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Foreword

The most recent national bridge inventory of deficient bridges conducted by the Federal Highway Administration (FHWA), U.S. Department of Transportation, indicates that an alarming number of the nation's bridges are classified as structurally deficient or functionally obsolete. Forty-two percent of the 575,607 bridges in the national system are so classified. Twenty-eight percent of the 271,300 bridges in the federal-aid highway system fall in the same category.

The problem is widely recognized, and increasing federal, state, and operating agency appropriations are being made available for bridge research, design, maintenance, and rehabilitation. The Transportation Research Board addressed this problem at the First Bridge Engineering Conference, held in St. Louis, Missouri, September 1978, and again at the Second Bridge Engineering Conference, held in Minneapolis, Minnesota, September 1984. Proceedings of these conferences were published in *Transportation Research Records 664*, *665*, and *950*, Volumes 1 and 2. This Record contains the papers prepared for the Third Bridge Engineering Conference, to be held in Denver, Colorado, March 10–13, 1991.

The papers in this Record highlight research and practice resulting from bridge studies performed by the National Cooperative Highway Research Program and sponsored by the American Association of State Highway and Transportation Officials as well as from federal, state, and other research agency programs. Areas addressed include safety and vulnerability of bridge structures; bridge maintenance; repair and rehabilitation; bridge inspection, testing, and evaluation; concrete and steel bridge design; segmental and cable-stayed bridges; seismic design and retrofit; bridge management; and scour prediction and countermeasures. The conference program includes working sessions on bridge specifications for the future. The proceedings of these sessions are not published in this Record because of space limitations.

Organization and direction of the conference were the responsibilities of the Planning Committee listed on the reverse side of the title page. Committees listed on the same page conducted technical reviews of the papers.

FHWA partially funded the Bridge Conference. The following organizations cooperated to make the conference possible:

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Cable-Stayed Houston Ship Channel Crossing

HOLGER S. SVENSSON AND THOMAS G. LOVETT

The bridge crosses the Houston Ship Channel 20 miles east of Houston between the cities of Baytown and LaPorte, Texas. It is the first dual superstructure cable-stayed bridge, and its total deck area of about 350 000 ft^2 makes it one of the largest cable-stayed bridges to date.
Each of the dual cable-stayed bridges has a composite superstructure girder with a main span of 1250 ft and a navigational clearance of 175 ft.
The twin concrete towers with a double diamond shape configuration rise 426 ft above the ground. In the transverse direction they carry the loads by truss action, in the longitudinal direction they are fixed to the foundations and elastically supported by the stay cables.

Each of the two girders is 78'-2" wide and consists of a 5'-3" deep steel grid. Exterior main edge girders and transverse floor beams at 17 ft spacing are plate girders with an 8 in composite concrete roadway slab. An additional wearing surface is 4 in thick. The roadway slab was designed for composite action under dead and live load.

The stay cables vary between 19 and 61 strands of 0.6" diameter. Their corrosion protection consists of Polyethylene (PE) pipes with cement grout. In addition, they are wrapped with a white weather-resistant tape.

Construction started in 1987, completion of the bridge is anticipated in 1992.

1 INTRODUCTION

The Houston Ship Channel Crossing, located 20 miles east of Houston, Texas, will replace the existing two lane Baytown Tunnel with a dual eight lane high level structure connecting Baytown and LaPorte. Presently, trucks with hazardous cargoes are prohibited from using the tunnel and must make a 16 miles detour. Completion of the bridge will allow the tunnel to be closed and permit the deepening of the ship channel.

At the bridge site the water is about 1500 ft wide with a required navigation clearance of 175 ft over a width of 600 ft. One tower is located on the existing levee, the other is placed in shallow water. For protection against ship collision the second tower is surrounded by an artificial island which also serves as a staging area for the construction.

The approach bridges were designed by the Texas Department of Highways and Public Transportation, using conventional precast prestressed girders with spans of up to 143 ft.

In accordance with Federal Highway Administration requirements alternate steel and concrete designs were prepared. In 1987 four bids ranging from 91.3 to 126.5 million dollars were received for the total project including the approach bridges, with all bidders selecting the subject steel-composite main bridge.

The structural design was done in accordance with the AASHTO Bridge Specification, amended as appropriate by other US and international codes.

All concrete and steel members were sized by the load-factor method and checked under working loads for fatigue and deflections.

Concrete for the roadway has a 7,000 psi compressive strength, concrete for the towers has a 6,000 psi compressive strength. All reinforcement is Grade 60, the structural steel is A 572, Grade 50.

2 GENERAL LAYOUT

The cable-stayed bridge is continuous over its length of 2214 ft, see Fig. 1. The girder is supported by stay cables in a semi-fan arrangement with anchorages at deck level at about 51 ft intervals.

The support conditions render a completely symmetric structure. The girders are connected to each tower with flexible neoprene bearings, 10 in high, which allow temperature movements by shear deformation and distribute longitudinal forces equally to both towers. The anchor piers are connected to the girders by rotational bearings. They are slender enough to allow temperature movements of the girder by deflection. The beam rotations are taken by strip seals above the anchor piers. All expansion movements take place at the two 30 in modular expansion joints on top of the piers at the ends of the two 130 ft flanking spans. Transverse wind loads are taken by bumpers at the towers, and at the anchor piers.

3 AERODYNAMIC INVESTIGATION

The bridge is located in a hurricane prone area near the Gulf of Mexico. The basic design wind speed for a 100 year

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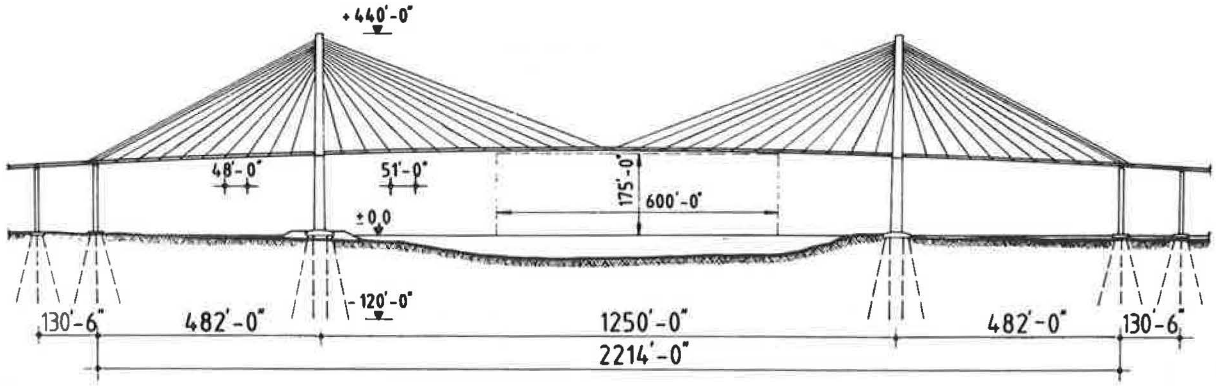

Fig. 1: General Layout

return period was determined as 110 mph at 30 ft elevation. By using an exponential function to reach the limiting wind speed of 200 mph at a 600 ft elevation, the basic design wind speed increases to 160 mph at deck level and 195 mph at the top of the towers. For the towers and piers an additional local gust factor of 7 % was applied.

The first 30 eigenfrequencies and corresponding mode shapes were calculated for a space frame. The important first natural frequencies in bending and torsion are f_B = 0.273 cps and f_T = 0.670 cps, resulting in a favorably high ratio of f_T/f_B = 2.45.

A high rotational superstructure girder stiffness for the torsionally weak open cross-section was achieved through the A-shaped upper tower legs. They form triangular space trusses with the towers as posts, the cables as tension diagonals and the girders as compression chords.Thus, providing much more rotational stiffness to the girder than H-shaped towerlegs would do.

The shape factors for the girder were determined in a wind tunnel from a section model with a scale of 1 : 96, Ref. (*1*).

In the final stage the traffic barrier and the safety fence create shape factors different from those during construction, see Fig. 2. Because of the interaction of the two girders different shape factors resulted also for the leeward and windward girder, see Table 1.

An analytical aerolastic analysis of the bridge in laminar and turbulent wind was then performed which rendered satisfactory results, Ref. (*2*).

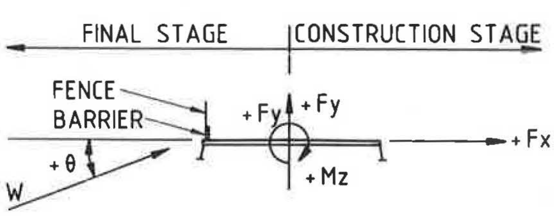

Fig. 2: Notation for Shape Factors

TABLE 1:
SHAPE FACTORS - MAIN SPAN BRIDGE DECK

Final Stage

Vertical Angle of Attack	Windward Bridge FX/Q	FY/Q	FZ/Q
- 2°	+ 14.537	- 32.062	39.392
0°	+ 14.075	- 21.724	- 46.300
2°	+ 14.028	- 9.044	- 113.503
Vertical Angle of Attack	**Leeward Bridge FX/Q**	**FY/Q**	**FZ/Q**
- 2°	+ 6.413	- 7.548	- 3.940
0°	+ 6.829	- 0.638	- 23.204
2°	+ 7.553	5.540	- 35.930

Windload = FT/Q x Q, with Q': Basic Static Wind Pressure

Construction Stage (W/O Fence and Barrier)

Vertical Angle of Attack	Windward Bridge FX/Q	FY/Q	FZ/Q
- 2°	+ 9.773	- 23.948	- 198.452
0°	+ 9.959	- 12.308	- 150.634
2°	+10.374	+ 2.357	- 118.323

Note:
Coefficients FX/Q and FY/Q are expressed in FT.
Coefficient MZ/Q is expressed in FT^2
For notation see Fig. 2

Because of the uncertainties involved with the interaction of the two girders, wind tunnel tests with a full bridge model were additionally executed, Ref. (*3*). The completed bridge was investigated as well as three construction stages, see Fig. 3. In both cases it was found that the aerodynamic interaction between the two beams by means of energy transfer to, and subsequent dissipation by, the leeward deck significantly enhanced the overall aerodynamic stability. The measured critical flutter wind speed was in excess of 150 mph for laminar flow. For an atmospheric turbulence of up to 12 % a peak to peak midspan amplitude of 5.42 ft is predicted for the completed stage, and of 8.22 ft for the critical construction stage just prior to reaching the anchor pier.

The computed results compared satisfactory with the wind tunnel measurements.

4 COMPOSITE GIRDER

4.1 Structural Details

The four lanes with full shoulders require a roadway width of 72 ft for each direction of travel. Two, three and four cable planes for supporting the roadway transversely were investigated. It was found that two independent girders - each supported by two outer cable planes, see Fig. 4, - are the most economical. Governing in this respect is the amount of steel required for the floor beams. It increases strongly if the transverse span length between cable planes is increased from 78 ft to 153 ft. The variation in the steel required for the main girders and the stay cables is comparatively minor.

The cross-section of an individual girder is shown in Fig. 5. It consists of a steel grid composite with a concrete roadway slab.

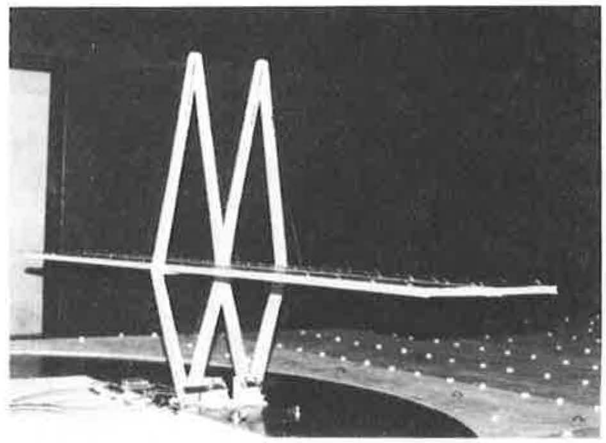

Fig. 3: Construction Stage Wind Tunnel Model prior to reaching the Anchor Pier, from Ref. (*3*)

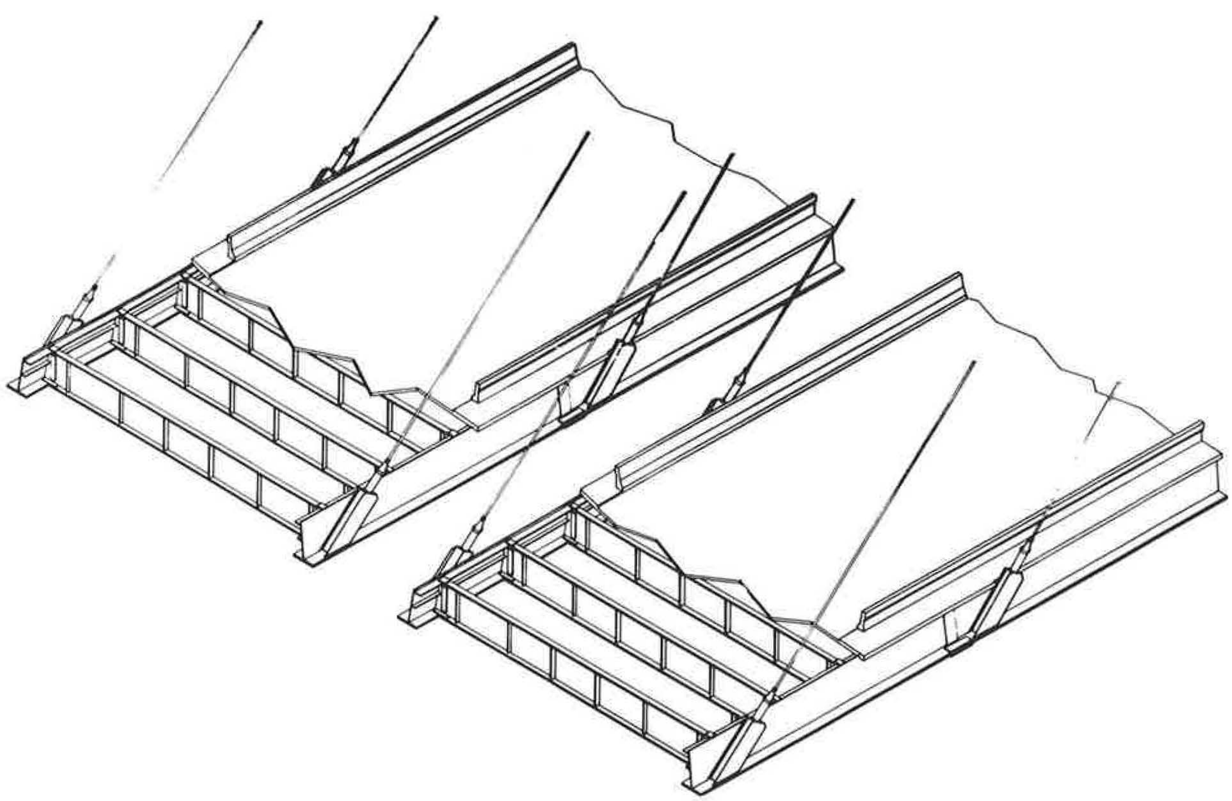

Fig. 4: Isometric View of Twin Girders

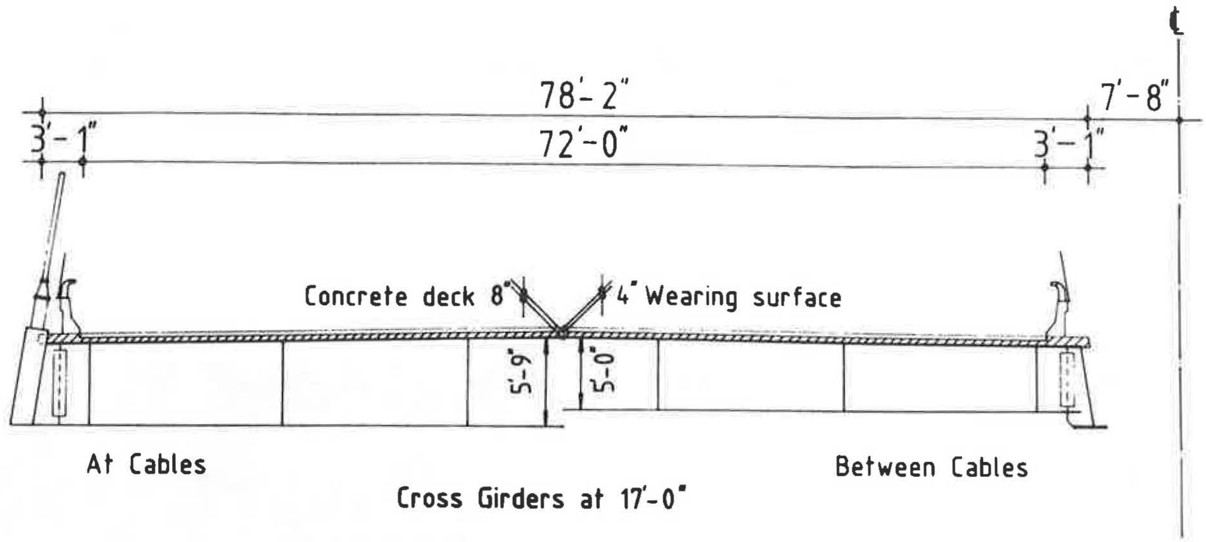

Fig. 5: Individual Girder Cross-Section

The outside main edge girders have one continuous longitudinal stiffener, see Fig. 6. The vertical stiffeners at 17 ft intervals are welded to the main girders, except for the regions of high moments near the center and the ends of the bridge where they are bolted to the bottom flanges due to fatigue. All floor beams are field bolted to the vertical stiffeners, see Figs. 5 and 6.

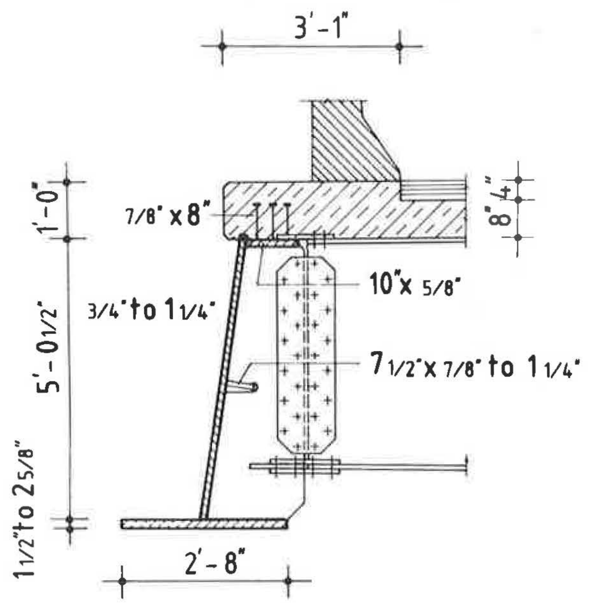

Fig. 6: Edge Girder Detail

The concrete deck is 8 in thick and has a 4 in reinforced concrete wearing surface. Such thick renewable wearing surface is used to safely protect the structural deck slab which would be more difficult to exchange. Longitudinal and transverse composite action is achieved by conventional shear studs. On the main girder top flange they are arranged in rows of three with a constant longitudinal spacing of 4 ½ in.

Crack control is achieved by a substantial amount of reinforcement with close spacing. At midspan where the compression is smallest 3/4 in diam. bars at 5 in spacing top and bottom (2.2%) are used longitudinally and transversely.

The cables are anchored in welded boxes bolted to the main girders, see Fig. 7. The eccentricity moment is carried by a force couple in compression to the roadway slab and in tension to the bottom flange of the full-depth floor beams.

4.2 Design

The overall girder forces under permanent loads were chosen similar to those for a beam rigidly supported at the cable anchorpoints, except for the midspan and end regions where a positive camber is introduced to provide additional compression in the roadway slab. Composite action for dead load was to be achieved by casting the roadway slab onto a continuously supported steel grid on ground . The deck was thus under compression in transverse direction also as top flange of a simply supported girder under dead load.

The shrinkage and creep values were calculated in accordance with the CEB-FIP Model Code (*4*), which resulted in approximately the following ratio of moduli of elasticity for permanent loads:

n_0 = 6.0 initially and for transient loads
n_1 = 12.5 at opening for traffic
n_∞ = 18.0 after creep has taken place
(W/C = 0.35, relat. humidity 75%, age of deck at installation 1 month).

The overall girder moments due to live load increased by up to 26% in the ultimate limit state due to non-linear effects of the rather slender deck with a main span to depth ratio of about 1 in 200, see Fig. 8 and (*5*). The edge girder shear studs are designed for the combined action of local

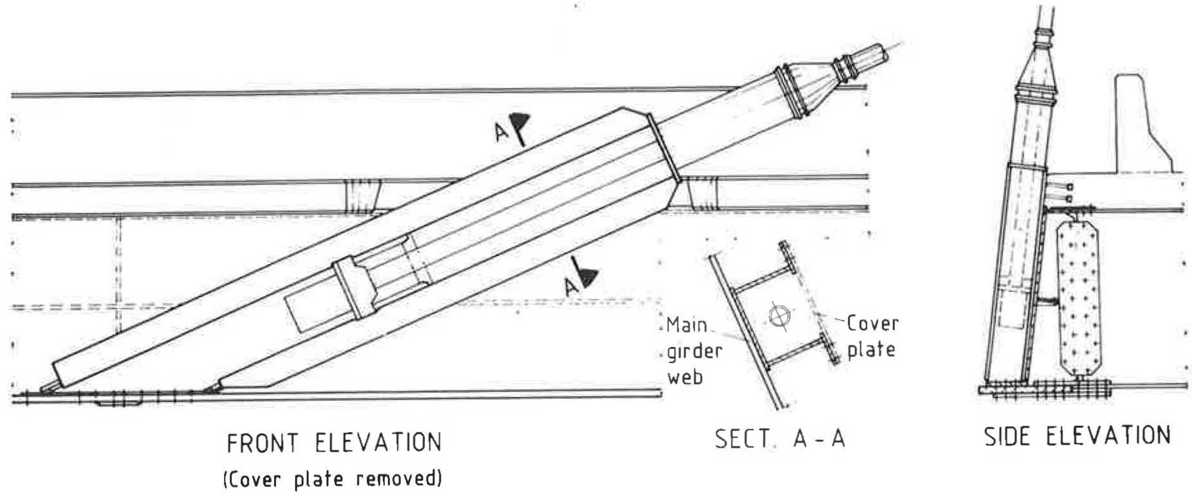

Fig. 7: Stay Cable Anchorage at Girder

and overall shear, and cable force introduction. An assumed limited amount of slip and plastic deformation of the studs in the ultimate limit state led to the uniform arrangement of shear studs over the length of the beam. The introduction of the shear force from the 12 in thick concrete edge beam into the regular 8 in thick slab (see Fig. 6) proved to be critical.

The sizing of the slab was governed by ultimate strength and crack control under service conditions. The plate girder stability was calculated in accordance with (*6*).

4.3 Contractor's Option

Instead of the proposed continuous roadway slab the contractor opted to use precast slabs connected by cast-in-place joints on top of the floor beams. Due to the resulting loss of composite action for dead load this required additional 1,412,000 lb of structural steel, or an increase of 17% from 24.8 lb/ft^2 to 28.9 lb/ft^2.

5 STAY CABLES

The stay cables were sized in accordance with the PTI-Recommendations (*7*), resulting in 19 to 61 7-wire 270 KSI strand with 0.6 in diam, see Fig. 9. They were specified as shop-fabricated parallel strand HiAm cables in PE-pipes with cement grout and a wrapping with a laminated Tedlar tape, see Ref. (*5*) for further details.

All stay cables are installed by jacking at the towerhead. They run through short steel pipes at the top and bottom anchorages, see Figs. 7 and 11. At the ends of these steel pipes annular neoprene washers are installed which act as dampers against cable vibrations.

The structure was designed to permit the exchange of any stay cable in conjunction with a reduction of live load to two lanes and reduced safety factors. Additionally, any stay cable may be accidentally severed under full live load without structural instability. Because of the close proximity of the backstay cables to one another, it was considered

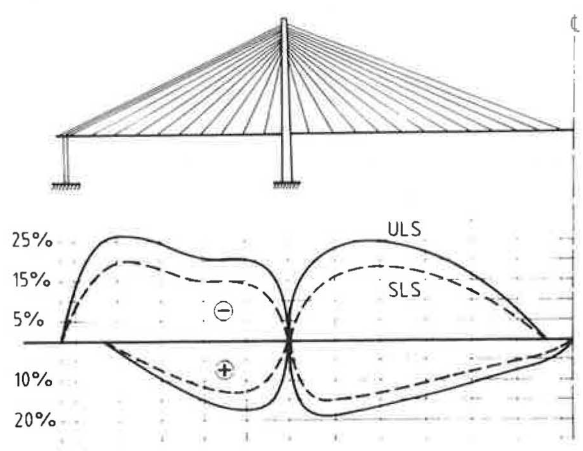

Fig. 8: Non-Linear increase of Live Load Movements

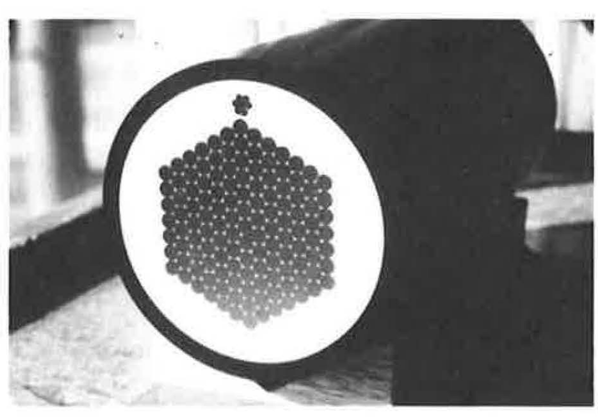

Fig. 9: Cross-Section of Stay Cable

prudent to protect them against simultaneous damage from a burning fuel truck. They are thus each surrounded by an additional larger PE-pipe, which reaches to 50 ft above the deck. The annular space of about 1 in between the inner and outer PE-pipes is filled with cement grout as an additional fire protection.

The contractor opted for site fabricated parallel strand cables with wedge anchorages in PE-pipes and cement grout.

6 TOWERS

The towers are shown in Fig. 10. Their legs and the tie beams underneath the decks have box sections with a minimum wall thickness of 12 in.

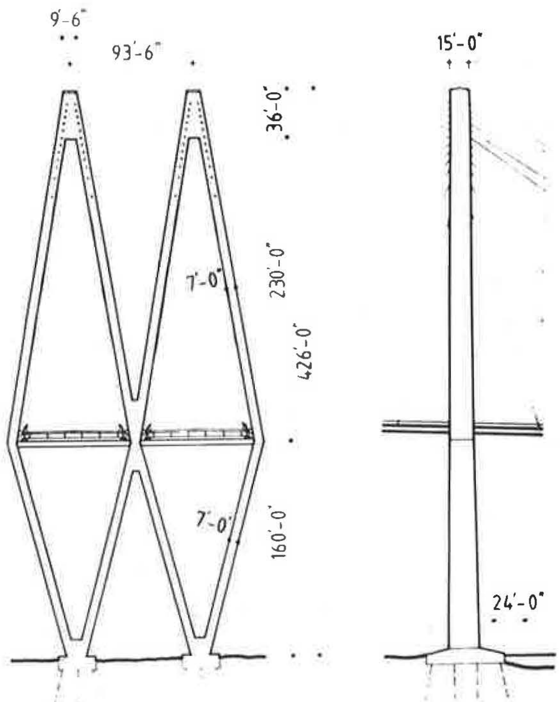

Fig. 10: Tower Layout

The double diamond shape is a natural progression from the twin decks. The A-frames on top of the decks reduce the rotations of the beam significantly by forming a triangular space frame, Ref. (5). By joining the two lower A-frames at deck level, a truss is created which carries the transverse wind loads in tension and compression to the two foundations. The transverse width of the towerlegs can thus be small. In longitudinal direction the tower legs act as cantilevers in bending, especially during construction. Those widths have thus to be significantly greater. The tie beams act as direct tension members and are fully post-tensioned against the outward thrust from the tower legs.

At the towerhead the stay cables pass through steel pipes embedded in the tower walls, see Fig. 11. They are individually anchored inside on steel bearing plates resting on concrete corbels. The horizontal cable components are tied back with alternating loop tendons so that each cable anchorage region is confined by the radial forces from the loops.

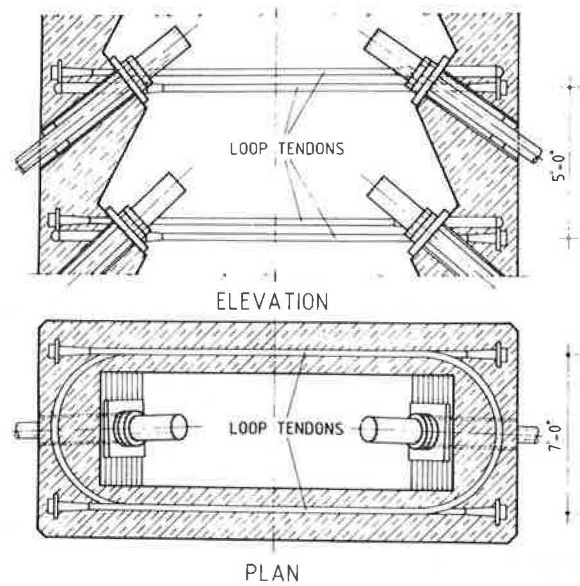

Fig. 11: Stay Cable Anchorage at Towerhead

7 CONSTRUCTION

From the four foundation alternates shown on the bid drawings the contractor opted to use 20 in. square precast prestressed concrete piles. 132 piles up to 136 ft long for each of the four tower foundations were driven from the artificial island on the LaPorte side and the existing levee on the Baytown side. 12 ft thick CIP pile caps form the basis for the towerlegs.

The towers are cast in 15 ft lifts with jumping forms, using trusses to support two cranes, see Fig. 12. All vertical reinforcement is spliced with mechanical couplers. They are squeezed hydraulically around the deformed bars.

Where the two inner legs meet, 220 rebars on each side cross one another. This cage was preassembled in a 45 ft high section, Fig. 13. Superplasticised concrete was vibrated into place without any significant honeycombing.

At about one half of the height above the deck temporary struts support the inclined legs against one another, Fig. 14.

The lower part of the 2nd tower is built with conventional jumping forms. First the inner legs are supported on trusses. Then the outer legs are tied back to the inner ones, Fig. 15.

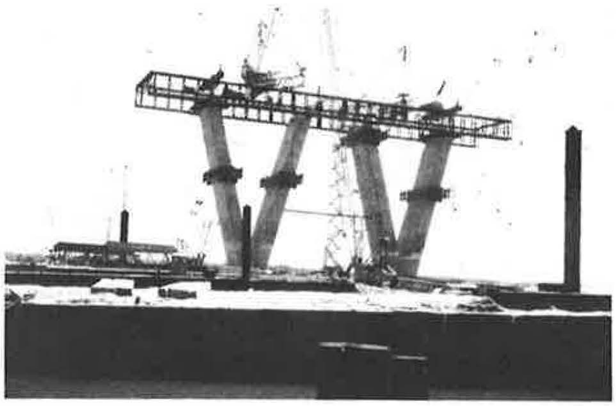

Fig. 12: First Tower Construction below Tie

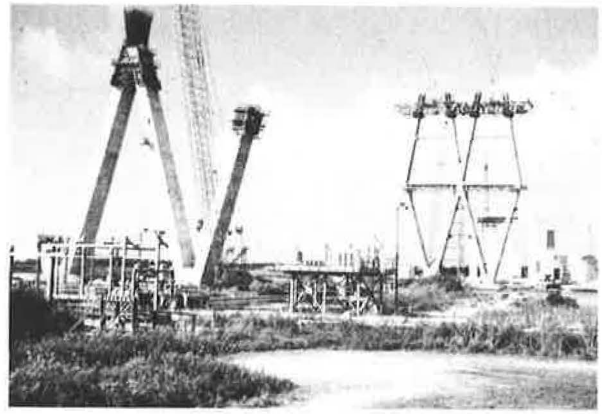

Fig. 15: 2nd Tower Construction below Tie

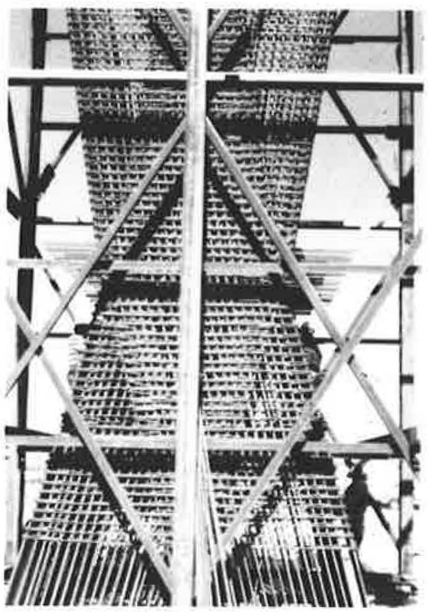

Fig. 13: Reinforcement Cage for Connection of inner Legs

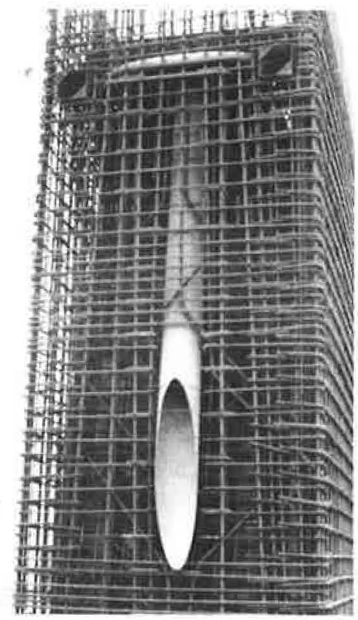

Fig. 16: Preassembled Section for Towerhead

Fig. 14: Construction of upper Tower Legs

Each lift in the anchorage zone is completely preassembled on ground, first the inner forms with the steel pipes, then the reinforcement is added, Fig. 16.

The about 5000 t of steel girders and their appurtenances are fabricated in South Africa. This is possible because the bridge is state-financed and the current Texas "Buy America" provisions were not yet law when the contract was signed in 1986, Ref. (*9*).

The stayed girders will be constructed by free cantilevering from the towers outwards. Construction started in 1987, and the bridge will be opened in late 1992.

The architectural model shown in Fig. 17 gives an impression how the completed bridge will look.

Fig. 17: Architectural Model

8 ACKNOWLEDGEMENT

Owner is the Texas State Department of Highways and Transportation. The cable-stayed main bridge was designed by Greiner, Inc., Tampa, Florida, in association with Leonhardt, Andrä and Partners GmbH, Stuttgart, Germany. Dr. Robert H. Scanlan served as aerodynamic consultant. Williams Brothers Construction Co., Inc., and Traylor Bros., Inc. (a joint venture), are the contractors. The stay cables are supplied by the VSL Corporation.

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Construction Design of the Dame Point Bridge

MAN-CHUNG TANG

Material quantities are usually not the most important items of such complex construction projects. A design with minimum materials does not necessarily represent the most efficient design. How a structure can be built is extremely important.

The main purpose of this paper is to describe how the bridge configuration and cable arrangement were modified to facilitate simpler construction and how the construction equipment was designed to make the operation more efficient.

The Dame Point Bridge in Jacksonville, Florida is a cable-stayed bridge with a mainspan of 1300 feet. It is presently the longest span cable-stayed bridge in the United States. It has a harp cable arrangement. The girder is 105 feet wide consisting of two solid edge girders. The bridge deck is supported by transverse floor beams framed into these edge girders. The deck slab varies from 9" to 2'-0". The floor beams are spaced at 17'-6" on centers.

The cables have Dywidag bar tendons grouted inside a steel pipe. This is the only cable-stayed bridge in North America that uses bar cables.

Towers are solid sections with cables enclosed in a crisscross pattern. The tower legs and columns are interconnected to each other by three bow-tie shaped cross-struts.

Construction of the deck was by cast-in-place method which used specially designed form travelers. Construction progressed smoothly.

Aerodynamic stability during construction was studied based on results from wind tunnel tests. Special tie-downs were designed and installed to safeguard against possible buffeting of the bridge under hurricane.

President, DRC Consultants, Inc. 34-36 Union Street, Flushing, New York 11354.

INTRODUCTION

The Dame Point Bridge is a 6,600 feet long bridge structure crossing the 1,800 feet wide St. Johns River in Jacksonville, Florida. It is a part of the eastern bypass around the City of Jacksonville. It carries six lanes of traffic divided into two roadways of three lanes each. However, the concrete medium barrier is removable to permit future reversible lane operations.

The main bridge is a cable-stayed structure with a center span of 1300 feet and two side spans of 650 feet each. The main bridge provides a vertical navigation clearance of 175 feet at midspan. This will permit passage of all large ocean-going vessels.

This is the largest concrete cable-stayed main span in the Western Hemisphere and the longest span in the United States of America.

The engineer produced two alternate designs for bidding purposes : a steel cable-stayed bridge with an orthotropic deck and a concrete cable-stayed bridge with a beam and slab type deck. The bridge was tendered in 1979 but due to an unfavorable financial situation the tender was cancelled. A new tender was called in 1986.

In the final tender, the Owner also permitted alternate designs submitted by the Contractors. However, the low bid in 1986 was based on the original concrete design at a price of $46.6 million. Very surprisingly, this was almost 30% lower than the lowest bid of $65.8 million seven years earlier.

The Contractor was a joint venture of Pensacola-Tyger. DRC Consultants, Inc. provided all construction engineering subcontracted through Dywidag Systems International. DRC also recommended certain modifications to the original design to simplify its construction.

The north and south approaches each

have two spans of 82 feet and 18 spans of 102 feet with precast I-girders and cast-in-place deck slabs with portal type piers. This is an alternate to the original W-shaped solid piers. The contractor was McCarthy Brothers and redesign was provided by DRC Consultants, Inc.

This paper will concentrate on the construction of the cable-stayed main span only.

CHARACTERISTICS OF THE MAIN BRIDGE

The general layout of the bridge is shown in Fig. 1. The pylons consist of solid columns connected by two bow-tie shaped cross beams. The solid column stems above the deck and are 7'-3" thick. Their width tapers from 32 feet at deck level to 15 feet at the top. The top of the tower is 466 feet above water level, Fig. 2.

The cables are in a harp arrangement rendering an aesthetically pleasing configuration. It also allowed for the simultaneous construction of the deck and pylon.

The deck is monolithically connected to the pylons. A hinge with an expansion seal is located at the midspan to accommodate all longitudinal movements.

The total width of the deck is 105'-9", Fig. 3. It has two 8 feet wide solid edge girders. The depth of the edge girders varies from 5 feet at midspan to 6'-1" at the pylon. The original design called for precast concrete floor beams spaced at 11'-8" on center. The cable spacings were 35 feet on center along the edge girders. Thus, each panel of the deck has three floor beams.

CONSTRUCTION DESIGN

For a complex structure such as a cable-stayed bridge a significant amount of engineering is required for its actual construction.

In most bridges construction engineering consists of analyzing each construction stage based on the actual construction loading, checking of stresses and capacity of the structure, providing camber curves to the contractor to set up the formwork and possible adjustments. However, when construction is as complicated as in the Dame Point bridge, the construction engineering consultant has also to develop a cable erection scheme, work out the pouring sequence of the segments, design the cable erection trusses and form traveler as well as the falsework and other temporary structures required for the pier table, center and end spans closure pours, among others.

The sequence of construction of certain elements is sometimes very important. By delaying construction of the bow tie strut between the tower legs, for example, cables could be erected easily. Such modifications of the construction sequence require very detailed analysis.

This deck configuration was modified in the final construction. Instead of the original double cables spaced at 35 feet, single cables at 17'-6" were used. The floor beam spacings were also changed from 11'-8" to 17'-6" thus making it equal to the cable spacing. This modification simplified the construction significantly. The precast floor beams were also changed to cast-in-place construction.

Modifications of the cable arrangement and floor beam configurations changed the structural system of the bridge. Extensive stress analysis was carried out to assure the safety of the structure under all possible loadings and that the modified structure met the intent of the original design.

To safeguard against possible future vibrations the cables were tied to each other by strands perpendicular to the cables. This was found to be very effective. No further vibrations have been observed since their installation.

FOUNDATIONS

Both end piers have pile footings and land piers. The Construction was quite conventional.

Construction of the two main piers, however, was more difficult. They stood on rather different foundations. At the south pier the water is about 15 feet deep. 704 steel H-piles with 300-ton capacities were driven to 80 feet length. The seal measured 187'-3" x 94' x 14' deep. It was poured in one single

operation in 35 hours.

The logistics of pouring the south pier was quite significant because the ready mix concrete was supplied to the north pier and the concrete trucks had to be barged to the south pier across the St. John's River. Six barges were used each carrying three to four 10 cubic yard concrete trucks. A total of 310 trips were made within the 35 hour concreting operation.

The water is about 40 feet deep at the north pier. The cofferdam was driven to 40 feet below the river bed. After the excavation was completed, a 33-foot deep tremie seal measuring 90' x 170' was poured by one single operation in 47 hours. That was about 1900 cubic yards of concrete. To assure a continuous operation a large fleet of concrete trucks were secured, some of them commissioned from hundreds of miles away.

DECK CONSTRUCTION

The pier tables, i.e., the first segments of the deck at the pylon was built on falsework supported directly from the footing. After erecting the first set of cables, the falsework was removed. Form travelers were installed on each end of the pier table.

Each form traveler weighed approximately 120 tons and was designed to allow pouring of the complete, 105' wide and 17'-6" long segment in one operation, Fig. 4.

The modification of the cables and floor beam spacings simplified the deck construction. All segments were 17'-6" long with one long floor beam. The rectangular shape of the floor beams also simplified the design and stripping of the formwork.

The traveler operations were semi-automatic. Hydraulic jacks were used to raise, lower and adjust the traveler. The launching operation of the traveler from segment to segment was also done by hydraulic jacks. The formwork on top of the traveler was supported by both hydraulic and screw jacks to facilitate adjustment of elevations. The traveler together with the formwork could be lowered sufficiently to clear the floor beams during the launching operation. At that time, it was supported by a pair of C-shaped hangers, one at each side of the girder. These C-shaped hangers were supported by rollers which ran on steel channels laid flat on the edge girders that served as rails.

During concreting operation the traveler was stressed to the edge girders by 16 vertical ties of 1-3/8" Dywidag high strength bar tendons.

WORKING CYCLE

After a learning period of several segments the Contractor was able to achieve a six day cycle for each segment at each form traveler. The cycle time was reduced even further to five days near the end of the construction. This is equivalent to the construction of 7350 sq. ft. of bridge deck per week. This reflected fully the advantage of segmental construction with a well designed form traveler. It is very efficient once the crew got used to the operation.

A typical working cycle of a segment is as follows:

1. Advance the form traveler to the next segment. Raise and tie the traveler to the previously completed segment. Align and grade the formwork according to the camber requirement provided by the construction design.
2. Set the reinforcing steel and post-tensioning steel in the slab, floor beams and edge girders.
3. Erect the new pair of cables and extend the ends of the cables to attach to the form traveler. Stress the new cables to a predetermined force. During this cable stressing operation each bar tendon is overloaded to 75% of its ultimate strength independently and then anchored at the required force.
4. Fine adjustment of elevation and then cast concrete.
5. Cure the concrete. Strip bulkhead and side forms after the concrete has properly set.
6. When the concrete strength reaches 4200 psi, stress the tendons in the floor beams to 70% of the design load and stress the tendons in the previous floor beams to 100% of the design load.
7. Adjust the newly erected cable to a predetermined cable force, if required.
8. Lower the traveler and formwork;

prepare for advancement to the next segment.

Cables were assembled on the deck at the same time as other work for the segment in the form traveler were progressing. Assemblage of the cables was, therefore, not at the critical path of construction.

A total of four form travelers were used simultaneously.

CABLES

This is the second major cable-stayed bridge that used large diameter high strength bars as cable tendons. The first one was the Penang Bridge in Malaysia completed in 1985.

Quality control of the bar cables was very stringent. The 1-1/4" high strength bars with 150 ksi ultimate strength were delivered to the site at 40-foot lengths. They were then coupled together using specially designed couplers which met strict fatigue and ultimate load requirements. The cable anchorages and the couplers were tested in the University of Munich to assure that they met the required specifications.

The cables were filled with cement grout after final adjustments. The bars and the steel pipe acted in composite action. The steel pipes were welded together by full penetration welds. The fatigue criteria requires all these welds pass the x-ray tests before a cable could be erected. After the total length of the steel pipe had been completed the bar tendons were coupled together at one end of the pipe as they were pulled into the steel pipe.

The assembled cables, steel pipes and bar tendons together, were hoisted and erected by means of cranes. For the longest cable, up to 728' long, three cranes were used to work in a synchronized fashion. Large spreader beams, Fig. 5, were used to reduce local bending moments of the steel pipes during erection until the cables attained sufficient tension to carry their own weight.

After the cranes picked up the cable, they first shifted the upper end of the cable into the tower. Then the lower end was placed in the edge girder form. It is significant to note that the erection of each cable, from the time the cranes picked up the steel pipe from the deck to the completion of the initial stressing, lasted only about 2 hours. Because reinforcing steel in the segment was being placed at the same time, the cable erection operation was practically outside of the critical path of the construction schedule.

The 288 cables came in four different sizes, Fig. 6. Six bar tendons in a 6" extra strong pipe; Seven bar tendons in a 6" standard pipe; Nine bar tendons in an 8" standard pipe; and Nine bar tendons in an 8" double extra strong pipe. All bar tendons were 1-1/4" diameter, 150 ksi Dywidag Threadbars per ASTM A722 Type 2 and all steel pipes were of ASTM A53 Grade B.

Both top and bottom ends of all cables have fixed anchorages. Angle changes at the cable ends due to structural deformations or changes in cable tensions may cause high bending stresses in the cables. A Neoprene collar was placed at the exit point of the cable and anchorage to provide an elastic support that reduced the bending stresses. This neoprene collar also served as a damper for the cable to reduce wind induced vibrations.

PYLONS

The upper pylon stems were built in 19 segments corresponding to the cable spacings using self lifting forms. The harp type cable configuration offered the great advantage of being able to build the pylons and the deck at the same time. The construction of the pylon stems were, in general, about two to three segments ahead of the deck girder, Fig. 7. This, however, required a camber to be considered in the pylon construction even though the pylon required no camber after the bridge was completed.

Because the tower columns are very slender in the transverse direction, they are very sensitive to all transverse loads before the cross girders are in place. A very detailed analysis considering the nonlinear behaviour of the structure and the material was carried out to determine the critical stage when the cross girder had to be in

place before further construction could proceed.

The most critical loading was from hurricane wind. But a hurricane can usually be forecast ahead of its arrival. To secure the stability of the towers a special cable bracing system was designed, fabricated and stored at the site. This bracing system could be installed within hours after a hurricane was forecast for the vicinity. However, this bracing system was never used because the critical construction stages were not in the hurricane season.

END SPAN CLOSURE

The end diaphragm located on top of the end pier was constructed well before the closure to receive the precast girders from the approach span. It was tied down to the cap of the end pier by temporary prestressing ties and blockings.

The traveler was attached to the diaphragm before the closure was poured. This connection was designed to resist all possible vertical and horizontal forces due to temperature and incidental loadings. The ties and blockings between the diaphragm and the pier cap were then released. The closure was poured using a special local formwork supported by the traveler.

MID SPAN CLOSURE

A hinge connects the two halves of the bridge girder at the midspan. A box-type floor beam at the end of each cantilever provides increased stiffness to the deck and the edge girders at this discontinuity points. Torsional rigidity of the box beam is further enhanced by connecting it with the adjacent floor beam by short longitudinal ribs.

This midspan closure was poured under a symmetrical loading condition. The form traveler at the north cantilever was removed. The remaining traveler was placed at such a location that its load was evenly distributed to both cantilevers, Fig. 8. The actual load distribution was verified by lift-off readings of the vertical tie-downs which were the only attachments the form travelers had with the girders. Although such an operation was not a requirement, it did simplify the analysis.

The hinge is a steel tongue and groove assembly. It is fixed to the girders by high tensile prestressing bars.

DECK PRESTRESSING

All floor beams have two tendons with 15 to 19 - 0.6" strands each. Because the edge girders are very stiff, the newly poured floor beams were stressed and reduced the prestressing force in the previously poured floor beams. This effect was compensated by a two-stage prestressing of the floor beam tendons.

The tendons in the newly completed floor beams were first stressed to 70% of their required force. These tendons were restressed to 100% of the required force after the succeeding floor beams were stressed. This not only reduced the effect from stressing the succeeding floor beams but also eliminated part of the losses due to creep and shrinkage of the concrete.

Longitudinal tendons were provided at the center portion and at both ends of the girders where axial compression from the horizontal components of the cable forces were small. They were stressed after the deck construction was completed.

AERODYNAMIC STABILITY DURING CONSTRUCTION

It is well known that cable-stayed bridges are more vulnerable to aerodynamic vibrations during construction stages than in completed stages. Extensive investigations were carried out for all critical construction stages of the Dame Point bridge before the final construction scheme was fully developed. A wind tunnel test was performed at the Low Speed Laboratory of the National Research Council in Ottawa, Canada. However, only a sectional model was used. Prior experience on other bridges showed that results from sectional models matches reasonably well with results of full model testings.

Investigations showed that the bridge built by cantilever method without any auxiliary supports or tie-downs was stable against vortex induced vibrations

-flexural, torsional and flutter vibrations. However, buffeting would cause significant overstresses in the structure at wind speeds at or over 80 mph. Buffeting might create significant vertical wind load on the bridge deck thus causing extremely high bending moments at the tower, Fig. 9.

The effect of buffeting can be reduced by increasing the natural frequency of the structure. This can be simply achieved by means of vertical or inclined tie-down cables connecting the bridge deck to a foundation. In the Dame Point bridge, at the most critical construction stage immediately before connecting the end cantilever to the end pier, the tie-down at the back span increased the natural frequency of the structure from 0.13 Hz to 0.22 Hz. This was sufficient to reduce the vertical, single peak amplitude at the tip of the main span cantilever from 10 feet to less than 4 feet. Comparing the weight of one 17'-6" segment will produce about 3 feet deflection at the end of the cantilever at this stage, a 4 feet oscillation under a hurricane wind was considered acceptable.

A tie-down system was designed and the required tension piles were driven and capped by a concrete foundation. The ties consisted of 2x19-0.6" prestressing strands each. These strands could be installed easily and stressed by regular prestressing jacks. However, this tie-down system, although ready and stored at the jobsite was never used because, as mentioned before, all critical construction stages were outside of the hurricane season.

An opportunity to measure the flexural vertical vibration in the field was provided when the bridge was being prepared for the side span closure pour. Because the end of the cantilever was very close to the end pier, it was easy to record the relative movement between the cantilever and the pier cap under a gentle cross wind. The frequency was found to coincide with the calculated first mode frequency of 0.13 Hz. The amplitude, as predicted, was small.

No noticeable vibrations of the cables were observed except in one or two incidents. Some long cables vibrated at a relatively low wind speed on a rainy day. Unfortunately, wind speed and amplitude were not recorded.

CAMBER CONTROL AND CABLE ADJUSTMENTS

The bridge deck of the Dame Point bridge is of a flexible type. For this type of bridge the most important geometry control is to assure that the local alignment is as close to the theoretical camber shape as possible. Global deviations can be corrected easily by cable adjustments. Any time during construction or after the bridge is completed, deflections as large as 2 to 3 feet can be corrected with relatively small changes in cable forces.

The analysis was based on linear theory except that the cable stiffness was adjusted according to the cable tension. A detailed nonlinear analysis, taking into consideration all displacements and variations in material stiffness was carried out for two construction cycles. Comparison of the results of the linear and nonlinear analysis showed that the nonlinear effects were negligible for the loading magnitude and sequence of the construction scheme.

Creep and shrinkage effects were calculated according to the CEB-FIP Model Code.

Camber curves for every construction stage were prepared before the start of construction. Comparison between the as-built curves and the theoretical curves was a regular control procedure for the elevations.

Modifications of the theoretical camber curves are sometimes necessary during the course of construction. Constant communication between the construction site and the design office is a prerequisite for a successful construction of such structures.

The maximum deviation of the bridge deck from the theoretically calculated camber curve at the time of the closure of the midspan hinge is less than three inches. Considering the span is 1300 feet long, this deviation is insignificant.

The cables were grouted after all permanent loads were in place.

CONSTRUCTION SCHEDULE

The bridge was tendered in November 1984. Notice to proceed for construction was given in March 1985. The contract stipulates a construction period of 36 months. Difficulties were encountered during pile driving for the south main pier. Additional load tests were carried out before final production piles were driven contributing to some delays. Other parts of the construction were slightly ahead of the schedule. The bridge was completed in September 1988.

ACKNOWLEDGMENT

The successful construction of the Dame Point Bridge is a result of a close cooperation between the Owner, the Contractor and the Engineers.

The Owner of the Dame Point Bridge is Jacksonville Transportation Authority (JTA). General Manager for the Owner is Sverdrup & Parcel.

The bridge was designed by Gerry Fox, Herbert Globig and Ray McCabe of Howard Needles Tammen & Bergendoff (HNTB).

Representing Jacksonville Transportation Authority to supervise and inspect construction was Leonard P. Druian of Sverdrup & Walter Sharko of HNTB.

William Praderio was project manager for the Contractor, a joint venture of Pensacola-Tyger. Manfred Spannring of Dywidag Systems International was their technical adviser at the site.

Man-Chung Tang of DRC Consultants, Inc., collaborating with Khaled Shawwaf of DSI, was the project manager for all construction engineering work.

DRC engineers Jee-Bong Louie provided design modifications, Nien-Sheng Chung - construction stage design, Hannskarl Bandel and Boris Levintov the form traveler and other equipment designs.

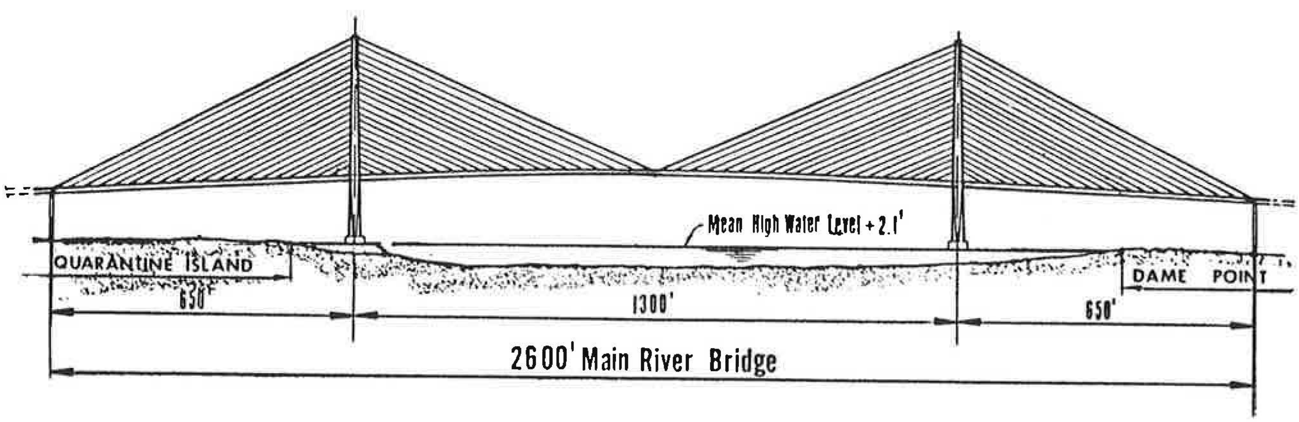
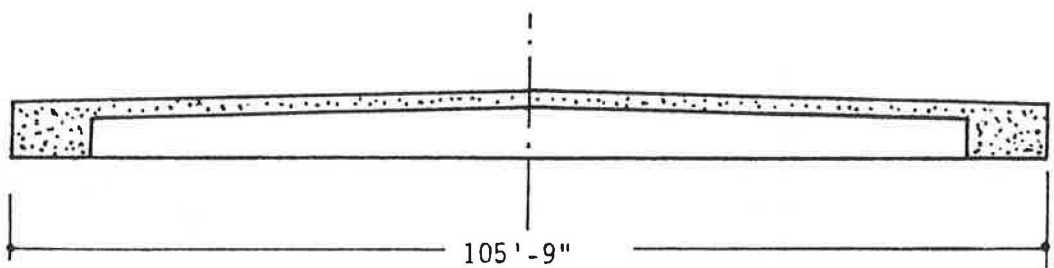


FIG. 1. ELEVATION OF THE DAME POINT BRIDGE

FIG 2. THE DAME POINT BRIDGE



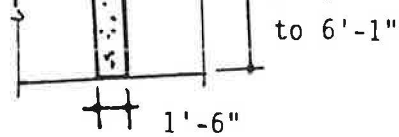


FIG. 3.
DECK CROSS-SECTION

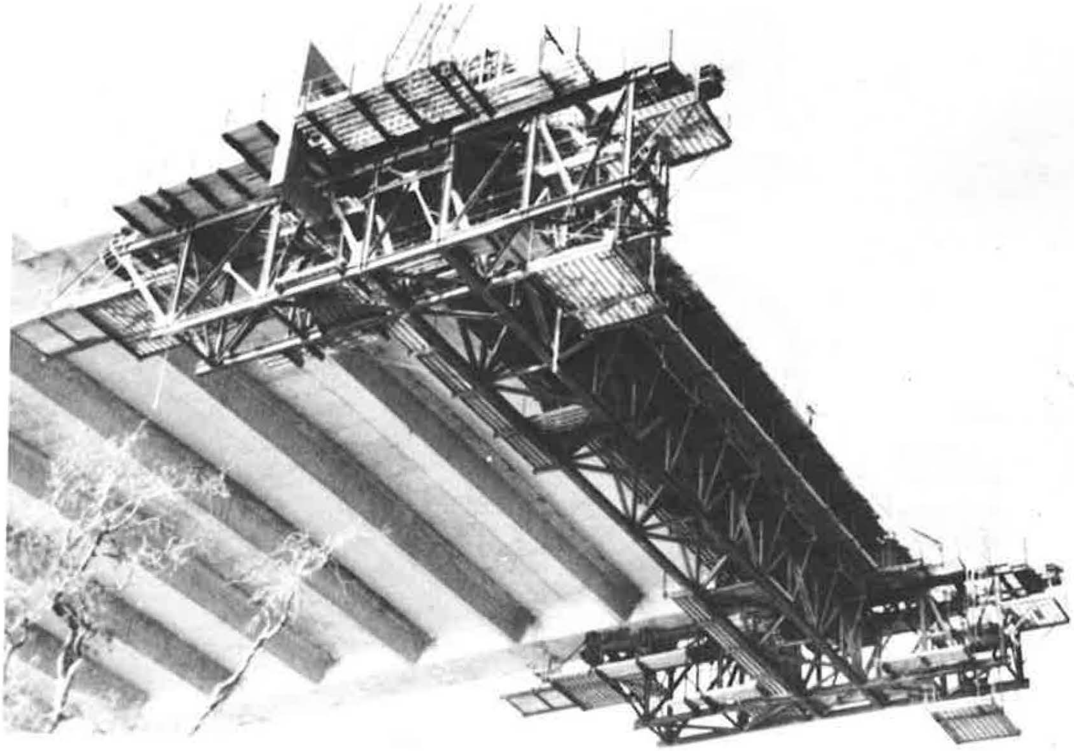


FIG 4. FORM TRAVELER

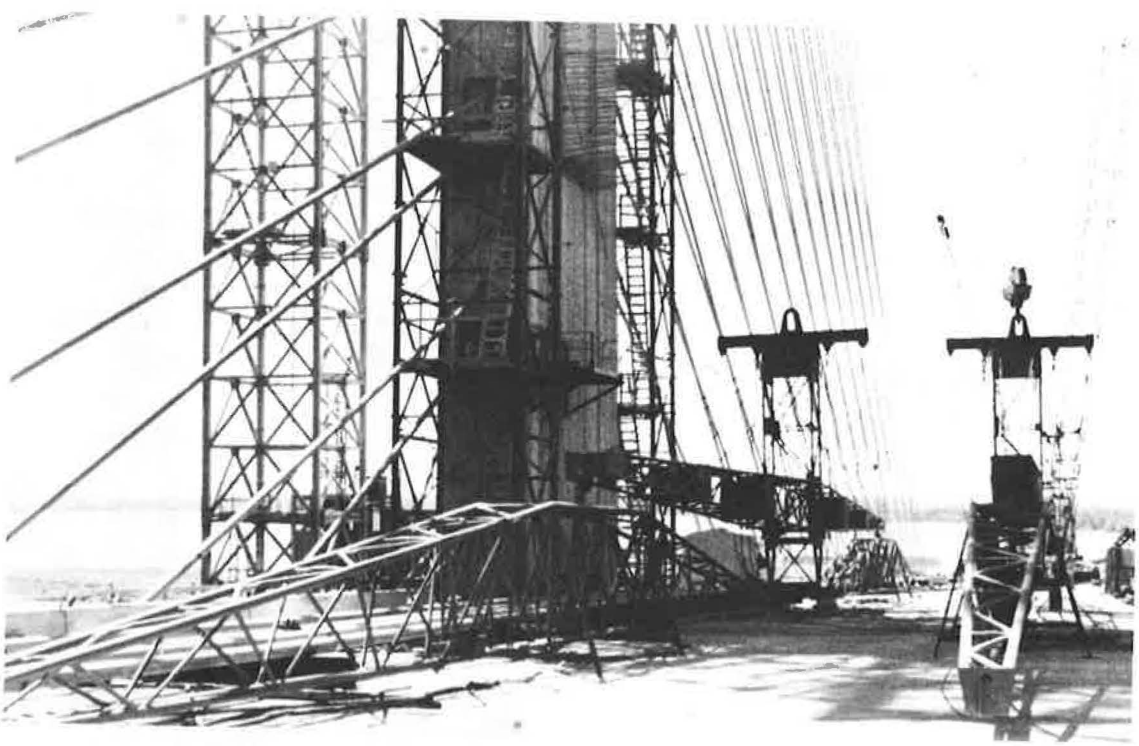


FIG. 5. SPREADER BEAM FOR CABLE ERECTION

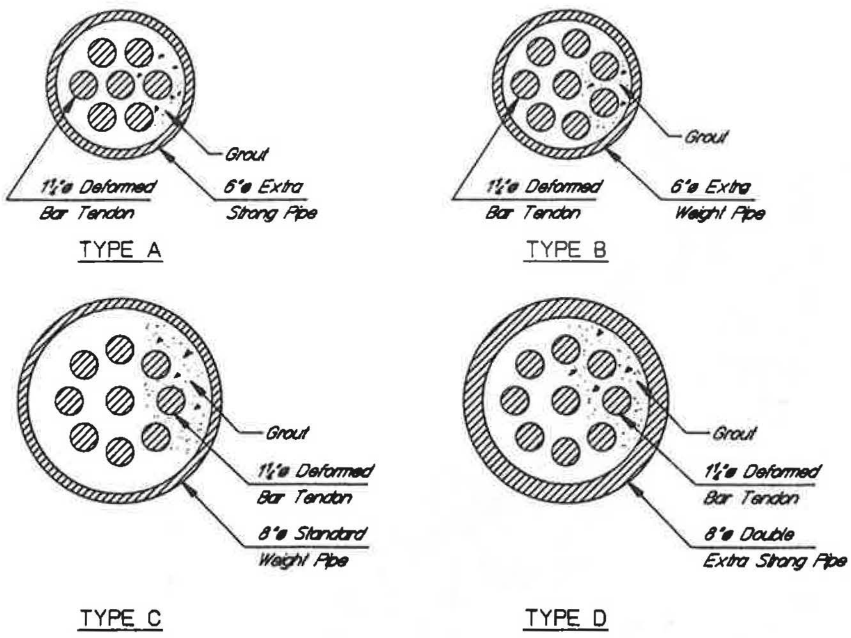


FIG.6. CABLE CROSS-SECTIONS

FIG. 7.

SIMULTANEOUS CONSTRUCTION OF GIRDER AND PYLON

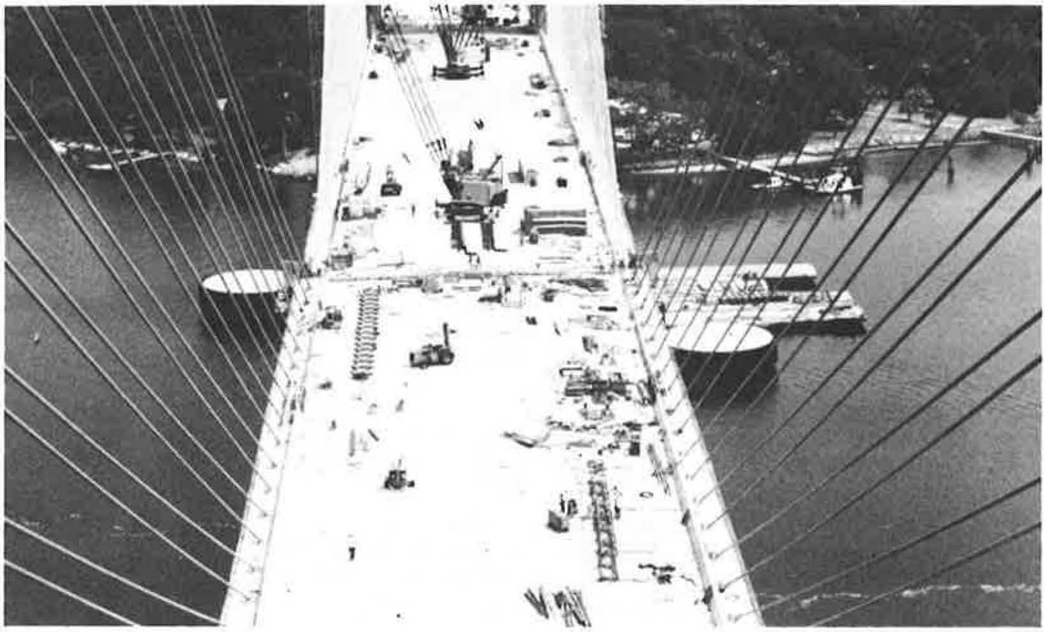


FIG. 8.
MID SPAN CLOSURE

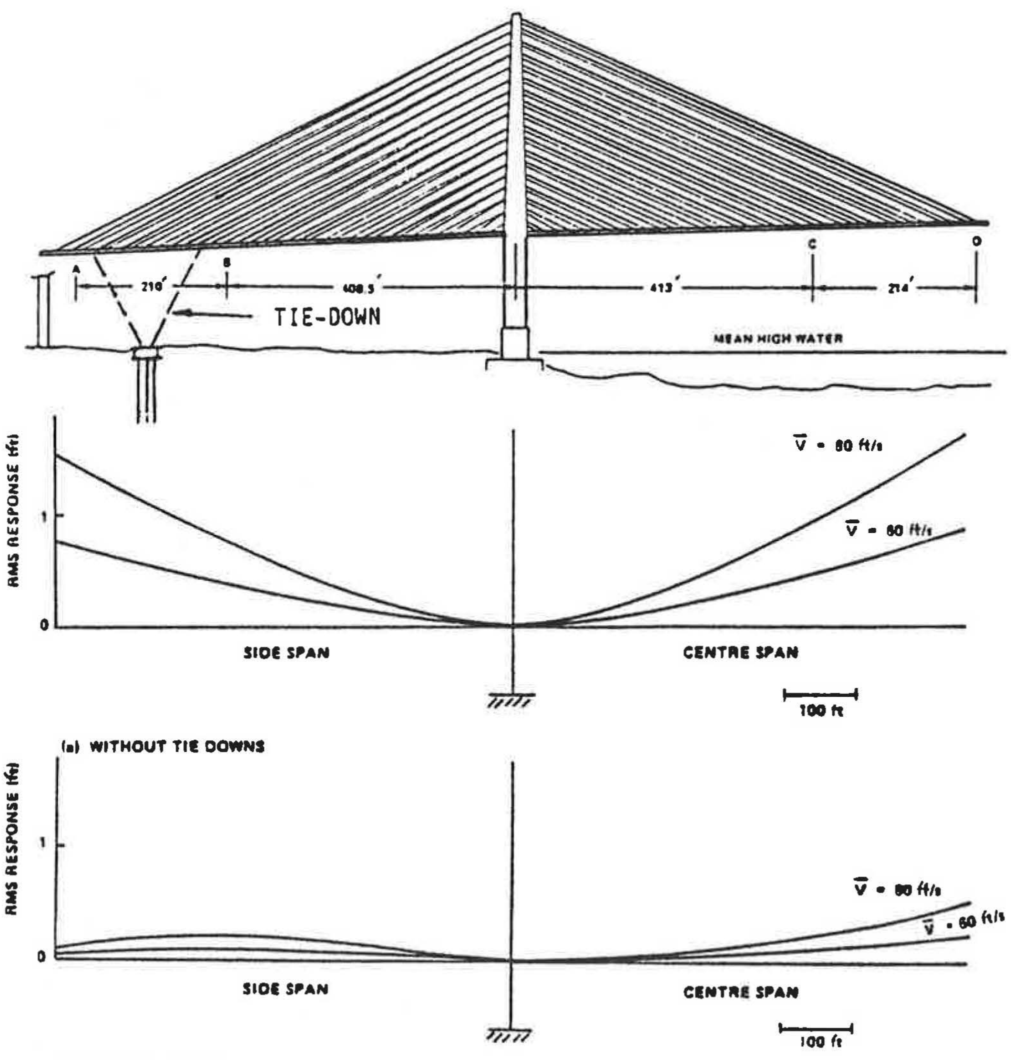


FIG. 9. BUFFETING RESPONSES

Creep and Shrinkage in Composite Cable-Stayed Bridges

S. G. ARZOUMANIDIS,[1] R. G. BURG,[2] AND J. SCHMID[1]

The application of strict controls on the creep and shrinkage exhibited by the roadway deck concrete of composite cable-stayed bridges is of primary importance. Precast deck panels from concrete specifically designed for minimized creep and shrinkage effects, carefully cured and matured are almost exclusively used in these structures. Such concrete shows a reduced long-term modular ratio which is quite different from the recommended by AASHTO for the design of composite girders. The provisions of ACI 209 can be used for the prediction of the creep and shrinkage effects of concrete. Creep tests are important for consideration of the specific material, project and site data.

Several cable-stayed bridges with composite girders have been designed and built in North America in recent years. The concrete roadway deck of these bridges is an integral part of the steel support system and carries vertical loads and significant horizontal compressive forces. The latter forces, typical in cable-stayed bridges, result from the inclined cables which support the composite girders.

Due to economic considerations, the concrete deck, which is efficient in carrying compressive forces, is made composite with the steel girders (built-up members or trusses) for live as well as dead loads. The effectiveness of this composite structural system is related to the creep and shrinkage properties of the concrete. Shrinkage is the decrease with time of the concrete volume due to changes in the moisture content and other physico-chemical changes. Creep on the other hand is the time-dependent increase of the concrete strain due to applied sustained loads. The effect of creep and shrinkage is the slow transfer of stresses from the concrete to the steel resulting in long-term reduced efficiency of the concrete in resisting loads.

The competitiveness of composite girders as opposed to other structural systems of cable-stayed bridges, entails the reduction of the concrete deck weight to the absolute minimum. This is achieved using high strength concrete, which reduces the thickness of the deck. Nevertheless, the high dead load stresses, primarily due to the horizontal forces from the cables throughout the length of the bridge, result in increased creep of the concrete.

The effect of creep on the carrying capacity of short and medium length composite girder bridges is considered in the AASHTO specifications. For cast-in-place concrete, AASHTO requires a threefold increase of the modular ratio, defined as the ratio of steel modulus of elasticity to concrete modulus of elasticity. For example, for 6,000 psi concrete, the modular ratio for loads of short duration (live, earthquake, wind loads etc.) is 6 while for loads of long duration (dead loads etc.) is 18. This increase of the modular ratio implies that the modulus of elasticity of concrete (or the concrete stiffness) for long-term loads is three times smaller than the modulus of elasticity (or concrete stiffness) for short-term loads.

Although shrinkage is not specifically mentioned as contributing in the increase of the modular ratio, the AASHTO procedure has apparently worked satisfactorily for conventional composite girder bridges. For cable-stayed bridges, however, this approach to resolving the creep and shrinkage problem results in an uneconomical solution and is clearly inadequate.

MODULAR RATIO

The size of concrete and steel sections in composite members depends on the relative stiffness of the two materials. Consideration of the creep and shrinkage effects is essential in the design of composite members. Thus, composite members are sized considering the short and long-term stiffness of concrete using the transformed area method and the modular ratio of concrete for short and long-term loads.

For composite cable-stayed bridges, the effect of creep and shrinkage is controlled through the application of strict requirements on the long-term modular ratio. In actual designs of composite bridges this ratio has been specified as low as 11 (1,2,3).

The forces in the composite top chord members of a recent design of a two lane cable-stayed truss bridge are used to demonstrate the benefit of using a low long-term modular ratio. Dead plus live loads due to HS-20 loadings are considered. Assuming uniform concrete properties throughout the deck, the modular ratio for live loads is taken as 6, while for dead loads it is varied from 11 up to 18.

Table 1 shows the stresses in the steel and concrete corresponding to different long-term modular ratio N_L values. It also shows the change of stresses in the steel and the concrete for modular ratio values higher than 11. It is seen that for the increase of the modular ratio from 11 to 12 the steel stresses increase by as much as 5.3% and the concrete stresses decrease by as much as 2.6%. Similarly, for the modular ratio increase

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TABLE 1: STEEL AND CONCRETE STRESSES IN TOP CHORD

Top Chord Member	STEEL							CONCRETE						
	Stresses in Ksi				Percentage Increase with respect to $N_l=11$			Stresses in Ksi				Percentage Decrease with respect to $N_l=11$		
No	$N_l=11$	$N_l=12$	$N_l=13$	$N_l=18$	$N_l=12$	$N_l=13$	$N_l=18$	$N_l=11$	$N_l=12$	$N_l=13$	$N_l=18$	$N_l=12$	$N_l=13$	$N_l=18$
U02'-U02	-10.3	-10.3	-10.3	-10.3	0.1	0.2	0.6	-0.13	-0.13	-0.13	-0.13	0.1	0.3	0.9
U02 -U06	-10.9	-10.9	-10.9	-11.0	0.2	0.3	1.0	-0.28	-0.28	-0.28	-0.27	0.1	0.3	0.8
U06 -U10	-18.9	-18.9	-18.9	-19.0	0.1	0.2	0.8	-0.30	-0.30	-0.29	-0.29	0.0	0.1	0.3
U10 -U14	-24.7	-24.8	-24.8	-24.8	0.0	0.1	0.2	-0.51	-0.50	-0.50	-0.48	0.9	1.8	5.7
U14 -U18	-24.0	-24.2	-24.4	-25.2	0.8	1.6	5.1	-0.68	-0.67	-0.66	-0.63	1.3	2.6	8.1
U18 -U22	-23.0	-23.3	-23.7	-25.1	1.5	2.9	9.1	-0.81	-0.80	-0.79	-0.73	1.6	3.2	10.0
U22 -U26	-23.8	-24.3	-24.7	-26.7	1.9	3.8	12.0	-0.96	-0.94	-0.93	-0.85	1.7	3.4	10.8
U26 -U30	-21.6	-22.1	-22.6	-24.7	2.4	4.6	14.4	-1.04	-1.02	-1.00	-0.91	2.1	4.0	12.6
U30 -U34	-21.3	-21.8	-22.4	-24.6	2.6	5.1	15.7	-1.16	-1.14	-1.11	-1.00	2.3	4.5	13.8
U34 -U38	-20.2	-20.8	-21.4	-24.0	3.2	6.2	19.1	-1.26	-1.23	-1.20	-1.08	2.4	4.6	14.3
U38 -U42	-20.3	-21.0	-21.7	-24.4	3.4	6.5	20.0	-1.35	-1.31	-1.28	-1.15	2.5	4.8	14.7
U42 -U46	-20.7	-21.4	-22.1	-25.1	3.6	7.0	21.4	-1.43	-1.40	-1.37	-1.22	2.5	4.8	14.7
U46 -U50	-21.2	-22.0	-22.7	-25.9	3.7	7.2	22.0	-1.51	-1.48	-1.44	-1.29	2.5	4.8	14.7
U50 -U54	-20.8	-21.6	-22.4	-25.5	3.9	7.5	22.8	-1.55	-1.51	-1.47	-1.32	2.6	5.0	15.1
U54 -U58	-20.9	-21.7	-22.5	-25.8	4.0	7.7	23.5	-1.60	-1.55	-1.52	-1.35	2.6	5.0	15.2
U58- U62	-20.7	-21.6	-22.4	-25.8	4.1	8.0	24.4	-1.64	-1.60	-1.56	-1.39	2.6	5.0	15.2
U62 -U66	-19.8	-20.7	-21.5	-25.1	4.5	8.8	26.8	-1.64	-1.60	-1.56	-1.39	2.6	5.0	15.2
U66 -U70	-18.0	-19.0	-19.8	-23.6	5.2	10.1	30.7	-1.73	-1.69	-1.65	-1.47	2.6	5.0	15.4
U70 -U74	-17.9	-18.8	-19.7	-23.3	5.2	10.0	30.6	-1.72	-1.67	-1.63	-1.46	2.6	5.0	15.3
U74 -U78	-17.5	-18.4	-19.3	-22.9	5.2	10.0	30.6	-1.69	-1.64	-1.60	-1.43	2.6	5.0	15.3
U78 -U82	-17.3	-18.2	-19.0	-22.6	5.3	10.2	31.2	-1.66	-1.62	-1.58	-1.41	2.5	4.9	14.9
U82 -U86	-16.5	-17.4	-18.2	-21.7	5.3	10.2	31.2	-1.59	-1.55	-1.52	-1.36	2.5	4.9	14.9
U86 -U90	-15.8	-16.6	-17.4	-20.7	5.2	10.1	31.0	-1.53	-1.49	-1.45	-1.30	2.5	4.8	14.7
U90 -U94	-15.2	-16.0	-16.7	-19.9	5.3	10.3	31.5	-1.47	-1.43	-1.40	-1.26	2.4	4.6	14.2

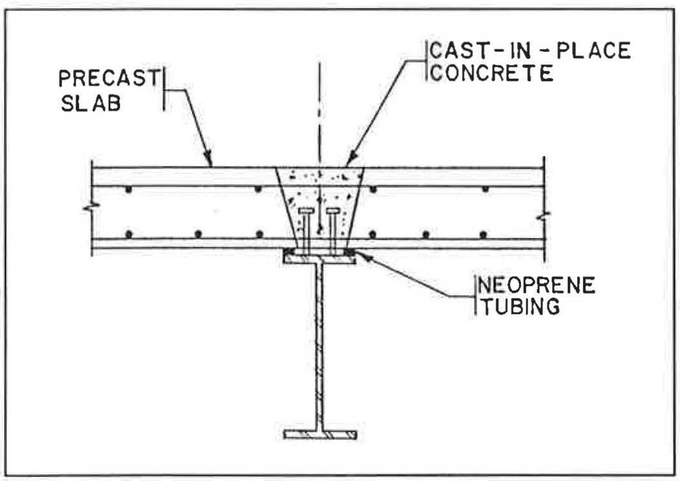

Figure 1 Precast panel connection detail

from 11 to 13, the corresponding maximum change of the steel and concrete stresses is 10.3% and 5.0% respectively.

It is further interesting to note the steel and concrete stresses for the long-term modular ratio 18 specified by AASHTO for cast-in-place concrete. The steel stresses increase by as much as 31.5% and the concrete stresses decrease by as much as 15.4%. This change of stresses implies a considerable reduction in the long-term participation of the concrete deck in carrying loads and, consequently, diminished effectiveness. For this reason, the use of concrete with long-term modular ratio of 18 would be uneconomical.

ERECTION CONSIDERATIONS

Although a few composite cable-stayed bridges adopted cast-in-place concrete roadway decks, most bridges have used precast deck panels. The precast deck panels are fabricated and cured under carefully controlled conditions and allowed to mature for an extended period of time. This procedure improves the creep properties of the concrete and, at the same time, removes a considerable percentage of the concrete shrinkage prior to the application of loads to the panels on the bridge.

Most often, the erection of the girders is performed by repeating a cycle of assembling steel components, cables and deck panels. The connection of the concrete with the steel is achieved through shear connectors. Figure 1 shows a connection detail of this type using shear studs. At the end of an erection cycle, cast-in-place concrete is used to fill small openings in the deck panels to achieve the composite action between concrete and steel. Additional cast-in-place concrete is used to fill openings between individual panels. Figure 2 shows two arrangements of deck panels on bridge roadways and the cast-in-place sections between them which have been used for securing monolithic action of the roadway deck.

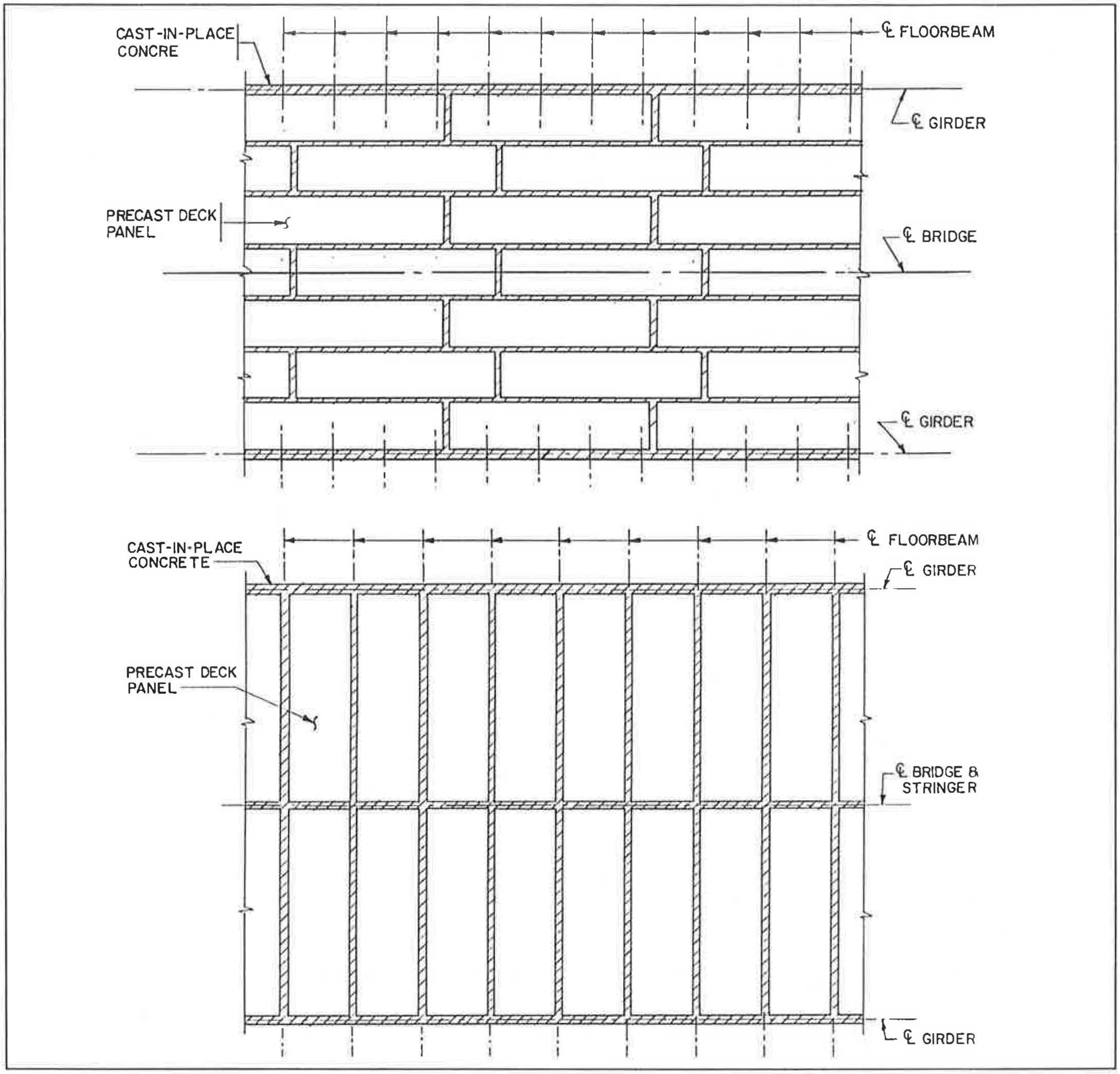

Figure 2 Two arrangements of precast deck panels on bridge roadways

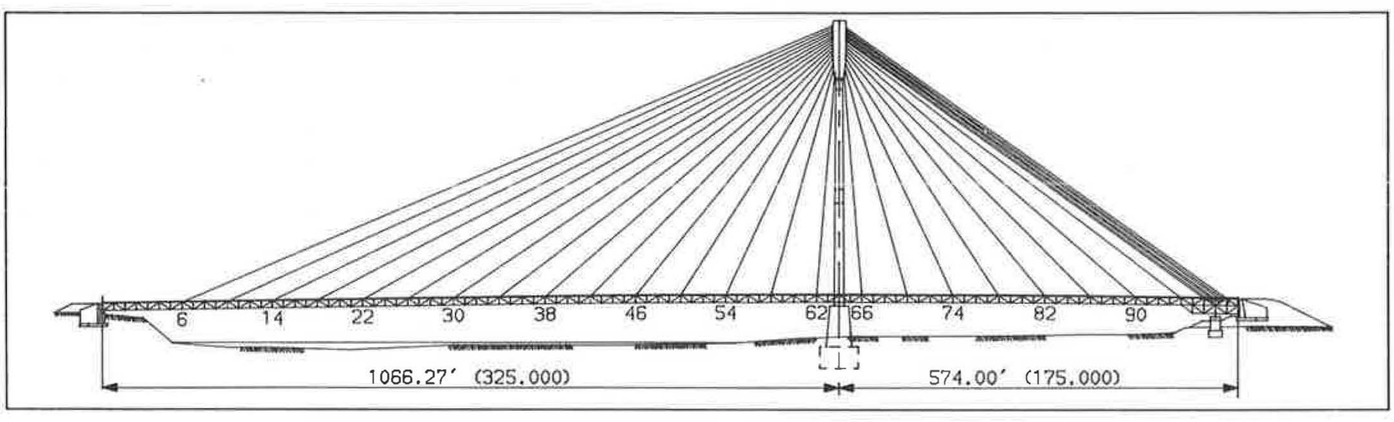

Figure 3 Karnali Bridge elevation

The erection procedures and in particular the timing of placing the cast-in-place concrete affect significantly the distribution of the dead load forces between concrete and steel. Consider the Karnali River Bridge which is a one tower asymmetric composite truss cable-stayed bridge, figure 3, currently under construction in Nepal. The distribution of the dead load forces in the steel and concrete components of the top chord prior to achieving composite action and at the completion of the erection are shown in figure 4. It is seen that the dead load forces of the concrete and steel components of the top chord are not the same throughout the length of the bridge. Sections of the chord near the tower carry higher forces than sections further away. By adjusting the erection procedures, it is possible to modify the level of dead load forces distributed between concrete and steel both prior to as well as after achieving composite action.

It is clear that the deck of composite cable-stayed bridges with precast panels essentially consists of precast and cast-in-place concrete sections. The cast-in-place sections do not undergo the rigorous curing and extended maturing of the precast concrete panels and they appear to be in relative disadvantage regarding their creep and shrinkage properties. To minimize or even eliminate the effect of shrinkage, shrinkage compensating cement may be implemented. Three factors appear to further limit the consequences from this apparent disadvantage of the cast-in-place concrete:

1. The relatively small percentage of the cast-in-place concrete which is typically around 18 percent of the total concrete volume.
2. The dead load force distribution in the roadway deck along the length of the bridge which shows a significant reduction of the forces away from the tower as shown in figure 4.
3. The history of dead load application during erection as discussed below.

Assuming a ten day erection cycle for a typical bridge segment between consecutive stays, figure 5 shows the loading history during erection of three sections of the concrete deck in the main span. It can be seen that the sections of the deck with the highest stresses are loaded at the slowest rate and receive their full load after a considerable period of time.

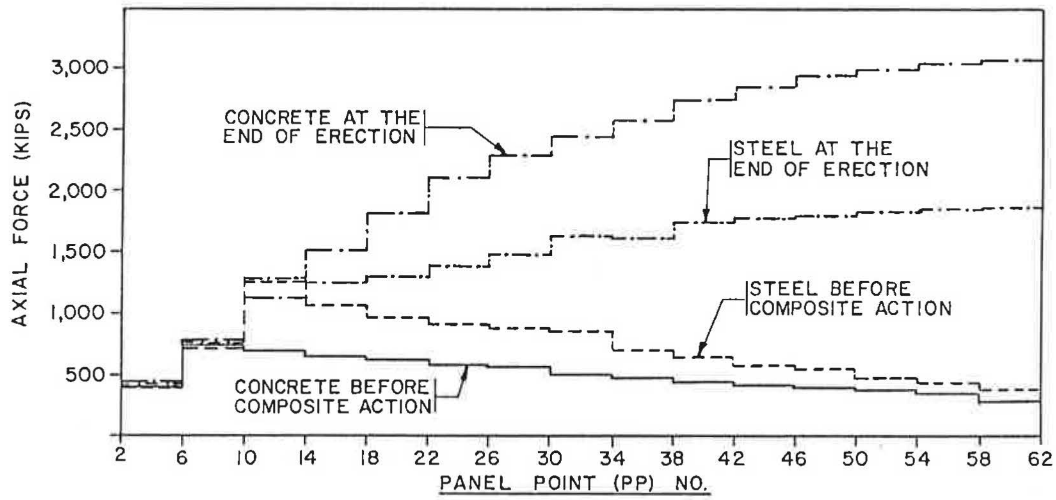

Figure 4 Dead load force distribution in the concrete and steel of the top chord in main span.

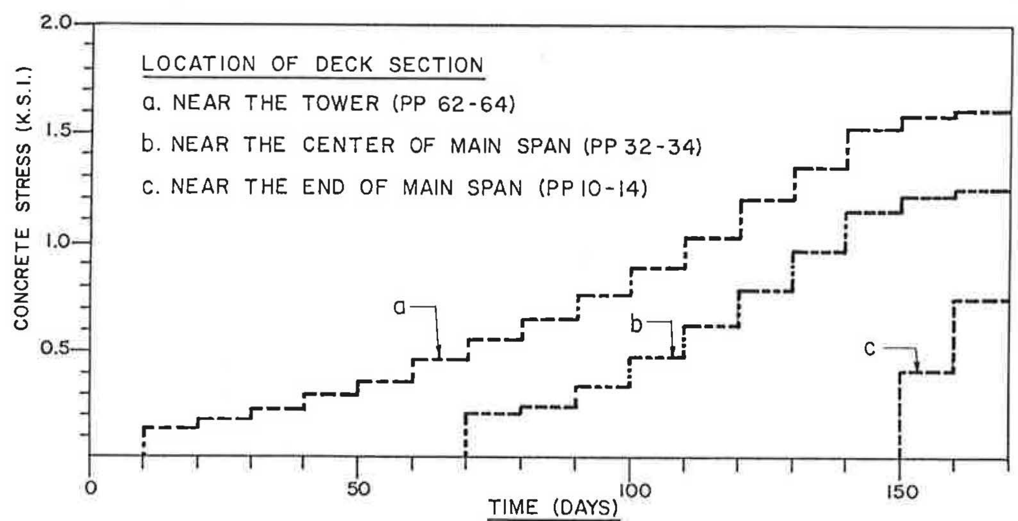

Figure 5 Loading history of three deck sections during erection.

CREEP TESTING PROGRAM

Although the long-term modular ratio of a given concrete mix can be estimated using calculation methods based on fresh and hardened concrete properties (4), a better value can be developed based on actual creep tests conducted on several candidate concrete mixes. Creep testing is conducted in accordance with ASTM C512-87 entitled "Standard Test Method for Creep of Concrete in Compression" (5).

Creep tests are conducted by subjecting standard 6x12 inch concrete specimens to a sustained compressive load and at specified time intervals measuring changes of the concrete strain. To account for strain resulting from drying shrinkage, drying induced strains in companion unloaded specimens are measured and the resulting strains are subtracted from load induced strains. Creep tests and the corresponding shrinkage tests, are conducted in a controlled temperature and humidity room maintained at $73.4 \pm 3.0^{\circ}F$ and $50 \pm 4\%$ relative humidity.

The imposed load for creep testing may be as high as 40 percent of the compressive strength of the concrete measured at the age of loading. If the stress level in the structure is known, it is desirable to conduct the test at that stress level. However, if the stress level in the structure varies or it is not known, a stress of between 30 and 40 percent of the concrete strength may be safely used. Several researchers (6,7) have reported that for stress levels less than about 40 to 50 percent of concrete strength, creep strains are approximately proportional to the sustained stress and obey the principal of superposition of strain history.

Because age of loading has a profound effect on the creep properties of any concrete, creep tests are conducted at several different ages. Typical loading ages include 2, 7, 28, 90 days and 1 year. Later age loading is desirable especially if the construction schedule is such that deck panels will not be loaded until long after they are cast. Because it is desirable to have at least 3 months of creep data on which to base long-term modular ratio predictions, it is apparent that creep tests must be started early in the construction phase of a project. If this is not possible, the effect of loading age on long-term modular ratio can be estimated from a series of creep tests performed at loading ages between 2 and 28 days.

LONG TERM MODULAR RATIO BASED ON CREEP TESTS

The short-term modular ratio is denoted as

$$N_s = \frac{E_s}{E_c} \qquad (1)$$

where

N_s = short-term modular ratio
E_s = modulus of elasticity of steel
E_c = modulus of elasticity of concrete

and the long-term modular ratio as

$$N_l = \frac{E_s}{E_{eff}} \qquad (2)$$

where

N_l = long-term modular ratio
E_{eff} = long-term effective modulus of elasticity of concrete

The long-term effective modulus of elasticity of concrete includes the effect of initial elastic deflection and long-term deflection due to creep and shrinkage. If the importance of shrinkage is minimized through measures as considered above, the long-term effective modulus of elasticity of concrete can be expressed in terms of the modulus of elasticity at time t and the ultimate creep coefficient as shown in the following expression (4)

$$E_{eff} = \frac{E_c(t)}{1 + \phi_u} \qquad (3)$$

where

$E_c(t)$ = modulus of elasticity of concrete at time t
ϕ_u = ultimate creep coefficient defined as the ratio of ultimate creep strain to initial strain.

Using the above relationships, the long-term modular ratio can be expressed in terms of the short-term modular ratio and the ultimate creep coefficient as follows

$$N_l = N_s(1 + \phi_u K_a \gamma_l \gamma_h) \qquad (4)$$

where

K_a = loading age correction factor, 1.0 at 7 days
γ_l = humidity correction factor
γ_h = element thickness correction factor

Creep analysis based on the above approach is appropriate only when the gradual changes of stress due to creep are relatively small and do not result in fundamental change in the distribution of stresses and the response of a structure.

The ultimate creep coefficient needed for the prediction of the long-term modular ratio can be established from creep tests. As shown in equation (4), several adjustments must be made to the ultimate creep coefficient to account for age of loading of the creep test specimens as compared to the actual members in the structure, effects of member size as compared to the standard test specimen size, and ambient humidity conditions at the site as compared to the humidity conditions at the laboratory. The following paragraphs describe how creep test data and knowledge of site and specific structure conditions can be used to estimate long-term modular ratio.

According to reference 4, the creep coefficient at time t for loading age of 7 days for moist cured concrete and for 1 to 3 days steam cured concrete can be expressed in the following form

$$\phi_t = \frac{t^{0.6}}{f + t^{0.6}}\phi_u \qquad (5)$$

where

ϕ_t = creep coefficient at time t

ϕ_u = ultimate creep coefficient

f = half time in days

Applying regression analysis to test data for concrete specimens loaded after 7 days of moist curing, values can be determined for the half time f and the ultimate creep coefficient ϕ_u. Reasonable values for the ultimate creep coefficient can be obtained after 90 days of creep test data become available.

Figure 6 shows creep test data the authors developed for the three potential concrete mixes of Table 2 for use for the Karnali River Bridge. Each mix was specifically designed to minimize creep and shrinkage. In the effort to minimize creep, the concrete compressive strength exceeded the required by strength considerations. The final selection of mix No. 2 was based on its creep as well as workability characteristics.

All three concrete mixes were loaded after 7 days of moist curing. Test data are expressed in terms of specific creep values which are converted to values of the creep coefficient ϕ_t by multiplying with the modulus of elasticity. Using these data, ultimate creep coefficients of 1.34, 1.42 and 1.57 were calculated for concrete mixes denoted 1, 2 and 3 respectively. Reference 4, indicates that the ultimate creep typically ranges from 1.3 to 4.15.

The same three concrete mixes were also subjected to creep tests after 14 and 28 days of moist curing to establish the effect of loading age on the ultimate creep coefficient. According to reference 4, the correction factor for loading age of concrete loaded at ages subsequent to 7 days of moist curing has the following form

$$K_a = A t_l^{-b}$$

TABLE 2: MIX PROPORTIONS AND PROPERTIES OF FRESH CONCRETE

Material	Quantity in a Mix per Cubic Yard No. 1	No. 2	No.3
Cement, lb	750	754	711
Fine Aggregates, lb	1128	1183	1186
Coarse Aggregates, lb	1886	1768	1771
NP-20 (HRWR), oz	274	184	142
Pozzolith 300-N (WR), oz	45	30	28
AEA 303A (AEA), oz	18.8	12.8	4.6
Water, lb	228	264	277
Parameter			
Slump, in	2.2	2.6	2.9
Unit Weight, lb/ft3	148.6	147.2	146.6
Air Content, %	3.5	3.1	3.2
Water to Cement Ratio %[1]	32.8	36.6	40.4

[1]Water to cement ratio includes water in admixtures

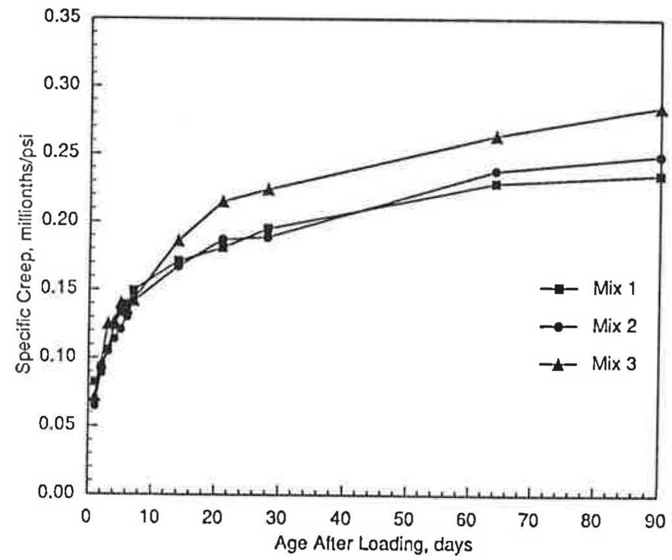

Figure 6 Specific creep of three concrete mixes loaded after 7 days moist cure

where

A = coefficient based on mix parameters
t_l = loading age in days
b = exponential factor based on mix characteristics

While reference 4 suggests average values of 1.25 for A and 0.118 for b, specific values for given mixes can be developed using creep data for concrete mixes loaded at varying ages. Figures 7 and 8 show creep data for the three mixes mentioned above loaded at 14 and 28 days. Although there was a measurable reduction in specific creep of mix No. 1 loaded at 14 days compared to 7 days, mixes No. 2 and 3 did not show any reduction and in fact were nearly identical to values for 7 days loading. This behavior may in part be attributed to the strength gain of mixes No. 2 and 3. Specific creep for all three mixes was reduced at loading age of 28 days as shown in figure 8.

Using the above data, it is possible to develop specific mix loading age correction factors. For the mix denoted No. 2 a value of 1.296 is obtained for A and 0.133 for b. Consequently, if the load in the structure is applied after the concrete reaches an age of one year, the ultimate creep coefficient determined using creep test loaded at 7 days is reduced by a factor of 0.59. Other loading ages for mix No. 2 will result in the following correction factors to be applied to the ultimate creep coefficient.

Loading Age	Correction Factor
7	1.00
14	0.91
28	0.83
56	0.76
120	0.69
180	0.65
270	0.62
365	0.59

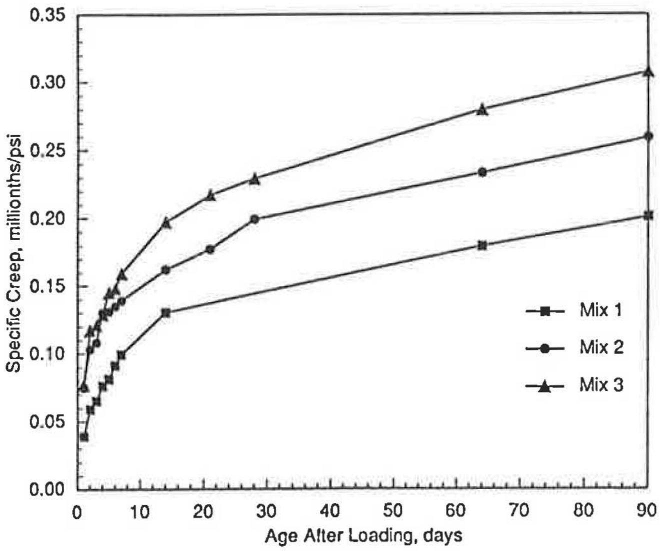

Figure 7 Specific creep of three concrete mixes loaded after 14 days moist cure

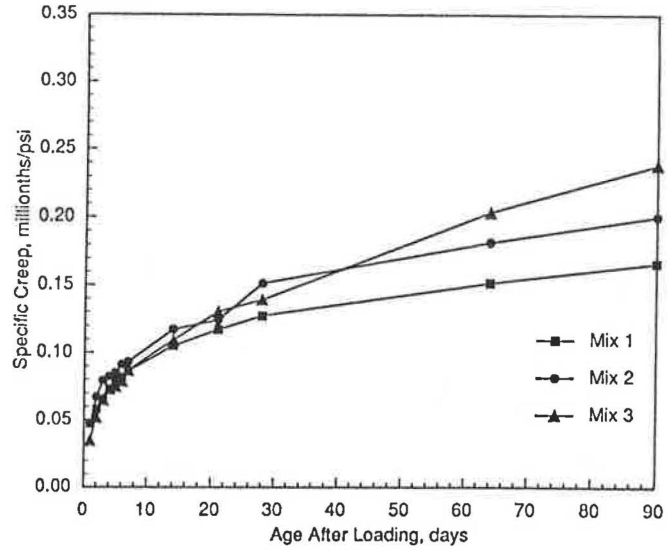

Figure 8 Specific creep of three concrete mixes loaded after 28 days moist cure

Thus, it is apparent that longer preloading periods can significantly reduce the ultimate creep coefficient resulting in a proportional reduction of the long-term modular ratio. If the entire load is not applied at a discrete point of time but rather over an extended period as shown in figure 5, a suitable correction factor must be selected to account for this effect. Furthermore, since the largest change in the loading age correction factor occurs during the early ages, much benefit can be gained by small delays in the early application of loads.

The ultimate creep coefficient must be further adjusted for the specific site conditions of average relative humidity and element thickness. Both of these adjustments are straightforward and well documented in reference 4.

Table 3 presents the long-term site specific modular ratios developed for the three concrete mixes investigated by the authors. As anticipated, the mix with the highest compressive strength and modulus of elasticity developed the lowest short and long-term modular ratios.

CONCLUSION

This paper identifies the requirements for the creep and shrinkage properties for the concrete deck of composite cable-stayed bridges and presents a rational evaluation of the modular ratio of the concrete mixes used in a project. This approach further provides the means for consideration of the specific material, project and site data in the evaluation of the creep and shrinkage effects. It was found that:

1. Long-term modular ratios based on code suggested values are overly conservative for certain concretes and values should be established using creep tests with material, project and site specific data.

2. It is possible to base long-term modular ratio estimates using creep test data from specimens which have been subjected to 90 days of loading.

3. Long-term modular ratios can be significantly reduced by delaying the application of loads.

4. Concrete mixes can be specifically designed to have low short and long-term modular ratios.

ACKNOWLEDGEMENT

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TABLE 3: LONG TERM MODULAR RATIO FOR THREE CONCRETE MIXES

Mix No	Compressive Strength 28 days, psi	Modulus of Elasticity ksi	Short-term Modular Ratio	Creep Coefficient 7 days	Loading Age Factor 1 Year	Humidity Correction Factor	Thickness Correction Factor	Long-term Modular Ratio
1	9500	4760	6.09	1.342	0.422	0.821	0.947	8.8
2	7890	4534	6.40	1.419	0.591	0.821	0.947	10.6
3	7300	4345	6.67	1.573	0.623	0.821	0.947	11.8

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Urban Second Level Bridges Built with Precast Segmental Construction

JOHN A. CORVEN

Transportation facilities in most major urban areas in the United States are operating far beyond their design capacities. Extensive rehabilitation, renovation and new construction is required if we hope to meet our future needs. The challenge facing the engineering community is to develop techniques to provide increased capacity, improved safety, at reasonable cost, while minimizing the impact on the operation of the current facility. Second level bridges built with precast segmental construction provide one important solution to this challenge.

Precast segmental second level bridges can offer an effective solution to relieving traffic congestion of many of our urban areas. Additional traffic lanes are added on the second level, typically for through traffic leaving the at-grade facility to operate within its design capacity. The new structures are designed in accordance with current geometric criteria (such as site and stopping distances and shoulder widths). The existing facility can also be upgraded to comply with current geometric standards. The construction costs of the elevated structures have proven to be very competitive, especially when comparing the high expense, and near impossible task, of securing the surrounding right-of-way. Finally the construction techniques developed of building from above and the elimination of underlying falsework allow the current facility to operate with minimal disruption during the construction of the elevated bridges.

The recent design and construction of three major urban viaducts has developed this special aspect of segmental construction, the construction of elevated urban bridges. Criteria for these bridges are not necessarily the same as other bridge structures. These criteria include aesthetic considerations, geometry layout of roadways to minimize impact on underlying traffic and reduce construction costs, innovative drainage techniques, and special substructure elements designed to meet the complexity of crossing over underlying roadways.
To describe these criteria three recent projects are discussed. These are:

• **I-110, Biloxi, Mississippi** — $40 million urban structures built through residential area, and over 4-lane U.S. 90 highway, while maintaining traffic. Segmental portion of the first contract contains 316,600 square feet, with 300,000 square feet in Contract II. The structures were opened to traffic February 19, 1988.
• **San Antonio "Y," Texas Projects IA, IIB and IIIA&B of I-10 & I-35** — $65 million urban structures built in limited right-of-way over streets and railroads. These projects are the first phase of reconstruction of the downtown urban freeways of San Antonio, which will ultimately consist of a new 10-lane expressway with provision for HOV lanes. Over 1.25 million square feet of elevated roadway deck.
• **U.S. 183 Elevated Viaduct Project in Austin, Texas** — The project has two parallel structures approximately two miles each providing a six-lane freeway and directional ramps to IH-35. The bridge deck area is approximately 1.25 million square feet with an estimated cost of $60 million. Construction is anticipated to begin in May 1991.

Figg Engineernig Group, 424 North Calhoun, Tallahassee, Fl 32301

PROJECT 1: BILOXI I-110 VIADUCT

An elevated viaduct constructed within minimal clearance and almost zero right-of-way limits solved a dilemma for Biloxi, Mississippi: How to expand interstate systems in developed urban areas where highways must wrap around existing structures; minimize the impact to the environment when urban structures are built; maintain the integrity of established neighborhoods; utilize construction systems that don't interfere with day-to-day traffic flow; and, provide an aesthetically pleasing road and bridge system for a city.

Highway Department officials had not completed the Biloxi Interstate-II0 connector between Interstate-10, a vital route, with U. S. Highway 90, which follows the coast of the Gulf of Mexico. The connecter stopped a mile before reaching U.S. 90. The missing link had to traverse an neighborhood and then a business community that was immediately adjacent to the Gulf of Mexico. The existing buildings made it almost impossible to acquire right-of-way land to build exit ramps. Additionally, highway department officials did not want to disrupt the community or stop traffic on what is a major tourist route.

An elevated viaduct with exit and entrance ramps that circle out into the Gulf of Mexico at its Southern end solved the problem. Concrete segmental technology was an excellent solution for zero right-of-way construction.

Project Description

The Biloxi Interstate-II0 Viaduct is a four-lane elevated highway consisting of 5,332' of mainline structure and 4,424' of ramps, for a total precast concrete segmental bridge deck area of 616,600 square feet. The bridge foundations are cast-in-place concrete columns supported on 95,000' of precast, prestressed concrete piles. Two of the five ramps reach across a U.S. Highway 90, on 600' and 650' radii curves using the cantilever erection method.

Twin concrete box girders, each made of 40' wide precast segments, comprise much of the mainline structure.

The ramps and transition areas feature 30' wide precast segments. All of the segments are 7' deep, with typical segments 10' in length. In ramp transition areas, as many as four adjacent box girders, joined with cast-in-place reinforced longitudinal closure strips, make up the bridge deck.

Casting of the prestressed piles and 2,164 segments required for the bridge was done at a casting yard 35 miles from the project site. The precast elements were then trucked to the project for incorporation into the structure.

Significant Bridge Features

• The viaduct completed a missing link connecting Interstate-10 with U.S. 90 on the Mississippi Gulf Coast. The new highway provides a much needed evacuation route in the event of hurricanes. (Figure 1)
• The aesthetics of the structure were also critical. Because the viaduct had to travel through an already developed residential and business community, residents and tenants tended to be reluctant to accept a new highway structure. (Figure 2)
• The design included a concrete box girder of the same depth and shape throughout the superstructure and the cast-in-place concrete piers were given a special rustication. In addition, a spray surface finish, Texcote, was applied to both the cast-in-place and precast concrete surfaces for added aesthetics. (Figure 3)
• The box girder cross section and substructure design allowed the bridge to be constructed within severely restricted rights-of-way. As many as four box girders are used transversely to form the bridge deck (Figure 4)
• This elevated, totally concrete, structure was built using two different construction methods, span-by-span construction and balanced cantilever construction. Traffic is riding on the as-cast concrete surface.
• The superior quality control achieved in the precast yard, together with both longitudinal and transverse prestressing maximize the structure's durability.
• Construction proceeded quickly. A maximum of 122 segments per month were produced at the casting yard using eight casting machines. The erection subcontractor was able to erect a maximum of 14 spans, or approximately 1600 linear feet of bridge, per month. Traffic was maintained on the busy U.S. Highway 90 while overhead ramps, with 160' and 180' spans, were being erected by using the cantilever erection method. (Figure 5)
• Precast prestressed concrete provided an economical solution for the Interstate-II0 connector in Biloxi. The $40.2 million bid for the project was $4.2 million less than the lowest bids for the conventional alternates.

PROJECT 2: SAN ANTONIO "Y"- INTERSTATES 10 & 35

Like many large American cities in the sunbelt, San Antonio's highway and road system was stretched to the limits during the population explosion of the early 1970s. Other cities have added mazes of interconnecting ramps and extensively widened highways. San Antonio chose a state-of-the-art highway-design concept -- building up instead of out -- to solve traffic problems downtown.

The concept of double-decking, or building a highway over an existing highway, was a excellent solution to a critical problem. Traffic capacity in the connection between Interstates-35 and -10 was effectively doubled without using a corresponding amount of right-of-way land for the expansion of highway space. Double-decking cancelled the problem many cities face: How to finance the purchase of fully developed commercial land to widen highways.

The concept of building the six-mile section of freeway over the existing facility with minimum disruption of traffic was simple but elegant. The sequence of construction was to build the elevated out-bound legs of the "Y" first, generally in the outer separations between the main lanes and the frontage road. After these sections were operating, the in-bound legs were let to contract. The ultimate section will provide as many as ten lanes of freeway.

The freeway design selected required a structural system that could be built over traffic. A significant feature of the concrete segmental box girder system designed for the project is that the surface area required to construct the overhead viaduct is only the dimensions of the supporting piers. The ground level freeway lanes, the shoulders, the ramps, and the frontage roads can be placed or allowed to remain under the box girders and the cantilever wings for a maximum utilization of space. In addition to the ability to be constructed over traffic, the structure had to be aesthetically pleasing to the community and traveling public. The design has a slender trapezoidal box and long tapering cantilever wings. (Figure 6)

Project IA The first design section of the Downtown "Y" was Project IA. This structure carries Interstate I-35 traffic south away from the interchange of I-10 and I-35. The project is 7,450' long. Four ramp structures totalling 2,540' in length give access to the elevated structure from the city streets. The total square footage is 366,615 square feet.

The typical main lane roadway width is 42'. Ramp structures are 26' wide. Transition zones widen where ramps and main line structures join. Span lengths vary between 70' and 110' to permit pier placements outside of existing roadway. The typical span is 100'.

The typical substructure of the IA Project consists of cast-in-place twin wall columns resting on cast-in-place footings that are supported by twin 42' diameter drilled shafts. The twin wall piers were selected for longitudinal flexibility that when used with elastomeric bearings allow continuous units of 720' in length.

Though span length adjustments were made as required to miss underlying facilities, special substructure elements were required. Three types of piers were used. These are cantilever piers, straddle bents, and T-Piers. All of these piers are cast-in-place concrete. Post-tensioning is used to control critical flexural stresses in the caps and shafts of these piers.

The bridge deck is a precast box girder with cantilever wings. The typical main line cross section has a 42' wide top slab and an 8' wide bottom slab. The depth of girder is 5'-10" and webs are sloped at 2.5:1. The resulting cantilever wings are 15'-4" long.

A typical span is is made of 11 typical segments 9' in

Figure 1. Biloxi, I-110. Aerial view of completed elevated structure.

Figure 2. Biloxi, I-110. Aesthetics were key to acceptance of the elevated structure as it passes through city parks.

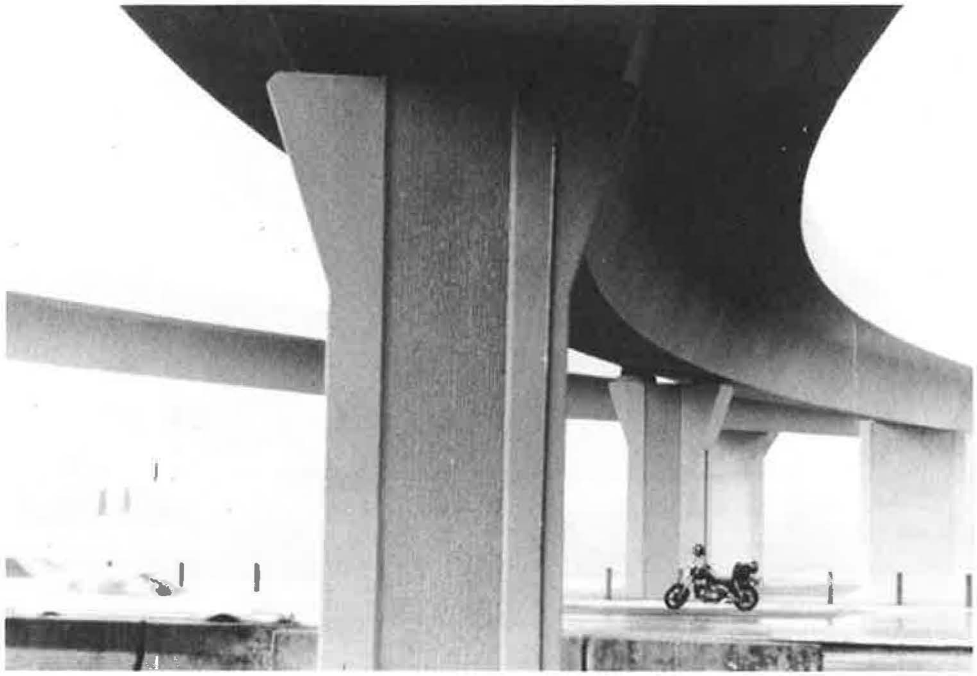

Figure 3. Biloxi, I-110. Aesthetic treatment of piers.

Figure 4. Biloxi, I-110. As many as four precast box girders were used side-by-side to provide the total bridge deck width.

Figure 5. Biloxi, I-110. Balanced-cantilever construction over U.S. 90. Lane closures were made only as segments directly above were placed.

Figure 6. San Antonio "Y". Aerial view of IIIA&B Project. Second-level bridges are placed within existing right-of-way. All aspects of highway geometry are accommodated in span-by-span precast segmental construction.

length. A typical segment weight is 90,000 lbs. Pier segments 9' in length are placed on top of the cast-in-place piers. The segments are assembled using the span-by-span method of construction. (Figures 7 and 8) Twin triangular trusses support the segments temporarily, while closure joints at either end of the span are being cast. Segments can be delivered on the ground or over the completed portion of the bridge deck. A typical span is prestressed with three 12 x 0.6" diameter strand tendons and one 19 x 0.6" diameter strand tendon per web. In addition seven and four strand tendons of 0.6" diameter are placed in the cantilever wings to assure longitudinal continuity.

Project IIB The second leg of the Downtown "Y" was Project IIB. This structure carries Interstate 35 traffic north away from the intersection of I-10 and I-35. The project is 5,817' long along the main lanes. Two ramp structures with a total length of 1,474' complete the bridge layout. The total square footage of this project is 350,055 sq. ft. The typical roadway for this project is 58' wide. Ramp structures as with the IA Project are 26' wide. The 63 main line spans vary in length between 78' and 110'. The typical span is 100'.

As in the IA Project, the typical substructure are cast-in-place twin wall piers resting on cast-in-place footings supported by two 48" diameter drilled shafts. Cantilever piers and straddle bents were required on this project as in the first project in order to provide horizontal and vertical clearances above underlying roadways.

A typical segment has a 58' wide top slab and a 10' wide bottom slab. The depth of the structure is again 5'-10" and the web slopes 2.5:1. The resulting cantilever wings are 22'-6" long. Unlike the IA Project the cantilevers for the IIB Project, which vary in thickness from 10" to 2'-3", are voided to reduce superstructure dead-load. A typical span on the IIB Project is made of 12 typical segments with length of 8' and 5'-6" pier segment halves placed a either end. Different from the IA Project were closure joints were made between the pier segments and the typical segments of the span, the IIB Project has closure joints centered over the tops of the piers between pier segment halves.

Construction is again by the span-by-span method and the post-tensioning of a typical span is 3 12 x 0.6" diameter strand tendons and 2 19 x 0.6" diameter strand tendons per web. The post-tensioning is stressed in two stages. First, two tendons of 19 strands each are stressed forming simple spans. Next, the cast-in-place closure joints between the pier halves over the piers are poured and the continuity 12 strand tendons are stressed from both ends of the four span units.

Project IIIA&B The third portion of the Downtown "Y" to be designed was the IIIA&B Project. (Figures 9 &10) This structure carries traffic on I-10 away from the interchange for a length of 8,955'. The three ramp structures have a total length of 2,996'. The precast superstructure of this bridge is similar to the IIB Project in that it has a 58' wide top slab. Ramps as before are 26' wide. The deck square footage of this project is 567,310 square feet. The main line superstructure has 89 spans varying between 80' and 110'. The typical span is again 100'.

The substructure for this project is cast-in-place solid rectangular shafts supported by a twin 48" diameter drilled shaft. Solid shafts were used on this project as opposed to the twin walled shafts of the previous two projects as a result of shorting the continuous units to two spans. Special substructure again was required in the form of C-Piers, straddle bents, and T-Piers.

A typical span of 100' was made up of 15 segments at approximately 6' in length. Two pier segment halves at either end of the span, approximately 4'-4" in length complete the typical span. The construction procedure is similar to the IIB Project where simple spans are first formed, closure joints placed between the spans and then continuity post-tensioning placed to complete the continuous unit.

Construction Bids for the first section were taken in December 1984, the second section was bid in June 1985, and the third section was bid in May 1986. In each case, the successful contractor bid the alternate precast segmental design. The total area of the three projects is 1,283,980 square feet at a total cost of $43,977,000 or $34.25 per square feet.

Austin Bridge Company of Dallas, Texas, is the contractor for the first and second project and The Prescon Corporation of San Antonio, Texas, is the contractor for the third. The first two projects of the Austin Bridge Company were completed in May and September of 1989, and The Prescon Corporation's project was completed in August 1989.

Each contractor established casting yards about nine and six miles, respectively, from the projects and hauled the segments over city streets for a portion of the distance. Segment weight limitations were established to avoid damage to the streets.

Span-by-span was the construction method selected for all projects. Erection trusses support the segments while the match-cast joints are epoxied together and the entire span is post-tensioned with external tendons inside the boxes. Austin Bridge Company placed three sets of erection trusses on their projects while The Prescon Corporation has a single set of trusses. Where space is available, each contractor uses ground-mounted cranes to set segments and move the trusses. However, many of the spans have been erected from the top of the previously completed structure using deck-mounted cranes because of limited space and traffic interference.

Many amenities of urban highways have been incorporated into the structures. Surface drainage is collected in roadway scuppers and carried to the interior of the box and finally discharged through the piers into a storm drainage system. Signal and electrical conduits are cast into the concrete so that the fixtures are attached to the structure without exposed wiring. Sign and luminary supports are bolted onto brackets extending from the cantilever wings and in an area that required special protection from hazardous highway cargos; a special barrier was designed and crash tested so that maximum legal loaded vehicles would be safely redirected into the traffic stream without overturning or penetrating the barrier parapet.

Figure 7. San Antonio "Y". Project IA construction. Typical segments are loaded onto twin triangular trusses in the span-by-span method of construction.

Figure 8. San Antonio "Y". Erection in the span-by-span method from above. Segments are delivered over the completed bridge. A deck mounted crane places segments on the erection trusses.

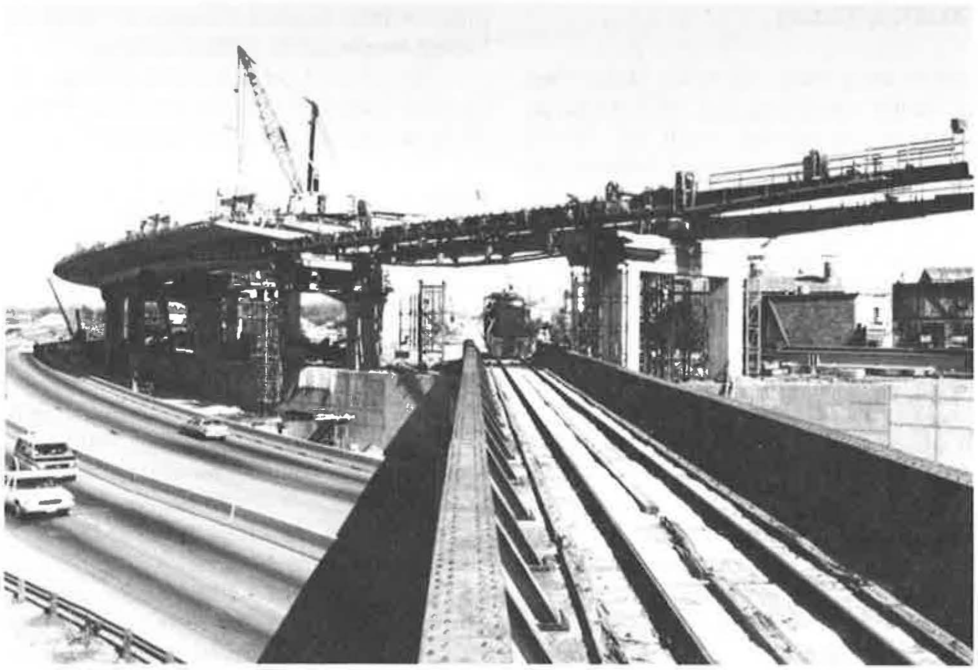

Figure 9. San Antonio "Y". Project IIIA&B construction over active rail lines. Construction from above eliminated traffic disruptions.

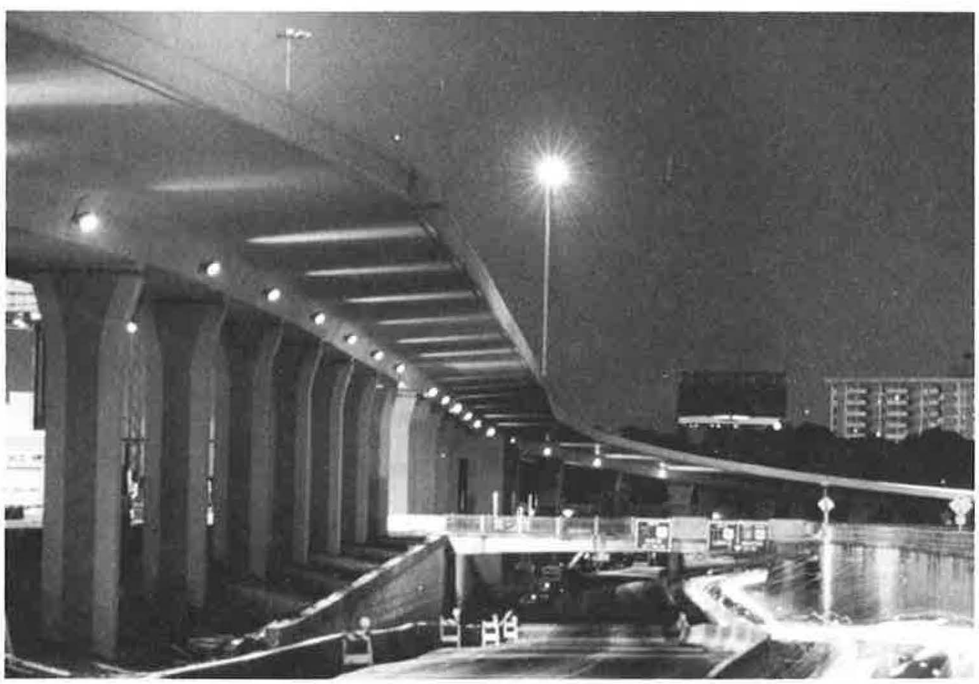

Figure 10. San Antonio "Y". Completed Project IIIA&B at night. Lighting for at-grade facility was provided by fixtures embedded in the elevated structures.

Project 3: US 183 - AUSTIN, TEXAS

U.S. Highway 183 serves as a main artery for cross-town traffic in Austin, Texas. To the northwest, U.S. 183 serves as the commuting route to industries such as Texas Instruments, IBM, and 3M. Between the Loop 1 interchange and IH-35, this expressway dually serves to carry local business traffic as well as through traffic to IH-35 and U.S. 290. Heavy congestion during peak operating hours and future traffic projections indicate the need to improve U.S. 183 from the existing six-lane expressway to a six-lane freeway with three-lane one-way frontage roads.

To accomplish these improvements U.S. 183 from RM 620 to east of U.S. 290 was divided into six design sections for the preparation of plans for roadway and bridge construction. Right-of-way limitations in the stretch of U.S. 183 between Peyton Gin Road and IH-35, designated Contract 5, require elevated structures to carry through traffic to the east of IH-35. Existing bridges at Lamar Boulevard divide the elevated structures of Contract 5. The resulting structures are designated the Peyton Gin Road to Lamar Boulevard and Lamar Boulevard to IH-35 elevated bridges. (Figures 11 and 12)

Peyton Gin Road to Lamar Boulevard

Beginning approximately 1,500' north of Peyton Gin Road at Station 332+20, the Peyton Gin Road to Lamar Boulevard elevated bridges will carry through traffic of U.S. 183 a distance of 1.44 miles.

The typical span length of these 76 span structures is 100', with variable spans between 80' and 110'. A 56' roadway width consisting of three 12' traffic lanes and two 10' shoulders are provided in each direction. Pier heights vary on the project between 10' and 45'.

Two ramp structures are required to move traffic on and off the elevated portion of U.S. 183 between Peyton Gin Road and Lamar Boulevard. Ramp roadways are 26' wide. This roadway is comprised of a single 14' roadway and shoulders of 6'.

The deck surface area for the Peyton Gin Road to Lamar Boulevard structure is 1,017,414 square feet. The northbound structure with off ramp at Peyton Gin Road and on ramp at Lamar Boulevard total 499,458 square feet and the southbound including the on ramp at Peyton Gin Road and off ramp at Lamar Boulevard structure 517,956 square feet.

Lamar Boulevard to IH-35 The northern abutments of the 4,220' (0.8 mile) Lamar Boulevard to IH-35 elevated bridges lie approximately 1,500' southeast of the Lamar Boulevard overpasses.

Two 26' roadway directional ramps are provided connecting traffic between U.S 183 and IH-35. The tallest piers of the project are on the on-ramp structure at IH-35. The piers of this ramp vary from a minimum of 25' to a maximum height of 70'.

The northbound elevated bridge and directional on-ramp at IH-35 have a deck area of 316,597 square feet. The corresponding square footage for the southbound bridge including directional off ramp is 294,663 square feet. This gives a total square footage 611,260 square feet for the Lamar Boulevard to IH35 structures.

The project total square footage for the Peyton Gin Road to Lamar Boulevard structure, the Lamar Boulevard to IH-35 structure, and all ramps is 1,628,674 square feet.

Precast Segmental Box Girder Superstructure

The mainline typical box girder superstructure segments are 58' wide along the top slab, 7' deep, and 10' in length. The top slab consists of side cantilevered slabs 14'6" long, varying in depth between 1'-3" and 10", and an interior top slab varying in depth between 1'-2" and 8" with a clear span of 25'. The bottom slab is 16' wide and is a constant 8" thick. Webs are inclined 90 vertical to 100 horizontal (90:100) and are 10" thick. Transverse flexural stresses in the webs are controlled by a 1'-4" rib placed at the centerline of each segment. This rib varies from 9" to 1'-8" in height. Typical segment weight is 60 tons.

The top slab of the box girder will be prestressed with a combination of pretensioning and post-tensioning. Pretensioned strands (0.5" will be placed straight at 6" centers across the entire width of the top slab and cantilevers. A post-tensioning tendon will be placed in each cantilever slab to provide additional prestressing. These tendons will turn down along the inside face of the stiffening rib of the web providing precompression against transverse forces.

Longitudinal post-tensioning will be applied by external tendons anchored in the pier segments. The deflected tendons will be held by deviation blocks cast at the juncture of the inclined web and the bottom slab. Continuity of the spans as designated will be produced by the overlapping of the tendons in the pier segments. Tendons will generally consist of 12 to 19-0.6"), 270 K ultimate strength low relax strands, encased in polyethylene pipe and subsequently grouted after stressing.

The typical ramp box girder segments are 28' wide along the top slab, 7' deep, and 10' in length. The bottom slab width is 8'. The web thickness (10") and slope (90:100) to match the mainline cross-section. Webs of the ramp girder are also stiffened with transverse ribs. Typical segment weight is 33 tons.

Pier and Foundation Elements

Typical mainline piers are 7'-0" wide at their base transverse to the bridge axis and flair at their top to 16' to accept bridge bearings. Typical mainline piers are 4' wide along the bridge axis, while expansion joint piers are 6' wide. Typical ramps piers are 4'-6" x 4'-0" at their base and flair to 8" wide at pier top. Expansion joint piers are 6' wide along the bridge axis. All piers are poured-in-place reinforced concrete.

Typical mainline and ramp dimensions were chosen in the conceptual design to provide an optimum pairing of form and function. The piers provide maximum clearance to frontage road traffic, while providing column shaft and pier cap dimensions required to carry the segmental box girder superstructure. Presented in the conceptual design was the impact of pier height on bridge aesthetics. This study indicated use of piers with an average height of 25' .

Figure 11. U.S. 183, Austin, Texas. Aerial view of interchange of 183 and IH-35. Direction ramps at 4th level are built in cantilever. Main lanes at 2nd or 3rd level are built in span-by-span.

Figure 12. U.S. 183, Austin, Texas. Twin parallel structures 58' wide each are used to produce this 1.25 million square foot bridge.

Studies in the Preliminary Design focused on two additional aspects of pier aesthetics. These were pier color and texture. Early in the Preliminary Phase it was confirmed that the natural color of the cast concrete would be used. To assist in determining whether or not texturing of exposed pier surfaces was desired, architectural renderings were prepared for various options.

Three of these options are presented in this section for both mainline and ramp typical piers. The first rendering is that in which no texturing is used on any exposed pier face. This is followed by a vertical texturing on the central portion of the transverse pier faces. Finally, texturing on both the transverse and longitudinal faces is presented. Textured surfaces would be provided by form liners attached to the pier forms.

Drilled shafts will transmit bridge reactions to underlying supporting strata. Based on existing soil information, the typical mainline piers rest on two 42" shafts Typical ramp piers are supported by two 36" shafts.

The bridge deck of the U.S. 183 bridges will transmit its reactions to the piers through elestomeric bearings. The horizontal flexibility of the bearings is varied by adjusting the bearing thickness. This control over substructure flexibility permits a more uniform transmission of load to the piers of a continuous unit. Typical span lengths are 100' and a typical unit consists of 6 typical 100' spans.

Construction Methods

The construction method proposed for the elevated U.S. 183 bridges is the span-by-span method utilizing twin triangular assembly trusses.

A typical assembly cycle begins with the assembly trusses resting on the temporary pier brackets of the span to be constructed. Superstructure segments, precast in a casting yard, are delivered over ground or along the completed portion of elevated bridge.

Either a ground based or deck mounted crane lifts the precast segments and places them on rolling supports on the assembly trusses. A hand winch brings the segments along the top cord of the twin triangular trusses to their final location. When all segments are in place, temporary blocking is placed across the closure joints, and a nominal prestress force is applied to insure tight fit of all the precast segments. Closure joint concrete is poured, longitudinal duct work secured, and the post-tensioning tendons threaded and stressed. The construction cycle is completed when the assembly trusses are advanced to the next span.

The assembly trusses for the U.S. 183 bridges may be self-launching to permit truss advancement when ground access is not available. Self launching of the assembly trusses takes place in two phases. At first, the front of the erection truss will ride on the temporary pier bracket of the pier at the open end of construction. The rear of the assembly truss rides along the completed superstructure with the aid of a C-hook and wheel. When the launching nose attached to the front of the assembly trusses lands at the pier bracket of the next pier, it supports the front of the truss eliminating the need for the Chook. The rear of the truss now rides along the temporary pier bracket at the open end of construction.

The span-by-span method of erection will accommodate all variations in span length on the U.S. 183 bridges except for the 180' span of the directional interchange ramp at IH-35. A recently developed technique of balanced cantilever construction incorporating the twin triangular trusses to provide cantilever stability will be used for this span.
The construction sequence of the five span continuous unit of the directional ramp is as follows:

1. The first span of 120' is built in conventional span-by-span fashion.

2. The assembly trusses are advanced across the 135' span adjacent to the main span. When secured, it provides the stability against cantilever overturning moments developed during balanced cantilever construction. Precast segments of the side span can be placed all at one time leaving gaps between the segments or placed one by one and attached directly to the cantilever.

The sequence of placing segments in cantilever is to always attach the side span segment prior to the placing of its balancing segment in the main span. In this way, the cantilever overturning is resisted directly by the temporary trusses.

3. After the cantilever is complete, the assembly trusses are taken down by crane and transported to the East side of IH-35 and assembled in the 135' span following the main span.

Balanced cantilever construction resumes, as described above, always assembling the segments of the side span to the cantilever prior to placing main span segments.

4. Once the balanced cantilever construction is completed for the second cantilever, the main span closure pour is made, and continuity post-tensioning of the main span is stressed completing main span construction. Following this, the remaining segments of the 135' span are assembled and continuity prestress is applied.

5. To complete the five span unit, the span-by-span trusses are advanced to the 120' span and typical span-by-span construction takes place.

CONCLUSIONS

• Precast segmental concrete box girder construction technology has been used successfully to minimize on-site construction time, speed erection and eliminate interference with traffic.
• The second-level structure utilizes a minimum of ground space, which permits a doubling of capacity with minimum acquisition of rights of way in congested urban areas.
• Transitions from single boxes to twin boxes for ramp structures are totally precast.
• Complex geometry was accomplished by using the short-line casting technique.
• Span-by-span erection method previously used for long bridges over water is extremely efficient in constructing urban structures with complex geometry.
• Span-by-span erection takes place with segment delivery over the completed portion of bridge and segment placement by deck mounted crane.
• Balanced-cantilever construction takes place over Interstates and U.S. highways with minimal disruption.

Segmental Concrete Arches on the Natchez Trace Parkway

DONALD W. MILLER AND JOHN A. CORVEN

Today's design and construction of major highway structures is increasingly becoming a "high tech" field for the structural engineer. These bridges are considered to be "high tech" because of the complex mathematical process that is required for their design and construction/erection. Designs such as these are only accomplished through the use of very complex computer-based design programs. The need for engineers to be capable of designing and constructing such structures is due in part by the need of the local community and the nation to get the most from the available highway dollar. Today's project requirements relative to environmental concerns such as wetlands and water quality also drive the demand for the design of longer span bridges with sophisticated erection schemes. Consequently, the engineering community is using more and more segmental concrete design and construction methods as a means to addressing these concerns. A segmental concrete arch bridge to carry the Natchez Trace Parkway over Tennessee Route 96 near Franklin, Tennessee, is such a structure.

BACKGROUND

The Natchez Trace Parkway is a two-lane roadway owned and maintained by the National Park Service which takes park visitors from Nashville, Tennessee, to Natchez, Mississippi, along an alinement that follows very closely the original Natchez Trace. The trace was first probably a series of hunters' paths that slowly came to form a trail from the Mississippi over the low hills into the valley of the Tennessee. As early as 1733, the French were familiar enough with the land to make a map that showed an Indian trail running from Natchez to the northeast. By 1785, American settlers in the Ohio River Valley had established farms and, in search for markets, had begun floating their crops and products down the rivers to Natchez or New Orleans. Returning home meant either riding or walking, for the flatboats, too, were sold for their lumber, and the trail from Natchez was the most direct route home. As the numbers of boatmen grew, the crude trail was tramped into a clearly marked path. Over the years, improvements were made, and by 1810 the trace was an important wilderness road, the most heavily travelled in the Old Southwest. Even as the road was being improved, other comforts, relatively speaking, were coming to the trace. During these years, many inns--locally called stands--were built. By 1920 more than 20 stands were in operation. Most of them provided no more than a roof over one's head and plain food. But even with these developments, the trace was not free of discomforts. Gangs of thieves added an element of danger that was only one more hazard in a catalog that included swamps, floods, disease-carrying insects, and unfriendly Indians.

Since the late 1930's, the National Park Service has been constructing a modern parkway that closely follows the course of the original trace and gives the present-day traveller an unhurried route from Natchez to Nashville. The construction of a structure which will carry the Natchez Trace Parkway over Tennessee Route 96 is one more link toward the final completion of the Parkway. The crossing is such that the parkway grade is approximately 155 feet in elevation above Tennessee Route 96. The north side of the crossing contains a tree-covered bluff that will be undisturbed during the construction of the bridge. In keeping with the history of the trace and desire by the National Park Service to have a structure at this crossing that will blend in with the surrounding terrain, an arch-type structure was selected.

STRUCTURE DESCRIPTION

The bridge is 1,648 feet in length with a main arch span of 582 feet, as shown in Figure 1. The superstructure, which is a post-tensioned concrete box section, rests on the arch section and is not a part of the arch construction as is done in most arch bridges. This is done to prevent the thermal effects of the arch from producing secondary forces in the main superstructure elements.

The superstructure is composed of span lengths from abutment 1 to abutment 2 (north to south) of 204' - 246' - 90' - 246' - 246' - 90' - 246' - 204'. The remaining bridge length is composed of 38-foot concrete tee-beam spans as part of the cast-in-place decked-over abutments. The two 90-foot spans are located on the top of each arch span. The 246-foot end spans then close the gap from the pier to the crown of the arch. The bridge is designed to be built as either a cast-in-place or precast, segmental concrete structure. Both designs use the same geometry and structural element sizes. That is, each construction type will produce identical finished structures.

CAST-IN-PLACE SEGMENTAL DESIGN

Foundation

All substructure units are to be founded in a rock layer locally known as Brassfield limestone. Preliminary field borings indicated that the limestone was located about 30 feet below the existing ground surface. At the base of the slope for abutment 1, the location of pier 1 and thrust block 1, preliminary foundation data indicated the possibility of cavities in the limestone formation. Since the footing for pier 1 is a thrust block, both vertical and horizontal forces have to be accounted for in the foundation design. Excavating for the foundation of pier 1 during construction and encountering a

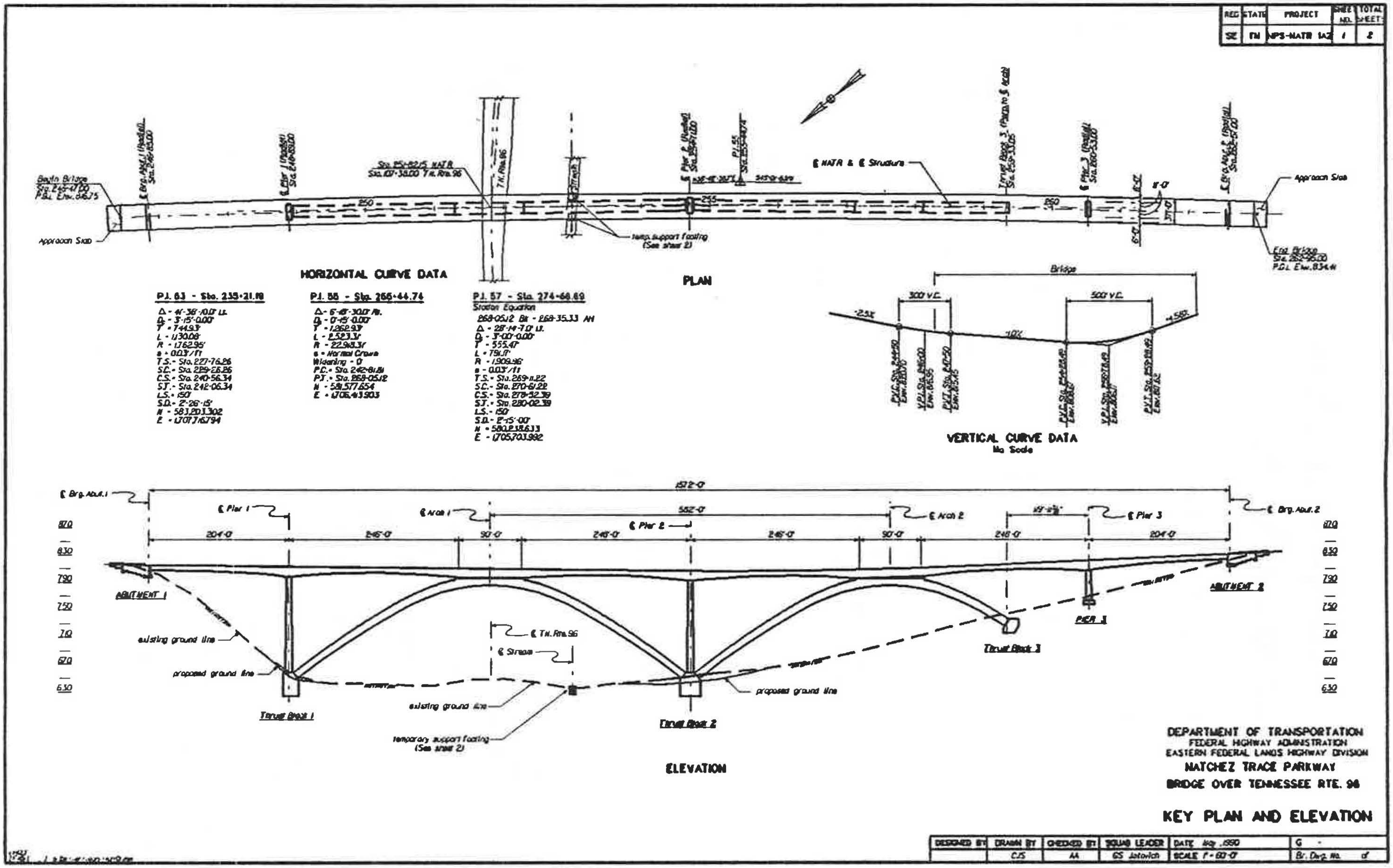

FIGURE 1 Bridge plan and elevation view

large cavity was not considered a desirable way to start work on a major bridge structure.

In order to thoroughly investigate the foundation for pier 1, two choices were considered. One choice was to drill the area with boreholes on about a 5-foot grid. Due to the size of the foundation, this was considered to be very expensive and time consuming. It also did not preclude the possibility of not locating a cavity. This was an unacceptable deficiency with this type of investigation.

The second choice and ultimately the method of investigation used was Crosshole Seismic Scanning and Tomography. This is a method of foundation investigation where boreholes are placed around the periphery of the footing, a thrust block in this case. Sensors are then placed in all but one borehole. The remaining borehole contains a seismic wave-generating device. By incrementally positioning these units in the boreholes, the entire foundation area is mapped by measuring the seismic velocity and wave attenuation between the seismic generator and the receiver. While the process of crosshole seismic scanning is not new, the ability to use this data along with a computer program to develop a tomographic image of the foundation material is gaining widespread use. Figure 2 is a sample of the tomographic image that is developed using this process. The dark area on the image is the cavity. By using multiple two-dimensional images developed between the seismic generator and the receivers, a three-dimensional map of the area can be developed showing the limits of any cavities.

Piers

The piers are designed as single-cell concrete box segments to be built using the cast-in-place slip forming method. The inside of the pier contains a ladder and landing system for future inspection. The pier heights range from a maximum of about 140 feet to a minimum of 43 feet. The piers are tapered in both directions with the taper for all piers being identical. The taper is set by pier 1, which is the tallest. Pier caps are 7 feet by 15 feet 6 inches. The base dimension for pier 1 is 10 feet by 22 feet.

Arches

The structure contains two arch supports. The main arch span is a symmetrical span of 582 feet and crosses Tennessee Route 96. The arch segment is to be constructed as a single-cell concrete box section. The inside of each cell contains a metal staircase ladder for future inspection. The second arch contains the same geometry as the main arch span but is only about 80 percent of the main arch. This is because the terrain beneath this span is sloping upward from pier 2 to pier 3. Consequently, the support for the second half of this arch does not coincide with the base of pier 3. This support base is located in the slope about 120 feet from pier 3 toward pier 2.

The arch geometry is a series of compound circular arcs along the intrados of the arch. This type of geometry was chosen so that minimal flexural stresses would be present in the legs of the arch. This is desirable in the design of each arch since the arch contains no spandrels. Each arch is also designed on an offset tangent horizontal alinement since the superstructure is on a 0° - 15' horizontal curve. This requires that the arch base and the pier base be offset by about 1 foot 4 inches.

The cast-in-place design requires that the arch be constructed in about 90-foot long segments using temporary tower support piers and a reusable support truss, as shown in Figures 3 and 4. As each section is built, the support truss is unloaded and moved to the next segment to be constructed. The completed arch segment then supports itself between each support or about every 90 feet. The arch is constructed simultaneously from both ends with the top or crown being cast last. All but the very crown and base of the arch is constructed using mild reinforcement. The crown and base of the arch contain post-tensioning reinforcement in addition to mild reinforcement due to the flexural stresses in the arch caused by the reactions of the 246-foot adjacent superstructure spans on the crown of the arch.

Superstructure

The superstructure for this structure is a single-cell concrete box with 8-foot-6-inch cantilevers supporting a concrete and aluminum bridge rail. The roadway carries two 11-foot traffic lanes with 6-foot shoulders. The railing width is 1 foot 6 inches, which gives an overall superstructure width of 37 feet out-to-out. The superstructure varies in depth from 14 feet at the piers to 7 feet 6 inches at the abutments and centerlines of each span. The section is a constant 12-foot 11 1/4-inch depth for the 90-foot span at the crown of the arch. The sides of the box are on about a 2 to 12 slope which gives the bottom of the box a varying width of about 17 feet 10 inches to 15 feet 4 inches. The superstructure contains both longitudinal post-tensioning in the webs and transverse post-tensioning in the deck.

The superstructure is designed to use the same temporary support tower piers used for the arch construction with a reusable truss support system, as shown in Figures 5 through 11. The 90-foot segment at the crown of the arch is cast using the arch as support. The abutment 2 end span is built using typical ground-based falsework as support. The end span at abutment 1 is built using an inverted Queenpost truss. This is done to keep from disturbing the fragile slope between abutment 1 and pier 1.

Abutments

After the main portions of the bridge are constructed, the 38-foot long decked-over abutments will be completed. The entire structure will finally receive a 1 1/4-inch thick latex modified concrete overlay as a final wearing surface. All exposed vertical faces of the structure and the concrete portion of the bridge rail will receive a hand-rubbed concrete finish. The abutment wings are to be faced with local stone.

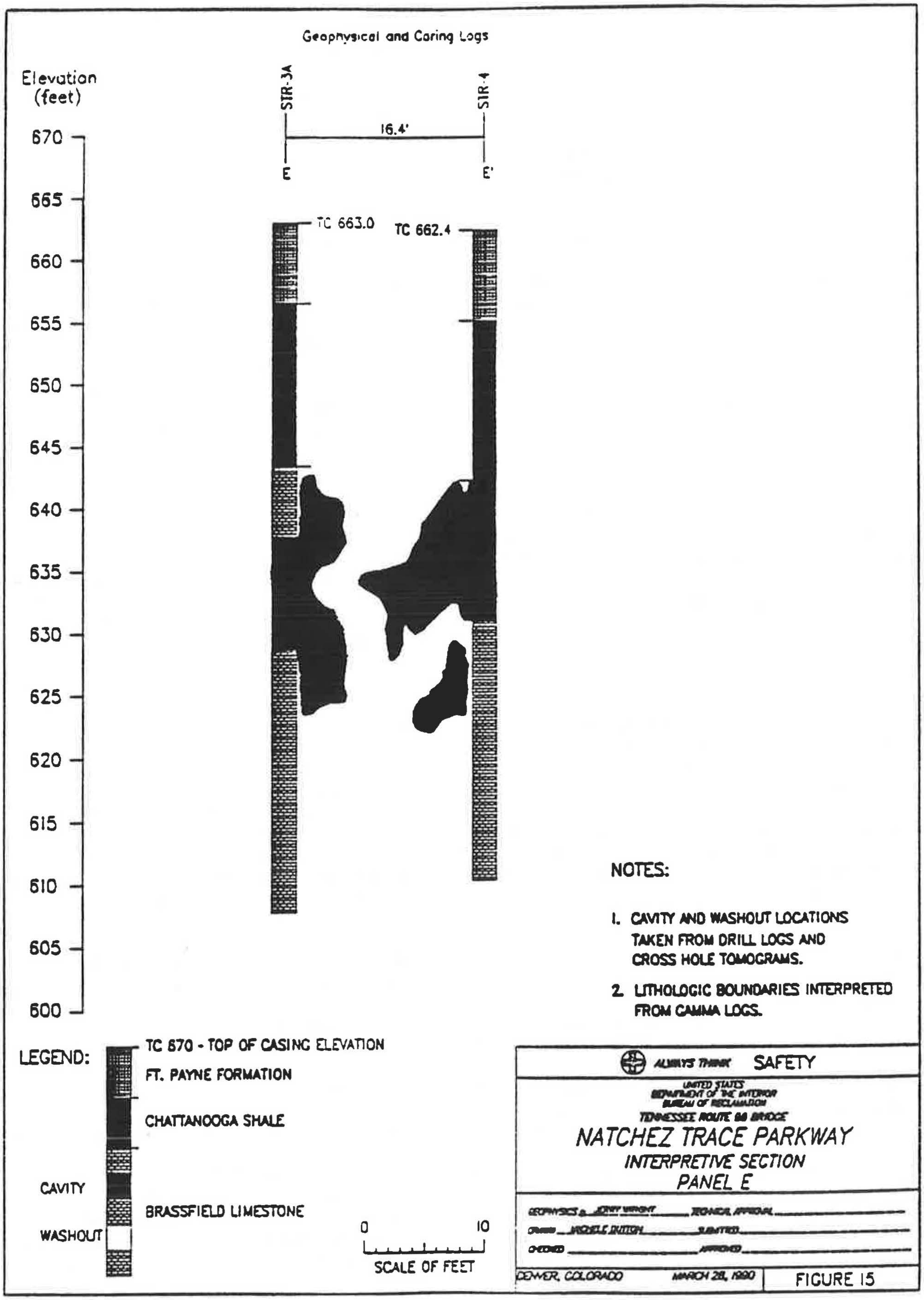

FIGURE 2 Tomographic image of cavities in limestone (dark area denotes cavity)

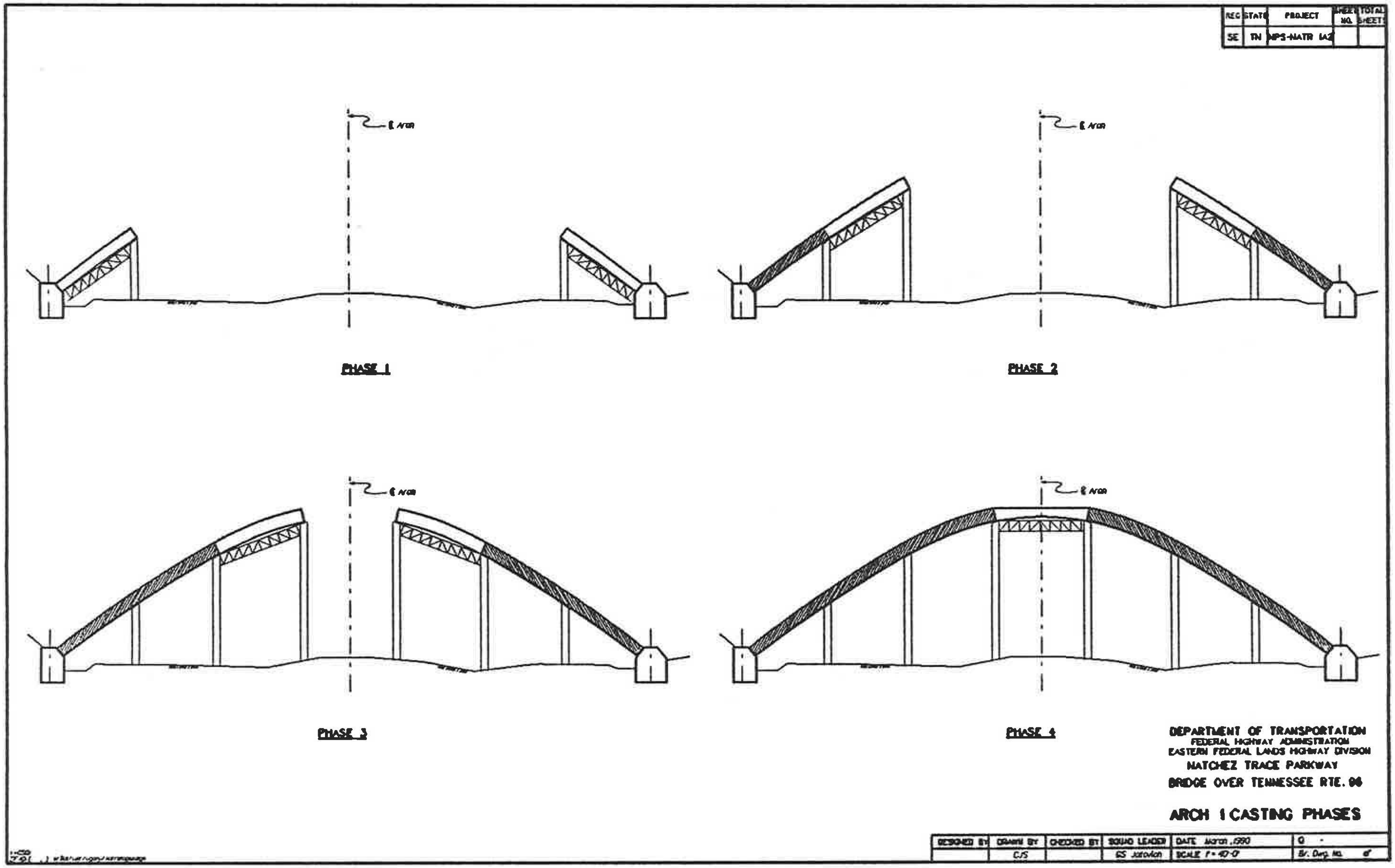

FIGURE 3 Arch 1 erection sequence

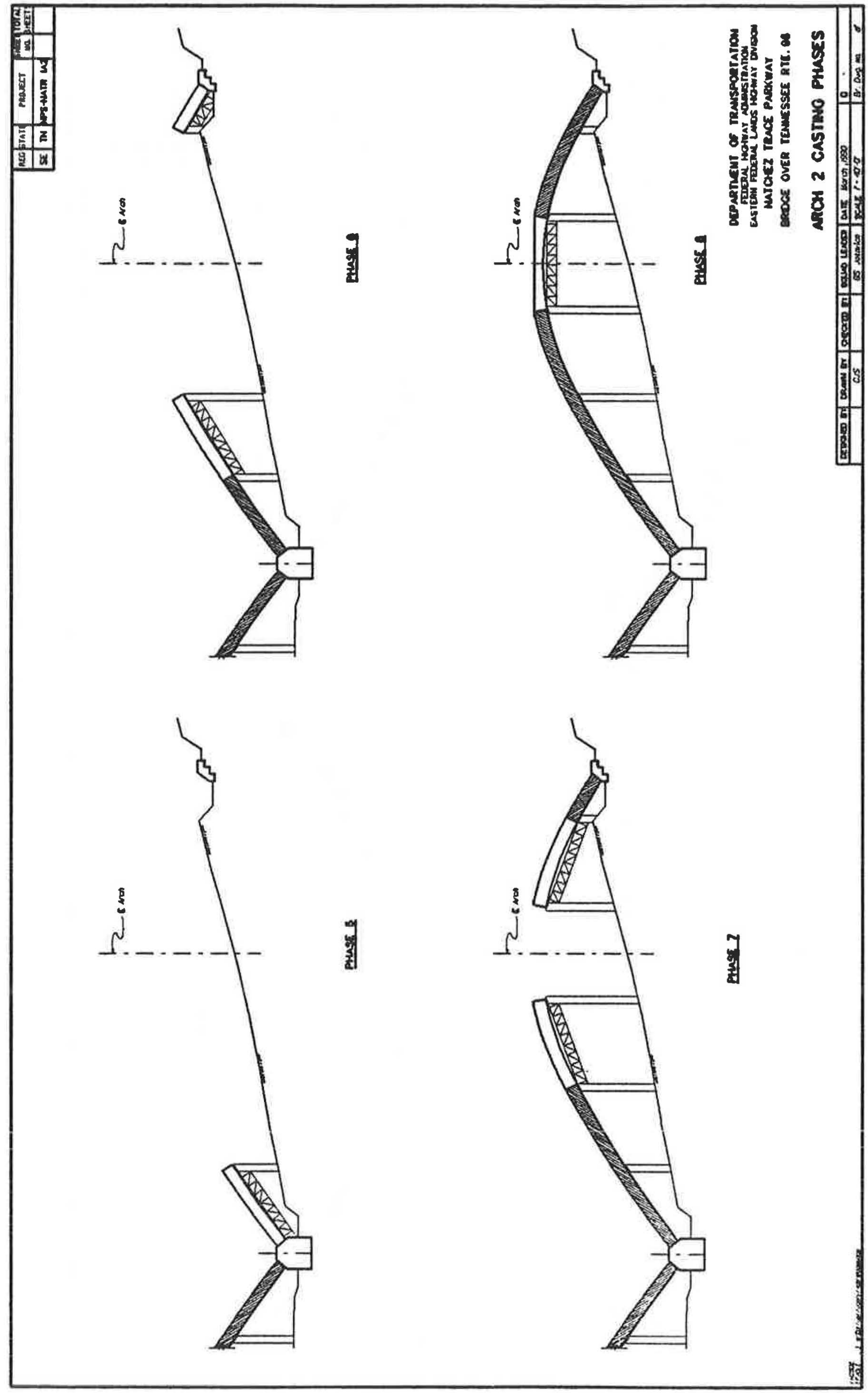

FIGURE 4 Arch 2 erection sequence

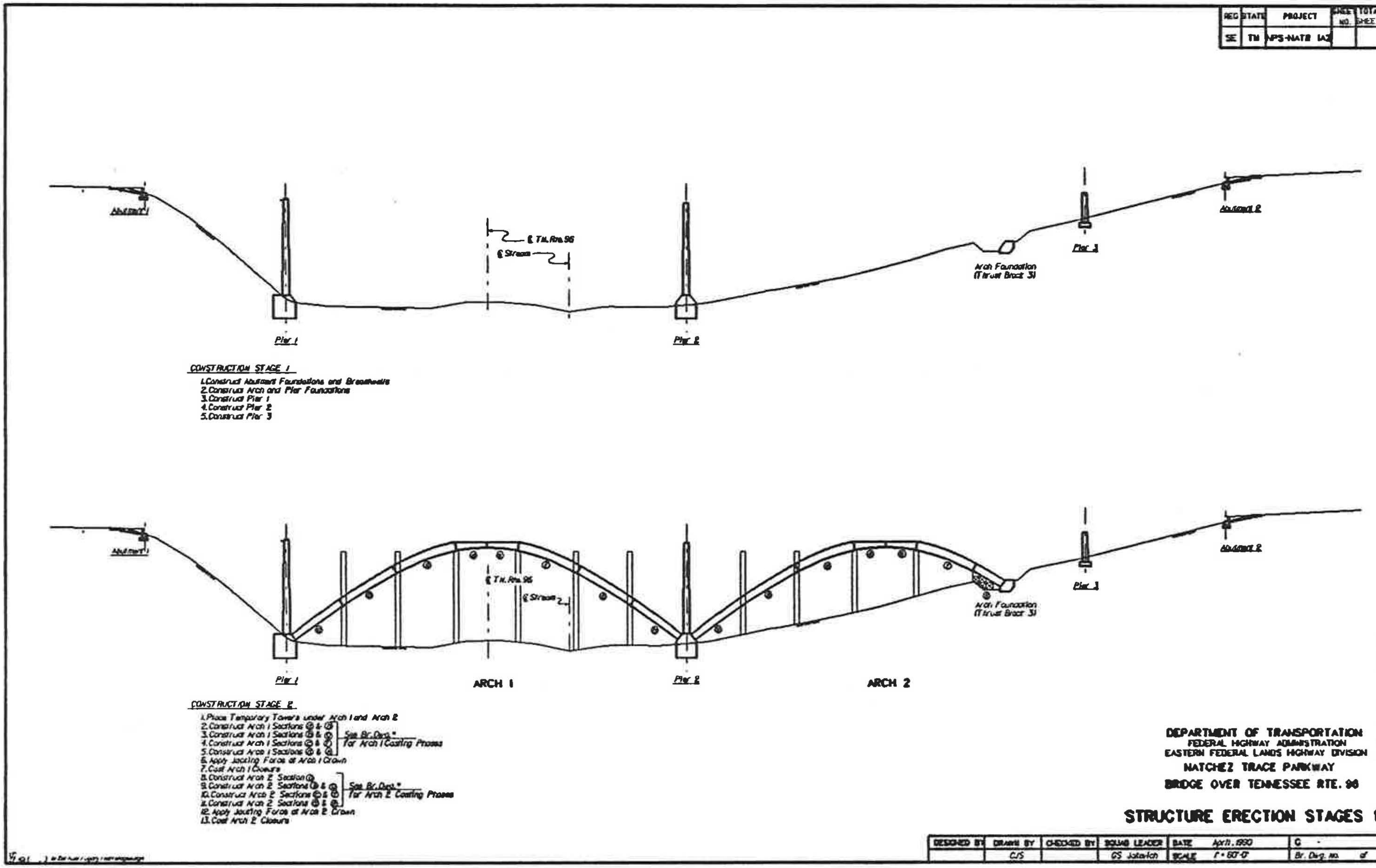

FIGURE 5 Construction stages 1 and 2 showing substructure construction

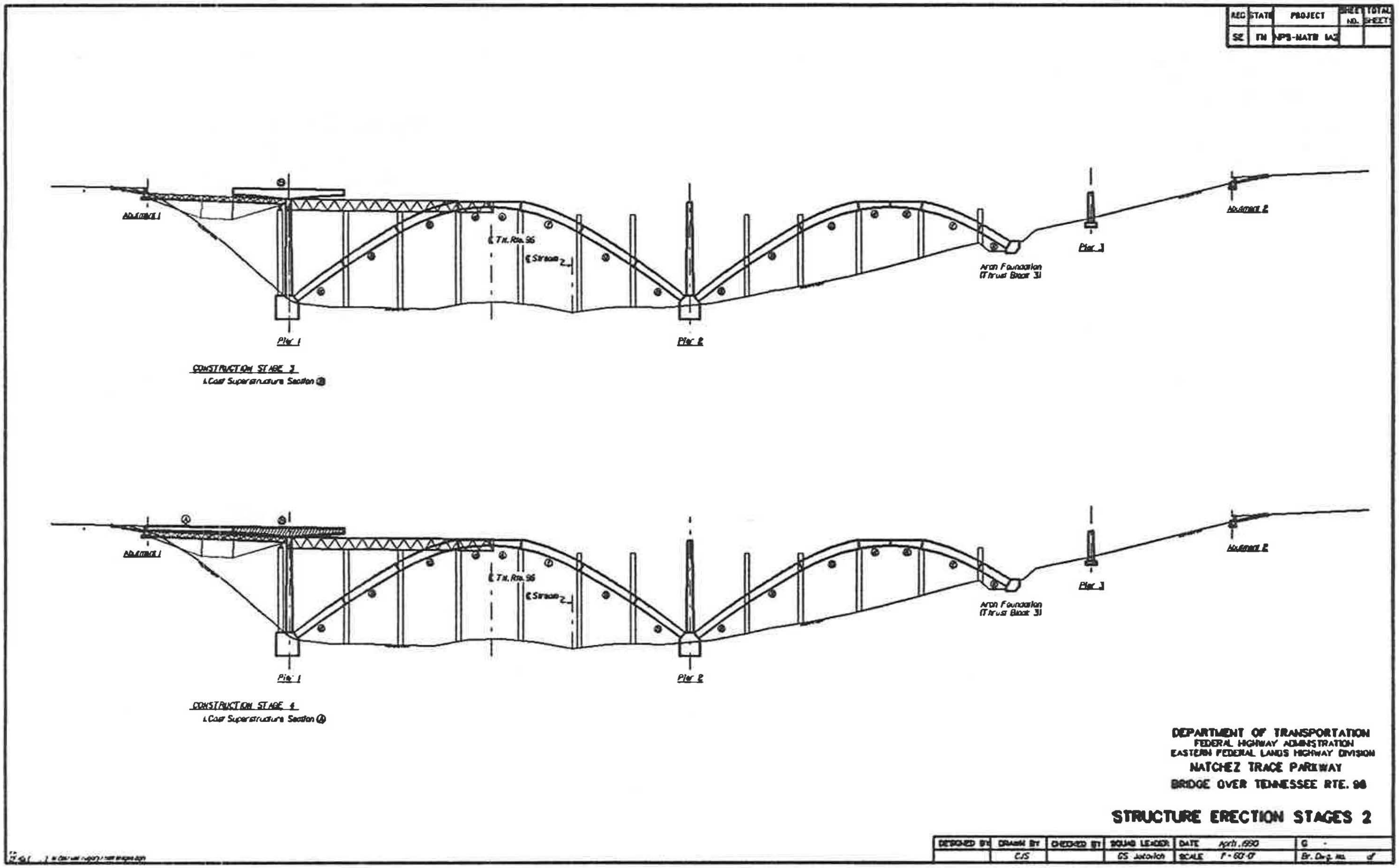

FIGURE 6 Construction stages 3 and 4 showing span 1 and span 2 cantilever placement

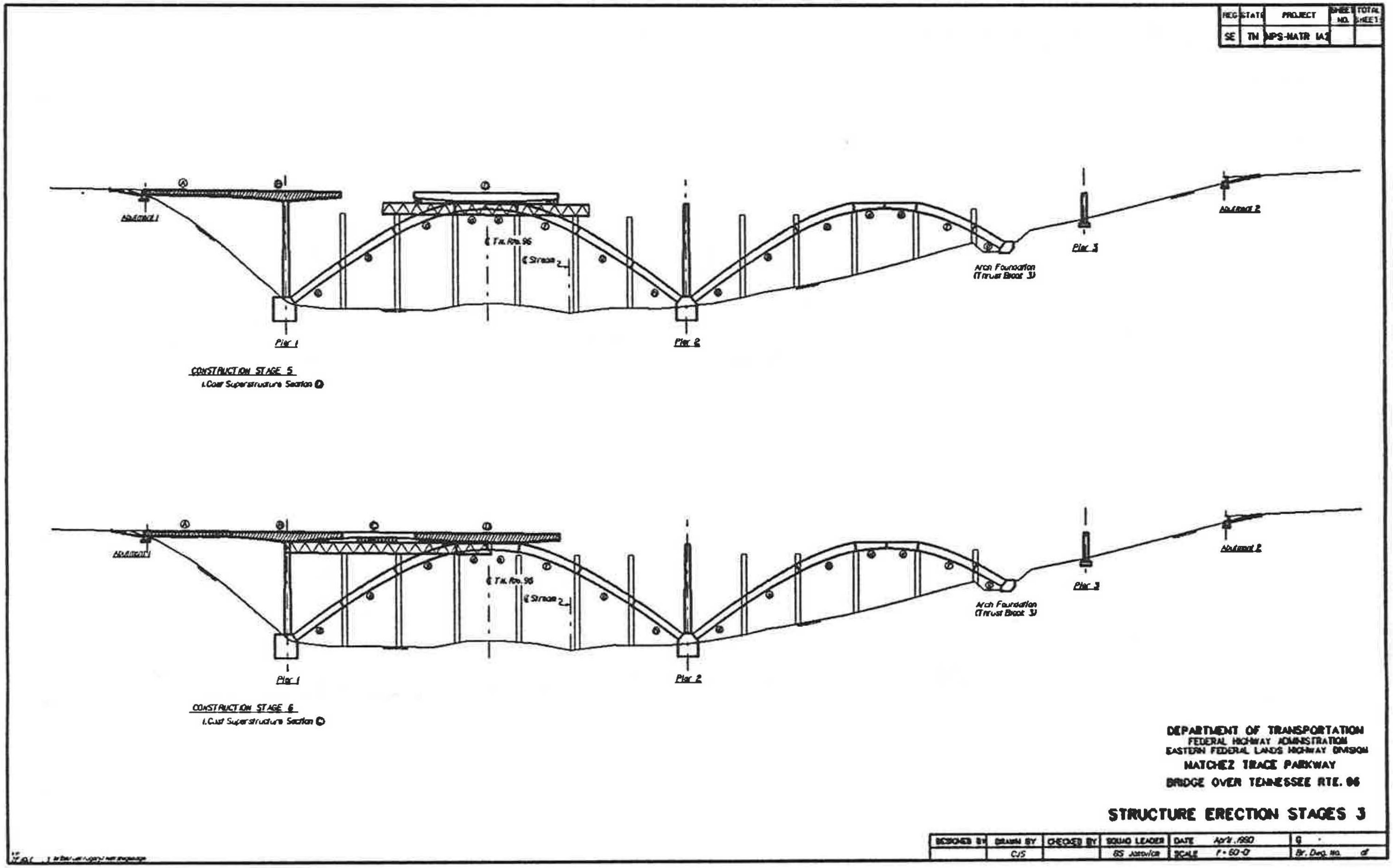

FIGURE 7 Construction stages 5 and 6 showing arch 1 (span 3) cantilever and span 2 closure placement

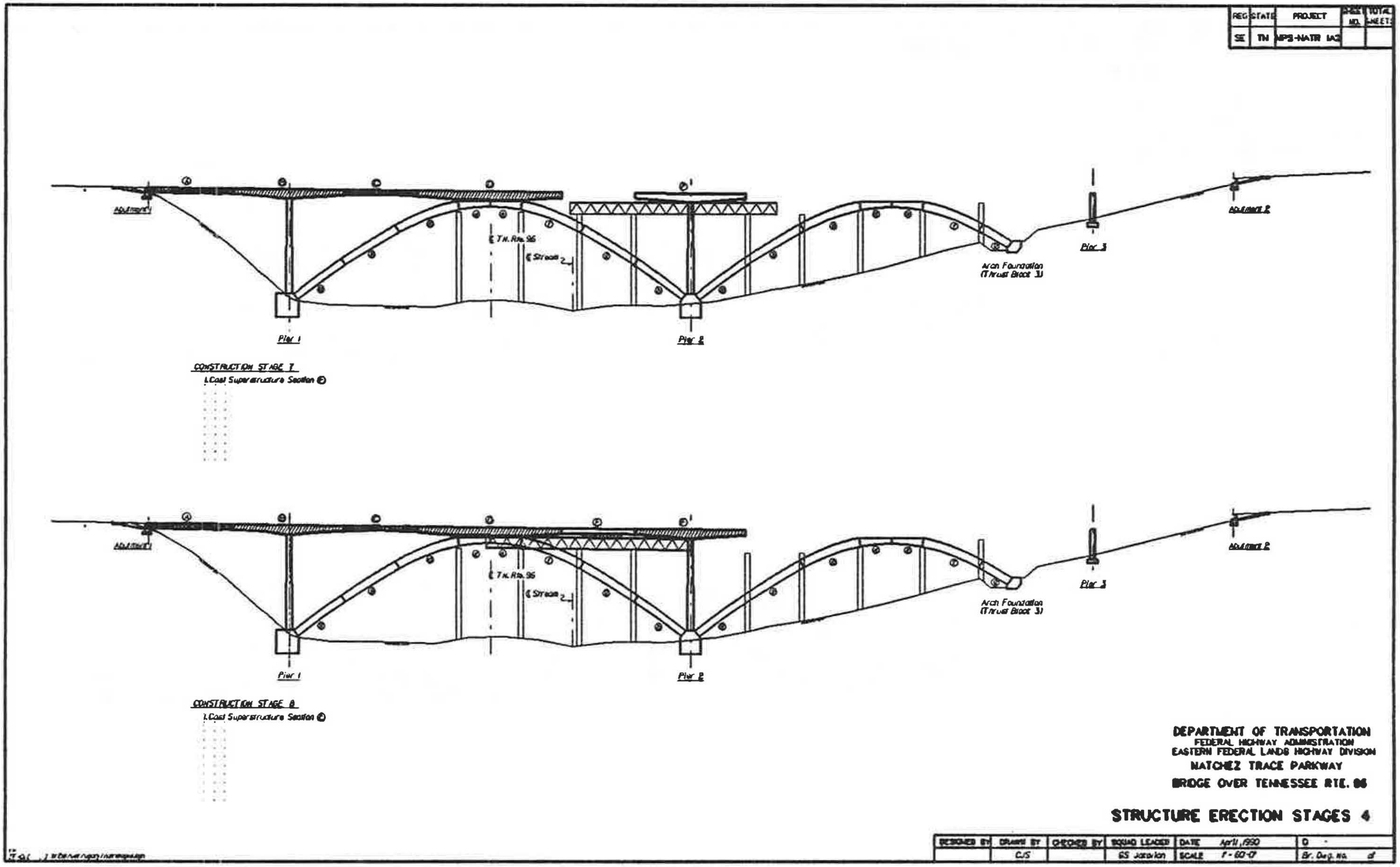

FIGURE 8 Construction stages 7 and 8 showing pier 2 cantilever and span 4 closure placement

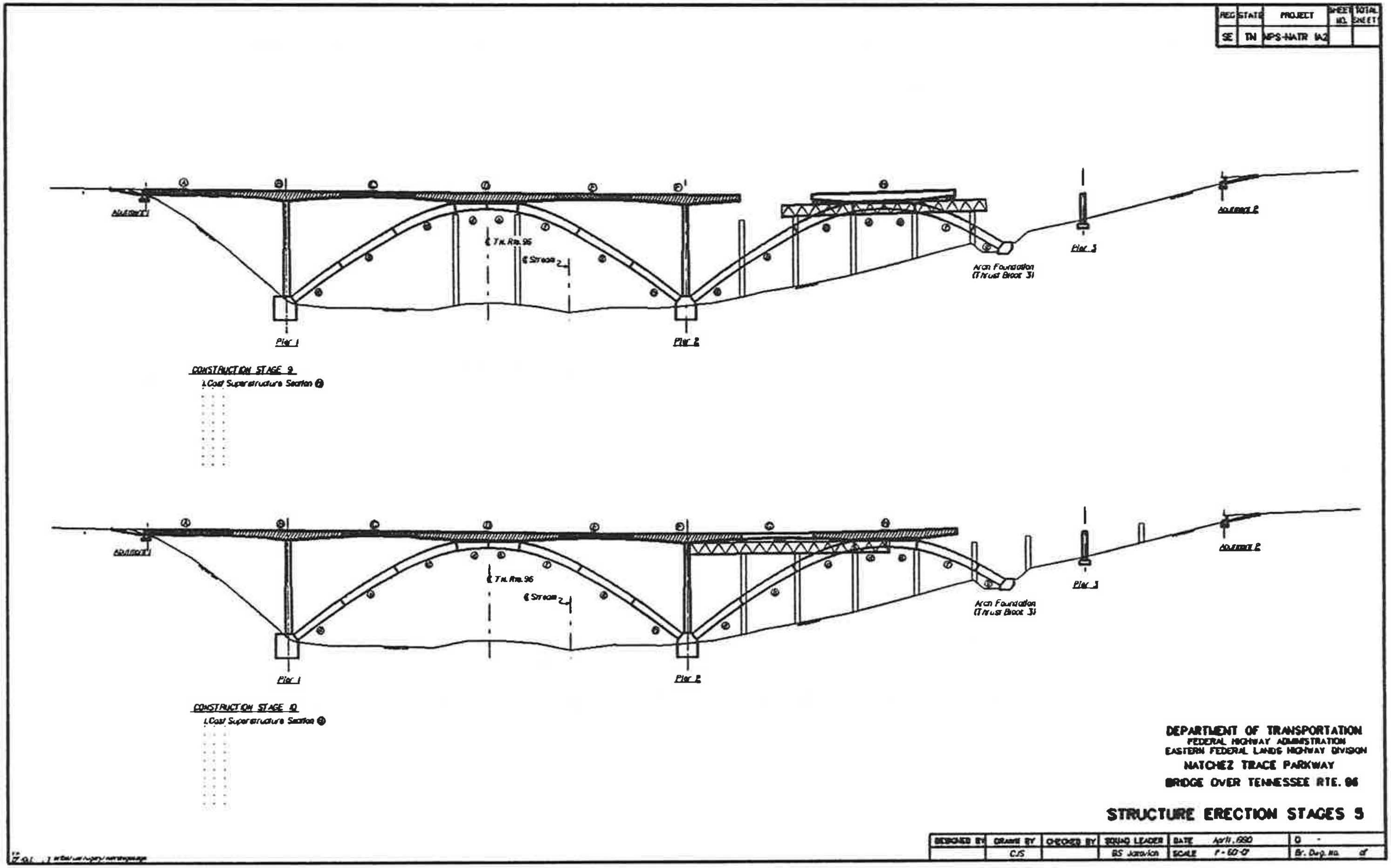

FIGURE 9 Construction stages 9 and 10 showing arch 2 (span 6) cantilever and span 5 closure placement

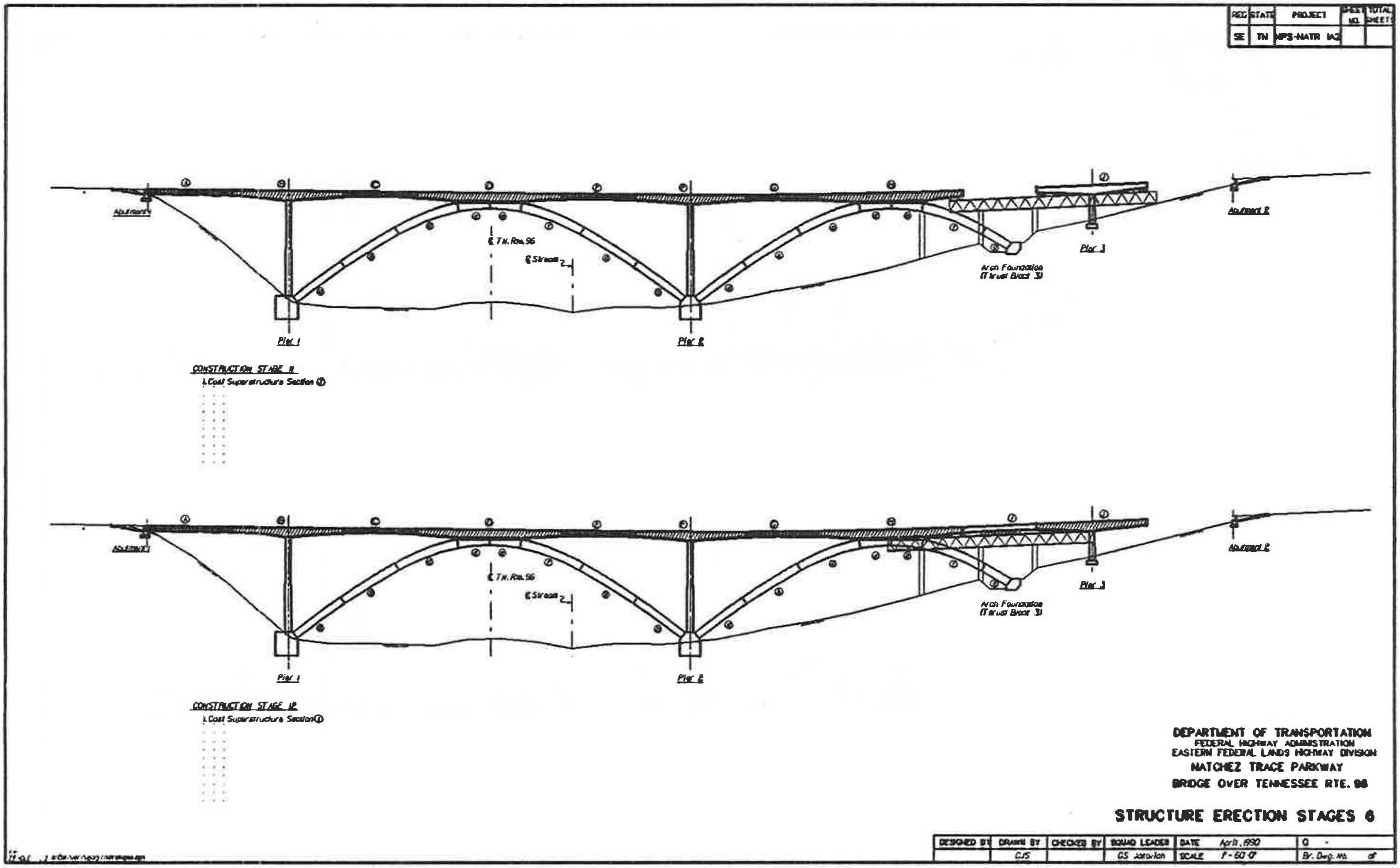

FIGURE 10 Construction stages 11 and 12 showing pier 3 cantilever and span 7 closure placement

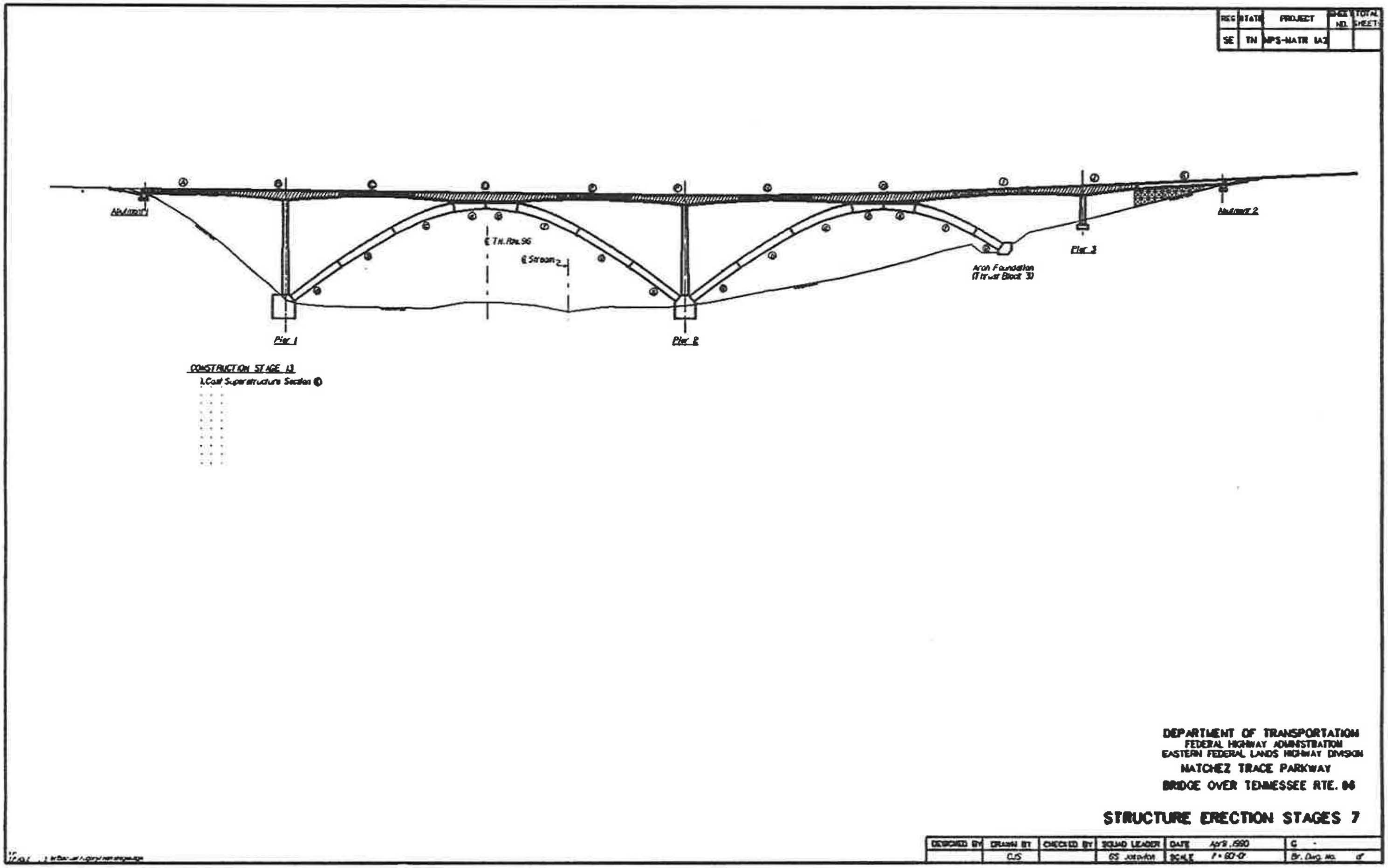

FIGURE 11 Construction stage 13 showing span 8 closure placement

PRECAST SEGMENTAL DESIGN

A second design and construction/erection method proposed for this structure, except for the foundations, uses precast segmental techniques. The piers, arches, and bridge superstructure are made of precast segments erected by either the span-by-span or balanced cantilever methods of construction. This approach represents the first use of precast segmental technology for an arch bridge in the United States. All major features of the precast segmental design are the same as the cast-in-place design.

Foundations

As previously mentioned, only the foundations for this design are constructed with cast-in-place concrete. These elements are abutments 1 and 2, thrust blocks 1, 2, and 3, and pier 3 footing. Each of these bear directly on the underlying rock stratum without the aid of piles or drilled shafts. The material encountered at the site at bearing level is mainly limestone with permissible bearing pressures between 14 and 24 tsf.

Thrust blocks 1, 2, and 3 are cast-in-situ concrete placed after excavation to acceptable bearing material. Ductwork and anchorages for the post-tensioning of the precast arch and pier segments are embedded in the thrust blocks. The quantity of concrete in the thrust blocks are 980, 870, and 490 cubic yards for thrust blocks 1, 2, and 3, respectively.

Piers

The three piers of the project are precast, segmental box piers. The tapers were established by setting the dimensions of the base of the tallest pier (pier 1). These dimensions are 10 feet longitudinally and 22 feet transversely. With the taper set, the variable segment dimensions are determined. The base dimensions of piers 2 and 3 are, therefore, a function of their height. Selecting dimensions in this fashion produces similar precast segments at the same relative elevation from the top of the piers.

The precast box pier segments are typically 10 feet in length. The maximum weight is 45 tons and the minimum weight is 33 tons. The pier caps are solid precast segments 7 feet by 15 feet 6 inches by 3 feet 9 inches and weigh 31 tons.

The precast segments are vertically post-tensioned with a combination of post-tensioning bars and strand tendons. Four 1 3/8-inch diameter high strength post-tensioning bars are typically placed between the precast segments. This number is increased to 10 bars for the last four segments of the taller piers. Two 19 by 0.6-inch diameter strand tendons are placed U-shaped through the pier foundation for the full height of the pier. All joints in the precast piers are joined with epoxy.

Arches

In plan view the roadway alinement follows a 0° - 15' curve over the entire length of the project. To accommodate this curvature, the arches are held straight and placed on chords which pass through the arch superstructure bearings. This results in an angle break between the lines of action of the arch thrusts at pier 2 and offsets between the centerlines of piers and the arch thrust lines at piers 1 and 2. Thrust block 3 is located independently of pier 3 as previously described.

The profile of the arches is a series of compound circular arcs on the intrados. The radii vary from 1,150 feet at the base to 240 feet at the crown. The thickness of the arches vary from 10 feet at the base to 13 feet at the centerline of bearing of the superstructure (45 feet off of centerline of arch). The arches then vary back to a depth of 10 feet at their centerline. These variations to establish the extrados are linear with length along the intrados.

The width of the arches is 16 feet 1 inch. The cross section is a single-cell box girder with 1 foot-thick walls. The maximum segment weight is 45 tons and the minimum weight is 29 tons. The typical length of the arch segments is 10 feet and a total of 68 segments are used to construct arch 1, and 54 segments are used in arch 2.

The erection procedure of the arches innovatively uses the span-by-span method of construction, as shown in Figures 12 through 18. This method was developed over 12 years ago for the erection of the Long Key Bridge in the Florida Keys. Since then numerous precast segmental superstructures have been assembled using this method and many refinements have been made. The span-by-span erection of the arches will demonstrate a new application of this efficient method of construction.

To build the 582-foot span arch (arch 1) along its chord, seven intermediate "spans" between 80 and 90 feet are established. These intermediate "spans" are supported by temporary piers which may be reused for the construction of arch 2. Construction begins at thrust block 1 (pier 1). Temporary pier 1 is erected and assembly trusses are positioned in the first intermediate span. Segment 1 is placed on the assembly trusses and stressed back to thrust block 1. In repeating fashion, each of the remaining first nine precast arch segments are epoxyed and stressed together. Typically, 10 1-3/8-inch diameter high strength bars are used to stress the precast arch segments together. Eight additional bars are used near thrust block 1 to help control flexural stresses. When all of the segments of this span are erected, the assembly trusses are lowered and the segments span between thrust block 1 and the first temporary pier.

Construction continues with the assembly of segments 10 through 19 and segments 20 through 29 in intermediate spans 2 and 3, respectively. Once these three spans are complete, the assembly trusses are moved to span seven adjacent to thrust block 2. Spans 5 through 7 are then assembled in reverse order which, when complete, produce two three-span structures symmetrical about the centerline of the arch.

In the last phase of construction of this arch, the assembly trusses are positioned in the fourth or central intermediate span. Segments 30 through 34 are epoxyed and stressed to the left three-span structure while segments 35 through 39 are assembled to the right three-span structure, leaving a 1-foot closure joint at the arch centerline. Hydraulic jacks are then positioned in the closure joint and the two arch halves are jacked apart with a force of 1,000 kips. The closure joint is then poured and after minimum compressive strength isreached four 19-inch by 0.6-inch diameter post-tensioning tendons per web are stressed completing the arch.

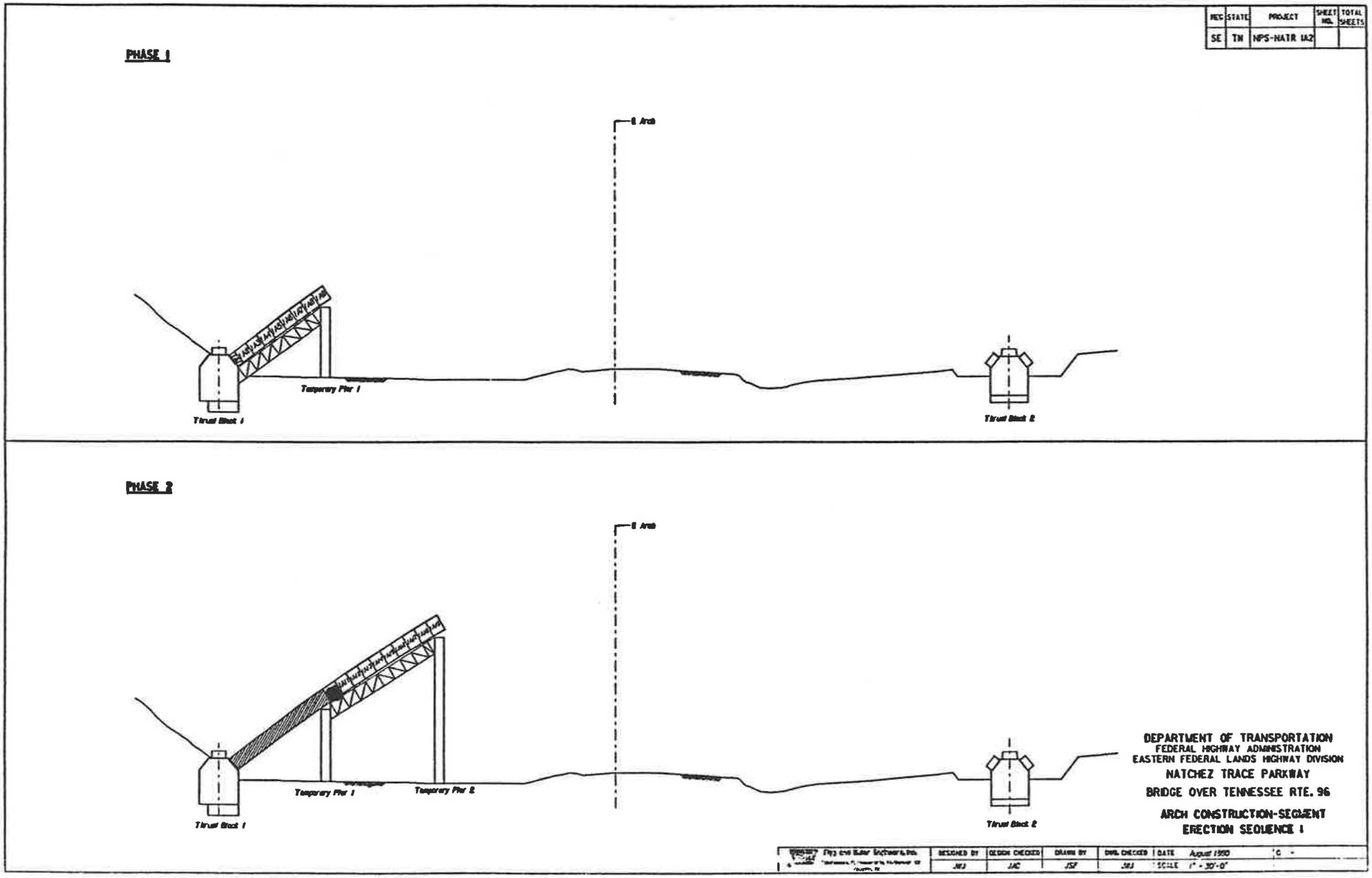

FIGURE 12 Arch construction - precast alternate - phases 1 and 2

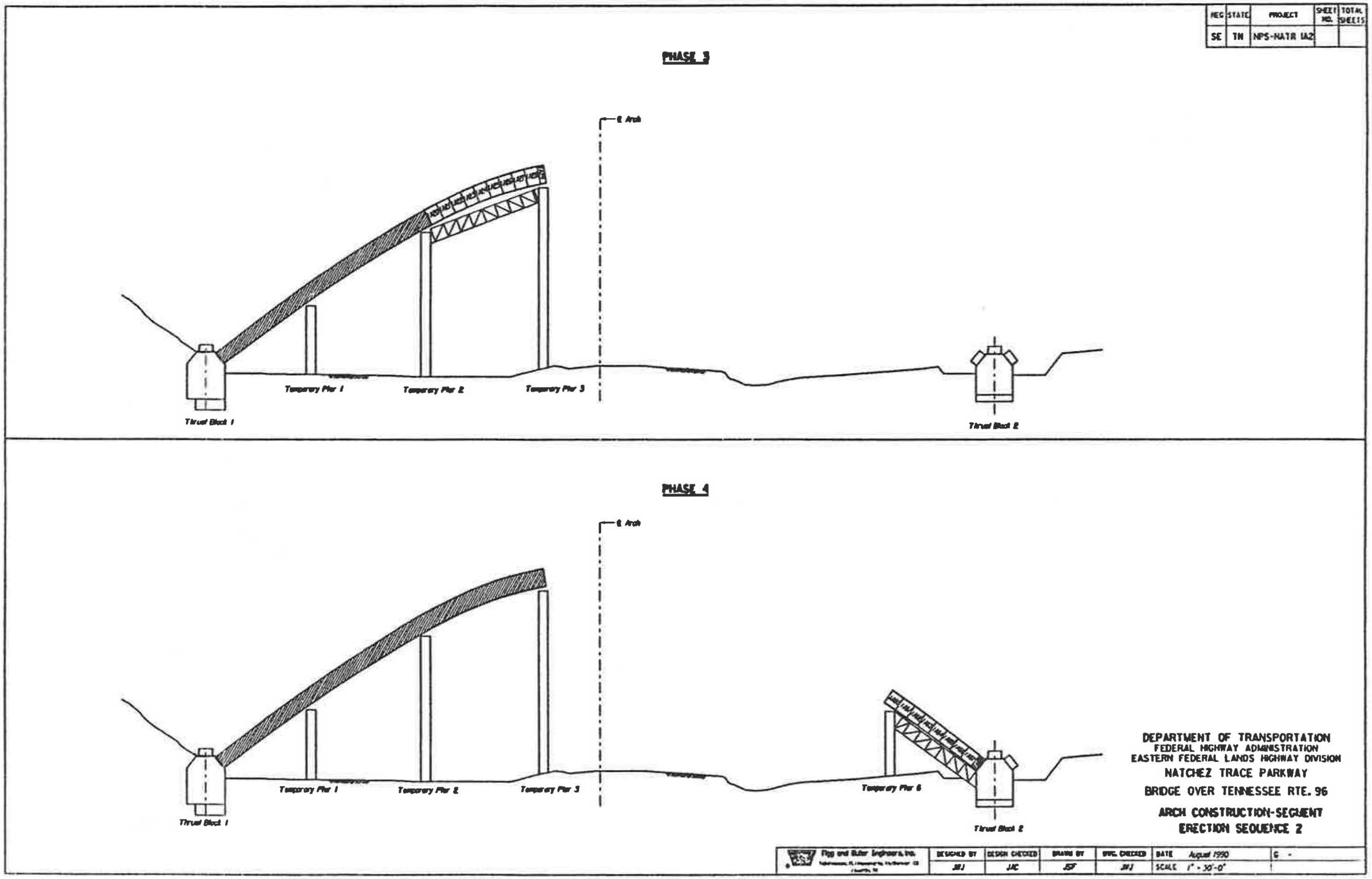

FIGURE 13 Arch construction - precast alternate - phases 3 and 4

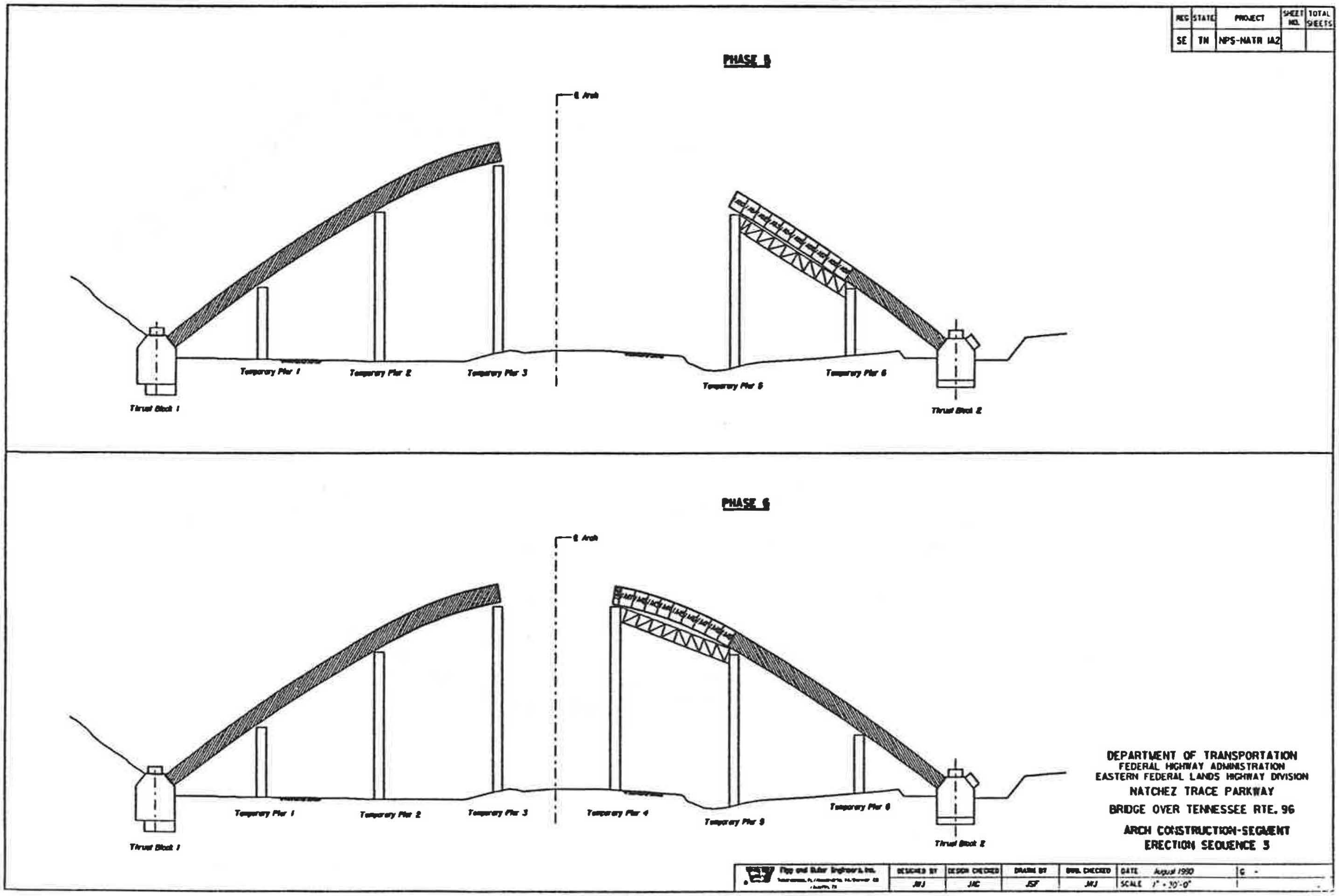

FIGURE 14 Arch construction - precast alternate - phases 5 and 6

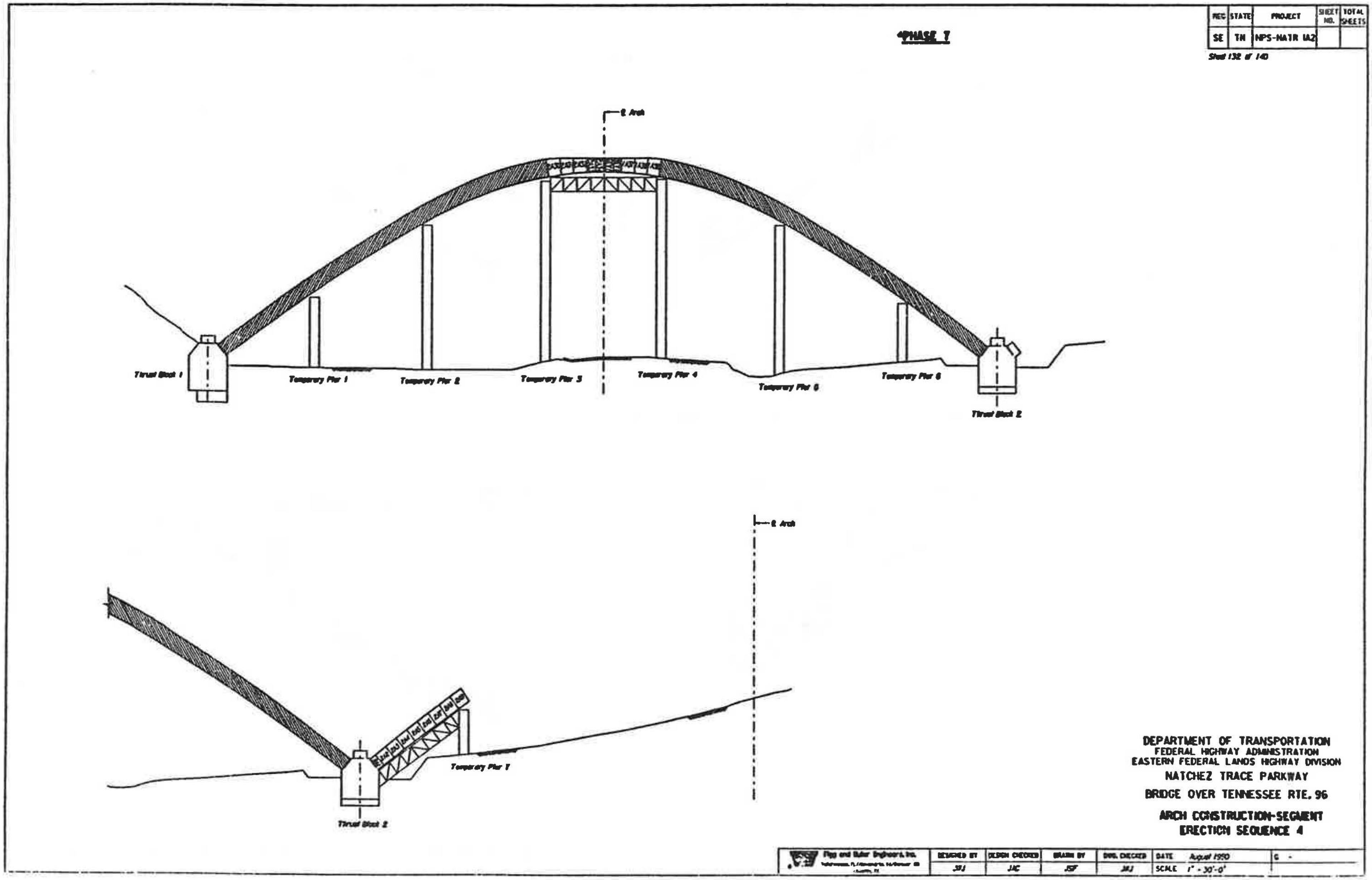

FIGURE 15 Arch construction - precast alternate - phases 7 and 8

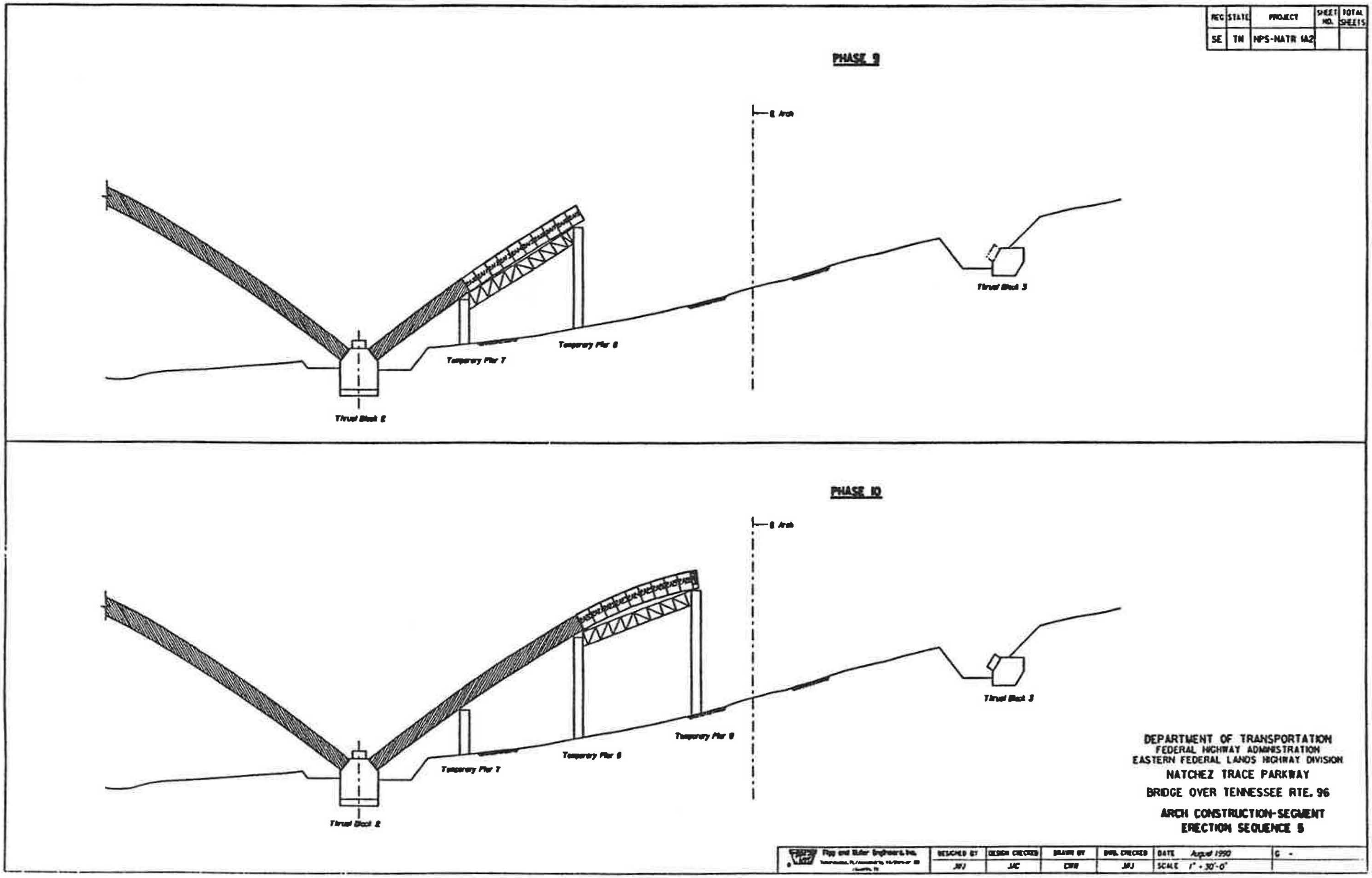

FIGURE 16 Arch construction - precast alternate - phases 9 and 10

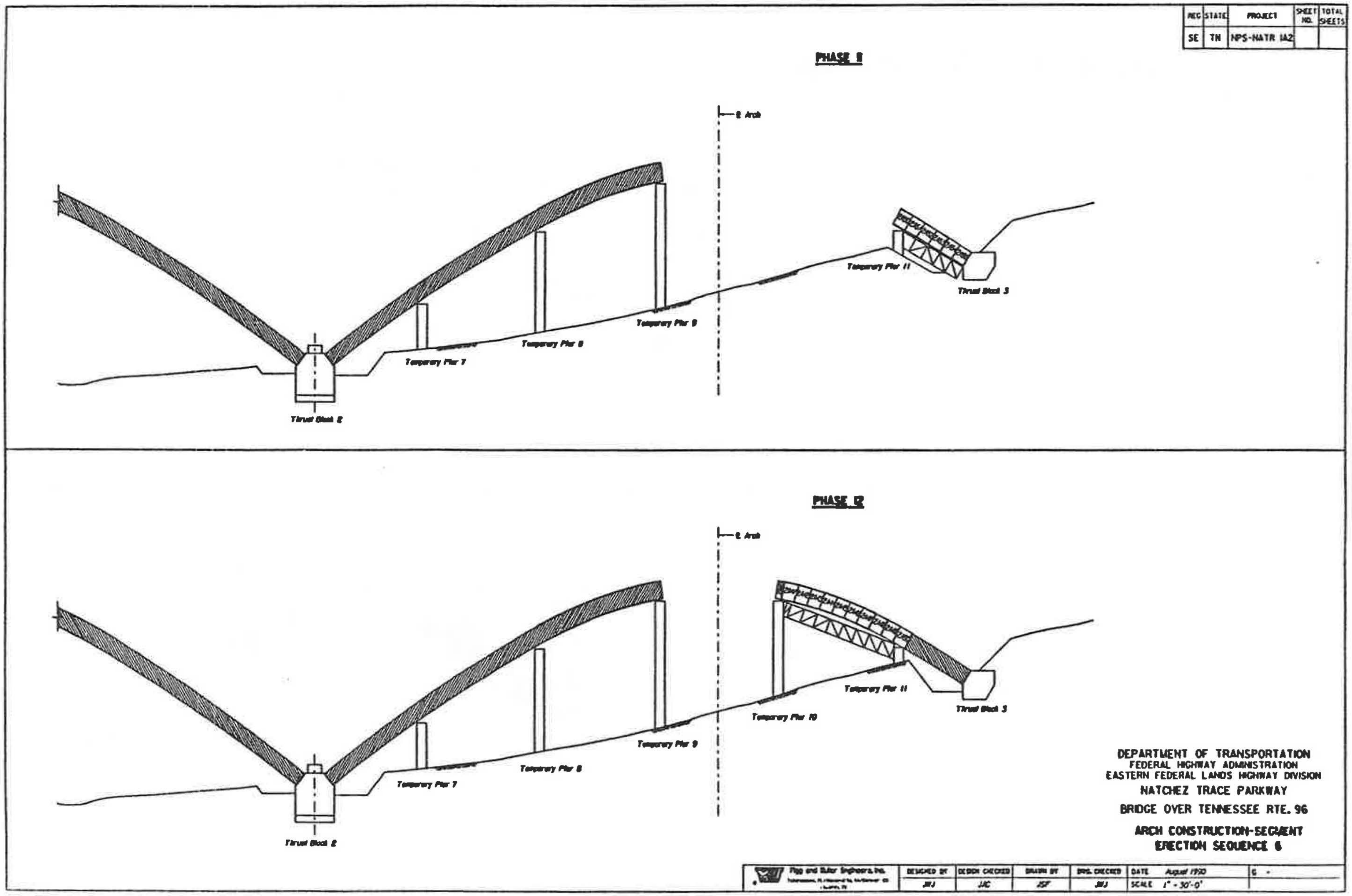

FIGURE 17 Arch construction - precast alternate - phases 11 and 12

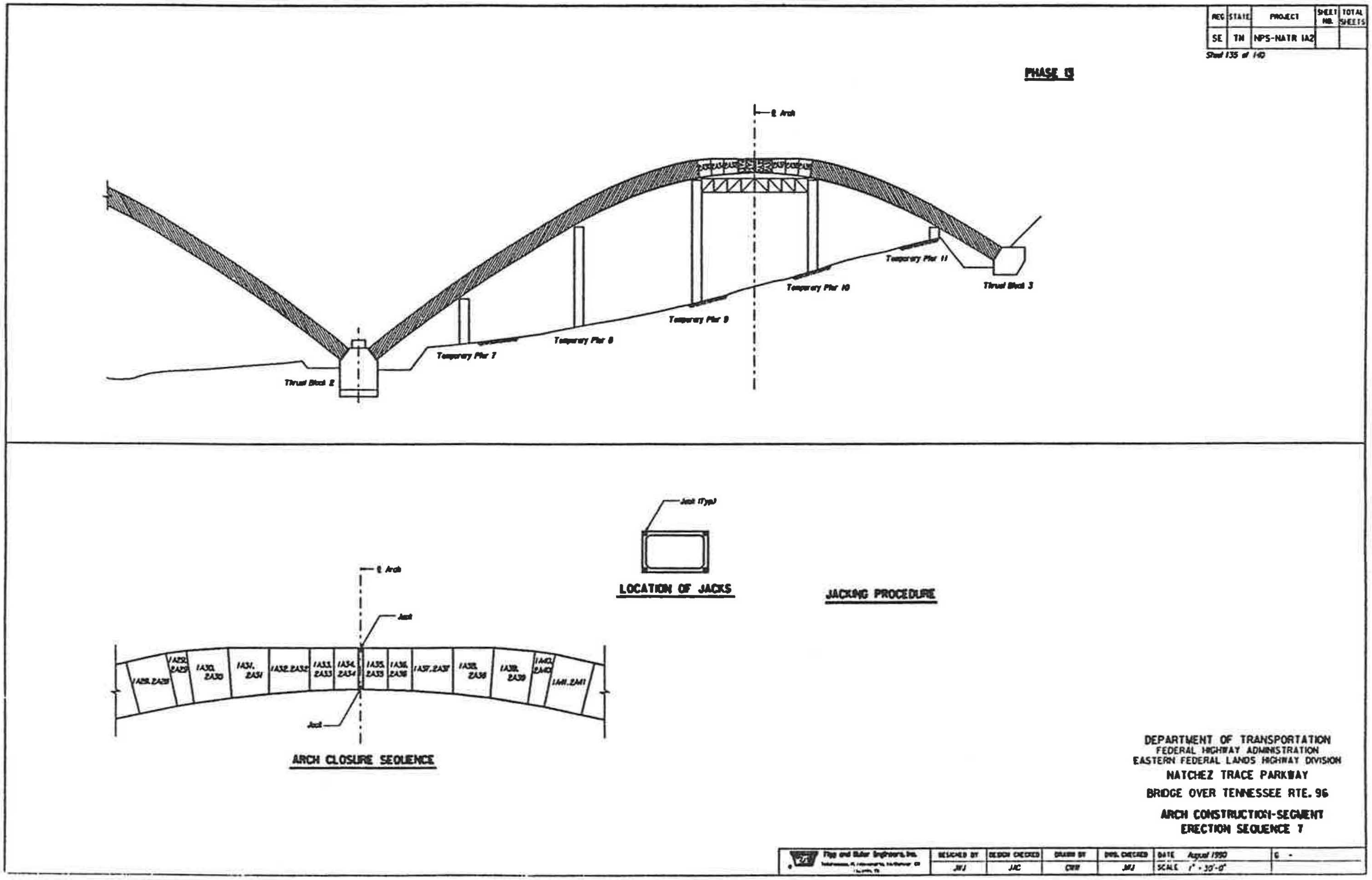

FIGURE 18 Arch construction - precast alternate - phase 13

Construction of arch 2 proceeds in similar fashion except that having a span of 462 feet, it contains only six intermediate spans.

Superstructure

A total of 196 precast segments are used to build the bridge superstructure. The typical segment length is 8 feet 6 inches. Maximum segment weight is 55 tons and minimum segment weight is 36 tons.

The superstructure of the bridge is designed to be constructed primarily by the balanced cantilever method of construction. Special modifications to this method are made for assembling the precast box girder segments adjacent to the abutments.

Erection begins after all precast piers and arch segments have been assembled, as shown in Figures 19 through 21. Access by ground-mounted crane is permissible for the entire length of the project except for the first 204-foot span. A steep slope rising approximately 150 feet over the span's length must not be marred by any of the construction activity. This requires a travelling beam-and-winch system for the construction of the first cantilever. The system must travel from a staging area at the base of pier 1, where segments are delivered, to the ends of the cantilever where the segment is being erected. This beam and winch has its rigging outside the width of the box girder to permit free passage of the segments.

The cantilever over arch 1 is assembled next. This cantilever is actually made up of two smaller cantilevers of six segments each. These smaller cantilevers are built about the bearing location of the bridge deck on the arch (45 feet off of centerline of arch). When each of the cantilevers are complete to a length of six segments, a 6-inch closure joint is poured over the centerline of the arch. Construction then continues on the now larger cantilever until reaching its full length in the 246-foot spans.

The construction procedure continues with balanced cantilever assembly over pier 2 with segments placed by a ground-based crane, or with the beam-and-winch system previously described, over arch 2 as previously described above, and over pier 3.

The last areas of segment placement are the approximately 70 feet of the first and last spans. As previously mentioned, this area near abutment 1 is not available for any construction activity. To overcome this difficulty, incremental launching techniques were investigated. Several options were investigated with regard to maintaining the stability of the segments being launched. It was decided to combine balanced cantilever construction with the concepts of incremental launching using the similar segments from the last span to build a complete cantilever.

Twin stub walls, 140 feet long, are built behind the back of abutment 1 bearing seats. These stub walls serve as the tracks for incrementally launching the first span. A balanced cantilever erection sequence is begun for a length of 140 feet on the stub walls, as shown in Figures 22 and 23. Segments from span 1 are used in their final location while the similar segments from the last span are used on the opposite side of the cantilever. When the balanced cantilever construction is complete, the girder is launched forward until the abutment segment comes to rest over the final bearings. The closure joint of span 1 is poured and the continuity post-tensioning is stressed, thus completing span 1. This is followed by the slackening of the cantilever tenndons and the removal of the segments of the last span. These segments are transported to the opposite end of the project where they will be assembled on falsework. After all tendons are stressed, the abutment backwalls are poured and the erection is complete.

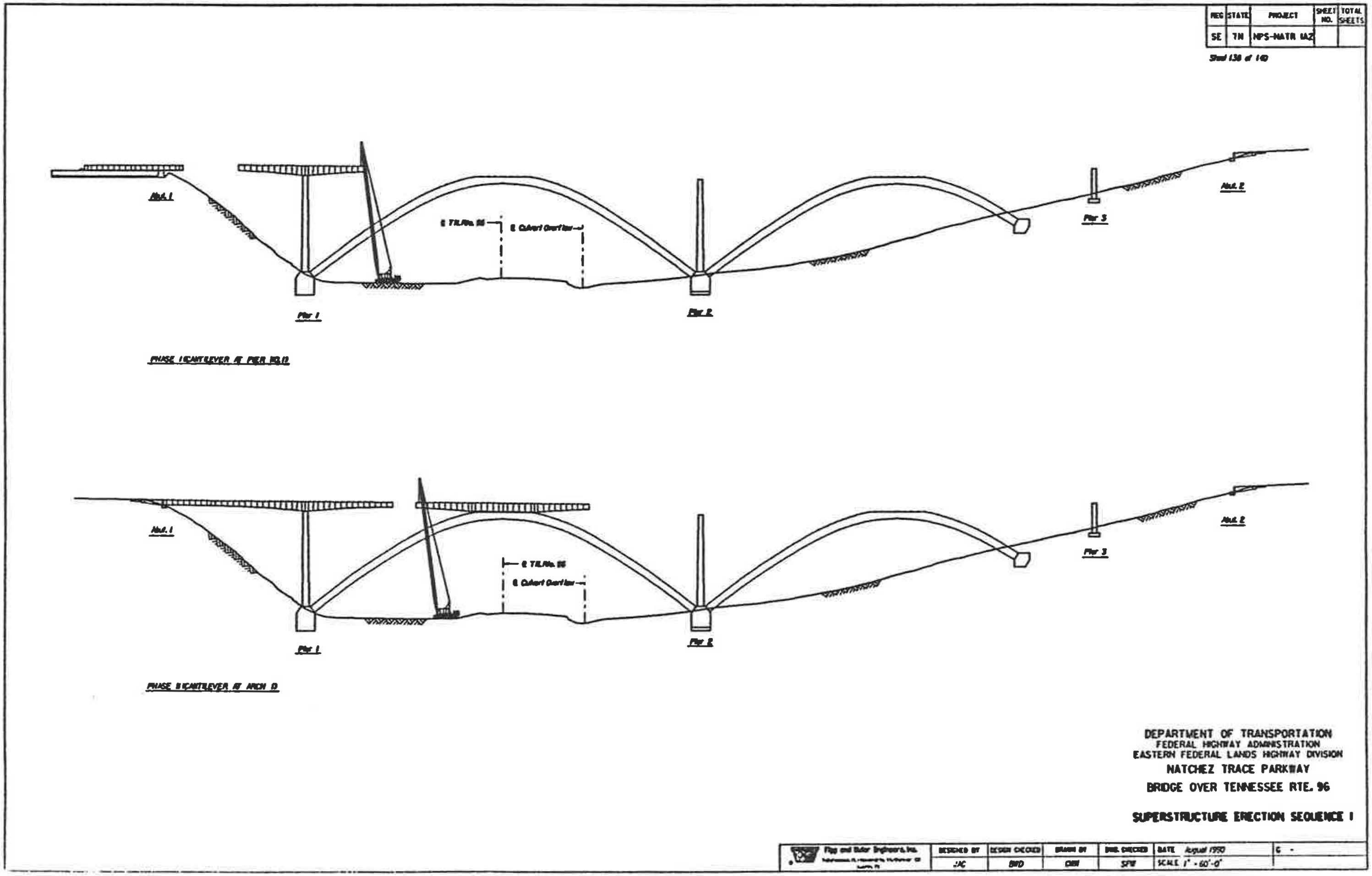

FIGURE 19 Superstructure construction - precast alternate - phases 1 and 2

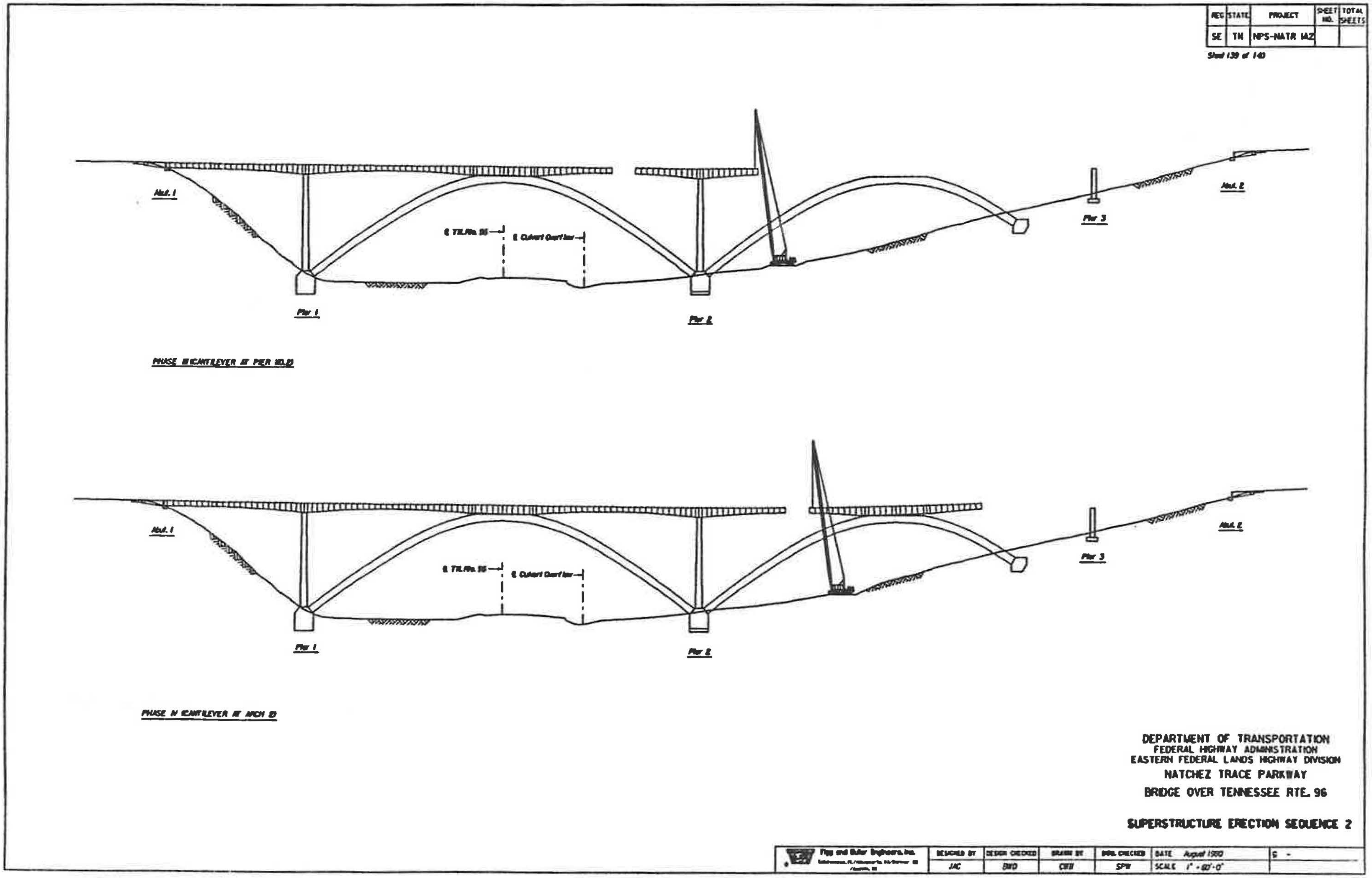

FIGURE 20 Superstructure construction - precast alternate - phases 3 and 4

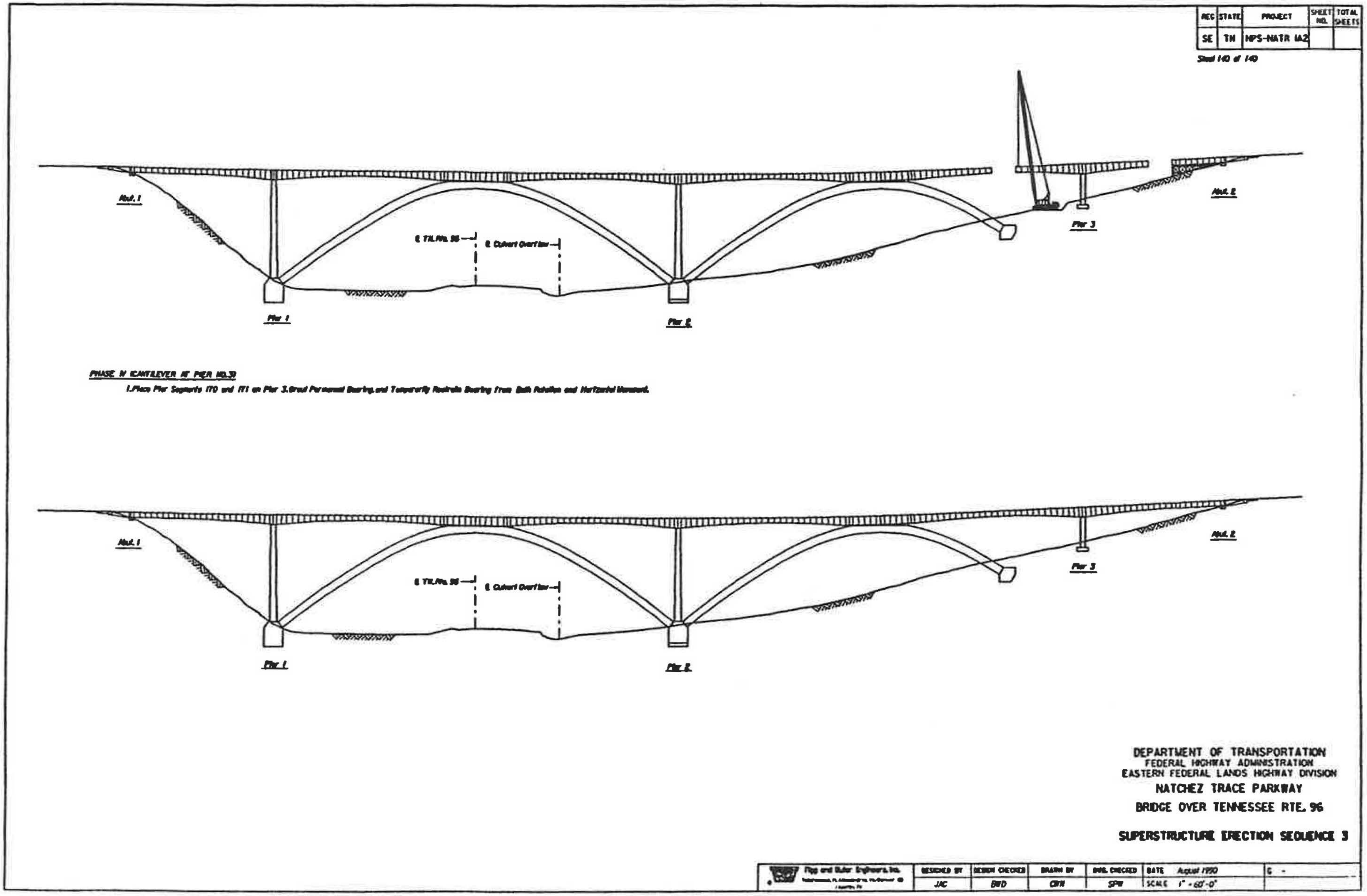

FIGURE 21 Superstructure construction - precast alternate - phases 5 and 6

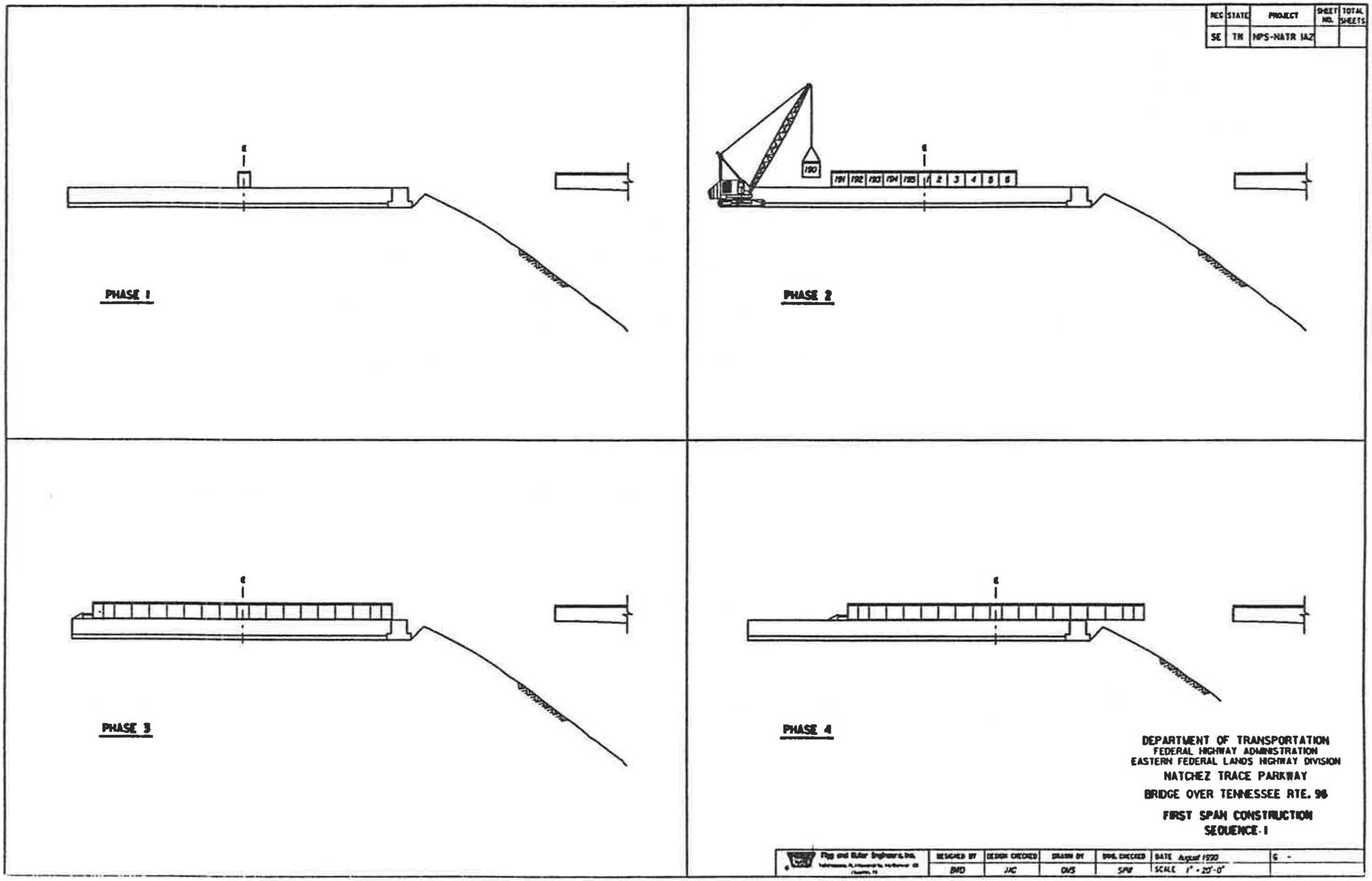

FIGURE 22 First span construction - precast alternate - phases 1 through 4

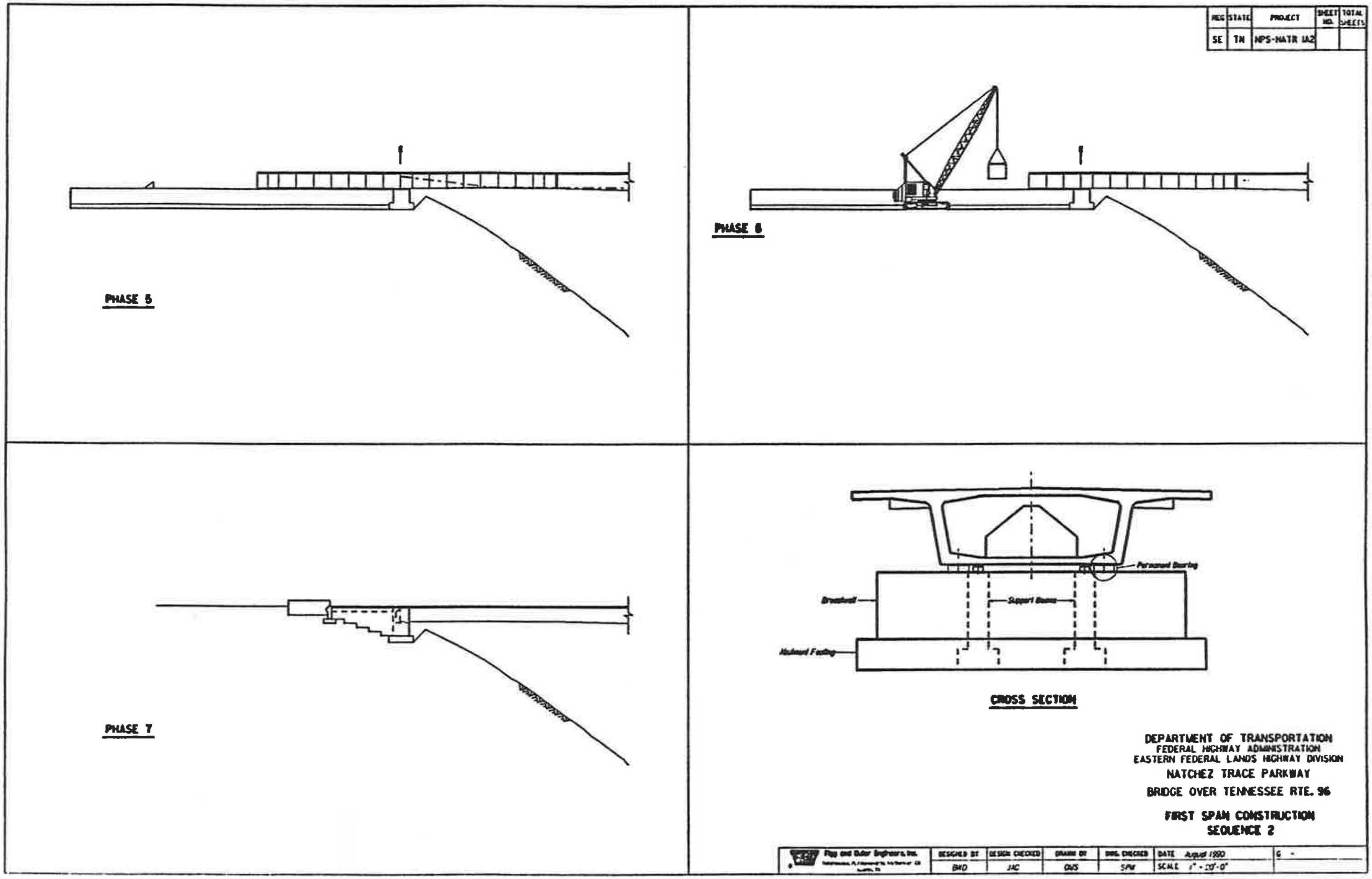

FIGURE 23 First span construction - precast alternate - phases 5, 6, and 7

Robert E. Lee Bridge, Richmond, Virginia

Man-Chung Tang, Nils D. Olsson, York-Kay Chan, and Philip J. Lang

The Robert E. Lee Bridge, is a segmental box girder structure which replaced the concrete arch bridge that had carried US Route 1 over the James River in Richmond, Virginia since 1934. The typical 285-foot span, variable depth girder was erected to emulate the arches of its soon-to-be-demolished predecessor.

Three northbound and three southbound lanes with full 10-foot shoulders marked as bicycle paths and 4-foot sidewalks on each side travel across 15 spans. The typical width of the northbound and southbound bridges is 54'-1 1/2". The bridge has bifurcated sections off the southbound structure at the south end and to the north. Both the northbound and southbound structures have ramps tying into Second Street which runs under the new bridge.

The fifteen spans (total length 3,755 feet) were built by balanced cantilever method with the exception of the first span from the south abutment and the three ramps which were built on falsework. The superstructure is divided into areas by six joints with typical joint locations at the abutments and at the center of the 285-foot spans.

The appearance from below the bridge was a major concern because of the public use of the island park. The cast-in-place solution provided smooth curved sections with monolithic pours providing a consistent soffit texture and color. In addition, the number of piers was minimized with the two-box system providing a more open view. Plans presently call for supporting a pedestrian bridge from the superstructure under four typical spans from the north shore to Belle Island.

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GENERAL

The Robert E. Lee Bridge in Richmond, Virginia was originally a concrete arch structure constructed from 1933 to 1934. The bridge spanned the James River connecting the downtown area of Richmond with the "Southside". In midstream is an Island, the Belle Isle. Here, the crossing maintained its elevation of about 90 feet above the river, while crossing the island. The bridge had served as a major link between the north and the south sides of the City for over 50 years.

Continued deterioration of the bridge deck of this structure and the associated increase in maintenance costs created the need for a new bridge. The new structure needed to maintain the existing alignment at the approaches in order to tie into existing road systems. Another stipulation was that the arch appearance of the old structure should be reflected in the new bridge.

The original Lee Bridge had four, 10-foot traffic lanes and 4-foot sidewalks on each side. The total roadway width was 40 feet at a typical cross-section, which meant that there were virtually no shoulders and no median barriers. At the North end (Richmond side), two ramps connected with Second Street while the main approach connected with Belvedere Street. On the South end, an off-ramp for south bound traffic was connected to Riverside Drive, and the main approach was Cowardin Avenue.

THE NEW BRIDGE

The new bridge called for three, 12-foot wide lanes and a 10-foot wide bicycle/emergency lane in each direction with sidewalks on each side. On and off ramps were to be provided to Second Street with an off-ramp for southbound traffic to Riverside Drive and an on-loop under the bridge on the South end from Riverside.

Two alternate designs, one concrete and one composite, were prepared for

bidding. The concrete alternate was designed by DRC Consultants, Inc. of New York and Hayes Seay Mattern & Mattern of Roanoke, Virginia. The composite alternate was designed by Beiswenger Hoch & Associates, Inc., Miami and St. Clair Callaway & Fryr, Richmond, Virginia. Both featured box type girders, a form established after extensive study of aesthetics.

Eight bids were received with all five low bids based on the concrete alternate varying from the $32.2 million lowest bid from McCarthy Brothers Co. to $45 million. The remaining three higher bids were all based on the steel alternate. Their bid prices varied from $52.6 million to $54.8 million. The concrete alternate bid was thus 39% lower than the steel alternate and the contract was awarded to McCarthy Brothers Co.

Under the design constraints mentioned above, the resulting concrete structure consisted of twin bridges, one for Northbound traffic (NBL) and one for Southbound traffic (SBL) with a nominal gap (1" for most of the structures) separating the two. Hexagonal shaped piers on spread footing founded on the rock, rose 75 to 90 feet above the river surface to the soffit of the box girder superstructure. The box girder consisted of both single and double celled sections with exterior web walls on a one to three slope and a depth varying from 16 feet at the piers to 7-1/2 feet at midspan. The average width of the top slab was 55 feet with variable widths to 89 feet at the ramp locations. Spans were 285 feet for the most part, Fig. 1. These span lengths, combined with the varying depth of the box section offer an arch appearance. It is aesthetically pleasing.

SUBSTRUCTURE

The Contractor mobilized in 1985, and started work on the footings of the piers located on Belle Isle. There was a causeway constructed in the south channel of the river to allow access to the island. Causeways had to allow for the passage of fish, and work in the river was not permitted from mid March to the end of May. Towards the end of 1985, serious flooding washed out the south and north channel causeways. Throughout the project, the James River periodically over-topped the causeways.

All footings, with the exception of the North end piers and North abutment were cast on granite bedrock. On land, excavations to bedrock were relatively shallow. A combination of blasting, rock splitting, and jackhammers exposed good granite surfaces. The footings varied in size with the typical span footings being about 7' x 25' x 25'. In the river, the Contractor made use of a cofferdam system called a "porta-dam". This system incorporated a series of frames which ringed the footing location. A waterproof fabric was then suspended from these frames and spread along the encircling riverbed. As water was pumped out, the fabric created a tight seal against the riverbed and was strong enough to span between the steel frames, Fig. 2. Blasting within the confines of the porta-dam posed no problems and the system worked very well. The porta-dam was limited to about 10 feet of depth. At the North end of the alignment, the banks of the river rose sharply to the North abutment. Piers 13 and 14 and the abutment itself were founded on piles.

The pier columns had a hexagonal shape, Fig. 3, and for the most part were solid cast-in-place sections. Some of the wider piers were originally designed with voids, but the Contractor opted to make these solid to avoid the inside forms. As the end section of all the piers were the same, the Contractor made use of steel forms with this shape built in, and with filler panels that would allow for the varying widths of the piers. Inserts in the pier shafts for the superstructure stability system and the pier table brackets consisted of posttensioning ducts that required accurate placement.

Pot bearings (both guided and fixed) were called for at each pier, abutment, and expansion joint. The bearing assemblies for the structure consisted of a masonry plate which was cast into the pier-head; a sole plate cast into the soffit of the pier table segments, and the pot/piston itself which was bolted to these two plates. It was a design requirement that the bearings be replaceable. The masonry plates were positioned at the top of each pier and their legs cast in. The gap between the top of the pier concrete

and the plate was grouted in at a later date. The rest of the bearing assembly was bolted in place when the pier table segments were formed.

SUPERSTRUCTURE

The superstructure construction can be broken down into two main areas : portions that were constructed on falsework, and portions that were built using cast-in-place balanced cantilever method. Span 1 on both the North and South bound structures were built on falsework, as were all the approach ramps. The remainder of the bridge was built using segmental balanced cantilever construction.

FALSEWORK CONSTRUCTION

On span 1, the falsework scheme called for concrete pours sequenced to reduce the effects of shrinkage. The Southbound structure had a span of 182 feet and had a widened section to accommodate ramp "A" which extended to Riverside Drive. The maximum cross-section was over 89 feet at the ramp expansion joint. Forming this section of the structure was complicated due to the varying sections, the large number of posttensioning buttresses required for the span itself and for the future cantilever work from pier 1 and, the changing crossfall for the ramp and the slight curve at the South approach. After these spans were completed and stressed, the "wing" falsework of the box section was removed but the main shoring towers had to remain in place until three or four segments had been cantilevered out beyond pier 1 to keep within the allowable stresses in span 1. Both the Northbound and the Southbound structures consisted of double-celled box sections at pier 1. Ramp "A" was cast shortly after the span 1 structures. All the ramps were solid box sections.

Ramp "A' on the South side was cast in one pour. The on/off ramps to Second Street on the North side had five spans each ranging in length from 58 feet to 164 feet. These ramps were cast in four pours each on falsework.

Because of the long spans involved, the Contractor cast 30-foot cantilevers beyond each pier as work progressed from the connection with the main structure towards the North. These 30-foot projections were used as a support for the main longitudinal falsework beams, eliminating the need for shoring towers at this location. 400 and 800-kip capacity towers were placed at a spacing of about 70 feet on average and two longitudinal W 36 x 260 beams were supported on the towers. Wide flange steel sections were laid transversely and forms consisting of aluma-beams and plywood completed the soffit forms. The wing and wall forms were prefabricated units about 18 feet long which could be lowered and winched along channel rails to the next pour.

Posttensioning of the ramps consisted of draped tendons which were coupled at each of the construction joints. The transverse posttensioning was 1-1/4" Dywidag Bars anchored in the 8" deck slab. The finished surface of the ramp sections was the riding surface; no overlay was placed on the ramps.

CANTILEVER CONSTRUCTION

The remainder of the bridge was built using balanced cantilever method. The cantilevers from piers 2 and 3 SBL and from piers 9, 10 and 11 consisted of double-celled box sections and added varying widths and superelevations to the other variables. The remaining piers were single-celled sections of constant width. Briefly, the construction sequence from each pier was as follows:

1. Brackets were fixed to the pier to support the pier segment forming system.
2. Bearings were bolted to the masonry plates in the pier head.
3. Soffit and exterior wall forms constructed.
4. Bottom slab and 3" of web wall cast.
5. Web walls formed and cast.
6. Top slab formed and cast; segment stressed.
7. Short side forms removed; stressed to bracket.
8. Long side forms removed; stressed to bracket.
9. Formtraveler 1 erected; segment "A" cast; traveler 1 advanced.
10. Second traveler erected; stability ties stressed in segment "A"; segment "B" was

cast; traveler was advanced.
11. Stability ties stressed in segment "B"; segment "C" cast; brackets free for removal.
12. Cast alternate segments then cast closure.

The brackets were large structural steel "knee" braces that were stressed against the piers using 1-1/4" Dywidag bars which had been fed through the embedded ducts mentioned above, Fig. 4. Due to the hexagonal shape of the piers, a precast wedge was used to ensure that the brackets were perpendicular to the pier. Once these brackets were fixed to the pier, transverse structural sections were placed on them and the soffit forms attached, Fig. 5. At this time, the bearing assemblies were bolted to the masonry plates that were cast into the pier.

The soffit forms and the other web wall forms were erected for the 38 feet long pier segments. Three inches of web wall were cast with the bottom slab to prevent cracking along the web/bottom slab interface. Once the bottom slab had been cast, the web walls and diaphragm over the bearings were completed. The top deck slab was formed using scaffolding to the transverse structural beams for the wing sections, and the interior soffit forms were attached to the tops of the web walls using inserts that were cast in. The completed pier segment (pier table) was 38 feet long with 23 feet on one side of the pier centerline, and 15 feet on the other. This 8-foot out of balance was maintained throughout the cantilever process.

Once the pier table was stressed both longitudinally and transversely, the soffit forms were removed. Sand jacks under the transverse beams supporting the forming system were lowered on the short side, the forms were removed and, the pier table rested on the long side with its 8-foot out of balance. Dywidag bars were then attached to the short side bracket and stressed sufficiently to rotate the pier table off of the long side forms so that they too could be removed. Bars were then stressed from the bottom slab on the long side to the brackets and final adjustment to the rotation of the pier table was made with the bars on each end of the segment. It should be noted that the stability system for this project was a soft system, using strand ties, and the rotations needed to be worked back from the required closure orientation to the initial pier table posttensioning. Elongation of the stays/bars and deflection of the piers at each stage was calculated and checked in the field (on some occasions it was necessary to adjust the stay cables to ensure the proper final alignment).

With the forms removed, and the pier table fixed to the brackets, the first formtraveler was erected on the short side. Segment "A" was placed and stressed, and the traveler advanced onto it. Once the traveler was advanced ahead, the second traveler was erected and the stability stays to segment "A" were installed and stressed. The bars to the bracket on the short side were not removed and segment "B" was cast. The second traveler was advanced and the stability installed and stressed in segment "B". Now that the cantilever out of balance was entirely on the stay cables of the stability system, the brackets were removed for use elsewhere and, cantilever production continued alternately on each side of the pier to closure.

The stability system, designed to compensate for the 8 feet out of balance, consisted of the brackets for the first segment and pier table and, the stability stays for the remaining segments. The stays consisted of 0.6 strand tendons designed to carry the maximum out of balance at less than 50% of the ultimate strength of the strand. The typical stay arrangement called for two 19/0.6 tendons on each side of the pier. The tendons were anchored at stressing blocks attached to the pier; they then went through the pier and curved towards the web walls, 27 feet from the pier centerline. They went through the bottom slab and were anchored in a buttress that was cast with the web walls. Extra reinforcement at the buttress locations was required for the shear stresses produced by the stability stays, Fig. 6.

Most cast-in-place cantilever bridges used only knee bracings to stabilize the superstructure. The Contractor elected to use the cable stabilizing system in this structure.

Because the cable stabilizing system

at the piers during cantilever construction was flexible, maintaining the correct geometry and camber for the casting operations involved the rotation of the structure. To account for the rotations when setting forms, baselines were established on each cantilever. Elevations given to the crew to set the forms for each segment were thus adjusted to compensate for the rotation at any given time.

The average cycle time for the single-celled, constant width segments was one cycle per week. Each cycle consisted of:

Day 1 : Stress tendons, advance the form traveler, set preliminary grade, and start forming.

Day 2 : Install the bottom slab and web wall reinforcing, and top slab-bottom mat rebar.

Day 3 : Install the longitudinal and transverse posttensioning and complete bottom buttresses (if any).

Day 4 : Complete top slab reinforcing and posttensioning, close in web walls, and cast segment.

Day 5 : Cure and prepare for stressing.

The two-celled segments required an extra two days per cycle on average due to the extra forming, larger size, and because adjustments to the formtraveler were required as the section width changed. Due to the changing width of the deck and other reasons, the original design suggested to build this portion of the bridge on falsework. The Contractor, however, elected to use formtravelers instead.

In span 12, the two ramps - C & D connected with the main structure. This occurred at a point roughly 65 feet North of pier eleven designated as expansion joint #4. In order to complete this section of the bridge using the travelers, the Contractor opted to use shoring towers as props to continue cantilevered construction, Fig. 7. Construction followed the normal sequence, and the expansion joint segment was cast on the north end of pier 11 cantilever. On the opposite end, under the previous segment the towers were in place, one under each of the three web walls. The next segment to the south was cast after the expansion joint segment was completed and the south cantilever came to rest on the temporary prop. Segments to the south were completed by cantilevering beyond the prop to closure.

On the North side of the joint, a similar approach was used on the cantilever from pier 12, but towers were required at two locations as indicated on the sketch.

On the North end of the main structure, the longer end spans negated balanced cantilever work so again the Contractor used temporary props to continue using the formtraveler. Extra posttensioning was added to complete work in this fashion. The last segments on both the NBL and SBL were formed using support from the abutments and hanging supports from the cantilever tips.

As the superstructure neared completion, barriers, sidewalks and overlay were being cast. The NBL structure was actually opened to traffic in June of 1988. This allowed three lanes of traffic to be diverted to the new bridge. The North end of the existing bridge was then demolished to allow the SBL structure to tie into the now widened approach on a similar alignment as the old bridge. The completed structure was officially opened to traffic in November, 1988.

The total bridge length of 3760 feet has over 41,000 cubic yards of 5000 psi concrete in the superstructure; 8,624,000 lbs. of reinforcing steel and approximately 3,220,000 lbs. of posttensioning steel.

The completed Lee Bridge now provides far superior access across the James River on an aesthetically pleasing structure, Fig. 8.

NEW PEDESTRIAN BRIDGE

Because the Robert E. Lee Bridge is a high level bridge, it does not provide any access from the North bank of the river to Belle Isle which is to be turned into a park. In order to offer this needed access to the citizens, Richmond Renaissance commissioned DRC Consultants, Inc. to design a pedestrian bridge. Aesthetics is an important consideration in this design. After extensive study of various bridge

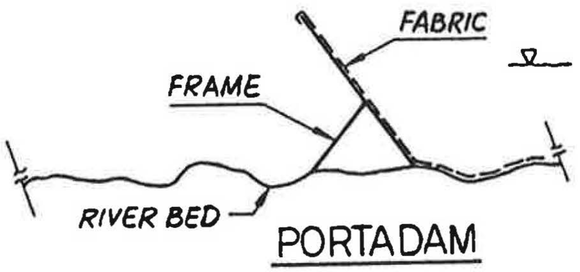


FIG. 2

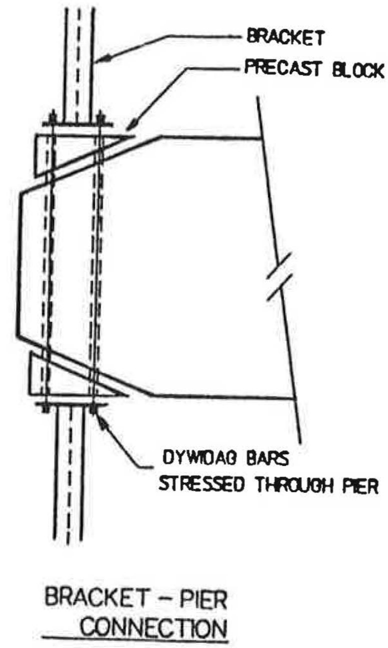


FIG. 5

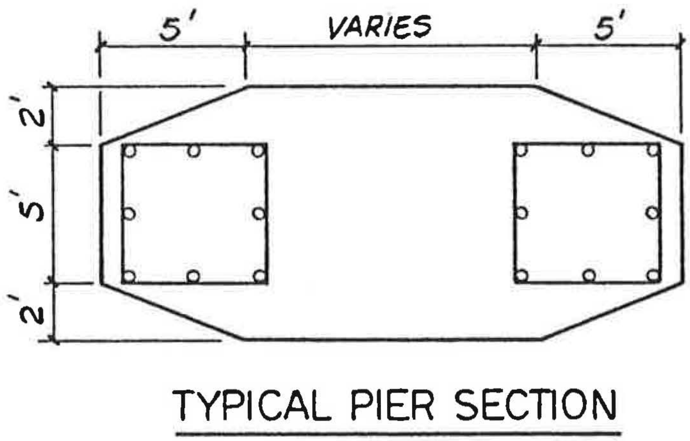


FIG. 3

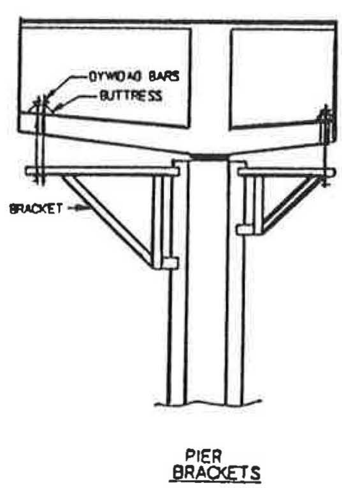


FIG. 4

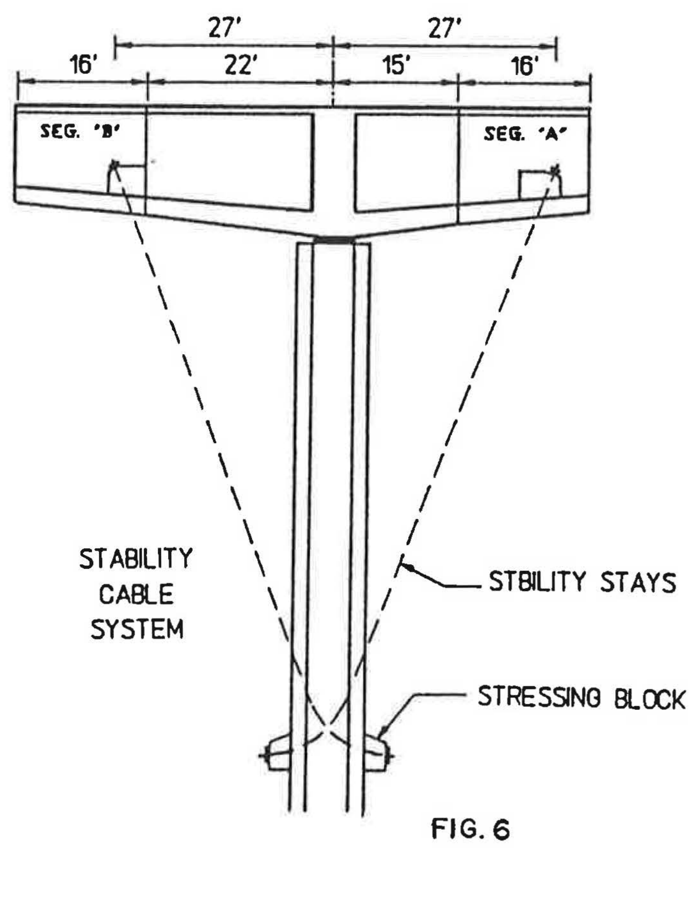
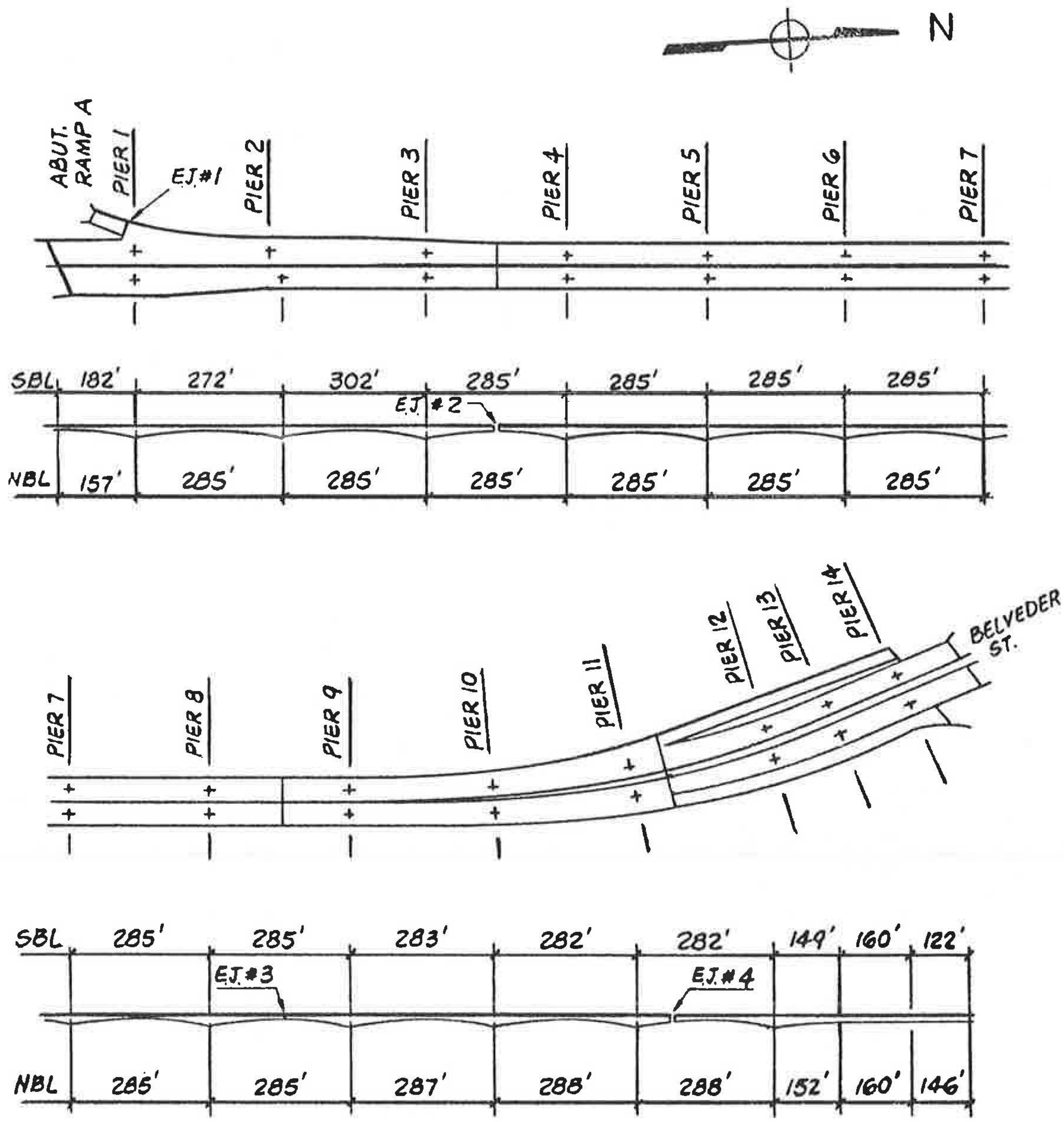


FIG. 6

configurations, DRC came up with a composite deck bridge to be suspended from the just completed Robert E. Lee bridge running between the northbound and southbound structures. The deck is curved to simulate the curved soffit of the high level bridge, offering a unique view of the river front, Fig. 9.

In October 1989, a contract was awarded to build this pedestrian bridge suspended from the Lee Bridge connecting the North bank of the river to Belle Isle.



ROBERT E. LEE BRIDGE

FIG. 1

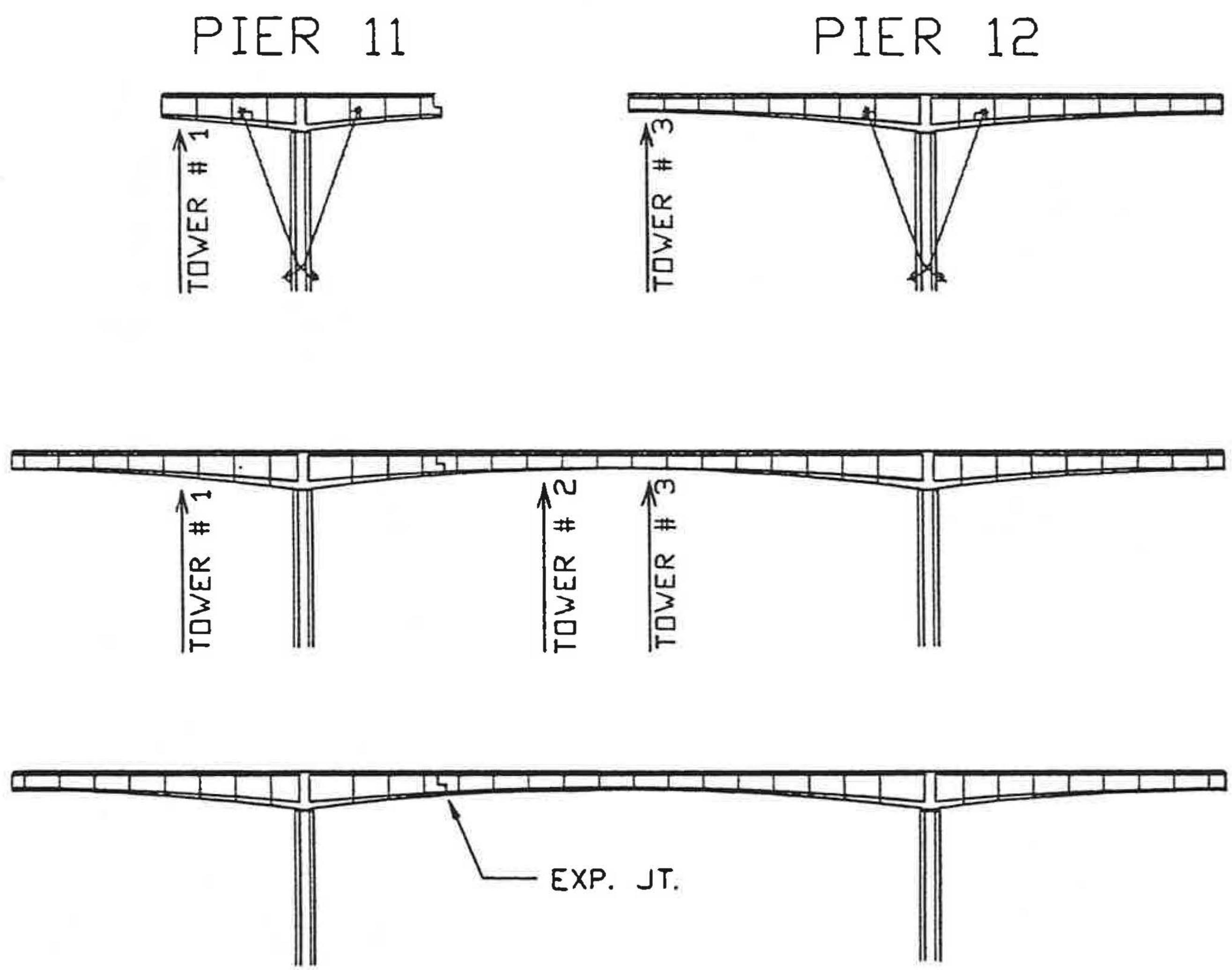


FIG. 7

Fig. 8. The Completed Bridge

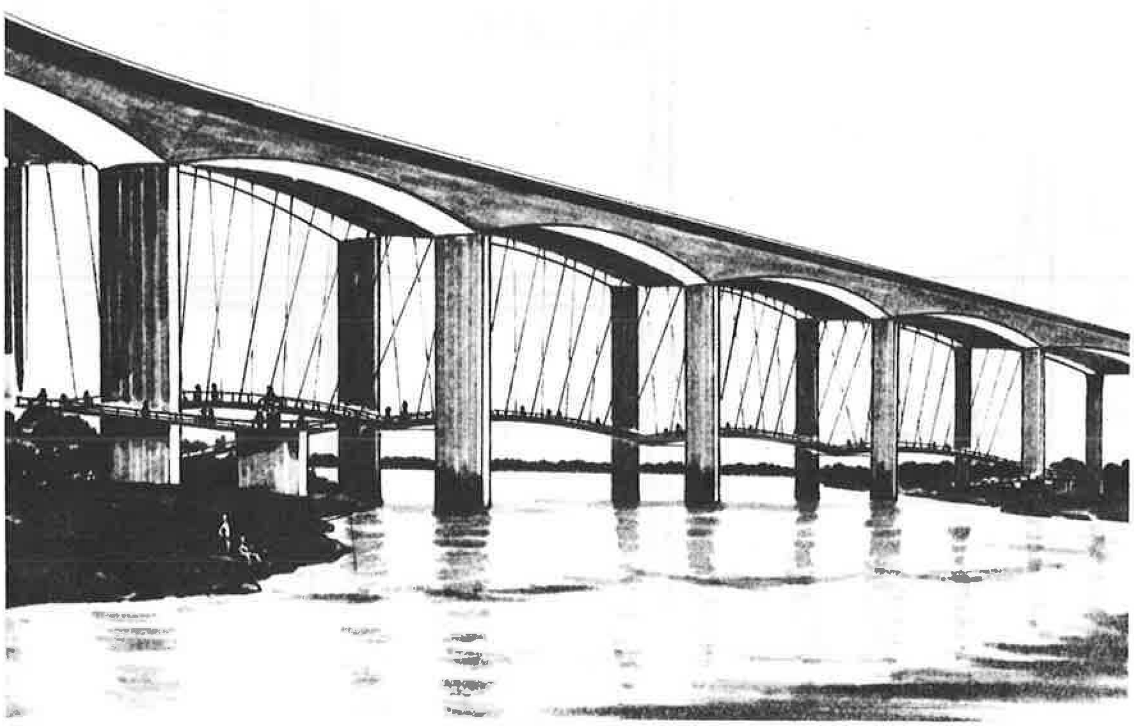


Fig. 9. The Pedestrian Bridge

Recent Advances in Seismic Design and Retrofit of Bridges

JAMES E. ROBERTS

This paper describes the seismic retrofit program implemented by the California Department of Transportation after the 1971 San Fernando Earthquake and accelerated after the 1989 Loma Prieta earthquake. The goal of the program is to reinforce most of the bridges designed and constructed prior to acquiring the current levels of knowledge of seismic loading and structure response (about 1978) to improve their structural ductility and resistance to the major factors which contributed to damage and collapse in the San Fernando and Loma Prieta events. The program has been executed in two phases. The initial phase, completed in 1988, provided reinforcement to superstructures of 1260 bridges by connecting all narrow expansion joint seats with hinge restrainers and anchoring girders and other superstructure elements to the substructure. The single and multiple column retrofit phase, now under active implementation, is designed to reinforce substructure elements and create ductile members by increasing confinement. A Risk Analysis procedure was developed to prioritize the bridges for seismic upgrading so that highest risk bridges are retrofitted first. Legislation was recently enacted to require the Department of Transportation to also take the lead role in seismically retrofitting all locally owned bridges in the state. Research has been conducted, and is currently underway, at the University of California, San Diego to test and confirm the validity of several proposed design solutions for seismic retrofitting of existing bridge single column bent substructure elements. Since the Loma Prieta earthquake additional research is being conducted at the University of California at Berkeley, Irvine and Davis to develop and test retrofit techniques for multiple column bents and double level structures, including abutment and footing details. Some full scale testing was completed on a standing portion of the Cypress Street viaduct in Oakland to determine its fundamental response characteristics and to test some techniques which may be applied to the double deck structures in San Francisco and on complex structures with outrigger supports and other non standard support configurations throughout the state.

BACKGROUND

The 1971 San Fernando earthquake caused substantial damage to recent bridge construction and exposed a number of deficiencies in the bridge design specifications of that time. These deficiencies have the potential to impact dramatically on transportation life- lines and the traveling public today. Bridge design specifications were immediately modified to correct the identified deficiencies on newly designed bridges. Structures in-place however, have served to be a substantially more challenging problem. Research was undertaken both in the United States and overseas to improve analytical techniques, and to provide basic data on the strengths and deformation characteristics of lateral load resisting systems for bridges.

The initial phase of the Bridge Seismic Retrofit Program involved installation of hinge and joint restrainers to prevent deck joints from separating. This was the major cause of bridge collapse during the San Fernando earthquake and was judged to be the highest risk to the traveling public. Included in this phase was the installation of devices to fasten the superstructure elements to the substructure to resist vertical accelerations and also to prevent superstructure elements from falling off their supports. This phase is now essentially complete after approximately 1,260 bridges on the State Highway System have been retrofitted at a cost of over 55 million dollars.

While the hinge and joint restrainers performed well, shear failure of columns on the I-605/I-5 separation bridge in the moderate Whittier Earthquake of October 1, 1987 reemphasized the inadequacies of pre-1971 column designs. Even though there was no collapse, the extensive damage resulted in plans for basic research into practical methods of retrofitting bridge columns on the existing pre-1971 bridges. The research program was initiated in 1987 and is currently being conducted at the University of California at San Diego.

The Loma Prieta earthquake of October 17, 1989 again proved the reliability of hinge and joint restrainers but the tragic loss of life at the Cypress Street Viaduct on I-880 in Oakland emphasized the necessity to immediately accelerate the column retrofit phase with a higher funding level for both research and implementation. Other structures in the earthquake affected counties performed well, suffering the expected column damage without collapse. With the exception of an outrigger column-cap confinement detail, those bridges using the post-1971 design specifications and confinement detail changes performed well. The effect of the response of deep soft soils in the structure foundations proved to be a contributing factor which must be analyzed and included in future design procedures, especially for long, tall structures with relatively high periods of vibration.

PRE AND POST 1971 COLUMN DESIGN

Bridge columns designed before the 1971 San Fernando earthquake typically contain very little transverse reinforcement. A common detail for both circular and rectangular columns consisted of #4 (12.7 mm diam.) transverse peripheral hoops at 12 inch to 18 inch (300 mm to 450 mm) centers, regardless of column size and area of main reinforcement. As a consequence of these details, the ultimate curvature capable of being developed within the potential plastic region is limited by the strain at which the cover concrete starts to spall. The result is flexural failure resulting from inadequate ductility capacity, or shear failure due to lack of adequate shear reinforcement. Tests conducted at UC San Diego confirm this theory. Several bridges suffered column shear failures due to the elastic design philosophy under which they were designed prior to 1971.

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Columns designed since 1971 contain approximately eight times the confinement reinforcing steel than pre-1971 designs. All new columns, regardless of geometric shape, are reinforced with one or a series of spiral wound circular cages. The typical transverse reinforcement detail now consists of #6 (19 mm diam.) hoops or spiral at three inch (76 mm) pitch. All main column reinforcing is continuous into the footings and superstructure. Splices are mostly welded or mechanical, both in the main and transverse reinforcing. Transverse reinforcing steel is designed to produce a ductile column by confining the plastic hinge areas at the top and bottom of columns.

RETROFIT PHILOSOPHY

There are some 25,000 highway structures for which the State of California is responsible in some capacity. It is economically unrealistic to suggest that every structure be immediately retrofitted to withstand maximum magnitude earthquakes without some damage. The retrofit philosophy adopted at the start of the Restrainer Retrofit Phase of the program offers reasonable direction to the remaining phases of the Retrofit Program.

It was and still is Caltrans philosophy to first retrofit those structures which pose the greatest risk to the public and are the most vital to the transportation system. The ultimate goal is to see that all of the bridges in the state will be capable of surviving maximum credible earthquakes without collapse. Some damage is inevitable but, with proper retrofitting, it is believed that collapse is preventable and, further, it is believed that damage can be held to a minimum, to the extent that the transportation system can remain open and functioning during repair. As we get into the actual analysis and design it is becoming apparent that retrofitting many older structures to this standard may not be cost effective. It is highly probable that we may have to accept some period of closure on many structures to effect repairs. Time and further analysis will bring this problem to the decision makers. As a direct result of the one month loss of the San Francisco-Oakland Bay Bridge during the Loma Prieta earthquake, it has been recommended that major transportation structures be designed for higher elastic seismic force levels and longer shaking periods to reduce the damage to non structural type. To accomplish this goal a new "importance factor" will be introduced into the design and retrofit criteria. This represents a major change in the seismic design criteria for bridges.

PRIORITIZATION PROCEDURE

Identification of bridges likely to sustain damage during an earthquake was an essential first step in the Bridge Seismic Retrofit Program. Damage analysis after the San Fernando earthquake led the bridge engineers to the conclusion that unconnected joints at hinges, bents and abutments posed the highest threat to collapse. It was also determined that single column supported bents posed the next highest threat. These judgements have been confirmed by performance of both retrofitted and non-retrofitted bridges in several subsequent earthquakes. The overwhelming evidence from Loma Prieta supports the priority of retrofit that had been established in 1973 after analysis of bridge damage caused by the San Fernando earthquake.

After the hinge restrainer phase of the retrofit program was completed, the department began to prioritize the single and multiple supported structures for sequence of upgrading. What can be classified as a level one risk analysis was employed as the framework of the process which led to a consensus list of risk prioritized bridges.

A conventional risk analysis produces a probability of failure or survival. This probability is derived from a relationship between the load and resistance sides of a design equation. Not only is an approximate value for the absolute risk determined, but relative risks can be obtained by comparing determined risks of a number of structures. Such analyses generally require vast collections of data to define statistical distributions for all or at least the most important elements of some form of analysis, design and/or decision equations. The acquisition of this information can be costly if obtainable at all. Basically, what is done is to execute an analysis, evaluate both sides of the relevant design equation, and define and evaluate a failure or survival function. All of the calculations are carried out taking into account the statistical distribution of every equation component designated as a variable throughout the entire procedure.

To avoid such a large time consuming investment in resources and to obtain results which could be applied quickly as part of the Single Column Phase of the Retrofit Program, an alternative was recognized.

What can be called a level one risk analysis procedure was used. The difference between a conventional and level one risk analysis is that in a level one analysis judgements take the place of massive data supported statistical distributions.

The level one risk analysis procedure used can be summarized in the following steps:

1. Identify major faults with high event probabilities (priority one faults).

This step was carried out by consulting the California Division of Mines and Geology and recent US Geological Survey studies. A team of seismologists and engineers identified seismic faults believed to be the sources of future significant events. Selection criteria included location, geologic age, time of last displacement (late quarternary and younger), and length of fault (10 km min.). Each fault recognized in step 1 was evaluated for style, length, dip and area of faulting in order to estimate potential earthquake magnitude. Known faults were placed in one of three categories; minor (ignored for the purposes of this project), priority two (mapped and evaluated but unused for this project), or priority one (mapped, evaluated, and recognized as immediately threatening). After identification of single column supported bridges close to priority one faults, all remaining structures were evaluated equally affected by priority one and priority two faults.

2. Develop attenuation relationships at faults identified in step 1.

An average attenuation model was developed by Mualchin of the California Division of Mines and Geology to be used throughout the state. It is the average of several published models. Mualchin, a well known seismologist, is now a member of the Caltrans Office of Earthquake Engineering

3. Define the minimum ground acceleration capable of causing severe damage to bridge structures.

The critical (i.e., damage causing) level of ground acceleration was determined by performing nonlinear analyses on a typical highly susceptible structure (single column connector ramp) under

varying maximum ground acceleration loads. The lowest maximum ground acceleration that demanded the columns to yield (provide a ductility ratio of 1.3) was defined as the critical level of ground acceleration. That level of ground acceleration was 0.5 g. Loma Prieta proved this assumption to be wrong and adjustments have been made in current evaluations. Based on input from our Geologists and the type of material over the bedrock, an acceleration level as low as 0.25 g could be critical.

4. Identify all the bridges within high risk zones defined by the attenuation model of step 2 and the critical acceleration boundary of step 3.

The shortest distance from every bridge in California to the two closest priority one faults was calculated. Each distance was compared to the distance from each respective level of magnitude fault to a 0.5 g decremented acceleration boundary. If the distance from the fault to the bridge was less than the distance from the fault to the 0.5 g boundary, the bridge was determined to lie in the high risk zone and was added to the screening list for prioritization. The prioritization procedure is described below.

The Caltrans Division of Structures has developed a computerized data base which has the coordinates of all 25,000 State, County and City bridges stored. We can produce a map of the entire state or any portion of the state showing the bridges, the major faults and an overlay of the combinations. These maps can be viewed on the computer screen or printed for use by designers in screening to identify high risk bridges. The procedure is quite simple, using the computer data base.

a) Locate all Highway Bridges on the State System
b) Locate all Earthquake Faults
c) Determine those structures that are in a high risk zone.

5. Prioritize the threatened bridges by summing weighted bridge structural and transportation characteristic scores.

This step constitutes the process used to prioritize the bridges within the high risk zones to establish the order of bridges to be investigated for retrofitting. It is in this step that a risk value is assigned to each bridge. A specifically selected subset of bridge structural and transportation characteristics of seismically threatened bridges was drawn from the California Department of Transportation structures computer database. Those characteristics were:

Ground Acceleration
Route Type-Major or Minor
Average Daily Traffic(ADT)
Column Design-Single or Multiple Bents
Confinement Details of Column(relates to age)
Length of Bridge
Skew of Bridge
Availability of Detour

After evaluating the results of the 1989 Loma Prieta earthquake, Caltrans engineers modified the Risk Analysis Algorithm by adjusting the weights of the original characteristics and adding the following new characteristics to the list. A total of 18 different characteristics are now evaluated.

Soil Type
Hinges, Type and Number
Exposure (Combination of Length, Height & ADT)
Abutment Type
Type of Facility Crossed

All of the components listed above were considered in the Caltrans risk evaluation. A team of experts representing hundreds of years of experience in the fields of bridge design, construction and maintenance engineering and geotechnical and geological sciences were employed to identify and weight appropriate risk components. Normalized preweight characteristic scores from 0.0 to 1.0 were assigned based on the information stored in the database for each bridge. Scores close to 1.0 represent "high risk structural" characteristics or "high cost of loss" transportation characteristics. The preweight scores were multiplied by prioritization weights. Post-weight scores were summed to produce the assigned prioritization risk value.

In summary, an evaluated risk number is calculated respecting source, distance, local geologic site conditions, bridge structural components and what is at risk in addition to the bridge.

Determined risk values are not to be considered exact. Due to the approximations inherent in the judgements adopted, the risks are no more accurate than the judgements themselves. The exact risk is not important. Prioritization list qualification is determined by fault proximity and empirical attenuation data and not so much judgement. Therefore, a relatively high level of confidence is associated with the risk ranking and identified bridges on the initial list of threatened bridges. Relative risk is then used to establish the order of bridges to be investigated in detail for possible need of retrofit by the designers. The risk analysis offers consistency in applying the judgements adopted to all bridges in the state.

A number of assumptions were made in the process of developing the prioritized list of seismically threatened bridges. This is typical of most engineering projects. These assumptions are based on what is believed to be the best engineering judgement available. It seems reasonable to pursue verification of these assumptions some time in the future to insure that we have not missed anything. Two steps seem obvious: (1) monitoring the results of the design engineer's retrofit analyses, and (2) executing a higher level risk analysis where necessary and better data are available.

Important features of the prioritization procedure are the ease and minimal cost with which it was carried out and the database, highlighting bridge characteristics, to identify structures in need of retrofit. This database will serve as part of the statistical support for the current risk analysis and of any future conventional risk analysis. The additional accuracy inherent in a higher order risk analysis will serve to verify previous assumptions, provide very good approximations of actual structural risk, and develop or evaluate postulated scenarios for emergency responses. It is reasonable to analyze only selected structures at this level. A manual screening process was used here which included review of "AS-Built" plans by three engineers to identify bridges with column and footing details that appeared to need upgrading.

DESIGN

The California Department of Transportation implemented the single column phase in 1986 and the multiple column phase (covering all

remaining bridges) of the Bridge Seismic Retrofit Program in 1990, in which entire structures may be subject to modification to reduce the likelihood of catastrophic failure during a large earthquake. The main goal of the program is to prevent collapse, but a secondary goal is to increase serviceability by reducing damage to a level that can be repaired without closing the structure to traffic. This is a goal that may have to be modified as cost- benefit considerations are evaluated. Special attention is being focused on overall structure response. Two key items are recognized as being primary in achieving this goal: (1) providing continuity in superstructures at joints to prevent supported elements from collapsing, and (2) increasing ductility at certain locations throughout the structure and specifically in columns. The key to the column retrofit phase is ductility provided by the supplemental external confinement and improvements to foundations and abutments. Currently, that external confinement consists of steel shells designed to resist the column shear forces and provide confinement of main column reinforcing at the plastic hinge location. Other techniques for wrapping with cables or fiber reinforcing are also being tested. Foundations and abutments are being reinforced with additional piles, confinement reinforcement, soil anchors, pile shaft retainers, bolster walls and other details to improve seismic resistance.

Design engineers have been assigned the final task of verifying or discrediting the prioritized bridges' need for retrofitting and then, if necessary, developing retrofit contract plans. Verification of the need for retrofit is necessary due to the possibility of prioritized bridges already being capable of withstanding the maximum credible earthquake. This can only be determined by additional analysis and will be the case when judgements made in the prioritization process prove to be too conservative. Emphasis is being placed on evaluation of the total bridge during this design phase. In most cases a dynamic analysis is necessary before the final decision to retrofit or not can be made.

A number of structure modification schemes are under consideration in the bridge seismic retrofit program (since the Loma Prieta earthquake the Single and Multiple Column phases of the program have been combined with the old superstructure phase into the single, "Bridge Seismic Retrofit Program with no phases). Restrainer and superstructure retrofit techniques which have proven to be successful during recent earthquakes will continue to be used to effectively force superstructures to act more like single units. The problems associated with preventing the type of substructure failures seen at San Fernando, Whittier and Loma Prieta are complicated. If all columns are made to carry earthquake loads, then so must the footings and the pile groups. This is not an acceptable solution economically. Some columns may be allowed to pin at a point of contraflexure under dynamic loading while selected retrofitted columns and the abutments absorb the seismic energy and continue to hold the damaged bridge up, preventing bridge collapse. Substructure modifications are currently being evaluated by researchers at the University of California, Berkeley and San Diego. The conclusions drawn from their work will be used to standardize retrofit design schemes.

RETROFIT DETAILS

Some typical superstructure retrofit methods used to date have been to add restraining cables or rods at piers and hinges and add shear keys at abutments. In some cases new, longer hinge and abutment bearing seats had to be installed. Where this was not practical heavy duty pipe hinge extenders were installed to resist both horizontal and vertical seismic forces. Additionally, these hinge extenders carry the supported portion of a bridge in the unlikely event it were to move off the narrow hinge bearing seat. The continuing Legislatively mandated Bridge Seismic Retrofit Program will include this type of retrofitting on all state and other publicly owned bridges (i.e. County, City, Transit Systems, Other State agencies).

Work is beginning on the multiple column structure retrofit as research results become available. The total program will consist of retrofitting approximately 4500 bridges on the state system and 1500 bridges on the local city and county systems. During this phase we will go back and look at all phase I bridges again to insure that none are missed by the initial screening processes.

RESEARCH AND PROOF TESTING

Work at the University of California at San Diego was funded in 1987 and consisted of half scale model testing of the various single column bent retrofit techniques. Theoretical calculations and research work previously conducted in New Zealand by Doctor Nigel Priestley showed that enclosing the columns in steel casings could significantly increase their shear strength and ductility by providing the additional confinement at the hinge areas. A series of tests have been completed on round columns with outstanding results. Based on this work, the first contracts for bridge column retrofit were advertised in January, 1990 and work is underway in the Los Angeles area. A second series of tests was begun in February, 1990 on rectangular single column bents and the results will be available this summer. Both series of tests include models of the prototype columns with the pre-1971 reinforcing details without retrofitting, retrofitted columns using the steel shell confinement and a post damage retrofitted column using the steel shell to determine whether a non-retrofitted damaged column can be salvaged after an earthquake. Typical displacement ductility factors on retrofitted undamaged columns are 6 to 8. On the post damage retrofitted column a ductility factor of 2 was achieved. Even though displacement ductility factors of 6 to 8 have been common in these first tests, our analysis procedure is based on moment ductility demand no greater than 4. Future tests are scheduled to be conducted on columns retrofitted by wrapping pre-stressing strand and fiber reinforced sheet wrapping similar to the technique used on large concrete tanks and industrial smoke stacks.

Work at the University of California at Berkeley is funded and was started in November, 1990 to test retrofit techniques on multiple column bents. These substructure configurations are more complex and difficult to retrofit but on the other hand they have not demonstrated as high a risk as do single columns. During the aftermath of the Cypress Street Viaduct cleanup efforts a three span segment of the standing portion of the viaduct was instrumented and tested by the University of California researchers to determine its fundamental period. Column jacketing retrofit techniques proposed for use on the double deck viaducts in San Francisco were tested to prove their theoretically calculated value and actual reliability in increasing column ductility and shear capacity. This was a unique opportunity to test full scale structure frames to yield, apply several retrofitting techniques and retest the upgraded structure. The results have been published by the University and offer some degree of confidence for use in very specific applications.

SUMMARY

The procedure used by the California Department of Transportation to identify and prioritize seismically threatened bridges to be investigated for possible retrofit has been presented. The original decision to retrofit deck joints first and columns later was based on experience from the 1971 San Fernando earthquake. Subsequent earthquakes, including the recent Loma Prieta event, have proved the validity of this decision. Several hundred bridges with only the deck joint restrainers in place have performed well during these earthquakes. The level one risk analysis used to prioritize the structures in the single column phase was discussed in which decisions were made based on reasonable judgement instead of massive statistical data. The level one risk analysis offers a procedure to consistently apply knowledge gained from past earthquakes and known characteristics of bridges throughout the state which can be carried out quickly without developing a large, more sophisticated statistical database. Steps will be taken to verify assumptions made in the risk analysis in an attempt to improve confidence in this analysis. A modification of this risk analysis procedure will be used to prioritize the more complex bridges remaining in the program as the candidate list is screened. Research on column retrofit techniques will be continued to refine and improve those techniques. Research on the effects of soft foundation materials and the effects of variable foundation material response on long structures will also be continued.

Finally, the legislative direction and funding is being made available to accelerate the California Bridge Seismic Retrofit program for all publicly owned bridges which require upgrading to modern seismic standards.

Seismic Design Criteria for Highway Bridges

IAN G. BUCKLE

The catastrophic collapse of the Cypress Street Viaduct during the Loma Prieta earthquake of October 17, 1989 and the loss of spans in both the Struve Slough Bridge and the San Francisco-Oakland Bay Bridge has emphasized, very forcibly, the importance of rigorous seismic design procedures for highway bridges. More than eighty bridges were damaged in this moderate earthquake; some were old, some were new, some were on soft soil and some on rock. Damage patterns confirm the seismic vulnerability of older structures on poor ground, but equally disturbing is the damage sustained by some new structures designed to so-called "modern" codes.

This paper first, reviews common bridge failures during earthquakes. It then summarizes the present situation regarding seismic codes for bridges in the United States, and finally it speculates on future developments. It is concluded that existing codes should be reexamined in the light of recent events and in the process, the following issues should be addressed: the design loads (acceleration coefficients, seismic hazard maps, and soil types); the response modification factors (redundancy and ductility); the seismic performance categories (importance issues); methods of analysis (curved bridges, single vs multimode methods, inelastic methods); seatwidths and design methods for reinforced concrete joints.

BRIDGE DAMAGE DUE TO EARTHQUAKES

Earthquakes damage civil structures every year and bridges are no exception. Historically, bridges have proven to be vulnerable to earthquakes, sustaining damage to substructures and foundations and in some cases being totally destroyed as superstructures collapse from their supporting elements. In 1964 nearly every bridge along the partially completed Copper River Highway in Alaska was seriously damaged or destroyed. Seven years later, the San Fernando earthquake damaged more than 60 bridges on the Golden State Freeway in California. This 1971 earthquake is estimated to have cost the State approximately $100 million to repair and replace these bridges, including the indirect costs due to bridge closures. In 1989, the Loma Prieta earthquake in Northern California damaged more than 80 bridges in a five-county region, and caused the deaths of more than 40 people in bridge-related collapses alone. The cost to repair these structures has been estimated at between $1.8 and $2.0 billion; a figure which also includes the cost of completing the California Department of Transportation (Caltrans) bridge retrofit program but excludes the societal costs of bridge closures [1].

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Common Bridge Failures

Earthquake damage to bridge structures may occur in the superstructure, the substructure or the approaches. Typical types of damage are summarized in the subsections below. Connection failures are the most common type of damage and these may take several forms. They include, for example, the failure of bearings and expansion joints which connect the superstructure to the substructure and those shear or flexural failures that occur within the substructure itself (e.g. in the column/footing joint or the column/capbeam joint). Connection failures have been identified as the principal reasons for the collapse of the Cypress Street Viaduct in Oakland and the link span of the San Francisco-Oakland Bay Bridge during the Loma Prieta earthquake (Figure 1).

Superstructures

Since earthquake loads are predominantly horizontal in-plane loads, and because bridge superstructures are inherently very strong in-their-own-plane, earthquake related structural damage to a bridge superstructure is very rare. On the other hand, loss of support due to gross horizontal movement of one or more segments of a superstructure is very common and this may lead to the partial or total collapse of one or more spans of the bridge.

Loss of support may be caused by a connection failure, a lack of continuity in the superstructure, inadequate support lengths for the girders, skew supports

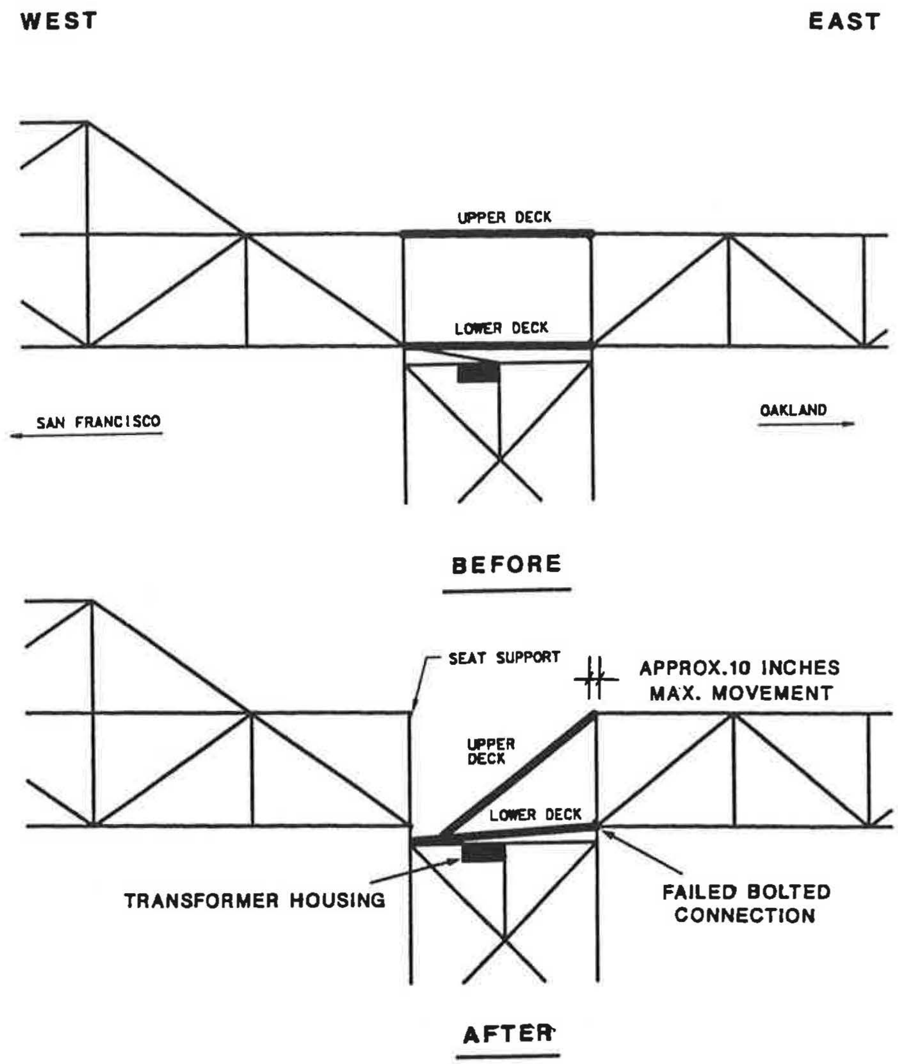

Figure 1. The link span at Bent E-9 in the San Francisco-Oakland Bay Bridge collapsed because of a failed bolted connection of the eastern truss to the Bent
Source: Loma Prieta Earthquake Reconnaissance Report Supplement to Vol.6, Earthquake Spectra, May 1990 (EERI)

(which encourage rotation of the superstructure about a vertical axis), or gross movements at the superstructure supports due to some form of soil failure under the piers or abutments. Reduction of this type of failure has been the principal aim of Phase I of the California seismic retrofit program (the cable restrainer program).

Substructures

Substructure damage includes the structural failure of the columns, abutments and foundations (piles, footings). Column damage can be caused by flexural failure, shear failure, and anchorage failure of the longitudinal reinforcement. All of these types of failure modes may also cause collapse of the superstructure by removal of support for the superstructure.

Foundations

foundation failures may result from excessive ground deformation and/or the loss of stability and bearing capacity of the foundation soils. As a result, substructures may tilt, settle, slide, or even overturn, thus experiencing severe distortion or complete failure. It is also well established that soft soils can amplify bedrock ground motion and that relatively harmless rock accelerations can be intensified by overlying soil to become destructive events at the surface. Deep deposits of both natural and manmade fill are potential sites for soil amplification effects. Even if piles are used to support a bridge through soft material to bedrock (or similar) large horizontal movements may still occur in the superstructure because of the flexible nature of the soil/pile/substructure system.

Abutments

By virtue of their high lateral stiffness, abutments may attract the largest share of the seismic inertia forces developed in the superstructure. These forces can be very high and may cause severe failures, often of a brittle nature. The interaction of the abutment with the backfill may also cause the wing walls to separate from the abutments. Backfill settlement resulting from compaction is often observed.

Vulnerable Bridges

Two factors determine bridge vulnerability to seismic effects: first, their ability to resist earthquake forces; and second, their ability to tolerate large superstructure movements. Earthquake forces are generally higher in bridges supported on stiff substructures (i.e. in short period bridges) and deflections are usually larger in the heavier decks on flexible substructures (i.e. in long period bridges).

Bridges with the greatest vulnerability for seismic damage are therefore multi-span structures that have one or more of the following

: simply supported spans which have deficient bearings and inadequate seatwidths

: continuous spans which have intermediate hinges with deficient bearings and inadequate seatwidths

: nonductile substructures

: under-reinforced footings

: under-reinforced abutment backwalls and wingwalls

: unusual geometry (severe curvature, severe skew, tall piers, piers of differing heights, long continuous spans, piers in deep water)

: hazardous site conditions (near active faults, on or near unstable slopes, on liquefiable foundations, on deep soft soil sites).

On the other hand, bridges with the least seismic vulnerability include

: single span bridges with either integral abutments or generous seatwidths and adequate connection details at the abutments

: continuous bridges with either integral abutments or generous seatwidths and adequate connection details at the abutments, that have redundant substructures and no internal hinge seats

: bridges with earthquake protective systems such as base isolation devices which reduce seismic forces and control large superstructure movements.

SEISMIC HAZARD TO BRIDGES

Seismicity of the United States

Although the earthquake "problem" is widely believed to be unique to California, it is in fact a national problem. This was recognized by Congress when Public Law 95-124 was passed in 1977 which identified 39 states as being exposed to a moderate level (and higher) of seismic threat.

Figure 2 shows the location of those earthquakes which have been reported in the United States since early colonial times through to the present day. Most of these are small magnitude events but a few have been very large and if they were to occur today they would be very destructive. The largest earthquakes to have been experienced in the continental U.S. were those in New Madrid on the Mississippi River during the winter of 1811 and 1812. Three magnitude 8 earthquakes occurred in a period of 3 months and were felt throughout the eastern United States. Figure 3 shows the extent of the felt area and compares it to that of similar magnitude events elsewhere in the U.S. It is clear that the affected areas are much larger in the east than in the west due, most probably, to the homogeneous nature of the basement rock in the east. The fact that large magnitude earthquakes have occurred in the East in the past and that they were felt over large areas of the continent, must be kept in mind when deciding seismic criteria for bridges in the East and Central United States.

Seismic Design Codes for Bridges

Two sets of provisions are commonly used for the seismic design of bridges outside of California. One is the AASHTO Standard Specification [2] and the other is the AASHTO Guide Specification for Seismic Design (1983) [3]. The California Department of Transportation uses a hybrid set of specifications, modified for local conditions [4]. The Guide Specifications are identical to the ATC-6 Guidelines prepared under contract to Federal Highway Administration in 1981 and are more rigorous than those provisions in the Standard Specifications.

During the 1990 Annual Meeting of the AASHTO Bridge Subcommittee, it was agreed that the seismic provisions in the Standard Specifications be replaced by those from the Guide Specification. A formal ballot will be made of all members of the Subcommittee in the Fall of 1990 and if successful, the Guide Specifications will become the new national standard for seismic bridge design in the United States.

However, these guidelines are now 10 years old, and it is appropriate that they be reviewed in the light of experience with their use over the last decade and the performance of various bridges in the Loma Prieta earthquake.

Retrofit Guidelines have also been prepared by ATC under FHWA sponsorship [5] but these have not yet been formally adopted by AASHTO as a Guide Specification. FHWA has also funded the publication of a seismic design and retrofit manual [6] and an instructional short course for bridge designers [7] offered through the National Highway Institute.

Seismic Design Philosophy of the Guide Specification

The design earthquake motions and forces specified in the Guide Specification are based on a low probability of their being exceeded during the normal life expectancy of a bridge. Bridges and their components that are designed to resist these forces and that are constructed in accordance with the design details contained in the provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking.

The principles used for the development of the provisions are [3]:

1. Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.
2. Realistic seismic ground motion intensities and forces are used in the design procedures.
3. Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.

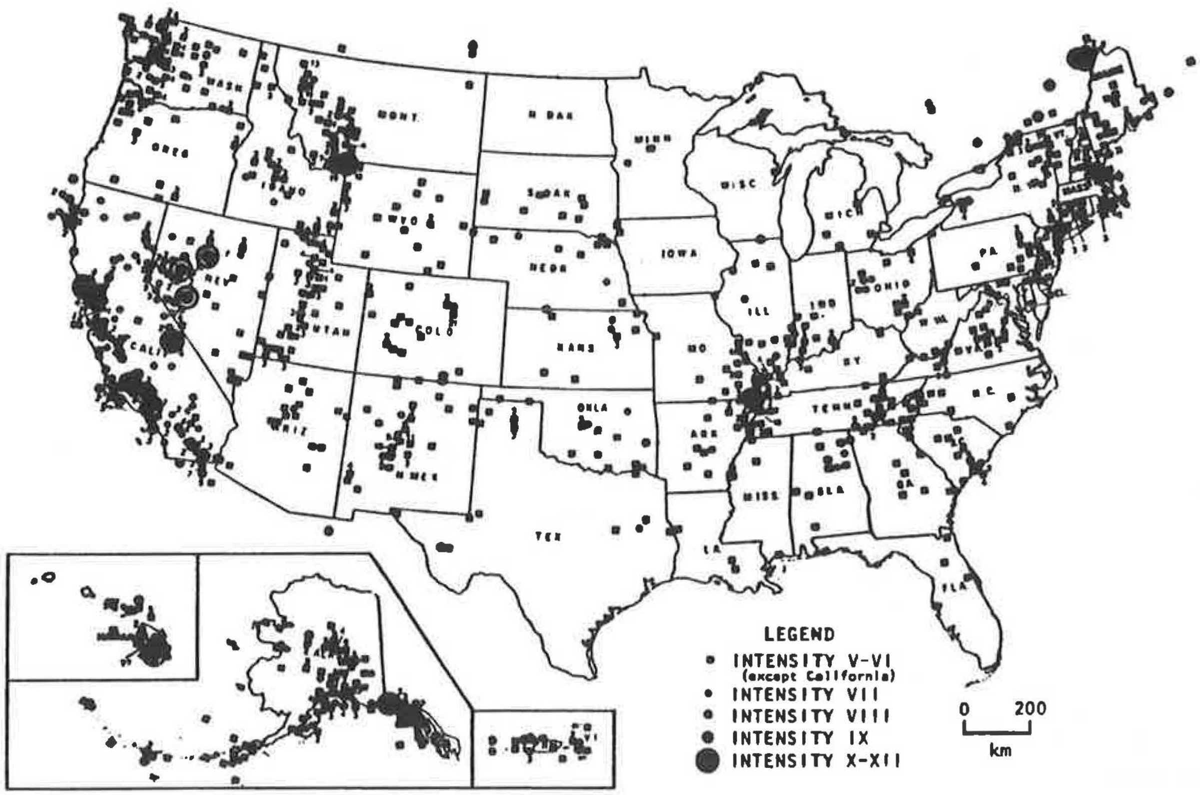

Figure 2. Earthquake with maximum Modified Mercalli intensities of V or above in the United States and Puerto Rico through 1976
Source: "An Introduction to the Seismicity of the United States", by S.T. Algermissen, EERI Monograph Series, 1983

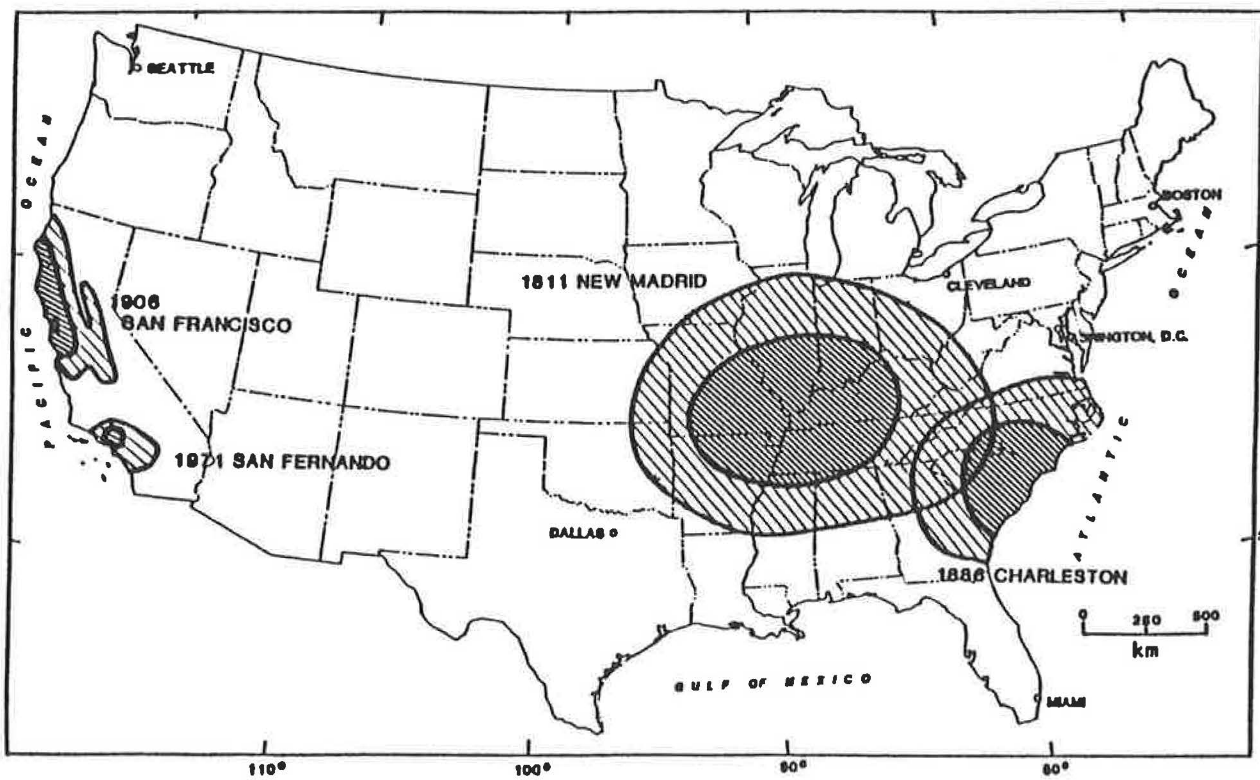

Figure 3. Regional areas affected by major earthquakes in the United States
Source: "Geology in the Siting of Nuclear Power Plants", Hatheway, A.W. and C.R. McClure, Eds., 1979. Reviews in Engineering Geology, Vol. 4, pages 67-94. Geological Society of America. Ed. note: Adapted from Nuttli, O.W. "Seismicity of the Central United States".

A basic premise in developing these seismic design guidelines was that they be applicable to all parts of the United States. The seismic risk varies from very small to high across the country. Therefore, for purposes of design, four Seismic Performance Categories (SPC) are defined on the basis of an acceleration coefficient (A) for the site, determined from the map provided (Figure 4), and the importance classification (IC). Different degrees of complexity and sophistication of seismic analysis and design are specified for each of the four Seismic Performance Categories.

An essential bridge is required to function during and after an earthquake. In areas with an acceleration coefficient greater than 0.29 essential bridges must meet additional requirements. A bridge is designated essential on the basis of social/survival and security/defense classifications presented in the commentary to the provisions [3].

One of the consequences of the 1989 Loma Prieta earthquake in California, has been a call for a reexamination of this philosophy. Accordingly, several efforts are underway at the time of writing (May 1990) including studies by Caltrans, the Governor's Board of Inquiry, the General Accounting Office and the Transportation Research Board through NCHRP Project 12-33.

The TRB/NCHRP activity is part of a much larger exercise sponsored by AASHTO through NCHRP to prepare a comprehensive bridge specification and commentary based on LRFD principles [8]. The Code Coordinating Committee for NCHRP 12-33 has formed an Earthquake Provisions Advisory Group to prepare the seismic provisions for the code. This paper summarizes the current thinking (as of May 1990) of this Group regarding some of the issues and concerns that need attention.

ISSUES AND CONCERNS

Loads: Acceleration Coefficient

A new map for the acceleration coefficient (A) has been prepared for the NEHRP'88 provisions for buildings [9] by the US Geological Survey. Higher acceleration coefficients are recommended in all parts of the United States (for the same probability of exceedance as previously used) as shown in Figure 5.

At the 1990 AASHTO Annual Meeting of the Bridge Subcommittee, it was agreed to update the map in the Guide Specification (shown in Figure 4) by replacement with this new map (Figure 5). The revised map is much closer to the map used by Caltrans for bridges in California and it also reflects an improved understanding of the seismic risk in the East.

Notwithstanding these steps in the right direction, it is still worthwhile to consider the advantages and disadvantages of a two-level earthquake load requirement. The first level would be the existing design event (Figure 5), and be used for determining design forces; the second level would be the maximum credible event and be used for determining superstructure displacements (e.g. Figure 6). The Guide Specification uses the first level event (the design event). Caltrans uses the second level event (the maximum credible event). It will be recalled that Caltrans does not use either the Standard Specification or the AASHTO Guide Specification for seismic design but has adopted its own set of specifications based on higher input loads and more generous response modification factors.

A two-level earthquake approach could permit the introduction of fully elastic performance under the design event and inelastic response (but not collapse) under the maximum credible event. This may have particular appeal for critically important bridges. In this way it would also be possible to make a rational allowance for the difference in seismicity between the East and West. For example, the maximum credible event in the East is expected to be 3 times larger than the design event in the East, whereas in the West, this ratio is about 1.5. Ensuring satisfactory behavior during the maximum credible event in the East is therefore more difficult than in the West, especially if only a single design event is used.

An alternative procedure to the two-level event is to use the Caltrans' philosophy nationwide (i.e. use the maximum credible event to define the acceleration coefficient). At the same time more generous response modification factors are adopted so that the design forces will be

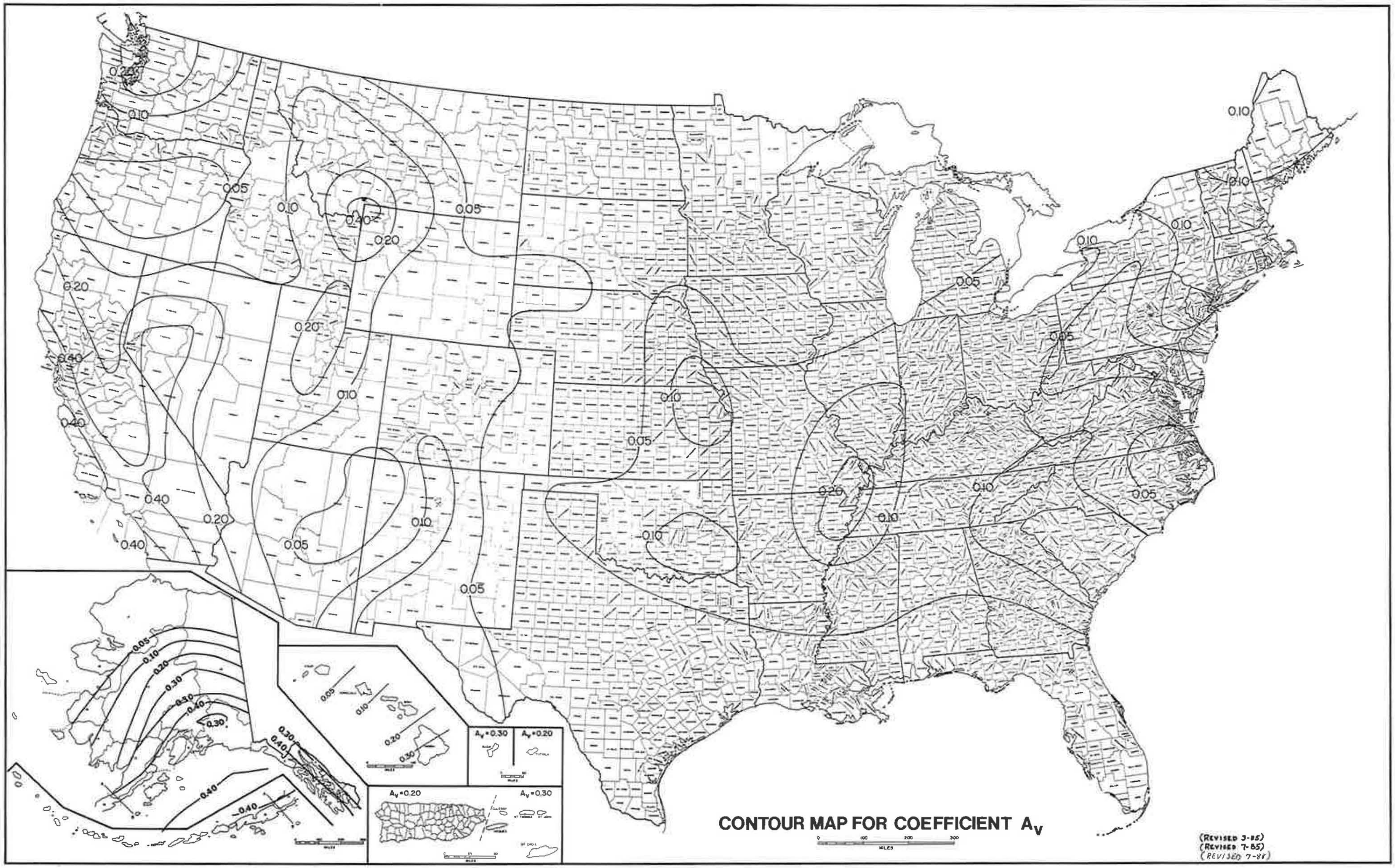

Figure 4. Acceleration Coefficient

Based on 10% probability of exceedance in 50 years.

SOURCE: ATC 6 and AASHTO 1983 Guide Specifications for the Seismic Design of Highway Bridges

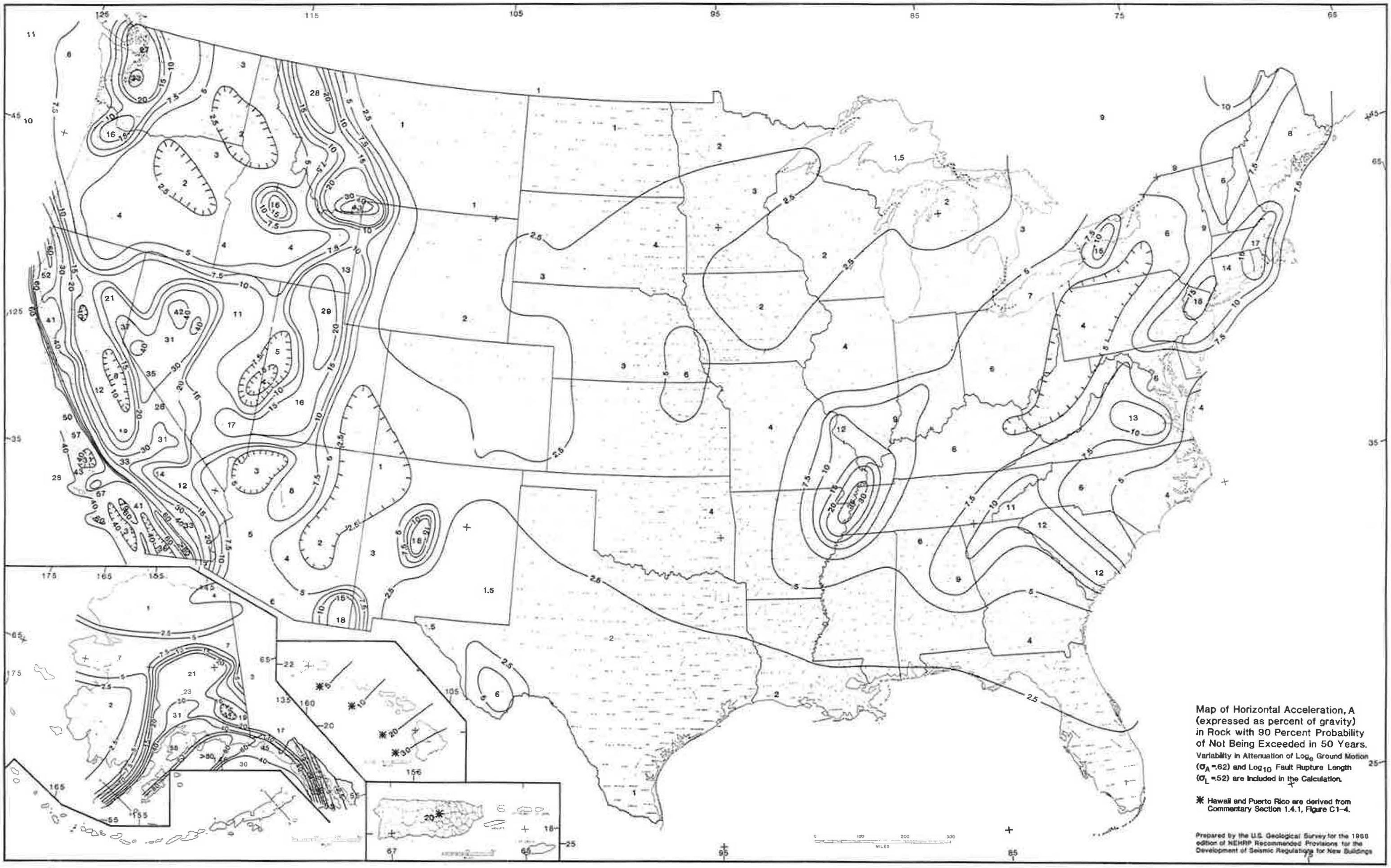

Figure 5. Revised Uniform Seismic Risk Map

Based on 10% probability of exceedance in 50 years.

SOURCE: USGS for 1988 Edition of NEHRP Provisions

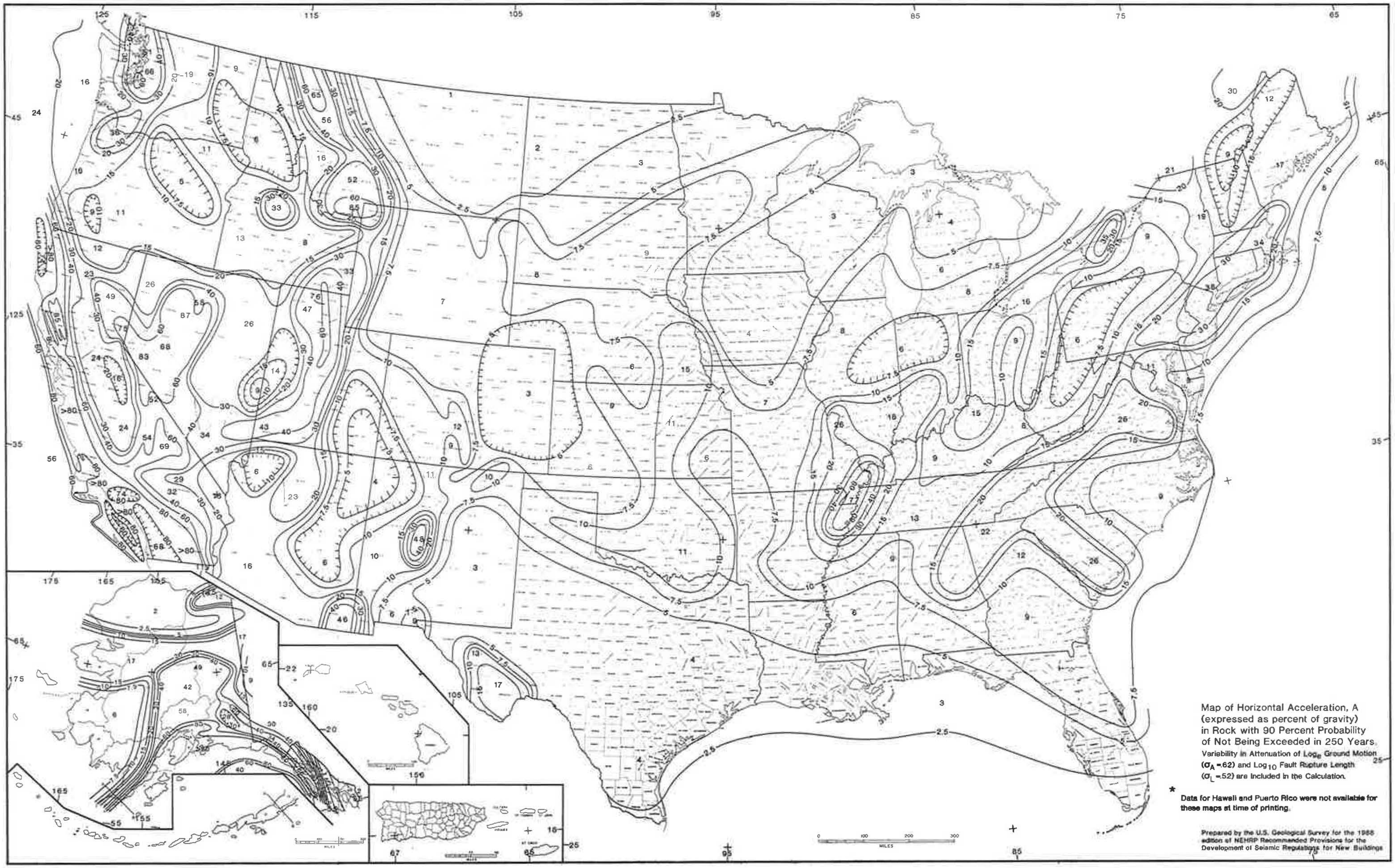

Figure 6. Revised Uniform Seismic Risk Map

Based on 10% probability of exceedance in 250 years.

SOURCE: USGS for 1988 Edition of NEHRP Provisions

about the same. However, the displacements will be significantly larger (2-3 times larger in the East than presently required in the East).

This will lead to greater seatwidth requirements in the East and Central U.S. than presently required. Such a move is necessary in order to prevent loss of girder support in a maximum credible event as required by the stated design philosophy. Seatwidths are further discussed in later section.

Loads: Vertical earthquake loads

Some substructure elements, including bearings, are thought to be susceptible to the vertical component in earthquake ground motion. Provisions that require analysis for these effects may be necessary. Further study is required.

Loads: Soil types and site coefficients

A fourth soil type for particularly soft materials has been adopted in the Uniform Building Code for buildings [10] and should be considered for bridges. The shape of the spectra for softer sites also needs to be defined. Provisions for the use of site-specific spectrum should also be made.

Response Modification Factors (R-Factors)

Present Response Modification Factors (R-factors) make allowance, in a somewhat arbitrary way, for redundancy and ductility in a bridge structure. The designer is permitted to reduce the elastic forces (obtained from a linear elastic analysis) by dividing by R to obtain the member design forces. For a multicolumn bent, R=5 (in the AASHTO Guide Specifications; R=8 for this same bent in the Caltrans Specifications). Because of the high degree of redundancy in a multicolumn bent, the inherent ductility in a modern, confined, reinforced concrete column and because the risk of total collapse is low, this factor is relatively high. Single column bents which are not redundant have R-factors as low as 3. Obviously, a great deal of professional judgement is required to select appropriate values for R. Note that the displacements of the superstructure are not reduced by R to obtain the design displacements. (Alternatively, displacements may be calculated from inelastic time-history analyses).

There is some merit in clearly separating the two components that comprise the R-factor, particularly as research has now progressed to the point where analytical and experimental methods are available to give specific values for member deformation capacity and demand, leading to an identifiable ductility component in the R-factor.

If a numerical measure of redundancy could also be defined, it would help the designer to see more clearly the risk he/she is assuming and the benefits of redundancy in bridge substructures.

Since less risk should be taken for important structures, such a step would also lead to an improvement in the manner "importance" is included in the design process (see section below).

Seismic Performance Categories (SPC)

Importance is used, along with the acceleration coefficient (A), to define 4 seismic performance categories in the Guide Specifications [3] as shown in Table 1.

It is seen in this Table that importance affects only those bridges in locations where A>.29 . Further, the only difference in the design requirements between SPC C and D is in the section on foundations and abutments. In general, more rigorous site inspections are required, soil degradation must be considered, and settlement or approach slabs must be provided at the abutments. All other design requirements are the same as for nonessential bridges (in the same seismic zone).

Recent experience with the Bay Bridge and the Cypress Street Viaduct in the San Francisco Bay Area would indicate that this treatment of importance is inadequate. It is therefore suggested that the "importance" requirements be moved and clearly identified in the R-factor Table and that SPC D be eliminated.

For example, for all elements in a critically essential bridge R should be essentially unity; for most other bridges the present R-factors may be adequate; for bridges in between these two extremes, the

TABLE 1: SEISMIC PERFORMANCE CATEGORIES (GUIDE SPECIFICATIONS)

Acceleration Coefficient	Importance	
	Essential	Other
< .09	A	A
< .19	B	B
< .29	C	C
> .29	D	C

TABLE 2: SELECTED R - FACTORS (PROPOSED)

Substructure	Importance		
	Other	Essential	Critical
Multicolumn bent	5	3	1.5
Single column bent	3	2	1.5
Wall pier	2	1.5	1.5

TABLE 3: METHODS OF ANALYSIS (GUIDE SPECIFICATIONS)

Seismic Performance Category	Geometry	
	Regular	Irregular
A	*	*
B	SM	SM
C	SM	MM
D	SM	MM

* = no seismic analysis is required regardless of geometry
SM = single-mode method
MM = multi-mode method

TABLE 4: METHODS OF ANALYSIS FOR MULTISPAN STRUCTURES (PROPOSED)

Seismic Zone	Importance					
	Other		Essential		Critical	
	regular	irregular	regular	irregular	regular	irregular
1	*	SMM	*	MM	MM	MM
2	SMM	SMM	SMM	MM	MM	IA
3	SMM	MM	MM	MM	MM	IA
4	SMM	MM	MM	MM	IA	IA

* = no seismic analysis required
SMM = single-mode method (modified)
MM = multi-mode method
IA = inelastic analysis

R-factors might be about one-half of the present values. Table 2 illustrates this thought process. Of course if an inelastic method of analysis is used, the assignment of an R value is no longer necessary and for critically important structures this type of analysis should be encouraged. See section on inelastic methods.

Note that if this concept is implemented, the term "Seismic Performance Category" has no useful purpose and could be removed from the jargon. Analysis and design requirements could be specified by seismic zone (for example: 1-4).

Definitions for importance will vary but factors influencing the importance classification of a bridge will include average daily traffic volume, available detour, use by emergency vehicles, replacement cost, and the nature and importance of the route being crossed by the bridge. It is possible that this classification will change with time and if it should increase (in importance), retrofit will be necessary. The owner should be involved in selecting the importance category.

Methods of Analysis

Curved bridges

All curved bridges should be analyzed by the multi-mode spectral analysis method but the definition of "curved" needs to be resolved. At present any bridge with a subtended angle (at the center of the arc) less than 90° may be analyzed by the single-mode method. This is too generous and leads to unconservative results. It is suggested that a "curved" bridge be defined as one for which the subtended angle is greater than or equal to 50°.

Single vs Multi-Mode Spectral analysis

The AASHTO Guide Specifications determine which method of analysis shall be used based on geometry, as in Table 3.

An irregular bridge is one for which there is an abrupt change in stiffness or mass from one bent to another and/or excessive curvature such that the subtended angle is greater than 90°. All other bridges are considered "regular".

The single-mode method can be performed manually for a simple structure, such as a uniform 2-span bridge. However, for all other bridge types, a hand solution is impractical and a computer-based solution becomes a necessity. The particular advantage of this simplified method is therefore lost and the retention of the single-mode method is consequently in doubt. If deleted, the multi-mode method could be used wherever the single-mode method was previously specified. However, there are good practical reasons for permitting a single-mode type of analysis for regular bridges in low seismic zones. In these cases, a "user friendly" version of the single-mode method would be particularly useful and should be investigated further. There is also a good argument to be made for requiring inelastic analysis for important, irregular bridges. Given these changes the table of recommended methods of analysis might be as shown in Table 4.

Inelastic Methods

Certain classes of irregular bridges in high seismic zones (especially those that are absolutely essential or monumental in size or both) should be analyzed using an inelastic, time history method of analysis. (If elastic methods are used, the R-factor should be, say, 1.5 for essentially elastic behavior, see Section on seismic performance categories.)

Bridges with intermediate hinges, soil nonlinearities, energy absorbers and/or base isolators are also candidates for inelastic analysis.

The method of analysis remains to be decided, and also the number and type of time histories (spectrum compatible or actual records?)

Special Requirements for the Various SPCs

Some changes are thought to be necessary to the various Special Requirements (for Analysis and Design) in the four Seismic Performance Categories.

These are summarized as follows:

General: No seismic analysis is required for any single span bridge regardless of seismic zone.

However minimum seatwidth requirements must be satisfied and all connections must be designed for a lateral force = A x dead load.

SPC A — No analysis is required for regular bridges but minimum seatwidths must be satisfied and connection forces must be 0.2 x dead load (except single span bridges).

SPC B — Analysis is required; minimum seatwidths must be satisfied. Member design forces are calculated from elastic-analysis values divided by the R-factor. For foundation design forces, use R/2 values. No column hinging analysis is required. (To avoid undue conservatism here, the development of an approximate column-hinging analysis method would be a useful design aid.)

SPC C — Analysis is required; minimum seatwidths must be satisfied. Member design forces are calculated from elastic analysis values divided by the R-factor for the substructure. Column hinging analysis is required; foundations must be designed for either the maximum shear corresponding to column hinging or the unreduced (R=1) elastic shear from the analysis whichever is the smaller.

SPC D — As for SPC C, with additional requirements for foundations and abutments as presently required in the Guide Specifications.

Seatwidths

The seat width requirements for SPC A and B should be reconsidered. In regions of the US where these categories apply, the maximum credible earthquake is of the same order of magnitude as the design earthquake in SPC C and D. It follows therefore larger seatwidths than currently specified may be necessary to prevent girders being unseated in an event larger than the design event and to satisfy the stated design philosophy as expressed in Section on seismic design philosophy. Attention must also be given to hinge seatwidths in both skewed and curved bridges.

Joint Shear

The shear reinforcement in the reinforced concrete joints of multicolumn bents needs to be reexamined. Requirements for joint steel vary from code to code, and from country to country. Damage sustained during the Loma Prieta earthquake have highlighted the need to change the design provisions for concrete joints, especially knee joints.

SUMMARY

This paper has identified some of the issues needing review in the current codes for seismic design of bridges. They include the design loads, the response modification factors, the seismic performance categories, methods of analysis, seatwidths and design methods for reinforced concrete joints. Current efforts are addressing these issues, with the expectation that improved seismic performance of bridges will be the end result.

ACKNOWLEDGEMENTS

Many of the issues discussed in this paper were identified by the Earthquake Advisory Group for NCHRP Project 12-33. The contribution of this group to the content of this paper is hereby acknowledged. The present membership of this group is as follows: Ian G. Buckle (chair), Robert C. Cassano, James Cooper, James H. Gates, Roy Imbsen and Frieder Seible.

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Seismic Retrofit of Bridge Columns by Steel Jacketing

Y. H. Chai, M. J. Nigel Priestley, and Frieder Seible

Inadequate flexural strength and ductility of shear strength of concrete bridge columns has resulted in collapse or severe damage of a number of California bridges in recent moderate earthquakes. In general these bridges were designed prior to the new seismic design methods which were implemented in the mid-seventies. Bridges constructed in accordance with the new design methods have performed well in recent earthquakes. However, the large number of older bridges that are in service, particularly freeway overpasses designed and constructed in the 1950's to 1970's, are now recognized to have substandard design details and is presenting a cause for major concern. This paper reports the results of a theoretical and experimental program investigating retrofit techniques for circular columns by encasing the critical regions with a steel jacket. The jacket is bonded to the column using grout. Results from six large-scale column tests show that the casing acts efficiently as confinement reinforcement enabling displacement ductility factor of greater than 6 to be achieved. The casing also inhibits bond failures at the laps of longitudinal reinforcement in the critical regions of the column by restraining the dilation and spalling of the cover concrete which degenerates into bond failure. Comparisons of 'as-built' and retrofitted columns are presented, and experimental strengths and ductilities are compared with analytical predictions.

INTRODUCTION

The 1971 San Fernando Earthquake caused substantial damage to a number of recently completed bridge structures and forced a reassessment of the design philosophy for bridges. Research was undertaken both in the U.S. and overseas to improve on the analytical techniques and to provide basic data on both the strength and deformation characteristics of lateral load resisting mechanism in bridges. In the U.S., research emphasis was primarily directed towards development of sophisticated time-history analysis techniques for bridges. Experimental research was mainly pursued as a means of verifying the analytical techniques.

Parallel to the analytical development in the U.S., a comprehensive research program pertaining to the strength and ductility of bridge columns was carried out at the University of Canterbury, New Zealand, under the sponsorship of the New Zealand National Roads Board. The research program produced detailed information on the flexural strength and ductility, and on the shear strength, of both reinforced concrete columns and steel-encased concrete piles. Particular emphasis was placed on quantifying the influence and effectiveness of lateral confining steel in the plastic hinge region of the column to increase ductility.

While basic research was being carried out, the California Department of Transportation (CalTrans) was making an initial impact on the difficult problem of improving the safety of older bridges. Although column failure was recognized as a major problem, the greatest risk was assessed to be due to inadequate connection between adjacent spans of the superstructure across movement joints. Consequently, a major retrofit program was undertaken by CalTrans to install restrainers across movement joints to reduce the risk of span collapse when excessive relative movement occurs. This retrofit program has recently been completed.

The recent shear failure in the columns of the I-5/I-605 Separator (a major freeway overpass) during the Whittier Earthquake of October 1, 1987 [1] and the tragic collapse of the Cypress Viaduct, and other bridge failures, during the Loma Prieta Earthquake of October 17, 1989 re-emphasized the inadequacies of the pre-1971 design and the urgent need in upgrading the seismic resistance of older bridge substructure.

The structural inadequacies inherent in many of the older bridge columns can be categorized as follows:

- **Inadequate Flexural Strength**
  Lateral force coefficients for seismic design were typically less than 10% in pre-1971 designs and are comparatively low by the current standard. Although the use of elastic design generally resulted in the actual flexural strength being significantly higher than that required by the assumed lateral load, low lateral flexural design strength results in high potential ductility demand in many cases.

- **Inadequate Flexural Ductility**
  Bridge columns designed before the 1971 San Fernando Earthquake typically contain insufficient transverse reinforcement. A common provision for both circular and rectangular columns involved the use of #4 (12.7 mm diameter) transverse peripheral hoops placed at 12 inches (305 mm) centers regardless of the column section dimensions. These hoops were often closed by lap splices in the cover concrete, instead of being lap welded or anchored by bending back into the core concrete. As a result, the ultimate curvature developed within the potential plastic hinge region is limited by the strain at which the cover concrete begins to spall which is typically in the range of 0.005 strain. At higher longitudinal strains the hoop steel unravels and the meager amount of confinement provided by the hoops becomes ineffective.

- **Undependable Flexural Capacity**
  In many of the tall bridges designed using the pre-1971 guideline, the column longitudinal reinforcement was spliced with starter bars extending from the footing with a lap length of 20 times the bar diameter. This lap length is insufficient for developing the yield strength of the longitudinal bars especially when large diameter bars are involved. As a consequence flexural strength degrades rapidly under cyclic loading. Occasionally the column longitudinal reinforcement was extended straight into the footing or pile cap without 90 degree hooks. Such details

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allow pulling out of column reinforcement when subjected to large intensity seismic load reversals [2].

- **Inadequate Shear Strength**
  Conservative flexural design, using elastic methods coupled with less conservative shear strength provisions of the 1950's and 1960's, typically result in actual flexural strength of short columns exceeding their actual shear strength. Inadequate anchorage of the transverse reinforcement in the cover concrete compounded the problem. As a consequence, the probable failure mode for shorter columns involves brittle shear failure with low ductility and energy absorption characteristics.

- **Footing Failures**
  Pile caps and footing in older bridges are often provided with only a horizontal layer of reinforcement in the bottom region of the member. Top steel and shear reinforcement were considered unnecessary and routinely omitted. Such practice may be attributed to the use of elastic design which assumes full gravity load acting during the seismic event while concurrently prescribing unrealistically low values of lateral seismic forces.

- **Joint Failures**
  Joint regions either between column and footing or between column and bent-cap beams are subjected to very high shear stresses during a severe seismic attack. These regions traditionally have not been designed to resist this high level of seismic shear stresses.

Although the above design deficiencies have been rectified in current seismic codes and should no longer affect new bridge design, the condition of many old bridge columns built before the 1970's is a cause for major concern. This paper describes the initial phase of a research program funded by CalTrans, and the Federal Highway Administration on flexural strength and ductility of bridge columns and on developing retrofit techniques for upgrading the seismic performance of existing bridges. Experimental testing is being carried out at the Large-Scale Structural Testing Facility at the University of California, San Diego.

CONFINEMENT BY STEEL JACKETING

Recent research [3] has established that closely spaced lateral confinement reinforcement in the potential plastic hinge regions increases both the compressive strength and the effective ultimate compressive strain in the core concrete. The ultimate compressive strain increases from a value of about 0.005 in unconfined concrete to a value of 0.03 or higher in confined concrete. The increase in ultimate compressive strain significantly enhances the ductility capacity of the concrete section. Provided that the transverse reinforcement is spaced no wider than 6 times the longitudinal bar diameter and is properly anchored by either lap welding or bending back into the core, the ultimate compressive strain in the concrete corresponds to longitudinal strain at fracture of the transverse reinforcement. A method for estimating the ultimate compressive strain of confined concrete by equating the strain energy capacity at fracture of the transverse reinforcement to the additional energy stored by the confined concrete above its unconfined state has been proposed [4]. The enhancement in compressive strength and strain due to confinement is illustrated in Fig.1.

Research results [5] have shown that columns designed with reasonable volumetric ratios of confinement reinforcement ($0.005 \leq \rho_s \leq 0.03$) can develop stable hysteresis loops during inelastic cycling to displacement ductilities exceeding $\mu = 6$. In columns where axial loads are high, significant enhancement in flexural strength can occur. Although it is technically feasible to place external hoops on existing circular columns which would later be lap welded and sprayed with gunite to ensure rigid connection with the existing concrete, the method would be costly and may be aesthetically unacceptable. Confinement by enclosing the potential plastic regions of circular columns with a site-welded cylindrical steel sleeve or jacket would be notably less expensive and contributes minimal visual impact. The jacket is introduced slightly oversize for ease of construction and the gap between the column and jacket is filled with a cement-based grout. The jacket is terminated about 2 inches (50 mm) above the critical section at the column base to avoid additional strength enhancement resulting from end bearing of the sleeve on the footing when in compression. Significant flexural strength enhancement can however be expected since an increase in concrete compressive strength will result from the confining action of the steel jacket. The behavior of jacketed columns is expected to be similar to that of the steel-encased concrete piles which have shown experimentally to possess remarkable ductilities in severe cyclic load tests [6].

The confining action of the steel jacket is illustrated in Fig.2. Under the combined effect of axial compression and column flexure, the compression zone attempts to dilate as the flexural strength of the member is approached. The dilation is restrained by the radial stiffness of the jacket; placing the jacket in circumferential tension and the concrete in radial compression. Ignoring any contribution from existing hoops to the confinement of concrete core, the radial confining stress at yield of the steel casing is given by:

$$f_l' = \frac{2f_{yj}t_j}{D_j - 2t_j} \tag{1}$$

$$= \frac{1}{2} f_{yj}\, \rho_{sj} \tag{2}$$

where

$$\rho_{sj} = \frac{4t_j}{(D_j - 2t_j)} \tag{3}$$

represents the volumetric confinement ratio of steel jacket. The variables f_{yj}, t_j and D_j are the yield strength, thickness and outside diameter of the steel jacket respectively. Hence for a column 60 inches (1524 mm) in diameter and retrofitted with a 0.5 inch (12.7 mm) thick A36 steel jacket (f_{yj} = 36 ksi or 248 MPa), the equivalent volumetric confinement ratio ρ_{sj} would be 0.0328 when 0.5 inch gap is used for grout. Thus the maximum confining stress f_L' due to steel casing would be 590 psi (4.07 MPa). The level of confinement provided by the steel jacket would correspond to the upper

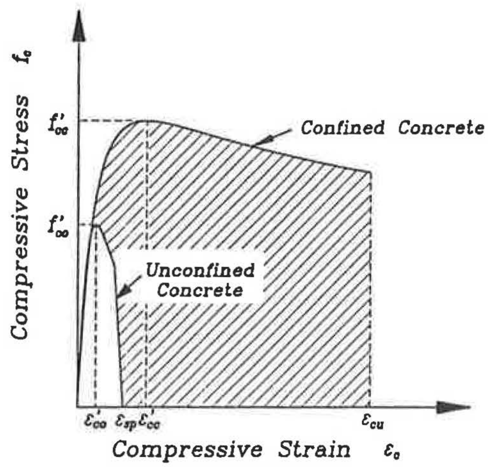

Fig. 1 Stress-Strain Model for Confined Concrete

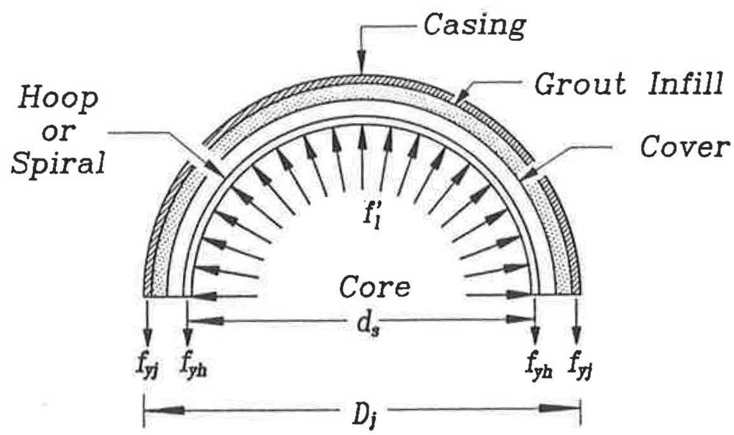

Fig. 2 Confining Action of Steel Casing

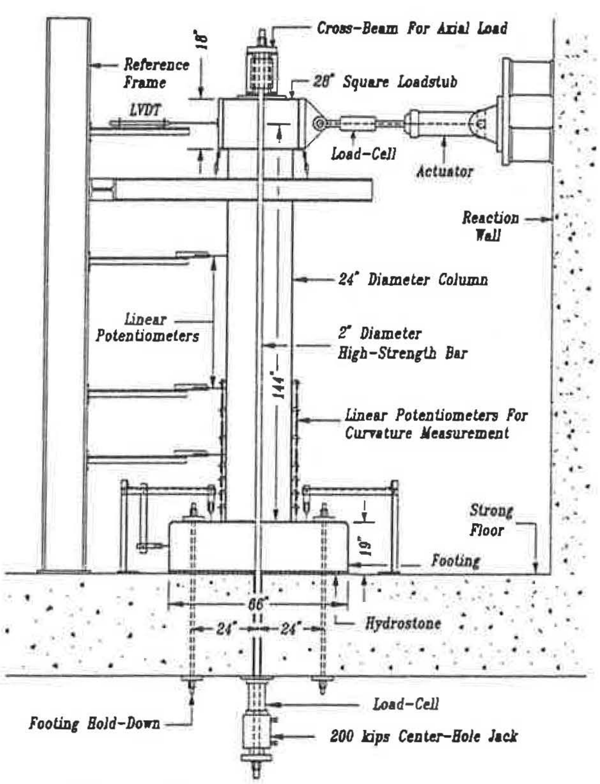

Fig. 3 Experimental Test Setup

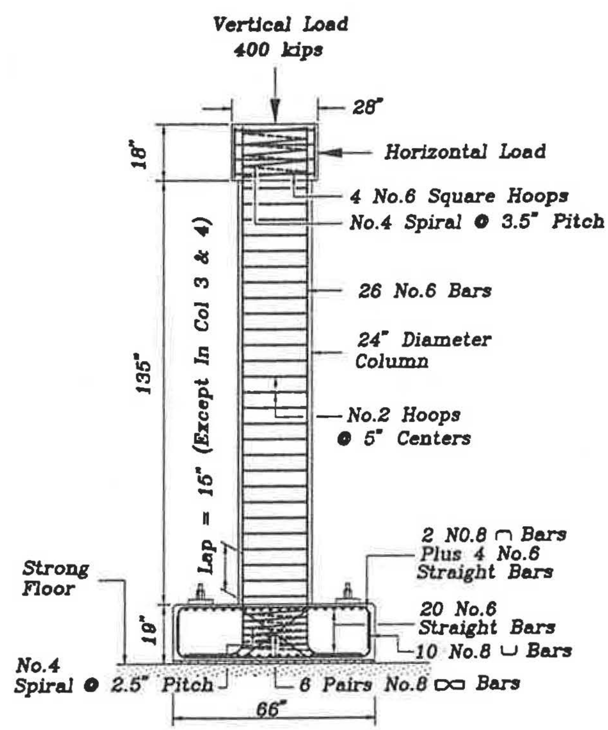

Fig. 4 Typical Reinforcement Details

limit of confinement provided by hoop reinforcement in current column design.

It should be clear that the steel jacket would also be effective in resisting a portion of the total column shear force. Analogous to a circular column having transverse reinforcement of either hoop or spiral, the contribution to shear strength from the steel casing, assuming an equivalent 45 degree truss mechanism, can be shown to be:

$$V_{sj} = \frac{\pi}{2} f_{yj} t_j (D_j - t_j) \quad (4)$$

It may also be expected that the lateral confining pressure from the casing would improve the bond transfer at the lap splices of column longitudinal reinforcement, possibly inhibiting bond failure in the potential plastic hinge region.

CIRCULAR COLUMN TESTS

In order to investigate the expected improved performance of the columns from steel jacketing, six large scale column models of 24 inches (610 mm) diameter and 12 ft. (3.657 m) height were recently tested using the test configuration shown in Fig.3. The columns were considered to be 0.4 scale models of a prototype 60 inches (1524 mm) diameter bridge column. The test columns were constructed with a footing to allow foundation influence or interaction to be monitored. A target concrete compressive strength of f'_c = 5000 psi (34.5 MPa) at 28 days was used to represent a 67% overstrength when compared to the typical 1960's design strength of 3000 psi (20.7 MPa). The overstrength is to reflect both the conservative concrete mix design and batching practices of the 1960's and the strength gain that has occurred in more than twenty years of natural aging. A vertical load of 400 kips was applied to the test column which corresponds to an axial load ratio of $0.18\ f_c A_g$ where Ag denotes the gross sectional area. Even though the axial load could not be kept constant during lateral displacement of the column, the variation of axial load is within ± 17 %.

Longitudinal reinforcement for the column consisted of 26 #6 Grade 40 deformed bars; thus representing a longitudinal steel content of 2.53%. Yield strength for the #6 bar averaged 45.7 ksi (315 MPa). Transverse reinforcement consisted of circular hoops (#2 Grade 40 plain bars) placed at 5 inches (127 mm) centers uniformly up the column. The corresponding confining steel ratio is 0.18%. The hoops were spliced in the cover concrete with a lap length of 12 inches (305 mm). Typical reinforcement details for a test column is shown in Fig.4. The design represented a 60 inches (1524 mm) diameter prototype column reinforced with 32 #14 longitudinal bars and #4 circular hoops at 12 inches (305 mm) centers.

Design variations between columns are summarized in Table 1. Column 1, 2, 5 and 6 were built with lap splices of 20 times the longitudinal bar diameter in the potential plastic hinge region. Column 3 and 4 were reinforced with continuous longitudinal bars which were anchored with 90 degree hooks in the footing. Steel jackets for the columns were fabricated from 3/16 inch (4.76 mm) thick A36 hot-rolled steel. A 1/4 inch (6.35 mm) gap was provided between the column and jacket and was pressure-injected with water/cement grout. Typical compressive strength of 2 inch (51 mm) diameter grout cylinder was between 2000 and 2500 psi (14 and 17 MPa) at an age of 14 days. To ensure that the jacket does not bear against the footing when in compression a vertical gap of 1 inch (25 mm) was provided between the jacket and footing. The length of jacket was chosen to be 48 inches (1219 mm) to ensure that the moment demand immediately above the jacket does not exceed 75% of the uncased flexural capacity. The first pair of columns were constructed with a 1960's footing design using only straight reinforcement (two orthogonal layers of 24 #6 bars each) in the bottom region of the footing. The footing was supported on 1 inch (25.4 mm) high rigid pile-blocks. A strong footing detail was used in the remaining four columns after footing shear failure was noted in column 2. Reinforcement for the strong footing was redesigned to include top and bottom layers of #8 bars bent at both ends, 6 pairs of #8 diagonal bars placed close to the column/footing joint and #4 spiral at 2.5 inch (64 mm) pitch within the joint. Instead of using rigid pile-blocks, the footings were uniformly supported on a thin layer of hydrostone and clamped against the test floor. A partial retrofit approach was undertaken in column 5 to limit the amount of enhancement in flexural capacity which was noted in columns 2, 4 and 6. A thin sheet of Styrofoam (1/4 inch or 6.35 mm thick) was added between the column and the grout infill to allow a controlled dilation of cover concrete at large lateral displacement. Complete loss of cover concrete was prohibited by the presence of steel casing. The program also investigates the possible use of steel jacket for post-earthquake repair of bridge columns. Column 1 was fitted with a steel jacket after initial test (indicated as 1-R in the text matrix) and retested using the same load history. Loose cover concrete around the splice region of the main reinforcement was removed before installing the steel jacket. In order to provide better seal against grouting pressure, the jacket was extended to the top of the footing without any vertical gap. The weak footing was strengthened by external prestressing to a total of 300 kips at mid-height of the footing and in the direction of lateral load. Instead of being supported on pile blocks, the repaired column is placed on uniform bearing similar to the strong footing setup.

Table 1: Test Matrix

Test Unit	Column & Footing Details		Remarks
1	$20d_b$ Lap For Long. Bars Without Steel Casing	Weak Footing	Reference
2	$20d_b$ Lap For Long. Bars With Steel Casing	Weak Footing	Full Retrofit
3	Continuous Column Bars Without Steel Casing	Strong Footing	Reference
4	Continuous Column Bars With Steel Casing	Strong Footing	Full Retrofit
5	$20d_b$ Lap For Long. Bars 1/4" Styrofoam Wrap	Strong Footing	Partial Retrofit
6	$20d_b$ Lap For Long. Bars With Steel Casing	Strong Footing	Full Retrofit
1 - R	$20d_b$ Lap For Long. Bars Repaired By Steel Casing	Weak Footing 300 kips Prestress	Repair

All test columns were subjected to the same lateral displacement pattern of increasing magnitude, as shown in Fig.5 (a). The experimental yield displacement Δ_y was determined by extrapolating a straight line from the origin through ± 40 kips which approximately corresponds to the theoretical first yield of extreme tension steel to the ideal capacity V_i which is calculated using the Mander's model for confined concrete [4]. As shown in Fig.5 (b), the average of the two displacements was adopted as the experimental yield displacement.

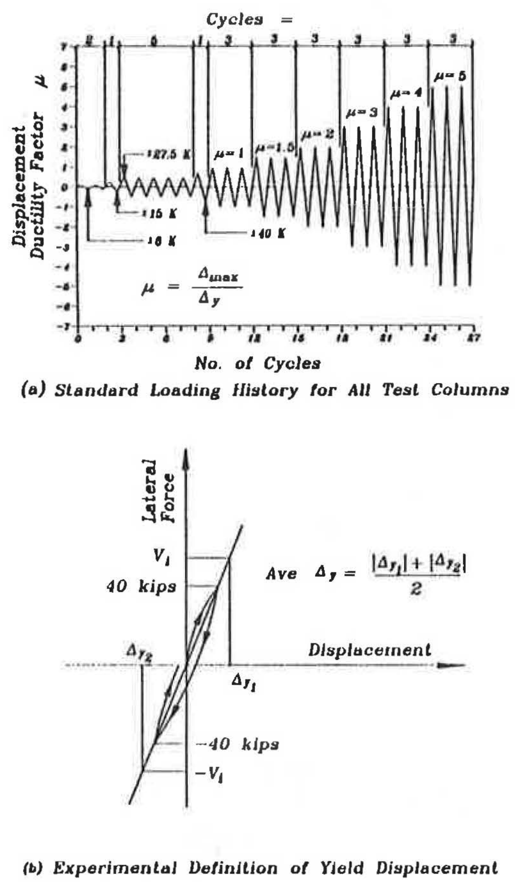

FIGURE 5: Standard Load Pattern for Test Column

RESULTS

Columns with Lapped Starter Bars

Column 1 was observed to suffer early bond failure at the lap of longitudinal reinforcement. Rapid strength degradation occurred after displacement to ductility factor μ=1.5. Maximum lateral load of 49 kips (218 kN) was noted during the push cycle to μ = 1.5 and was 97% of the theoretical ultimate capacity V_u. Significant drop in lateral load to 83% of theoretical capacity occurred in the pull direction of the same cycle. Bond failure in the lap was initiated and caused serious strength.degradation under additional cycling (see Fig.6). The strength envelope is seen to degrade asymptotically after μ = 1.5 to the moment resisted purely by the axial load which is estimated to be 19 kips (85 kN). In comparison, column 2 allowed the theoretical flexural capacity to be achieved. There is a 10% increase in theoretical flexural capacity due to confining pressure from the steel jacket. Lateral stiffness of column 2 shows a 19% increase after retrofit. The lateral stiffness of the column is defined as the theoretical flexural capacity (without strain-hardening) divided by the experimental yield displacement. Hysteretic loops for the column were stable up to μ = 3 when footing failure occurred resulting in rapid drop in vertical load carrying capacity (see Fig.7). There was however no sign of bond failure in the lap splices of longitudinal reinforcement in column 2. Hysteretic loops for the pair of columns are shown in Fig.8.

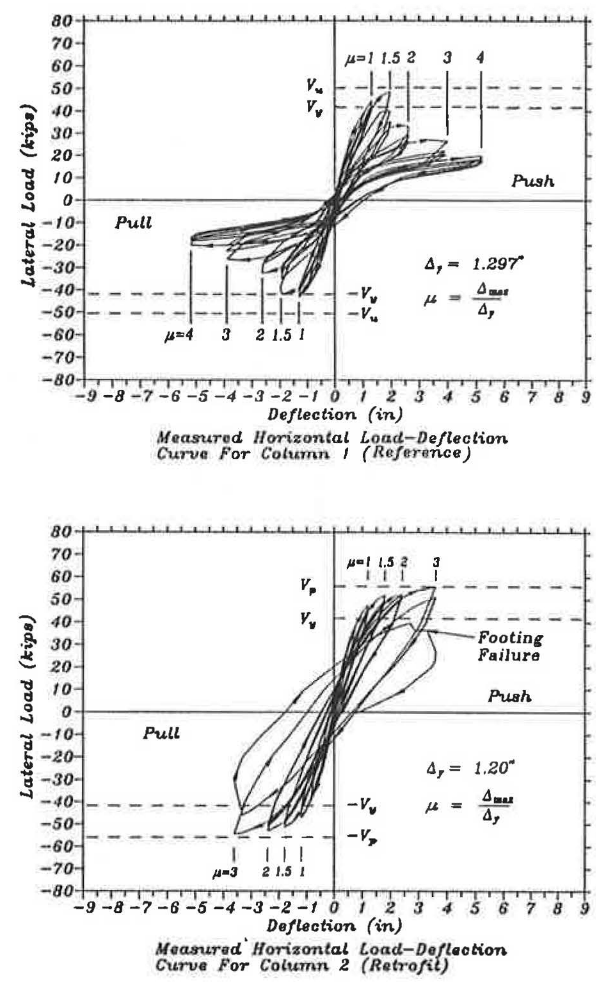

FIGURE 8: Hysteretic Response of Columns with Starter Bar and Weak Footing

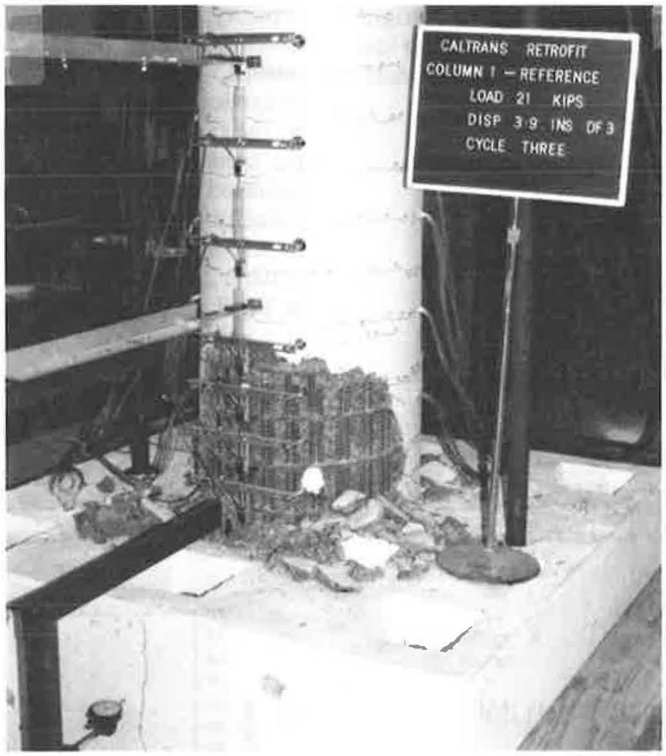

Figure 6: **Bond Failure in Column 1**

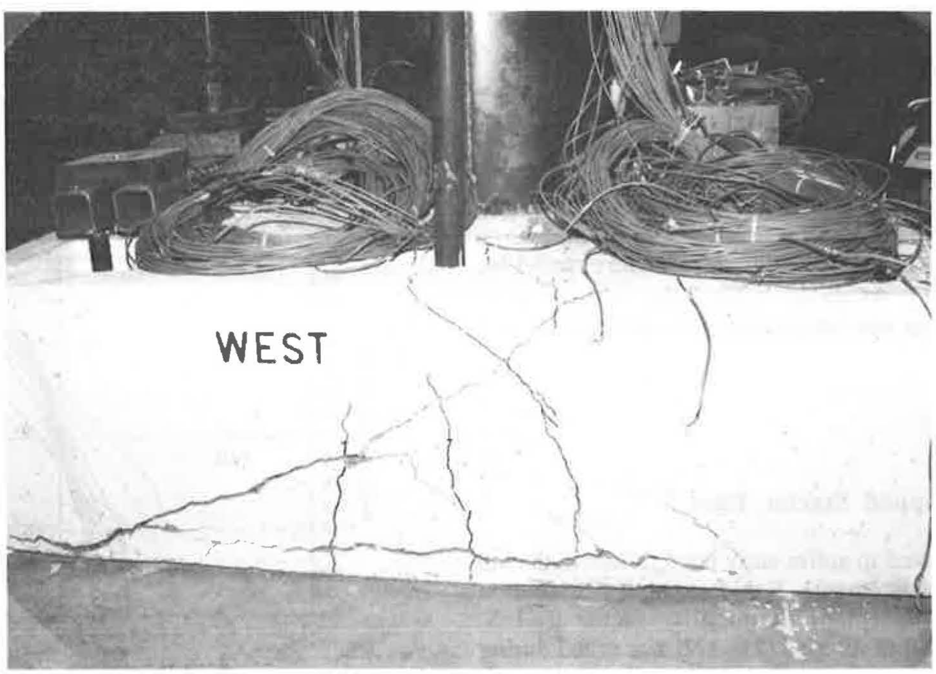

Figure 7: **Joint Shear Failure in Weak Footing - Column 2**

The weak retrofit approach adopted in column 5 uses a soft Styrofoam wrap as cushion to allow dilation of cover concrete and relative slip between main reinforcement and starter bars to occur. Without complete loss of cover concrete, strength degradation is not expected to be as rapid as the uncased column and vertical load carrying capacity can be maintained even after excursion to large lateral displacement. The response of column 5 and column 1 were very similar during the initial stages of loading (see Fig.9). Bond failure was again initiated at $\mu = 1.5$ but subsequent degradation of strength is comparatively more gradual. Theoretical capacity of column 5, as predicted without steel jacket, again could not be achieved.

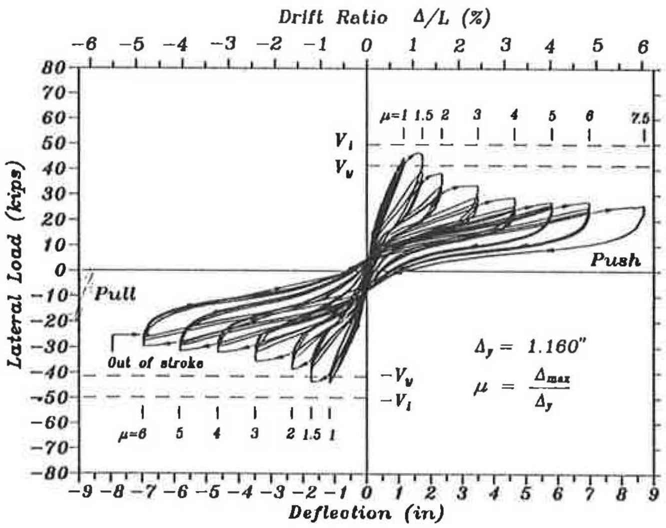

FIGURE 9: Hysteretic Response of Weak Retrofit Column

Column 6 was constructed and retrofitted with a steel jacket identical to column 2 except with a strong footing so that response at large displacement would be studied. Because columns 1 and 2 have been supported on 1 inch (25.4 mm) high pile-blocks, the compliance of the footing is reflected in the larger experimental yield displacement when compared with that of column 6 whose footing was placed in uniform bearing. The response of column 6 shown in Fig.10 is very similar to that of column 4 which was constructed with continuous reinforcement. Hysteretic loops for column 6 were stable up to $\mu = 7$ after which extreme tension reinforcement fractured due to low-cycle fatigue (see Fig.11). Bond failure which would otherwise prevail without retrofit was completely inhibited.

Columns with Continuous Reinforcement

The use of continuous reinforcement in column 3 gives a favorable increase in the displacement ductility capacity when compared to column 1. Experimental flexural capacity exceeded the theoretical capacity V_u by 6%. Very little degradation in flexural strength was noted between cycles of the same displacement magnitude except at $\mu = 5$ when failure was initiated by compression buckling of the longitudinal reinforcement. In comparison, column 4 showed significant increase in both the flexural strength and ductility. A maximum displacement ductility factor of 8 was observed in column 4 in the push direction. The displacement in the pull direction was limited by the travel in the actuator to $\mu = 6.7$. Failure of column 4 was caused by low-cycle fatigue fracture of the extreme tension reinforcement as with column 6. Theoretical capacity of $V_p = 55.6$ kips (247 kN) predicted using only the yield stress of the tension reinforcement significantly underestimated the ultimate flexural capacity of the column. Strain-hardening of longitudinal steel was estimated to have occurred at about $\mu = 3$. Theoretical strain-hardened flexural capacity of $V_{sh} = 70$ kips (311 kN calculated using an ultimate stress of 1.5 f_y in the longitudinal steel was only 4% larger than the observed maximum strength. Column 4 showed a 13% increase in the lateral stiffness over column 3. Hysteresis loops for column 3 and 4 are shown in Fig.12.

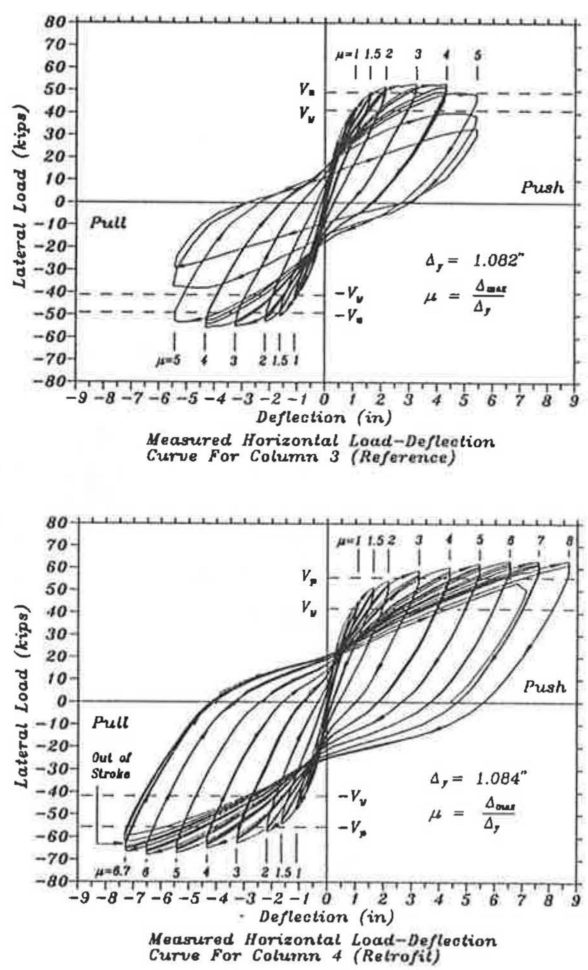

FIGURE 12: Hysteretic Responses of Columns with Continuous Reinforcement and Strong Footing

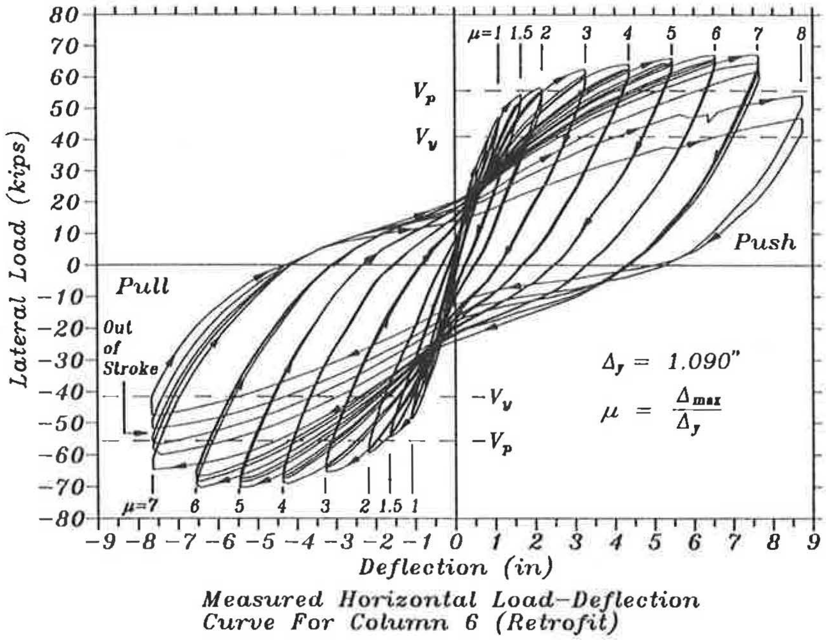

Figure 10: **Hysteretic Response of Column with Starter Bars and Strong Footing**

Figure 11: **Low-cycle Fatigue Fracture of Main Steel in Column 6**

Behavior of Repaired Column

Hysteretic response after repair of column 1 is shown in Fig.13. Compared to first testing without a steel jacket, significant improvement in the cyclic behaviors of repaired column was observed. Theoretical flexural capacity of the original column (50.6 kips or 225 kN) was exceeded by 6% at $\mu = 3$ after which gradual degradation of strength occurred due to development of bond slip in the lap of main reinforcement. There is no observed enhancement of flexural capacity due to strain-hardening of longitudinal steel as evident in the full retrofit case.

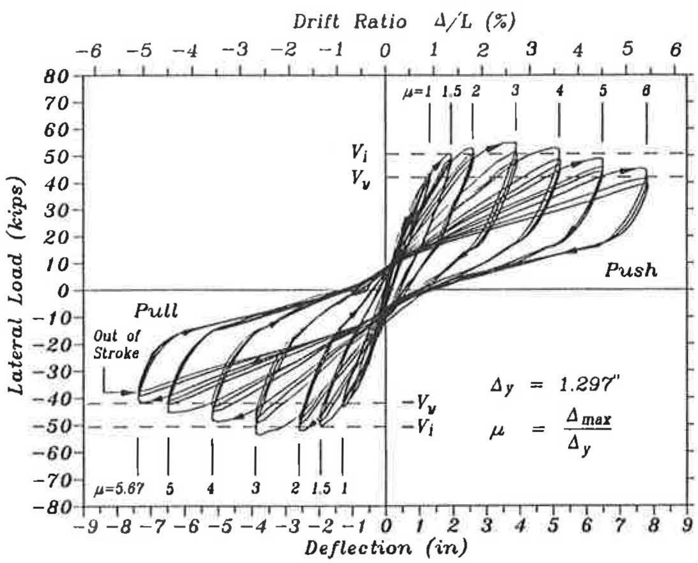

FIGURE.13: Hysteretic Response of Repaired Column

Conclusions

Experimental verification of strength and ductility enhancement by use of steel jacket is presented in the paper. A lap length of 20 times the longitudinal bar diameter is insufficient to develop yield stress of the longitudinal bar. Rapid strength degradation due to bond failure can be expected in these type of columns without retrofit. The introduction of fully grouted steel jacket as a retrofit measure is effective in providing confinement to the concrete. The cover concrete is completely contained within the steel casing eliminating bond failure. A ductile mode of flexural failure with good energy dissipation can be achieved by steel jacketing. Lateral stiffness increase due to fully grouted steel jacket is in the range of 10 to 20%. Due considerations must however be made to ensure comparable footing strength is available when the columns have been retrofitted with steel jackets. Full grouting of the jacket encourages penetration of large inelastic strain into the footing which can cause brittle footing shear failure. Post-earthquake repair of bridge columns using steel jackets can be expected to restore or even improve the seismic performance of columns with lap-splice details.

Acknowledgments

The research described in this paper has been funded by the California Department of Transportation and the Federal Highway Administration under Grants RTA599267 and F885D06. The helpful comments of Guy Mancarti and James Gates are gratefully acknowledged. The comments and conclusions in this paper are those of the authors alone and do not necessarily reflect the views of CalTrans or the FHA.

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Training Program for Implementation of Newly Developed Guidelines for Seismic Design and Retrofitting of Highway Bridges

ROY A. IMBSEN AND ROBERT A. SCHAMBER

This paper describes the training program sponsored by the National Highway Institute (NHI) in cooperation with the Federal Highway Administration (FHWA) to implement the latest technology in seismic design and retrofitting of highway bridges on a national basis. This training program includes the development of training materials, presentation of two pilot workshops and the presentation of several 4 - 1/2 day workshops. The workshops are divided into lecture sessions and class assignments. The class assignments focus on practical applications. Although the workshop does not include computers the class assignments include input coding and output results from computer programs typically needed for seismic design and analysis.

The primary objectives of the workshops are to train bridge engineers to use the American Association of the State Highway and Transportation Officials (AASHTO) Guide Specifications for Seismic Design of Highway Bridges and to introduce bridge designers to the procedures for retrofitting bridges as described in the FHWA publication *"Seismic Retrofitting Guidelines for Highway Bridges"*. The AASHTO Guide Specifications for Seismic Design are currently being incorporated into the AASHTO Standard Specifications for 1991.

Basic principals of seismology, structural dynamics and foundation modelling are presented. These basic principals are combined with current recommendations in the guidelines and the state-of-practice to give the practicing bridge engineer the knowledge required to design and retrofit bridges.

The procedures and methods available to engineers for the seismic design and retrofitting of bridges have increased markedly since the 1971 San Fernando earthquake in California. Knowledge and experience gained in research studies, post-earthquake field reconnaissance investigations, and the practical application of damage mitigation measures have contributed to this increase. In addition, the need for seismic design and retrofitting has subsequently increased nation wide due to the recent 1989 Loma Prieta earthquake. Several of these improvements in procedures and methods since 1971, however, have yet to find their way into many bridge design offices. Although some pilot courses have been presented in the past to help implement new seismic procedures, only a small percentage of practicing bridge engineers have been able to attend and many more training courses are needed. The courses now being taught utilize the state-of-practice consistent with the recently developed guide specifications *(1)* and the current available methodology.

One of the first efforts at training was the presentation of two one-week pilot workshops in 1981 designed to give working level bridge engineers hands-on experience in seismic analysis and design of bridges *(2)*. The development of these workshops was prompted by a successful Federal Highway Administration (FHWA)-sponsored Seismic Bridge Design Workshop held in Boise, Idaho in 1976 *(3)*. Both the Boise workshop and the two pilot workshops relied heavily on the methods developed by the California Department of Transportation (CALTRANS) following the San Fernando earthquake. In general, the two pilot workshops were enthusiastically received by the students, but they were handicapped at the time by the absence within the profession of the following three major elements:

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1. A nationally recognized seismic design specification for bridges that reflected the latest state-of-the-art.
2. Comprehensive, practical guidelines for seismic retrofitting of bridges.
3. User friendly computer programs readily available for the seismic analysis of bridges.

Recent research efforts and experience have made available the above three missing elements, and it is now possible to conduct much more effective training courses.

The first element was fulfilled in 1983 when the American Association of State Highway and Transportation Officials (AASHTO) adopted as a guide specification the report entitled "Seismic Design Guidelines for Highway Bridges" (ATC-6)*(1)*. These guidelines were prepared by Applied Technology Council (ATC) under FHWA contract with guidance from a panel of distinguished experts in the seismic design of bridges. The improvements in seismic design criteria contained in these guidelines have had a major impact on bridge seismic design practice.

A recognized weak point in the guide specifications was their treatment of foundations. In 1983, the Federal Highway Administration awarded a contract to the Earth Technology Corporation to develop a seismic design guide for bridge foundations*(4)*. This project is now complete, and the results, which complement the guide specifications, are useful in any training efforts relative to the seismic design and retrofitting of highway bridges.

The second missing element listed above was addressed by a follow-up report to ATC-6 entitled "Seismic Retrofitting Guidelines for Highway Bridges" (ATC-6-2)*(5)*. This report, which was developed with a separate panel of experts, established a comprehensive approach to the problem of seismic retrofitting. The guidelines include methods for rapidly identifying those bridges that present the greatest seismic hazard, methods for quantitatively evaluating local and global weaknesses and identifying economical retrofitting techniques for individual bridges, and a description of specific retrofit measures that can be used to increase the seismic resistance of selected bridge components. The guidelines have been subsequently published by FHWA*(6)* and are used as a reference for the current training course being sponsored by FHWA.

CALTRANS has recently completed a 54 million dollar program (Phase 1) to seismically retrofit approximately 1260 bridges. This program focused on expansion joints and sought to reduce the vulnerability to collapse introduced by the discontinuity in the superstructure that occurs at these joints. Simplified analysis methods and many retrofitting details and construction procedures were the by-product of this program. The experience gained as a result of this program is invaluable and is included in the current FHWA course. In addition, CALTRANS is currently in the process of retrofitting bridges with single column bents for the Phase 2 retrofit program.

The third and final missing element is readily available computer programs which may be used in a production environment for bridge designers. A user-oriented computer code was written specifically for the Seismic Analysis of Bridges (SEISAB) *(7)*. This code, developed by Engineering Computer Corporation (currently Imbsen & Associates, Inc., (IAI)) with

funding by the National Science Foundation, was specifically written to accommodate the provisions of the guide specifications. The elastic analysis portion of this computer code has been completed, and it is now possible for bridge designers, with minimum knowledge and training in theoretical structural dynamics and computer technology, to perform multi-modal response spectrum analyses of bridges. In addition, following the San Fernando earthquake CALTRANS implemented STRUDL, currently known as GTSTRUDL *(8)*, to conduct seismic analysis of bridges. A pre-processor has been developed specifically for this program to simplify the input coding.

Another factor related to implementation of seismic design procedures is the development of a "Seismic Design and Retrofit Manual for Highway Bridges" *(10)*. This document, which summarizes much of what is known about the seismic design and retrofitting of bridges, was written to update the original manual developed for the first two pilot workshops *(2)*. It therefore is an excellent reference document for the current workshop.

The recent development of design and retrofitting guidelines and the SEISAB computer code, plus experience gained from implementing improvements in seismic design and retrofitting practice have resulted in the conditions that made it timely to develop a comprehensive training course on the seismic design for highway bridges. This current course de-emphasizes hands-on computer use, but includes handout problem solutions using SEISAB. In addition, the course mentions other programs (i.e., GTSTRUDL, SAP, etc.) which can be used in the seismic analysis of bridges.

TRAINING COURSE DEVELOPMENT

The course outline that was prepared for this course reflected a balanced up-to-date approach to the seismic design and retrofitting of bridges and reflected as near as possible the consensus views of the project team. It included, where appropriate, items included in previous training courses as well as new, updated information. The course is organized into modular sessions to facilitate tailoring of the course to a specific audience (e.g. training and experience of attendees, seismic zone of interest, etc.). Each session contains certain components that can be deleted or emphasized as required. The contents of the course includes those sessions listed in the outline shown in Table 1. The workbook, instructor's guide, and visual aids were organized according to the course sessions. The workbook was designed to be useful before, during, and after the course presentation (see Figure 1). Before the presentation, the workbook is used to brief students on the material to be covered and orient them toward the objectives of each course session. During the presentation, the workbook facilitates note taking and helps students focus on the topics being discussed. After the presentation, the workbook will be a useful reference. The problem assignments that are included (12 total) can be referenced in the bridge engineers procedures for seismic design. Worked example problems included in the participant workbook utilize SEISAB and other computer programs. These examples emphasize the interpretation and use of output results. The problem assignments are presented by filling in selected calculations as shown in Figure 2. The following paragraphs discuss the content of each of the sessions presented as shown in Table 1.

Introduction to Seismic Design and Retrofitting of Bridges

Session 1 of the course provides the student with a conceptual understanding of the issues and goals of seismic design and retrofitting of bridges. This includes knowledge of the currently applicable design and retrofitting criteria, design and retrofitting strategies, the effect of siting, structural configuration, detailing on the seismic performance of bridges, and the performance of bridges during past earthquakes. The latest bridge damage from the 1989 Loma Prieta earthquake is presented in this session. In addition, a brief overview of the AASHTO Guide Specifications *(1)* is included. The preliminary seismic design concept which is introduced in shown in Figure 3.

Fundamental Concepts in Structural Dynamics

This session emphasizes physical concepts as opposed to mathematical derivations. Considerable attention is given to achieving this goal through the extensive use of graphical illustrations, models, and a VCR presentation that helps the students visualize basic concepts. A model of three single degree of freedom oscillators mounted on a common base is used for explaining the concept of natural period and the effect of earthquake ground motion on different types of structures. The concept of various modes of response is illustrated by a three degree of freedom model. Workshop assignments and example problems are included to help students become familiar with these critical concepts. It should be emphasized that this session uses bridge schematics in the text to teach the concepts of structural dynamics as shown in Figure 4.

Seismic Loading

This session covers the sources and characteristics of ground motion and other critical earthquake effects. The development of seismic zones is also discussed. The characterization of these phenomena for design loading is covered to introduce students to the concept of response spectrum, acceleration time history, the directionality of ground motion, etc. This information reflects the latest knowledge relative to the nature of earthquakes, including information from the 1989 Loma Prieta earthquake. The normalized response spectra sited in the AASHTO Guide Specifications *(1)* is shown in Figure 5. In addition, the new accceleration coefficient maps now adopted by AASHTO for the 1991 standard specification are shown in Figure 6.

Seismic Response Analysis

Session 4 includes a description and comparison of analysis methods. The material includes the ATC single mode spectral approach (procedure 1) as well as the multi-mode spectral approach (procedure 2). A brief overview of the example problem in Appendix A of the AASHTO Guide Specifications *(1)* is also included by projecting mode shapes of that bridge model on a screen to show participation factor contributions.

Structure modeling is a critical part of seismic response analysis. Correct idealization of the mass and stiffness characteristics of a structure can make the difference between reasonable or unreasonable analysis results. In addition a group assignment is given as shown in Figure 7 to introduce the participants to seismic structural form. A sample of the group assignment presentation from one of the courses is shown in Figure 8.

Design Concepts

This session is subdivided into lectures and workshop problems covering the various aspects of component design. The concept of ductility demand and the use of response modification factors are explained so that participants can get a true conceptual

feeling for how a bridge is likely to perform during an earthquake. Recent knowledge gained from research on the ductile behavior of reinforced concrete columns is presented. This includes studies relative to the role of transverse confining steel, main longitudinal reinforcement, splices in reinforcement and the role of concrete in resisting compression and shear forces. A discussion of the behavior of wide piers is also included.

The actual design of a reinforced concrete column including both longitudinal and transverse steel design is covered in example and workshop problems. This includes an introduction to the concept of capacity design as it applies to multi-column bridge bents. Proper emphasis is also given to the importance of good detailing practice for seismic design.

Modeling of bridge abutments and foundations is a subject that has been clarified by the recent development of guidelines for foundation design. Because modelling of these elements has always been a problem for engineers in the past, special treatment is given to this subject. Workshop assignments are provided so that students can have first-hand experience in calculating the stiffness and damping characteristics of the most common types of abutment and foundation elements. The use of design charts are covered in the lectures and example problems. This part of the session reflects the findings of the recent FHWA-funded study conducted by the Earth Technology Corporation on the seismic design of bridge foundations *(4)*.

The design of conventional bearings, expansion joints, restrainers, and shear keys is a critical issue that is covered during session 5. The design of other types of bridge components is discussed briefly to introduce students to some of the latest thinking on this subject. The use of special motion restricting devices, energy dissipation devices and base isolation bearings is covered along with their related design philosophies and considerations.

The subject of ground stability is an important consideration during both the planning and design of bridges and their accompanying roadway approaches. A lecture is provided to give students knowledge in assessing probable ground stability during an earthquake.

Retrofitting

The seismic retrofitting guidelines developed by ATC *(5,6)* as well as the latest practical experience obtained from CALTRANS and others who have attempted to retrofit bridges is discussed.

The subject of seismic retrofitting is divided into three parts as presented in the seismic retrofitting guidelines *(5,6)* (see Figure 9). The first part is preliminary screening which is a planning function used during the implementation of an area-wide retrofitting program.

The detailed evaluation of an individual bridge to determine its capacity to resist earthquake loads is important when identifying the need for retrofitting and the most appropriate retrofitting strategy to be used. The method for calculating component capacity/demand ratios as proposed by the retrofitting guidelines is covered through lectures, example problems and workshop assignments.

Design of component retrofitting requires considerable innovation since no two structures are the same. Retrofitting techniques is discussed in terms of concepts. Several concepts are presented in the retrofitting guidelines. A considerable amount of standardization of these concepts as they relate to retrofitting of bearings and expansion joints was achieved by CALTRANS during their retrofitting program.

The retrofitting concepts for other components such as columns and footings is discussed briefly. Also included is a discussion of retrofitting of geotechnical components of the bridge.

Advanced Topics

Session 7 of the course will address advanced topics that may be of interest to certain groups. These topics include special bridge types and nonlinear analysis *(11)* for seismic analysis of bridges.

PILOT PRESENTATIONS AND COURSE PRESENTATIONS

Following two pilot presentations, the courses have been scheduled throughout the United States to state department of transportations or private agencies (over 20 courses have been given since January of 1990). Each course, which averages 40 students, is tailored to meet the needs and interests of the attendees and the particular seismic zone of interest. The principal instructor is Dr. Roy A. Imbsen, President of IAI. He is assisted by either Mr. Robert A. Schamber of IAI or Mr. James H. Gates of CALTRANS as one of his Co-Instructors and either Dr. Geoffrey R. Martin of the University of Southern California or Mr. Ignatius (Po) Lam of Earth Mechanics, Inc. as his other Co-Instructor. The FHWA Contracting Officer's Technical Representative (COTR) is Mr. Larry Jones.

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11. Imbsen, R. A., Penzien, J., "Evaluation of Energy Absorption Characteristics of Highway Bridges Under Seismic Conditions," Vol. 1 and 2, Report No. EERC 85/17, September, 1986.

TABLE 1 OUTLINE FOR FHWA TRAINING COURSE ON SEISMIC DESIGN OF HIGHWAY BRIDGES

TABLE OF CONTENTS

SESSION DESCRIPTION

1.0 INTRODUCTION TO SEISMIC DESIGN AND RETROFITTING OF BRIDGES (4 hours)

- 1.1 The Effect of Earthquakes on Bridges
- 1.2 Seismic Design and Retrofitting Since the San Fernando Earthquake
- 1.3 Seismic Design and Retrofitting Philosophy
- 1.4 Planning Considerations
- 1.5 Seismic Design and Retrofitting Strategies
- 1.6 AASHTO Guide Specifications (Example Problem Assignment 1)
- 1.7 Seismic Base Isolation

2.0 FUNDAMENTAL CONCEPTS IN STRUCTURAL DYNAMICS (4 hours)

- 2.1 Dynamic Loading and d'Alembert's Principal
- 2.2 Single Degree-of-Freedom Systems
- 2.3 Muilti Degree-of-Freedom Systems (Example Problem Assignment 2)

3.0 SEISMIC LOADING (2 hour)

- 3.1 Basic Seismology
- 3.2 Characteristics of Earthquake Ground Motion
- 3.3 Seismic Design Loadings

4.0 SEISMIC RESPONSE ANALYSIS (6 hours)

- 4.1 Analysis Methods for Design
  - 4.1.1 Equivalent Static Force Methods (Example Problem Assignment 3)
  - 4.1.2 AASHTO Guide Specifications
    - 4.1.2.1 Single Mode Spectral (Example Problem Assignment 4)
    - 4.1.2.2 Multi-Mode Spectral
- 4.2 Modeling for Analysis
  - 4.2.1 Practical Modeling Guidelines
  - 4.2.2 Structural Form and Details (Example Problem Assignment 5)

5.0 DESIGN CONCEPTS (8 hours)

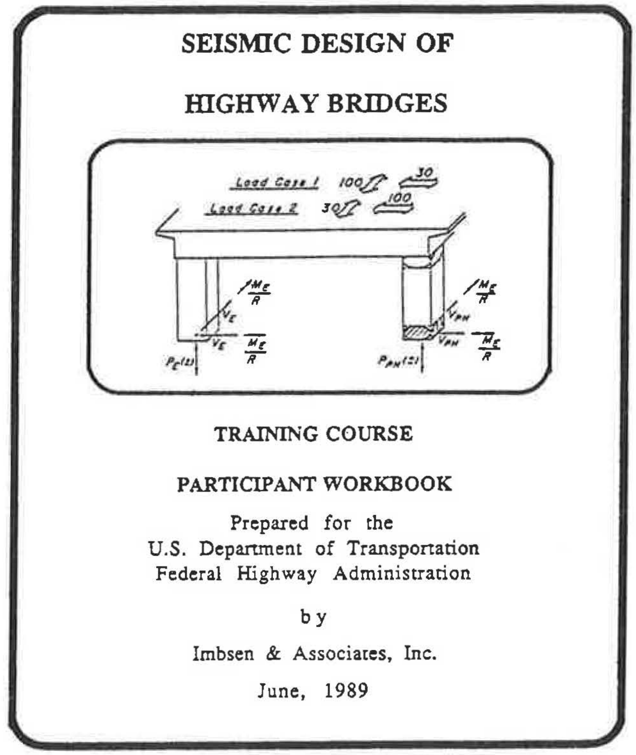
- 5.1 Component Design
  - 5.1.1 Reinforced Concrete Columns and Piers (Example Problem Assignment 6)
  - 5.1.2 Abutments and Foundations (Example Problem Assignment 7 and 8)
  - 5.1.3 Bearings, Expansion Joints, Restrainers, Shear Keys, and Hold-Down Forces (Example Problem Assignment 9)
- 5.2 Ground Stability Considerations

6.0 RETROFITTING (8 hours)

- 6.1 Preliminary Screening (Example Problem Assignment 10)
- 6.2 Detailed Evaluation (Example Problem Assignment 11)
- 6.3 Retrofitting and Bearing Concepts (Example Problem Assignment 12)

7.0 ADVANCED TOPICS (4 hours)

- 7.1 Nonlinear Analysis
- 7.2 Complicated Structures


SEISMIC DESIGN OF

HIGHWAY BRIDGES

TRAINING COURSE

PARTICIPANT WORKBOOK

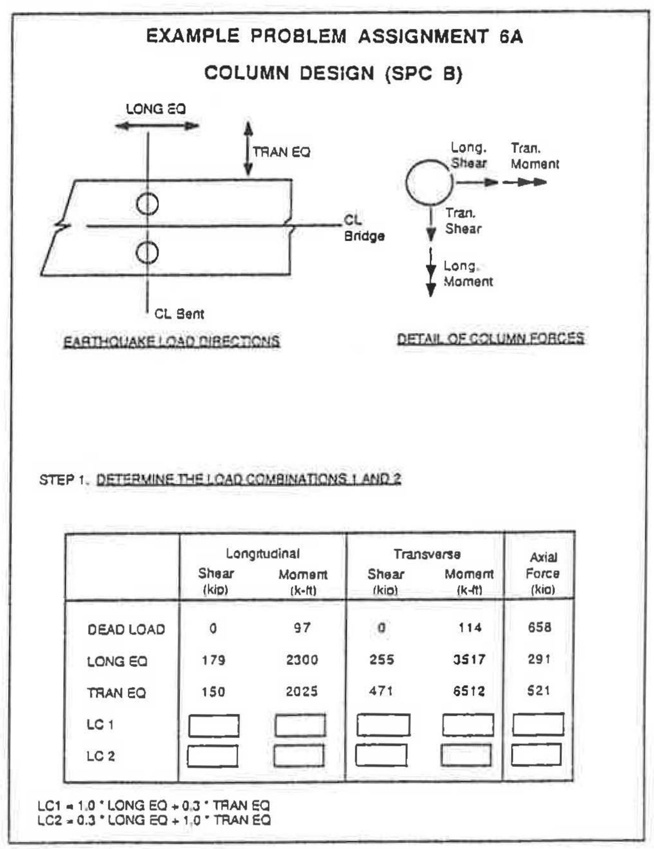
Prepared for the
U.S. Department of Transportation
Federal Highway Administration

by

Imbsen & Associates, Inc.

June, 1989

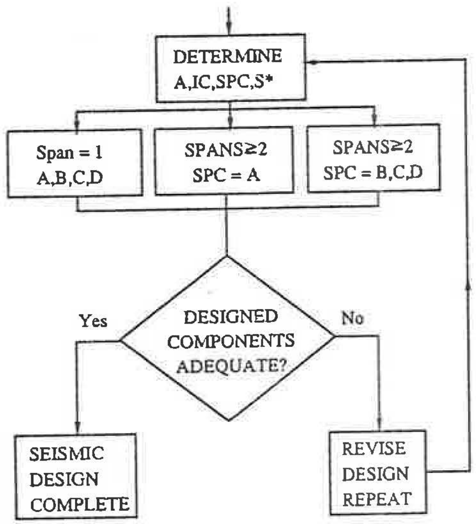
FIGURE 1 Cover on Participant Workbook



	Longitudinal Shear (kip)	Longitudinal Moment (k-ft)	Transverse Shear (kip)	Transverse Moment (k-ft)	Axial Force (kip)
DEAD LOAD	0	97	0	114	658
LONG EQ	179	2300	255	3517	291
TRAN EQ	150	2025	471	6512	521
LC 1					
LC 2					

LC1 = 1.0 * LONG EQ + 0.3 * TRAN EQ
LC2 = 0.3 * LONG EQ + 1.0 * TRAN EQ

FIGURE 2 Example Problem Assignment



*A = Acceleration Coefficient
IC = Importance Classification
SPC = Seismic Performance Category
S = Site Coefficient

FIGURE 3 Preliminary Seismic Design Concept

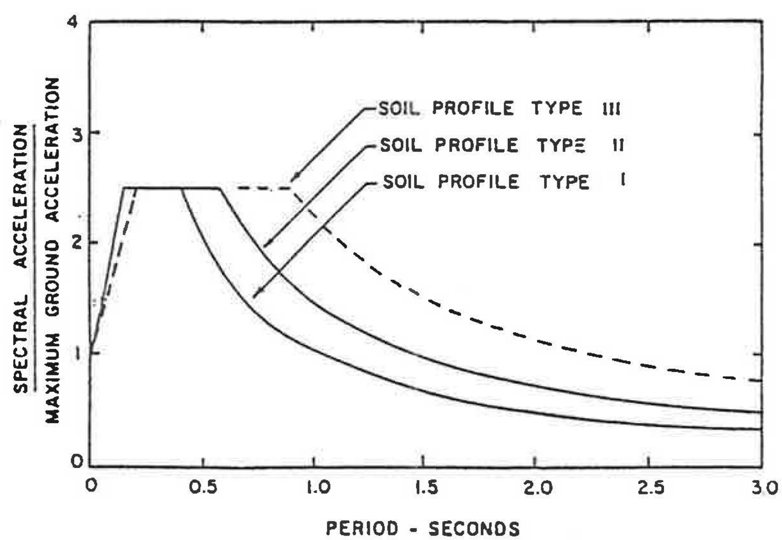


FIGURE 5 Normalized Response Spectra

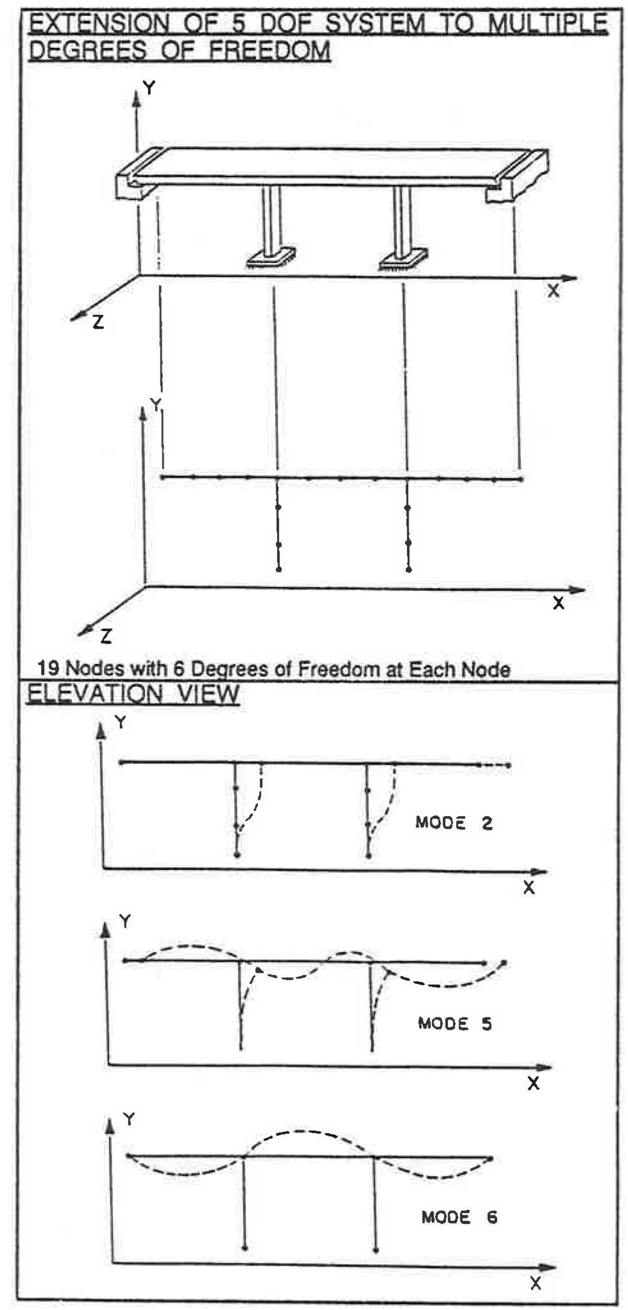


FIGURE 4 Typical Bridge Schematic to Illustrate Dynamic Concepts

Western Part of the United States

FIGURE 6a Acceleration Coefficient (expressed as percent of gravity) Using the Adopted AASHTO Standard Specification Map for 1991 in Rock with 90 Percent Probability of Not Being Exceeded in 50 years.

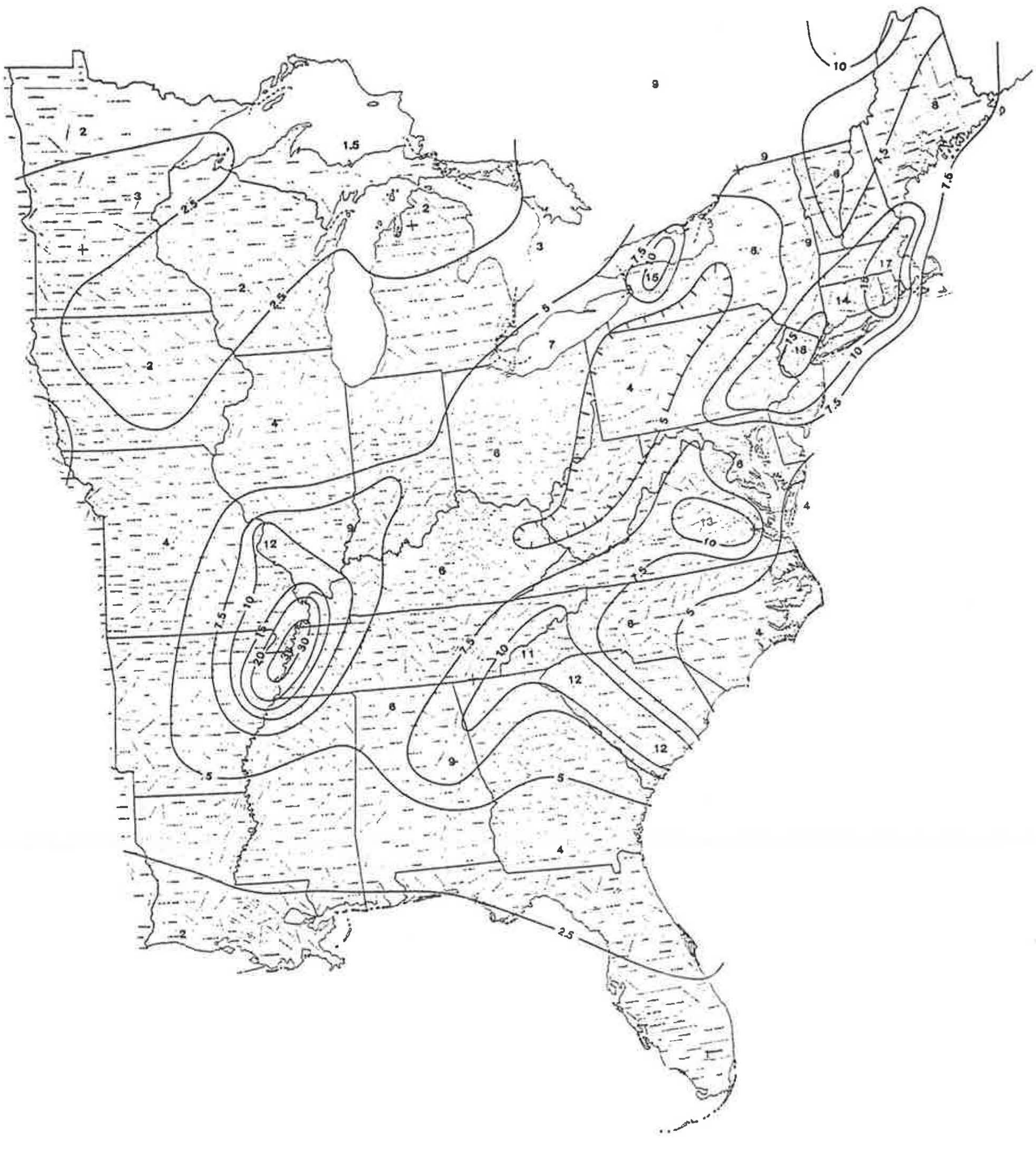
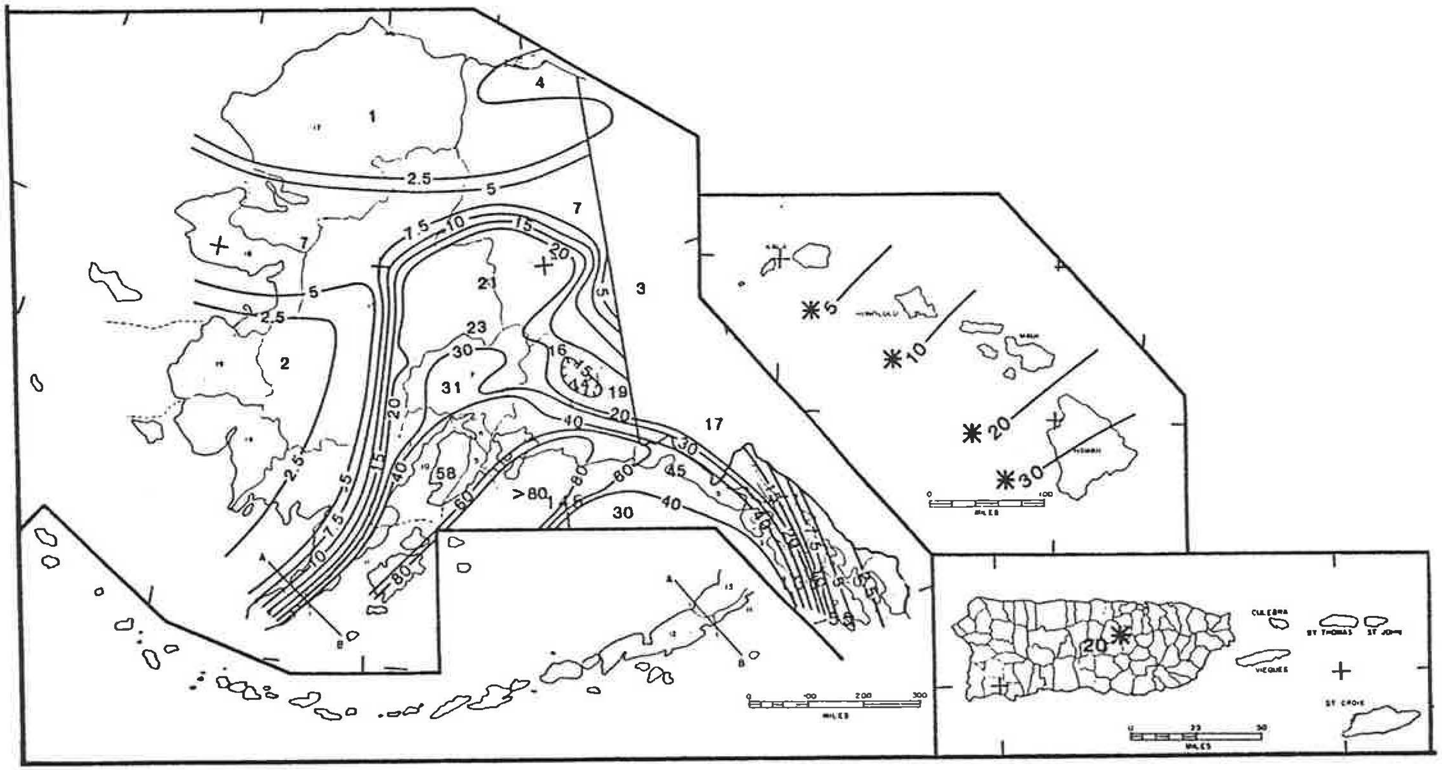
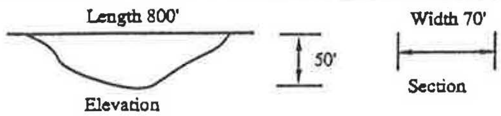


FIGURE 6b Acceleration Coefficient (expressed as percent of gravity) Using the Adopted AASHTO Standard Specification Map for 1991 in Rock with 90 Percent Probability of Not Being Exceeded in 50 years.



Alaska, Hawaii and Puerto Rico

FIGURE 6c Acceleration Coefficient (expressed as percent of gravity) Using the Adopted AASHTO Standard Specification Map for 1991 in Rock with 90 Percent Probability of Not Being Exceeded in 50 years.



Assume AASHTO Guide Specifications Apply
High Seismic Area, Type III Soils

Superstructure Type to be Selected By Group
Span Range: 70' to 160'

In Small Groups (45 min.)

(1) Select two different structural configurations which the group feels would be adequate for this site. Do not get too bogged down at this point, just pick two reasonable bridges based on your experience.
(2) Evaluate each of the bridges on the basis of seismic performance. List positive and negative aspects of each bridge. Select one bridge to present and discuss to the total group.

Discussion (15 min.)

(3) Each group to present and discuss the positive and negative seismic aspects of their selected design. Discussion of total group should focus on advantages and disadvantages of each design, if possible, determine a single design which appears reasonable.

FIGURE 7 Group Assignment for Structural Form Example Problem

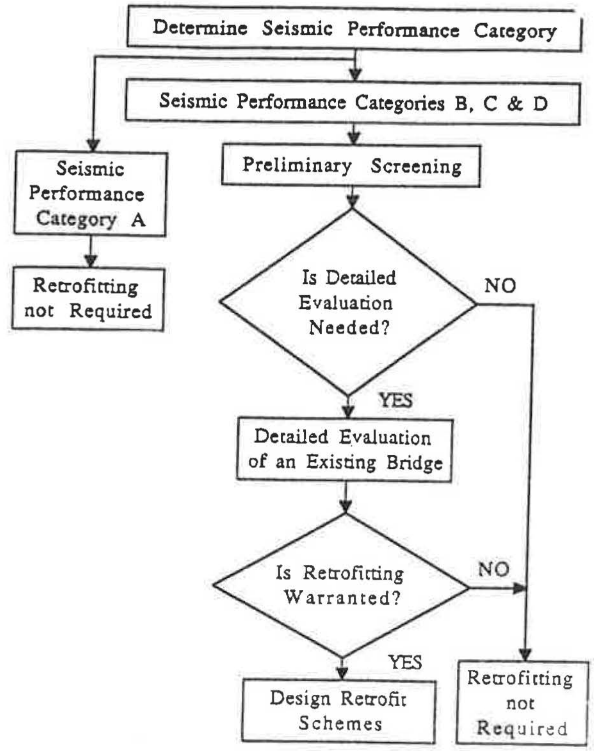


FIGURE 9 Seismic Retrofitting Process

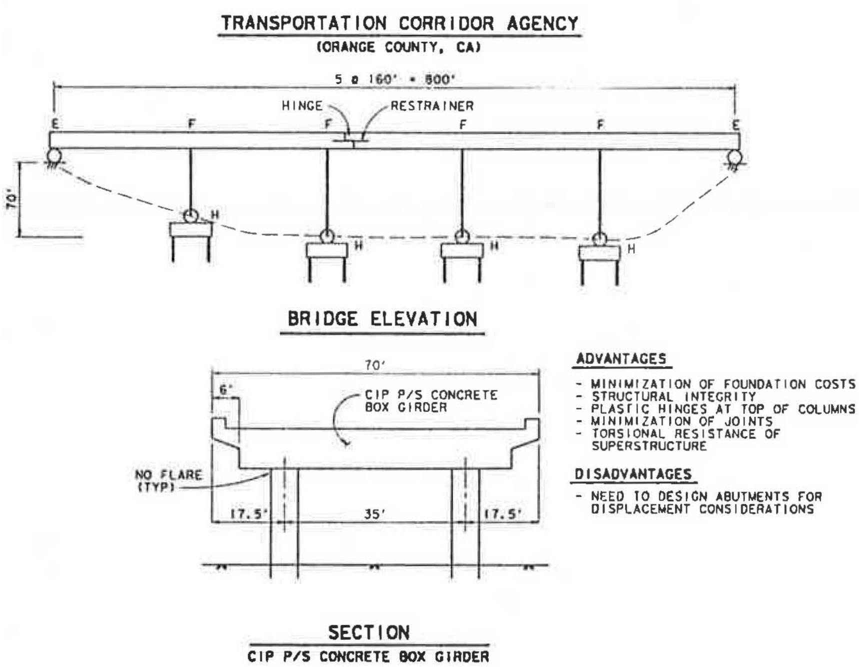


FIGURE 8 Sample of Group Assignment Presentation for Structural Form Example

Modeling Bridge Foundations for Seismic Design and Retrofitting

IGNATIUS PO LAM,[1] GEOFFREY R. MARTIN,[2] AND ROY IMBSEN[3]

Past experience and recent research indicates that proper modeling of abutments and foundations is very important in the evaluation of dynamic response of an overall bridge structure. This paper presents simplified procedures with accompanying design charts for the development of stiffness coefficients for abutments, piles and spread footing foundations for highway bridges. The presented procedures have been calibrated to design practice adopted by bridge engineers. Several examples are presented in the paper to highlight various sensitivity issues in abutment and foundation design.

INTRODUCTION

Highway bridges are known to be highly susceptible to damage under earthquake loading. One major reason for the poor performance relates to the complexities of the bridge structural and substructural systems as compared to other structures. Some of these complexities are listed below:

- There are wide variations in structural types and configurations (bridge decks, bents and abutments). Furthermore, the need for expansion joints to accommodate temperature effects increases the potential for collapse and introduces significant complexities in design.

- Variations in soil conditions are common along the length of a highway bridge which could lead to different types of abutments and foundation systems. The wide range of combinations of structures, abutments, foundations and connections introduces many unique design problems for bridge engineers.

- Bridges are supported on multiple support points that are spaced relatively wide apart. As a result, variations in ground motion (magnitude and phase shift) could result in differences in force excitation levels and differential movements at support points.

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[2] Professor, Dept. of Civil Engineering, Univ. of Southern California, L.A., CA.
[3] President, Imbsen and Associates, Inc., Sacramento, CA.

An excellent literature survey (Iwasaki et al., 1973) chronicles earthquake damage to bridges up until 1971. This report concluded that foundation behavior played a major role on performance of highway bridges during past earthquakes. Extracts from the conclusion section of the report are quoted below to illustrate the significance of foundation systems in seismic performance of bridges:

- "Seismic damage, particularly to low bridges, are most commonly caused by foundation failures resulting from excessive ground deformation and/or loss of stability and bearing capacity of the foundation soils......"

- "Backfills exert large forces on abutments which can at certain times be in-phase with the seismic inertia forces developed in the superstructures. These forces in combination may cause severe failures, often of a brittle nature, in the substructures."

- "To minimize damage, bridge structures should be designed with proper recognition of the stability and bearing capacities of foundation soils, force-deformation and energy absorption characteristics of substructure, superstructure and connecting elements, the dynamic nature of structural response, and the dynamic characteristics of all forces acting on the complete soil-structure system."

Poor soil conditions (soft soil and a high water table) contributed to most of the structural and substructural damage during many of the past earthquakes, often of a severe catastrophic nature. Sites that have sustained heavy damage during past earthquakes due to ground motion amplification, slope failure or liquefaction would be likely candidates for damage in future earthquakes. Therefore, existing earth-science information

(seismologic, geologic, geotechnical, hydrological) should be used in regional earthquake preparedness and planning programs as well as in site specific designs.

Geotechnical considerations for highway bridges can be divided into two categories:

- o Ground Stability. Site response, liquefaction potential, embankment slope stability and assessments of the magnitude of ground deformations (including cyclic and permanent deformations of the free-field site soil) and the evaluation of the nature and the magnitude of foundation movements and their implications on structural loading, are critical elements in foundation design of highway bridges.

- o Substructures and Abutment Models. Evaluation of the fundamental periods and mode shapes of the bridge structure is needed for the determination of the seismic force level for design. For some bridges, especially the shorter and low bridges, the effects of foundation stiffness could significantly affect the overall dynamic response characteristics. Therefore, evaluation of foundation stiffness is an important part of the overall bridge structure model.

This paper focuses on modelling procedures for estimating stiffnesses of abutments, spread footings and pile footings (pile groups) for dynamic response analysis of an overall bridge.

ABUTMENTS

For many highway bridges, the abutments attract a large portion of the seismic force, particularly in the longitudinal direction. Therefore, the stiffness of the backfill at the abutments must be considered. However, further research is needed to improve our understanding and to develop improved design procedures related to the following aspects:

- o Stiffness of abutment systems in both the longitudinal and transverse loading directions.

- o The magnitude and distribution of soil pressure on end and wing walls.

- o The relative significance in the induced soil pressure from inertia forces of the bridge deck versus the inertia forces of the soil mass acting on the abutment walls and the appropriate procedures for selection of design soil pressure to account for the interplay of the two loading mechanisms.

Due to the wide range of combinations in connection details (bridge-abutment and abutment-foundation), many of the above issues remain to be resolved and currently, a wide variation in design practice of abutments prevails.

Some guidance is currently provided by the Caltrans Bridge Design Aids Section 14- "Dynamic Model Assumptions and Adjustments" (Caltrans) and the AASHTO Guide Specifications for Seismic Design of Highway Bridges" (AASHTO, 1983). Both documents recognize the highly nonlinear behavior in abutments due to failure of the backfills and from structural nonlinearity at expansion joints. The load-displacement characteristics for a typical monolithic and a seat-type abutment are shown in Figure 1.

An iterative design procedure as shown in Figure 2 and described in the following steps is needed to account for the nonlinear behavior of abutment systems in a linear dynamic response analysis.

(1) Assume an Initial Abutment Stiffness. This stiffness should be compatible with the configuration of the structure and connection details and the assumed peak displacement of the bridge deck.

For longitudinal loading, it should be recognized that in the course of an earthquake, the stiffness of only one abutment would be mobilized (i.e., soil resistance is mobilized when the structure is moving toward the soil, whereas, no soil resistance is mobilized when the abutment moves away from the soil). Therefore, the stiffness would be too high if the full soil stiffness were used at both abutments. For most non-curved bridges, as an approximation, one-half of the total stiffness should be allocated to each abutment at both ends. When the "half-half" stiffness approach is used, the resulting abutment forces should be doubled for design. For curved bridges, it may be necessary to assign the full abutment stiffness at one end of the bridge while assigning a zero stiffness on the other end. Two dynamic response analyses are needed (each run using a full stiffness at each abutment) for this "full-zero" stiffness approach.

The transverse stiffness should reflect the potential reduction in stiffness arising from the deformability of wing walls (relative to the bridge) and the partial contact

and sloping configuration of the backfills at the exterior surfaces of wing walls.

(2) Conduct Dynamic Response Analysis. Using the above abutment stiffness, conduct a dynamic response analysis of the overall bridge to determine the forces and displacements.

(3) Check Abutment Force Capacity. Using peak abutment force and the effective area of the abutment wall, solve for the peak soil pressure and check that the soil capacity has not been exceeded. The soil capacity should be based on the properties of the backfill. Caltrans have recommended a soil capacity of 7.7 ksf (370 kPa) for typical California abutment-backfill conditions (sandy soil with shear wave velocity of about 800 fps or 240 meter /sec). If the peak soil pressure exceeds the soil capacity, the analysis should be repeated with a reduced abutment stiffness to reflect plastic yielding of the backfill soil. Iterations should be conducted until the force levels are below the acceptable capacity of the abutment, prior to proceeding to the next step.

(4) Check Abutment Displacement. Compare the computed displacements against the value assumed in Step 1 in relation to the load-displacement characteristics (See Figure 1) of the abutment system for the configuration of the expansion joints and the soil capacity value. This cross check is needed to ensure that the assumed abutment stiffness reflects the load-displacement characteristics properly. If the error in the assumed stiffness is excessive, the analysis should be repeated with a revised stiffness.

The converged displacement value of the abutment (with respect to the nonlinear load-displacement characteristic) should then be evaluated against the acceptable level of displacement. Excessive deformations at the abutment may cause problems. Field inspections after the 1971 San Fernando earthquake suggest that abutments which moved up to 0.2 feet (6 cm) in the longitudinal direction into the backfill soil appeared to survive with little need for repair. Therefore, if possible, this limit should be maintained. Excessive deformations may create stability and integrity problems both at the abutment and at the bents. Deformations greater than 0.2 feet (6 cm) at abutments should be evaluated for these effects.

The above steps outlined the various design considerations that should be addressed in abutments. Development of the load-displacement characteristics of the abutment-backfill system forms the basic requirement in abutment modeling. It depends on three parameters:

(1) The initial abutment-backfill interaction stiffness prior to soil failure.

(2) The ultimate resistance of the load-displacement characteristics from backfill soil failure consideration.

(3) The magnitude of the gap at expansion joints for seat-type abutments.

Discussions of the latter two parameters (based largely on Caltrans recommendations) have been presented earlier. Caltrans Bridge Design Aids Section 14 recommended an abutment-backfill interaction stiffness coefficient of 200 kips per inch of deflection per linear foot (115 MPa) along the length of the abutment wall as a starting point for iterative analysis. This abutment-soil stiffness coefficient would be appropriate for typical California abutment backfill conditions (material with shear wave velocity of about 800 ft/sec (240 m/s) and approximately 8 feet (2.4 m) of effective height of abutment walls).

For abutment configurations and soil conditions that differ significantly from the above condition, a more general form of abutment wall-backfill stiffness equation, which considers the passive resistance of the soil, as recommended by Wilson (1988) could be used to develop the longitudinal stiffness of the end wall and the transverse stiffness of the wing wall. The equation is given by:

$$K_s = \frac{E_s}{(1-\upsilon^2)\ I} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (1)$$

where K_s is soil stiffness per unit deflection per unit wall width; E_s is the Young's modulus of the backfill soil; υ is the Poisson ratio of the backfill soil; and I is the shape factor as shown in Figure 3.

The above equation allows for input of site specific soil parameters and abutment wall configurations. As the length to height ratios for wing walls are somewhat smaller than end walls, the above equation suggests a lower shape factor I, or a higher soil stiffness coefficient (K_s) for wing walls as compared to end walls.

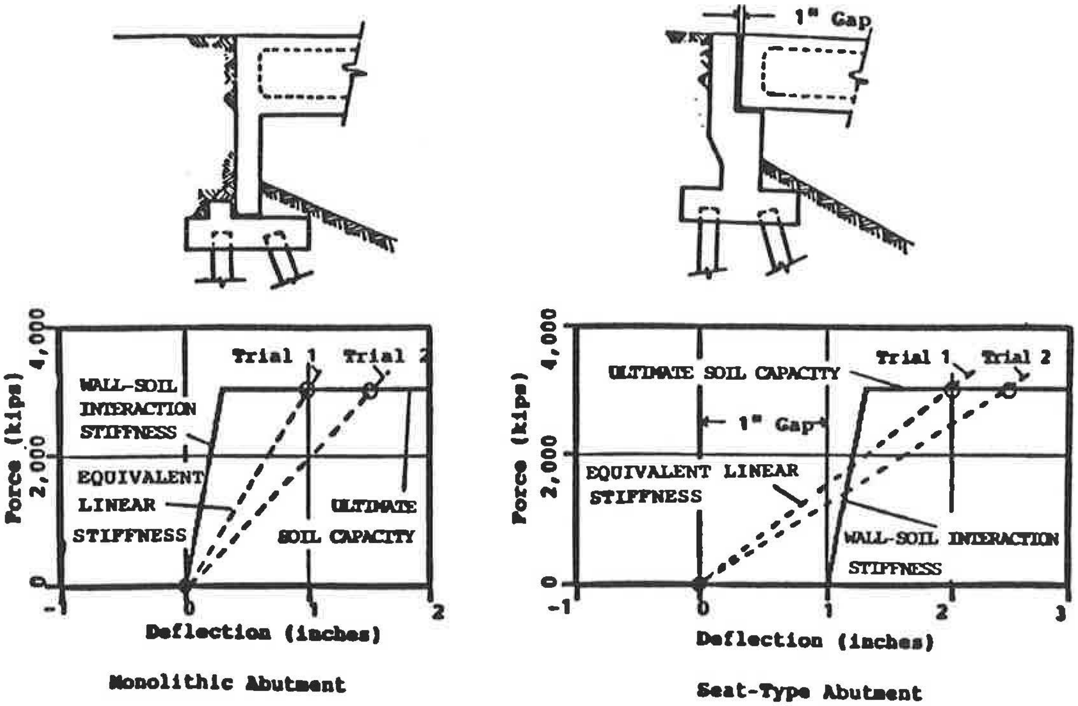


Figure 1. Load-Displacement Characteristics of Abutments

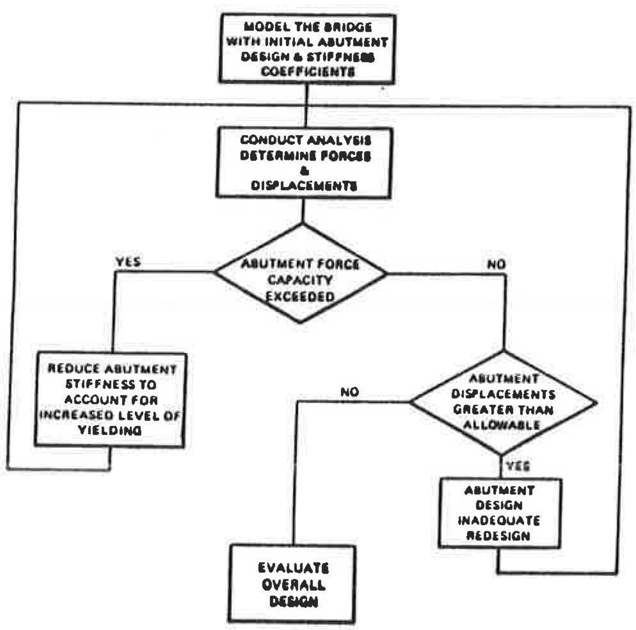


Figure 2. Iterative Procedure for the Determination of Abutment-Soil Interaction (After AASHTO, 1983)

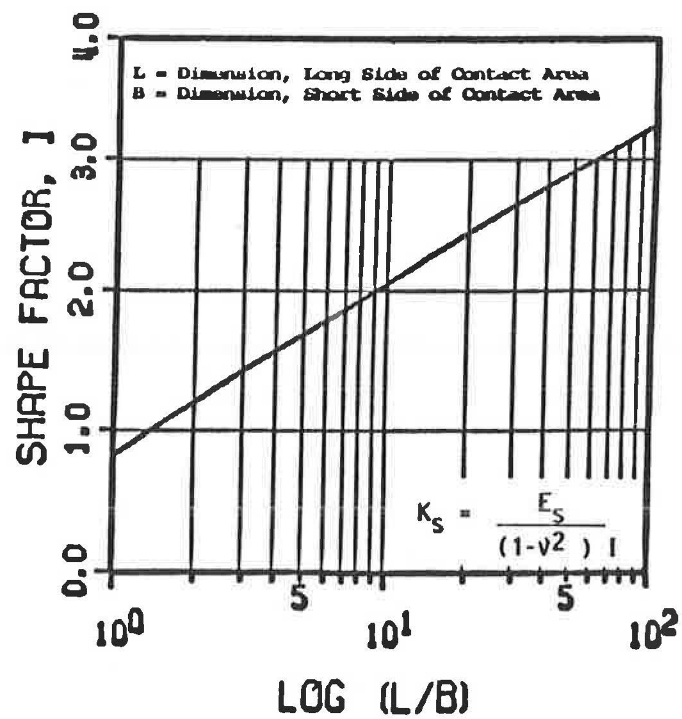


Figure 3. Shape Factor for abutment Stiffness in Equation 1

<u>Comparison to Caltrans.</u> The soil stiffness (K_s) from Wilson's equation can be compared to Caltrans recommendation through an example calculation for the longitudinal stiffness of a typical California abutment end wall-backfill system:

- Configuration of end wall: 50-ft (15.2 m) wide by 8-ft (2.4 m) high. For this configuration, a shape factor, I of 1.84 is obtained from Figure 3.

- Shear wave velocity of backfill: (800 ft/sec or 240 m/sec).

- Using a Poisson ratio of 0.3, a Young's modulus corresponding to a shear wave velocity of 800 ft/sec would be about 6.2 x 10^6 psf (300 MPa). However, a reduction factor is normally needed to adjust for a soil modulus based on shear wave velocity measurement to account for nonlinear soil behavior at higher strain levels. Some typical soil modulus variations with shearing strain curves have been recommended by Lam and Martin (1986). At a typical average shear strain value of about 0.01 percent, a reduction factor of 0.7 would be reasonable. The correspon-ding reduced Young's modulus is estimated at about 4.34 x 10^6 psf (210 MPa).

- The soil stiffness coefficient K_s for the above end wall is estimated at 216 kip/in of deflection/lineal foot of end wall (124 Mpa) as compared to the 200 kip/in /ft (115 MPa) as recommended by Caltrans.

It can be concluded that the Wilson's equation compares favorably with Caltrans recommendations and provides a rational basis for extrapolation to non California design conditions (different soil types).

SPREAD FOOTINGS

The current state-of-practice in analyses of footings involves the use of stiffness equations of a rigid footing on a semi-infinite elastic half space. Typically, footings used for highway bridges are rectangular in shape and are embedded beneath a layer of ground cover soil. A procedure has been developed for solutions of stiffnesses of embedded rectangular footings. The procedure involves first solving for the radius of an equivalent circular footing as shown in Figure 4. The cross-coupling effects between moment and shear for most shallow footings are small and can be neglected. The diagonal stiffness terms in the stiffness matrix of an embedded rectangular footing can be obtained from equations 2.

<u>Component</u>	<u>Stiffness Coefficient</u>
Vert. Translation	$\alpha\ \beta\ 4\ G\ R\ /\ (1-\upsilon)$ . . (2a)
Hor. Translation	$\alpha\ \beta\ 8\ G\ R\ /\ (2-\upsilon)$ . . (2b)
Tors. Rotation	$\alpha\ \beta\ 16\ G\ R^3\ /\ 3$ . . (2c)
Rocking Rotation	$\alpha\ \beta\ 8\ G\ R^3\ /\ 3\ (1-\upsilon)$ (2d)

where G and υ are shear modulus and Poisson ratio for an elastic half-space material; R is the equivalent radius as shown in Figure 4. α and β are the embedment and shape correction factors, respectively (See Figure 5).

It should be noted that the design chart was developed for a special case of zero ground cover thickness (soil thickness above the top of the slab). We recommend that, for conservatism, the thickness of the slab be used as the embedment depth (D in Figure 4) rather than the full depth from ground surface to footing base. This approximation is needed to avoid the need of an extended set of design charts to accommodate the wide combinations of ground cover and slab thicknesses.

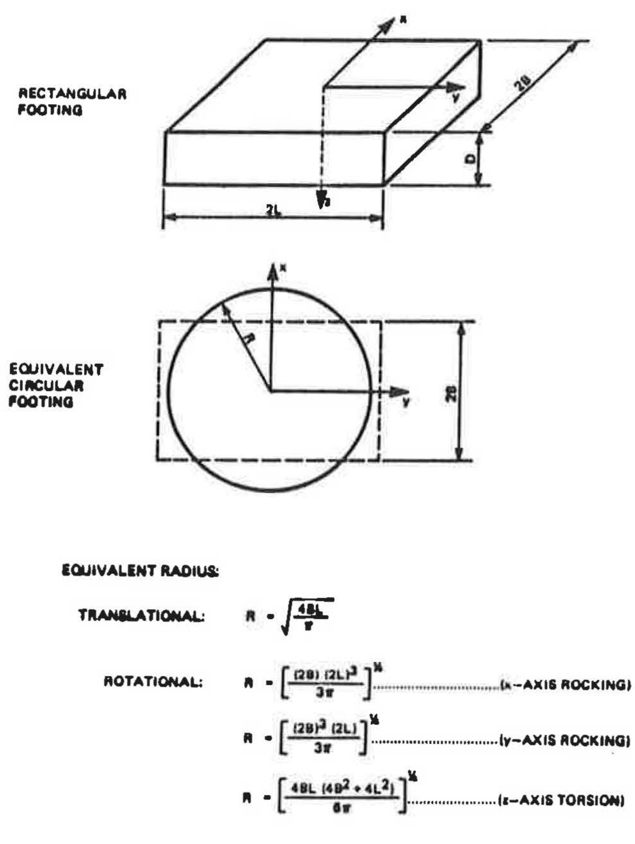


Figure 4. Procedures for Equivalent Radius of a Rectangular Footing

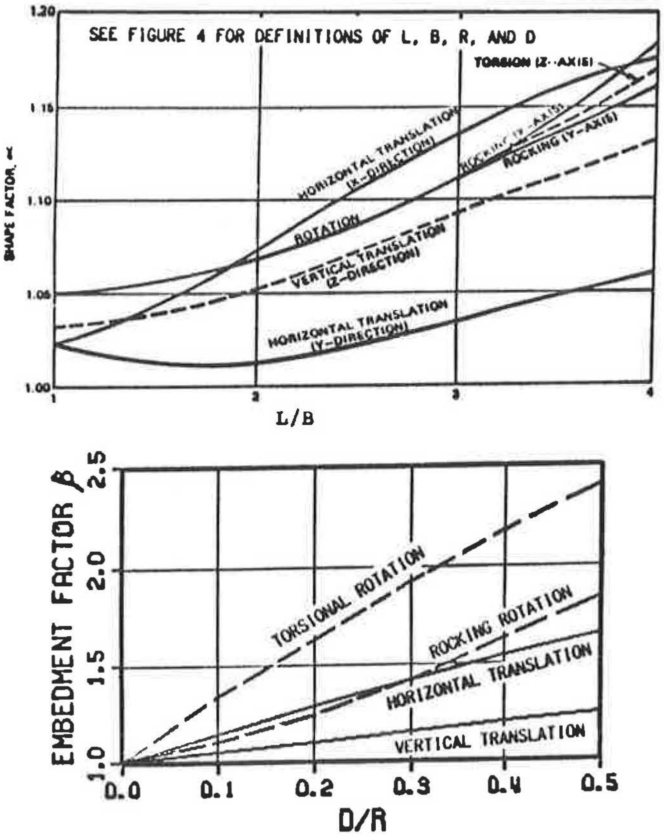
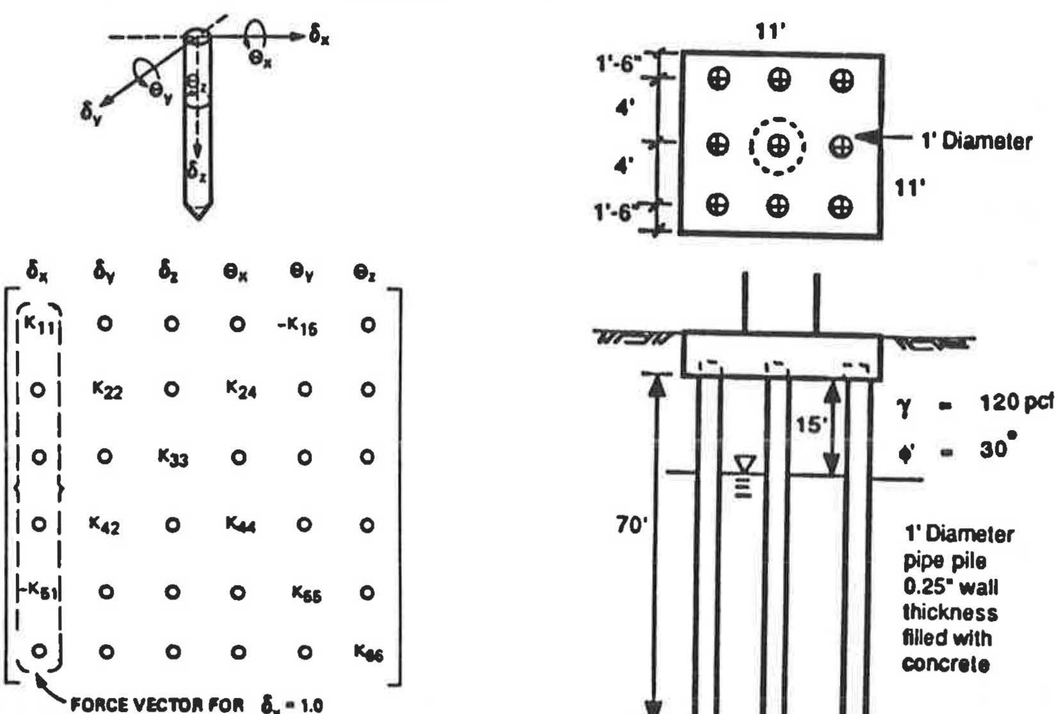


Figure 5. Design Charts for Footings

PILE FOUNDATIONS

Pile footings (pile group systems) are the most commonly used foundation systems for support of bridge structures. In a dynamic response analysis of an overall bridge structure, a pile foundation can be incorporated in the bridge model by several methods, including: (1) equivalent cantilever models, (2) uncoupled base springs models and (3) coupled foundation stiffness matrix models.

Among the three methods, the equivalent coupled foundation stiffness matrix model is the most general method of representation of foundation stiffness in a dynamic response analysis of the overall bridge. In fact, it can rigorously represent both the equivalent cantilever and the uncoupled base spring models. Because of its generality, it can represent all types of foundation systems including abutments, spread footings, pile groups and drilled shafts. The main draw back relates to the added effort to develop the coefficients in the stiffness matrix. Solutions of stiffness coefficients of a full 6 X 6 stiffness matrix as shown in Figure 6a are needed for this method. A simplified procedure has been developed for the solution of the stiffness matrix coefficients of pile groups.



(a) DEFINITIONS OF COORDINATE SYSTEMS

(b) EXAMPLE OF A PILE FOOTING

Figure 6. Form of a Pile Head Stiffness Matrix

The five basic steps involved in this simplified procedure for a pile footing such as that shown in Figure 6b are:

(1) Solve for the stiffness matrix of a single pile under lateral loading.

(2) Solve for the stiffness matrix of a single pile under axial loading.

(3) Superimpose the stiffness of individual piles to obtain the pile group stiffness.

(4) Determine the stiffness contribution of the pile cap (pile footing).

(5) Superimpose the stiffnesses of the pile cap to the pile group.

Details of the 5-Step Procedures are described below.

<u>Lateral Load-Deflection Methods</u>. Currently, in practice, the lateral pile-soil interaction problem is obtained by solving the problem of a beam member (modeled by finite elements, difference equations, or by discretized mechanical analog) supported on closely spaced linear or nonlinear elastic soil springs. Due to the dominance of the elastic pile stiffness over the nonlinear soil and the localized zone of influence (confined to the upper five to ten diameters), linear solutions were found to be adequate for pile stiffness evaluations (Lam and Martin, 1986). Most available linear non-dimensional solutions have been geared toward development of the total pile solution (including distributions of deflection, slope, moment and shear along the entire pile length). For the purpose of foundation stiffness evaluation, the total pile solutions can be simplified to provide coefficients of pile-head stiffness matrix as shown in Figures 7 through 9. Pile-head stiffness coefficients can be obtained for a combination of bending stiffness of the pile (EI) and the coefficient of variation of soil reaction modulus E_s with depth (f).

Recommendations for the coefficient f for sand are available in the literature (Terzaghi, 1955; and O'Neill and Murchison, 1983). At normal working load levels (pile-head deflection between 0.5 to 1.0 inch or 1.3 to 2.5 cm), Terzaghi's recommendation is considered reasonable for sand and his recommendation is presented in Figure 10. Recommendations for the coefficient f for clays have been developed from correlations of nonlinear computer solutions at a pile-head deflection (fixed head condition) of about 1-inch (2.5 cm) by

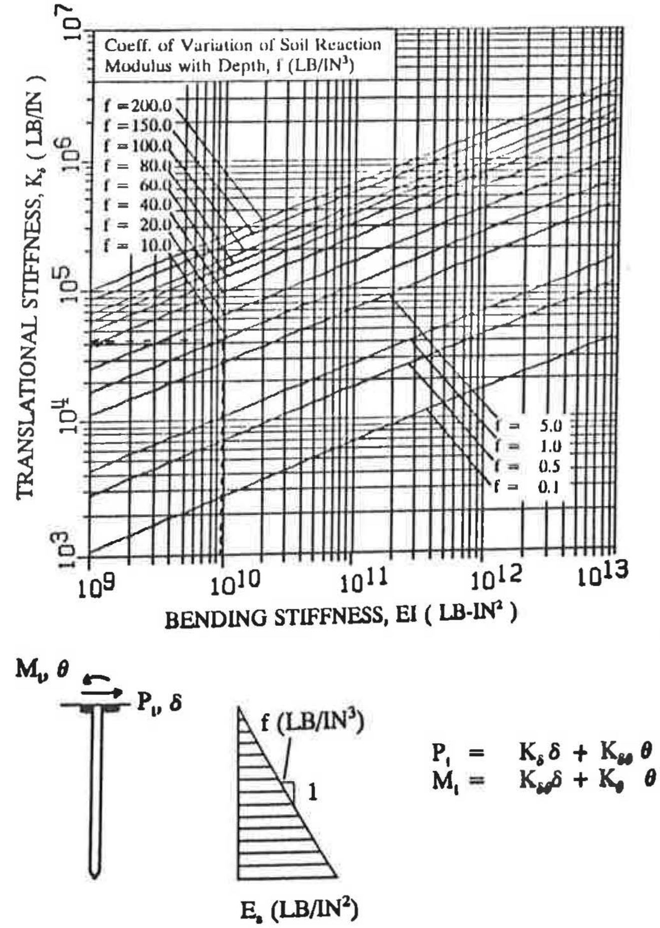


Figure 7. Pile Translational Stiffness, K_δ

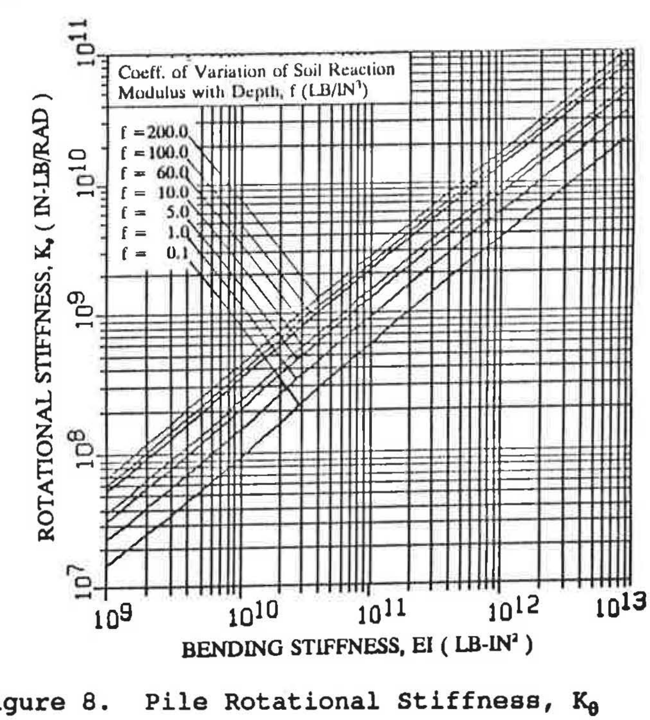


Figure 8. Pile Rotational Stiffness, K_θ

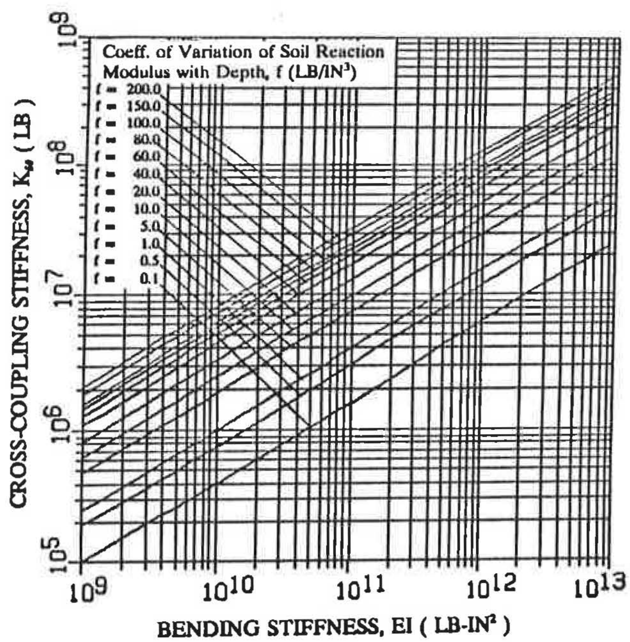


Figure 9. Pile Cross-Coupling Stiffness, $K_{\delta\theta}$

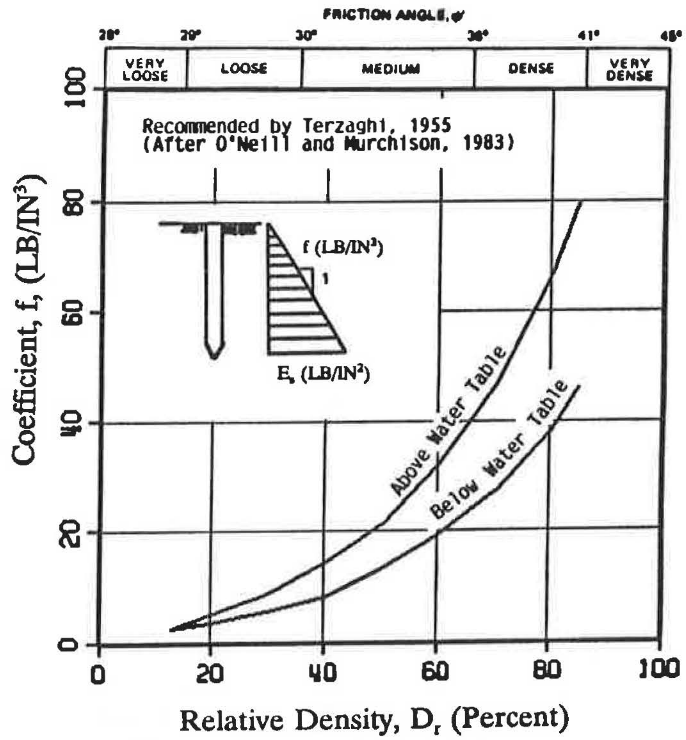


Figure 10. Recommendations for Coefficient f for Sands (Note: 1 LB/IN^3 = 0.27 N/cm^3)

the authors. This recommendation and results of the correlation for clay are shown in Figure 11. Only the upper five diameters of soils (soil type and ground water) need to be considered in usage of the presented design charts.

Limitations of Approach. There are several simplifying assumptions in the presented approach. The coefficient f is not an intrinsic soil parameter. The recommendations for f presented in Figures 10 and 11 are appropriate for piles in typical highway bridge foundations (i.e. smaller piles). Furthermore, the embedment effect has not been taken into account in the procedure. Therefore the recommendations are conservative and appropriate for shallow embedment conditions (say less than 5 feet or 1.5 m).

Although correlations for the coefficient f can be conducted for other conditions (e.g. larger piles and bigger embedment depths), the additional complexity negates the merits of the use of simplified linear elastic solutions. For such cases, computer solutions, which can readily accomodate nonlinear effects and more general boundary conditions, are recommended.

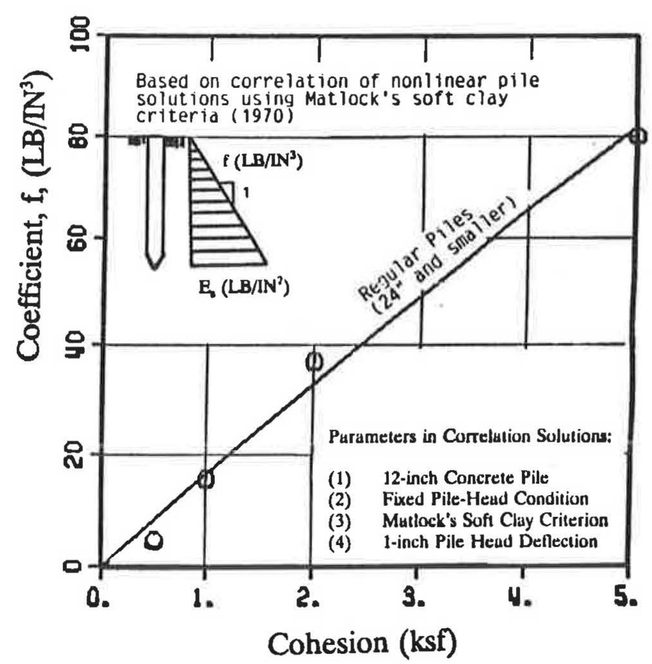


Figure 11. Recommendations of Coefficient f for Clays (Note: 1 LB/IN^3 = 0.27 N/cm^3)

Comparison to Caltrans Practice. The above procedure can be compared to the practice adopted by Caltrans. In Caltrans

Bridge Design Aids Section 14, a pile head stiffness of 40 kips per inch deflection (70 kN/cm) is recommended for a standard 16-inch (40-cm) CIDH (Cast-in-Drilled-Hole) pile. The bending stiffness (EI) of the above pile is estimated at 9.7×10^9 in^2-lb (2.8×10^{11} N-cm^2). For a typical sandy soil condition (friction angle = 30°), the coefficient f, obtained from Figure 10, is about 10 pci (2.7 N/cm^3). For the above combination of bending stiffness and subgrade stiffness, a lateral pile-head stiffness coefficient (K_δ) of 42 kips per inch (73.5 kN/cm) of deflection is obtained from Figure 7. The 42 kips per inch compares favorably with the Caltrans recommendation of 40 kips per inch (70 kN/cm).

<u>Sensitivity of Boundary Condition</u>. The above stiffness coefficient, K_δ, represents the lateral stiffness of a fixed-head pile (zero rotation). The rotational and the cross-coupling stiffness terms for the above (16-inch or 40.6-cm CIDH) pile can be obtained from Figures 8 and 9, respectively and summarized in the following paragraph:

- o Rotational stiffness (K_θ) (From Figure 8) = 2.3×10^8 in-lb/rad (26 MN-m/rad)
- o Cross-coupling stiffness ($K_{\delta\theta}$) (From Figure 9)= 2.3×10^6 lb (10 MN)

The above coefficients can be used to solve for the load/moment-deflection/rotation relationship for any combination of shear and moment or pile-head constraint condition. For example, if the pile head condition is a pinned head connection detail, the zero pile-head moment leads to the following relationship between pile head deflection (δ) and rotation (θ):

$$\theta = -(K_{\delta\theta} \,/\, K_\theta)\, \delta \quad \ldots\ldots\ldots \quad (3)$$

The above relationship can be substituted into the pile head force equation in Figures 7 through 9 for the lateral stiffness (ratio of force to pile-head deflection) of a free-head pile as presented below:

Lateral stiffness of free head pile
$= K_\delta - K_{\delta\theta}^{\,2} / K_\theta$ = 19 kips/in (33.3 kN/cm)

From the example, it can be observed that the lateral stiffness could vary from 42 to 19 kips per inch (70 to 33.3 kN/cm) for a fixed versus a free pile-head condition. It can be concluded that a realistic representation of the pile-head connection is very important, and often of more significance than the selection of soil parameters.

<u>Role of Axial Stiffness</u>. The role of lateral loading on piles is usually emphasized for earthquake consideration. However, in a pile group system, the base moment from structural loading is reacted largely by the moment couple from variation in axial load among individual piles as compared to reaction from pile-head bending. Experience from recent earthquakes and research have provided ample evidence (Rosenblueth, 1986; and Douglas et al. 1984) that the rotational stiffness of a pile group, which is related to the axial pile stiffness, will have a dominating effect on the overall structure as compared to the lateral stiffness. Therefore, evaluation of axial pile stiffness is critical for realistic modeling of foundation stiffness.

For a generalized soil-pile condition, the following factors need to be accounted for in evaluation of the overall axial pile behavior and pile head load-displacement characteristics:

- o The stiffness characteristics of the pile (AE) and pile length (L).
- o The shear transfer-displacement characteristic of the soil along the length of the pile shaft (related to the cumulative ultimate skin-friction capacity).
- o The end-bearing load-displacement characteristic of the soil at the pile tip (related to the ultimate end-bearing capacity).

In a normal design condition, a pile foundation derives a significant portion of the soil reaction throughout the pile shaft as well as the pile tip. Unless the pile is very lightly loaded, plastic slippage at the pile-soil interface will occur along a significant upper portion of the pile. Furthermore, the nonhomogeneous, layering nature of soil deposits must be accounted for in axial stiffness evaluation. Due to these complexities, linear analytical procedures would be of limited practical applications and nonlinear analyses are preferred for axial pile response.

<u>Solutions for Axial Stiffness</u>. Uncertainty in axial soil-pile interaction analysis relates largely to uncertainties in soil parameters including the ultimate pile capacity (skin-friction and end-bearing) and load-displacement relationships. Computer solutions can be used for a rigorous nonlinear solution. An approximate nonlinear graphical solution method has been developed and presented by Lam and Martin (1984, 1986). It will be described below. The procedure is

schematically shown in Figure 12 and involves the following steps:

(1) Soil Load-Displacement Relationships. Side-friction and end-bearing load-displacement curves are constructed for a given pile capacity scenario (accumulated skin-friction and ultimate tip resistance). Various forms of curve shape recommended by researchers can be used to develop the above load-displacement curves. Vijayvergiya's recommendation (1977) was adopted (for simplicity) in the example shown in Figure 12.

(2) Rigid Pile Solution. Using the above load-displacement curves, the rigid pile solution can be developed by summation of the side-friction and the end-bearing resistance values at each displacement along the load-displacement curves.

(3) Flexible Pile Solution. From the rigid pile solution, the flexible pile solution can be developed by adding an additional component of displacement at each load level (Q) to reflect the pile compliance. For the most flexible pile scenario, corresponding to a uniform thrust distribution along the pile shaft, the pile compliance is given by Equation-4.

$$\text{Pile Compliance } (\delta_c) = \frac{Q\,L}{A\,E} \quad \ldots\ldots \quad (4)$$

where L is the pile length; A is the cross-sectional area, and E is the Young's modulus of pile.

(4) Intermediate Pile Stiffness Solution. The "correct" solution, as indicated by the computer solution, is bounded by the above rigid pile and flexible pile solutions. In most cases, a good approximation can be developed by averaging the load-displacement curves for the rigid and the flexible pile solutions. The above graphical method can be used to solve for the load-displacement curve for any combinations of pile/soil situations (end-bearing and friction piles as well as any pile type: long or short and any pile material).

(5) Selection of Secant Stiffness. The secant axial pile stiffness appropriate for use in dynamic response analyses should reflect the unloading and reloading behavior. The secant stiffness from the origin of the load-displacement curve to the cyclic load level (without the static bias load) would be appropriate for dynamic response analyses.

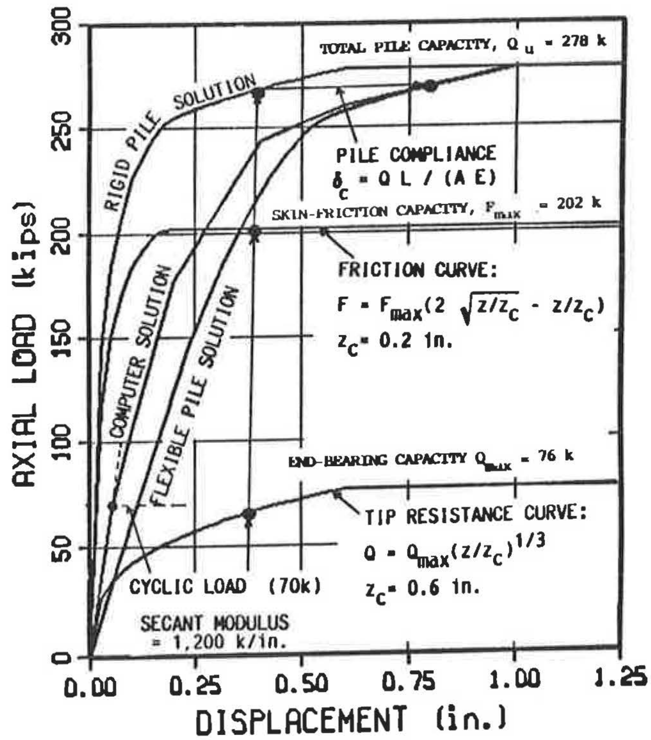


Figure 12. Graphical Solution of Axial Pile Stiffness

Stiffness Matrix of a Single Pile. Using the above described procedures, results of the pile-head stiffness coefficients of a single typical pile for the pile footing shown in Figure 6b are tabulated in Table 1. It should be noted that the torsional stiffness of a single individual pile can usually be ignored and assumed to be zero. The torsional moment on a pile group is usually reacted by the torsional moment couple from variation in lateral shears among individual piles in a pile group.

Stiffness for a Pile Group. The single pile stiffness matrix in Table 1 can be used in the next step to develop the pile group stiffness matrix. For a vertical pile group such as that shown in Figure 6b, the form of the stiffness matrix will be identical to the individual pile (as shown in Figure 6a). Also the stiffness summation procedure is relatively straight forward. For battered-pile systems, computer solutions are recommended. A PILECAP computer program that can be used to conduct the summation of individual

pile-head stiffness for an overall pile group stiffness matrix has been documented in the FHWA report by Lam and Martin (1986). The program can also be used to distribute the overall foundation load to individual piles.

For a vertical pile group, the stiffness for the translational displacement terms (the two horizontal and the vertical displacement terms) and the cross-coupling terms can be obtained by merely multiplying the corresponding stiffness components of an individual pile by the number of piles. However, the rotational stiffness terms (the two rocking and the torsional rotations) require consideration of an additional stiffness component. In addition to individual pile-head bending moments at each pile head, a unit rotation at the pile cap will introduce translational displacements and corresponding forces at each pile head (e.g. vertical forces for rocking rotation and lateral pile forces for torsional rotation). These pile-head forces will work together among the piles and will result in an additional moment reaction on the overall pile group. The following equation can be used to develop the rotational stiffness terms of a pile group:

$$R_g = N\, R_p + \sum_{n=1}^{N} K_{\delta n}\, S_n^2 \quad \ldots\ldots\ldots\ldots \quad (5)$$

where R_g and R_p are the Rotational Stiffness of the pile group and an individual pile, respectively. N is the No. of piles in the pile group. $K_{\delta n}$ is the appropriate translational stiffness coefficient of an individual pile, and S_n is the spacing between the n^{th} pile and the point of loading (center of the pile group).

The subscript $_n$ denotes the pile no. Summation is conducted for all the piles in the pile group in the above equation.

Using the described procedure, the pile-group stiffnesses of the overall pile group system shown in Figure 6 is developed and presented in Table 1. It can be observed that the rocking rotational stiffness coefficients of the pile group are dominated by translational stiffnesses of piles. The rotational stiffness from pile bending can virtually be ignored for most bent foundations.

The above presented procedure for a pile group does not account for the "group effects" which relate to the influence of the adjacent piles in affecting the soil support characteristics. There exists a wide range of opinions among geotechnical engineers on the significance of the "group effects". The importance of "group effects" would depend on many factors including the configuration of the pile group (number of piles, spacing, direction of loading in relation to the group configuration), soil types and pile installation methods. In view of the lack of evidence that "group effects" contributed to failure of bridges and the lack of well proven approaches for treatment of "group effects"; and above all for the sake of simplicity, we have neglected "group effects" in our presented procedures.

Stiffness of Pile Cap. So far, the presented procedure deals with stiffness of the pile-soil system. In a typical highway bridge situation, there will be additional stiffnesses arising from the pile cap. Our experience indicates that the lateral stiffness from (1) passive resistance on the vertical surface/s and (2) tractional shear forces at the base of a pile cap could be very significant as compared to the lateral pile stiffness. Wilson's equation for abutment stiffness presented earlier can be used for evaluation of the passive resistance component of cap stiffnesses. The spread footing procedure for an unembedded (surface) footing can be used for evaluation of the cap stiffness for soil reactions at the cap base. The pile footing example problem shown in Figure 6b is used to illustrate the relative stiffness of the pile group versus the pile cap. The stiffness contribution from piles have been developed earlier (See Table 1). The pile-cap stiffnesses including: (1) the lateral passive resistance component developed from Wilson's method (Equation 1) and (2) the 6 degrees-of-freedom soil reaction at the base of the cap (from spread footing Equations 2) are presented in the following table for comparison. A shear modulus of 7.2×10^5 psf or 34.5 Mpa (a conservative, or low value for compacted backfills in most construction practice) and Poisson ratio of 0.35 were used in the pile-cap stiffness evaluation presented in Table 2.

From Table 2, it can be concluded that, the vertical pile stiffness dominates the response of the vertical translational and the rocking rotational stiffnesses of the overall foundation. The influence of the pile cap is relatively minor for these two modes. However, the lateral footing stiffness is quite high relative to the lateral pile stiffness and dominates the lateral and the torsional rotation stiffness terms.

Table 1. Pile Stiffness Solution

Stiffness Coefficient	Single Pile	Pile Group
Lateral Translation $k_{11} = k_{22}$,(kip/in)	42	$9 \times 42 = 378$
Vertical Translation k_{33} , (kip/in)	1,200	$9 \times 1{,}200 = 10{,}800$
Rocking Rotation $k_{44}=k_{55}$ (in-kip/rad)	193,000	$N R_p + \sum_{n=1}^{N} K_{\delta n} S_n^2$ (Eq. 5) $= 1.74 \times 10^6 + 1.66 \times 10^7$ $= 1.83 \times 10^7$
Torsional Rotation k_{66}, (in-kip/rad)	0	$4 \times 42 \times 48^2 + 4 \times 42 \times (48^2+48^2)$ $= 1.16 \times 10^6$
Cross-Coupling $k_{15}=k_{51}=-k_{24}=-k_{42}$, (kip)	-2,250	$9 \times -2{,}250 = -20{,}250$

See Figure 6 for definition of stiffness coefficients and example problem. Note that the pile size is different from that used in earlier discussion on Caltrans standard 16-inch CIDH pile.

Table 2. Comparison of Pile vs. Cap Stiffness

Stiffness Component	From Piles	From Pile Cap: Base of Footing	From Pile Cap: Passive Pressure Vert. Face
Lateral Translation (k/in)	378	1,833	1,167
Vertical Translation (k/in)	10,800	2,333	N/A
Rocking Rotation (in-k/rad)	1.8×10^7	5.2×10^6	N/A
Torsional Rotation (in-k/rad)	1.2×10^6	1.2×10^7	N/A
Cross Coupling Between (k) Lateral Trans. and Rocking Rotation	2.0×10^4	0.0	N/A

See Figure 6b for configuration and Table 1 for pile stiffnesses.

Notes: 1 k/in = 1.75 kN/cm; 1 in-k/rad = 11.3 cm-kN/rad; 1 k = 4.45 kN

Significant engineering judgement is required on the use of the above estimated stiffness coefficients for a pile footing. Considerations regarding interaction between piles and the footing and other factors such as the contact condition between the cap bottom and soil in the presence of the piles introduce uncertainties on the use of the footing base stiffnesses (the second column Table 2). However, it would be prudent to add the pile-cap lateral passive soil resistance (the third column) to the pile stiffness in design practice. Caltrans have adopted similar view point in their design practice. Ignoring the footing base is considered conservative. It can be observed that even if the bottom tractional stiffness is ignored, the lateral passive resistance of the pile cap would dominate the lateral stiffness of the pile group.

CONCLUSIONS AND RECOMMENDATIONS

Procedures and accompanying design charts to facilitate practical solutions of abutment and foundation stiffnesses for dynamic response analyses of typical highway bridges have been developed and presented. The presented procedures are relatively simple and emphasis has been placed on hand and graphical solution methods for easy application. An in-house Earth Mechanics project to computerize the presented procedures is presently being undertaken.

In view of uncertainties in ground motion which could lead to differential foundation movements and other geotechnical concerns (e.g. ground stability and liquefaction problems), ductility design has significant technical merit for earthquake resistance. Allowance for ductility tends to lead to a design more tolerant to foundation movements. However, ductility design requires realistic evaluation of the magnitude of displacements and deformations and design provisions (e.g. allowance of minimum seating widths as recommended by Caltrans) to accommodate the displacements.

Incorporation of foundation and abutment stiffness in design and retrofit analyses of highway bridges leads to an improved solution of the overall seismic load level and the distribution of the overall load among various bents and abutments. More importantly, it leads to better estimates of displacements. However, uncertainties in ground motion and other geotechnical concerns warrant an even more prudent approach to provide for potential bridge displacements. This has led to the specification of minimum seat widths by Caltrans and bridge designers should be cognizant of this issue. Provision of a sufficient seat width represents excellent earthquake design practice and can be accommodated economically for most new designs. The role of foundation stiffness becomes more important in retrofit situations especially when a more realistic analysis approach is warranted to reduce the level of conservatism associated with uncertainties in analytical procedures.

The above procedures on foundation stiffness can also be applied for temperature loading evaluations. Significant reduction (relaxation) in the structural stresses can usually be realized if the foundations are allowed to deform. Therefore introduction of foundation stiffnesses in bridge analysis would usually lead to a more economical design.

There are other geotechnical considerations, especially those related to ground stability that have not been addressed in this paper. Cooperation between structural and geotechnical engineers is strongly recommended to address such issues, especially for poor soil conditions. In addition, a number of sensitivity issues have been discussed in this paper. Although an attempt has been made to ensure our example problems reflect real typical situations, one must be careful in extrapolating the presented discussions and results to other design conditions.

ACKNOWLEDGEMENT

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Enhancing the Seismic Performance of Toll Road Bridges

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Caltrans and AASHTO highway bridge criteria focus on collapse prevention during an extreme seismic event, and allow or require substantial damage to occur as part of the mechanism resisting extreme earthquakes in high seismic zones. Such damage implies possible extended closure of the facility until repaired, and, in the case of toll roads, loss of revenue until traffic is restored. A special study was made to determine the feasibility of bridge design criteria which will obtain a higher degree of seismic reliability for the Orange County, California toll roads. The study included a seismic hazard analysis of the area, development of site specific response spectra with several different probabilities of exceedance, a review of Caltrans criteria, and evaluation of various strategies for improving the structural performance or controlling damage. The principal strategies considered were: a two level design approach; the use of energy dissipators at abutments only; seismic isolation; and lighter weight superstructures. Evaluations were made by preparing a range of bridge designs using a linear elastic analysis for strength design and a nonlinear analysis to assess performance. Cost estimates were prepared using the results of the analytical work. Criteria are proposed for implementation in the design of bridges in the Orange County, California toll road corridors.

The first U.S. toll roads in a high seismic area are being built by the Transportation Corridor Agencies (TCA) in Orange County, California. Funded in part by bonds, the reliability of the bridges is important to the revenue base of the project. Once opened to traffic these 65 miles of new roads will be turned over to the California Department of Transportation (Caltrans) for traffic operation and maintenance. Following the October 17, 1989 Loma Prieta earthquake, TCA sponsored a study to determine the feasibility of using seismic design criteria for bridges which would exceed the minimum requirements of Caltrans and thereby improve the performance of the structures during moderate earthquakes. An additional objective was less damage and shorter closure time to repair damage from an extreme event.

Computech Engineering Services, Inc. (CES), Imbsen & Associates (IAI) and Woodward/Clyde Consultants were contracted to develop such criteria. Woodward/Clyde Consultants performed a seismic hazard analysis of the area and developed site-specific response spectra with different probabilities of exceedance. CES and IAI were responsible for investigating the performance and cost implications of different design strategies that would achieve the basic objectives of the study, and then develop project-specific design criteria.

Elements of the study included a review of Caltrans design criteria, consideration of a two level design approach, the use of energy dissipators at abutments, seismic isolation and lighter weight superstructures. The cost and performance of these various strategies are calculated and compared to possible repair and closure costs.

The analytical phase of the study consisted of the design and analysis of two basic bridge configurations; a typical three-span bridge with 22-foot-high columns and a nine-span bridge with 59-foot-high columns. The three-span bridge was designed with both steel and concrete superstructures, providing a total of three primary structural configurations. The columns of each bridge were designed with various levels of strength (Z-Factors). As alternate design strategies, each of the above configurations, incorporated lead rubber isolation bearings throughout, and as an energy dissipation mechanism only at the abutments. An analysis matrix was developed to provide a range of bridge designs for performance assessment and criteria development. The matrix was separated into two distinct phases; linear elastic response spectrum analysis to establish adequate strength, and full nonlinear analysis to assess performance. Within the linear elastic phase, bridge designs were developed using standard Caltrans design strategies with varying damage and risk (Z-Factor) adjustment factors. Each of the designs were then modeled with all structural elements exhibiting their non-linear characteristics and subjected to a series of spectrum compatible time histories in order to establish their performance level.

The analytical results were used to develop cost estimates of initial construction, cost to repair column earthquake damage, and estimates of the closure costs while significant column damage is repaired. Abutment damage was not assessed.

Based on the cost and performance evaluation, the project-specific design criteria was developed to consist of a two level procedure. Although this is a departure from current Caltrans and AASHTO practice, it meets the objective of the study as it provides a higher and more uniform level of safety and reliability. Briefly, the design criteria requires two levels of analyses: one corresponding to a 72 year return period event and the other corresponding to a maximum credible design event. Design forces and displacements resulting from the lower level event will be used so that minimal damage will occur as a result of this event. The forces and displacements resulting from the Maximum Credible analysis will be used to

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protect against the structure collapse.

SEISMIC HAZARD STUDY

A map showing the location of the three transportation corridors is presented in Fig. 1. Included in this figure are faults in the area and seismic contours showing the acceleration levels developed by the California Department of Mines and Geology which vary form 0.6g to 0.3g. Woodward-Clyde performed a seismic hazard analysis of the three corridors which resulted in uniform risk site specific response spectra with varying return periods for five different sites (A to E) shown in Figure 1.

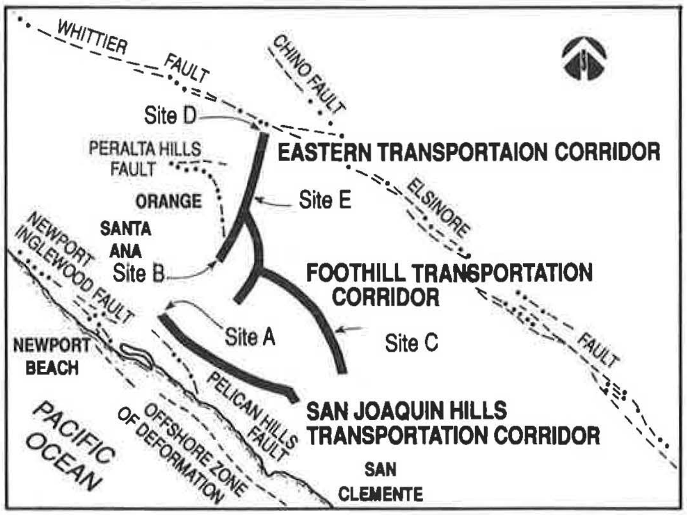


FIGURE 1 Location Map

In a seismic hazard study several terms are used which are defined for the purpose of clarity.

a. Response Spectrum - a plot of the maximum earthquake response with respect to natural period or frequency of the structure. Response spectra can show acceleration, velocity or displacement. This paper will only include acceleration response spectra.
b. Return Period - an appropriate response spectra with a return period of 72 years means that such an earthquake will occur approximately every 72 years. Because of the major uncertainties involved with earthquakes an event larger than the return period event may occur and therefore return periods are also expressed in a more meaningful term called probability of exceedance.
c. Probability of Exceedance - this expresses the probability that a given event will be exceeded in a certain time frame. For example, a 72 year return period response spectra has a 50% chance of being exceeded every 50 years. See Table 1 for the relationship between return period and probability of exceedance.
d. Maximum Credible Spectrum - a maximum credible response spectrum is a measure of the maximum amount of energy that can be released by a given fault.

TABLE 1 Return Period & Probability of Exceedance

	Probability of Exceedance	
Return Period	*in 50 Years*	*in 100 Years*
25 Years	86%	98%
50 Years	63%	86%
72 Years	50%	75%
150 Years	28%	49%
250 Years	18%	33%
475 Years	10%	19%
2500 Years	2%	4%

Fig. 2 is an example of a response spectra plot at a typical site, showing the relative magnitude of spectra with return periods of 25, 50, 72, 150, 475, and 2500 years as well as the appropriate Caltrans Maximum Credible Spectra. Fig. 3 compares the 475 year return period spectra at the five different sites. Note there is a factor of 2 or more difference between extreme sites. Site A is different from the other 4 sites in that it has a soft ground condition.

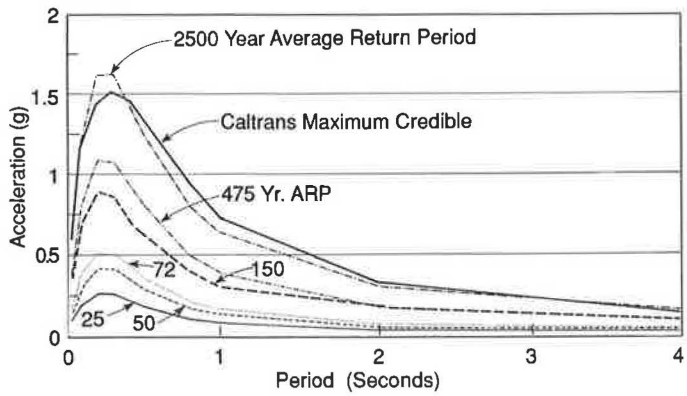


FIGURE 2 Site Specific Response Spectra - Site E

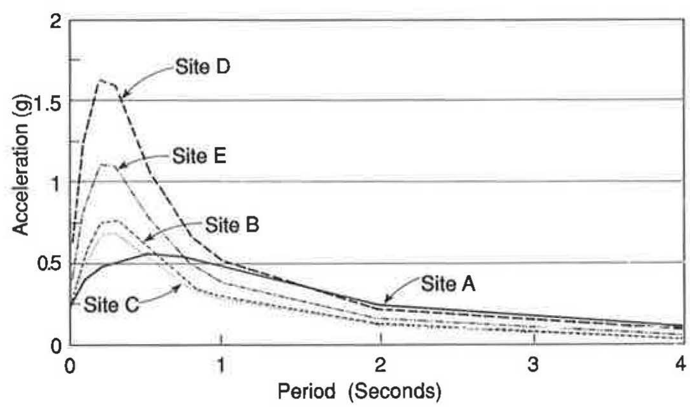


FIGURE 3 Site Specific Response Spectra - 475 Yrs ARP

CURRENT CALTRANS DESIGN PHILOSOPHY

Both current U.S. seismic highway bridge design codes (AASHTO and Caltrans) are one-level design approaches, i.e.,

an analysis is performed for only one level of response spectra and all design forces and displacements are derived from this analysis. The primary advantages of a one-level design approach is its simplicity. The disadvantages are that the elastic capacity of the columns, shear keys, etc. correspond to a lower-level design event that has an unknown return period or probability of exceedance. The primary differences between the Caltrans design requirements and the AASHTO requirements are that Caltrans uses a maximum credible response spectrum for the analysis, whereas the AASHTO Specifications use a 475-year return period spectrum. In addition, the reduction factors used for column design Z-Factor in Caltrans, a R-factor in the AASHTO Specifications) are also different. For Caltrans, the Z-Factor (Fig. 4) varies between 8 and 4, depending on period, for multi-column bents, and between 6 and 3 for single columns. In the AASHTO Specifications, the R-factors are 5 and 3 for multi-and single-column bents, respectively.

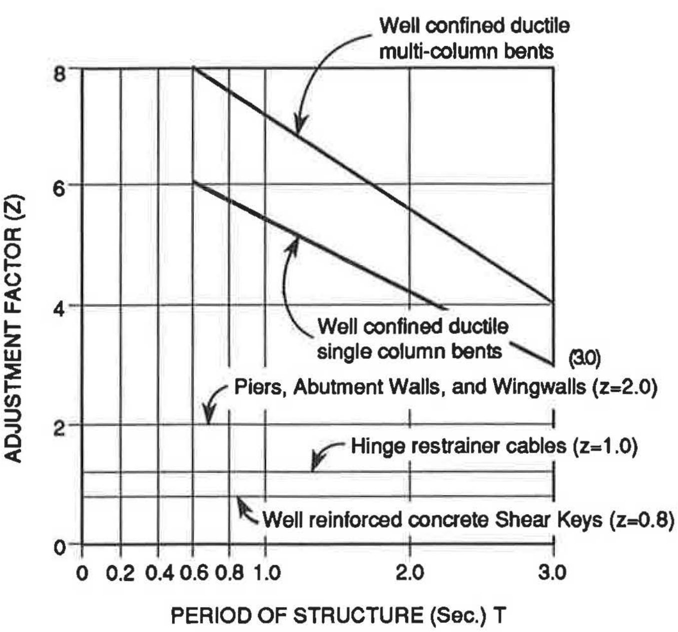


FIGURE 4 Adjustment of Ductility and Risk (Caltrans)

Design Strategies

For monolithic bridges with multi-column bents, the current Caltrans design philosophy is to provide a fully pinned connection at the base of the column. This evolved as a cost-saving measure due to requirements embodied in AASHTO Specifications and the Caltrans design criteria. The current design codes state that the foundations must be designed for the maximum forces developed by the formation of column plastic hinges. This design requirement was developed to ensure that any damage that does occur in a bridge is readily detectable and repairable. The disadvantage of the pinned column base design philosophy is that redundancy is reduced and the ductility demand on the plastic hinge that will form at the top of the column will be increased. The redundancy reduction also eliminates an energy-dissipating mechanism, i.e., column plastic hinge; thus, if one joint should fail, there are fewer joints to provide a resisting mechanism.

For larger bridges, a thermal separation is generally required at the abutment. An earlier Caltrans design concept was to permit the box girder to impact the backwall of the abutment, thereby mobilizing the soil backfill. Empirical stiffness relationships were developed for the abutment-soil-pile interaction in order to estimate the displacements that occurred. The problem with this concept is that significant damage may occur to the abutment backwall and abutment piles in addition to potential damage to the wing walls. This concept was modified to provide a knock-off or release mechanism at the bottom of the abutment backwall (see Fig. 5). This concept has the advantage of permitting the soil backfill to be mobilized as a resisting mechanism. The disadvantage is that it is very difficult to inspect the damage that may occur at the knock-off plane. Damage may still occur in the wing walls and the abutment backwall. In the transverse direction, shear keys are generally provided at the abutments. If their ultimate capacity is below the ultimate capacity of the piles in the transverse direction, the transverse shear keys will act as a force-limiting mechanism to protect the piles.

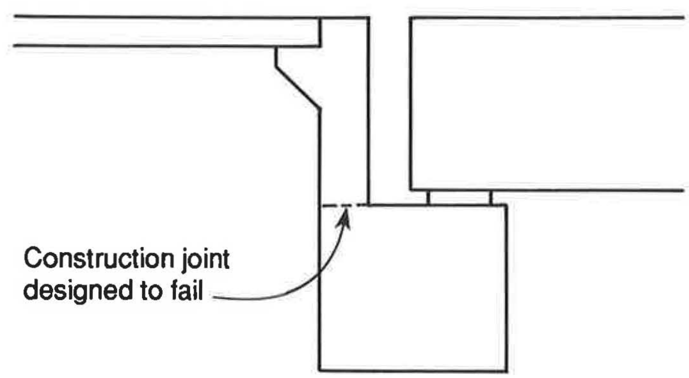


FIGURE 5 Bottom of Backwall Knockoff Detail

Return Period Implicit in Column Z-Factors

In the Caltrans design criteria, the Z-Factor is a reduction factor applied to the forces of a given structural component from an elastic analysis using the maximum credible earthquake. These reduced forces are then used for the design of the component. The Z-Factor thus becomes a measure of the ductility demand required in columns or other members critical to seismic loading. As shown in Fig. 4 the Caltrans Z-Factor also adjusts for risk by requiring greater strength for longer period (tall column) structures.

Even though the Caltrans procedures use the Maximum Credible Spectra for design, it is possible to estimate the return period for which such a design produces a purely elastic response in the columns. Two methods can be used:

1. Divide the Caltrans maximum credible spectra by the col-

umn Z-Factor and compare this reduced spectra with various site specific return period spectra. Fig. 6 is such a plot for Site E.

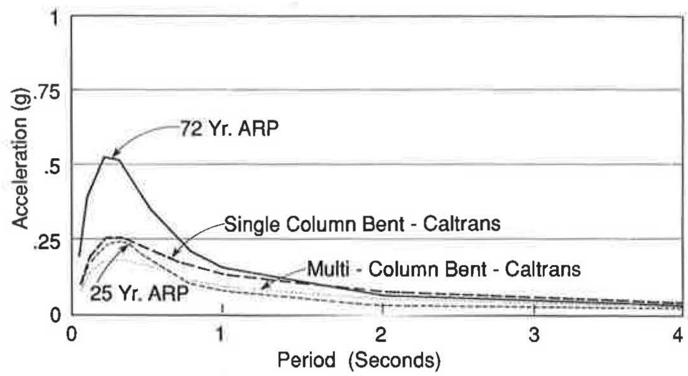


FIGURE 6 Maximum Credible Spectra Divided by Z-Factor - Site E

2. Divide the Caltrans maximum credible spectra at a given period by a given return period site specific spectra at the same period to obtain the Z-Factor implicit in the given return period spectra. Fig. 7 plots the results for Site E for the 72 Year event.

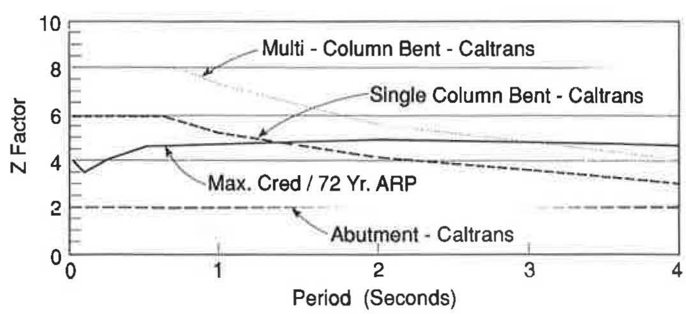


FIGURE 7 Comparison of Z-Factors - Site E (72 Yrs ARP)

This analysis of implicit Z-Factors shows the approximate range of elastic behavior which results from a design using Caltrans minimum standards. For multi-column bents current Caltrans minimum elastic design corresponds to less than a 25 year return period for bridges with a natural period less than 0.8 seconds and a 50 year return period for natural periods between 0.8 to 2.0 seconds. For single column construction the current Caltrans minimum elastic design corresponds to a 50 year return period for bridge periods less than 0.8 seconds and a 72 year return period for bridge periods between 0.8 and 2.0 seconds. By means of comparison Department of Defense essential facilities including hospitals require elastic design for a 72 year return period event.

ALTERNATE DESIGN STRATEGIES

Several different design strategies were evaluated as part the investigation to develop design criteria with a higher level of safety and reliability. These were as follows:

1. While retaining the Caltrans Maximum Credible design earthquake and a one level design approach use a lower Z-Factor in the design of the column. This will increase column cost but reduce ductility demand. The threshold of damage will be increased, and the severity of damage at extreme events reduced.

2. Return to the use of moment-resisting connections at the column bases of multiple column bents. This adds more energy dissipating mechanisms to the structure but increases the cost of the foundations.

3. At abutments, use a knock-off detail at the top of backwall to limit damage, rather than at the base of the backwall which is current Caltrans practice. A detail such as shown in Fig. 8 has been tested and used extensively in New Zealand.

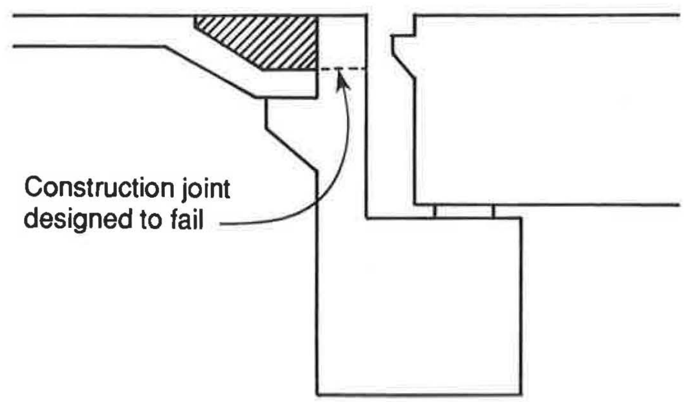


FIGURE 8 Top of Backwall Knockoff Detail

4. At abutments design transverse shear keys to release before damage to wing-walls or abutment piling can occur.

5. Energy Dissipation Devices - In Caltrans current design philosophy low height elastomeric bearings are generally used at the abutments. An alternative consideration is to use energy dissipating devices or bearings at the abutment to increase the amount of damping at this location. Significant additional damping will decrease the column forces, superstructure displacements and column ductility demands by 30 to 50%.

6. Seismic Isolation - This design concept works by reducing the seismic forces that the column and abutments must resist. The isolation bearings lengthen the period of the bridge and add a significant amount of damping. The

concept is quite different from current Caltrans practice in that a bearing must be introduced at either the top or bottom of all columns and at the abutments. The actual displacements that result from the use of seismic isolation are not very different from those that result from a non-linear analysis of a current Caltrans design The advantages of the seismic isolation design concept are that damage to the columns can be eliminated and forces on the abutments can be significantly reduced.

7. Two level Design Strategy-as previously noted, current U.S. Bridge design codes use a one level design approach. For a two level design approach which is used in some other seismic codes, an analysis is performed for an upper and lower level design event. The upper level response spectra can be, for example, either a maximum credible or 2500 year return period event and a lower level response spectra is a 72 year or 150 year return period event. The lower level results would be used together with design requirements to ensure that there is no significant damage for that event. Columns for example would be designed using a strength design approach for forces that resulted from this lower level analysis. The upper level results would be used to ensure that there is no collapse potential under a severe earthquake. The primary advantage of the one level design approach is that it is similar to current Caltrans practice and there is only one analysis to be performed. The disadvantages are:

   a. The return period at which onset of damage would occur is difficult to determine.
   b. There is no logical basis to determine what the longitudinal abutment gaps should be before the knock-off device at the abutment is activated.
   c. There are problems in deciding whether to include or exclude the transverse abutment shear keys in the analytical model.

The philosophical advantage of the two level approach is that it directly considers the behavior of the structure during events that are almost surely to occur in its lifetime as well as safeguarding against collapse during an extreme event. The disadvantage with this approach is that two dynamic analyses are required. The two level design approach is currently required on all Department of Defense essential facilities with the lower level event specified as a 72 year return period event. For a design criteria with a higher degree of reliability the advantages of a two level approach are:

   i. Design forces for all critical components can be used to ensure that the bridge will have no significant damage for the lower return period.
   ii. Transverse shear keys can be included in the analytical model for the lower level design event. The forces imposed on the key can be used to design its capacity as well as ensure that the capacity of the piles and the wing walls exceed the capacity of the shear keys.
   iii. The displacements that occur in the longitudinal direction can be used to size the abutment gaps such that the knock-off detail is not activated for this lower level event.
   iv. For the upper level design event there would be no confusion on what to do with the transverse shear key in the analytical model, since it would no longer be effective for this event.

PERFORMANCE AND COST STUDY

Coupled with the investigation of different design philosophies it was also necessary to determine what increase in performance (i.e., reduction in damage) was achieved with the different design strategies and how this related to both cost of column repair and initial cost of construction. In order to achieve this objective a number of different designs were performed on a 3 and 9 span bridge configuration. Each design was analyzed incorporating a full non-linear analytical model of the structure to evaluate its performance for varying return period events. Bridge configurations used in the study were:

1. A 3-span 155'-190'-155' bridge with 22' high columns and 83-½ ft. deck width.

2. A 9-span (7 at 140'and ends spans of 150,'and 140') bridge with 59' high columns and 71 ft. deck width.

3. Super structure types were:
   a) Concrete box girder monolithic with columns for both the 3- and 9-span configurations. The concrete box girder had two columns fixed at the superstructure and fully pinned at the foundation. Columns with fixed based were also included in the investigation.
   b) Steel Girder connected with pins to the cap beam for the 3-span configuration. The weight of the steel superstructure was approximately ⅔ of the concrete box girder. The steel superstructure had a cap beam connecting the two columns. The columns were fixed at the foundation level.

4. Energy Dissipators at Abutments - The 3-span concrete configuration was designed and analyzed with the columns in their conventional configuration, but with energy dissipating bearings at the abutments.

5. Seismic Isolation Designs - Each of three primary configurations were designed and analyzed in a fully isolated and conventional configuration. The isolated concrete box girder superstructure had the isolators at the abutments and at the base of the columns. Similar results would be obtained if they were at the top of the columns. The steel superstructure had the isolators under the girders.

6. Column Design - The columns were designed for the Caltrans maximum credible response spectra for Site E. The following Z-Factors were used with octagonal columns.
   a) 3-span Concrete Box Girder - Z = 7.5, 5 and 3
   b) 3-span Steel Superstructure - Z = 5 and 3.25
   c) 9-span Concrete Box Girder - Z = 4.5, 2.75 and 1.0

The 3-span concrete box girder bridge was also analyzed with a rectangular column with a Z-Factor of 7.75.

It was determined from preliminary studies that a bridge designed for a given Z-Factor for one site and subjected to a 2500 year return period seismic input for that site produced results that were reasonably similar to using the same Z-Factor at a different site and subjecting the bridge to the 2500 year return period for that site. As a consequence, this simplified the number of analysis and enabled the study to focus on the design and seismic input for just one Site. Site E was selected.

The seismic input used for the nonlinear time history analysis for Site E used the following return period events 2500; 475, 150, and 72 years. For each return period six spectrum compatible time histories were developed. Each set of six time histories included three orthogonal components (two horizontal, 1 vertical). Each set was frequency scaled such that two horizontal components were compatible with the appropriate site specific spectra and the vertical component was compatible with two-thirds of the horizontal component. All analyses were performed with 100% of each of the two horizontal components and the full vertical (⅔ horizontal) component.

SUMMARY OF ANALYTICAL RESULTS

A small selection of the analytical results are shown in Figs. 9, 10, and 11 for the conventional 3-span bridge with a concrete superstructure with varying Z-Factors 7.5, 5 and 3 with octagonal columns and for a 7.75 Z-Factor for a rectangular column. Figs. 12, 13, and 14 are similar results for all 3 global design options, conventional, with a Z-Factor of 5, seismic isolation and energy dissipators at the abutments. For the 9-span bridge with the conventional concrete superstructure a selection of results are shown in Figs. 15 and 16 with variations in Z-Factors. Although isolation was included in the study of the 9-span bridge it is not a good seismic isolation candidate, because the tall (59 ft. high) piers provide a flexible bridge without the incorporation of seismic isolation.

Three Span Bridge

Three global design options have been examined in terms of their impact on the response of the 3-span bridge. The improvement in seismic performance has been assessed in terms of both the columns and the abutments. From a design strategy perspective the results are summarized as follows:

The column performance is assessed in terms of column ductility demands.

1. Z-Factor - the column ductility demand reduces as the Z-Factor decreases. Up to a 50% reduction in ductility demand was achieved as Z decreased from 7.5 to 3. Thus increasing the column strength improves the performance of the columns.

2. Use of Seismic Isolation - this provided the most dramatic improvement in column performance in that the column ductility demand was eliminated.

3. Use of Energy Dissipators at Abutments - this provided up to a 40% reduction in column ductility demand with more significant reductions achieved with higher Z-Factors.

The abutment performance is more difficult to assess, since it is dependent upon the strategy adopted for the design. Displacements at the superstructure level are used to assess the potential abutment performance.

1. Z-Factor - in general the abutment performance will improve, although not significantly, with decreasing Z-Factors, since at higher return period events displacements are not reduced significantly. In terms of an abutment design strategy, changes in the column Z-Factor will not have a significant impact on the design forces or displacements.

2. Use of Energy Dissipators - this design option has the most dramatic impact in terms of abutment performance since it provides up to a 50% decrease in the superstructure displacements. It also provides a 40% reduction in the forces the abutments are required to resist.

3. Use of Seismic Isolation - compared to conventional Caltrans abutment design the primary benefit provided by the use of seismic isolation are the reduced forces for the abutment design. Design displacements are similar to the conventional design and therefore the performance in terms of displacements will be dependent on the design strategy and detailing (knock-off detail, engagement of backwall, etc.).

Nine Span Bridge

The 9-span bridge was designed with the conventional Caltrans configuration with 2 intermediate hinge joints and three different Z-Factors (1.0, 2.75 and 4.5). The bridge had reasonably tall columns (59 ft. high) and thus was quite flexible. Consequently, it would not be considered to be a good candidate for seismic isolation or the use of energy dissipators at the abutments.

The column ductility demands, as with the 3-span bridge, decrease as the Z-Factor reduces. There is no actual ductility demand for either the Z=2.75 or Z=4.5 designs for the 72 Year event and there is approximately a 25% reduction in ductility demand as Z decreases from 4.75 to 2.75 for the 2500 Year event.

In assessing the abutment performance greater clearance will be required for the 72 Year event, compared to the 3-span bridge with shorter columns, if abutment backwall engagement is to be avoided. The displacements in the longitudinal direction are reduced by approximately 30% as Z decreases from 4.5 to 2.75 for the higher level events. Depending upon the abutment design strategy this would improve the abutment performance. In the transverse direction the displacements are also reduced, but not as significantly.

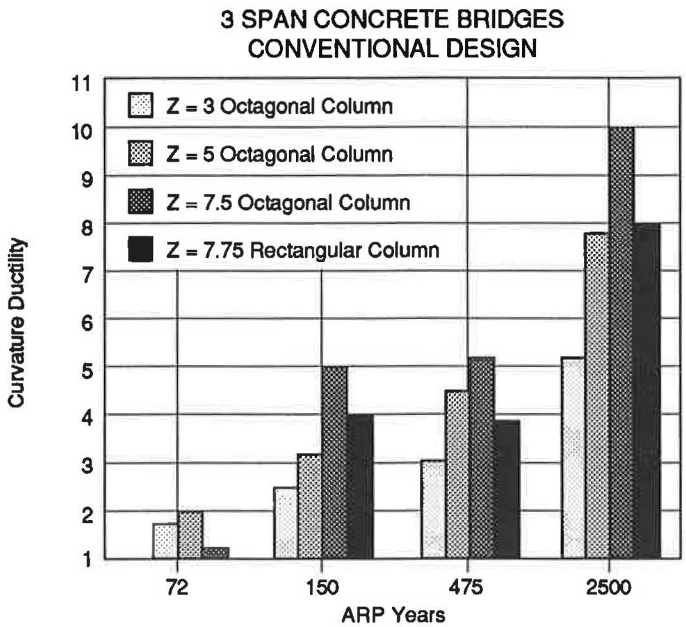


FIGURE 9 Curvature Ductility Demand

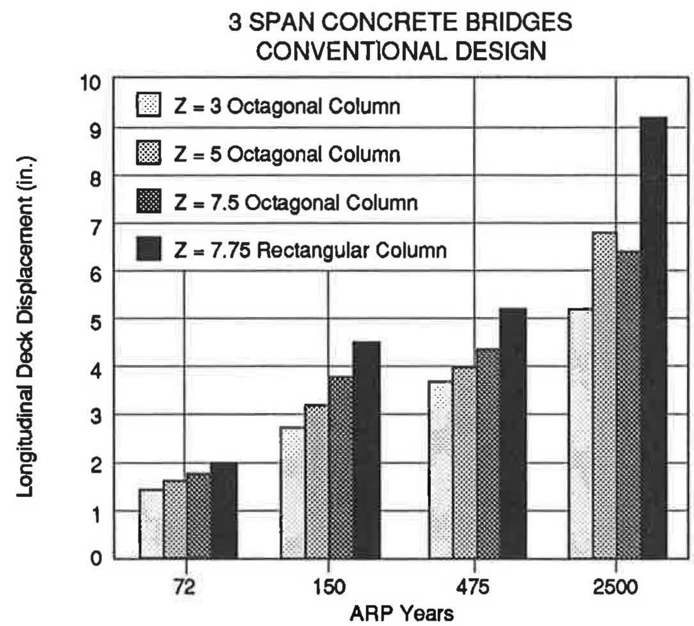


FIGURE 10 Longitudinal Deck Displacement

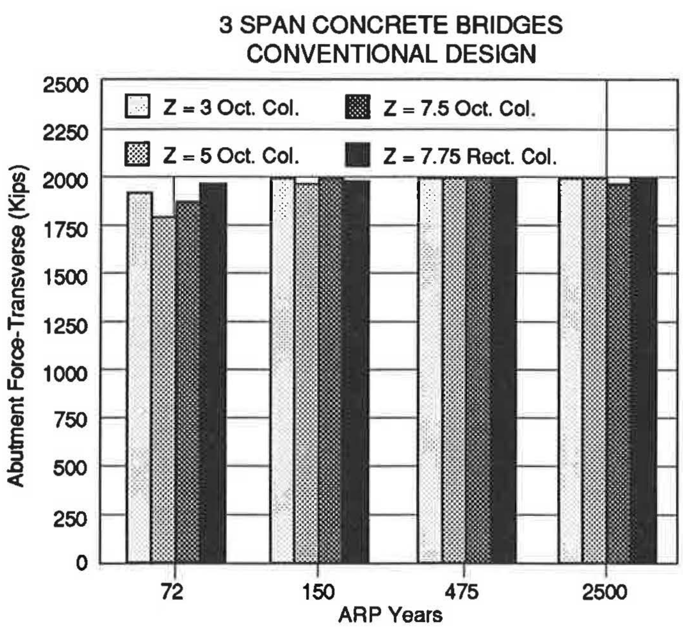


FIGURE 11 Transverse Abutment Force

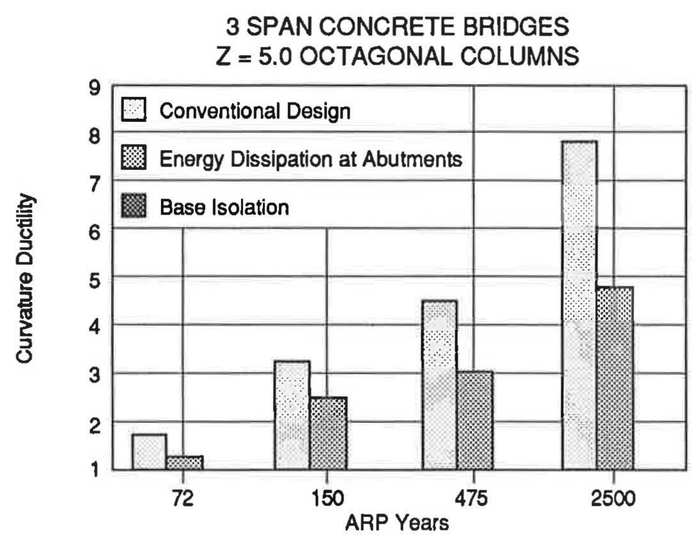


FIGURE 12 Curvature Ductility Demand

Effects of Vertical Accelerations

To assess the impact of vertical acceleration one of the models (the three span concrete bridge) was selected and analyzed both with and without the vertical earthquake component of ground motion. The analyses show that the vertical accelerations have a marked effect on the vertical deck displacements and bending moments. The difference in moments in each span between the analyses with and without vertical earthquakes is equivalent to the moments caused by a uniform load approximately equal to the dead load.

The conclusion from this limited study of vertical effects is that bridge girders may be in the period range where vertical amplification may occur and so the vertical component of motion should be included in the response spectrum analysis. The added forces from this component may affect the design of the girders and column. The vertical earthquake effects are less likely to influence the ductility demands on the columns.

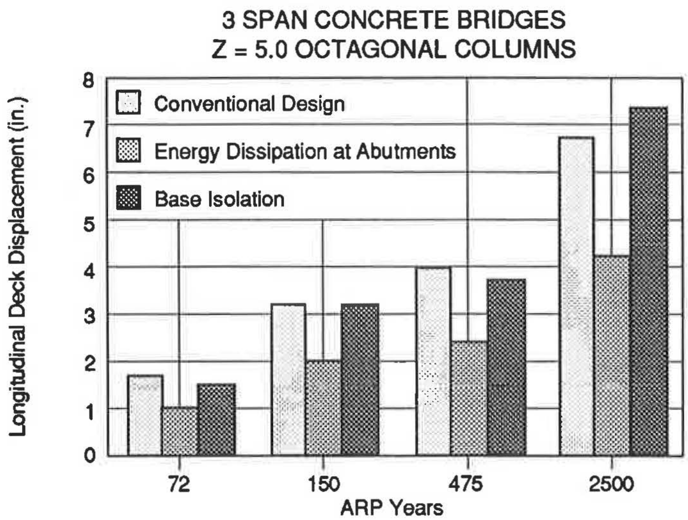


FIGURE 13 Longitudinal Deck Displacement

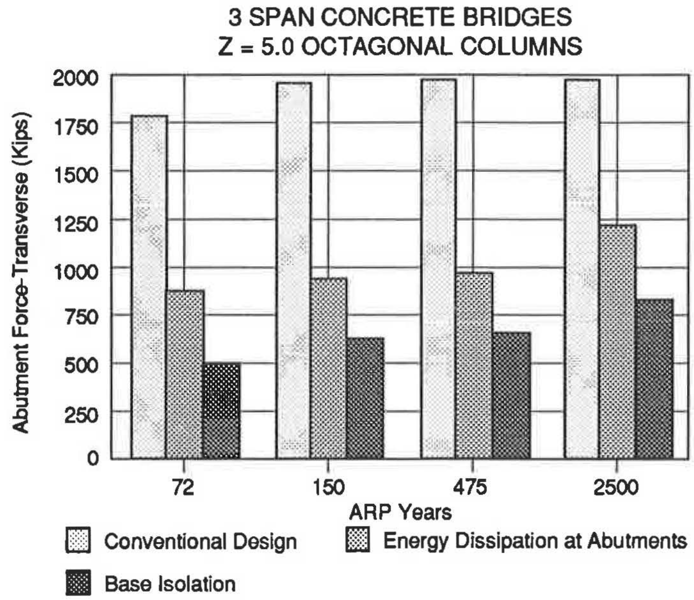


FIGURE 14 Transverse Abutment Force

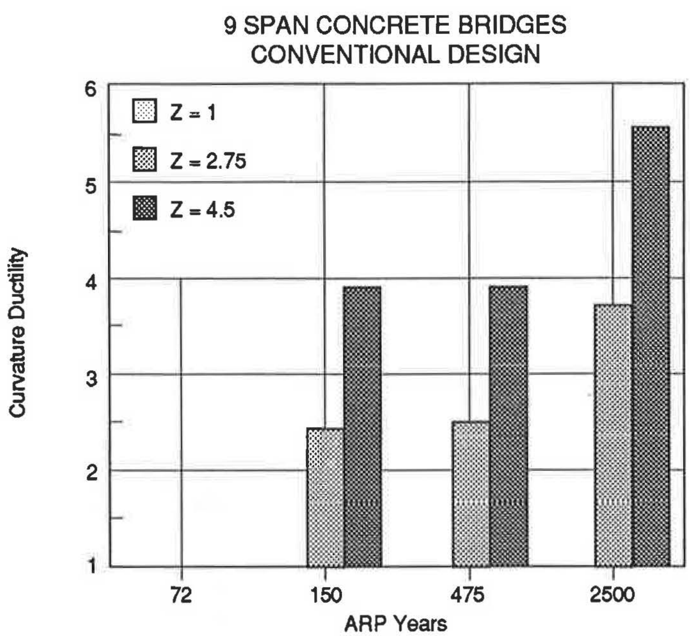


FIGURE 15 Curvature Ductility Demand

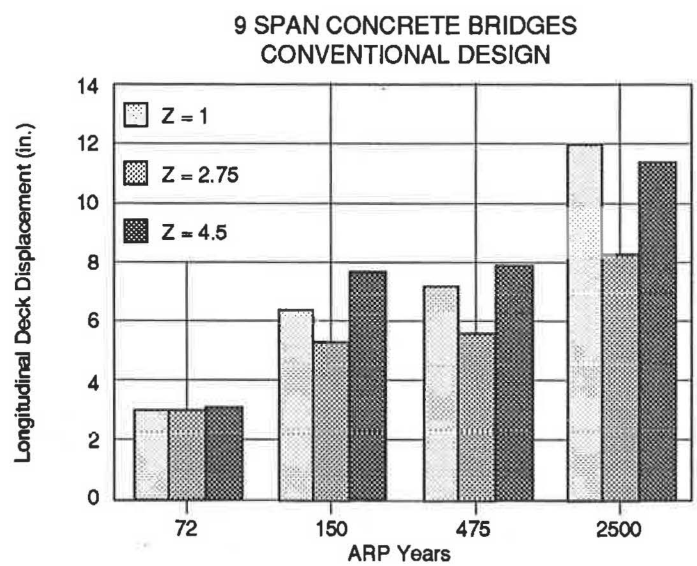


FIGURE 16 Longitudinal Deck Displacement

Summary

A significant number of analyses were performed on a range of design configurations and design options. The results are useful in providing trends of the impact of the different variables considered. Because of the limited range of configurations utilized some generalizations on the impact of the different design options are provided as an aid to designers.

1. Increasing the column strength will decrease the column ductility demand and provide some small decrease in the deck displacements.

2. The use of energy dissipators at the abutments will decrease column ductility demands and deck displacements. These reductions were significant (up to 40%) in the three-span example. These trends would probably occur for continuous bridges in the 2-to 5-span range. For bridges with larger numbers of spans and with intermediate hinges, abutment energy dissipators would only impact the

response of the segment adjacent to the abutment.

3. Seismic isolation provides the ability to eliminate the ductility demand on the columns provided the period of the bridge in the isolated configuration is less than approximately 1.5 seconds. Therefore bridges with tall columns are not good seismic isolation candidates. The deck displacements resulting from the use of seismic isolation are similar to those of conventional construction when analyzed in a non-linear configuration.

4. The use of light weight superstructures does not have an economic advantage in designing for greater seismic reliability.

ECONOMIC ASSESSMENT OF RESULTS

For TCA the economic impact of bridge seismic design and performance is a combination of the following: the cost of initial construction, the cost of earthquake insurance, and the cost of closure of the system during repairs. For Caltrans the impact is the cost to maintain the bridge and repair damage that occurs. In addition, the seismic performance of a bridge as a component of a transportation system will have an economic impact on the surrounding community if the bridge must be closed. The cost components considered in the economic assessment are as follows:

Construction Costs

The baseline cost assumed is for a standard Caltrans type of design. The costs, without construction engineering, are assumed as follows:

Concrete Box Girder Bridge	\$65.00/sq. ft.
Steel Girder Bridge	\$65.00/sq. ft.

A cost of \$65.00/sq ft. was assumed for both girder types to show only the relative effect of the impact of seismic design strategies. It is acknowledged that there maybe an initial cost differential between steel and concrete and that this will vary with time and location. The costs include all standard details including abutments and joints. No attempt was made to include cost increases in any of the designs for changes that may result in the abutments and joints. The cost increases in initial construction only included the increase in column costs and the cost of isolators and energy dissipators where appropriate.

Other items in the cost tables are expressed as a percentage of this initial cost. If it is assumed that all bridges have the same percentage increase, then the project bridge cost also increases by this percentage. A 1% increase in bridge cost represents a 0.17% increase in total project cost (estimated total project cost \$2.1 billion, estimated bridge cost \$0.36 billion). All costs are present value, assuming both the cost of money and the cost of inflation over time are equal. Additional assumptions are as follows:

1. The cost of abutment damage has not been included in any of the designs. The anticipated amount of damage is dependent on the design strategy used for the abutment (e.g. gap provided, backwall engagement, knock-off detail, wing wall capacity etc.).

2. For the steel alternate (fixed base columns) changing the Z-Factor changes the foundation cost. For the concrete alternate (pinned base columns), changing the Z-Factor does not change the foundation cost. It was assumed that the dead and live load requirements govern the pile design and that this provides sufficient lateral capacity to resist the overstrength column shear of 1.3M/L. In some cases, such as poor soil conditions, this assumption may not be accurate, so foundation costs would increase.

Closure Cost

The estimated annual revenue of the three systems is \$43 million per year for 1992-93, \$100 million per year for 1996-97, and \$147 million per year for 1999-2020.

Closure costs for the system would range form \$118,000 to \$402,000 per day. Assuming the year 2,000 revenues, the cost per week of closure expressed as a percentage of bridge cost is:

$$\frac{147 \ X \ 100}{52 \ X \ 360} = 0.8\% \ \textit{per week of closure}$$

The estimate of closure time is based on the level of inelastic behavior in the bridge columns. Significant abutment damage may also cause closure but abutment damage has not been included in this economic assessment.

Column Displacement Ductility	*Weeks Closed*
<3	0
3-4	1
4-5	1-3

Repair Costs

The repair costs are based on the level of the inelastic behavior of the columns. No abutment repair costs have been included. The repair costs do not provide for shoring or difficult access to the repair location.

Cost Comparison

Tables 2 and 3 illustrate the nominal costs of designing a bridge for a higher level of seismic performance. In general, performance of all alternatives is reasonable; for the 72 year ARP event, all alternates remain open assuming no abutment damage occurs. For only one alternative (steel superstructure, Z=5) is closure anticipated for a 475 year return period event again assuming no abutment damage occurs. For the 2500 year ARP event, more severe column damage, requiring closure and increased repair cost occurs for the conventional alternates, except for the Z=3 conventional concrete, where

closure is not required. There is improved performance as the design Z-Factor decreases. Only the fully isolated structure remains damage-free for the 2500 year return period event. The steel structure's performance is marginally worse than the concrete alternative.

TABLE 2 Three-Span Bridge. Costs of Alternate Design Strategies (All Percent Values Are Expressed as a Percentage of Standard Caltrans Design)

Design Alternate	Construction Cost		Column Repair Cost	Lost Revenue	Downtime (weeks)	Total TCA Cost Increase %
	$/sq. ft.	% Incr. of Bridge Cost	% of Initial Cost			% of Initial Cost
Concrete Box Girder - Minimum Design (Z=7.75)						
75% Prob. of Exceedance (72 Year Return Period)			1	0	0	4
20% Prob. of Exceedance (475 Year Return Period)			1-2	0	0	4-5
4% Prob. of Exceedance (2500 Year Return Period)	$67	3	3-5	1	1	6-8
Concrete Box Girder - Minimum Design (Z=7.5)						
75% Prob. of Exceedance (72 Year Return Period)			1	0	0	1
20% Prob. of Exceedance (475 Year Return Period)			1-2	0	0	1-2
4% Prob. of Exceedance (2500 Year Return Period)	$65	-	6-8	1-3	1-3	7-11
Concrete Box Girder - Reduce Z-Factor from 7.5 to 5						
75% Prob. of Exceedance (72 Year Return Period)			1	0	0	2
20% Prob. of Exceedance (475 Year Return Period)			1-2	0	0	2-3
4% Prob. of Exceedance (2500 Year Return Period)	$66	1	3-5	1	1	5-7
Concrete Box Girder - Reduce Z-Factor from 7.5 to 3						
75% Prob. of Exceedance (72 Year Return Period)			0	0	0	3
20% Prob. of Exceedance (475 Year Return Period)			1	0	0	4
4% Prob. of Exceedance (2500 Year Return Period)	$67	3	1-3	0	0	4-6
Concrete Box Girder - Isolation at All Supports						
75% Prob. of Exceedance (72 Year Return Period)			-	0	0	4-6
20% Prob. of Exceedance (475 Year Return Period)			-	0	0	4-6
4% Prob. of Exceedance (2500 Year Return Period)	$68-$69	4-6	-	0	0	4-6
Concrete Box Girder - Energy Dissipation Devices at Abutments (Z=7.5)						
75% Prob. of Exceedance (72 Year Return Period)			1	0	0	4-6
20% Prob. of Exceedance (475 Year Return Period)			1-2	0	0	4-6
4% Prob. of Exceedance (2500 Year Return Period)	$67-$68	3-5	3-5	1	1	7-11
Concrete Box Girder - Energy Dissipation Devices at Abutments (Z=5)						
75% Prob. of Exceedance (72 Year Return Period)			1	0	0	5-7
20% Prob. of Exceedance (475 Year Return Period)			1	0	0	5-7
4% Prob. of Exceedance (2500 Year Return Period)	$68-$69	4-6	1-2	0	0	5-8
Lightweight Superstructure - Steel, Conventional Z=5						
75% Prob. of Exceedance (72 Year Return Period)			1	0	0	2
20% Prob. of Exceedance (475 Year Return Period)			3-5	1	1	6-8
4% Prob. of Exceedance (2500 Year Return Period)	$66	1	6-8	1-3	1-3	9-15
Lightweight Superstructure Steel, Conventional Z=3.25						
75% Prob. of Exceedance (72 Year Return Period)			0	0	0	3-6
20% Prob. of Exceedance (475 Year Return Period)			1-2	0	0	4-8
4% Prob. of Exceedance (2500 Year Return Period)	$67-$69	3-6	6-8	1-3	1-3	11-20
Lightweight Superstructure - Steel, Fully Isolated						
75% Prob. of Exceedance (72 Year Return Period)			0	0	0	3-6
20% Prob. of Exceedance (475 Year Return Period)			0	0	0	3-6
4% Prob. of Exceedance (2500 Year Return Period)	$67-$69	3-6	0	0	0	3-6

Note: Z = 7.75 is the 4'x8' oblong column. All other columns are octagonal.

TABLE 3 Nine-Span Bridge. Costs of Alternate Design Strategies (All Percent Values Are Expressed as a Percentage of Standard Caltrans Design)

Design Alternate	Construction Cost		Column Repair Cost	Lost Revenue	Downtime (weeks)	Total TCA Cost Increase %
	$/sq. ft.	% Incr. of Bridge Cost	% of Initial Cost			% of Initial Cost
Concrete Box Girder with Fixed-Base Design (Z=4.5)						
75% Prob. of Exceedance (72 Year Return Period)			0	0	0	0
20% Prob. of Exceedance (475 Year Return Period)			3-4	0	0	3-4
4% Prob. of Exceedance (2500 Year Return Period)	$65	-	5-7	0	0	5-7
Concrete Box Girder with Fixed-Base Design Reduce Z-Factor from 4.5 to 2.75						
75% Prob. of Exceedance (72 Year Return Period)			0	0	0	1-3
20% Prob. of Exceedance (475 Year Return Period)			3-4	0	0	4-7
4% Prob. of Exceedance (2500 Year Return Period)	$66-$67	1-3	3-4	0	0	4-7

RECOMMENDED PROJECT SPECIFIC CRITERIA

The intent of the recommended seismic design criteria for all bridges on the TCA project is to provide a higher level of safety and reliability than the current Caltrans design criteria. The criteria are as follows:

Design Response Spectra

The design response spectra for a bridge shall be obtained by using the appropriate site specific spectra for both the maximum credible and 72 year return period earthquake. The spectral ordinates for vertical earthquake ground motions are obtained by multiply the horizontal values by two-thirds.

Methods of Analysis

With the exception of single-span bridges, multi-mode response spectra analysis shall be used for all bridges. The maximum response (displacement or component force) shall be estimated based on the CQC (Complete Quadratic Combination) modal combination procedure. If discontinuities or other sources of nonlinearity exist, use an equivalent linear solution procedure with iteration as required to satisfy equilibrium of forces and displacements. Non linear analysis may be used.

Preliminary Design

During the preliminary design stage, due considerations should be given to the following items:

1. The selection of span configuration. Unbalanced dead load moments in the columns should be minimized as much as possible;
2. The selection of optimal column cross section;
3. For multiple column bents, whether to pin columns at the base to minimize foundation costs or to fix column bases to achieve more redundancy under seismic loads and smaller moment magnifications under live loads.

Column Design

Column Design forces are based on the forces obtained from the lower level design earthquake. This will ensure that there is no ductility demand on the columns for this design event. A second analysis is required to ensure that the ductility demand on the columns is limited to acceptable levels for the upper level design event ($\mu < 3$ for single columns and $\mu < 4$ for multi-column bents). For this design check it is assumed that there will be no resistance provided by the abutments. This will ensure that the columns will provide adequate resistance and acceptable performance regardless of what damage may occur at the abutments.

Abutment Design

Design displacements and forces are obtained from an analysis using the lower level design spectra and appropriate stiffnesses of columns and abutments. There should be no engagement of the abutment backwall and there should be no failure of any fuses.

For the upper level design event an analysis is required to determine the maximum displacements and forces that may occur. This analysis is performed assuming any fuses or keys have failed. If the abutment backwall is engaged, limits on the displacement are provided to avoid excessive damage.

If energy dissipation or seismic isolation bearings are used the site specific response spectra should be modified to incorporate

the additional damping provided by these devices.

Design Forces

Load Case 1: 100% of the absolute values of force and moment in transverse direction are added to 30% of the corresponding forces and moments from the longitudinal direction.

Load Case 2: 100% of the absolute values of force and moment from the analysis in the longitudinal direction are added to 30% of the absolute value of the corresponding forces and moments from the transverse direction.

Columns: Each column of the structure is designed to withstand the forces resulting from each load combination from the lower level event according to Caltrans specification.

Foundations

Foundations shall be designed for the forces resulting from plastic hinging of the top and bottom of the column. The column plastic moment capacity shall be 1.3 times the moment capacity obtained using a Ø-factor of 1.

Transverse Abutment Shear Keys

The transverse shear keys, if used, shall be designed for the forces resulting from each load combination from the lower level event according to Caltrans specifications using a Ø-factor of 1.

Abutment Piles

The combined pile and wingwall capacity at the abutments shall have sufficient lateral capacity to resist the design forces required. If fuses are provided for the lower level event, these will provide the lower limit of the design forces. If no fuses are provided, design forces should be obtained from the upper level event.

Connections

The connections of the columns to the foundations, superstructure or bent cap shall be designed for the forces resulting from column plastic hinging using a Ø-factor of 1.3. If bent caps are used, the joints of the bent caps shall be designed to resist the plastic hinge moments developed by the column and the bent cap beam both determined using a Ø-factor of 1.3.

Girder Support Length

The minimum support lengths (N) of all girders shall be the greater of either those obtained from an analysis using the maximum credible spectra or

$$N=12+0.3L+0.12H \text{ (inches)}$$

where

L = length (ft.) of bridge to adjacent joint or bridge end.
H = height (ft.) of columns.

Ductility Demand Design Check

The column forces and displacements resulting from the maximum credible analytical model shall be used to check that each column has a ductility demand less than 3 for single columns and less than 4 for multicolumn bents. The column forces shall be determined by incorporating the P-Δ effect if the column displacements are judged to be significant.

CONCLUSIONS

A two level design criteria has been developed for the bridges in California's first toll roads. The intent of the criteria is to have a higher level of safety and reliability than Caltran's current minimum requirements. The criteria when implemented should prevent significant damage for at least a 72 year return period event. By comparison Caltrans current requirements would prevent significant damage for a 25 to 50 Year return period event.

Several different design strategies were included in the performance and cost study that was performed to aid in the development of the criteria. It was shown that for relatively small increases in initial cost there were several options for enhancing the seismic performance of the bridges. These included designing the columns for higher force levels to reduce column damage; the use of energy dissipators at abutments for 2 to 5 span bridges which will provide a 30 to 50% reduction in displacements and column ductility demands; and the use of seismic isolation which can eliminate column damage.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the work that was performed by David Jones, Marcial Blondet, Trevor Kelly of Computech Engineering Services, and David Liu of Imbsen and Associates. They were responsible for the design and analysis work performed as part of the study. Woodward-Clyde's seismic hazard analysis was performed by John Barneich, Yoshi Moriwaki, and John Waggoner.

Lateral Load Test on Driven Pile Footings

JACK L. ABCARIUS

The October 17, 1989 "LOMA PRIETA" earthquake provided an ideal time and situation to perform for the first time, at no installation cost, a full scale lateral load test on a group of driven piles in typical bridge footings at the Cypress Street Viaduct. Since these foundations were no longer going to be used, we were able to load them laterally to failure and record load versus deflection.

Testing was performed at two different locations. Location #1 consisted of 60'± long piles in a clayey (bay mud) material, and location #2 consisted of 15'± long piles in sandy silt.

Preliminary results were very encouraging. Lateral pile capacities were observed at ¼" deflection, which greatly exceed our Bridge Design Specifications criteria. Reduced data produced a range from 17 kips to approximately 26 kips per pile in these two soil types.

Considering that we use 5 kips/pile in today's design criteria for this type of pile, one can readily see that if this number were increased to just 10 kips, the number of piles required for lateral forces would be significantly reduced, thereby achieving an appreciable reduction in cost for foundations of this type.

Pile capacities are correlated with appropriate soil parameters for the two soil types. Bridge Design Specifications are being proposed to take advantage of the increased lateral load capacities obtained in this research project.

INTRODUCTION

For the past fifteen years, Caltrans foundation pile designs for lateral forces have been based on full scale testing of single piles in predrilled holes with a two layer soil system consisting of a compacted embankment on an underlying natural deposit of silt or clay [1]. An earlier study for a sandy soil at Occidental Drive Overcrossing in Sacramento was published in "Lateral Resistance and Deflection of Vertical Piles, Interim Report #1 [2]. Several experimental studies concerning the behavior of piles and pile groups subjected to lateral loading have been conducted at the University of Houston Pile Test Facility. None of these tests, however, loaded the piles to failure.

To load the piles to failure at the Cypress Street Viaduct, there were two points to consider:

1) Ultimate capacity of the pile
2) Capacity of jacking frame

Since the maximum number of piles under the footings to be tested was seventeen (17) and our Bridge Design Practice specifies an ultimate lateral resistance of 40 kips per pile for earthquake loads, the Caltrans Substructure Committee assumed that a minimum force of 680 kips would be needed to fail the piles; furthermore, to make sure the jacking frame would not fail, the Committee agreed on a maximum force of 1500 kips for the frame to assure its structural integrity throughout the testing. Christie Constructors, Inc. was chosen to provide a calibrated jack and to design the jacking frame (Figure 1).

State of California, Department of Transportation, Division of Structures, Sacramento, CA 94274-0001

FOUNDATION INVESTIGATION

A foundation investigation was completed at the site in February 1990 by the Engineering Geology Branch of the Office of Transportation Materials and Research. Rotary sample borings of 4" were drilled at both test locations. Eight samples from the boring near Bent #97 and two samples from the boring near Bent #61 were analyzed.

Boring log (B-6) near bent #97 is shown in Figure 8, and boring log (B-4) near bent #61 is shown in Figure 9.

TEST PROCEDURE

Bent #97:

1) Excavate between footings to expose inside face (perpendicular to centerline of bent).
2) Locate centerline of bent on each footing and mark.
3) Set a transit on one mark, sight at the other and turn 90 degrees to set a hub at some distance (10'±), then rotate the eyepiece 180 degrees to set another hub.
4) Repeat process for each footing.
5) Place jacking frame between the right and center footing.
6) Dry pack all voids between end plates and face of right footing to assure full bearing.
7) Provide additional steel plates between jacks and face of center footing for a tight fit.
8) Fix measuring tapes on the footings parallel to centerline of bent to measure deflections.
9) Perform Phase I test, then release jacks (Figures 3 and 4).
10) Place bracing frame between the left and center footing; provide additional plates between end plates and face of footing for a tight fit.
11) Perform Phase II test (Figure 5).

Bent #61:

1) Eliminate the passive pressure created on the other side face of the footing.
2) Eliminate the dry pack and use steel plates throughout.
3) Perform Phase I test only.

CONCLUSION

Caltrans Bridge Design Specifications presently specify 5 kips of lateral resistance at ¼" deflection with a standard penetration resistance value, N of 10 for a (12" flange) steel pile or a 12" driven concrete pile. This value appears to be extremely conservative as borne out by the Cypress tests (see Summary Table).

On an interim basis, until more site tests at other locations and in other soils can be made, it was recommended that the Caltrans Specifications be increased to 10 kips lateral resistance for the above conditions. Caltrans specifications are being revised accordingly.

Further tests at other sites using this jacking frame are being planned one of which is at the Terminal Separation replacement in San Francisco.

SUMMARY TABLE OF RESULTS

Location	*Load/Pile @ ¼" Deflection*
Bent #97 Center Footing	17.7 kips (Phase I)
Bent #97 Right Footing	20.9 kips (Phase I) and 25.6 kips (Phase 2)
Bent #97 Left Footing	32.9 kips (Phase 2)
Bent #61 Center Footing	24.7 kips
Bent #61 Right Footing	25.0 kips

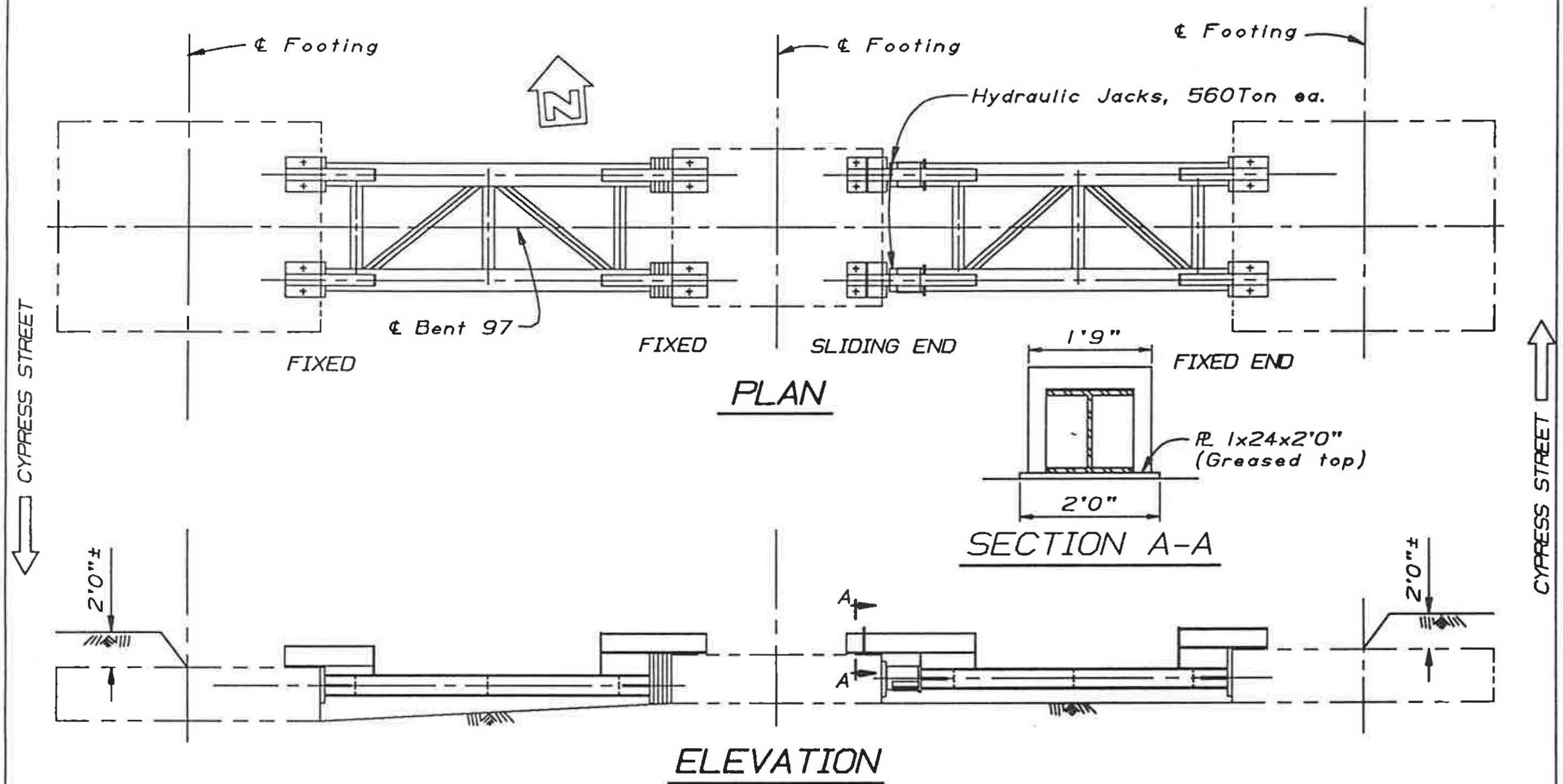
Table 1. Calibration of Jacks

Pressure Gage (psi)	Load (kips)
100	.0
1,000	217
2,000	445
3,000	675
4,000	905
5,000	1,140
6,000	1,370
7,000	1,602
8,000	1,830
8,800	2,010

REFERENCES

[1] Yee, Wilfred S., "Lateral Resistance and Deflection of Piles – Final Report – Phase I," State of California, January 1973.

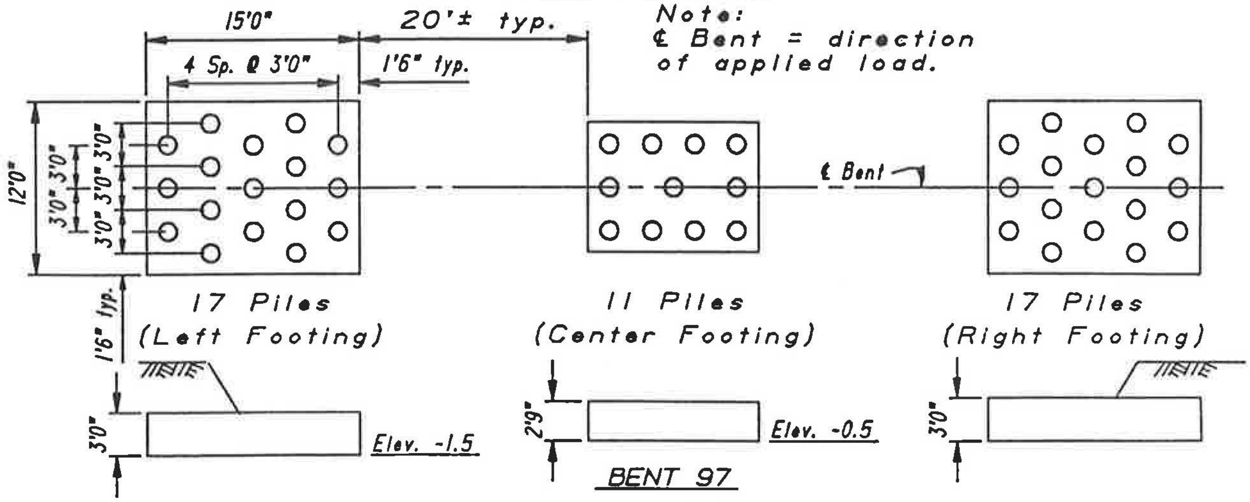
[2] Yee, Wilfred S., "Lateral Resistance and Deflection of Vertical Piles, Interim Report #1," State of California Business and Transportation Agency, Department of Public Works, Division of Highways, Bridge Department, R&D Report No. 6-71, September 1971.

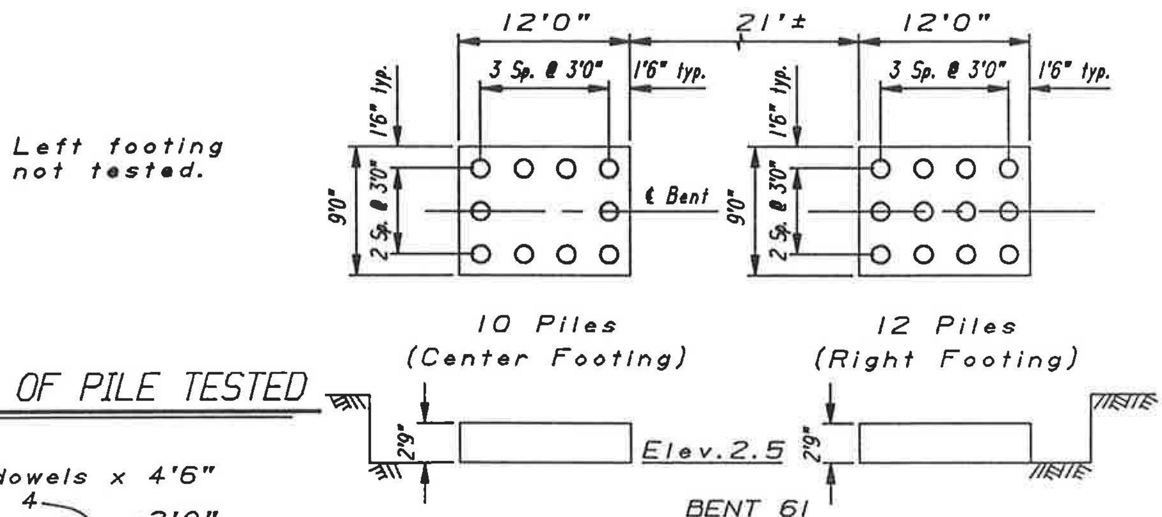


LATERAL LOAD TESTING APPARATUS

Figure 1

FOOTING AND PILE LAYOUT





TYPE OF PILE TESTED

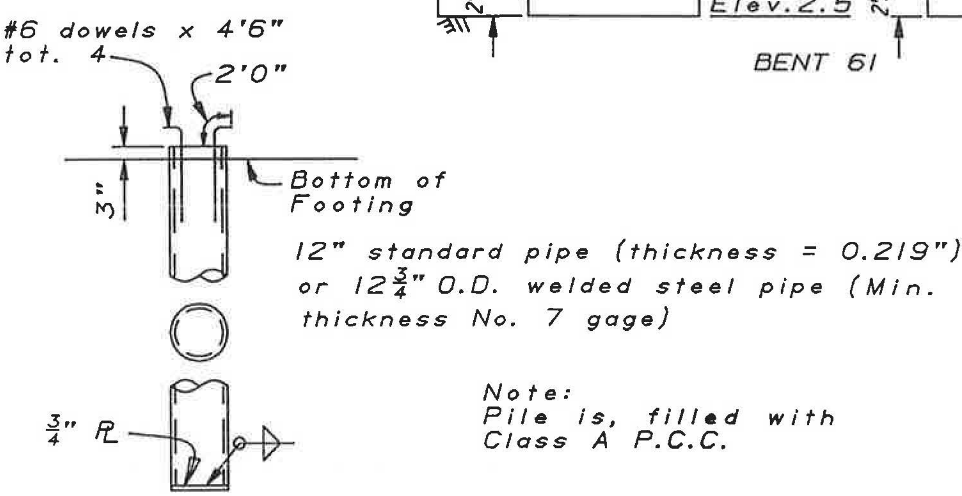
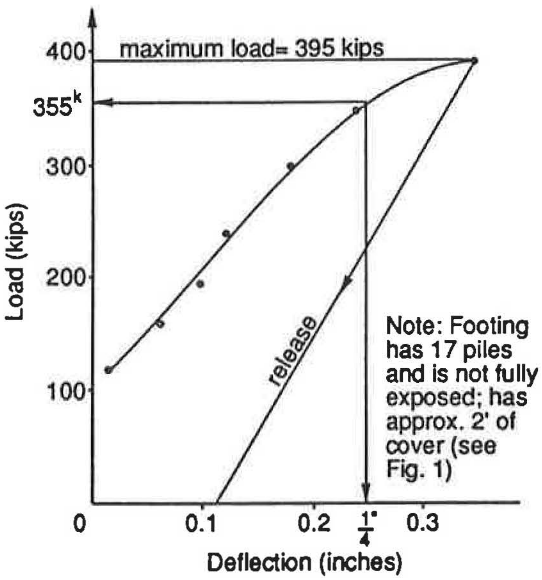
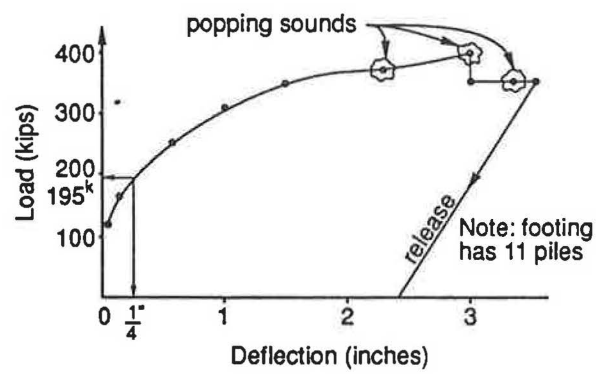


Fig. 2

Phase I Testing

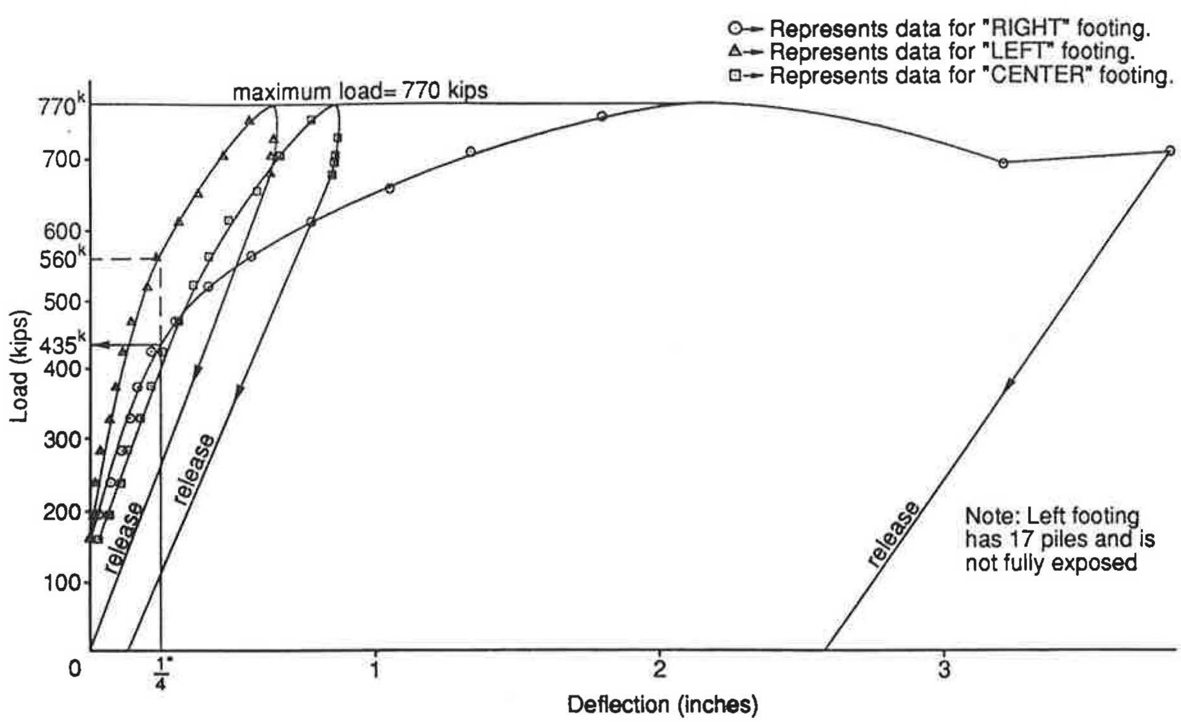


Load vs. Deflection Curve for Bent #97 "Right" Footing **Fig. 3**

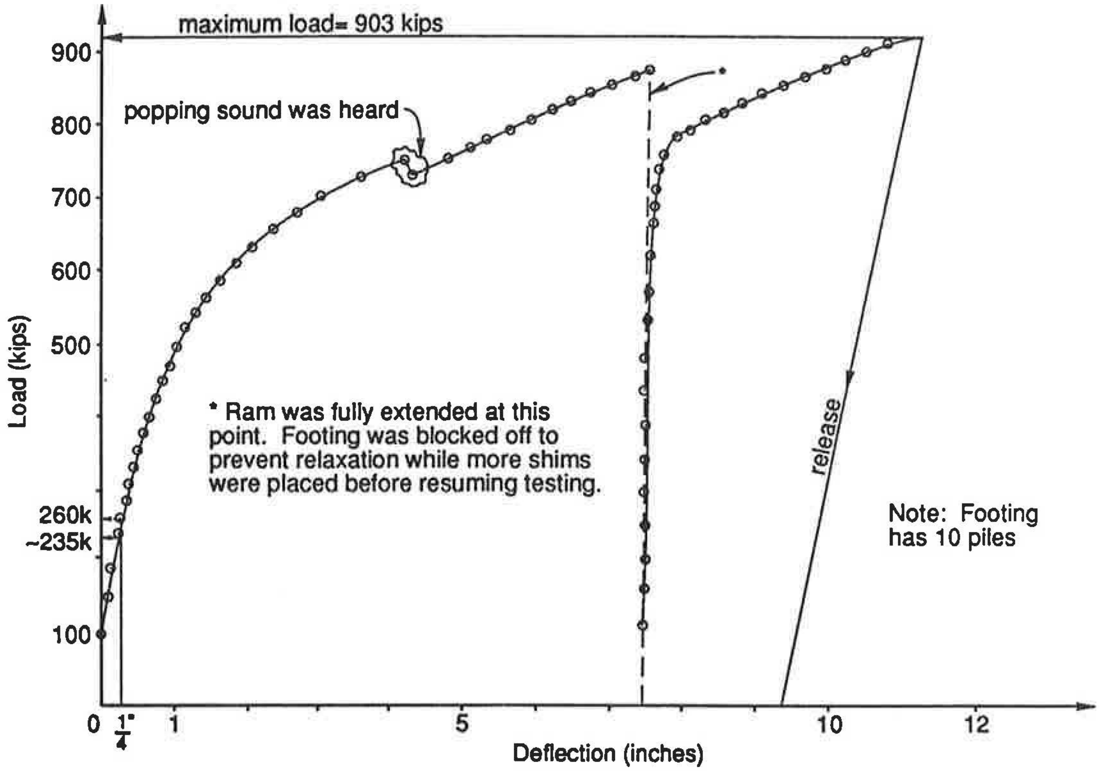


Load vs. Deflection Curve for Bent #97 "Center" Footing **Fig. 4**

Phase II Testing

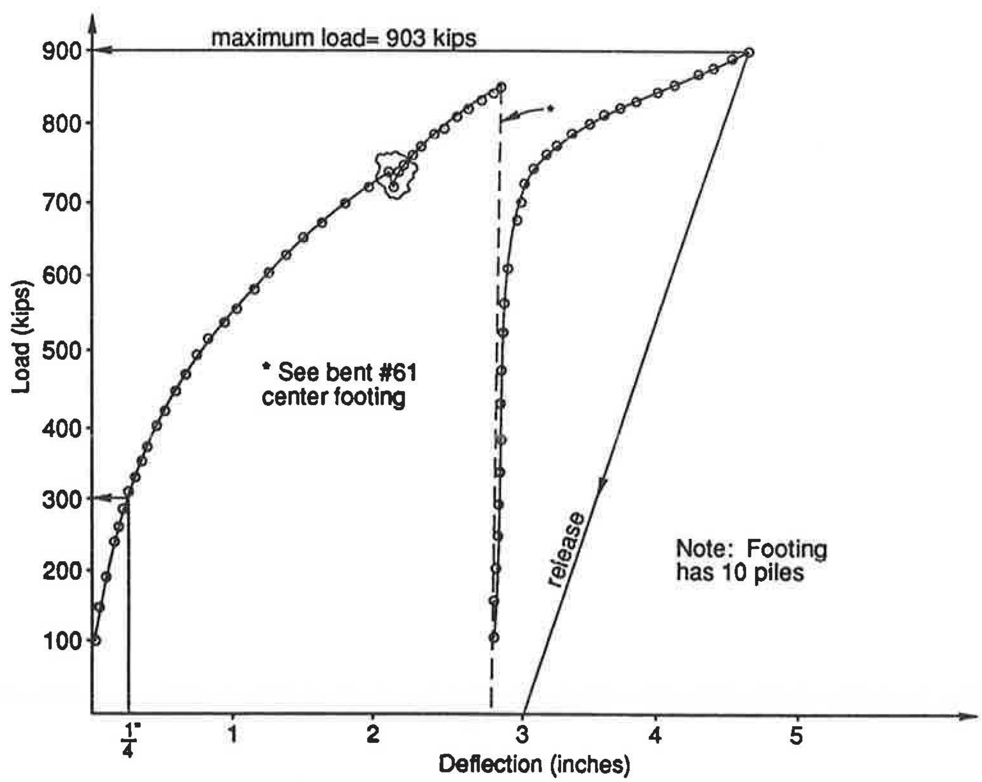


Load vs. Deflection Curve for Bent #97 Footings **Fig. 5**



Load vs. Deflection Curve for Bent #61 "Center" Footings

Fig. 6



Load vs. Deflection Curve for Bent #61 "Center" Footings

Fig. 7

Expert System for Determining the Disposition of Older Bridges

WILLIAM ZUK

The paper describes the development of an expert system that was designed to make recommendations for bridge management as to the proper courses of action that should be taken with regard to older highway bridges. Five basic options are possible; namely rehabilitation, improvement, replacement, abandonment, and routine maintenance. Based on an extensive set of rules, criteria and procedures as currently used by bridge engineers, this expert system offers a computerized approach that should reduce the time needed to evaluate the many thousands of older bridges yearly, as well as to provide a consistent basis for decision making.

INTRODUCTION

It is assumed that most engineers have by now heard about "expert systems;" that is, computer driven systems that generate decisions similar to those of human experts, founded on extensive knowledge bases of information. Such expert systems are particularly useful where there are complex and often fuzzy situations. One such situation, which this paper deals with is that of deciding what to do with the many thousands of older highway bridges in the inventory of the State of Virginia. As part of bridge management programs, a similar situation exists in every state in the country; representing in total, several hundred thousand bridges. Not only are there many factors to be considered for each bridge, but such bridges are generally re–evaluated every year. The collective task is thus quite enormous.

To provide a rapid and consistent way to make these many evaluations, an expert system was developed for the Virginia Department of Transportation (VDOT) at the Virginia Transportation Research Council (VTRC). Specifically, the expert system was designed to provide the user with a recommendation as to the disposition of a given bridge by one of the following five options.

1. Rehabilitate: To rehabilitate a bridge is to restore it to its original condition. The recommendation may stipulate rehabilitation of the deck, the superstructure, the substructure, or any combination of the three.
2. Improve: To improve a bridge is to improve its characteristics over and above its original condition. This is usually either by strengthening, widening, or both.
3. Replace: To replace a bridge is to remove or bypass the old bridge and replace it with a completely new one on the same or on a new alignment.
4. Abandon: To abandon a bridge is to take it out of service from the highway system. An abandoned bridge may be destroyed or put to a new use, possible at another site.
5. Maintain: To maintain a bridge is to keep it functioning essentially in the condition that it is in. The recommendation may call for maintenance of the deck, the superstructure, the substructure, or any combination of these three components.

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DEVELOPMENT OF THE EXPERT SYSTEM CALLED DOBES (*D*isposition of *O*lder *B*ridges *E*xpert *S*ystem)

The development of DOBES proceeded in four related phases. The first phase was devoted to a general overview of the subject to acquaint potential uses what expert systems are and how they could be useful to them in a variety of applications.[1] This phase was believed necessary as many engineers are fully acquainted with the emerging subject of expert systems. The second phase narrowed the subject down to a specific bridge management problem as explained in the Introduction. It also encompassed the acquisition of the necessary knowledge base and development of the inference procedure (*2*). Details of these acquisition and inference efforts are described later. The third phase utilized the knowledge base and inference procedure developed in phase two and converted them into user–friendly software (*3*). The resulting DOBES program is contained on two standard floppy disks intended to run on any IBM or compatible personal computer. Details of phase three are described later. The fourth and last phase deals with the field testing of DOBES and its possible refinement. As of this writing (September, 1990) only partial results are in, but preliminary indications are that this expert system is working rather well (*4*). More on this final phase is described later.

The Knowledge Base

Since DOBES is expected to reach essentially the same conclusions as human experts, the exact method experts arrive at their decisions had to be obtained. This was done by the author through a series of lengthy taped recorded individual interviews with a number of senior bridge engineers in VDOT whose responsibility it was to make decisions related to older bridges. Although their thinking processes were similar in general, there were some differences in regard to details and secondary considerations. Following the completion of the interviews, this bank of knowledge had to be evaluated, adjusted to resolve differences, organized, and then couched in the form of "if–then rules" as required for expert systems programming. A draft of these rules was written up and then re–submitted to the experts interviewed for their review. In the review process, some minor changes and adjustments were made. These rules form the basis of DOBES; of which there are 35. Since not all the rules were judged to be equally important, some were designated as being primary (P) and some secondary (S). A complete description of the rules can be found in reference (*2*); however, several are listed as follows for illustration.

Rule 1. If the sufficiency rating is more than 80, then recommended maintenance (P).

Rule 2. If the sufficiency rating is more than 50 and the sufficiency rating is less than 81, then recommend rehabilitation and recommend improvement (P)

Rule 8. If the substructure condition rating is less than 6, then recommend substructure rehabilitation (P)

Rule 13. If the sufficiency rating is less than 50 and the average daily traffic is less than 10 and the detour length is less than 2 and historic is no and political objection is no, then recommend abandonment (P)

Rule 15. If classification is Primary and road width is less than 12 times the number of lanes and the sufficiency rating is greater than or equal to 50 and the average daily traffic is more than 100 and type is deck truss or deck girder, then recommend improve by widening (P)

Rule 23. If the material is concrete and the classification is Primary or Interstate and loading capacity is less than 22 and average daily traffic is more than 100 and historic is no, then recommend replacement (S)

Rule 26. If the material is metal and the classification is Secondary and the loading capacity is less than 15 and the sufficiency rating is more than 50 and the average daily traffic is more than 100, then recommend improve by widening (S)

Rule 35. If type is through truss and material is metal and classification is Secondary and loading capacity is less than 15 and average daily traffic is more than 100 and historic is no, then recommend replacement (S)

As can be seen, some of the rules are rather involved, which suggests that a computer would be more likely to keep track of all these factors than would a human; thus providing a greater degree of decision making consistency.

In addition to a set of rules, a knowledge base also must contain specific data or attributes of a given bridge. These attributes relate to the needs of the various rules. For example in Rule 1, the sufficiency rating must be known. In other rules, the condition rating, average daily traffic, bridge type, material and the like must be known. In all, 24 attributes of a bridge are needed plus 4 relating to name and location of the structure. Six of these are designated as secondary (S) attributes; such as the detour length if the bridge were to be abandoned, or the bridge's historic importance. It is to be noted that DOBES is designed to work, even if these secondary attributes are not known; although the final recommendation would be qualified.

In actual use, a question concerning each the 28 attributes appears on the monitor with appropriate prompts, so that the user has only to select or type in a simple answer. The rules are stored in memory, but could be displayed on the monitor or printed out on command.

The Inference Procedure

The inference procedure by which a final recommendation is reached is based on a forward chaining process coupled with a multi–factored weighting system. The process begins by applying each of the separate rules to each of the separate attributes. This results in a listing of all applicable recommendations in tentative form. However, some rules conflict or overlap with others, and some rules and attributes are more important than others. It was therefore necessary to develop an inference or evaluation process to deal with these factors. A multiweighting system was chosen.

First, each attribute and rule is designated as primary (P) or secondary (S). These determinations are based on the judgments of the bridge engineers involved in this study. Each status of P or S can be changed if necessary by minor reprogramming of DOBES. When the rules are applied to the attributes, the possible combinations of options (such as replace, rehabilitate, etc.) involve primary rule with primary attribute (PP), primary rule with secondary attribute (PS), secondary rule with primary attribute (SP), and secondary rule with secondary attribute (SS). For any given bridge, a list is made of the various resulting options generated in applying the rules to the attributes, along with their (PP), (PS), (SP), (SS) category.

To provide for a wide range of weighting factors, 15 constants (potentially adjustable) were applied in the following manner to the 5 basic options: rehabilitate, improve, replace, abandon, and maintain. For example, with regard to rehabilitation, a numerical constant A is assigned as a multiplier or weighting factor to the sum of all the times on the option list cited that call for rehabilitation in the (PP) category. Then another numerical constant B is assigned as a multiplier to the sum of all the times on the option list that call for rehabilitation in the (PS) or (SP) category. The categories (PS) and (SP) are assumed to be of equal weighting value. Still another numerical constant C is assigned as a multiplier to the sum of all the times in the option list that call for rehabilitation in the (SS) category. By adding all the terms above involving A, B, and C, a new sum value D is determined.

In like manner, four other sum values for the improve, replace, abandon, and maintain options can be determined.

The final recommendation would be the option with the largest numerical sum value. In the DOBES program as currently written, the constants A, B, and C were chosen as 10, 7, and 3, respectively. Parallel constants for the improve option are 8, 5, and 2. For the replace option, the constants are 10, 7, and 3. For the abandon option, they are 6, 4, and 1. Finally, for the maintain option, the constants are 7, 5, and 2. The larger the constant, the more important is the related option. These constants were chosen intuitively as trial values. These values can and will be adjusted as further testing and evaluation indicate.

Two checks related to cost are built into the program.

1. If the rehabilitation or improve option has the highest sum value, but the cost exceeds 70 percent of the cost of a replacement bridge, then replacement is selected as the final recommendation.

2. If the rehabilitate or improve option has equal sum values, and these sum values exceed all others, then the rehabilitate or improve option costing the lesser amount is selected as the final recommendation.

The entire inference procedure described is done in a micro–second simply by selecting the "Analyze" item displayed on the main menu of the program. Both a summary of the numerical scores and a final recommendation are given. Should there be any qualifications or conditions regarding the final recommendation, such as the omission of any particular secondary attribute, that too will be noted. It is left for the user to decide how important the qualifications or conditions are, with respect to the final recommendation. All final recommednations are of course subject to the ultimate judgement of the user.

Software

Certain aspects of the software associated with its development and operation should be noted for a proper understanding of DOBES.

1. LISP programming language was used in the development of the program. As such, licensing by Gold Hill computers as well as approval by the Virginia Dept. of Transportation is required before duplicate disk copies of DOBES can be made.
2. DOBES is configured for two 5 1/4 inch floppy drives for IBM or compatible microcomputers. One disk contains

the master program and the other is used for collection and storage of the data base (attributes).

3. The program is essentially menu driven. For example, in connection with the record (data base) one can chose to add to the record, change the record, delete from the record, inspect the record, list the records or print the records. For other operations, one can choose from this main menu to inspect the rules, to analyze the problem (which leads to a final recommendation) or to exit the program.
4. Changes to the rules or any other aspect of this expert system are possible, but a knowledge of the LISP language is required. This feature should enable DOBES to stay up to date should situations or conditions change.
5. Because of time and budget constraints, the following limitations had to be imposed on the current DOBES program.
   a. A few of the rules are couched in overly precise terms. For example, several rules set the ADT (average daily traffic) at a precise 100. Obviously a count of 99 or 101 would make little difference in the final result. However, an incorporation of "fuzzy logic" in the program would add considerable complexity, which is not believed to be warranted.
   b. Interactive questions and answers concerning how the rules were arrived at, where data can be obtained, or the reasoning behind the inference have been left out. Should such information be necessary in an operational program, it could be added at a later date. Much of this information, however, already is available in the report cited in reference (2).
   c. The program is applicable to all single or multiple span girder or truss highway bridges of wood, metal, or concrete (either normally reinforced or prestressed). If in a multiple span bridge, one span (or more) is of a different basic material and/or of a different structural type from the rest of the bridge, that span or spans should be treated as if it were a separate bridge. The remaining span or spans would then also be treated as a separate bridge.
   d. A few relatively rare bridge types have not been included, such as covered wooden bridges, arch bridges, cable–suspended bridges, and plastic bridges. So few of these exist that it is believed these special ones could be handled individually.
   e. Recommendations concerning which of the many old bridges deserve expenditure of funds in a given year are not part of this program. Such recommendations would require that all of the old bridges be evaluated by this DOBES program, after which the total required costs would be compared to the total available funds on some rational basis such as a bridge management system using deficiency point ratings as a guide to prioritization. An expert system prioritization is quite possible, but would require additional programming.

Testing and Refining

At the time of this writing (September 1990) field testing of DOBES has been initiated, but not yet completed. Senior VDOT bridge engineers in all nine regions of the state who have responsibility for making decisions of the kind generated by DOBES, have been supplied with DOBES disks and instruction manuals. On–site field visits by the author or his associate were made as needed in order to see that the operation of this expert system was understood and working. These nine bridge engineers were asked to test DOBES using a number of actual older bridges they are familiar with (at least 15 per person) and answer the following eight questions:

1. Are the final recommendations from DOBES the same as would be obtained by a human expert? If not, what would be the expert's recommendation?
2. Are any of the rules incorrect? If so, what would be the correct rule?
3. Are any necessary rules omitted? If so, what are they (indicating whether primary or secondary)?
4. Should any of the attributes (bridge data) listed as primary be listed as secondary? If so, which ones?
5. Should any of the attributes listed as secondary be listed as primary? If so, which ones?
6. Should any of the rules listed as primary be listed as secondary? If so, which ones?
7. Should any of the rules listed as secondary be listed as primary? If so, which ones?
8. How could the DOBES program be improved to make it easier to use?

In addition to answers to the above questions, print–outs of the "Analysis" of the bridges investigated using DOBES were requested to assist in the evaluation.

When all replies to the questions have been received, they will be carefully evaluated. Changes and refinements in DOBES will then be made as required. Of particular focus will be the matching of recommendations between those by the experts and those by DOBES. Should there be inconsistencies unaccounted for by possible rule changes, or changes concerning the primary and secondary importance of attributes, a systematic exploration will be undertaken with regard to the 15 numerical weighting factors in the inference procedure. A specially written iterative computer program will be used to find the best match. Ultimately, by late 1990, it is expected that a refined DOBES program will result that will accomplish its intended mission.

CONCLUSIONS

At the initiation of this expert system project, there was an element of skepticism among some of the intended users, that any computer program could successfully deal with the many variables and uncertainties involved in a decision making process of this nature. However, as the work progressed, which required the involvement of the potential users from time to time, skepticism grew less and less, as the basis for the computer program became understood. It was realized that the factors (whether firm or fuzzy) used by experts in reaching a decision were the same ones used in this expert system. Furthermore, the software as developed is quite easy to use, requiring very little introductory instruction. Acceptance for accredited operational use, however is not to be expected before 1991, after the field tests have been run and the program fine–tuned. So far, based on preliminary results, DOBES appears to be working rather wellwith close to 90% agreement between recommendations made by DOBES and those by bridge engineers.

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Bridge Management Systems—State of the Art

A. M. SHIROLE, W. J. WINKLER, AND J. J. HILL

During the past decade, many agencies responsibile for bridges in the U.S. and abroad have been actively involved in the development of operating bridge management systems (BMSs). The Federal Highway Administration, American Association of State Highway and Transportation Officials, and Transportation Research Board have encouraged and supported such efforts. This paper presents an overview of major approaches to BMS development that have emerged during the past decade. It evaluates the role of differing needs, specific to different agencies, and the way in which they are addressed in developing customized bridge management systems. The paper reviews use of large mainframe as well as microcomputers for applications suitable for large and small agencies. Further, the paper presents some suggestions and insights about the future of the state-of-the-art of BMSs.

The information presented was compiled from available literature (1)(2)(5)(6)(8) and from results of a bridge management questionnaire summarized in Table 1.(10)

INTRODUCTION

During the past decade, agencies responsible for bridges have come to recognize the severity and enormity of problems associated with their bridge populations. As a result, significant efforts have been undertaken to analyze and find well-researched engineering solutions to these problems. Attention has also focused on finding better ways to manage all bridge-related activities in order to avoid similar problems in the future. Many agencies began developing comprehensive bridge management systems (BMSs) toward this goal.

Some states, such as Pennsylvania, North Carolina, and Indiana began developing their own BMSs. Others, in need of some assistance and guidance, have opted to wait and clearly understand ongoing efforts in development of BMSs. To provide an overview of these efforts, this paper reviews current practices, prevailing environments, and major approaches to BMS development. It also summarizes BMS-development activities in different agencies and highlights some primary features of those efforts. Further, the paper discusses BMS automation needs of small and large agencies and presents some helpful suggestions and insights with respect to the future of BMSs.

BRIDGE MANAGEMENT PRACTICES

Current attention toward developing BMSs should not be interpreted to mean that bridge-related activities were not managed in the past. Over their service life, some bridges were being managed better than others. As time passed, the numbers of bridges and their needs grew larger and larger. No longer could manual methods, "seat of the pants" type approaches, and use of only available resources satisfy these needs. Inadequacies of traditional management practices became obvious. The catastrophic bridge collapse at Point Pleasant, W. VA., in 1967 was the turning point in management of bridges

in the U.S. Bridge inventory, inspection, rating, and posting programs were initiated, thus beginning a systematic approach to bridge management. As inspection practices improved, ability to assess bridge condition needs correspondingly improved. This improvement, coupled with rapidly deteriorating infrastructure condition, provided sufficient evidence to conclude that bridge condition needs far exceeded available resources and that comprehensive BMSs were needed.

As efforts to develop them began, the following inadequacies in current management practices became evident:

- Bridge data available from the Federal Structure Inventory and Appraisal (SI&A) forms represented a basic level that was not comprehensive enough for a desirable BMS.

- Most bridge management activity relied heavily on bridge condition while ignoring many other bridge needs.

- Current deterioration models were general in nature and not comprehensive enough to use for forecasting future bridge conditions. Further, load ratings were not included in deterioration models, rendering them ineffective in predicting remaining service life.

- Most bridge management practices pertained to project level decisions on a specific bridge. Network-level (a level of analysis that reviews and assesses groups of bridges) bridge management practices were either non-existent or essentially inadequate.

- Most bridge management activities were not based on sound economic analyses. Even when these analyses were done, user costs were usually not included or, if included, were not realistic.

- Bridge maintenance data are generally inadequate or non-existent.

- Systematic prioritization and optimization that could assure effective use of available fiscal and human resources, although needed, did not take place.

ENVIRONMENT FOR BMS DEVELOPMENT

Experiences of agencies that have undertaken BMS development clearly indicate that the development process and its success are very strongly impacted by the organizational environment in which the BMS is being developed.

The current BMS development environment typically has the following key characteristics:

- Bridge management decisions are made at different levels in each organization. For example, project-level decisions are being made at the operating level, while network-level programming decisions are being made at the planning level.

- BMS decisions will always be made by managers and not by any management systems. Managers do, however, need management systems assistance in making these decisions.

- Managers need decision-making assistance. Some of the areas involved are: ranking of needs, developing programs, predicting future conditions, etc.

- BMSs must include life-cycle-cost strategies. These are difficult to develop because existing maintenance data bases are inadequate to assist in these activities.

- Network-level bridge management decisions most often will require automation due to the size of data requirements and complexity of bridge decisions.

- Most managers are not familiar with mainframe computers although they are receptive to using microcomputers.

- User involvement during development is essential for success of any operating system.

MAJOR APPROACHES TO BMS DEVELOPMENT

Agencies involved in the BMS development process are inclined to adopt certain major approaches. These approaches provide the basis for development activities and also provide a logical methodology for decision-making.

For many agencies developing BMSs, basic objectives are similar. Consequently, the major approaches are also similar. Approaches, along with objectives, do differ in some BMS areas because of unique circumstances created by particular needs and available data.

Some common major approaches to BMS development activities are as follows:

- The first step has been a user survey to identify available and desired data, various report-needs and need for assistance in making decisions.

- Agencies have identified needs for decision support at the network and project levels. Typical network-level support is in terms of identifying, prioritizing, and predicting needs of a group of bridges. Typical project-level decision support is in terms of maintenance, repair, rehabilitation, or replacement decisions for individual bridges.

- Most emphasis to date has been on developing network-level decision support systems.

- Agencies developing BMSs systems have found it necessary to expand existing data bases to pick up data elements, such as vulnerability data not currently collected but essential for development of appropriate BMSs.

- BMSs are being developed in modular format, according to priority of their importance to the agencies.

- Basic network-level BMSs consist of the following core decision support modules: needs analysis, maintenance, rehab and replacement, work selection strategies, and development of capital and operating (maintenance) programs. The more comprehensive network-level BMSs consist of the basic BMS and certain other peripheral decision support modules. These may include prioritization, optimization, forecasting, estimating, and program monitoring.

- Basic project-level BMSs consist of the following core decision support modules: bridge-specific needs analysis and bridge-specific maintenance, rehab, and replacement work selection strategy. The more comprehensive project-level BMSs consist of the basic BMS and certain other peripheral decision-support modules, such as life-cycle analysis.

- Individuals involved with BMS development activities recognize that decisions will ultimately be made by managers, with the BMS providing necessary decision support. This is reflected in the manual override being built into most systems.

STATUS OF BMS DEVELOPMENT EFFORTS

The BMSs are in various stages of development in the United States and abroad. Most development efforts have been initiated by agencies responsible for large networks of bridges. The American Association of State Highway and Transportation Officials (AASHTO) and the Federal Highway Administration (FHWA) have for some time encouraged development of BMSs.

Pennsylvania has been a leader in BMS development.(9) The Penn DOT effort began in 1984 and was implemented as an operational BMS in 1987. It integrated

several data bases containing bridge data into one data base with approximately 400 data elements. This BMS includes a priority ranking procedure based upon minimum acceptable level and desirable level of service and the Federal Sufficiency Rating (FSR). It can provide cost estimating for maintenance/rehab/replace alternatives and is expected to have a future "what if" capability.

North Carolina has been also been active in BMS development.(3) It has a partially developed and implemented BMS, which began in 1983-84 with initial emphasis on maintenance management and is expected to be completed in 1991. This effort also utilizes a level-of-service concept to prioritize potential bridge projects. North Carolina's level-of-service concept includes load capacity, clear deck width, and vertical roadway under/over clearance.

Indiana started BMS development in 1987 and has partially completed and implemented its system.(4) Completion of a fully operational system is expected in 1992. Indiana's approach is similar to Pennsylvania and North Carolina in that it uses level of service to prioritize bridge work. In addition, it also evaluates bridge traffic safety. This approach enables Indiana to evaluate and account for bridge characteristics that may constitute hazards to traffic.

While Pennsylvania, North Carolina, and Indiana have pursued BMS development vigorously, BMSs are also being developed in other states, and in some provinces of Canada and abroad(7). There have been some concurrent efforts to create a generic BMS adaptable to specific needs of individual agencies. TABLE-I presents a brief summary of the status of these efforts.

AUTOMATING THE BMS

In concept, a BMS can be a manual system operable without use of a computer. In practice, agencies developing BMSs have recognized the need to automate their systems because of the size of their bridge networks and the complexities and inter-relationships among bridge-related decisions. User expectations play a decisive role in the selecting automating equipment for a BMS. Some considerations found by different agencies in selecting their computer equipment are as follows:

. Most agencies maintain their agency-wide data on either mainframe or mini-computers. There is a clear trend to have all transportation infrastructure information stored on the mainframe computer a state agency already has or is planning to acquire. Mainframe computers offer the most security, data storage capacity, speed, and potential for multi-terminal networking.

. Decision-makers who would use a BMS for their decision-support prefer the simplicity and user-friendliness of microcomputers. This is because use of menu-driven programs, cursor-controlled operation, and intra-program windows simplify the use of a program. Needless to say, most potential BMS users lack the sophistication to use modern high-powered main-frame computers. Their familiarity with microcomputers has improved greatly in recent years.

. Based on software applications and design, the latest generation of microcomputers has reached a very competitive processing speed. Their ability to store, retrieve, and process data can be made adequate to accommodate even larger BMSs. It is recognized that very large data bases may reside more conveniently on a mainframe computer that could be networked to a microcomputer system to act as a hybrid. Networking micros is possible and cost-effective.

. Agencies have recognized that program construction and modification, as well as the use of menu-driven software, is more difficult on a mainframe and requires personnel well versed in the

more complex operating languages of most mainframes. They have also realized that the many computer languages available for use on microcomputers (BASIC, C. Turbopascal, DBase, etc.) can make for a more flexible customized BMS. Further, report generation and graphics capabilities appear to be more readily available and useable with the microcomputers and accompanying color monitors.

Review of BMS development activities in the U. S. and abroad indicates a trend to rely on microcomputers in developing individual BMSs. This trend appears to be clearly justifiable on the basis of rapid improvements in microcomputer capabilities, in evidence so far and realistically expected to continue in the future.

STATE-OF-THE-ART OF FUTURE BMSs

BMS is an evolutionary process and the current state-of-the-art for BMSs is in very preliminary stages. For future BMSs to be truly effective operating decision-support systems, they must have the following features:

- A larger data base that would include bridge construction, inventory, inspection, maintenance, safety assurance (such as scour-related), planning, and programming information. This data base will typically reside on mainframe computers for large agencies (i.e., States) and on microcomputers for smaller agencies or as an interconnected version for joint use.

- The BMS structure will be in modular form, with each module packed with built-in sophisticated techniques, independently operable and subject to input data availability. These modules will use highly sophisticated methodology to provide better decision support. Data significantly impacting outcome of a decision will be clearly identified as decision-critical.

- The BMS will provide management and engineering support for the decision-maker and also allow for his/her manual override. The decision-maker will be able to reasonably predict, with help of the BMS, consequences of his/her decisions under various scenarios and will be able to run a sensitivity analysis to evaluate the impact of variations in underlying assumptions.

On the network-level, future BMSs will have the following specific capabilities(11):

- Needs Analysis: present as well as future program needs such as maintenance, rehabilitation, replacement.

- Program Selection and Coordination: capital and maintenanance programs under a variety of constraints.

- Program Effectiveness: monitoring and evaluating.

- Sensitivity Analysis: program effectiveness under a variety of assumptions and scenarios, e.g., level of funding and condition improvement of a network over a certain period.

- Report Generation: prompt and extensive sorting of available information.

On the project-level, future BMSs will have the following specific capabilities(11):

- Evaluation of current needs and prediction of future individual bridge needs: e.g., overlay in...years, painting...years.

- Prediction of remaining service life under a variety of scenarios: e.g., under varying levels of maintenance or repair.

TABLE 1. STATUS OF BMS EFFORTS

AGENCY	MAIN FEATURES AND STATUS
California	System being developed for FHWA, ranking "minimum acceptable" and "desirable" levels-of-service. Anticipated completion: 1992.
Florida	Ranking basis: level-of-service, system prioritizes repair/rehab needs. Anticipated completion: 1995.
Indiana	Ranking basis: level-of-service and bridge traffic safety. Anticipated completion: 1992.
Michigan	System to include network and project level analysis. Anticipated completion: 1993.
New Jersey	PC-based system to facilitate budget and resource allocations, project selection, and maintenance management. Anticipated completion: 1991.
New York	Prototype developed, ranking basis: structural condition, vulnerability, essentiality and serviceability. Anticipated completion: 1994.
North Carolina	Ranking basis: level-of-service, system partially implemented. Anticipated completion: 1991.
Pennsylvania	Ranking basis: level-of-service, large data base, provides cost estimation for repair/rehab/maintenance alternatives, operational.
Texas	System to assist forecasting maintenance and capital needs, project selection, and integrated planning. Anticipated completion: 1993.
Washington	Bridge deck management system operational, BMS will assist resource allocation and preventative maintenance planning, Anticipated completion: 1995.
Wisconsin	Life-cycle cost analysis and network optimization employed by system. System operational, continuing updates.
FHWA	DP-71 provides basis for network-level BMS development by states, identifies candidate elements for data base.
NCHRP	Report 300 published, Project 12-28(2) being completed, network and project-level BMS, identifies and details six BMS modules.
Ontario	Ranking basis: level-of-service, cost/benefit program now operational. Anticipated completion: 1993.
Manitoba	System to schedule work based on optimized cost and priority. Anticipated completion: 1991.
Denmark (Thailand)	Emphasis on bridge safety and optimal allocation of funds. Mostly complete and operational.
Finland	Ranking basis: level-of-service, network and project-level analysis planned. Anticipated completion: 1992.
Saudi Arabia	Emphasizes stochastic optimization and maintenance work scope selection. System operational.

- Available life-cycle strategies, their costs, and impact on required maintenance and life expectancy.

- Selection of appropriate work based upon a variety of criteria such as fiscal constraints, maximum detour lengths, etc.

- Engineering support for load rating, design, and drafting to ensure uniformity, consistency, and increased productivity.

CONCLUSION

The past decade has seen significant advances in development of bridge management systems to coordinate and improve management of all bridge-related activities. Some agencies have taken the lead, under constraints of the prevailing environment, and have developed their own bridge management systems while many others are in the process of developing them. BMS development has been summarized in this paper by focusing on major approaches taken by different agencies, status of their development activities as reported by them, and the state-of-the-art of automation relevant to BMS development. Current trends point to future BMSs being flexible, adaptable, user-friendly, and packed with built-in sophisticated techniques.

It must be noted that the purpose, comprehensiveness and capabilities of each BMS is solely determined by each developing agency.

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Analytical Approach to the Development of a Bridge Management System

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A study to determine feasibility and define the general structure of a bridge management system (BMS) for the Texas State Department of Highways and Public Transportation's 47,000 highway bridges is described. The decentralized nature of bridge management in the SDHPT's district offices suggests that a BMS for Texas should have primary applicability at the district level. Specific recommendations for the characteristics and scope of a BMS are presented. Development of the recommended BMS is planned to culminate with implementation in September, 1992, followed by a period of evaluation and refinement.

INTRODUCTION

A bridge management system (BMS) can be defined as a comprehensive method for making decisions about bridge management activities in a systematic manner. According to such a definition, the bridge management activities of most state transportation departments can be termed bridge management systems. The term BMS implies somewhat more, however. The FHWA defines a comprehensive BMS as "an integrated set of formal procedures for directing or controlling all activities related to bridges," (1). In this paper, the term BMS is used to mean *a computerized decision-aiding system incorporating both rational models and expert knowledge of bridge deterioration processes and effectiveness of management activities, and economic models of life-cycle costs, including an economic accounting of user as well as agency benefits and costs, in a process designed to provide a predefined level of service to the public at a minimum cost.* In the past few years, a significant amount of study has been directed at development of methods to accomplish bridge management with systems which approximate the above definition. Based on recent federal data, it appears that approximately forty percent of the states have developed, are developing, or are planning development of some form of a BMS (2). It is clear that the needs of the various states are different, and these differences are reflected by the various approaches taken in the development of the various BMS which have been reported in the literature. The FHWA is encouraging the various States to develop their own bridge management systems (1), and it is generally recognized that bridge management decisions supported by BMS will carry increased credibility in the federal Bridge Replacement and Rehabilitation Program (BRRP).

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SCOPE OF THE PROBLEM

There are at present approximately 46,783 bridges on Texas highways. This figure includes nearly 32,000 bridges "on system", or on highways designated as eligible for federal-aid, with the remaining approximately 15,000 bridges designated as off-system bridges, for which primary responsibility for maintenance belongs to a county, city or some other entity rather than the State Department of Highways and Public Transportation (SDHPT) (3). The approximately 47,000 bridges is easily the largest number of bridges of any state in the United States. Recent studies (3) have concluded that approximately 19.8 percent of all on-system bridges in Texas are classified as substandard due to structural deficiency or functional obsolescence. A much higher fraction of the off-system bridges are so classified. Projected needs for rehabilitation of existing bridges peak in the five-year period beginning 1995, at $2 billion/5 years or $400 million per year. This is more than 2.5 times the entire bridge expenditures by the Department in 1988, and will undoubtedly require additional sources of revenue.

The magnitude of the projected rehabilitation needs emphasizes the importance of good bridge management to Texas. The advantages of an operational BMS for Texas are anticipated to include:

- Better data and better access to data on bridge deterioration rates,
- More rational definition of level of service goals,
- Identification of optimal activities and budgets for both short and long-range planning scenarios,
- Better understanding of effectiveness of maintenance and management activities,
- Future integration with pavement management system (PMS) into a roadway management system (RMS),

- More uniform application of bridge management strategies across geographically and politically diverse districts, and
- More rational means for programming improvements and forecasting needs.

To study the feasibility of developing a BMS for Texas, and to define the recommended scope and content of a BMS, an SDHPT-sponsored study conducted by the Texas Transportation Institute (TTI) was initiated in September 1988. The recommendations of this two year study are reported in this paper. It is the present goal of the SDHPT to implement a BMS by September 1992.

DESCRIPTION OF THE STUDY

The primary objective of the study was to design a plan to achieve a cost-effective, comprehensive bridge management system which meets the needs of the SDHPT. To accomplish this objective, the research team worked closely with an advisory committee consisting of bridge managers from several districts, and various individuals in the bridge and maintenance divisions of the SDHPT. The purpose of the advisory committee was to obtain feedback on the effectiveness of present bridge management practices and to obtain expert advice on the needs and problems of bridge management at both the district and state levels. A review of bridge management systems and related literature nationwide was also accomplished, which included discussions with FHWA personnel and bridge managers and researchers associated with successful or evolving bridge management systems in other states, including North Carolina, California, Indiana, Pennsylvania, and New York.

In reviewing the needs expressed by district and state personnel, it was determined that many could be satisfied directly through implementation of a comprehensive bridge management system. Listed below is a summary of some of the key needs identified in the study that can be aided by development of a BMS for Texas:

- A supplement to the sufficiency rating system for setting planning priorities,
- An analytical approach to identification of bridge needs,
- Consideration of user benefits and level of service in making funding decisions,
- Consideration of life-cycle costs in programming activities at the state and federal level
- An intermediate-range plan for scheduled maintenance to accompany the replacement and rehabilitation funding programs,
- A method of gauging maintenance effectiveness, and
- A method of evaluating alternatives, including maintenance, over a network as well as for single bridges.

With these needs and goals in mind, a plan for the development and implementation of a comprehensive BMS was formulated. In the course of performing this research, several major tasks were accomplished, including:

- Identification of specific data required for the BMS,
- Establishment and evaluation of a list of cost-effective maintenance and rehabilitation work items, and
- Identification of suitable and necessary models and algorithms for incorporation into the BMS, including deterioration, life-cycle cost, and optimization procedures.

DETAILS OF PROPOSED BMS

The BMS recommended to the SDHPT includes the following elements: databases containing inventory data and unit costs data, deterioration models, feasible improvements knowledge-based system, life-cycle costs models, and optimization procedures. Each of these proposed elements will be addressed below.

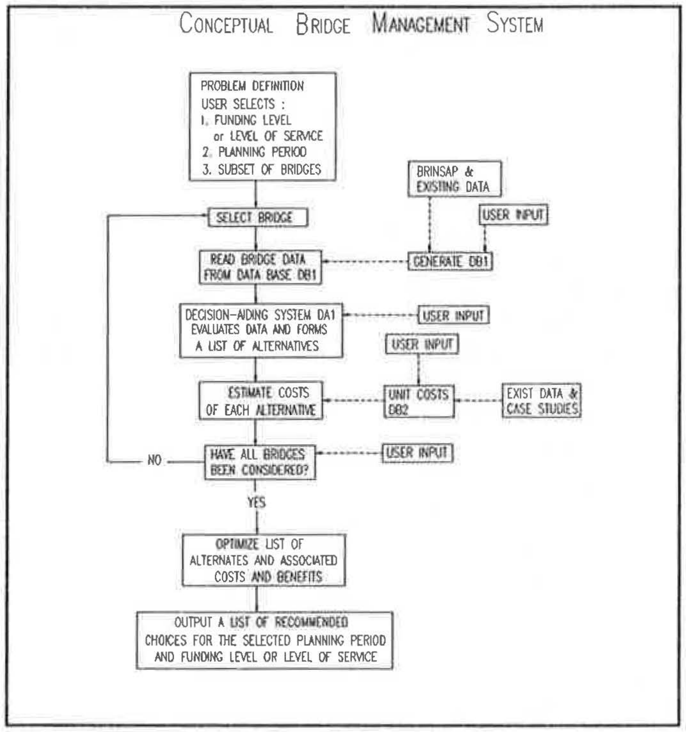

FIGURE 1 Flow chart illustrating recommended BMS for Texas.

The interaction between the elements is as described in FIGURE 1. The BMS may be activated by the user, for purposes of this discussion, a district bridge engineer, in one of two modes; short-term program planning, or long-term needs prediction. These two modes are distinguished by the different unknowns which will be determined by the BMS in each case. In the case of short-term program planning, it is assumed that the planning period is one or two years, and

that the budget available for that period is known and constrains the problem. In this case the problem becomes a determination of the optimal selection of projects and bridge management activities yielding the greatest economic benefit subject to the budget constraint. In the case of long-term needs prediction, the user is seeking to determine the required funding level to provide a predefined level of service for a longer period, such as five, ten, or twenty years. The user also enters the BMS with a user-defined subset of bridges under consideration. In the default case, this subset will be all the on-system bridges in the district, but other subsets can be defined by the user for specific studies.

The BMS considers each bridge in the selected subset individually, in turn, determining the required condition data from the database DB1. With the knowledge represented by this data, along with any other runtime data input by the user for specific bridges, the decision-aiding system DA1 synthesizes several feasible alternative bridge management activities. This list of feasible alternatives may be supplemented by user-generated alternatives in the case where the user desires to force economic consideration of some specific activity for a selected bridge. For each of the synthesized activities an initial cost and a life-cycle cost are generated. The agency benefits of the alternative are determined by comparison of the present value life-cycle cost with a reference life-cycle cost, and the user benefits are computed and summed with the agency benefits. The optimizing routine then determines the optimal selection of alternatives for that bridge. Once all bridges have been considered, the BMS outputs the recommended list of bridge management activities for the predetermined planning period, along with the initial costs and calculated benefits.

The user interface for the BMS is planned to allow runtime user interaction in both pre- and postprocessing modes. Through this interface, the user will be able to make queries to the databases, manipulate output reports, and provide input that might otherwise not be considered by the BMS. A geographic information system (GIS) is one viable alternative for meeting these pre- and postprocessing needs.

Bridge Inventory Data

A comprehensive and effective BMS requires a database or system of databases which is capable of supporting the various analyses involved in the various submodels of the BMS. As part of the development of a BMS, it was thus necessary to first identify the expected users of the BMS--in this case the various SDHPT district bridge management personnel, and the data needed by them, and also determine what types of BMS analyses can be accomplished utilizing the data items available in the existing databases.

There are basically three major types of data required to implement a comprehensive BMS: bridge inventory, condition and rating data; cost data; and improvement activity data, including replacement, rehabilitation and maintenance data. At the Texas SDHPT, the primary database containing such data is the Bridge Inventory, Inspection, and Appraisal Program (BRINSAP) database. An evaluation of the adequacy of the BRINSAP database included a study of the database structure itself and the relevance or usefulness of its data contents for incorporation into a BMS. One of the goals of this effort was to determine whether BMS analyses using only the existing data could yield useful results, or whether any additional data is needed. This question is still under study. As part of the BMS, a core database DB1 will be created primarily from the existing BRINSAP database to provide necessary bridge inventory, condition and rating data for analyses within the BMS.

Texas' Bridge Inventory, Inspection and Appraisal Program (BRINSAP)

In an effort to perform bridge planning, programming, project decision-making, as well as implement the National Bridge Inspection (NBI) standards as issued by the Federal Highway Administration (FHWA), a manual of procedures was prepared by the SDHPT (4) defining maintenance of the department's BRINSAP file. BRINSAP uses a computerized database to record, store, and update all data related to inventory, inspection, and appraisal activities on each bridge and tunnel on Texas public roads. While the main electronically-stored data is maintained by the department's headquarters in Austin, Texas, SDHPT District Engineers also keep a complete, accurate, and sometimes more detailed file on each bridge located in their respective districts.

The bridge inventory data in BRINSAP consists of items identifying the bridge type, location, general description, age, functional classification, condition ratings, etc. In addition to the basic NBI items, BRINSAP includes modifications to suit the Department's own needs, similar to databases kept by DOTs in other states, including New York and North Carolina (5). This is especially apparent in the case of condition ratings; NBI includes six condition ratings, but BRINSAP contains ten possible condition ratings (0 - 9) for each item, and additional details are also recorded during the bridge field inspection.

BRINSAP allows the bridge inspector to record the condition rating after condition assessment of the bridge, allocating a rating out of a possible ten values (0 - 9) as stipulated by the National Bridge Inventory Coding Guide (6). The condition rating of a major component (SI & A Item) is derived by selecting the lowest rating of any element of that component as illustrated in FIGURE 2. In addition to the ratings, the inspector may record comments on the condition of the component or its elements. While these comments and individual ratings are useful for detailed bridge analyses, only the overall component condition ratings are stored on the computerized BRINSAP database; the

remaining information can be accessed only manually, through the individual bridge folders maintained by the Districts.

TEXAS BRINSAP INVENTORY ITEM 58 - ROADWAY

Minimum		Condition Rating
1	Deck___	___
6	Wearing Surface___	___
6	Joints, Expansion, Open___	___
6	Joints, Expansion, Sealed___	___
6	Joints, Other___	___
6	Drainage System___	___
6	Curbs, Sidewalks & Parapets___	___
6	Median Barrier___	___
6	Railings___	___
7	Railings Protective Coating___	___
7	Delineation (curve markers)___	___
6	Curbs, Sidewalks & Parapets___	___
	Other___	___

Component Rating___

Comments:___

FIGURE 2 Excerpt from Texas' BRINSAP inspection form 1085-1.

Since any form of network analysis requires computer accessibility to the data required, the level of detail of any analysis conducted using the current data in BRINSAP will be limited to a consideration at the bridge component level only. To illustrate this point, consider the SI & A Item 58--Roadway. The condition performance of sealed joints, railings, etc. which are elements of this component, cannot be monitored on a network basis; the required condition rating of the joint or railing has been overridden by the available single overall rating of the Roadway component. In order to improve the current level of detail in network analyses, the element's specific condition ratings should be included in BRINSAP. Since these data items are already collected at the time of regular bridge inspection, no additional cost would be incurred; only a slight modification of the BRINSAP database structure is needed to accommodate this change. Moreover, these additional data items will give a more complete picture of the bridge condition and the maintenance and rehabilitation needs. The need for such data was initially identified during one of the research team's interviews with SDHPT District bridge personnel.

With the aid of a coding guide, the items in the computerized BRINSAP database were interpreted and reviewed to evaluate their relevance to the development of a comprehensive and effective BMS. An example listing some of the many items being considered for inclusion in the core database DB1 for the proposed BMS framework are shown in TABLE 1, along with a description of the relevance of each item.

The database DB1 is designed to interface with the main framework of the proposed BMS, with the option of user input provided to override or validate the data items generated from BRINSAP. The deterioration submodel and the knowledge-based feasible improvements generator will be able to read bridge data directly from this database.

Improvement Costs Database

Many of the analyses to be done within the proposed BMS will require cost data. To facilitate this, two steps will be taken in building a database DB2 which will contain unit costs of feasible bridge improvement and maintenance activities. First, it is necessary to identify the various possible bridge maintenance and rehabilitation (M&R) activities which are being performed, or which should be performed, and to evaluate the effectiveness and frequency of application of each of these activities on Texas bridges. Secondly, unit costs for each of the M&R activities will be developed, using historical data where available, or expert opinion when historical data is lacking. It may also be necessary to establish a procedure to capture future bridge maintenance and rehabilitation cost data.

Sources of available data on bridge-related expenditures at the SDHPT include the BRINSAP database, the Construction Projects' Average Low Bid Listings, Bridge Projects' Bid Listings, the Design and Construction Information System (DCIS), the Financial Information Management System (FIMS), and the newly developed Maintenance Management Information System (MMIS).

Bridge Improvement Activities

The only formal system for monitoring maintenance activities on the Texas Highway System is the recently developed Maintenance Management Information System (MMIS). In its original form, the MMIS was designed to monitor 21 highway maintenance functions, with none specifically designated for bridge maintenance (7). MMIS has recently been modified to monitor the ten bridge-related maintenance and rehabilitation functions shown in TABLE 2.

While these activities may be adequate for a summary level analysis of bridge expenditures, a network level bridge management scheme will require the monitoring of M&R activities at a more detailed level. There is a vast number of possible bridge rehabilitation and maintenance activities. For example, the bridge management systems used by the transportation agencies of Pennsylvania and North Carolina monitor 71 and 40 activities respectively (8, 9). While efforts are underway to increase the current number of bridge maintenance activities monitored by the SDHPT, the activities recommended for consideration by a BMS should be limited to the most commonly applied activities.

To achieve this goal, first, a "raw" list of possible bridge improvement activities was generated through a literature

TABLE 1 SAMPLE LISTING OF BRINSAP ITEMS FOR BMS CORE DATABASE DB1

BRINSAP Item	Relevance to a BMS
Items 5.1 - 5.6 (Principal Route)	Level of Service (LOS) Criteria
Items 6.1 - 6.2 (Principal Route)	LOS Criteria
Item 10.4 (Widening projects' history)	Rehabilitation Effectiveness
Item 19 (Bypass, Detour Length)	User Costs/Benefits
Item 20 (Toll)	Life-Cycle Cost Analyses
Item 24 (Federal Aid System)	LOS Criteria
Item 27 (Year built/last rehab.)	Deterioration/Rehabilitation Study
Item 28 (Lanes on structure)	LOS Criteria
Item 29 (Average Daily Traffic)	LOS Criteria and Deterioration Study
Item 32 (Approach Roadway Width)	Estimate Improvement Costs
Item 36 (Traffic Safety Features)	LOS Criteria and User Costs/Benefits
Item 41 (Operational Status)	User Costs/Benefits
Item 43 (Structure Type)	Deterioration Study and Feasible Improvement Strategies
Item 58 (Roadway Condition)	Deterioration Study
Item 59 (Superstructure Condition)	Deterioration Study
Item 60 (Substructure Condition)	Deterioration Study
Item 63 (Estimated Remaining Life)	Deterioration Study
Item 66 (Inventory Rating)	LOS Criteria
Item 67 (Structural Condition)	LOS Criteria
Item 68 (Roadway Geometry)	LOS Criteria
Item 69 (Vert/Lateral Clearance)	LOS Criteria
Item 70 (Safe Load Capacity)	LOS Criteria
Item 71 (Water Adequacy)	LOS Criteria
Item 72 (Approach Rdwy Alignment)	LOS Criteria

survey and review of historical data on bridge activities - SDHPT's Average Bid Listings, Bridge Cost Data Files, etc. Secondly, the initial raw list was refined to a shorter list using a structured opinion survey of the SDHPT bridge engineers and inspectors. At this stage, the importance of each activity was assessed based on the expert's own experience with bridge engineering and management in Texas. Because the feasibility study revealed a scarcity of bridge cost records, old bid records on bridge projects and expert opinions of SDHPT bridge personnel, regarding the relative importance and frequency of application of the activities, were determined in order to select and recommend the most important bridge rehabilitation activities to be monitored by the Texas BMS.

The MMIS should provide data and statistics on work performed as well as costs expended on various maintenance activities. It will also be a useful tool in analyzing and improving the productivity and efficiency of bridge maintenance programs. Since the system is designed to track activities on a highway by milepost location, it may be necessary to modify the MMIS database structure in order to track bridge maintenance activities. This problem is apparent in the case of two adjacent bridges on a divided highway; the two bridges have the same milepost. Possible solutions are to use bridge structure number instead of the milepost, or use milemarkers instead of mileposts. The MMIS appears to be flexible enough that such changes and addition of maintenance functions could be easily accommodated. In the future, it may be necessary to add to the ten MMIS bridge maintenance and rehabilitation functions, because of the many activities which may impact the decision processes in a BMS.

In addition to its current link with three other databases maintained by the SDHPT--Salary and Labor Distribution (SLD), Equipment Operating System (EOS), and Material Supply and Management System (MSMS)--MMIS could potentially be linked to other pertinent SDHPT systems such as BRINSAP, DCIS, FIMS, and the Roadway Information System (RIS). A similar integrated system has been successfully implemented by the Pennsylvania Department of Transportation (8).

TABLE 2 BRIDGE FUNCTIONS IN THE TEXAS MMIS

Texas MMIS Function No.	Bridge M&R Activity
610	Bridges, Movable Span
620	Channel Maintenance
625	Channel Maintenance (Under Bridge)
630	Bridges, Rail
640	Bridges, Joints
650	Bridges, Deck
660	Bridges, Superstructure
670	Bridges, Substructure
970	Bridges, Inspection (BRINSAP)
971	Bridges, Routine Inspection (non-BRINSAP)

Unit Costs Estimates

As mentioned above, for a comprehensive BMS to function accurately, it is necessary to develop unit costs for each of the considered bridge improvement activities. This could be done through statistical analyses of historical data where available; otherwise preliminary cost estimates could be used. Preliminary cost estimates of bridge replacement or major rehabilitation projects are usually obtained through the aggregation of unit costs of different work items as retrieved from low bid listings for past similar projects. The total project unit cost, or major component--deck, superstructure, or substructure--unit costs can be estimated using the bridge deck area as a unit of measurement. Simplicity aside, it is not practical to use a common unit cost value for all bridges, even for the same type of improvement activity. This is simply because bridge improvement projects are often unique to specific classes of bridges, especially substructure rehabilitation projects. It will therefore be necessary to categorize project unit costs by bridge type, improvement activity type, and also by geographic location. In a similar fashion, unit costs for each bridge maintenance activity can be estimated using preliminary cost estimates.

It will be necessary to establish a procedure and framework to capture appropriate cost data as costs accrue during the service life of the bridge. Hopefully, the MMIS will be modified to be capable of capturing bridge maintenance cost data on a regular and historical basis. After collecting this data on a historical basis, it will then be feasible to conduct case studies on bridge maintenance expenditures, which will contribute to the refinement of a comprehensive bridge management system for Texas.

Deterioration Models

Models of bridge condition deterioration may be classified as empirical, mechanistic, or stochastic. Empirical or regression based models are models in which the condition of a component is assumed to deteriorate at a prescribed rate determined by fitting quantitative data, sometimes supplemented with the use of expert knowledge. Regression models are the most commonly used models in existing BMS (5, 9, 10, 11, 12, 13, 14, 15), although examples of mechanistic models (16), and stochastic models (17, 18, 19) may also be found in the literature. Because of the present lack of comprehensive mechanistic models, it is not practical at present to base a deterioration model solely on a mechanistic approach, no matter how aesthetically appealing that may be.

The most commonly used stochastic model for representing the bridge deterioration process is the Markov chain approach. One mathematical limitation of deterioration models using stationary Markov chains is that the probability of transition from one state to the next is independent of the time that the bridge has occupied the present state. In reality, bridges stay in a specified condition state many years, and the probability of transition to a lower condition state will increase with time in any given state. Non-stationary semi-Markov processes are more suitable, but have not yet been applied to this problem.

The most sophisticated and realistic of the existing regression models is the nonlinear model developed by West et al. (15). The basis of the model is a condition rating function which depends on time in the following manner:

$$CR(t) = (1-X)\beta_1 e^{-\frac{t}{\beta_2}} + X\left(\beta_1 e^{-\frac{t_r}{\beta_2}} + \beta_3\right) e^{\frac{-(t-t_r)}{\beta_4}}$$

where

CR(t)	=	component condition rating,
t	=	age of the bridge,
t_r	=	age at time of major rehabilitation,
β_1	=	initial condition rating,
β_3	=	improvement due to rehabilitation,
β_2, β_4	=	reciprocal decay coefficients (yr), and
X	=	0 before rehabilitation, 1 after rehabilitation.

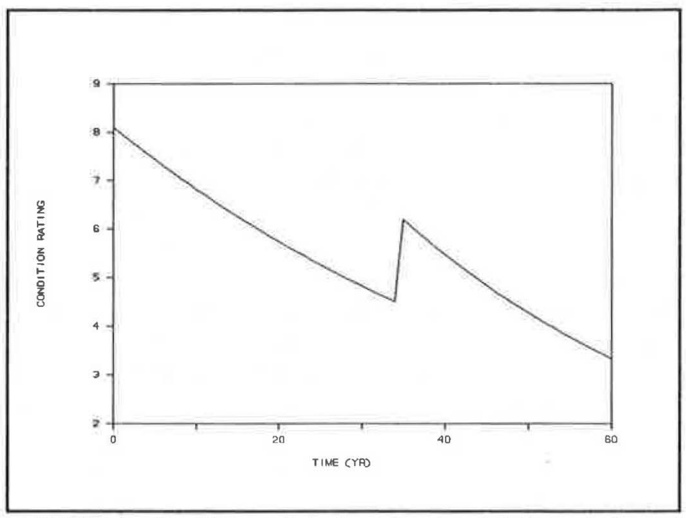

FIGURE 3 Nonlinear deterioration model for reinforced concrete bridge decks on prestressed concrete girders, with rehabilitation at 35 years (15).

This model is represented in FIGURE 3, with the parameters recommended by West et al. (15) for concrete bridge decks on prestressed concrete girder bridges carrying average daily traffic (ADT) in excess of 10,000 vehicles.

In its most detailed form, the model is a six-parameter nonlinear model which can include the effects of two different types of rehabilitation; a rehabilitation after 25 years of service being considered to have slightly different effects on the resulting increase in condition rating and on

the subsequent condition rating deterioration than a rehabilitation accomplished before 25 years of service.

Because of the simplicity and availability of regression models, the use of regression models from the literature, fit to Texas bridge condition data supplemented where needed by expert knowledge obtained of Texas SDHPT bridge engineers, is recommended for incorporation into the proposed BMS.

Feasible Improvements Model

The feasible improvements model will be a knowledge-based system which will synthesize a list of feasible alternatives for each bridge under consideration. Rules for such a knowledge-based system are available. The present SDHPT practice for identifying bridges eligible for rehabilitation and replacement have been incorporated into a series of computer programs (20) to allow an automated listing of bridges eligible under FHWA guidelines as defined in Texas SDHPT's Roadway Information System User's Manual (21). From this existing procedure, a series of rules will be developed which will approximate the existing procedure, generating a feasible alternative from the following set: do nothing, rehabilitate, or replace. The present procedure does not include more detailed alternatives, nor does it include consideration of different levels of maintenance effort. However, along with the rules developed from existing practices, other expert knowledge such as that suggested by Harper et al. (22) and Zuk (23), will be used to generate additional alternatives.

In the current SDHPT process, each bridge is assigned a weighted score, which is a linear combination of five variables; ADT, the average daily traffic; CPV, the cost of the proposed improvements divided by ADT; SR, the FHWA's sufficiency rating; DSS, the minimum value of the deck, superstructure and substructure condition ratings; and BWC, the bridge width condition, which is a binary-valued measure of the adequacy of the roadway width for ADT. The rules forming the basis for this process involve comparison of the weighted scores with user-defined limits for qualifying/marginal and marginal/non-qualifying bridge categories. In the present method, the bridges classified in qualifying and marginal categories are then ranked by increasing initial improvement cost, normalized by ADT. In the proposed procedure, the life-cycle costs and benefits will be compared for all feasible alternatives for all bridges under consideration, to select an optimal program of improvements.

As mentioned above, knowledge-based rules derived from the present logic used by the SDHPT, as well as rules suggested by Harper et al. (22), Zuk (23) and others, will be employed to assure that the set of feasible alternatives includes the optimal solution. Harper et al. (22) define a series of 84 rules which synthesize feasible maintenance and rehabilitation options from among approximately 40 possible scopes of work. The possible scopes of work include various feasible permutations of routine maintenance, repairs, rehabilitation and replacement of deck, superstructure, and substructure components. Zuk (23) describes an expert system with a set of 32 knowledge-based rules. These rules are fundamentally based on the sufficiency rating, but include consideration of historical significance, roadway width adequacy, replacement of obsolete bridges that inherently cannot be widened, strengthening of bridges to improve load capacity, terminal maintenance of bridges which are to be replaced in the near future, detour length during rehabilitation, replacement of bridges requiring frequent major repairs, timing of bridge replacement for bridges in areas of rapid ADT growth, and limited network considerations.

Life-Cycle Costs Model

In order to evaluate the alternative improvement strategies for every bridge on a network system, it is necessary to consider the corresponding costs to be incurred by the highway agency. These costs include the first or initial one-time cost, periodic maintenance costs, and possibly a rehabilitation or replacement cost as the bridge approaches the end of its service life.

While many of the current bridge project selection methods consider only the first or initial construction costs (5), it is more appropriate that the entire life-span costs, or simply the life-cycle costs associated with each alternative, be considered. Life-cycle cost analysis is particularly suitable for evaluating multiple alternatives which have unequal life expectancy, level of service, and/or maintenance costs. Based on the expected deterioration rate, costs required to bring a bridge back to a desired level of service are utilized to generate a life-cycle profile. Since life-cycle costs include future costs, a discount rate must be selected in order to combine future and present costs.

In the economic analysis, the cost of a bridge improvement alternative will be taken as the initial capital cost, or first cost, of the project. In addition, two types of benefits will be considered: agency benefits and user benefits.

Agency Benefits

Agency benefits are categorized as reductions in life-cycle cost resulting from actions of the agency, such as preventive maintenance, certain types of rehabilitation, and any other actions that effectively extend the service life of the bridge. More specifically, agency benefits are defined as the present worth of future cost savings to the department due to a bridge expenditure (5).

To determine agency benefits, the present worth of all future costs to the agency over the life of the bridge, such as

maintenance, rehabilitation, and replacement expenditures, are calculated for two different life-cycle cost scenarios. The first assumes no improvement is made to the bridge, thus the service life is not extended and replacement takes place at the end of the bridge's remaining life. This scenario constitutes the base line or reference life-cycle cost. The second case assumes that an expenditure is made to extend the life of the bridge for a specified number of years. This scenario determines the life-cycle cost for the improvement activity. The net benefits are thus equal to the life-cycle costs for the replacement alternative minus the life-cycle costs for the extended life alternative.

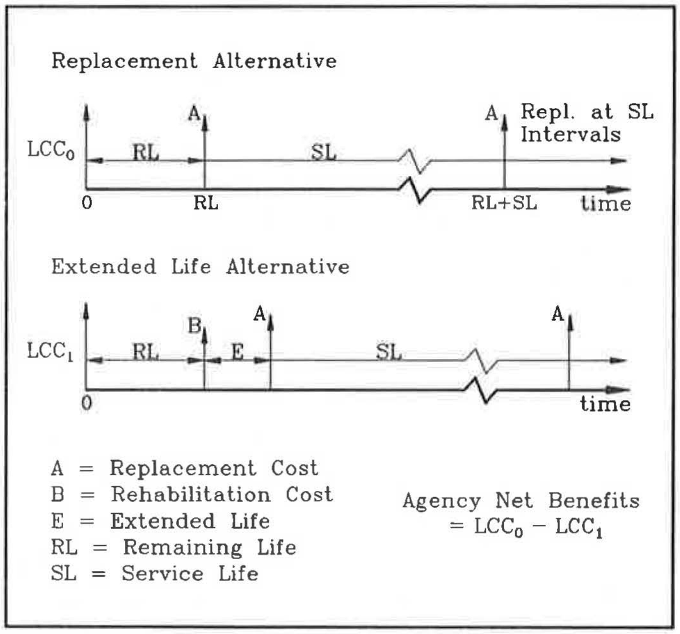

FIGURE 4 Calculation of agency benefits using life-cycle cost analysis (5).

Consider, for example, the illustration shown in FIGURE 4. There are two possible courses of action: (1) replacement of the bridge, or (2) rehabilitation and maintenance of a bridge to postpone eventual replacement. For a life-cycle cost comparison of these alternatives, the following information is needed:

E = the extended life of the bridge due to rehabilitation and maintenance.

LCC_0 = Present worth of life-cycle costs for the replacement case.

LCC_1 = Present worth of life-cycle costs for the extended life case.

The agency net benefit, B_a, associated with the extended service life case is computed as the difference between the two life-cycle costs involved, that is,

$$B_a = LCC_0 - LCC_1$$

If the rehabilitation expenditure is in fact determined to be of net benefit to the agency, the reduction in future agency costs, discounted, will be greater than the expenditure. If such a benefit is not feasible, the alternative was not of net benefit, and the agency should search for other options.

User Benefits

The second type of benefits to be considered by the BMS is user benefits. These are the benefits to the public resulting from actions that reduce user costs. User costs can be generated due to narrow width, low clearance, poor alignment, and reduced load capacity. Likewise, improvements that functionally upgrade a bridge, such as straightening the approach alignment, removing a load posting restriction, or increasing horizontal and/or vertical clearances all serve to reduce user costs. In general, any improvement that alleviates user costs prior to the end of a bridge's economic life is taken as a user benefit (5). These benefits are calculated by subtracting the user costs before an improvement from the user costs after the improvement. The user costs are generally computed annually and then discounted to a present worth in the same manner as agency costs. The benefits that will be considered primarily fall into three categories:

- Reductions in accident costs,
- Savings in vehicle operating costs, and
- Reductions in travel times.

In the last several years major advancements have been made in life-cycle cost analysis procedures and methods of calculating user costs for bridge alternatives. Two references present the state of the art in this analysis very well. The first of these is Federal Highway Administration's procedure for analyzing bridge alternatives (5). The second is the research done in North Carolina, particularly the report by Chen and Johnston (9). These life-cycle cost procedures represent advancements in the state of the art for bridge management and will provide the basic framework for the BMS described herein.

Accident Costs

Savings in accident costs result from bridge improvements that eliminate width restrictions and poor approach geometry. Several studies have shown that bridge width, roadway width, bridge rail design, roadway marking and signing, and roadway geometry are important in determining accident rates and severity. However, a review of these bridge studies showed that several had emphasized the importance of bridge and roadway width in determining the accident rate

(24). That is, vehicles tend to strike the bridge rail or bridge rail end when the bridge is narrow, either absolutely, or relative to the roadway width.

A more recent review of the literature (25) and a recent study of a large number of bridge accidents (26) confirmed this relationship and developed improved estimating procedures. The latter study used all reported accidents from the states of Arizona, Michigan, Montana, and Texas over a 3-year period, occurring on or within 500 feet of a bridge. This included 11,880 bridges and 24,809 accidents.

Other recent research has developed improved estimates of bridge-related accident costs (27). This research showed that fatality and injury rates for bridge-related accidents were among the highest cost accidents. The annual accident benefits of bridge widening are calculated as follows:

Annual Accident Benefits = (Change in Accident Rate) (ADT) (365) (Cost/Accident)

Time and Vehicle Operating Costs Associated With Detours

The cost of detouring vehicles when the load capacity or size of a bridge requires such action includes the extra time and vehicle operating costs associated with traveling the alternative route instead of using the preferred route. The benefit of not having to detour is equal to this cost and is calculated using the following two formulas (5):

Vehicle Operating Cost Savings From Not Detouring:
Savings = (ADT) (365) (Change in Fraction of Trucks Detoured) (Change in Distance Traveled by Trucks Detoured, in Miles) (Operating Cost Per Mile).

Value of Time Savings:
Savings = (ADT) (365) (Change in Fraction of Trucks Detoured) (Change in Distance Traveled by Trucks Detoured in Miles) (Value of Time Per Hour)/ (Speed in Miles Per Hour)

The FHWA manual uses vehicle operating costs developed in a New York study in 1981. Truck and passenger values of time have also recently been developed by McFarland and Chui (28). In addition, very detailed procedures for estimating the number of trucks that will be detoured for load limits and size restrictions have been developed by North Carolina (29). The above estimation procedures have been proposed for use in the BMS for Texas. However, the procedures for calculating time and vehicle operating costs associated with detours use average values and, consequently, are rough approximations of the expected costs of detours. Therefore, in order to provide more accurate calculations in cases where these costs are especially significant, it has been further proposed to supplement these procedures with an improved method that is based on research conducted at TTI.

This improved procedure was originally developed for evaluating highway bypasses and uses traffic relationships in the new highway capacity manual. It can be used in both rural and urban areas, and can be used to model the effects of closing one or more lanes, including complete shutdown of the bridge during major rehabilitation. This ability will allow the estimation of user cost savings experienced from building a new bridge at a different location while keeping the old bridge open to traffic during construction. This may be very significant if a critical bridge with a high traffic volume must be closed during rehabilitation.

Since general modeling procedures have already been developed for estimating the user costs described above, emphasis will be placed on refining these procedures. Special attention will be placed on developing procedures for each type of bridge alternative. Attention will also be devoted to evaluating procedures for estimating user costs during rehabilitation requiring bridge or lane closures.

Once the agency and user benefits for a prescribed alternative, have been determined, the total net benefits can be simply determined. For any proposed improvement activity,

Total Net Benefits = Agency Benefits + User Benefits

Optimization Routine

The bridge rehabilitation/replacement model is actually a special case of discrete optimization models. Discrete optimization problems abound in all situations concerning the management and efficient use of scarce resources to increase productivity. Particularly, in the last few years, applications of discrete optimization have rapidly developed because of the widespread use of microcomputers and the data provided by information systems.

The purpose of the optimization routine is to select one maintenance, rehabilitation, or replacement activity, including the do-nothing alternative, for each bridge of a given system, such as a state, district or part of a district. This is done in such a way that the total benefit derived from the implementation of the selected projects is maximized without exceeding a known budget. It is assumed that the following information is available:

- The group of bridges to be considered,
- A set of feasible alternatives for each bridge, and
- The cost and benefit associated with each alternative.

The basic bridge alternative selection model can be mathematically formulated as follows:

$$\textit{Maximize} \quad Z=\sum_{i=1}^{n}\sum_{j\in s_i} b_{ij}X_{ij}$$

$$\textit{Subject to} \quad \sum_{j\in s_i} X_{ij} = 1 \textit{ for each bridge } i$$

$$\sum_{i=1}^{n}\sum_{j\in s_i} c_{ij}X_{ij} \le B$$

$$X_{ij}=0,1 \quad \textit{for all } i,j$$

where the following notation is used:

n = number of bridges,
S_i = set of alternatives for bridge i,
b_{ij} = benefits associated with bridge i, alternative j,
c_{ij} = cost of bridge i, alternative j,
B = specified budget for a given planning horizon, and
X_{ij} = 1 if alternative j is chosen for bridge i; else 0.

The objective function maximizes the benefits resulting from a set of feasible bridge alternatives. The first set of constraints allows only one alternative to be selected for each bridge. The last constraint ensures that the total budget available is not exceeded. The above mathematical model is an integer programming model. A computationally efficient solution for the model can be obtained from a branch-and-bound integer programming procedure due to Nauss (30) or a special-purpose methodology based on a systematic analysis of incremental costs and benefits due to McFarland, et al. (31). Optimization routines based on these two procedures have been coded in FORTRAN and are referred to as INTPROG and INCBEN, respectively.

The computational requirements of INCBEN and INTPROG for the optimization of bridge-improvement alternatives for single-period problems have been investigated in a microcomputer environment. Both programs have been used successfully with hypothetical bridge data to identify an optimal set of bridge-improvement alternatives for up to 5,000 bridges with up to seven alternatives each. A comparison of the optimality of the solutions was accomplished, as well as a comparison of the computation time and hardware requirements for the two methods. Although INCBEN requires up to 1.5 times the memory storage capacity of INTPROG, INCBEN runs up to 3 times faster than INTPROG for certain data sets. For all practical purposes, the accuracy of the solution sets were identical for all data sets and budget constraints used in the investigation.

Description of Basic INCBEN Procedure

The basic procedure of the INCBEN algorithm (31) is summarized below. This algorithm ensures that the optimal set of bridge alternatives will be chosen for any cumulative cost:

1. For each bridge arrange all alternatives in increasing order of cost.

2. If there are several alternatives having the same cost for the same bridge, delete all alternatives except the one resulting in the largest benefit.

3. Calculate the ratio of incremental benefit to incremental cost for each remaining alternative for each bridge.

4. Delete any alternative for which the incremental benefit-cost ratio is less than one. This step is optional, and its applicability depends on accuracy of the quantified benefits.

5. For each bridge, compare the incremental benefit-cost ratio of the first alternative to that of the second one. If the second ratio is larger than the first ratio, combine the two increments to form a marginal benefit-cost ratio. Leave the first alternative in the array in case that budget limitations exclude the second alternative while allowing the first. Then compare the marginal benefit-cost ratio of the first and second alternatives to the benefit-cost ratio of the next lower cost alternative, and repeat this basic procedure. This will yield an "average" benefit-cost ratio.

6. Arrange all alternatives, along with their relevant corresponding marginal costs, in decreasing order of their relevant incremental benefit-cost ratios.

7. Initially, choose alternatives in order from highest to lowest incremental benefit-cost ratios, accumulating the corresponding marginal costs to determine which alternatives to include in a budget. Only the most attractive alternative is chosen for each bridge, and all less expensive alternatives for the same bridge are then excluded.

8. The last bridge alternative selected is dropped from the list of chosen projects, and the selection process continues adding as many projects as the remaining budget will allow. After this process is completed, the total net benefit of the initial set of bridge alternatives is compared to the total net benefit of the second set. The set having the larger total benefit is selected as the optimal solution.

Multi-Period Optimization

The above basic model can be efficiently used to develop the most cost-effective, or the most beneficial, bridge rehabilitation/replacement alternatives for a short-range scenario. If the problem of interest, however, is determining optimal

bridge decisions for each period of a long-range planning horizon, then it is necessary to combine the basic model with a dynamic programming approach in which the stages correspond to the periods.

In this multi-stage optimization problem, the state variables are defined as the budget remaining and the bridge conditions at each stage. The decision variables are binary variables indicating which strategy should be selected for each bridge in each period of the planning horizon. Since the condition of a bridge changes from period to period, the bridge condition deterioration process must be incorporated into the optimization methodology to take into consideration the effect of selecting or not selecting improvement projects for each bridge. This multi-period optimization approach exhibits the following attractive features:

- It allows the study of the interaction between periods, since a decision made in one period may affect decisions made in future periods,
- It is suitable for the investigation of several funding levels in each period of the planning horizon, and
- It accepts input from the bridge deterioration process for each intermediate period of the planning horizon.

The following notation will be used in the formulation of the dynamic programming recursive relationship for a multi-period optimization model:

A_t = amount available at the beginning of period t,
X_t = set of projects to choose from in period t,
$b(X_t)$ = total benefit for period t,
$c(X_t)$ = total cost for period t,
R_t = bridge conditions at the beginning of period t, and
$P_t(R_t)$ = set of feasible projects for period t.

Using the above notation, the subproblem for period t can be formulated as follows:

$$g_t(B_t) = \max_{0 \le A_t \le B_t} [f_t(A_t) + g_{t-1}(A_t + c(\vec{X}^*_{t-1})]$$

where

X^*_{t-1} = most benefit-effective decision at period t-1
$f(A_t)$ = max $b(X_t)$

subject to

$$c(\vec{X}_t) \le A_t$$

$$\vec{X}_t \in \vec{P}_t$$

In the above formulation, $f(A_t)$ can be determined in one pass of the INCBEN algorithm for all desirable values of A_t.

Geographic Information System

Geographic Information Systems, or GIS, provide an innovative approach for managing the large and diverse information required to support decisions concerning the highway and bridge infrastructure. A GIS can integrate the data collected by a transportation agency for use in both analytical and graphical applications. This technology is rapidly gaining popularity and is expected to play an increasingly major role in the daily operations of state transportation organizations in the immediate future. In fact, there are already several state agencies that are sponsoring projects related to the development and implementation of a GIS. These include the transportation departments in Texas (32), North Carolina (33), Pennsylvania (34), Colorado (35), and Alaska (36). The California Transportation Agency has digitized its entire highway network and can now graphically display every highway for which it is responsible. In addition to the activities at the state level, there are also ongoing efforts at the federal level by the FHWA and the American Association of State Highway and Transportation Officials (AASHTO) directed toward development of GIS technology and its applications in the area of transportation.

A GIS can be defined as a method of representing tabular data in a visual format as that information relates to a physical geographic location. Any type of data--physical, economic, demographic or other applied data--can be spatially defined within a georelational reference system. Layers of thematic data such as bridges, roadways, watercourses, district boundaries, county or municipal divisions, soils, and soils can be entered into the system as a series of map databases. The information is then joined or related to a series of tabular databases, such as inventory and cost databases, for analytical purposes.

The maps in the GIS are input as a series of arcs and nodes. Closed arcs form polygons which contain common thematic types such as soils, geologic formations, or ownership boundaries. This then becomes known as a polygon coverage. The area and perimeter of each polygon is automatically calculated by the program. A coverage which contains arcs representing linear data such as roads or streams is a line coverage. Nodes are placed at the intersection of arcs which represent roadway intersections, bridges, etc. Both distance and direction between nodes, or along a route, can be automatically calculated. Thus, within the context of a bridge management system, the GIS could be used to determine alternate routing around an obstruction such as a construction site, and to determine the length of that alternate route. Possible applications of these features include detouring traffic around a bridge rehabilitation project or rerouting oversize loads around low clearance or load-posted bridges.

Visual impact adds a second dimension to tabular data. A list of all bridges twenty-five years old with four lanes that have not had preventative maintenance in the last five years

is an easy search within a database. With the GIS, answers to such questions are readily accessible, and the locations of each of the structures satisfying a given set of variables are immediately available on a computer screen. Visual proximity analysis can lead to more cost effective repair schedules by optimizing the location of work crews and equipment. Examples of specific GIS applications include:

- Production of color coded maps showing bridge structures satisfying certain selection criteria or possessing certain attributes (e.g., deck condition rating < 3 and average daily traffic > 10,000), and
- Graphic representation of recommended activities such as scheduled maintenance, rehabilitation, or replacement.

In the proposed BMS for Texas, a GIS is being considered for use as both a pre- and postprocessor. Use of a GIS will enable the user to interface with the system and its various databases to obtain graphic images in response to various input and output queries and to automatically input the variables needed to run the model for a selected network of bridges. The results from the BMS can be made available in both tabular and visual format. In the pre-processing mode, the system can aid the bridge manager in selecting and visualizing a particular network of bridges. In the post-processing mode, the GIS can enable the bridge manager to query the tabular output from the BMS in order to visualize the answers to pertinent management questions, such as which bridges on a particular route need to have a concrete deck replaced, or be replaced entirely.

Another major advantage of the GIS system is its ability to establish a relational link among multiple databases. Once established within the bridge management system, the GIS could be linked with other databases within the SDHPT. Although beyond the scope of the proposed study, this type of communication among systems is feasible and would be the first step in the development of a total roadway management system.

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Demonstration Bridge Information System for Connecticut

ROBERT G. LAUZON AND IVAN KUZYK

The Connecticut Department of Transportation utilizes laser videodisc technology to store millions of frames of roadway images. The Photolog Laser Videodisc (PLV) System is a menu-driven PC-based workstation that is used to access images of any active state route. To maximize usage of the PLV hardware, the Department is actively developing other applications.

This report addresses the development of a Demonstration Bridge Information System (BIS) that makes use of the current PLV image-retrieval concept in conjunction with a relational database. The system was designed for storage and quick retrieval of both alphanumeric data and images concerning bridges.

The alphanumeric data is divided into four general areas on a bridge-by-bridge basis. Three areas contain static information such as structural dimensions, construction-project details, and on-site utility information. The fourth area chronologically lists events concerning a bridge, such as an inspection, rehabilitation, or maintenance activity.

Captioned and dated images of the bridge selected in the alphanumeric data areas are accessible as a menu selection. A hard-copy output of both the alphanumeric data and images are also available.

The entire State-maintained roadway system in Connecticut is filmed annually by The Connecticut Department of Transportation (ConnDOT). In 1985, the Department began using laser videodiscs to store its roadway images. (1)

A PC-based workstation, controlled by user-friendly menus, utilizes this technology to enable quick and easy access to images of any State-maintained roadway. This workstation is commonly known as the Photolog Laser Videodisc (PLV) System.

The PLV System has many users, primary among which is the Division of Pavement Management; this division conducts its pavement ratings of the State Highways with the System. Others, within and outside the Department, use the system to gather information about the roadway environs that would normally require field investigation. The images are also used to document past roadway conditions.

In April 1988, a PLV-based system dedicated to bridges was proposed. The proposal identified the inherent problems of the "metal file" system of storing and retrieving bridge-related information at ConnDOT. These problems have been accentuated by a major growth in this type of information, resulting from the State's 10-year $6.2-billion Infrastructure Renewal Program (IRP). The proposal called for the prioritization of available data, development of a methodology to store these data and retrieve them easily, demonstration of the feasibility of linking a database with corresponding images, while maintaining hardware compatibility with the existing PLV system, and integrating the developed system into a proposed management system. The expected benefits of such a system include the ability to simultaneously view images of, and obtain comprehensive information on any of the more than 3,500 State-maintained bridges in a timely and cost-effective manner.

The Connecticut Deprtment of Transportation, Office of Research and Materials, P.O. Box Drawer A , 24 Wolcott Hill Rd. Wethersfield, CT. 06109

GOAL OF THE DEMONSTRATION SYSTEM

Sources of bridge-related data are currently maintained by several different units in a variety of formats. These include both computer- and paper-based files. A thorough review of all these sources was conducted, as were interviews with operations personnel in these units.

As a result of this research, the following goal was established for the Demonstration Bridge Information System: provide a comprehensive easy-to-access source for Bridge Information that would be compatible with exisitng PLV hardware, consisting of:

a) a comprehensive history of what has occurred to a particular bridge, including references to inspections, construction projects, maintenance work, and any notable event related to the structure;

b) a comprehensive source of structural data, such as dimensions and foundation information;

c) a cross reference between bridge number and construction-project number;

d) a dedicated area where emergency information would be located; and,

e) access to video images of a bridge.

DEVELOPMENT OF THE DEMONSTRATION SYSTEM

Bridges Selected

Bridges of various type, size and age were selected for inclusion in the demonstration system in order to ascertain all the types of data that might be associated with bridges, and to demonstrate the image capability of a laser videodisc system for storing static photographic images. Personnel in the Department's Bridge Design, Bridge Safety and Evaluation (BSE) , and Bridge Maintenance Units were asked to recommend candidate structures for inclusion in the Demonstration System. Forty-three bridges were selected on the basis of these recommendations. The demonstration bridges were distributed among the State's four Transportation Districts to allow inspection of each District's record-keeping methods, and to increase the likelihood that at least one structure on the

TABLE 1 Bridges Included on Demonstration BIS Laser Videodisc

NUMBER	TOWN	LOCATION	TYPE
District 1			
448	Wethersfield	Rte 15/Berlin Tnpk	Steel frame
807	Wethersfield	Ridge Rd./ Rte 15	Concrete frame
808	Wethersfield	Rte 15/Folly Brook Blvd #3	Steel frame
809	Wethersfield	Rte 15/Wolcott Hill Rd.	Steel girder
810	Wethersfield	Rte 15/ McMullen Av.	Concrete frame
811	Wethersfield	Rte 15/Hartford Ave. & RR	Steel frame
860	Middletown	Rte 17/ Main St. Ext.	Steel stringer
1477	Newington	Rte 173/ Amtrak	Steel stringer
2163	Wethersfield	Rte 15/ Folly Brook	Concrete culvert
2299	Cheshire	Rte 42/ Ten Mile River	Concrete T-beam
3031	Rocky Hill	Gilbert Ave./ I-91	Plate girder
3242	Vernon	Factory Walk/ Rte 74	Wooden truss
4315	Plainville	Rte 72/ Rte 177	Steel girder
5231	Vernon	I-84 WB/ Rte 30 &83	Steel girder
5512	Southington	I-691 WB/ I-84	Steel box girder
5543	Meriden	Rte 15/ Abandoned RR	Concrete slab
District 3			
196	Branford	Rte 95/ Rte 1	Rolled beam
709	New Canaan	Rte 15/ Rte 106	Concrete frame
759	Stratford	James Farm Rd./ Rte 15	Concrete frame
770	New Haven	Rte 15/ Rte 243	Concrete arch
2475	Bridgeport	Rte 751/ Pequonnock Rv.	Lift Span
3772	Trumbull	Rte 25 SB/Private Rd.	Plate girder
District 2			
232	Westbrook	Rte 153/ Rte 95	Steel girder
343	Guilford	Rte 1/ West River	Pony truss
346	Madison	Rte 1/ Neck River	Masonry arch
362	Stonington	Rte 1/ Mystic River	Leaf bascule
484	Bolton	Rte 6 &44A/RR	Concrete tunnel
501	Killingly	Rte 395 NB/Rte 6	Steel girder
852	East Hampton	Rte 16/ Poco Creek	Concrete T-beam
1138	Haddam	Rte 82/ Ct. River	Swing span
1385	Westbrook	Rte 153/ Amtrak	Concrete box
1388	Old Saybrook	Rte 154/ South Cove	Timber beam
1415	Lisbon	Rte 169/ Shetucket Rv.	Steel deck arch
1918	Norwich	Rte 2/ P&W RR	Masonry tunnel
3979	East Hampton	Rte 66/Abandoned RR	Plate arch
District 4			
464	Watertown	Rte 6/ Steele Brook	Prstrsd conc slab
972	Barkhamsted	Rte 44/ Morgan Brook	Concrete culvert
1164	Middlebury	South St./ I-84	Steel arch
1338	Sharon	Rte 128/ Hstnc. River	Covered bridge
1343	Bridgewater	Rte 133/ Hstnc. River	Thru truss
1561	New Hartford	Rte 219/ Frmgtn. River	Thru truss
3349	Thomaston	Rte 8 NB/Rte 6 EB	Prstrsd conc gird
4385	Beacon Falls	Lopus Rd./Rte 8	Con. box beam

System would be familiar to a demonstration audience. The bridges are listed in Table 1.

Bridge Information Acquisition

As previously mentioned, bridge information within the Department exists in a wide variety of sources and formats. The value of this information is currently based on its use within individual units. The BIS is an attempt to assemble and centralize this information, and make it available to all units within the Department.

Construction plans were the first source of information reviewed for inclusion in the BIS. From these, structural information, such as skew angle, foundation type, etc., was accumulated. This information was stored in a large three-ring binder and later input to the BIS database. A complete listing of the attributes of the BIS database are shown in Table 2.

Although not comprehensive, a database dedicated to bridges already exists on the Department's mainframe computer. The information it contains is essentially static, i.e., location, route, span length, etc. Utilizing these data, the Department's Inventory, Planning and Data Section publishes a Bridge Log on an annual basis. This database was appended to the BIS database.

Original construction contracts were also reviewed. They are kept at the State Records Center in their original paper form and are often the only source for such diverse data as paint type and color, bridges included within a project, and contractor information. They are not readily accessible to ConnDOT personnel. The Record Center currently discards these contracts after 20 years.

Lists of all construction projects are available in one of two formats: on index cards kept at the Map & File Room, or on a sizable computer listing, generated by the Office of Scheduling and Monitoring, commonly known as the MIS.

These sources were reviewed. Any citations of relevance to the bridges in the demonstration system were recorded. In this way, a project history was established for each bridge.

Bridge Maintenance Information

After BSE personnel conduct their inspections, any work that they identify that could be performed by State Maintenance crews is assigned a priority. These priority assignments are reviewed by the Bridge Supervisor and Superintendent before maintenance work is scheduled. Over 95 percent of maintenance activity is scheduled in this manner. Monitoring of this

procedure, aspects of which are already computer-based, proved to be the most effective way to track and document the Department's maintenance activity. After the scheduled work has been completed, a confirmation letter is sent to Staff Maintenance, and the work item is purged from the computer file that lists pending maintenance work. Copies of the original work request and the confirmation letter are kept in the respective bridge paper-file in each district, as well as in the Staff Maintenance office. In a spot check of one District's Maintenance files, no reference to known work on a particular bridge was found.

The BIS database could be used to maintain this information as a precursor to a bridge management system.

Images

Since no comprehensive database dedicated to bridges exists at ConnDOT, considerable freedom could be exercised in developing the database for the BIS. Although the BIS was required to be compatible with the PLV workstation hardware, key elements, such as the programming language and database structure were chosen primarily on their relative merits. Development of the image feature of the BIS required much closer alignment to ConnDOT's existing image storage and retrieval methods. As stated previously, ConnDOT uses videodiscs to store its Photolog roadway images. The Department oversees the annual production of fifteen, Phillip's format, double-sided, constant-angular-velocity (CAV) videodiscs each year. These videodiscs hold over 700,000 normal roadway images, as well as an equal number of enlarged views of the pavement. These images are accessed using Photolog Laser Videodisc (PLV) workstations. Eleven workstations are currently in use throughout the Department.

Since their introduction at ConnDOT, the PLV workstations have come under increased usage each year. Annual savings from their use is estimated at $800,000. The idea of adding videodisc images to a comprehensive bridge database was based on the PLV System's success, and the desire to take greater advantage of the Department's existing PLV hardware.

Photographic images are an effective way of communicating complex technical information. Indeed, ConnDOT personnel and their consultants take thousands of bridge-inspection photographs each year. These photos are filed by the BSE unit and used as attachments to the Bridge Inspection Reports and circulated within the Department. The quality of the photocopied photographs is poor, often voiding the usefulness of the information they were meant to convey. In addition, the Department employs a photographer who, along with his other duties, obtains hundreds of unique bridge pictures each year. At present, the whereabouts of many of these photographs is not known, nor is there a medium for their distribution within the Department.

Videodisc Process

The Department photologs the entire state highway system using modified 35mm motion-picture cameras mounted in vans traveling at speeds of up to 40 m.p.h. The camera is attached to a sensor which monitors the vehicle's wheel rotations and triggers the shutter at one-hundredth-mile intervals. The Photolog film is then comprised of individual sequential images of the roadway taken at 52.8-foot intervals.

Once a route has been filmed, the film is developed by a private firm and edited by ConnDOT personnel. After editing, it is sent to a video-production facility for pre-mastering. Pre-mastering involves reviewing the film, fixing color and exposure levels, setting cues and transferring the images to 1-inch videotape. The 1-inch tape is then forwarded to a videodisc-production facility.

Excluding the occasional effect of heavy canopy or blinding sun, the exposure level on photolog film varies little from one frame to the next. When the film is pre-mastered, color and exposure levels can be set for a length of film comprised of thousands of frames. Once these levels are set, the film is transferred to tape at the rate of 30 frames-per-second.

Forty-three bridges were included on the Demonstration BIS videodisc. They were photographed, by a two-man crew, using 35mm cameras loaded with color slide film. Images of the elevations, the deck approach and egress, joints, abutments, piers, bearings and any other feature unique to the structure were obtained. The number of images for each bridge ranged from fifteen for a small structure to over one hundred for a large, complex structure. The average number of images per bridge was twenty seven. Aerial photographs of bridges in complex interchanges are available and were included.

After all 43 bridges had been photographed, the developed slides were organized and taken to a videotape-production facility in New York City for pre-mastering. Subsequently, the videotape was forwarded to the videodisc-production facility. The total turnaround time between the start of the pre-mastering process and receipt of the completed videodiscs was about three weeks.

Although the production of videodiscs from slides requires the same pre-mastering process as motion-picture film, slides have a distinct disadvantage. Unlike motion-picture film, the slides must be individually fed into the video-transfer device and taped one-frame-at-a-time. Exposure and color levels on motion picture film often remain constant for long sequences; bridge slides require constant adjustment since the light level can vary significantly from one slide to the next. The additional time required for the transfer of slides was well illustrated during the production of the Demonstration BIS videodisc. Over three days of studio time were required to transfer two thousand slide images to 1-inch video tape, while thirty thousand images on motion picture film were transferred in one afternoon. These factors seem to preclude the use of slides as viable image-capture medium because of the prohibitive time and labor requirements for pre-mastering. Possible alternatives to slides are currently being investigated by ConnDOT personnel. These alternatives include the use of half-frame format cameras and still-video cameras. In-house duplication of slides to motion picture film stock is another possible alternative. The Demonstration BIS videodisc contained images of approximately 1 percent of Connecticut's bridges.

With its capacity to store over 100,000 images, it is estimated that a single, double-sided, CAV format videodisc

would adequately store the images for all ConnDOT maintained bridges.

Software Development

The software used in the development of the BIS was chosen because of its compatibility with both the existing PLV-system hardware and the database software used within the Department for the storage of bridge-related information. Another consideration was the ability to support a communications library for control of the videodisc player. The configuration of the existing PLV hardware employs a nonstandard method of communication between the Personal Computer and the laser videodisc player. To avoid the problems inherent to this nonstandard communication, a switch was added to the hardware allowing direct communication between these two components. With this configuration, a suitable database compiler, and a communications library, the hardware can be utilized as either a PLV or BIS workstation. The BIS can then be appended to any existing PLV workstation at minimal cost. The compiler chosen allows unlimited distribution of the executable program code with no licensing limitations or cost. The database files that currently exist within the Department are in the dBase format and are utilized and maintained by their individual units. The dissemination of information in these files, as stated previously, is the main function of the BIS.

THE SYSTEM

The BIS was designed to be user-friendly and requires minimal computer knowledge. It is menu-driven and easily operated. The program flow is shown in Figure 1. First, the user is asked whether the video option is desired. Access to the video images can be suspended to increase the speed of the program initially. The main menu becomes operational after this choice is made. It consists of two levels. The first level allows selection between areas titled "Bridge Log," "Chronology," "Project," and "Crisis." The second level allows three means of access to the Bridge Log, Chronology, and Crisis areas. These are by bridge number, bridge name, or route, town and intersecting feature.

The Bridge Log area holds information that is currently available in the Department's Bridge Log, which is supplemented with structural information now only available on construction plan sheets. Items such as foundation type, skew angle, deck-membrane type, and utilities carried by the bridge can be displayed on the computer monitor or output as hard copy. The video images are also available from this area as a menu selection. They are displayed on a video monitor adjacent to the computer monitor. Their captions appear on the computer monitor and are scrolled concurrently with the images using the computer keyboard.

The Chronology area lists events that have occurred with regard to bridges since their construction, on a bridge-by-bridge basis. Included in this area are references to any construction project, inspection, maintenance work, or other notable event concerning a bridge. Project numbers, inspection dates, and maintenance reference numbers, all provide a cross reference to other sources of information within the BIS

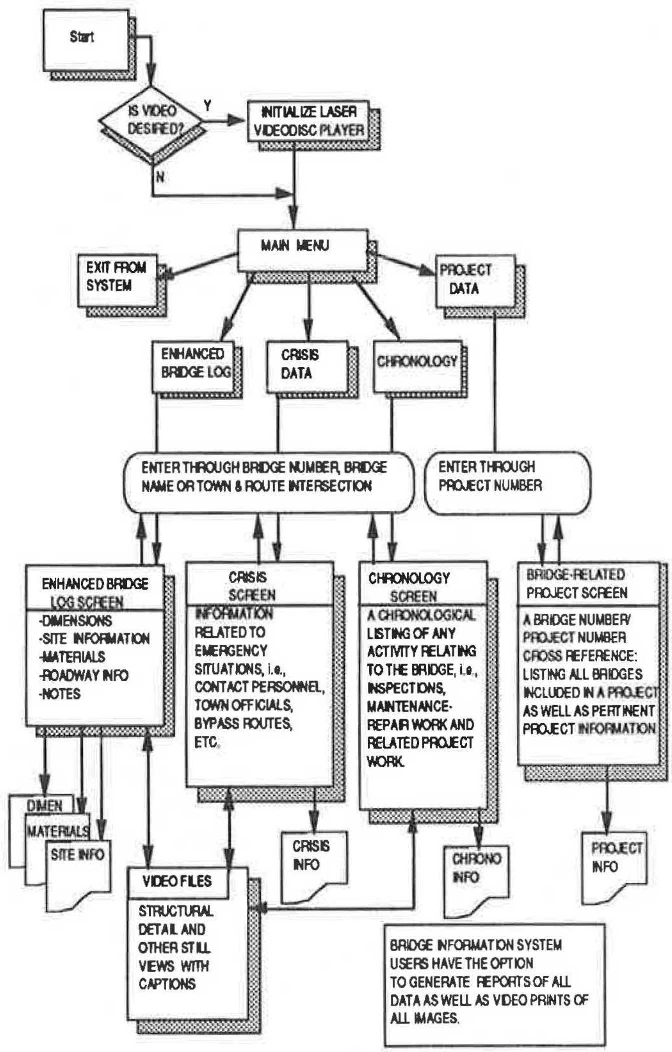

FIGURE 1 Bridge Information System Program Flow

and in other units supplying these data. Details of any particular event are also available as a menu selection. All the information described is displayed on the computer monitor and can be output as hard copy. The images can also be accessed by a menu selection in the same fashion as in the Bridge Log area.

As its name implies, the Project area contains specific information about particular bridge-related construction projects. The information is accessed by keyboard entry of the project number. Project information such as job description, designer, contractor, costs, and design specifications are listed. For projects involving many bridges, a listing of the bridge numbers is available.

The Crisis area contains information that would be required in an emergency situation. This includes telephone numbers of town officials, utilities, local and State police officials and Department operations personnel in the affected area. It also includes a list of on-site utilities, suggested bypass routes and storm-related design information, such as design flow for bridges over water. Images are also accessible in the previously described manner.

TABLE 2 Bridge Information System Data Attributes

Area 1 - Bridge Log		
Bridge Number		
Date Number Retired	Previous Bridge Number	Date Present Number Assigned
Bridge Maintainer	New Bridge Number	Bridge Owner
District	Historical Significance	Town
Longitude & Latitude	Location	Feature Intersected
Ramp Number	State Coordinates	Route Number(s)
Truck ADT	Milepost	Average Daily Traffic
Approach Span Type	Year of ADT	Main Span Type
Future Paving Allowance	Posted Weight Limit	Design Live Load
Construction Plan File No.	Bridge Designer	Contractor Information
Skew Angle	Number of Main Spans	Number of Approach Spans
Max Span Length	Flared Structure (Y/N)	Structure Length
Curb to Curb Width	Right Curb Width	Left Curb Width
Number of Lanes on Bridge	Deck Width, Out to Out	Approach Roadway Width
Bearing Type	Number of Lanes under Bridge	Median Type
Lighting Type on Bridge	Bearing Location	Railing/Fence Type
Membrane Type	Signs Located on Bridge	Wearing Surface Type
Deck Type	Type of Drainage on Bridge	Type of Drainage near Bridge
Length of Joint Types	Deck Protection Type	Deck Joint Types (A&B)
No. Pin & Joint Assemblies	Foundation Type	Number of Pins
Vert. Clearance Over Bridge	Number of Suspended Spans	Parallel Structure Number
Inv. Rte. Horiz. Clearance	Vert. Clear. under Bridge	Inv. Rte Vert. Clearance
	Lateral Clear. under Bridge	Utilities on/under Bridge
Area 2 - Chronology		
Year Built	Original Project	Maintenance History
Inspection History	Construction Project History	
Area 3 - Crisis		
Bypass Routes	Towns Impacted by Bypass	Emergency Contact Officials
Utility Company Contacts	Waterway Design Flow Rate	
Area 4 - Project		
Contractor(s) Information	Cost - estimated	Cost - actual
Bridge Number Cross Ref.	Date of Plans	Date of Completion
Federal Aid Number	Designer	State Form and Specification
AASHTO Soecification		

REVIEW OF GOAL

As a result of the demonstration of the BIS to personnel within the Department, it has been concluded that the goal of providing a easy-to-access source for bridge information with existing PLV technology was met. The system provides a historical record of all activity on a bridge, a cross reference between bridge number and project number, and captioned images of the structure. A dedicated area for emergency information is also provided.

CONCLUSIONS

The units responsible for routine maintenance work on bridges currently lack the resources to adequately document their work, while for project-level work, the documentation is given a low-priorty. With the vast amount of work being done under the IRP, little has changed to ensure that current information will exist 25 years from now. The same system that has tracked records for the last 100 years is being used to document the six-billion dollar IRP. The use of all available sources for bridge information, both textual and photographic, will provide an invaluable operational tool for every unit associated with bridges. The BIS will also establish basic information for use in the maintenance and design of bridges in the future.

REFERENCE

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Reliability and Load Modeling for Bridge Management

Michel Ghosn[1] and Fred Moses[2]

Managing the nation's bridge infrastructure requires major decisions on design and construction of new bridges, replacement, strengthening or posting of deficient bridges, issuing permits for truck overloads, and the implementation of truck weight regulations. These decisions have to be made after a careful review of the safety of the affected bridges. This paper illustrates how structural reliability theory can be used to provide tools for bridge management decisions. Using a reliability index as safety criteria, this paper describes reliability-based methods for the development of: a) Criteria for selection of load and resistance factors for a new LRFD bridge design code; b) Flexible load capacity evaluation or rating techniques for existing bridges; c) Fatigue evaluation procedures for steel bridges; And d) new bridge formulas for establishing truck weight regulations.

INTRODUCTION

Managing the nation's bridge infrastructure requires major decisions on design and construction of new bridges, replacement, strengthening or posting of deficient bridges, issuing permits for truck overloads, and the implementation of truck weight regulations. These decisions have to be made after a careful review of the safety of the affected bridges.

Insuring bridge safety requires a balance between the loads imposed on a bridge and the capacity of its members. Evaluating member capacity has traditionally relied on a visual inspection of the bridge while estimates of truck loads were obtained from the load envelope provided by AASHTO's HS or legal vehicles [1]. This traditional approach while very expedient ignores two fundamental issues: First, the current inspection process is very subjective and estimates of member capacities are associated with very high levels of uncertainty. Second, AASHTO's HS vehicles do not provide a consistent envelope over the entire range of bridge spans and geometries. Over the last few years, major efforts by the authors and other researchers have led to the development of new techniques for the evaluation of bridge safety using structural reliability theory [2,3,4,5,6,7,8].

Structural reliability methods contain the necessary ingredients for providing rational bridge management tools. Safety is expressed in terms of a measure of the probability that the capacity will exceed the load effect due to an extreme truck loading event. The maximum loading event is characterized by the number of trucks simultaneously on the bridge, their gross weights, axle spacings and the relative position of these vehicles. In addition, uncertainties associated with estimating the dynamic response and the distribution of loads to each bridge member are considered. A data base is used reflecting measured truck spectra, analysis variables, strength of materials, as well as projections of expected future loadings over bridge life spans. In this approach, design and evaluation factors are correlated to the uncertainties inherent in estimating the strength of members, the behavior of structural systems and the loading of bridges. In addition, the consequences of a single member failure for redundant or nonredundant spans can be accounted for using this reliability technique.

This paper illustrates how structural reliability theory can be used to provide tools for bridge management decisions. The work is based on the results of recent studies by the authors for FHWA, TRB, NCHRP and Ohio DOT on load modeling and reliability analysis of highway bridges [3,4,6,7,8]. Using a reliability index as safety criteria, this paper describes reliability-based methods for the development of: a) Criteria for selection of load and resistance factors for a new LRFD bridge design code; b) Flexible load capacity evaluation or rating techniques for existing bridges; c) Fatigue evaluation procedures for steel bridges; And d) new truck weight formulas for establishing truck weight regulations. The latter application requires a review of the existing bridge inventory and allocation of resources to improve productivity while maintaining a consistently safe bridge system.

STRUCTURAL RELIABILITY THEORY

Load intensity, load effect analysis and structural strength parameters are not known with certainty. The aim of structural reliability theory is to account for the uncertainties in evaluating the load carrying capacity of structural systems or in the calibration of safety factors for structural design codes. Such uncertainties may be represented by random variables and their probability distributions.

The standard deviation of a random variable R with a mean $\bar{R}$ is defined as σ_R. A nondimensional measure of the uncertainty is the coefficient of variation (COV) which is the ratio of standard deviation divided by the mean value.

Typical COV's for structural applications range from 8-15% for material strength, 5-10% for dead load, and 15-30% for live load and even higher for wind and seismic effects.

Codes often specify safe or nominal values for the variables used in the design equations. These nominal values are related to the means through bias values. The bias is defined as the ratio of the mean value to the nominal value used in design. For example, if R is the member resistance, R can be obtained from the nominal or design value R_n using a bias factor such that:

$$\bar{R} = b_r R_n \qquad (1)$$

where: b_r is the resistance bias and R_n is the nominal value as specified by the design code. For example, A36 steel has a nominal design yield stress of 36 ksi but coupon tests show an actual average value close to 40 ksi. Hence the bias of the yield stress is 40/36 or 1.1.

In structural reliability, safety may be described as the situation where capacity (strength, resistance, fatigue life, etc.) exceeds demand (load, moment, stress ranges, etc.). Probability of failure, i.e., probability that capacity is less

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than applied load may be formally calculated; however, its accuracy depends upon detailed data on the probability distributions of loads and resistances. Such data is often not available so approximate models are used for calculation.

Let the reserve margin of safety of a bridge component be defined as, Z, where:

$$Z = R - S = R - (D + L) \qquad (2)$$

R is the resistance or member capacity , S is the total load effect (S=D+L), D is the dead load effect, and L is the live load effect.

Risk is often described by a safety index (beta):

$$\beta = \frac{\bar{Z}}{\sigma_Z} \qquad (3)$$

where $\bar{Z}$ is the mean safety margin, σ_Z is the standard deviation of the safety margin. Probability of failure is the probability that the resistance R is less than the total applied load effect $P_f = Pr [R < S]$

If R and S follow normal distributions then:

$$P_f = \Phi [- \beta] \qquad (4)$$

Where Φ is the normal probability function that gives the relation between the probability that the normalized random variable exceeds a given value.

In general, beta' s from either normal or lognormal models are used as estimates of the reliability of a structural member even is its capacity and applied load are neither normal nor lognormal. To improve on these estimates "Level II" methods have been developed [9]. Level II methods involve an iterative calculation to obtain an estimate to the failure probability. This is accomplished by approximating the failure surface (Z=0) by a tangent hyperplane at the point on the surface closest to the origin, after the surface is mapped into a normal space.

The safety index approach has been used by many code writing groups throughout the world to express structural risk. Betas in the range of 2-4 are usually specified for different structural applications. For new bridge constructions, specified target betas are relatively high, say on the order of 3.5. But, looking at the entire bridge life span, typical betas may fall to about 2.5. These usually correspond to the failure of a single component. If there is adequate redundancy, overall system reliability indices (betas) will be higher.

Calculation of beta is not made solely for making statistical risk statements but rather for recommending the proper load and strength safety factors for design or evaluation specifications. One commonly used approach is that each type of structures should have uniform or consistent reliability levels over all applications. For example similar beta values should be obtained for bridges of different span lengths, number of lanes, continuous spans, roadway categories, etc.

Appropriate target betas are obtained based on existing designs. That is, if the public is generally satisfied with the safety performance of bridges designed according to current criteria, then the safety index obtained from current designs is used as the target that any new design should satisfy. This calibration with past performance also helps to minimize any inadequacies in the data base. The calibration effort is usually executed by code groups as follows:

- Safety indices are calculated for current code design and performance of existing structures based on statistical information about the randomness of the strength of members and the statistics of applied loads. For medium to short span bridges, the load S in equation (2) is divided into two parts: Dead load and live load. R on the other hand is determined by looking at the statistics of the resistance of typical bridge members. This is usually done for a range of applications such as different span lengths, girder spacings, material types and traffic conditions
- In general, there will be considerable scatter in such computed safety indices. If the existing code is believed to provide an average satisfactory performance then, a target *beta* can be directly extracted. This is done by examining the performance and experience with selected bridge examples and averaging the beta 's
- Safety factors and nominal loads and strengths for a new format are selected by trial and error to satisfy the target beta as closely as possible for the whole range of applications.

DATA BASE AND LIVE LOAD MODELS

To execute the safety index calculations, one needs to obtain the statistical data of all the random variables that affect the safety margin Z of equation 2. These are the member resistances, the dead load effect and the live load effect. Experimental and simulation studies have developed estimates for the statistics of member resistances for different types of bridges. Data on the live load statistics are however less common and in fact, besides the limited data from the Weigh-In-Motion studies at Case Western Reserve University, little information is available on bridge related truck load statistics in the United States [16]. This section presents a summary of the statistical data used by the authors and their colleagues in several studies on bridge reliability.

Dead Load

Dead load effects are obtained from the self-weight of the structure accounting for the weight of asphalt and other non-structural members. Hansell and Viest [10] found that the nominal dead load effect for steel members is related to the design live load and the span length by the formula

$$D_n = 0.0132 (L_n + I_n) SL \qquad (5)$$

where L_n is AASHTO's design live load effect on a member. I_n is the nominal impact load effect on the member. SL is the span length in feet. The mean dead load value D was also obtained from equation (5) ie. the dead load bias is estimated at 1.0. The dead load coefficient of variation used is 9% based on the typical values given in ref [11]. The same relationship between the dead load and span length was used for prestressed concrete bridge members based on the data provided by James et al [12]. Imbsen et al. [13] recommended a similar formula for concrete T beams such that

$$D_n=(L_n+I_n)(0.6967-0.00762SL+0.0002554SL^2) \qquad (6)$$

Resistance Data

Summarizing earlier work by Galambos [14] and Ellingwood [15], Moses and Verma [3] established biases and COV for different categories of steel members and prestressed concrete members. For example, steel members in new

condition were assigned a bias of 1.1 relative to the nominal capacity as specified by AASHTO procedures and a COV of 12%. Partially corroded steel members with some slight loss of section were associated with a bias of 1.05 and a COV of 16%. Severely corroded sections with noticeable loss of section have a bias of 1.0 and a COV of 20%. For prestressed concrete members in good condition a bias of 1.05 and a COV of 9% were used. These biases and coefficients of variation account for the uncertainties in the material properties, fabrication and scatter in prediction theory.

Live Load Modeling

Bridges are designed to safely withstand the maximum load expected over the service lifetime of the structure. In short to medium span bridges, maximum live load is usually due to the occurrence of several heavy trucks simultaneously on the bridge. Each occurrence of one or more vehicles on the bridge (herein called loading event) is characterized by the number of trucks in the event, their gross weights, axle spacings, axle weight distribution and the relative position of these trucks with respect to each other. All these factors are random variables which should be accounted for in a model to calculate the maximum loading on a bridge.

Simulation programs have been developed to study the truck loading problem [6]. In these programs, the bridge is divided into slots and a truck loading event occurs when there is at least one truck on any one of the assumed slots. Each truck involved in the loading event was assumed to be either of the single or the semi-trailer type. Each truck in the event will also have a different gross weight. Depending on the truck type, each truck involved in the event will be associated with a gross weight and a corresponding probability obtained from gross weight histograms for the different truck types considered.

Given the truck positions and given the gross weights of all the trucks in the event, the maximum moment response associated with the event can be easily calculated from the influence line of the bridge. The response of the bridge due to the event is also associated with a headway probability and probabilities of the gross weights of the trucks. The corresponding moment response is then associated with a probability equal to the product of the headway probability and the gross weight probabilities. This assumes independence between the headway and the gross weights and between the gross weights of the different trucks in the event.

Based on the results of the simulation program, the median of the total response of the maximum load in 50 years for a general truck traffic at a given site is approximated herein by the load formula [6]:

$$M = a\, m\, W_{.95}\, H \qquad (7)$$

where a is a deterministic value dependent on the standard truck configuration used in the simulation, the span length and the response variable (midspan moment, end shear ...). m is a random variable reflecting the type of truck traffic configuration present at the site e.g. single, semis, etc. It is also a function of span length. H is a random variable and gives the overload factor due to the presence of closely spaced vehicles, side-by-side and following vehicles. H also reflects the probability that vehicle weights exceed the 95th percentile in combination with closely spaced events. It was found from the simulation model as discussed in the previous paragraph and in reference [6]. H is a function of the truck volume and depends on the span length. $W_{.95}$ is a 95th percentile characteristic value of the truck gross weights.

Equation 7 is used in estimating the maximum live load applied on a bridge structure in its lifetime (usually taken as 50 years). This equation gives the total static load on a bridge. To obtain the load on a member under highway traffic, two additional factors are required and these are the impact factor (or dynamic amplification factor) ,i, and the girder distribution factor, g. The total load effect on a bridge member L is then the product of the maximum lifetime static load effect, the girder distribution factor and the dynamic amplification factor:

$$L = a\, m\, W_{.95}\, H\, g\, i \qquad (8)$$

For a fifty year design, possible growth in the weights of heavy trucks traveling over the highways should be included in the reliability analysis. The approach adopted here is to include load growth explicitly as one of the variables denoted as Gr. A mean Gr factor of 1.15 along with a C.O.V. of 10% were assumed for the evaluation of the safety indices. The live load formula used in the safety index calculations for new designs becomes:

$$L = a\, m\, W_{.95}\, H\, g\, i\, Gr \qquad (9)$$

Except for the factor a, all the variables of Eq. 9 are random variables with statistics based on examination of a number of sites as given in Table 1 for a fifty-year projection of the maximum load effect based on Weigh-In-Motion data collected at several sites [6].

Statistical data based on field measurements and theoretical analysis were collected by Moses and Verma on the girder distribution factor g. They found that for steel bridges a bias of 0.90 with a COV of 13% exists between the AASHTO recommended girder distribution factor and the values obtained by researchers. For concrete T beams, the bias obtained was 1.05 with a COV of 13%. Prestressed concrete bridges were associated with a bias of 0.96 and a COV of 8%..

Similarly, the impact factor was found to be a function of surface roughness. Three different values were recommended for the mean dynamic impact, these are 1.1, 1.2 and 1.3 for smooth, medium and rough surfaces respectively, these were all associated with a COV of 10%.

Two different $W_{.95}$ values are used depending on whether the span length is dominated by the loading of single unit trucks (spans less than 60 ft.) or dominated by the semi-trailer load (greater than 60 ft.). These are found to be 47 kips and 75 kips respectively.

The data used for new designs has to be averaged from several sites because no specific information is available on the type of trucks or the loading and response of the bridge before its construction and opening to traffic. Therefore, different statistics are used for the evaluation of existing bridges since site-specific data can be collected for those. Also, projections of maximum loads over fifty years are needed for the assessment of the risk over the design life span of the bridge. Shorter projection periods (two years) were recommended by Moses and Verma [3] for the evaluation of the safety of existing bridges. The two-year period was chosen because it corresponds to the bridge inspection period mandated by FHWA.

Moses and Verma also observed that truck traffic data can be divided into four categories which correspond to combinations of light truck traffic or heavy truck traffic on sites that have high truck weight enforcement levels or

unenforced sites. Different H and W values are recommended for each type of traffic [3].

BRIDGE CODE CALIBRATION

The first and most direct application of structural reliability theory and the bridge load modeling procedure outlined above is in the calibration of a new bridge design code. As stated above, development of a new code requires first the determination of a target safety index. Since engineers are generally satisfied with the performance of bridges designed with the current standards, Ghosn and Moses [7] decided to use the average safety index inherent in the current design practice as the target safety index that the new steel design code should achieve uniformly for all design applications.

Determination of Target Beta

The resistance R of a member designed using the current code has a nominal value that can be calculated from either the WSD or the LFD formulas. In both cases, the nominal load is obtained from the AASHTO design vehicle.

The calculations of the safety indices inherent in the existing code were executed assuming that all the random variables follow lognormal type distributions, the failure function used becomes of the form:

$$Z = R - D - a\, m\, W_{.95}\, H\, g\, i\, Gr \qquad (10)$$

The results of the safety index calculations are given in Figure 1 for the two design cases considered. As can be seen, the safety index beta for all of AASHTO's design possibilities is inconsistent from span to span. For example the WSD method gives a maximum of 4.33 for the 200 ft span while the minimum is 2.38 at 30 ft. For LFD, the maximum index is 3.70 at 60 ft and the minimum value is 2.56 at 30 ft. These inconsistencies in betas suggest two needed changes in the current design: 1) changes in the nominal design load formula to give safety indices compatible with the actual loadings on the members and 2) revision of the load factors to provide more uniform betas. The target beta was chosen here to be 3.5 based on the average index from AASHTO's WSD and LFD which have a combined average from all span lengths considered of 3.46. This same target value was also used by Nowak [17] in the calibration of Ontario Highway Bridge Code and is currently being used by Kulicki [5] for the calibration of the new AASHTO design code.

Proposed Strength Design

In order to obtain more uniform beta's over the range of span lengths considered, a new nominal design load was recommended [7]. This new design model consists of using different vehicle types with corresponding truck gross weights to calculate the maximum design load effect on a bridge structure. In this respect, it is closer to the load model contained in the AASHTO rating manual [18]. The partial safety factors associated with the proposed loading shown in Figure 2 are : a resistance factor of 0.9, a dead load factor of 1.3 and a live load factor of 2.35. These factors were obtained by a trial and error selection to achieve the target safety index of 3.5 as closely as possible for all span lengths. The proposed design checking equation becomes:

$$0.9\, R_n = 1.30\, D_n + 2.35\, (L'_n + I'_n) \qquad (11)$$

R_n is the nominal girder strength, D_n is the calculated dead load effect, L'_n is the static load due to the maximum effect from the three design vehicles shown in Figure 2. These vehicle configurations and axle loads were also selected by trial and error to achieve uniform betas over the range of span lengths considered (30-200 ft). Notice that the first two vehicles of Figure 2 are similar to the single unit and semi-trailer trucks most commonly observed on U.S. highways. The third vehicle in Fig. 2 is similar to double trailer type trucks and its moment effect replaces AASHTO's Lane Loading for simple spans up to 200 ft. Calculation of I'_n follows the same impact calculation contained in AASHTO. The load effect on a member is obtained using AASHTO's distribution factors on the wheel load, (such as S'/5.5 for steel girders where, S' is the maximum spacing between girders in feet).

The safety indices obtained from the new checking equation are very uniform when compared to the existing code [7]. For example, the maximum beta is 3.60 for a 150 ft span and the minimum value is 3.44 for the 200 ft span. These small differences between the maximum and minimum beta values confirm that the goal of more uniform betas was achieved from the proposed new design format including the changes in nominal load model and the load factors.

LOAD CAPACITY EVALUATION OF EXISTING BRIDGES

In a study for NCHRP, Moses and Verma [3] also proposed to use reliability theory and the load modeling techniques presented above to develop load capacity evaluation procedures for existing bridges. The new rating equations proposed by Moses and Verma have the following format:

$$\phi\, R_n = \gamma_D\, D_n + \gamma_L\, (L_n + I_n)\, R.F. \qquad (12)$$

where ϕ is the resistance factor, γ_D is the dead load factor and γ_L the live load factor. R.F. is the rating factor. Therefore, bridges which are just safe, according to this equation, have a R.F. equal to 1.0.

Determination of Target Beta

Establishing the target safety index was again an important step in the calibration process. In evaluation, there are at least four different procedures for rating currently in use in different States. These include the working stress WSR operating (WSR-O) and inventory levels (WSR-I) and the load factor LFR operating (LFR-O) and inventory (LFR-I) levels. Also, States may apply different distribution factors or nominal loadings (e.g. AASHTO legal vehicles, HS20 or other vehicles).

Figure 3 compares the betas for a set of hypothetical simple spans which just satisfy a R.F. = 1 based on WSR-I using present procedures. The live load loading category assumed is unenforced, heavy traffic and the nominal loading is taken to be the HS-20 design vehicle. The betas are compared to spans with a R.F. = 1 using LFR-I. It can be seen that the WSR-I betas are in the range of 3.8 to 4.6 for most spans while LFR-I betas vary between 3.0 and 4.6. The target beta (based on past practice) therefore depends on what criteria is used. Calculations for all the four traffic categories considered

show a large variation in the safety indices, depending on the severity of traffic (and hence the projected live load effect). The target beta values were selected based on the operating LFR for situations of enforced heavy traffic criteria. Using this approach the resistance and live load factors were calibrated to achieve a beta of 2.3.

For good condition nonredundant elements, the target safety index was derived from current inventory levels and an average value of 3.5 was chosen. It should be noted that the calibration was done using different levels of traffic for redundant and non-redundant elements. This is because calibration to the original design or inventory level should account for the severest traffic possible (which corresponds to unenforced, heavy traffic) whereas calibration to the operating level should be for enforced, heavy traffic ("reasonably controlled" according to the AASHTO Manual).

Calibration of Resistance and Load Factors

Based on the calibration procedure, a value of $\phi = 0.95$ was selected as the reference case for a good condition element for both steel and prestressed concrete. The value of ϕ is subsequently adjusted to recognize the data available from inspection and the degree to which maintenance can be expected to detect any important defects. For example, a member that shows a high amount of corrosion will have larger uncertainty in strength. This will produce a set of resistance factors as shown in Table 2. Thus, nonredundant components which require higher betas must be assigned a smaller ϕ to provide the higher reliability. γ_D was selected to be 1.20 .

Because they provide generally uniform reliability levels, and because they also are familiar to engineers, AASHTO's rating vehicles were used as the reference nominal loadings in the new rating formulation. In addition, a lane loading was developed to cover the long and continuous spans(Figure 4).

Different load factors are used to realize constant reliability for each of the truck traffic cases presented above. For example, the selection of γ_L for each case is illustrated in Table 3 to produce the same uniform beta value over all spans. Thus, sites with the heavier truck traffic will require a higher load factor.

In addition, the proposed format gives corrections on the resistance factors and the live load factors depending on the level of maintenance and inspection, the use of advanced methods or field measurements to estimate the girder distribution factors and the number of lanes of the bridge.

DESIGN AND EVALUATION FOR FATIGUE DAMAGE

Moses, Schilling and Raju [4] also developed a similar approach for the fatigue design and evaluation of steel members. The nature of the fatigue process however, required a modification of the failure function. The safety margin was then defined as:

$$Z = Y_F - Y_S \qquad (13)$$

where failure occurs if $Z < 0$. Y_F is life at which failure actually occurs and Y_S is the specified or calculated deterministic life. A linear Miner cumulative damage rule was assumed such that failure occurs when the nondimensional damage accumulation sum equals 1.0.

A new fatigue design and evaluation equation was then proposed involving a safety factor γ. Different safety factors γ achieve different safety index values (beta's). The safety factor to be selected is sensitive to the target beta and hence to the statistical parameters of the variables in the reliability expression. During the calibration process, the target beta was selected as an average of the beta's implicit in present design practice. Fourteen AASHTO design cases with different truck volumes, detail categories, spans, impact factors, lateral distribution factors and support conditions (simply supported or continuous) were used to evaluate an average beta implicit in the present AASHTO design practice. This is done by taking sections which just satisfy the AASHTO fatigue criteria and computing the implicit safety factor, β , by the methods described above. It was found that most of the design points fall around a beta of 2.0, where the range of betas is between 0.7 to 3.6. The target beta 2.0 corresponds to a safety factor, γ , of 1.35. The same analysis is repeated for nonredundant details and the results showed that the mean of the range of betas for nonredundant details appears to be about 3.0. This corresponds to a safety factor, γ of 1.75. From this analysis the target safety index for redundant and nonredundant members was fixed at 2.0 and 3.0 respectively, producing safety factors of 1.35 and 1.75. A fatigue design vehicle with a 54 kip equivalent weight along with a new method to select the girder distribution factor and the design stress range were also proposed to produce more uniform safety levels than currently observed [3].

DEVELOPMENT OF NEW TRUCK WEIGHT FORMULA

The same reliability and load modeling techniques were also used by Ghosn [8] to develop a new truck weight (bridge) formula to regulate maximum weights of trucks over federal interstate highways. The load modeling equation (eq. 9) was used to develop the new legal weight limit formula. The proposed procedure accounts for the truck load effect by considering the statistics of current truck gross weights, multiple occurrence on the bridge and truck configuration. Also indirectly considered in the H factor is the truck traffic volume. It is also assumed that changes in truck volume and other regulation changes over the service lifetime of the bridge might produce an average increase in the live load effect of 15% (Gr =1.15) the COV associated with this random variable is 10%.

Calculation of Required Load Effect Envelope

A target beta was also extracted from current design and loading as executed in the development of new design procedures. It was decided to use a target index equal to 2.5 to calibrate the new bridge formula. This value is similar to the average safety index for operating stresses which are regularly used by a number of states for rating existing bridges.

During the execution of the calculations, it was assumed that changes in the legal weight limits will produce a shift in $W_{.95}$ such that the ratio of $W_{.95}$ to the legal limit remains constant. The live load envelope required to produce an acceptable safety level is caculated as follows: Assuming that

current bridges are designed according to AASHTO's WSD method using HS 20 vehicles, the new mean live load $\bar{L}$ is calculated such that the target reliability of 2.5 is exactly matched for each span length considered.

Calculation of Truck Weight Formula

Using the load envelope developed, a bridge formula was obtained using the procedure outlined by the TTI study [12]. The bridge formula was designed to give a relationship between the weight of a truck and its length. The steps involved are summarized as follows:

- A truck satisfying the bridge formula was assumed to have a total weight W_t and total truck length B.
- Assume the truck weight W_t is uniformly distributed over the truck length B.
- Several values of B are used such that B varies between 1 and 120 ft.
- Given a truck length B, find the moment effect M_r of a unit load uniformly distributed over B. This is done for each span length.
- For every span length, find the load envelope M_t required to obtain the target safety index 2.5.
- For each span length, find $W_{.95\ t}$ (target weight) such that $m M_r\ W_{.95\ t}\ H$ is equal to M_t. a is not included herein since it is implicitly considered in M_r.
- Repeat the previous steps for every truck length B. For each truck length, several $W_{.95t}$ values are obtained corresponding to the load effect of every span considered.
- For each truck length B find the lowest $W_{.95t}$ value: $W_{.95t\ min}$.
- A bias factor gives the relationship between the legal load W_t and $W_{.95t}$. A bias of 1.07 is used for spans greater than 60 ft based on current truck weight statistics for semi-trailer trucks. The bias factor for shorter spans governed by single unit trucks is 1.09.
- Plot $W_{t\ min}$ versus B. This curve provides an envelope that the distributed load W_t should satisfy in order to insure that all span lengths will produce a safety index of at least 2.5.
- Find an algebraic expression that will fit the $W_{t\ min}$ versus B curve as close as possible. The proposed truck weight (bridge) formula obtained by fitting the $W_{t\ min}$ vs. B results is as follows:

$$W = (1.64\ L + 30)\ 1000 \quad \text{for } L < 50 \text{ ft} \qquad (14)$$
$$W = (0.80\ L + 72)\ 1000 \quad \text{for } L > 50 \text{ ft}$$

Cost Analysis

A major portion of the total cost impacts for new regulations is the effect on the existing bridge population. Some 130,000 bridges are now rated structurally deficient with an estimated $53 billion replacement or upgrading cost. If a new truck weight regulation introduced higher legal loads, a larger number of structural deficiencies will increase replacement costs. The procedure outlined herein for the cost analysis, follows the method developed by Moses for TRB's Truck Weight Study [19].

To provide a base case for the cost analysis, bridges under current truck regulations are checked by vehicles corresponding to the current Federal Bridge Formula. This means that they should provide adequate capacity under the AASHTO legal vehicles. A bridge is considered deficient under this base case scenario if these AASHTO vehicles cause stresses that exceed the operating stress level plus 5% tolerance. The operating stress level is obtained based on a safety factor 1/0.75 and a load L_n obtained from the AASHTO rating vehicles not the HS loading. These proposed criteria are similar to the rating methods used by many state agencies.

The cost allocation study considered a large sample of bridges that represent the highway classifications and regions in the U.S. Bridges of different spans, geometries, material and age were analyzed. The sample was obtained from the Federal Highway National Bridge Inventory System (NBI). The predictions using the Base Case model produced a total number of deficiencies close to the 130,000 estimated deficiencies which appears in the Secretary of Transportation annual report.

The current US bridge network consists of 600,000 bridges some of which would need upgrading if a new legislation allowing higher truck weights is implemented. In this analysis, it is assumed that all deficient bridges under any proposed legislation will have to be replaced. The possibility of posting or closing bridges has been ignored because these options will entail economic and productivity costs to the shipping industry exceeding the cost of replacing the affected structures. Upgrading options are seldom used in practice because of Federal rules requiring that upgraded bridges should satisfy all regulations on geometry, lane widths, side barriers... Thus the cost estimates given herein are upper bounds to the values that will actually be encountered.

As previously mentioned, additional deficiencies are defined with respect to operating stress levels plus 5% load tolerance using legal vehicles satisfying the bridge formula under consideration. Replacement cost of deficient bridges do not reflect the existing condition and age of the structures. Table 4 shows the estimates of current number of deficient bridges in comparison to the expected number under the new regulations given in eq. (14). This is presented for two different highway categories: primary systems and non-primary systems. The estimate is also given separately for steel, reinforced concrete and prestressed concrete bridges. The cost implications of such number of deficiencies is given in terms of total length of deficient bridges. Construction costs are usually given for unit area and since bridges have standard widths, the total length of deficient bridges is directly related to the total cost.

The results of Table 4 show that a large increase in the total length of deficient steel bridges on primary highway systems will accompany the implementation of equation (14). The increase in the length of deficiencies is about 5.7 times the current length. The additional length of deficiencies for the other types of bridges and highway classification is in general less than 2.9 the current levels.

One reason for this large number of deficiencies is that many existing bridges do not satisfy the WSD HS-20 criteria used to determine the target reliability level.

CONCLUSIONS

A review of recent work on the application of structural reliability theory and load modeling for the development of bridge evaluation methods has been presented. Bridge management requires the consideration of the structural safety of the bridge and the safety of its users. The techniques presented above illustrate how safety criteria can be included while decisions on bridge design, closing or rating are done or while determining the load capacity of existing bridges in order to allow higher legal limits or permit loads.

ACKNOWLEDGEMENT

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TABLE 1. Input Data for Reliability Analysis

Span (ft.)	a	m mean	m C.O.V.	H mean	H C.O.V.
30	6.07	0.92	15%	2.63	10%
40	8.57	0.93	12%	2.69	10%
60	13.57	0.94	6%	2.75	10%
80	13.40	0.93	9%	2.78	7%
100	18.40	0.95	7%	2.80	7%
125	24.40	0.96	6%	2.86	7%
150	30.90	0.96	5%	2.87	7%
175	36.90	0.97	4%	2.98	7%
200	43.40	0.97	4%	3.05	7%

Table 2. Resistance Factors for Different Member Categories

Resistance factors ϕ nonredundant	Resistance factors ϕ redundant	Comment
0.80	0.95	good condition
0.70	0.85	slightly corroded
0.60	0.75	heavy corrosion

Table 3. Live Load Factors for Bridge Evaluation

Category	Factor
- Low volume with reasonable enforcement and control of overloads	1.3
- Heavy volume with reasonable enforcement and control of overloads	1.45
- Low volume with significant overloads	1.65
- Heavy volume with significant overloads	1.80

TABLE 4. Consequences of Implementation of Proposed Bridge Formula on Simple Span Bridges

	Total # of Bridges	Current Deficiencies Number	Current Deficiencies Length x 10^3 ft	Increase in Expected Deficiencies Number	Increase in Expected Deficiencies Length x 10^3 ft
Primary Highways					
Steel	41140	1738	119.3	7248	679.5
R/C	25510	1174	43.7	2346	124.4
P/C	26474	1264	95.6	2896	249.7
Non-primary Highways					
Steel	154401	80243	3759.1	18742	1234.5
R/C	81671	21510	652.8	11957	521.0
P/C	48568	5503	247.5	6079	378.9

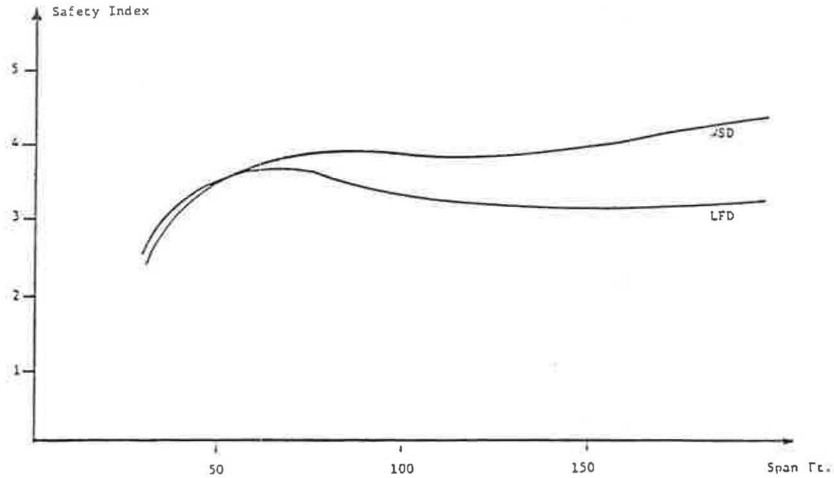


Figure 1 - Safety Indices with Current AASHTO

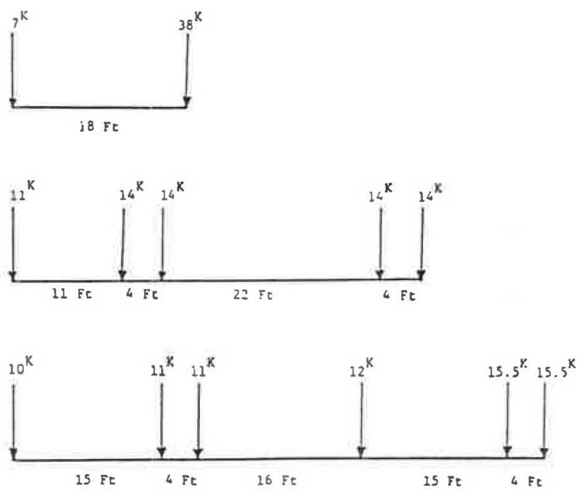


Figure 2 - Proposed Design Vehicles

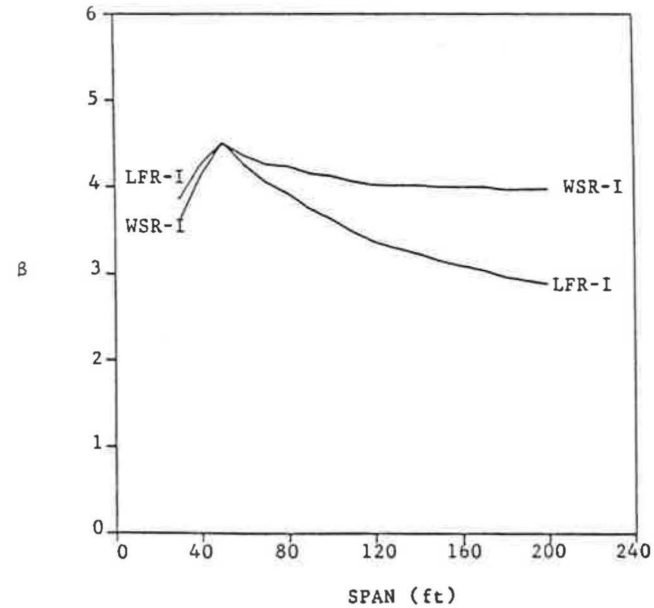
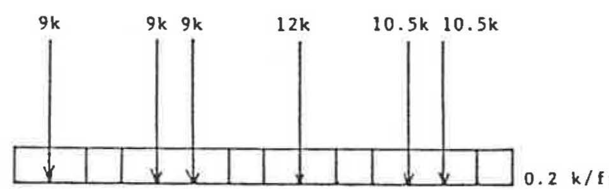


Fig. 3 - β for inventory checks (unenforced, heavy traffic)



- Lane loading

Figure 4 - Proposed lane loading for Load Capacity Evaluation

Bending and Bond Behavior of Concrete Beams Reinforced with Plastic Rebars

SALEM S. FAZA AND HOTA V. S. GANGARAO

Corrosion related deterioration of constructed facilities, such as coastal and marine structures, bridge decks, chemical and wastewater treatment plants, results in costly repairs and leads to user inconveniences. To improve the longevity of these facilities, the use of non-corrosive fiber reinforced plastic (FRP) rebars offers an alternative to mild steel rebars as reinforcement in concrete structures. Because the mechanical properties of FRP rebars are different from those of steel rebars, the performance of the FRP rebars embedded in concrete is not fully understood. As an example, the tensile strength varies with the diameter size of the rebar. For an E-glass FRP rebar with 55% glass volume fraction, the ultimate strength is 130 ksi for a #3 rebar and 80 ksi for a #7 rebar, based on gross area of rebar. In addition, the modulus of elasticity is about 7 x 10^3 ksi which is about 1/4 of mild steel. There is an urgent need for information regarding the bending and bond behavior of concrete sections reinforced with FRP rebars. A series of laboratory tests on concrete beams reinforced with FRP rebars is outlined in this paper. The major emphasis of these tests is to evaluate concrete beams reinforced with FRP rebars in terms of the stress-strain behavior, load-deflection variations, load carrying capacities, crack patterns, modes of failure and bond strength.

Recent reports and publications on the use of glass fiber reinforced plastics suggest the desirability of a review of research and development on the use of this material for reinforced concrete (1,2,3,4). In addition, published reports on the prestressing work in West Germany on the UlenBergstrasse bridge were reviewed (5,6). Current engineering reports and publications present very few experiments conducted on the bending and bond behavior of beams reinforced with FRP rebars with inconclusive results and no established design recommendations.

The influence of the following variables of FRP rebars on bending and bond strengths is evaluated on a systematic basis:

1) Rebar surface condition (smooth versus ribbed versus sand coated).
2) Diameter.
3) Fibers and resins types.
4) Fiber volume.

More specifically, this paper synthesizes the experimental results obtained by testing twenty-two rectangular beams under pure bending and twelve tests under bond forces (pull-out) using a stub cantilever test frame. The experimental results of the beams reinforced with FRP rebars tested under bending and bond will be evaluated in terms of the existing theories that are being employed for concrete specimens reinforced with mild steel rods after the completion of the testing program. The mathematical models and design equations of concrete specimens reinforced with mild steel will be modified according to the experimental results generated for FRP reinforced concrete beams.

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EXPERIMENTAL PROGRAM

Specimen Fabrication and Curing

Bending specimens were cast in wood forms whereas the bond specimens were cast in steel forms. In the case of the bond specimens, a thin wall conduit was inserted over and along the unloaded end of the test rebar to eliminate bond, thus giving the desired embedment length, EL.

After assembling the forms, the surfaces were oiled for easy removal. Before redimix concrete was placed in the forms in layers of about one third of the depth of the specimen, the reinforcement was positioned carefully and the overall dimensions were checked. Next, a portable electric vibrator was used to vibrate each layer. The surfaces of the specimens were leveled to a smooth surface. Test cylinders were cast at the same time as the specimens. For each mix of concrete, the slump was measured.

The specimens and cylinders were then covered with plastic sheeting and allowed to cure in the forms for at least seven days. After removing the specimens from the forms, they were allowed to cure at ambient conditions for three weeks. For each bending or bond test, two cylinders were tested. ASTM C-39 test procedure was followed to determine the concrete compressive strength f_c'.

Bending Test

The rectangular beams, Figure 1, were tested under pure bending, (as simply supported under four point bending), using different configurations of FRP reinforcements, such as rebar size, type of rebar (smooth, ribbed), and type of stirrups (steel, FRP smooth, FRP ribbed). The applied force was measured by a load cell connected to a strain indicator, Strain gages were carefully selected and placed on the reinforcement as well as on the concrete top surface. At every load stage, the strains were recorded with the use of strain indicator. A dial gage positioned at the center of the beam was used to measure deflection at each load stage. In addition, cracks were monitored, sketched and measured.

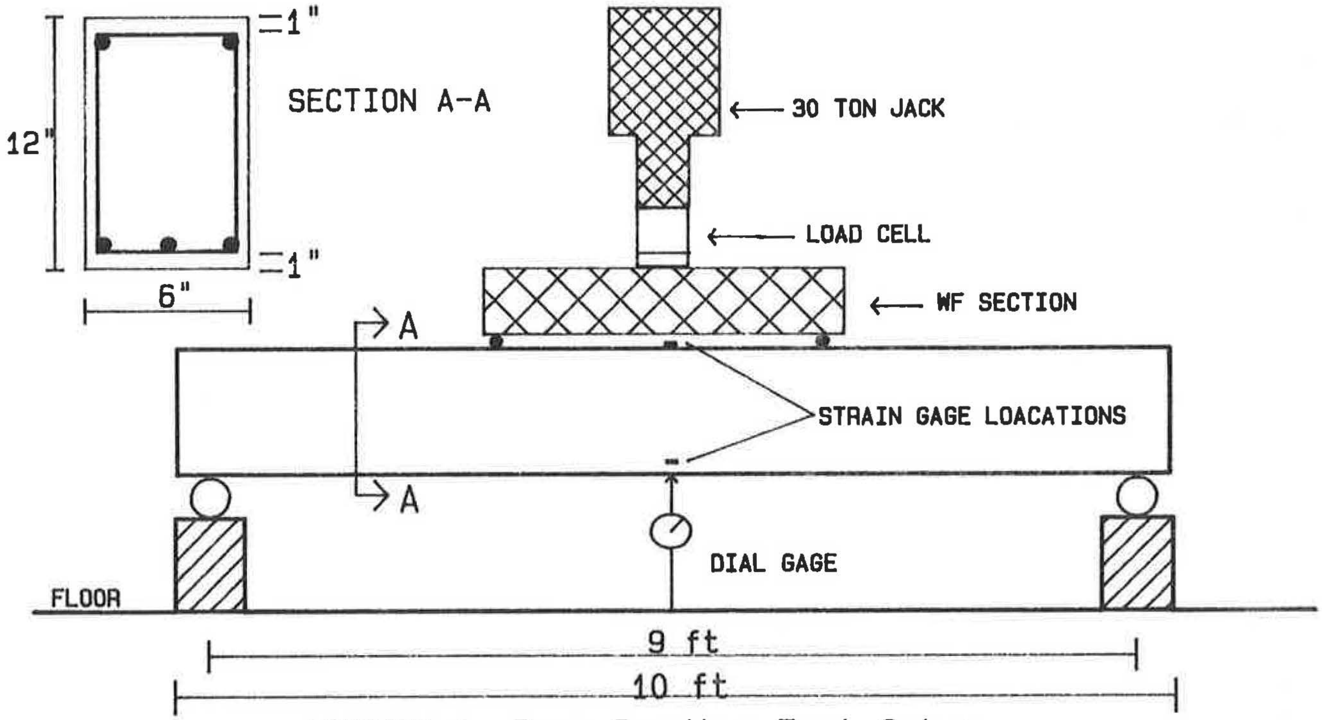

FIGURE 1 Beam Bending Test Setup

Bond Test

All specimens were similar to the WVU modified cantilever type shown in Figure 2. The front concrete end block was cut out in these specimens for modeling the portion of a beam adjacent to a diagonal crack. The compression zone of the beam was designed to prevent premature crushing failure. The specimens were designed so that no shear or moment failures were expected to occur during the tests.

Testing Rig

The specially designed and fabricated testing rig utilized for this study was originally developed by the Civil Engineering Department at WVU in the early 1970's (8). It consisted of a frame and two load reaction systems as shown in Figure 3. One of the load reaction systems was a buildup steel bearing beam, used as the horizontal support at the front end of the specimen. The other load reaction system, used for the vertical support at the rear end of the specimen, was composed of a 100-ton jack, a 60-ton load transducer and a steel bearing plate. The bearing plate was used to reduce the local bearing stresses which might affect the failure mode and crack pattern. At the front end of the specimen was an adjustable WF beam carrying the vertical reaction down by 4 vertical rods to the horizontal girder. Two 1 x 12 x 26 inch steel plates were bolted together to act as load distribution plates with an adjustable gap as shown in Figure 4.

The load distribution plates rested on four bolts. During testing, the bolts were released to avoid any undesirable restraints. For additional details refer to Figure 4.

Loading System

The applied horizontal bar force was produced by a pair of 60-ton jacks and was transmitted to the FRP rebars through the distribution plates and sand coated grips shown in Figure 5. The applied forces were measured by 2 load cells connected to a strain indicator and were increased in increments of 1 kip.

Gripping Mechanism

In earlier stages the design and development of suitable grips to pull FRP rebars embedded in concrete have presented some difficulties. Typically, the FRP rebar is found to fail in the grip itself due to combined effects of local horizontal shear and crushing. An ideal grip must grasp the rebar in a manner as to avoid failure of the rebar at the grip. Several methods for anchoring the FRP rebars have been investigated as a part of this study. However, the sand-coated grips filled with sand are found to be most suitable to transfer axial bar forces, Figure 5.

After placing the FRP rebars between the two sand coated grips, the set of grips are tightened together by six bolts.

Test Procedure

The specimen was seated in its proper position as shown in Figure 3 on the testing frame. After applying the initial load of 5 kip, the bolts which support the distribution plates were released. The test was continued by loading the specimen in an increment of 1 kip per load stage. At each load stage, dial gages were recorded and the crack pattern was sketched on the specimen. The test was stopped when the rebars could not hold any extra load, i.e. slippage was evident.

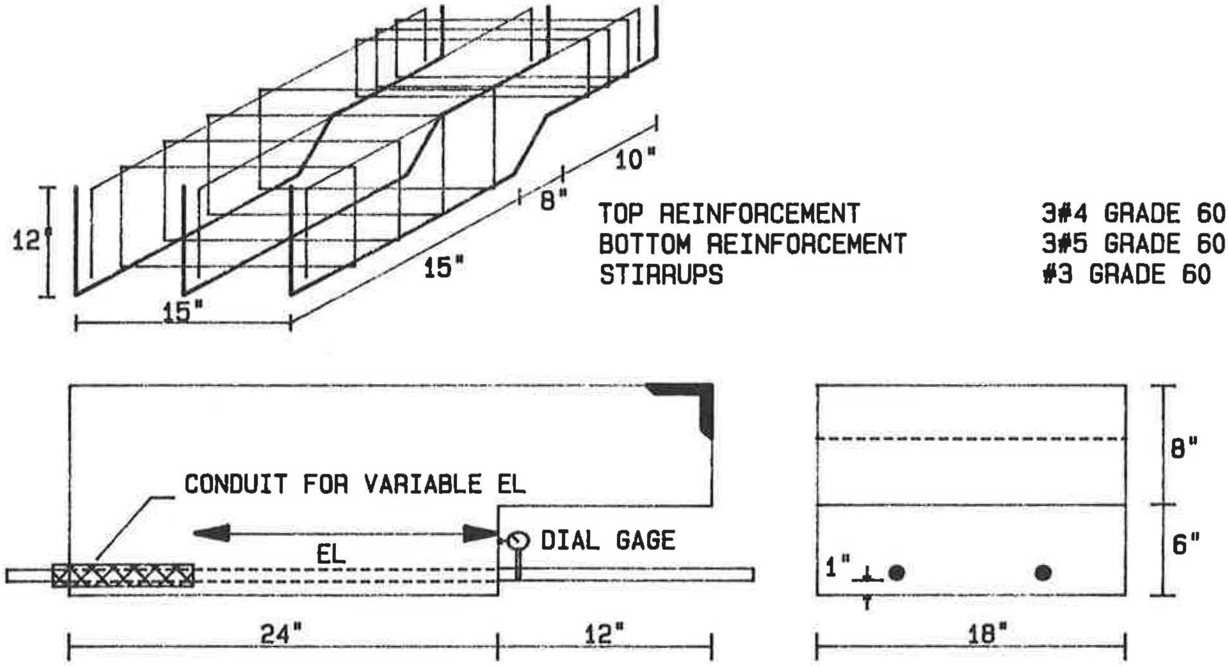

FIGURE 2 Bond Specimen Reinforcement Detail

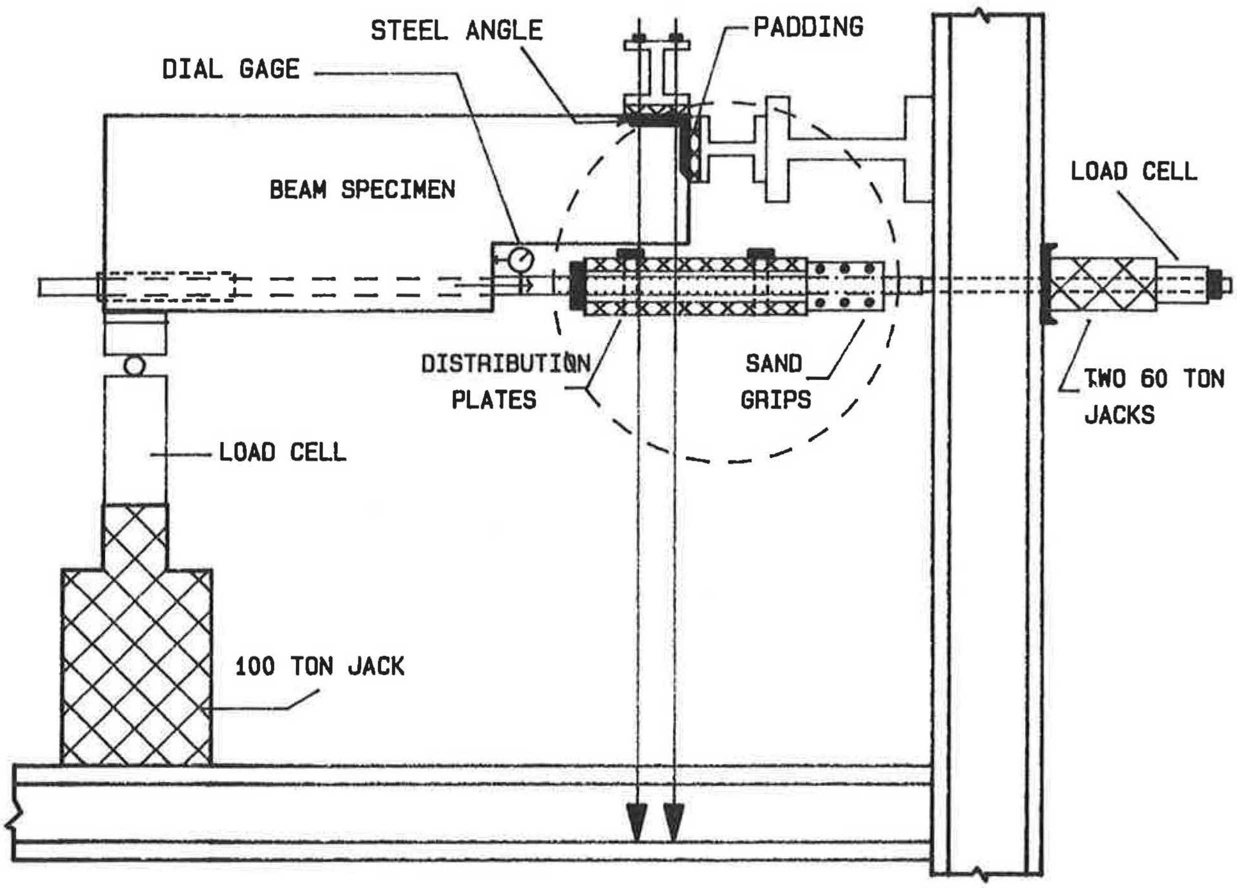

FIGURE 3 Bond Test Setup

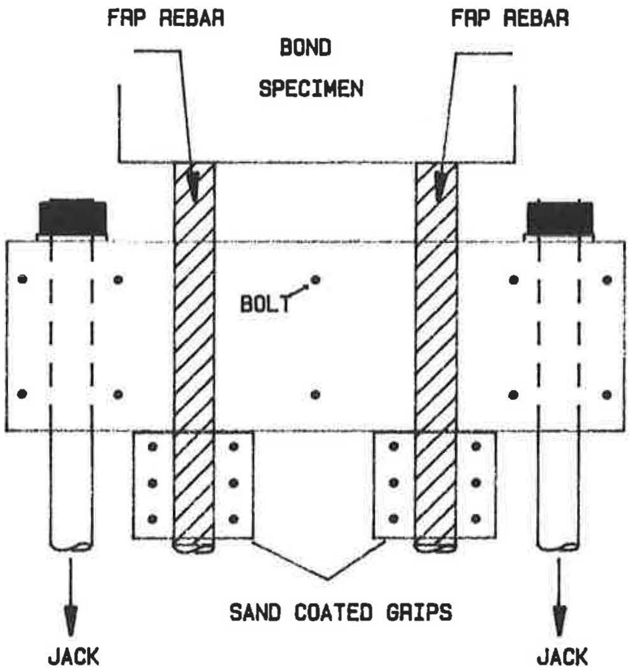

FIGURE 4 Detail of Load Distibution Plates

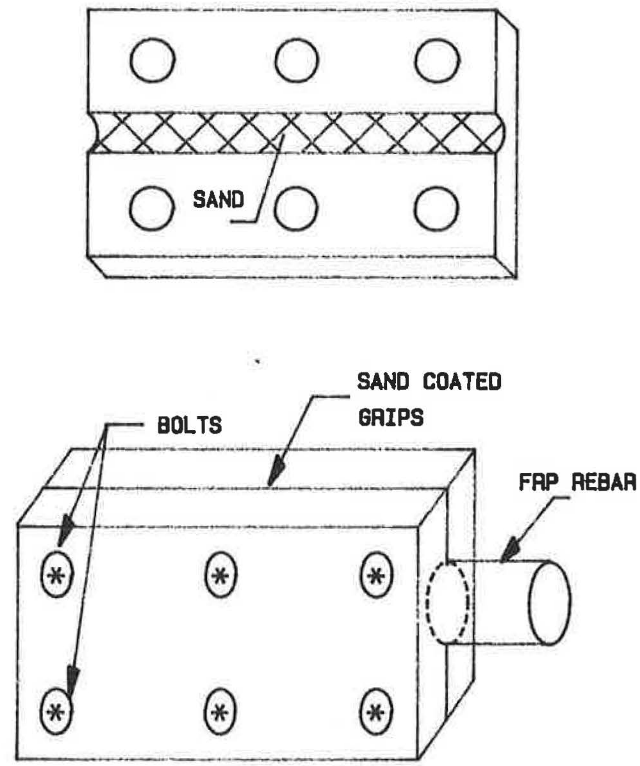

FIGURE 5 Detail of The Sand Coated Grips

EXPERIMENTAL RESULTS

Bending Tests

A series of laboratory tests on concrete beams reinforced with FRP rebars were conducted. The major emphasis of these tests was to investigate whether or not concrete beams reinforced with FRP rebars behave in a manner similar to those reinforced with mild steel rebars in terms of:

* Stress-Strain behavior.
* Load-Deflection variations.
* Load carrying capacities.
* Crack patterns (spacing, width, propagations).
* Modes of failure.

The simply supported rectangular beams of 9 ft lengths, Figure 1, were tested under pure bending, under four point bending load condition, using different configurations of FRP reinforcements. In order to take advantage of the high tensile strength of the FRP rebars, beams with high strength concrete (6500 and 7500 psi) were tested for the purpose of maximizing the bending resistance of the beams. A 90% increase in ultimate moment capacity was obtained when FRP rebars of ultimate tensile strength of 130 ksi were used in lieu of mild steel rods. Cracks in FRP reinforced concrete beams were found to be sudden and crack widths were larger than the corresponding steel reinforced beams; however this behavior has improved by using higher concrete strengths.

In order to compare the beam bending test results and to evaluate the effect of changing some parameters, the results are grouped into five categories, Group A through E. Table 1 summarize the beams reinforcement, and test results.

Group A

This group consists of two beams reinforced with 3#3 FRP rebars with #2 steel stirrups. All other parameters were kept the same for this group except for rebars, which were supplied from two different companies. A maximum difference of 10% was found in the moment capacities of the two beams in this group of which Beam #2 reached an ultimate moment capacity of 27.75 kip-ft

Group B

The behavior of the beams reinforced with 2#4 FRP rebars were investigated in terms of the type of stirrups used. The use of smooth FRP stirrups resulted in a bond failure between the smooth stirrup and concrete, thus reducing the moment capacity of the beam by 35%.

Group C

This group of beams was reinforced with either 2#7 or 2#8 FRP rebars. The use of smooth FRP rebars in Beam # 1 reduced the moment capacity of the beam by 60% compared to Beam #8, which was reinforced with ribbed rebars. By using high strength concrete (7.5 KSI) versus regular strength concrete (4.2 KSI) the moment capacity of Beam #C was increased by 50% compared to beam #4, due to better utilization of the rebar high strength.

Group D

In order to take advantage of the high tensile strength of the FRP rebars (130 ksi), the use of high strength concrete (7.5 ksi) was used in beams A & B and (6.5 ksi) was used in beams H1 & H6. The ultimate tensile strength of the FRP rebars were reached in beams H1, H6, A and B, thus increasing the moment capacity of the beams by a factor of TWO. The moment capacities of beams H1 and H6 did not reach the ultimate moment of beam A due to the lower ultimate tensile strength of the rebars, (100 vs 130 ksi), which were supplied from another company. The tensile strengths were also verified by conducting tension test on both rebars.

The use of an additional FRP reinforcement in an arch form in Beam #B has increased the moment capacity of the beam by 60% as shown in Table 1. This increase is attributed not only to the increase in FRP area but also to the shape of FRP, in resisting bending. In addition, the crack width was reduced by 50% when using the arch form.

The effects of changing concrete strength in addition to the use of the arch arrangement are best illustrated by the load versus deflection plot in Figure 6.

The experimental deflections of FRP reinforced beams (Beams 9 & 10) were about four times larger than the beam reinforced with steel (beam #11). These larger deflections were expected due to the low modulus of elasticity of the FRP rebars, which is about 7×10^6 psi.

Group E

In this group, the effect of increasing the perimeter of the reinforcement in addition to the use of sand coated rebars was investigated. The use of 5#3 rebars (0.55 in^2) versus 3#4 rebars (0.59 in^2) increased the ultimate moment capacity by 20%. The crack pattern in terms of the crack width, their propagation and distribution has vastly improved by the use of the sand coated rebars. This behavior is related to a better bond between the sand coated rebar and concrete. The crack pattern is very similar to a pattern expected of a beam reinforced with steel rods. The load versus deflection plot in Figure 7 illustrates an increase of 40% in the cracking moment when sand coated rebars are used.

Bond Behavior

Bond strength development is a complicated phenomenon which is influenced by concrete strength, embedment length, concrete cover, rebar spacing and associated shear and flexure. The bond strengths are expected to arise either from the FRP rebar anchorage to concrete or changes in bending moment along the specimen length.

In order to study the bond performance (bond force and failure pattern) of the FRP rebars, a set of experiments were conducted using the cantilever specimen developed by the Civil Engineering Department at West Virginia University, illustrated in Figures 2 and 3.

The cantilever specimen tests are more realistic in simulating bond behavior conditions, less expensive and preferred over the conventional pull-out specimen tests since they provide more realistic strain gradient through the depth of the specimen which is similar to the gradient expected in a flexural member. While the pull-out test does not represent the actual behavior in a beam, subjecting concrete to compression forces and rebars to tension forces, the cantilever specimen produces tension forces in both concrete and rebars.

As shown in Figure 2, the compression zone of the beam is designed to prevent premature crushing failure and is designed to prevent shear or moment failures during experiments. A concrete block is cut out of the specimen for modeling the portion of a beam adjacent to a diagonal crack.

For each experiment in the test program, the loading and slip of the rebar were recorded and plotted. A summary of the bond test experiments and the results are outlined in Table 2. A typical bond stress versus net slip at the loaded end is illustrated in Figures 8 and 9.

CONCLUSIONS

Based on the limited tests (22 in bending and 12 in bond) that were conducted under this program, the following conclusions may be drawn.

1) Cracks in FRP reinforced concrete beams are found to be developing suddenly and crack widths are found to be larger than the corresponding ones in steel reinforced beams.

2) The fact that the bending cracks were developed at uniform intervals is a clear indication that there was no bond failure between deformed FRP rebars and concrete.

3) The crack pattern in terms of the crack width, their propagation and distribution has vastly improved by the use of the sand coated rebars due to a better bond between the sand coated rebar and concrete which has increased the cracking moment by 40%..

4) A 50% increase in ultimate moment was obtained when deformed FRP stirrups were used in lieu of smooth FRP stirrups. Bond failure between smooth FRP rebar and concrete is observed. Therefore, use of smooth FRP rebars and stirrups are ruled out.

5) The ultimate moment capacity of high strength concrete beams ($f_c' = 7.5$ ksi) was increased by 90% when FRP rebars of ultimate tensile strength of 130 ksi were used in lieu of mild steel rods (60 ksi).

6) More bond tests are required before determining the minimum development length for the FRP rebars.

7) Current procedures for steel reinforced beams do not predict ultimate moment capacity of beams reinforced with FRP. However, modifications in the present ACI equations will be made upon the completion of the experimental program.

RECOMMENDED RESEARCH AREAS

In order to reduce the crack width in beams reinforced with FRP rebars and to confine it to the maximum width set by AASHTO specifications (10), the following options will be investigated:

TABLE 1 BENDING TEST RESULTS

GROUP #	BEAM #	FRP REINF.	STIRRUP SIZE	CONCRETE STRENGTH (ksi)	ULTIMATE MOMENT (kip-ft)	MODE OF FAILURE
A	2	3#3	#2 S	4.2	27.75	BEND/COMP/REBAR
A	3	3#3	#2 S	4.2	24.67	BEND/COMP/SHEAR
B	5	2#4	#2 S	4.2	27.75	BEND/SHEAR
B	6	2#4	#3 F	4.2	24.67	BEND/SHEAR
B	7	2#4	#2 fs	5.0	16.96	BOND IN STIRRUP
C	1	2#7SMOOTH	#2 S	4.2	46.50	BOND
C	4	2#8	#2 S	4.2	40.00	COMP/SHEAR
C	8	2#7	#2 S	5.0	41.63	COMPRESSION
C	H5	2#8	#3 F	6.5	54.75	COMP/SHEAR
C	C	2#8	#3 F	7.5	60.00	COMP/SHEAR
D	9	2#3	#2 fs	5.0	12.95	BEND/BOND
D	10	2#3	#2 S	5.0	10.64	TENSION/EX.CRACKING
D	11	2#3 STEEL	#2 S	5.0	13.88	TENSION
D	H1	2#3	#3 F	6.5	18.00	TENSION
D	H6	2#3	#3 F	6.5	16.50	TENSION
D	A	2#3	#3 F	7.5	27.75	TENSION
D	B	2#3/ARCH	#3 F	7.5	43.50	TENSION
E	H2	3#4	#3 F	6.5	31.13	COMPRESSION
E	H4	5#3	#3 F	6.5	37.50	COMP/TENSION
E	D	3#4/SAND	#3 F	7.5	40.50	SHEAR/TENSION
E	E	5#3	#3 F	7.5	40.50	SHEAR/TENSION
E	F	3#4	#3 F	7.5	34.50	SHEAR/TENSION

F : FRP STIRRUP S : STEEL STIRRUP fs : SMOOTH FRP STIRRUP

TABLE 2 BOND TEST RESULTS

BEAM #	REBAR SIZE	EMBEDMENT LENGTH (in)	ULT. LOAD @ FAILURE (kip)	EXP. BOND STRESS (psi)	REMARKS
BO.1.1	#8	16	22.46	450	BOND
BO.1.2	#8	16	24	480	BOND
BO.1.3	#8	24	29	387	BOND
BO.1.4	#8	24	30	400	BOND
BO.2.1	#3	16	*	*	FAILURE IN GRIP
BO.2.2	#3	16	*	*	FAILURE IN GRIP
BO.2.3	#3	24	11	>389	REBAR FAIL./NO SLIP
BO.2.4	#3	24	10.9	>389	REBAR FAIL./NO SLIP
BO.H1	#3	12	8.2	>580	REBAR FAIL./NO SLIP
BO.H2	#3	12	8.1	>573	REBAR FAIL./NO SLIP
BO.H3	#3	8	9.4	>997	REBAR FAIL./NO SLIP
BO.H4	#3	8	8	>849	REBAR FAIL./NO SLIP

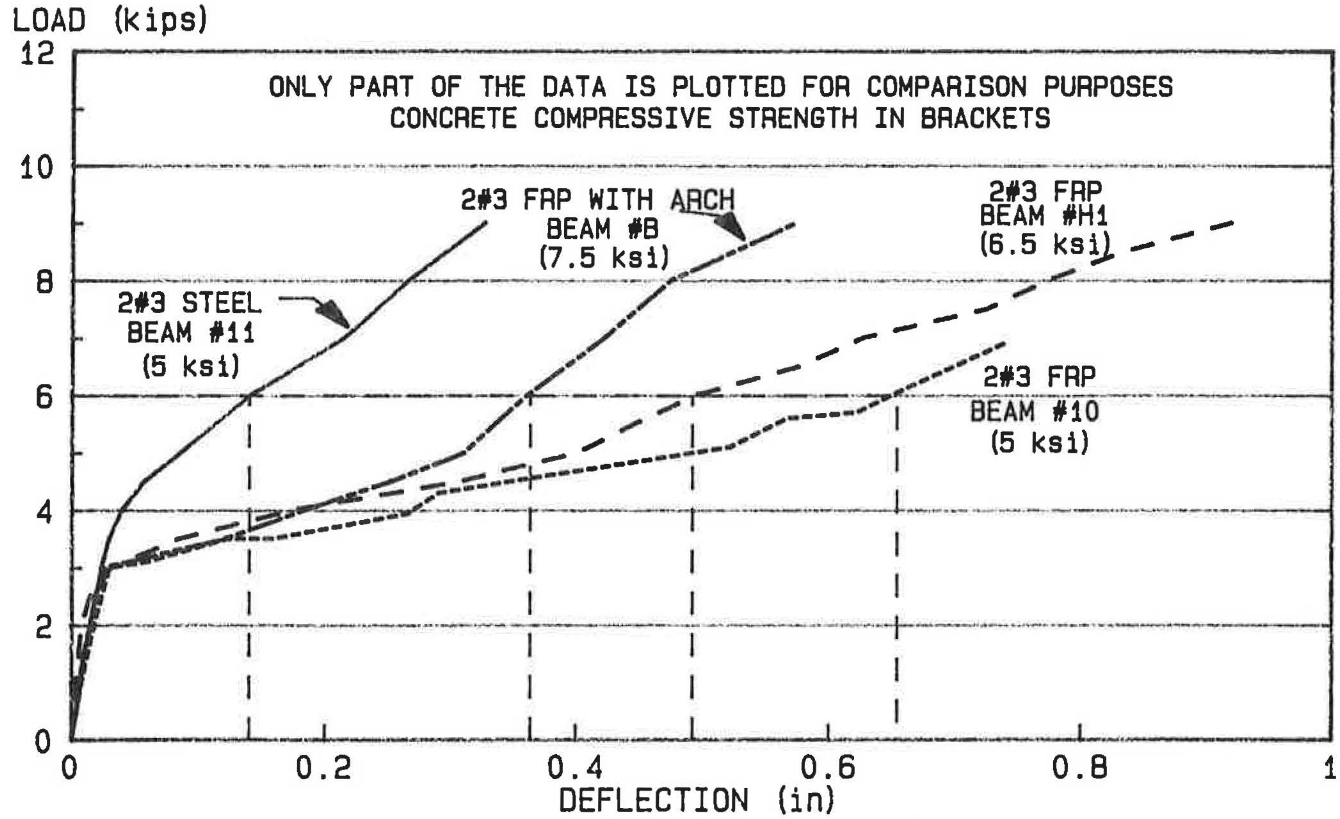

FIGURE 6 Load vs Deflection (GROUP D)

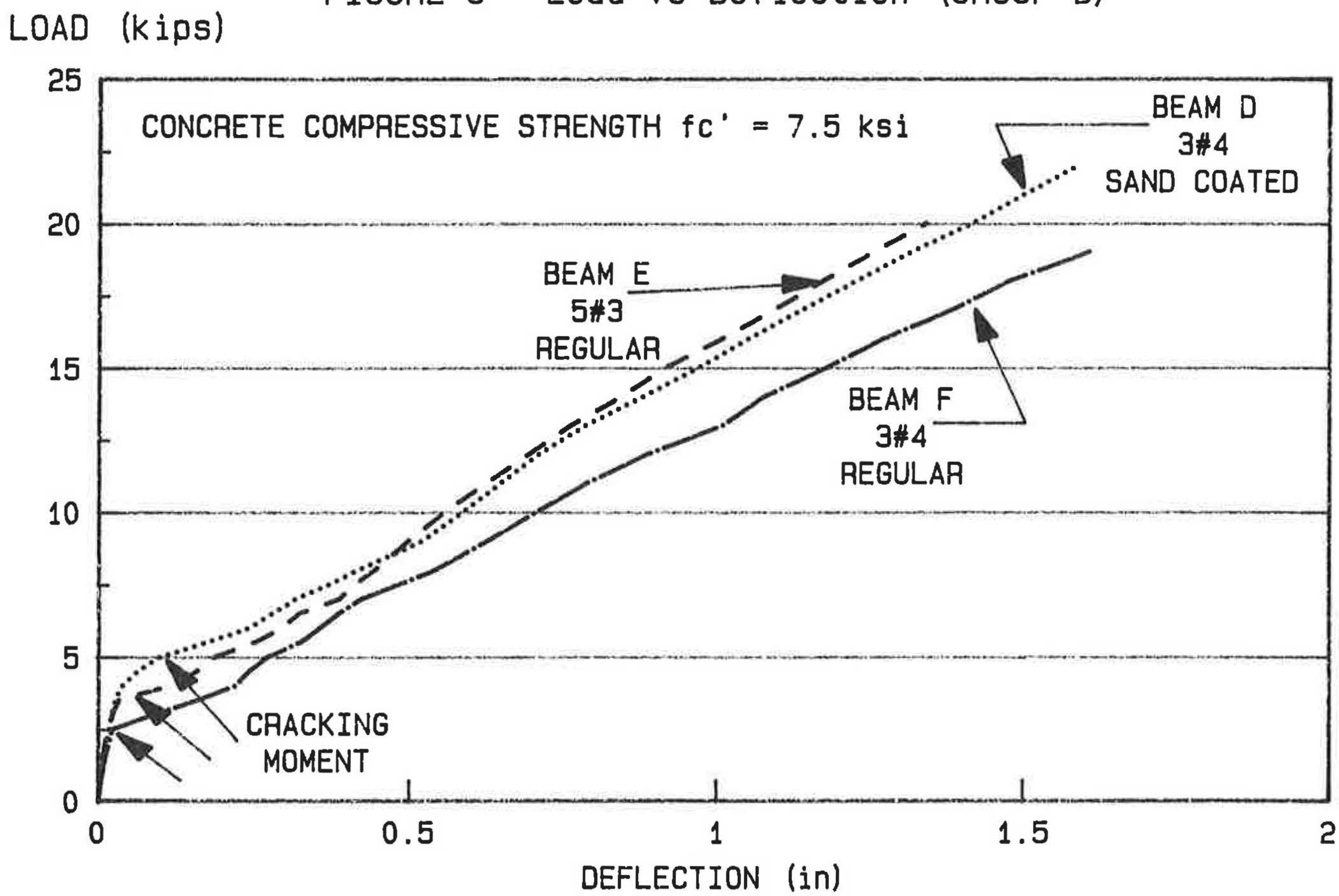

FIGURE 7 Load vs Deflection (GROUP E)

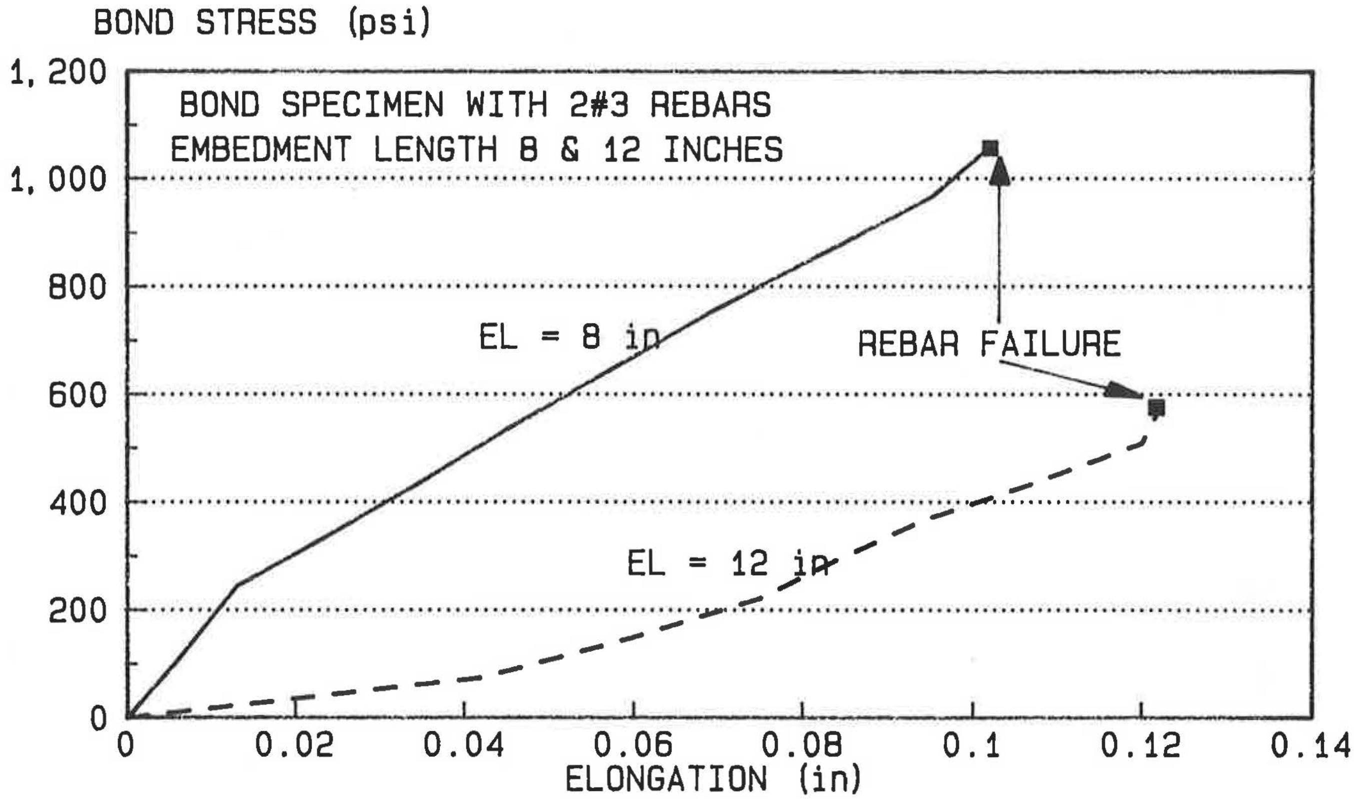

FIGURE 8 Bond Stress vs Elongation

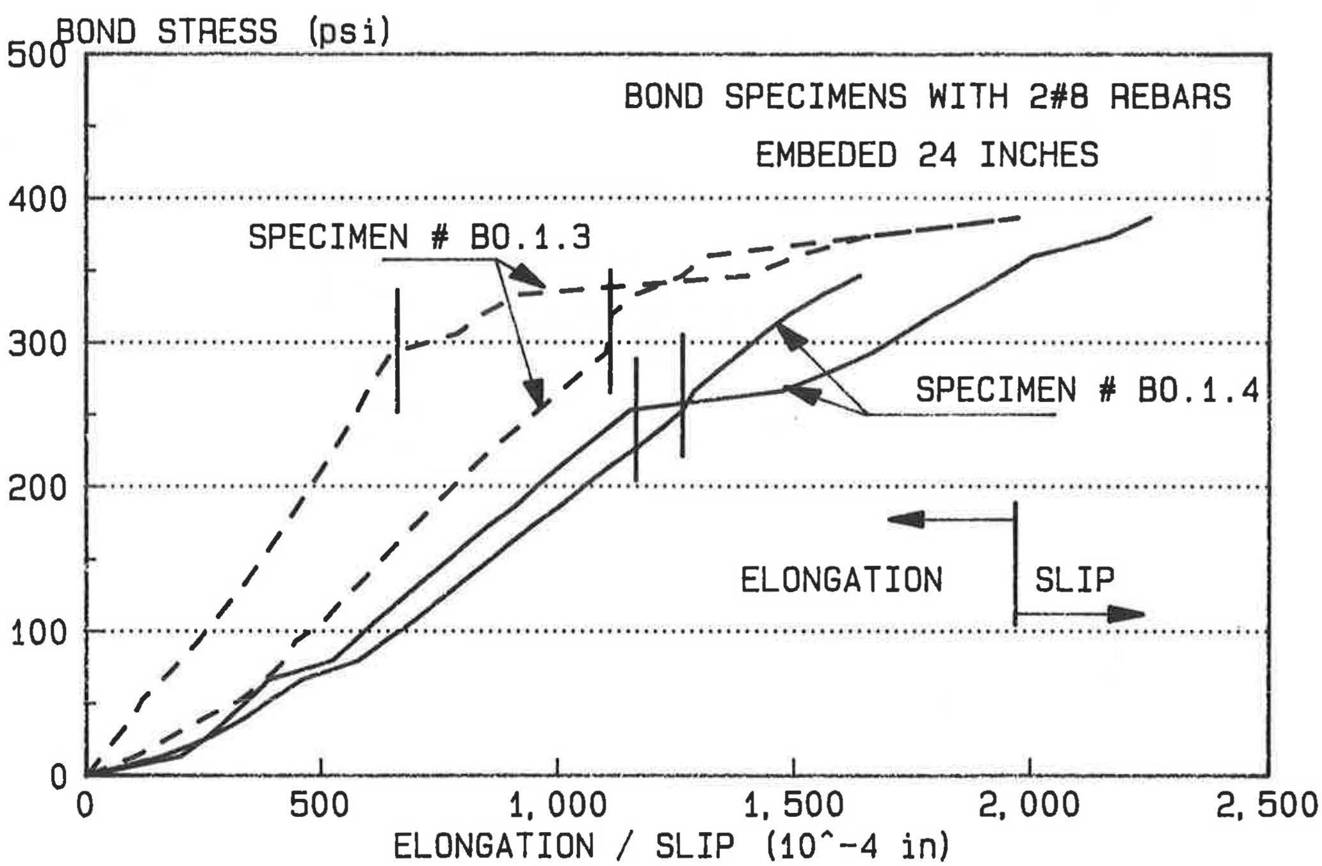

FIGURE 9 Bond Stress vs Net Slip

a) Reduce the elongation mismatch between the FRP rebar and concrete by either increasing the ductility of concrete or increasing the stiffness of the rebar by using different kind of fiber, i.e. kevlar, or byincreasing the strength of concrete, i.e. 8 ksi inlieu of 4 ksi.
b) Increase the stiffness of the beams by using 2 to 3 times the required cross-sectional area of the FRP rebar needed.
c) Increase the perimeter of the reinforcement.
d) Introduce a new detailing idea in the reinforcing of the beams.

Acknowledgement

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AASHTO Bridge Design System: A Status Report

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The Bridge Design System (AASHTO-BDS) is a proprietary computer software product under development by the American Association of State Highway and Transportation Officials (AASHTO). AASHTO-BDS is a product of the joint development process by which AASHTO member departments voluntarily pool their resources to produce mutually acceptable computer software products, and to propose specifications for unique transportation-oriented computer equipment.

Over twenty state highway departments, a Canadian province, and the Federal Highway Administration are contributing to the development of computer software which can be utilized by all parties for the design of highway structures. BDS, like other joint development products, is available for licensing to public and private agencies through AASHTO.

BDS software planning and development began in 1986 with selection of an engineering software contractor, BAKER/IAI, a joint venture of Michael Baker, Jr., Inc., and Imbsen & Associates, Inc. The development is monitored by a BDS task force comprised of representatives from five state bridge agencies and assisted by an AASHTO liaison officer. Release 2.6 of the BDS product is currently in use by more than twenty-five AASHTO members for production purposes.

AASHTO BDS is a bridge design system under development by the American Association of State Highway and Transportation Officials (AASHTO). This system is being implemented into production use by more than twenty-five AASHTO members since July, 1989 (AASHTO BDS Release 2.4). The complete bridge design process has been carefully considered in developing the BDS program architecture. The traditional practice of analysis and design on a component basis, the considerable body of software in use by design organizations, as well as the regional variation in type of bridge construction, has been factored into the system design.

These considerations resulted in a bridge design system which uses a database, database management system and a standards processor to allow iterative component-by-component design. When complete, BDS will allow an engineer to analyze, design and rate bridges within a single integrated software system.

In developing BDS, two general categories or user options have been established: A bridge model generation option for the routine bridge design and a general structure option for the finite element analysis of complex structures.

The bridge model generation option includes model generation for the analysis and subsequent design check of steel, reinforced concrete and prestressed concrete girder bridges. Bridges may be single span or multiple span with simply-supported or continuous spans. Structural framing characteristics include intermediate hinges, monolithic connections between the superstructure and the substructure, skewed supports at the abutments and bents, and sloped leg supports. Boundary conditions for the bridge model generation may be fixed, pinned, or roller. Bridge cross sections include slabs, T-beams and I-girders. I-girders may be either steel (built-up or rolled) or prestressed with noncomposite and composite concrete or steel decks.

Design checks are made for compliance with the AASHTO's *Standard Specifications for Highway Bridges, 13th Edition* for either the Service Load Design Method or the Strength Design Method. Future enhancements will include expanded design-check capabilities for rating and automated design. Force envelopes for the AASHTO design truck (with variable rear-axle spacing) and corresponding lane loadings are automatically generated. Additionally, special permit vehicles or alternative design vehicles may be stored in the vehicle library and easily referenced to produce design-force envelopes. The bridge model generation option available in Release 2.6 (June, 1990) includes only superstructure design. A high-priority enhancement to include substructure analysis and design is scheduled for the next release.

The general structure option was developed to give the designer access to the sophisticated analysis capabilities of the finite element method often needed for complex structures and unusual loading conditions. To date, bridge designers have relied on general-purpose structural analysis programs for these capabilities. Release 2.6 of BDS includes the finite element capabilities needed to conduct plane-truss, plane-frame, plane-grid and space-frame analyses. Node point and element generation capabilities are included to simplify the task of modeling large and complex structures. The node point and element generation capabilities are also useful in modeling a bridge for a seismic analysis in accordance with procedure 1 of the AASHTO "Guide Specifications for Seismic Design of Highway Bridges, 1983." BDS loading capabilities are tailored for bridge design needs to enable the designer to produce force envelopes for live loads, gravity loadings for members that are arbitrarily oriented in space (e.g., truss members, diagonal bracing), and other loads which are applied as permanent loads or temporary loads. The BDS general structure option may be used to simulate construction stage loadings by activating previously-stored members and supports. Boundary spring elements included in the general structure option allow the user to model unusual support conditions. Future releases of BDS will include special finite elements used in modeling bridge components and connections. Additionally, these capabilities may be expanded to include dynamic analysis capabilities needed for seismic analysis.

The *BDS POL Reference Manual* and its two companion documents, *BDS Users Guide* and *BDS Example Problem Manual*, describe the current capabilities and use of BDS. The *Users Guide* is intended to introduce the user to BDS. The *POL Reference Manual* describes in detail the use of the system, the language syntax and its semantics. The *BDS Example Problem Manual* presents selected problem solutions to illustrate how BDS is used for particular applications.

To date, the development of BDS basic architecture and database management capabilities have pre-empted the development of design features and capabilities. With the basic system developed, future enhancements will easily expand BDS into a comprehensive bridge design system.

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CURRENT STATUS OF AASHTO-BDS

The initial release of AASHTO-BDS (release 2.4) was delivered to participating states in July 1989 for implementation in production environment. A number of additional states have also licensed the product and taken delivery of the system following the initial release. Current BDS user states include Arkansas, California, Colorado, Illinois, Iowa, Kansas, Kentucky, Maine, Maryland, Missouri, Nebraska, Nevada, New Mexico, New York, North Carolina, Ohio, Oklahoma, Oregon, Pennsylvania, South Dakota, Tennessee, Texas, Virginia, Washington, West Virginia, Wisconsin and Wyoming, Federal Highway Administration and Province of Manitoba in Canada. AASHTO-BDS is now fully operational on VAX/VMS, IBM-MVS/TSO, and IBM-VM/CMS, and Intergraph Clipper platforms. The Clipper work station version of BDS was ported to for the State of Nevada and was subsequently accepted by AASHTO for maintenance and licensing.

The current release of BDS (release 2.6) was delivered in July, 1990. This release includes some of the enhancements which were suggested and prioritized by the users. Major releases of AASHTO-BDS to date are shown in the following table:

Release	Versions	Purpose
1.0	VAX, IBM	Interim Release
2.0	VAX, IBM	Beta Test
2.4	VAX, IBM	General Implementation
2.6	VAX, IBM, Intergraph	Enhanced Superstructure

The distribution package includes a tape containing the software, an *Installation Guide and System Managers Document, Users Guide, POL Reference Manual, Example Problem Manual* and *System Design Documentation and Programmers Guide.*

The AASHTO-BDS system is available for licensing on VAX and IBM equipment through AASHTO. workstation versions for Vaxstations and Integraph Clipper workstations are also available. The contractors will be available for continued support and maintenance of all of the released versions of the system.

Major enhancements are currently underway which will enhance the capabilities of the program in superstructure rating, user interfaces and .substructure design.

BACKGROUND OF AASHTO-BDS

The primary objective in developing BDS as a comprehensive bridge design system was to automate the total design by coupling the design of individual components with a database and an interactive process control system. Traditionally, bridges have been designed on a component-by-component basis using computer programs to complete each task. This approach has resulted in the development of a large number of programs which are not readily shared by the AASHTO member departments or by the private sector providing consulting services to these member departments. The AASHTO Joint Development Process was organized to provide a means whereby AASHTO member departments can pool their resources on a voluntary basis to produce a system such as BDS.

A secondary objective was to utilize, to the greatest extent possible, existing software in order to minimize the development costs and to preserve the confidence acquired in using home-grown software developed within the member departments. This second objective was somewhat difficult to achieve, in some cases, because the data handling by the database management system required substantial modifications to the existing software. Additionally, BDS was to be developed using stringent programming standards for ease of maintenance and future expansions. Software which potentially could be incorporated into the system was grouped into four categories, as shown in the following table. The categories were determined on the basis of such factors as internal documentation, programming style, functionality and completeness.

Level of Use	Modification	Examples
Entire Programs	None-Minor	POLO, SICAD
Group of Subroutines	Minor	OMBAS
Selected Subroutines	Minor-Moderate	SEISAB, STDS
Selected Algorithms	Rewritten	BRASS, SIMON

The level to which existing software was used varies from the incorporation of entire programs to the selection of computation algorithms which were modified for use in the environment of a database management system. As shown in the table, some programs were used with little or no modification while others were rewritten to achieve the same results.

AASHTO-BDS SYSTEM ARCHITECTURE

BDS is a modular system, designed for expandability and ease of maintenance. The overall architecture of the system is shown in Figure 1. Each of the elements shown in the diagram is, with the exception of the central database, a subsystem or module with prescribed tasks as explained briefly below.

Database

Database files may be viewed as the core of BDS as depicted in Figure 1. This is the control location where all bridge description data, run time data, and library information are stored. The BDS system contains some permanent database files and some temporary work files. The permanent database files include a DBMS control file, used by POLO-II; a standards file, storing a compiled form of the standard specifications; a library file, storing standard cross sections, vehicles, rebar information, etc.; a message file, storing system information, warning, and error messages; and a multi-bridge database, storing bridge descriptions for an unlimited number of bridges. The database work files are used for communication between modules. The pertinent data is obtained from these files via the DBMS and made available for each subsystem or module.

Database Management System - POLO II

POLO is the name of a software package developed at the University of Illinois under the direction of Dr. L. A. Lopez. The POLO DBMS constitutes a set of specialized modules designed to allow the programmer to construct complex hierarchical data structures that would appear to be in memory even though the total data space is many times the size of available primary memory.

Process Control System

This subsystem controls the flow of the program from subsystem to subsystem. The Process Control System (PCS) is responsible

for overall management of the system. PCS can be viewed as the connection between DBMS and various subsystems as depicted in Figure 1. The functions of PCS can be listed as follows:

- Invoke execution of a portion of the program. This portion may consist of a set of subsystems, single subsystems, or portions of subsystems.

- Allow communication between subsystems through database and/or memory.

- Provide user flexibility and control in processing parts of the system. The user is allowed to process geometry alone, or change the list of output reports requested and receive a new output, or redirect the output file or device, etc.

- Promote modularity. It is important for the PCS to work in a way to encourage programmers to write modular code. This is achieved by an easy interface for PCS which allows a list of requests to be specified easily and efficiently.

User Interface

The user interface for the current release of BDS is a problem-oriented language (POL). The POL may be input through a file, it may be created via an integrated text editor (TXEDIT), or entered directly. In either case, the PCS will interpret the incoming POL command and activate appropriate modules or subsystems which may receive input data (POL) or perform calculations. When POL commands are read from a file, or created by the BDS text editor, they will be available for modifications or corrections during that session. The POL consists of control commands, which invoke various processes in BDS, and input data blocks which receive the bridge description and save it in the database.

Commands given for the input data description are organized into blocks such as BASIC DATA, BRIDGE COMPONENTS, etc. Each block may be repeated as many times as desired and there is no required order. Comments can be included in the command input lines for additional description and are separated by a semicolon (;)

START ABUTMENT 1;	Abutment and bent numbers are
OVER BENT 1;	arbitrary and should be
END ABUTMENT 2;	described separately.

Once a syntax error is detected by the system, the appropriate action is taken depending on the mode of input and process. In batch mode, the execution is halted. If the mode of input is interactive and the data is entered from a file or created by TXEDIT, the control is given to the BDS text editor at the erroneous line and the user is requested to correct the syntax. If the input is given directly, a message indicating the error is given and the user will have the chance to correct the input.

A utility program (MAKE_POL) is added to the VAX version of the system in Release 2.6. This utility allows BDS users to specify the bridge properties for common bridge configurations without having to learn the POL. The MAKE_POL utility will then use this information and prepare the POL input file for BDS.

Geometrics

This subsystem, shown in the top right corner of Figure 1, processes the input data related to the geometrics; i.e., reference systems, alignments, cross-section properties, and cross-section capacities. Many different local coordinate systems may be defined for node points and support directions. The input data are processed and relative angles between reference systems are calculated. During alignment processing, nodal coordinates are obtained from alignment stations and offsets. Cross-section processing includes calculation of area and inertia values for standard sections or obtaining data from BDS libraries. Section capacities are calculated to be used for specification checks. The geometrics subsystem also calculates the stress coefficients for various cross sections. These coefficients are applied to forces and moments to obtain stresses during solution. Figure 2 shows some cross sections processed by BDS.

Load Builder

The load builder subsystem is responsible for calculating element stiffness matrices and preparing load vectors for self weights, additional loads, moving loads, influence lines, and prestress loads. When the structure is constructed in stages, this subsystem calculates the equivalent loads for all leaving or entering elements and supports. This subsystem also calculates the losses in prestress forces, digitizes the cable forces, calculates the element loads due to prestressing, and builds the load vectors. In summary, this subsystem performs the finite element idealization of the bridge and prepares a mathematical representation of it for the solution subsystem.

Analysis, Modeler, Solver

This subsystem is normally invoked after the Load Builder, as shown in Figure 1. The three major tasks performed within this subsystem include analysis, model generation, and solution of equations. In the following, first the modeler, then the solver and analysis are described briefly.

The model generation capabilities of BDS allows the users to describe the bridge using bridge terminology without using nodes and elements directly. A bridge may be modeled as continuous beam, a plane frame, or a space frame. The three levels of model generation provide an option for the user to build a model which is appropriate for a given bridge. These models are referred to as Model 1, Model 2, and Model 3 in BDS. Figure 3 shows schematics of the three models available in BDS.

Model 1 is a continuous beam model with optional intermediate hinges. To describe this type of model, location of abutments and bents must be specified, the girder must be described, and located, and the support conditions must be specified by describing bearings and locating them at abutments and bents.

Model 2 is a plane frame model which is suitable for analyzing straight bridges with integral piers. The piers may be vertical or sloped. For Model 2 description, column and pier descriptions are needed in addition to Model 1 input.

Model 3 is a space-frame model which would be suitable for bridges on other than straight alignment or bridges with skewed abutments or bents. For Model 3 description, alignment description is needed in addition to Model 2 input.

The analysis package prepares the global system stiffness and load matrices. The solver then performs forward reduction and back substitution; i.e., solution of the equations. The remainder of the analysis package will calculate the forces, moments, stresses, and reactions due to various load cases, such as self weight, influence line, prestress, etc.

Loading Response Processor

This subsystem is responsible for organizing the results obtained by solver, loading the influence lines using the truck

and lane loadings specified earlier, and calculating the load combinations.

The dead load results applied in each stage are accumulated and prepared for output. The self weight and prestress results are also obtained at specific points specified by the user. The user may request results at specific points in the girder by the distance into the girder or by percentage of span length. It is also possible to ask for results at specific intervals such as 5th points or 10th points.

The response processor will also apply the truck and lane loads to the influence lines generated by the Solution Module, and obtain the maximum results at the specified points. The impact factors are calculated based on the span length and applied to the results along with the input load distribution factors.

The response processor also calculates load combinations according to the AASHTO Table 3.22.1A. The factors in the table may be modified by the system managers at the installation time, but the individual users are not allowed to make any modifications.

The system was enhanced for release 2.6 to calculate and report the effect of sidewalk live loads, as well.

Standards Processing and Specification Check

The standards processor/specification checker subsystem consists of two parts. One is the standards processor (SICAD), and the other is the AASHTO specifications checker.

The SICAD is a standards processing system developed under the guidance of Professor Lopez at the University of Illinois for the National Bureau of Standards. Since this system is fully compatible with POLO, it was incorporated into BDS without much difficulty. The SICAD system compiles the standards and stores them in database files. The run time portion of SICAD performs the retrieval of standards and makes them available for specification check.

The specification checker portion of this subsystem includes the mapping of the results from the database to SICAD nodes and performs the actual check. The current release of BDS includes the AASHTO specifications for primary superstructure members. Steel, reinforced concrete, and prestressed concrete are available. The girders may be composite or non-composite, and both Load Factor Design and Working Stress Methods are available for either flexure or shear.

This subsystem was enhanced for release 2.6 to allow specification check on stiffeners, shear connectors and fatigue criteria. Also, a design option is added which calculates the required stiffener spacing and shear connector spacing.

The modular design of BDS and the structure of the specification checker are such that specification modifications, changes, or updates can be easily incorporated (such as those anticipated with the completion of NCHRP Project 12-33, "Development of a Comprehensive Bridge Specification and Commentary").

PLANNED ENHANCEMENTS

Three major enhancements are planned for near future, namely: rating, user interface, and substructure modeling and design.

The rating module will include calculation of the deteriorated section properties and calculation of operating and inventory ratings. The AASHTO Maintenance Manual and the new guide specifications both will be available, and the user can select either the working stress or the load factor method. It is anticipated that these enhancements will be available within the two releases following the release 2.6.

The new user interface will be based on menus and will allow interactive invokation and use of the system. This user interface will concentrate on the frontend, i.e. the data entry and processing tasks.

The substructure module will bring a number of new capabilities to the system. Some of these capabilities include Seismic analysis using response spectrum, bent model generation, transverse loading of a space frame model, substructure response calculation, bent cap design check, and column design check. As part of the substructure loading the truck and lane loads will be moved longitudinally and transversely to determine the maximum effect in the bent cap and columns.

Other enhancements include further upgrade of the user interface to a "point and shoot" windowing system; incorporation of on-line help; knowledge/rule based "expert" system to aid novice designers; more control of output options and format; and schematic graphics. The following schedule lists the tentative sequence in which the enhancements will be added to BDS:

- Superstructure — Haunced Element
- Substructure — Superstructure Load Transfer
- Substructure — Substructure Load Descriptions
- Substructure — Load Combinations and Envelopes
- Substructure — Seismic Analysis
- Substructure — Bearings
- Substructure — Bent Design
- Substructure — Foundations
- Substructure — Pilings
- Translators — BRASS
- Translators — AASHTO BARS
- Superstructure — Longitudinal Stiffener
- Superstructure — Slab Analysis
- Superstructure — Iterative Design
- Superstructure — Secondary Members
- Superstructure — Multiple Girder Lines
- Superstructure — Camber, Blocking Diagrams, etc.
- Superstructure — Grid Analysis
- Superstructure — Bid Quantities
- Superstructure — Schematic Graphics of Input
- Graphics — Moment Diagrams, Envelopes, etc.
- Graphics — Plan Sheet Generation
- Graphics — Three-dimensional Renderings
- Attaching Software to BDS
- User Friendliness — On-line Help
- User Friendliness — "Point and Shoot" Graphical Input
- User Friendliness — "Expert System"
- User Friendliness — Output Control

TESTING AND VERIFICATION

One of the important tasks in developing a comprehensive and versatile software such as BDS is the verification and testing. BDS is capable of analysis and design check of reinforced concrete, prestressed concrete and steel bridges, and many different configurations and loadings can be considered. In order to test and verify all aspects of the program an organized and rigorous testing procedure was developed and the product is tested according to this procedure before every release. This procedure is as follows.

During the first phase of testing and verification, i.e., the alpha testing, a number of bridges of various types are used

and all major parts of the system are verified. In a subsequent beta testing period the system is installed and tested by a minimum of four highway departments of transportation. The problems which are identified during the alpha and beta testing are resolved to prepare the product for distribution. It must be noted that a system as large as BDS cannot be fully tested in a few months and minor problems may be found by the users. However, the contractors are available to receive problem reports and resolve them in a timely fashion.

SUPPORT AND MAINTENANCE

A team of qualified engineers, made of design engineers and software developers, is available for user support. This team has been helping the users with the installation, customization and use of the system since it was released. Some bugs have also been discovered and fixed since the initial delivery of the system. However, none of the bugs was serious enough to warrant an interim release of the system.

IMPLEMENTATION AND TRAINING

BDS release 2.6 has been distributed to the licensees. This release of BDS is available for VAX/VMS, IBM-VM/CMS, IBM-MVS/TSO, and Intergraph Clipper systems. User documentation to accompany the system includes Users Guide, POL Reference Manual, Example Problem Manual, and System Managers Guide. A Programmers Guide and System Design and Documentation Manual is also provided so that the participating states can develop their own subsystem or modify the current subsystems to fit their needs. It is anticipated that the workshops, documentation, and user support will provide enough information for BDS users to take advantage of many unique features of AASHTO-BDS system and provide enthusiasm and guidance for its continued maintenance and enhancement.

SUMMARY

The Bridge Design System (AASHTO-BDS) is a proprietary computer software product developed by AASHTO using the Joint Development Process. Over twenty state highway departments, a Canadian province and the Federal Highway Administration have contributed toward the development of computer software which can be utilized by all parties for the design of highway structures.

AASHTO-BDS is designed to be a comprehensive bridge design system. Special features of this system include a Database Management System, an interactive process control system, and modular design. Strong geometrics and analysis subsystems are included in this software as well as model generation, loading response processing and specification check capabilities. The input to BDS is in problem-oriented language which can be entered either interactively or from a file. The output reporting is very versatile in that reports are user selectable and any set of units or mixed units may be used for various reports.

BDS Product Release 2.6 has been distributed and is available on VAX/VMS, IBM/CMS, IBM/TSO, and Intergraph Clipper computers. BDS documentation includes a Users Guide, a POL Reference Manual, an Example Problems Manual, a System Managers Guide, and a System Design Documentation and Programmers Guide.

Enhancements planned for the near future include: substructure analysis and design check, rating, menu-driven user interface, and graphics. It is anticipated that with the support, maintenance, and enhancements planned for the system, coupled with the enthusiasm and feedback from the users, BDS would be one of the most useful tools for bridge design in the nation.

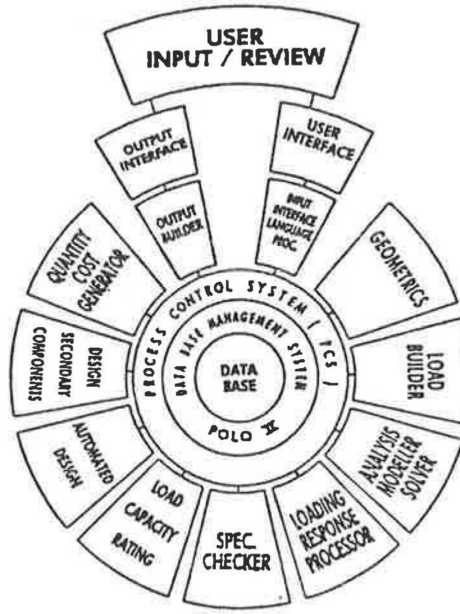

FIGURE 1: Overview of AASHTO-BDS System Architecture

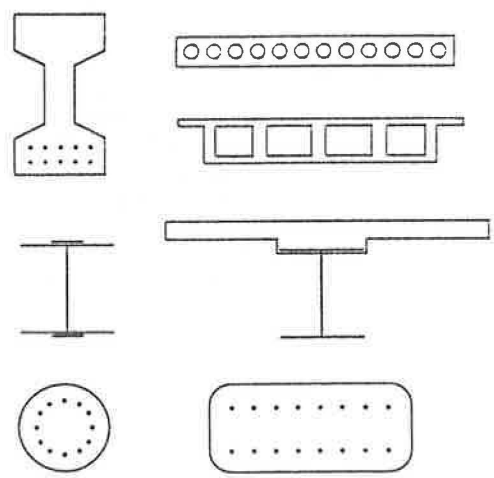

FIGURE 2: Typical Cross Sections

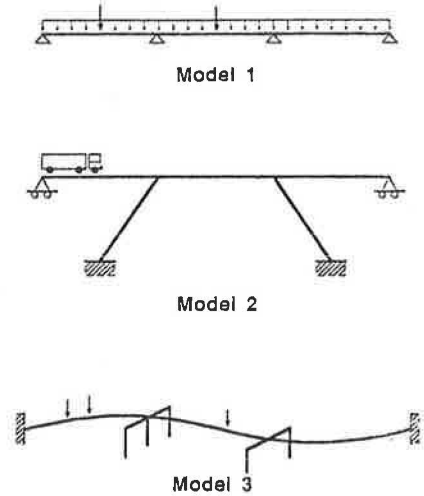

FIGURE 3: Model Generation in BDS

Design of Laminated Elastomeric Bridge Bearings

CHARLES W. ROEDER AND JOHN F. STANTON

Elastomeric bearings have been used for many years. They are economical and require minimual maintenance, however, the historic AASHTO design provisions have limited the range of applications of these bearings. Recent research has resulted in recommendations which can increase the versatility of elastomeric bearings, but the recommendations require increased design calculations. A simple method of satisfying these more complex design equations is presented. The method uses a standard spread sheet computer program, and it automates and simplifies the design equations. These proposed design equations increase the load, movement and rotational capacity of many elastomeric bearings over those permitted with existing AASHTO design provisions, but the recommendations require greater care in design and quality control in manufacture of the bearings.

INTRODUCTION

Elastomeric bridge bearings have been used and have provided economical and maintenance free service for many years. Provisions for them were first included in the 1961 American Association of State Highway Officials (AASHO) Bridge Design Specifications (1), based on experimental research performed on unreinforced elastomeric pads (2). This first specification limited the average compressive stress on the bearings to 800 psi and the average compressive strain to 15%. The maximum thickness of the pad could be no more than 20% of its smallest plan dimension to assure stability, and the minimum thickness was controlled by the expected movement of the bridge.

This specification was simple, and it was used with only minor changes for nearly 25 years. Most bearings design to it performed well, but over time some problems and limitations came to light. First, the specification was unduly restrictive, because the stress and strain limits combined with the geometric constraints meant that elastomeric bearings could only be used for relatively small bridges with light loads and short movements. In fact, the original specification restricted elastomeric bearings to bridges shorter than 80 ft. Second, quality control problems occurred in some states, and a few bridge engineers became disenchanted with elastomeric bearings. These problems were usually caused by shoddy manufacturing, but the early specification contained only material tests, and required no load tests on finished bearings with which to separate well and badly made bearings. Finally, bearing use changed over the years, partly through the influence of practice in other countries, and reinforced elastomeric bearings started to be used more widely in place of the unreinforced pads for which the early specification was developed. However the specification did not permit the designer to take advantage of the superior properties of reinforced bearings.

The NCHRP 10-20 Research Project was initiated to overcome these difficulties. In 1985, a new specification was approved and now forms part of the 1989 AASHTO Specification (3) as a result of a state of the art study (4,5) performed as part of the 10-20 Project. Since then, considerable research (6,7) has been completed within the NCHRP 10-20 project, and new design recommendations (7) have been developed.

These latest design recommendations (7) have not yet been adopted by AASHTO, but they are based on extensive experimental and theoretical studies (4-11) of the behavior and modes of failure of elastomeric bearings. They differentiate between plain pads and reinforced bearings, and allow engineers to choose between two levels of design. The first restricts the load to relatively low levels but requires only modest design effort and quality control, while the second takes advantage of the better performance obtainable from reinforced elastomeric bearings at the expense of more complicated design procedures and more stringent quality control.

The purpose of this paper is to outline some of these advantages, and to show that highest level design procedures may be executed quickly and easily using a spreadsheet. The paper provides a brief overview of the behavior of and recommended provisions for steel laminated elastomeric bearings. It does not deal with plain pads, fiberglass or fabric reinforced pads, since the stresses allowed on them have not changed significantly. The benefits of the changes are illustrated, and it is shown that steel laminated elastomeric bearings can support larger loads and accommodate larger movements and rotations than were possible under earlier specifications (1,3), often allowing good quality elastomeric bearings to be used in place of other more troublesome and expensive bearing systems.

FUNDAMENTALS OF BEARING BEHAVIOR

Steel reinforced elastomeric bearings are built as illustrated in Fig. 1. Alternate layers of elastomer (typically 1/4 to 3/4" or 6 to 18 mm thick) and steel (typically 1/16" to 3/16" or 1.5 to 4.5 mm thick) are hot bonded together during vulcanization. The elastomer may either be natural rubber or polychloroprene (Neoprene). Cover layers of elastomer are placed around the

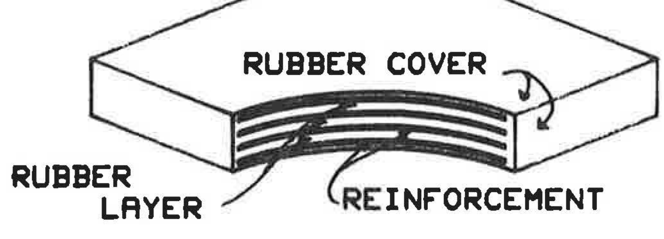

FIGURE 1 Typical steel reinforced elastomeric bearing

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edges of the reinforcement to protect it from corrosion, and they may also be placed above and below the top and bottom reinforcement layers as shown in the figure. Top and bottom cover layers are are typically about one half the thickness of the internal rubber layers, but are sometimes left off to simplify attachment of the top and bottom steel plates to the structure.

The geometry of the complete bearing and the individual layers dominates the performance of the bearing. The shear strain at the edge of the steel-rubber interface proves to be the critical response quantity in an elastomeric bearing layer, and Figure 2a shows the shear strain caused by compressive load. The reinforcement restrains the lateral displacement of the elastomer and so stiffens the bearing against compressive load, but the resulting bulge pattern causes the shear strains illustrated in the figure. The shape factor, S, is the characteristic used to describe the influence of the layer geometry on the compressive stiffness and the induced shear strains. For a single rubber layer it is defined as the plan area divided by the area free to bulge.

Bridge bearings must accommodate rotations and lateral movements due to dead, live and seismic load, creep and shrinkage and thermal effects. They do so by deformation of the elastomer as shown in Figs. 2b and 2c. The lateral stiffness of a bearing is dominated by shear effects and so is controlled by the plan area and total rubber thickness, and it is not sensitive to the individual layer thicknesses or shape factor. It is given approximately by $K_t = GA/h_{rt}$, where A is the plan area, G is the shear modulus of the elastomer and h_{rt} is the total rubber thickness. The rotational stiffness is also strongly dependent upon the geometry of the bearing (and in particular, the shape factor) and the stiffness of the elastomer. Both lateral movement and rotation cause shear strains in the elastomer as illustrated in Figs. 2b and 2c.

PROPOSED DESIGN PROVISIONS

Bearing design consists largely of selecting appropriate material properties and bearing geometry.

The most important material property for design is the shear modulus, G, which can vary significantly with temperature and to a lesser extent with test method and rate of loading. However durometer hardness has historically been used because it is so easily measured. Hardness is approximately related to shear stiffness, but the relationship between them is not always reliable, particularly at low temperatures. Thus, the use of elastomer hardness has tended to mislead or confuse engineers in the design of elastomeric bearings and their expectations of bearing performance. The proposed recommendations overcome these difficulties by basing design on the shear modulus and defining an appropriate test procedure for measuring it.

For designers who wish to continue using hardness, the range of shear modulus which corresponds to a particular hardness is given in the proposed specification, but each aspect of the design is to be based on the most disadvantageous shear modulus from the range. This is done to allow for the uncertainty in the true G value when the hardness alone is specified, and the penalty it imposes provides an incentive to measure and use the true shear modulus. The proposed specification also deals with the variation in elastomer stiffness with temperature (11) through appropriate material property and testing requirements.

The design of a laminated elastomeric bearing requires an appropriate balance between the compressive, shear, and rotational characteristics. The bearing must have an appropriate height, plan area, layer thickness, and elastomer stiffness to accommodate lateral movement and rotation without developing excessive force or moment or allowing

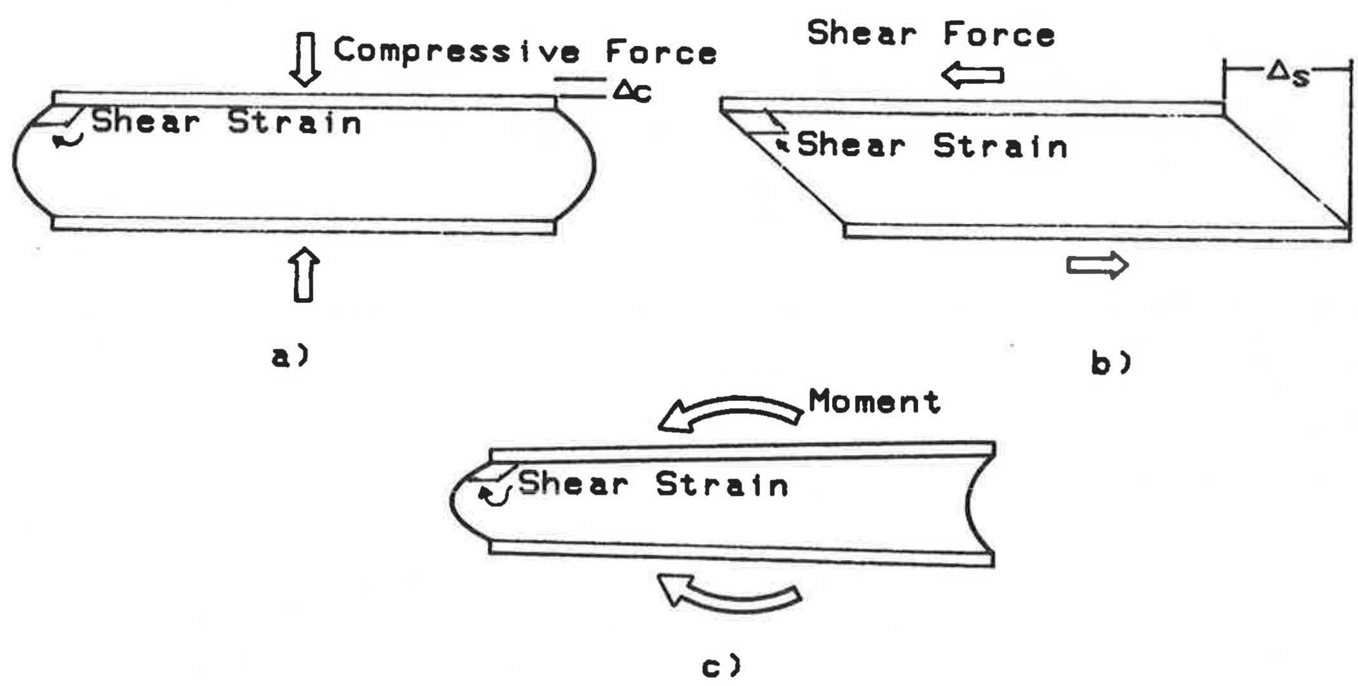


FIGURE 2 Strains in elastomeric bearings

excessive vertical deflections. In addition, the stresses and strains in the bearing materials must be restricted to tolerable levels. The total shear strains in the elastomer must be small enough to prevent fatigue (8) and delamination (4,9) of the elastomer from the reinforcement. The compressive stress on the bearing is further limited by stability (10), and the tensile strength of the reinforcement must also be adequate (4). These design considerations have been investigated in great detail (3-11) and design equations have been proposed (6-7) to control each mode of failure. The design equations are discussed in detail elsewhere (6-7), but they are briefly summarized in Fig. 3.

APPLICATION OF THE DESIGN EQUATIONS

The recommended design constraints of Fig. 3 appear to be complicated and must be simultaneously satisfied, but this process becomes simple with the aid of a personal computer and a spreadsheet. The equations and inequalities of Fig. 3 were programmed into an EXCEL spreadsheet on a Macintosh computer and Fig. 4 shows a typical printout. The row and column identifiers are retained in this figure to help identify terms, but they would normally be excluded. Five groups of numbers must be input, and they are highlighted by enclosing them in boxes. The upper left hand box (cells C5-9) contains the material properties. Allowance is made for a maximum and minimum shear modulus, in case the designer chooses to define the elastomer by its hardness. The program then selects the most disadvantageous G value at each step.

The upper right hand box (cells E5-9) contains the load, movement and rotation data. The remaining three smaller boxes contain the geometric properties of the bearing, such as the plan dimensions (L and W), the rubber layer thickness (h_{ri}), the steel laminate thickness (h_s) and the number of rubber layers (N), which should be entered in the order shown. Cells C13-28 contain the maximum and minimum permissible values for each quantity if the constraints of Fig. 3 are to be satisfied. As each dimension is chosen and entered in column E, the constraints on the next one appear in column C and some additional quantities (such as the shape factor corresponding to the chosen dimensions) appear in column E outside the boxes. In some cases, such as in cells C23 and 24, two or more criteria exist for one quantity, and the value chosen must satisfy both.

Figure 4 therefore represents an illustrative example

<u>Definitions</u>

Shape Factor : $S = \dfrac{\text{Plan Area}}{\text{Perimeter Area Free to Bulge}}$

Compressive Stiffness :

$$E_c = 3G[1+2\,kS^2] \text{ (psi)}$$

Shear Stiffness : $H = GA\dfrac{\Delta_s}{h_{rt}}$

<u>Deformation Limits</u>

Shear: $h_{rt} > 2\,\Delta_s$

Rotation : $\Theta_{TL,x} < 2\,\Delta_c / L$

<u>Compressive Stress Limits</u>

Delamination : $\sigma_{s,TL} < 1{,}600$ psi

Stability (sidesway prevented) :

$$\sigma_{c,TL} = \frac{G}{\left(\dfrac{1.92(h_{rt}/L)}{S\sqrt{1+2L/W}} - \dfrac{2.67}{S(S+2)(1+L/4W)}\right)}$$

Stability (sidesway permittted) :

$$\sigma_{c,TL} = \frac{G}{\left(\dfrac{3.84(h_{rt}/L)}{S\sqrt{1+2L/W}} - \dfrac{2.67}{S(S+2)(1+L/4W)}\right)}$$

<u>Combined Stress Limits</u>

Fatigue (with shear deformation) :

$$\sigma_{c,TL} \le \frac{1.66\ G\ S}{\left(1 + \dfrac{L\ \Theta_{TL,x}}{4\ \Delta_c}\right)}$$

and

$$\sigma_{c,LL} \le 0.66\ G\ S$$

Fatigue (no shear deformation) :

$$\sigma_{c,TL} \le \frac{2.0\ G\ S}{\left(1 + \dfrac{L\ \Theta_{TL,x}}{4\ \Delta_c}\right)}$$

and

$$\sigma_{c,LL} \le 1.0\ G\ S$$

<u>Reinforcement Thickness</u>

$$h_s \ge \frac{1.5\ (h_{r1} + h_{r2})\ \sigma_{c,TL}}{F_y}$$

and

$$h_s \ge \frac{1.5\ (h_{r1} + h_{r2})\ \sigma_{c,LL}}{F_{sr}}$$

FIGURE 3 Proposed design equations for reinforced elastomeric bearings

of the use of the design procedure with the spreadsheet program. In this example, the bearing is designed for a dead load of 78 kips, a live load of 33 kips, a translational movement of .87", and a rotation of .003 radians. The resulting bearing is 7" x 12" with 0.25" layers. It is clear that this 111 kip load is quite large for a bearing of this size compared to the stress level historically permitted for elastomeric bearings by the AASHTO Specification. It clearly illustrates the benefits of the proposed provisions because this increased load capacity allows broader use of elastomeric bearings.

	A	B	C	D	E
1		Elastomeric Bearing Design			
2		using			
3		AASHTO Method B			
4					
5		Gmin(ksi)=	0.100	P DL(kip)=	78
6		Gmax(ksi)=	0.135	P LL(kip)=	33
7		k bar =	0.6	P TL(kip)=	111
8		Fy (ksi) =	36	rot (rad) =	0.003
9		Fsr (ksi) =	24	Δs (in) =	0.870
10					
11		Max/min		Actual values	
12					
13		area	69.375	area	84
14		L	5.781	L(short)	7
15		W	9.911	W(long)	12
16				TL stress	1.321
17				LL stress	0.393
18		h ri (TL)	0.278		
19		h ri (LL)	0.371	h ri	0.250
20		S (TL)	7.960		
21		S (LL)	5.952	S	8.842
22				Ec	38.402
23		h s (TL)	0.0275		
24		h s (LL)	0.0123	h s	0.0625
25		N lay (Δs)	7.0	N layers	7
26		N lay (uplift)	1.0	h total	2.188
27		N lay (comp)	4.2	vol elastomer	147
28		N lay (stab)	19.0	vol steel	37
29				weight (lbs)	17

FIGURE 4 Sample output of spreadsheet program

The program is simple to execute and designs can be completed very rapidly. However, the program does not automatically choose the geometry of the bearing The designer must select a value for each input quantity, ensuring that it falls between the minimum and maximum permissible values calculated by the program. The example shown in Fig. 4 satisfies the design equations, because all of the dimensions lie between the minimum and maximum permissible values. Design with the spreadsheet is very quick and can be illuminating, since it illustrates how changes in the bearing dimensions and material properties affect the design.

The calculations in the lower right hand corner (cells E27-29) give the weight and volume of the selected bearing. These quantities are not required by the design provisions, but they provide useful information for the engineer. The weight and size of the bearing provide a direct measure of the ease with which it can be handled and installed and a basis for estimating its approximate cost.

Fig. 5 shows the formulas used in the spreadsheet of Fig. 4. All the design constraints of Fig. 3 are included, and the calculations are for bearings which are subject to shear displacements. Bearings fixed against translation lead to higher load capacity due to improved fatigue and buckling behavior and a separate spreadsheet could be written for that case. Other changes, such as some level of automatic redesign, could be incorporated if desired.

EFFECT OF PROPOSED PROVISIONS

The spreadsheet can be used to evaluate the effect of the proposed provisions on elastomeric bearing design and to compare the recommended provisions to the 1961 AASHO (1) and the 1989 AASHTO (3) Specifications. The comparisons are made using an elastomer with a nominal hardness of 55 on the Shore 'A' scale, which corresponds approximately to nominal, minimum and maximum shear moduli of 110, 95 and 125 psi (.76, .66 and .86 MPa). The comparisons are all based on rectangular bearings with W/L=2.0 and dead load/live load =1.5. These values were chosen because they fall within the practical range and because the rectangular shape allows more rotation than does a square.

The effect of the proposed provisions is illustrated in Fig. 6, which shows old and new load ratings for bearings with 0.25" and 0.75" thick rubber layers. Bearings with plan dimensions between 5" by 10" and 15" by 30" are represented in the graph. The proposed provisions confer a higher load rating than do the old ones on bearings with thin layers and high shape factors. Any bearing larger than 6" by 12" has a high enough shape factor that the load is governed by the 1600 psi limit. Load on the bearings with 3/4" layers is governed by the fatigue limit (a multiple of GS) which, for the shape factors in question, is less than 1600 psi, and the change from the old provisions is less pronounced. In fact, when the maximum rotation occurs, the allowable load is very similar to the value under the old provisions. Thus it can be seen that the proposed provisions offer the greatest advantage if a high shape factor is used.

The upper limit on load size of an elastomeric bearing is controlled by manufacturing techniques. The elastomer in a very large bearing cannot be cured uniformly, reliably and economically because the heat takes time to penetrate through the bulk of the material. There is no precise limit because each manufacturer has different processing equipment but, under the proposed provisions, 600 kips represents an approximate upper bound. The proposed provisions thus extend the load range for which elastomeric bearings are feasible, and offer the possibility of using them in situations where other more trouble-prone and potentially more expensive bearings have been used in the past.

The graph also shows that, for the bearings with 3/4" layers, application of the maximum rotation reduces the allowable load considerably. Rotation appears from the graph to have a larger effect on bearings with thicker layers, which

	A	B	C	D	E
1					
2			using		
3			AASHTO Method B		
4					
5			0.1		78
6			0.135		33
7			0.6		=E5+E6
8			36		0.0025
9			24		0.87
10					
11					
12					
13			=E7/1.6		=E14*E15
14			=C13/E15		7
15			=C13/E14		12
16					=E7/E13
17					=E6/E13
18			=E14*E15*0.5/(C20*(E14+E15))		
19			=E14*E15*0.5/(C21*(E14+E15))		0.25
20			=E16/(1.66*C5)		
21			=E17/(0.66*C5)		=E14*E15*0.5/(E19*(E14+E15))
22					=3*C6*(1+2*C7*E21^2)
23			=3*E19*E16/C8		
24			=3*E19*E17/C9		0.0625
25			=2*E9/E19		7
26			=0.5*E8*E14*E22/(E16*E19)		=E25*(E19+E24)
27			=0.25*E14*E8*E22/(E19*(1.66*C5*E21-E16))-0.4		=E13*E25*E19
28			=(C5/E16+2.67/(E21*(E21+2)*(1+0.25*E14/E15)))*E14*E21*SQRT(1+2*E14/E15)/(1.92*E19)		=E13*E25*E24
29					=0.285*E28+0.0434*E27

FIGURE 5 Formulae used in Spreadsheet

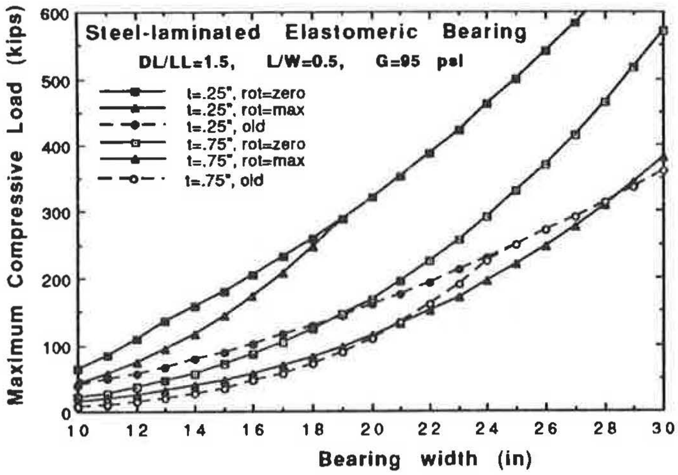


FIGURE 6 Compressive load capacity of a rectangular elastomeric bearing as a function of bearing width

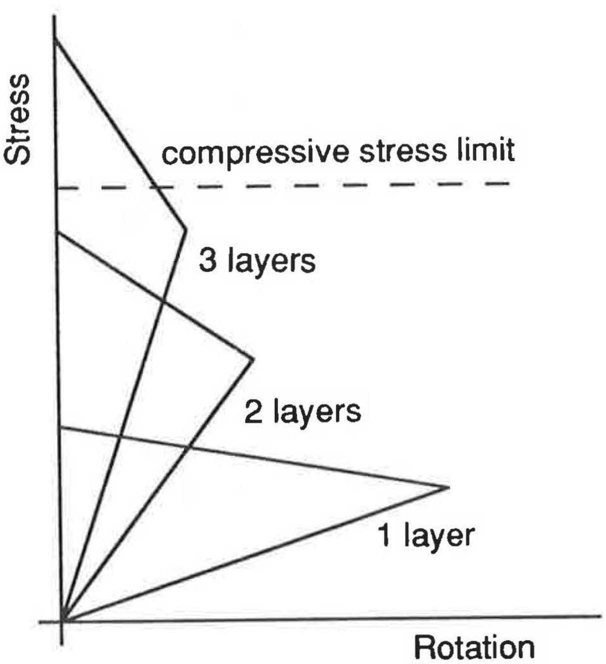


FIGURE 7 Interaction of rotation and compressive load

seems contradictory but the explanation lies in the fact that θ_{max} for the 3/4" layer bearings is much larger than θ_{max} for the 1/4" layer bearings. Rotation had no influence on load rating under the old specifications because it was not included explicitly.

The relationship between compressive stress and rotation is further illustrated in Fig. 7 in which stress is plotted against rotation capacity for a bearing with given plan dimensions and total rubber thickness. The three different curves represent different numbers of layers (and different layer thicknesses). The individual interaction curves resemble the axial force - bending moment interaction curves used in reinforced concrete design. The upper line represents 'tension-controlled' designs, in which rotation is limited by the requirement of no uplift, and the lower line represents 'compression-controlled' designs in which the fatigue or delamination limit is active. In most cases the maximum rotation capacity occurs at 2/3 the maximum stress allowed by the fatigue limit. If the bearing load is high but the rotation is large, the best design strategy may be to choose a shape factor which causes this 2/3 maximum fatigue stress to be equal to the 1600 psi delamination stress. For a 55 durometer elastomer, this would require a shape factor of about 11, which is higher than is commonly used today in the USA, but is typical in Europe and Australia. Such an approach makes use of the highest possible compressive stress yet optimizes the rotation capacity for that stress. If the rotation capacity is too small, a smaller number of thicker layers will be needed, but the allowable stress then decreases and so the bearing must be larger.

With hand design calculations, it is difficult and tedious to achieve these optimum conditions, but the spreadsheet speeds up the calculations so much that optimum or near-optimum bearings can be designed very rapidly.

Rotations are important under the proposed provisions whereas they have been ignored in previous specifications. This has two main consequences. First, the largest component of rotation comes from inaccuracies in levelling the bearing and in fabrication of the girders which rest on it. Thus better quality control in these operations would reduce the rotation demand on the bearing and allow a greater proportion of its capacity to be used in resisting compressive load. Second, a greater total rubber thickness leads to a larger rotation capacity, so rotation should be taken into account when selecting the bearing height.

The relationship between rotation capacity and height is illustrated in Fig. 8. The bearing properties are those used in Fig. 6, and in addition shear displacement is assumed to be

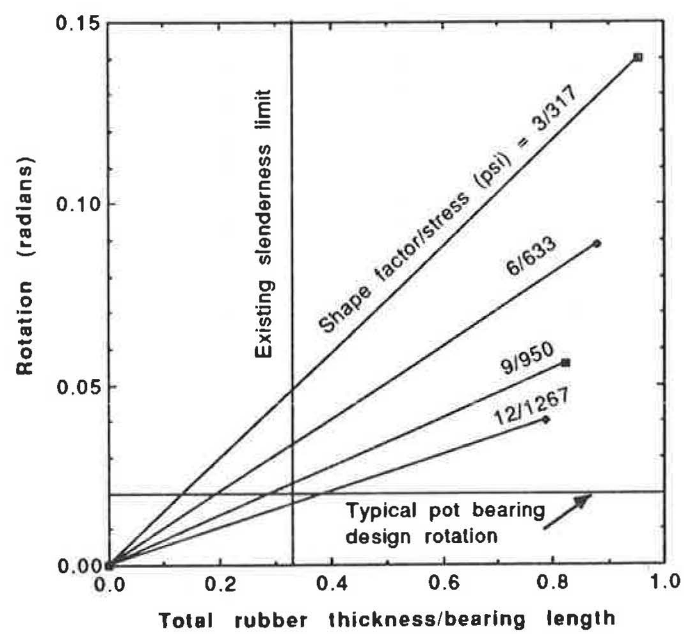


FIGURE 8 Rotational capacity as a function of slenderness

the maximum possible, but, for buckling calculations, sway is assumed to be prevented. These conditions correspond to those at the free end of a bridge in which the other end is fixed. For each shape factor, the compressive stress which allows the maximum rotation is used, and the curve stops at the stability limit. The figure shows that taller bearings are now possible with the improved buckling models (10) used in the recommended provisions. It also shows that the maximum rotation increases significantly with increasing slenderness or bearing height and with decreasing shape factor. The common pot bearing design rotation of 0.02 radians is marked on the figure, and many elastomeric bearings can be seen to have a larger rotation capacity than it. This is an interesting observation, because pot bearings are usually regarded as high rotation bearing systems whereas elastomeric bearings are not.

The footprint of a pot bearing is usually controlled by the allowable stress on the concrete seat, which is typically 1000 to 1500 psi. Since the proposed provisions permit compressive stresses on laminated elastomeric bearings of this order of magnitude, the space required for both bearings will be comparable. Fig. 8 shows that an elastomeric bearing could provide greater rotation capacity and could be a viable alternative.

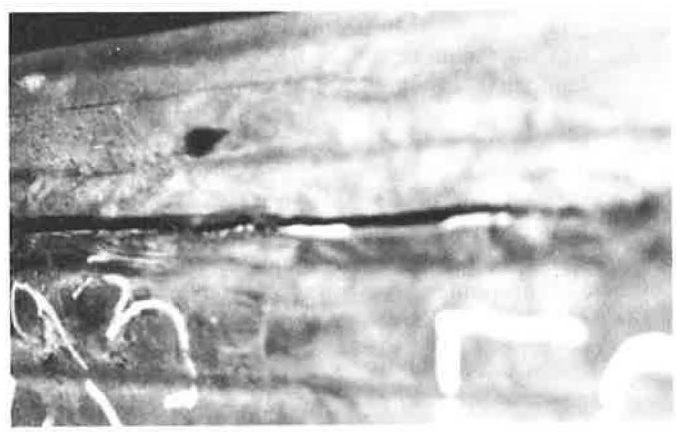
QUALITY CONTROL

The proposed provisions offer some real benefits but require some additional effort in the engineering design. However this paper has shown that the design calculations can be greatly simplified with the aid of a simple spreadsheet. The increased capacities also require tighter quality control during bearing manufacture and installation.

Problems associated with elastomeric bearings can generally classified as: poor design or choice of materials, poor quality materials, poor quality fabrication and improper installation. The first and last are controlled by the designer and installer and are not considered further here. The quality of the materials used is assured by ASTM material property tests, and most of those previously required by AASHTO (such as tensile strength and elongation at break) are retained with largely unchanged limiting values. Material stiffness tests are required in place of the former hardness test, since stiffness is a more direct and reliable indicator of bearing behavior. In addition, low temperature stiffness tests (7,11) are required for colder climates to assure that the elastomer has appropriate low temperature characteristics. The low temperature brittleness test is retained but the target values are modified to reflect more accurately the regional climate. These improved material property tests are a necessary complement to the proposed higher design stresses but they should add little or no cost to most bearings.

Fabrication quality is addressed by load testing. It is not foolproof, but it represents a good compromise among simplicity, reliability and cost. Previous specifications did not require load tests. Under the proposed provisions every steel-laminated elastomeric bearing would be subjected to a short duration load test to 150% of its maximum service load. Bearings which exhibit cracking of the elastomer or unusual bulging patterns such as shown in Fig. 9a or 9b would be rejected. These flaws typically indicate a poor quality elastomer, excessive strain in the bearing due to poor geometry control, or partial delamination of the steel from the elastomer.

a)



b)

FIGURE 9 Potential problems in the manufacture of elastomeric bearings

This test requires 10 minutes or less to complete and can be performed with equipment available to all elastomeric bearing manufacturers.

The short duration load test will show most fabrication flaws, but some appear only under long duration loading so a long duration load test is also proposed. Unusual or unacceptable bulge patterns or cracks or flaws in the elastomer as shown in Fig. 9a or 9b will again be the basis for rejection. The long duration load test is an effective but more expensive and time consuming quality control test. It requires that a load which is 150% of the maximum service load be applied and maintained for 15 hours. As a result, the long duration load test is required only for a sample of the bearings which have already passed the short duration load test. Failure

of a single bearing is cause for either rejecting the entire lot of bearings or additional testing of the entire lot.

Improved quality control during installation is also required (7) to assure that the bearing performs as expected. For example, the bearings must sustain rotations caused by out of level pier caps or excessive camber or twist in bridge girders. It is essential that tolerances be controlled on the construction site if the bearing is to perform properly. This can be done by requiring levelling shims or a grout bed to ensure that the bearing is properly positioned and levelled. Installation tolerances are more important than ever, because the recommended provisions have less inherent conservatism built in to them. The recommended provisions are safe, but they will not protect the bridge against careless workmanship. The good judgement of the bridge engineer combined with adherence to the required tolerances are needed for this last element of quality control.

PRACTICAL IMPLICATIONS

The proposed bearing design provisions offer substantial advantages, but require more detailed calculations and tighter quality control. This paper has reviewed all three elements. The major practical implications are -

1. Laminated elastomeric bearings can be designed for compressive stress levels well above the upper limit of 800 psi which has historically been used for elastomeric pads. As a result, smaller and more economical bearings can now be used, and more important, laminated elastomeric bearings can sometimes be used in place of other more expensive and often more trouble prone bearing systems.
2. Improved stability models have been developed. As a result taller laminated elastomeric bearings can be designed which can accommodate larger translational movements and larger rotations. This again indicates that elastomeric bearings can sometimes be used in place of other large movement or high rotation bearing systems.
3. Smaller and taller bearings will result in smaller forces and moments transferred between the substructure and the superstructure. This can lead to increased economy and greater reliability of the structural design.
4. The benefits are achieved at the cost of increased effort in the design calculations. However, the design calculations can be completed quickly, economically, and systematically with the aid of the simple spreadsheet described in this paper.
5. The benefits also require improved quality control in the manufacture and installation of the bearing. Suitable quality control measures are briefly reviewed in the paper. They will add slightly to the cost of an individual bearing, but the increased cost will be small compared to the potential overall savings.

CONCLUSIONS

This paper presents a brief overview of recent proposed provisions for the design of laminated elastomeric bridge bearings. It summarizes the design calculations, and shows how they can be easily executed with the aid of a computer spreadsheet. It illustrates the effect of the recommendations on bearing design, and clearly shows that there can be a very beneficial effect on the total bridge design.

ACKNOWLEDGMENTS

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Using the Workshop Process to Foster Innovative Designs

WALTER C. ROEHRS

ABSTRACT

The Workshop Process has proven to be an effective management technique which creates an environment that fosters innovative design. The process brings together a small group of experts with a variety of specialty talents to focus their creative energies on a specific design problem. The Workshop Process borrows many techniques from the Value Engineering (VE) Study Process. Like the VE Study, the Workshop Process is carried out in a short time period, usually five days following a specific agenda. A typical agenda for a workshop team includes the following:

1. Information Exchange Phase
2. Creative Phase (Brainstorm Design Alternatives)
3. Develop Weighted Criteria for Judging Alternatives
4. Evaluate and Concentrate Design Alternatives
5. Develop and Enhance Most Promising Alternatives
6. Written and Oral Presentation of Recommendations.

The Workshop Process was effectively used to develop the preliminary design for replacement of the Broad Street Bridge over the Scioto River in downtown Columbus, Ohio. Because this bridge is located in a prominent location within an historic district, special emphasis on aesthetics and historical compatibility was important. For the Broad Street Bridge, two design workshops were held. The first workshop generated 117 different alternative designs which were concentrated and refined to 13 design alternatives for public consideration. The second workshop incorporated the public response to the results of the first workshop and developed the recommended design alternative for the replacement of the Broad Street Bridge. Because aesthetics were an important consideration in the design of this bridge, scale models and numerous perspective sketches and renderings were prepared to assist both the workshop team and the public in their consideration of the various alternatives. The resulting final design, most likely would not have been conceived by routine design processes.

INTRODUCTION

The Workshop Process has proven to be an effective management technique which creates an environment that fosters innovative design. The process brings together a small group of experts with a variety of special talents to focus their creative energies on a specific design problem. The Workshop Process borrows many techniques from the Value Engineering (VE) Study Process. Like the VE Study, the Workshop Process is carried out in a short time period, usually five days following a specific agenda. Unlike the VE Process which is used to analyze something already designed, the Workshop Process is used to create a design. This paper will first describe the Workshop Process and then briefly illustrate its use in developing the preliminary design for the replacement of the Broad Street Bridge over the Scioto River in downtown Columbus, Ohio.

WORKSHOP PROCESS

The Workshop Process usually follows a specific agenda which is completed in about five days (see Figure 1). The Workshop Process is subdivided into six specific phases. Each phase, as with a VE Study, is important and must follow specific guidelines to be effective.

FIGURE 1. TYPICAL WORKSHOP AGENDA

PREWORKSHOP:

- Define the scope of the design problem
- Select Workshop Team
- Schedule Workshop

DAY 1

Phase 1 - Information Exchange (4 hours)
Phase 2 - Creative Phase (1-2 hours) (Brainstorm Alternatives)
Phase 3 - Develop Weighted Criteria for Judging Alternatives (2-4 hours)

DAY 2

Phase 4 - Evaluate and Concentrate on Design Alternatives (8-12 hours)

DAY 3

Phase 5 - Develop and Enhance Most Promising Alternatives (8 hours)

DAY 4

Phase 5 - Continued from Day 3 (8 hours)

DAY 5

Phase 5 - Concluded from Day 4 (4 hours)
Phase 6 - Written and Oral Presentation of Team's Recommendations (4 hours)

Preworkshop Activities

Working with the Owner, the Workshop Coordinator develops in detail the scope of the design problem. The scope of the design problem usually involves design of some facility such as a bridge. The Workshop Coordinator determines from the Owner such needs as:

- Required capacity, size, shape, durability
- Aesthetic considerations
- Performance criteria, operating characteristics
- Functions, primary and secondary
- Acceptable costs
- Safety requirements

Once the scope of the design problem is determined, the Workshop Team is selected. As with a VE Study, the Workshop Team seems to work well with from four to seven or eight members from various backgrounds depending upon the design problem. Often knowledgeable representatives of the Owner are chosen to participate on a Workshop team because of the special perspective that they bring. For example, a Workshop Team to study a bridge replacement project in an historic area could include: bridge engineers, cost estimator, traffic engineer, Owner's representative, construction specialist and an architect experienced in civic architecture.

It is important that the Design Workshop be held at a location isolated from everyday distractions. It is good to provide at least one banquet-size work table for each team member. The room should accommodate two or three easels and perhaps a chalkboard. Telephone and copying services should be available. These accommodations are often available through hotels. Each team member should commit to attend the entire Workshop. Sometimes it may be necessary to schedule a Workshop over a weekend to accommodate the schedules of busy experts desired for the team.

Phase 1–Information Exchange

The actual Workshop Process for the Workshop Team begins with Phase 1, the *Information Exchange*. Each team member is furnished with copies of material collected by the Workshop Coordinator. During the information exchange, the scope of the design problem is completely described. The team may visit the site of the design problem. Photographs of the site, where appropriate, should be available and described to the team. Presentations to the Workshop Team by individuals to outline the scope of the problem should be scheduled during this phase. This will allow the team to question individuals presenting information, thus reducing potential misinterpretation. Important information for the information exchange includes:

- Existing plans, maps, etc.
- Survey data, topography, traffic counts
- Environmental data, weather, floods, etc.
- Construction cost data
- Constraints; funding, legal
- Regulatory requirements, design standards, codes
- Specifications

Phase 2–Creative Phase

During the *Creative Phase* of the Workshop Process, the team "brainstorms" alternative solutions (ideas) to the design problem. In a VE Study, this often is called the *Speculative Phase*. Brainstorming is a deliberate effort to generate ideas without making any judgment regarding their practicality or usefulness. The following rules for effective brainstorming should be enforced by the Workshop Coordinator:

1. No idea may be criticized. Judgment and evaluation are postponed to a later phase.

2. All ideas are written and posted in full view of the team during this phase.

3. Freewheeling is encouraged. Wild ideas are welcomed. It is easier to tone down a wild idea than to juice up a tame one.

4. Strive for many ideas. The more the better. Greater numbers increase the potential for more good ideas.

5. Combine ideas. Improve ideas. Build on ideas of others.

Phase 3–Developed Weighted Criteria for Judging Alternatives

Alternatives or ideas generated in the brainstorming session of Phase 2 need to be evaluated based upon some criteria. A technique borrowed from VE involves using a matrix to weight selected criteria. The various criteria should be mutually exclusive, i.e., no over-lapping criteria will be allowed. The Workshop Team should carefully define and select the criteria for judging each alternative. Criteria for judging alternatives could include such items as:

- Initial cost
- Estimated life
- Aesthetic appeal; appearance
- Flexibility
- Operating cost
- Time to design and construct

Each criteria item is compared with the other criteria items using a matrix Similar to Figure 2. For example, in the A-C box, the notation of A-4 indicates a major preference (4 points) in favor of criteria A over criteria C. Total points are counted for each criteria and a weight of 10 assigned to the highest scoring criteria. Whole number weights are assigned in proportion to the raw score of the remaining criteria. It is important that the Workshop Team fully discuss and agree upon the criteria and the weighted criteria before moving to the next phase. The workshop Coordinator should strive to arrive at the weighted criteria by consensus of the entire team.

Phase 4–Evaluate and Concentrate Design Alternatives

Alternatives (ideas) generated in Phase 2 are now evaluated and concentrated in this phase. When ideas are evaluated, often new ideas are suggested. It is important to always be open to and accept new ideas or alternatives, especially in this phase. New ideas should be added to the listing developed from Phase 2.

Ideas can be screened using a form similar to Figure 3 which quickly notes perceived advantages and disadvantages. The Workshop Team, by consensus, rates each idea from 1 to 10 points. The highest scoring ideas are retained for further evaluation. The other ideas will not be evaluated

further.

FIGURE 2. CRITERIA WEIGHING

PROJECT	CRITERIA WEIGHTING	
LOCATION		
CLIENT	ITEM:	NO.
DATE		
PAGE OF		

PROJECT ITEM

TEAM DATE

CRITERIA	WEIGHT	RAW SCORE
A Initial Cost	10	21
B Maintenance	3	7
C Aesthetics	1	1
D Quality Control	9	18
E Space Flexibility	2	4
F Construction	5	11
G Other Project impact	2	3
H		
I		

CRITERIA SCORING MATRIX

HOW IMPORTANT
4-MAJOR PREFERENCE
3-MEDIUM PREFERENCE
2-MINOR PREFERENCE
1-SLIGHT, NO PREFERENCE
ONE POINT EACH
(LETTER/LETTER)

	B	C	D	E	F	G	H	I
A	A-4	A-4	A-3	A-3	A-3	A-4		
B		B-3	D-3	B-1/E-1	F-3	B-3		
C			D-4	C-1/E-1	F-3	G-2		
D				D-4	D-3	D-4		
E					E-1/F-1	E-1/G-1		
F						F-4		
G								
H								

Using the weighted criteria from Phase 3, the remaining ideas are scored by the team based on how well they satisfy each criteria (see Figure 4). Using a rating system from 1 to 4. A rating of 4 would indicate the idea scored <u>excellent</u> in satisfying the criteria and a criteria and a rating of 1, a score of <u>poor</u>. The score is multiplied by the fact or weight and totaled for each idea. When all the ideas, including any new ones, are evaluated, the highest scoring ideas are retained. The remaining ideas are dropped from further consideration.

Phase 5–Develop and Enhance Most Promising Alternatives

The most promising alternatives to the design problem selected in Phase 4 are now studied in greater detail. Preliminary sketches and written descriptions are made for each idea. Quantities and construction costs are estimated. If appropriate, life-cycle costs are computed. Advantages and disadvantages for the particular idea are tabulated. This work is done in a format and often on forms borrowed from VE. Several members of the team often work together on each idea. If the team includes a cost estimator, it makes good sense that person prepare all the cost estimates to insure consistency.

Phase 6–Written and Oral Presentation of Team's Recommendation

At the end of the Workshop, the team's recommendations are presented to the Owner and others. When Phase 5 is completed, the team considers the ideas developed and selects the most promising to recommend to the Owner. Scheduling an oral and written (photocopy of work papers) report to the Owner at the conclusion of the Workshop forces the team to concentrate on the design problem.

EXAMPLE OF WORKSHOP PROCESS

The Workshop Process effectively was used to develop the preliminary design features for the replacement of the Broad Street Bridge over the Scioto River in downtown Columbus, Ohio. Because of this bridge's prominent location within an historic district, special emphasis on historical compatibility and aesthetics was important. To fully develop the preliminary design for the Broad Street Bridge replacement, two design Workshops were held. The first Workshop generated 117 different alternative designs which were concentrated and refined by the Workshop Team to 13 design alternatives for public consideration. The second Workshop incorporated public response to the results of the first Workshop and developed the recommended design alternative for the replacement of the Broad Street Bridge.

FIGURE 3. IDEA EVALUATION

PROJECT / LOCATION / CLIENT / DATE / PAGE OF	CREATIVE PHASE — ALTERNATIVE LISTING		JUDGEMENT PHASE — ALTERNATIVE EVALUATION	
NO.	ALTERNATIVE	ADVANTAGES	DISADVANTAGES	ALT. RATING
A	Idea A	Low Cost	High Maint. Cost	5
			Poor Aesthetics	
B	Idea B	Higher Cost	Poor Durability	6
		Good Aesthetics	Safety Questioned	
C	Idea C	Moderate Cost		8
		Safe		
		Durable		

LIST ALL ALTERNATIVES BEFORE PROCEEDING TO JUDGEMENT PHASE
(10) MOST DESIRABLE (1) LEAST DESIRABLE

FIGURE 4. IDEA ANALYSIS

PROJECT
LOCATION
CLIENT
DATE
PAGE OF

ALTERNATIVE EVALUATION

ITEM: NO.

	Initial Cost	Maintenance	Aesthetics	Quality Control	Space Flexibility	Construction	Other Project Impact		TOTAL	RANK
FACTOR WEIGHT 10 = MAXIMUM	A 10	B 3	C 1	D 9	E 2	F 5	G 2			
Idea A	20 / 2	9 / 3	3 / 3	18 / 2	8 / 4	15 / 3	6 / 3		79	4
Idea B	40 / 4	12 / 4	3 / 3	27 / 3	8 / 4	20 / 4	8 / 4		118	2
Idea C	40 / 4	12 / 4	2 / 2	27 / 3	8 / 4	20 / 4	6 / 3		115	3
Idea D	40 / 4	12 / 4	2 / 2	36 / 4	6 / 3	20 / 4	8 / 4		134	1
Idea E	30 / 3	6 / 2	2 / 2	18 / 2	4 / 2	10 / 2	4 / 2		74	5

SCORING IDEAS vs FACTORS 4-EXCELLENT 3-GOOD 2-FAIR 1-POOR

WORKSHOP SESSION I

The Workshop Team for the first Workshop consisted of nine members described as follows:

Team Role	Representing
Workshop Coordinator	Consultant, Partner
Bridge Engineer	Consultant, Chief Bridge Engineer
Civil Engineer	Consultant, Transportation Engineer
Historical Specialist	Subconsultant, Civic Architect
Aesthetic Specialist	Subconsultant, Civic Architect
Owner Representative	Owner (County), Bridge Engineer
Estimator	Consultant, Estimator
Bridge Specialist	Subconsultant, Bridge Specialist
Shareholder Representative	Shareholder (City), Bridge Engineer

A five-day agenda similar to Figure 1 was used by this Workshop Team to identify 117 alternatives. These 117 alternatives were refined and condensed into 13 alternatives for further consideration and public presentation (see Figure 5). Sketches, cost estimates and discussion was developed for each of the 13 recommended alternatives (see Figure 6 and 7). After the first Workshop, the 13 alternatives were presented at public and special interest group gatherings. Public comment was taken and encouraged.

WORKSHOP SESSION II

The team composition and Agenda for the second Workshop was essentially the same as for the first one. The scope for this Workshop was to incorporate the public comments obtained with the 13 alternatives and develop three final alternatives for in-depth study. The team recommend one alternative as the most appropriate for the bridge replacement. Because aesthetics were an important consideration in the design of this bridge, scale models and perspective sketches from a variety of viewpoints were used by the team in consideration of the various alternatives. The final alternatives included two 5-span and one 3-span plate arched bridges which retained key visual elements of the existing bridge and surrounding architecture. Weighted criteria to judge the alternatives was developed by the team as follows:

Criteria	Weight
Life Cycle Cost	4
Compatibility	6
Constructability	2
Fundability	3
Historical	6
Aesthetics	10

A matrix analysis similar to Figure 4 performed by the team determined the recommended alternative. This alterative–which became the final design for the Broad Street Bridge replacement–most likely would not have been conceived by routine design processes. The Workshop Process allowed meaningful public input and openly demonstrated concern and care for other criteria beyond the normal least-cost criteria.

FIGURE 5. BRIDGE ALTERNATIVES SUMMARY

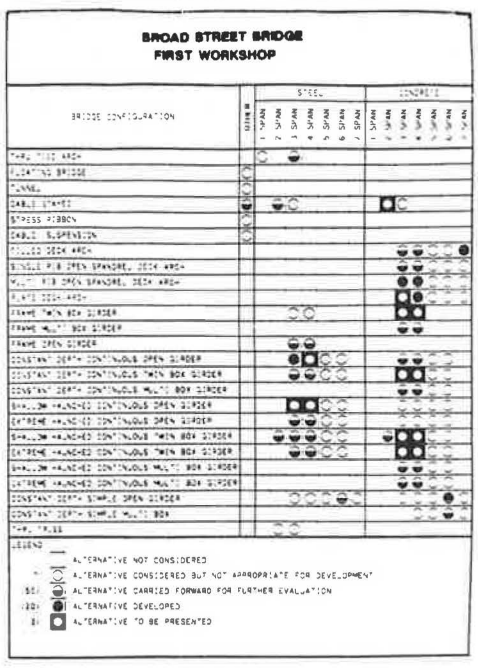
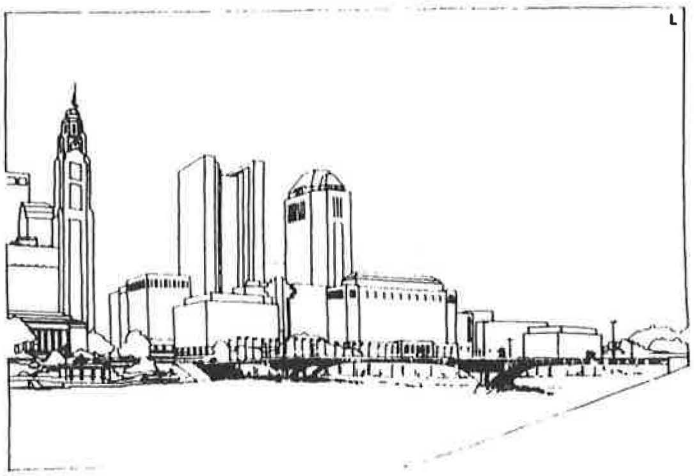
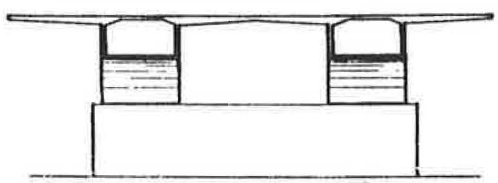


FIGURE 6. PRELIMINARY ALTERNATIVE L, SKETCH



BROAD STREET BRIDGE REPLACEMENT

FIGURE 7. PRELIMINARY ALTERNATIVE L, SECTION AND DESCRIPTION



Three-span, continuous, deep haunched, twin concrete box girder bridge with span lengths of 190'-300'-190'. The transversely posttensioned deck is supported by twin concrete box girders. Girders vary in depth from 17 feet at the piers to 6 feet at mid-span and abutments. Extra depth is provided at pier haunches to enhance the arch effect.

Cast-in-place methods of construction will be used. Construction will require a contractor familiar with posttensioning. Construction cost is estimated at $7,800,000. This bridge will require normal maintenance. Inspection will require special equipment for below the deck and access manholes into the box girders.

Two variants of the haunch girder have been studied, one with a 12-foot deep section at the piers and the other with this dimension increased to 17 feet. The difference is primarily a matter of choice on aesthetic grounds, the lesser-depth haunch providing a lighter and more open appearance, and the deeper one presenting a stronger profile with more pronounced curvature.

Both of the concrete haunched girder structures use box girder sections, resulting in pleasing smoothness of forms and surfaces with the possibility of interesting textures and colors. Particularly observable from below, the underside of the girders will be smooth.

SUMMARY

The Workshop Process provides an effective management technique to focus and concentrate a wide variety of talent for innovative design. It provides a systematic and democratic process to assimilate a wide variety of viewpoints into design solutions. It creates an environment favorable to innovation. The Workshop Process can reduce development time where innovative design is needed or desired. The process is usually found to be stimulating and fun by the participants. It helps make "early" design decisions before expending significant effort by using experience of the team.

Research and development areas to improve the Workshop Process could include:

- Workshop Team composition,
- Optimum team size,
- Idea generation techniques, and
- Idea scoring and rating techniques.

Use of the Workshop Process is not appropriate in every instance. Innovation is often not necessary or desired. The process is best suited where:

- Innovation is desired,
- The best solution is not obvious, and
- Where diverse training and experience need be balanced.

The process is capable of dealing with design of a wide variety of things in addition to bridges. For instance, it could be used to help design an organization with its organizational chart and position descriptions. The strength of the process is that it permits a wide variety of talent to work within an innovative environment toward a design solution. It provides a good tool to foster innovative design.

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Federal Highway Administration Bridge Scour Practice

LAWRENCE J. HARRISON

The national attention given to scour-associated highway bridge failures since 1987 coupled with the paucity of press coverage on this issue prior to that time suggests that scour failures are of recent vintage. This is, in fact, not so. For example, 17 highway bridges failed due to scour in the northeastern part of the Nation in the same relative time frame that the Schoharie Creek/New York Throughway bridge lost its battle to scour in April 1987, The Federal Highway Administration and State highway agencies have been conducting research on bridge scour and considering scour in the design process for many years. Periodic highway bridge failures due to scour from the 1950's onward have provided the impetus for continued investigation of the mechanics of such failures with some advances in the design process. It was not, however, until the 1987 Schoharie Creek bridge failure that sufficient national interest focused on the issue to generate the need for national direction within the Federal-aid highway program. In September 1988 the Federal Highway Administration published Technical Advisory 5140.20, Scour at Bridges, to address bridge scour in the overall context of the design process and in the inspection of existing structures within the National Bridge Inspection Standards Program. This paper presents the elements of the bridge scour challenge, the state-of-the-art basis of scour evaluation and countermeasures to protect bridges against scour, all based on the Federal Highway Administration Technical Advisory 5140.20 and Hydraulic Engineering Circular No. 18, both entitled Scour at Bridges. The national status of the program is also presented. All of these issues are presented from the perspective of the Federal Highway Administration's role in the bridge scour program.

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The Big Sioux River was on the rampage. Just upstream of its confluence with the Missouri River, the Interstate 29 crossing had been completed just the year before. One of the two bridges associated with this crossing fails due to scour - 1962. A vital bridge on the Interstate system in the northwestern part of the Nation, Interstate 80 over the John Day River in Oregon, was under siege during a series of storms that struck the entire west coast and fails due to scour - 1964. The northeast was being battered by heavy rains which swelled many streams. Schoharie Creek attacked the New York Thruway bridge over Schoharie Creek which fails due to scour - 1987. As well, a bridge carrying US 51 over the Hatchie River in Tennessee fails due to scour - 1989. Isolated incidents? Failures uncommon to United States highway experience? Flukes? The lack of national attention given to scour-associated highway bridge failures prior to 1987, coupled with the press coverage since then on isolated failures, suggests that scour failures are of recent vintage and limited in number. This is, in fact, not the case. For example, 17 highway bridges failed in the northeast due to scour in 1987 during the same time frame that the Schoharie Creek structure failed.

FEDERAL HIGHWAY ADMINISTRATION TECHNICAL ADVISORY

In September 1988 the Federal Highway Administration (FHWA) published Technical Advisory (TA) 5140.20, Scour at Bridges, to address bridge scour in the overall context of the bridge design process and the inspection of existing structures within the National Bridge Inspection Standards (NBIS) Program. The TA was formulated to provide guidance on developing and implementing a scour evaluation program for 1) designing new bridges to be scour-safe, 2) evaluating existing bridges for vulnerability to scour, 3) using effective scour countermeasures, and 4) improving scour estimating procedures. Scour evaluations of new and existing bridges should be conducted by an interdisciplinary team comprised of structural, hydraulic and geotechnical engineers.

NEW BRIDGES

New bridges over waterways with scourable beds should be designed to withstand the effects of scour without failing for the worst conditions resulting from floods equal to or less than the 100-year flood. All bridge foundations should be checked to ensure that failure does not occur for scour resulting from a superflood; i.e., a 500-year event. The geotechnical analysis should be performed on the basis that all stream bed material in the scour prism above the total scour line for the scour design flood has been removed and is not available for bearing or lateral support with a safety factor of from 2 to 3. The geotechnical analysis for the superflood should incorporate a safety factor of 1.0. The FHWA publication Hydraulic Engineering Circular (HEC) No. 18, Scour at Bridges, discussed below, should be followed in conducting and documenting the results of scour evaluation studies.

EXISTING BRIDGES

All existing bridges over waterways with scourable beds should be evaluated for the risk of failure from scour during the occurrence of a 500-year flood with a geotechnical safety factor of 1.0.

The three basic elements of the TA relative to the NBIS Program for existing structures are 1) screen all existing bridges to prioritize the order of scour evaluations on the basis of scour susceptibility, 2) evaluate scour of all bridges with scourable beds to determine whether the bridges are scour critical, and 3) establish a plan of action for each identified scour critical bridge. State highway agencies have been requested by FHWA to have all existing bridges screened by March 1991. No date has been suggested for completion of the evaluation phase of the process. A scour critical bridge is defined as one that is in imminent danger of scour failure based on analyzed conditions, using state-of-the-art scour technology. The FHWA expects scour procedures set forth in HEC No. 18 to be used for all Federal-aid highway design and within the NBIS Program for evaluating existing bridges. A State highway agency can use other technology if it is judged by FHWA to be sound design practice. The scour procedures were initially issued as an attachment to the TA and entitled "Interim Procedures for Evaluating Scour at Bridges" in September 1988. A plan of action for each structure determined to be scour critical should include monitoring its scour performance with contingency for closure and/or scheduling timely design and construction of scour countermeasures. The HEC No. 18 scour procedures discussed below should be followed in conducting and documenting the results of scour evaluation studies.

HYDRAULIC ENGINEERING CIRCULAR NO. 18

These procedures contain the state-of-the-art technology for evaluating scour at highway bridges. Chapter 1 gives the background of the problem and the general state of knowledge. Basic concepts and definitions are presented in Chapter 2. Chapter 3 gives recommendations for designing bridges to resist scour. Chapter 4 presents equations for calculating scour depths within bridge waterways, at piers and abutments. Chapter 5 provides procedures for conducting scour analyses and evaluations at existing bridges. Chapter 6 presents guidelines for inspecting bridges for scour. Chapter 7 gives a plan of action for installing countermeasures to strengthen bridges that are considered vulnerable to scour.

CHAPTER 1 - INTRODUCTION

The most common cause of bridge failures is from floods, and the scouring of bridge foundations is the most common cause of damage to bridges during floods. Studies by FHWA in 1973 and 1978 indicated damage to bridges about equally distributed between piers and abutments. Transportation Research Record 950 contains a number of case histories of scour problems at bridges. About 85 percent of the 577,000 bridges in the National Bridge Inventory are over waterways. Some risk of scour failure must be accepted from future floods in this large target population since it is not economically feasible to install scour countermeasures at all sites. Every bridge over a scourable stream should, however, be assessed as to its vulnerability to scour.

It was well recognized when the TA was published in 1988 that the recommended scour technology lacked field verification. This is still true. While this technology represents the best available methodology, experience with the scour equations since that time clearly shows that further research and development is vital to the pursuit of more satisfactory scour solutions. This is being accomplished through monitoring of bridge sites by a number of State highway agencies and through research studies by FHWA and others. The U. S. Geological Survey has been retained by FHWA to evaluate the scour data from the individual state monitored field scour studies for the purpose of validating improvements in the scour prediction equations. Additional State highway agencies are encouraged to initiate Highway Planning and Research studies for obtaining field measurements of scour.

CHAPTER 2 - BASIC CONCEPTS AND DEFINITIONS OF SCOUR

Total scour at a highway crossing resulting solely from a bridge is comprised of two components. These are 1) contraction scour caused by the constriction of the waterway through the crossing and 2) local scour generated by the obstruction of piers and abutments. Degradation, aggradation and lateral migration in the reach of waterway adjacent to the crossing should also be considered. Scour produced by the bridge is determined by the procedures in HEC No. 18 while bed and bank movement unique to the adjacent channel reach can be evaluated from the FHWA publications HEC No. 20, Stream Stability at Highway Structures, and Highways in the River Environment. The concept of clear-water and live-bed scour is discussed in conjunction with bridge scour.

CHAPTER 3 - DESIGNING BRIDGES TO RESIST SCOUR

Hydraulic studies of bridge sites are a necessary part of the preliminary design of bridges. These studies should address both the sizing of the waterway area and foundation design to resist scour. The scope of the analysis is commensurate with the

importance of the highway and the consequences of failure. Since the best current technology is based on laboratory model studies, a designer must apply sound engineering judgement in comparing scour results with available hydraulic data to arrive at a practicable foundation design. Available hydraulic data include 1) performance of contiguous existing structures during past floods, 2) effects of regulation and control of flood discharges, and 3) the relative stability of the stream reach. A bridge hydraulic design procedure is presented which is to be accomplished for both the 100- and 500-year floods. Scour is part of the design procedure. A checklist of design considerations provides the practitioner with valuable insights to lessen the risk of failure.

CHAPTER 4 - ESTIMATING SCOUR AT BRIDGES

Before the various scour estimating methods for determining contraction and local scour can be applied, it is necessary to 1) obtain the rigid boundary channel hydraulics; 2) estimate the long-term stream bed changes; 3) adjust the rigid boundary channel hydraulics to reflect the stream bed elevation changes; and 4) compute the bridge hydraulics. A scour design approach is presented which is the basis for the scour portion of the overall bridge hydraulic design process documented in Chapter 3.

Contraction scour is computed by comparing the conditions at an actual bridge site to those in four typical bridge site model solutions and choosing the most appropriate. Abutment scour solutions have proved to be the least reliable of the scour solutions within HEC No. 18. The original Interim Procedures recommended use of a matrix of field site conditions with the potential use of seven differing solutions for abutment scour. HEC No. 18 permits consideration of a single solution with the alternative of using the seven equations recommended in 1988. Riprap protection of abutments is rather common design practice in the Nation. Should it be determined in the original design process or in an evaluation of an existing bridge that riprap will protect an abutment for the superflood, it may be unnecessary to determine abutment scour. A single pier scour equation is presented. Little refinement has evolved in plotting total scour depths based on the contraction, abutment and pier scour solutions other than summing the appropriate components. This chapter also contains sample solutions.

CHAPTER 5 - ESTIMATING THE VULNERABILITY OF EXISTING BRIDGES TO SCOUR

A prodigious amount of work is necessary to accomplish an evaluation of the 577,000 bridges on the National Bridge Inventory. Some State highway agencies are well into this endeavor while others are just getting started. The former group are commended for their diligent efforts. The others are encouraged to persevere. A recommended procedure for accomplishing this gigantic task is presented. The short-term goal is to evaluate existing bridges with known problems. The long-term goal is to evaluate all existing bridges. Background information on many bridges is lacking. Foundation type and depth may be unknown. Unless a bridge lacking foundation data has a visible scour problem, State highway agencies may set such structures aside for potential future evaluation. The FHWA is investigating a risk management approach as well as potential instrumentation for addressing this issue.

CHAPTER 6 - INSPECTION OF BRIDGES FOR SCOUR

The factors to be considered in scour evaluation require a broader scope of study and effort than those considered in a bridge inspection. Whereas the major purpose of bridge inspection is to identify changed conditions which

may reflect an existing or potential problem, a scour evaluation is an engineering assessment of what might possibly happen in the future and what steps can be taken now to eliminate or minimize future damage. The NBIS Program involves inspections on a 2-year cycle of the 577,000 bridges in the National Bridge Inventory. The FHWA December 1988 publication "The Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges" specifies the bridge and channel hydraulics and scour data that are evaluated and reported within the NBIS. Typically the inspections are accomplished by bridge engineers. Item 60, Substructure; Item 61, Channel and Channel Protection; Item 71, Waterway Adequacy; and Item 113, Scour Critical Bridges, are included. This chapter presents material covering bridge inspection for all of these items with emphasis on Item 113. Should an inspector observe conditions of an emergency or potentially hazardous nature, there must be a positive means of promptly communicating these findings to proper agency personnel. Hydraulic and geotechnical engineers should be informed immediately of existing or potential problems so action can be taken to further evaluate the bridge.

CHAPTER 7 - PLAN OF ACTION FOR INSTALLING SCOUR COUNTERMEASURES

Scour countermeasures are features incorporated into the original bridge design or at a later date to make a bridge less vulnerable to damage or failure. Certainly the best solution for original design is to set the foundation sufficiently below the total scour level that the potential for failure is minimized. Other enhancements to scour reduction or minimization during initial design are streamlining bridge elements to minimize obstructions to flow and locating a bridge to avoid adverse flood flow patterns. In developing a plan of action for an existing scour critical bridge, considerations are 1) monitoring the site with contingency for closure; 2) installing temporary countermeasures such as riprap with monitoring during high flows; 3) scheduling countermeasure construction; and 4) designing countermeasures. Typical countermeasures included are riprap, guide banks (spur dikes), and channel improvements. Design procedures or references are presented for the countermeasures.

STATUS OF THE BRIDGE SCOUR PROGRAM

The FHWA initiated biannual status reports on bridge scour in 1990. State highway agencies are reporting data on the numbers of bridges 1) over waterways; 2) screened as low risk, scour susceptible, and with unknown foundations; 3) analyzed for scour; 4) scour critical; 5) with countermeasures; and 6) monitoring planned. Through March 1990, 24 percent of the 475,731 bridges reported over waterways had been screened. Of those screened, 73,407 were assessed as safe, 23,057 were determined to be scour susceptible, and 17,174 had been identified with unknown foundations. About 8 percent of the scour susceptible bridges had been analyzed for scour. About 35 percent of the bridges analyzed were scour critical. This represents data from the State highway agencies' March 31, 1990 status reports. Consistently improved information will be forthcoming from subsequent reports. Some States are doing all of the scour evaluations in-house while others are contracting much of the work to consultants. As well, the U. S. Geological Survey is accomplishing some of the work for States.

SUMMARY

The FHWA and State highway agencies as well as other parties have conducted research for many years to improve the estimating procedures for determining scour at bridges. The National Cooperative Highway Research Program

Synthesis of Highway Practice No. 5, "Scour at Bridge Waterways," 1970, summarized technology available at that time which was a mixture of analytical and empirical methodology. Significant research work within this discipline has been accomplished since that time. Even so, the best available current technology represented by HEC No. 18 needs improvement. Ongoing and proposed research within the highway hydraulic community addresses scour needs and will result in improved methodology. The incorporation of scour within the overall bridge hydraulic design process is well on its way. Within the NBIS Program, the evaluation of scour for existing bridges with acknowledged scour problems is also well on its way. The evaluation of a large percentage of existing bridges is, however, just getting underway. This endeavor will require the training of additional hydraulic engineers, professional dedication in the identification and treatment of scour critical bridges, and a greater commitment of resources by State highway agencies in its accomplishment.

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Nationwide Estimation of Extreme Floods for Bridge-Scour Analysis

W. O. THOMAS, JR., W. H. KIRBY, J. B. ATKINS, AND M. E. JENNINGS

Procedures are described for estimating flood-peak discharges and flood hydrographs for extreme floods, such as the 500-year flood, to evaluate scour at bridges. These procedures are being incorporated (June 1990) in a microcomputer program called "National Flood Frequency" developed by the U.S. Geological Survey in cooperation with the Federal Highway Administration and the Federal Emergency Management Agency. U.S. Geological Survey flood reports typically describe procedures for estimating flood-peak discharges for return periods of 2 to 100 years for rural and urban ungaged watersheds. This paper describes procedures in the National Flood Frequency program for extrapolating design floods with return periods of 2 to 100 years to the 500-year flood.

INTRODUCTION

The planning and design of bridges requires the estimation of extreme flood discharges to facilitate an adequate and safe design. Recent bridge failures, resulting from excessive scour, have prompted the Federal Highway Administration (FHWA) to develop procedures for evaluating scour at bridges. As part of this program, the FHWA advised the State Departments of Transportation (DOT's) nationwide to evaluate the risk of their bridges being subjected to scour damage during floods on the order of a 100- to 500-year or greater average return period. In response, State DOT's are developing and implementing improved procedures for evaluating the magnitude of scour during extreme floods and for designing bridges to minimize scour damage. Essential to these improved procedures is a consistent, unbiased, and easy to apply technique for estimating discharges of floods having return periods on the order of 100 to 500 years.

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The purpose of this paper is to describe a procedure, applicable nationwide, for estimating extreme floods that will meet the design needs of FHWA and State DOT's. The U.S. Geological Survey (USGS), in cooperation with FHWA and the Federal Emergency Management Agency (FEMA), is developing (June 1990) a National Flood Frequency (NFF) microcomputer program for estimating floodpeak magnitudes of T-year events (2 to 500 years) and for estimating a typical hydrograph associated with such events for ungaged rural and urban sites in most areas of the United States. Jennings and Cookmeyer *(1)* provide an overview of the plans for NFF. The flood-peak estimation techniques are a compilation of regional regression equations developed by USGS (primarily in cooperation with State DOT's)) over the last 15 years or so. The flood hydrograph estimation technique is based on unit-hydrograph theory and utilizes a dimensionless hydrograph that is defined by the flood-peak discharge and the watershed lagtime. This paper primarily is concerned with describing an extrapolation procedure for estimating the 500-year flood for bridge-scour evaluation.

ESTIMATING FLOOD DISCHARGES

The USGS has developed regional regression equations for estimating floods with selected recurrence intervals for rural watersheds for every State and Puerto Rico. In some areas of the Nation, however, data are inadequate to define flood-frequency characteristics. Jennings and Cookmeyer *(1)* report that the NFF microcomputer program will include about 1500 equations for 214 flood regions in the United States. USGS reports typically give regression equations for estimating the 2-, 5-, 10-, 25-, 50- and 100-year flood-peak discharges for an average of about four flood regions in each State. Only in a few States has USGS published

regression equations for the 500-year flood; hence the need for an extrapolation procedure. The regional regression equations are developed by relating the T-year flood-peak discharges at gaging stations to watershed and climatic characteristics by means of multiple regression equations. (The T-year discharges at gaging stations are determined by following guidelines adopted by the Interagency Advisory Committee on Water Data, *(2)*). Estimates at ungaged sites then can be computed by using the appropriate watershed and climatic characteristics which usually are available from topographic maps and climatic reports.

Regression equations for estimating flood-frequency characteristics for urban watersheds have been published in 18 States. In some instances, these equations are applicable statewide, in other instances, the equations are applicable only for one metropolitan area. These equations are based on watershed and climatic characteristics as well as on urban characteristics, such as the percentage of the watershed that is impervious, or a measure of the state of development of the drainage system, such as the basin development factor described by Sauer and others *(3)*. For those States without urban regression equations, a nationwide technique, developed by Sauer and others *(3)*, is used to adjust the rural frequency curve to urban conditions. Thomas *(4)* provides a listing by State of all USGS published reports (as of December 1986) for estimating rural and urban flood-frequency characteristics.

EXTRAPOLATION FOR EXTREME FLOODS

To date, the USGS has published regression equations for estimating the 500-year flood in only seven States. A procedure will be given in the NFF program for extrapolating the regional regression equations in any State to the 500-year flood. Basically, the extrapolation procedure consists of fitting a log-Pearson Type III curve to the 2- to 100-year flood discharges given by NFF and extrapolating this curve to the 500-year flood discharge. The procedure consists of the following steps for a given watershed:

1. Determine the flood-peak discharges for selected return periods from the appropriate regional regression equations given in NFF. At least three points are needed to define the skew coefficient required in a subsequent step. Use of additional points improves the definition of the frequency curve that is defined by the regional equations and helps to average out any minor irregularities that may exist in the relations among the regional equations. The NFF program will use all available regional equations to define the frequency curve.

2. Fit a quadratic curve to the selected points on log-probability paper using multiple regression analysis. The variables used in the regression analysis are the logarithms of the selected discharges and the standard normal deviates associated with the corresponding probabilities. The purpose of this quadratic curve is to obtain a smooth curve through the selected flood-peak discharges from step 1 above. The quadratic curve is an approximation of the log-Pearson Type III curve that will be computed.

3. Determine the skew coefficient of the log-Pearson Type III frequency curve that passes through the 2-, 10-, and 100-year floods defined by the quadratic curve. The skew coefficient is defined approximately by the formula (IACWD, *2*)

$$G = -2.50 + 3.12 \log (Q_{100}/Q_{10}) / \log (Q_{10}/Q_2).$$

4. Replot (conceptually) the selected discharges and return periods using a Pearson Type III probability scale defined such that a frequency curve with the computed skew plots as a straight line. This scale is defined by plotting probability values p at positions x on the probability axis, where x is defined by the standardized Pearson Type III deviate (K values) for the given skew and probability. A Wilson-Hilferty approximation (Kirby, *5*) is used to compute the K value.

5. Fit a straight line by least squares regression to the points plotted in step 4 and extrapolate this line to the 500-year flood-peak discharge. The variables used in the regression analysis are the logarithms of the selected discharges and the Pearson Type III K values associated with the corresponding probabilities.

Figure 1 is an example of a flood-frequency curve provided by this procedure for the Fenholloway River near Foley, Florida. The solid triangles shown in figure 1 are the regional flood-frequency values as estimated by the equations given in Bridges *(6)*, which will be incorporated in the NFF program. The 500-year value shown as a solid circle in figure 1 (12,800 ft^3/s) is estimated using the extrapolation procedure described above. Note that the extrapolated 500-year value is a reasonable extension (see dotted line) of the regional frequency curve.

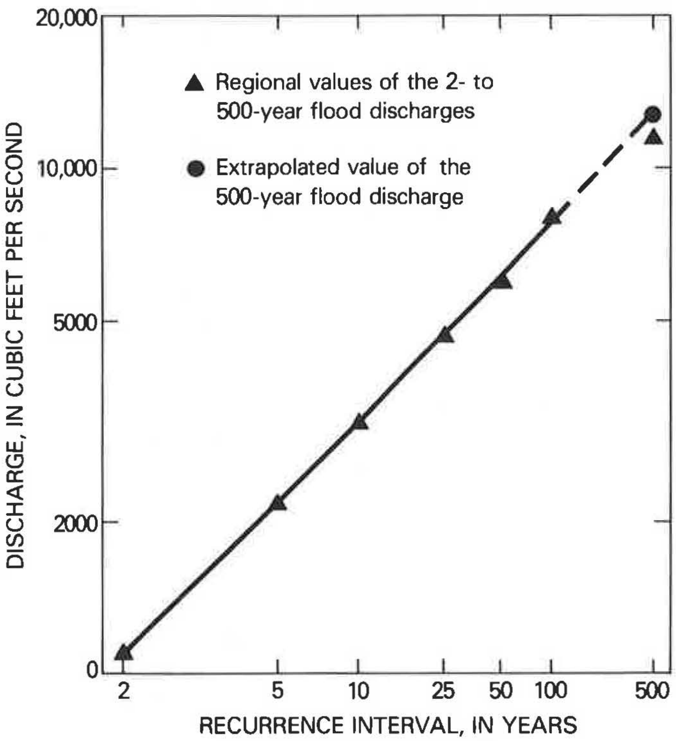

Figure 1. Regional flood-frequency curve for the Fenholloway River near Foley, Florida

The solid triangle shown in figure 1 (11,500 ft^3/s) for the 500-year value is the regional value as obtained directly from the 500-year equation given in Bridges *(6)*. The 500-year flood for the Fenholloway River can be estimated without extrapolation since Florida is one of the seven States for which 500-year regression equations have been published. The difference between the two 500-year values is 11.3 percent. This is typical of several comparisons of extrapolated 500-year floods to published regional equations that have indicated most results agree within plus or minus 15 percent.

Figure 2 is a summary of the input data, questions, and responses that created the frequency curve in figure 1. The information required to compute the regional flood-frequency curve in figure 1 is that the watershed is in Region B with a drainage area of 120 square miles and that 0.37 percent of the watershed area is covered by lakes. Figure 2 shows the regional flood frequency values and their associated standard errors.

For comparison and evaluation, the NFF program will compare each extrapolated 500-year flood-peak discharge with the maximum flood-envelope curves given by Crippen *(7)*. These flood-envelope curves were developed by plotting maximum known flood discharges against drainage areas for 17 flood regions of the United States. Therefore, these flood-envelope curves approximate the maximum flood-peak discharge that has been regionally experienced for a given size watershed. Since there is no frequency of occurrence associated with the envelope-curve estimates, the comparison of these values to the extrapolated 500-year flood is merely a qualitative evaluation. In general, one would expect the extrapolated 500-year flood-peak discharge to be less than the envelope-curve values, assuming that several watersheds in a given region have experienced at least one flood exceeding the 500-year value during the period of data collection. For the Fenholloway River near Foley, Florida, the 500-year flood estimates range from 11,500 to 12,800 ft^3/s. The envelope-curve value from Crippen *(7)* is 101,000 ft^3/s given that the watershed is in region 3 (this information must be provided by the analyst) as defined by Crippen *(7)*.

National Flood Frequency Log Session

```
NFF Log session started 07/03/1990 09:04
Enter state id code : FL
Enter name of basin under study: FENHOLLOWAY RIVER NEAR FOLEY, FL.
List of Hydrologic Regions
in Florida
Region
Number Region Name
==============================
   1   Region A
   2   Region B
   3   Region C
Is basin contained in more than one
hydrologic region? (Y/N) N
Hydrologic region? (1-3) : 2
Region B parameters:
Drainage Area (sq mi), DA (13.90-9640.0) : 120.0
Lake Area (%), LK (0.00-13.3) : 0.37
Enter maximum flood region within which the basin is contained (See Report).
Enter 0 if not applicable (e.g. outside of conterminous United States) : 3
Table of rural flood event values
FENHOLLOWAY RIVER NEAR FOLEY, FL.
  Recurrence  Peak,               Eq. Yrs.
| Inter.,yrs   cfs    % Std. Err.  Record
|===========================================
| RQ2          1050       60.9         2
| RQ5          2170       59.7         3
| RQ10         3150       59.9         3
| RQ25         4650       60.9         5
| RQ50         5980       61.9         5
| RQ100        8000       63.1         6
| RQ500       12800
============================================
      MAXIMUM FLOOD ENVELOPE = 101000 cfs
  List of Entered Parameters
      Name        Value
==============================
             DA : 120.000
             LK : 0.370
Do you want to calculate a weighted average of observed and regression
estimates? (Y/N) N
Do you want to perform urban calculations? (Y/N) N
Do you want to write a flood frequency plot input file for TELAGRAF? (Y/N) N
Do you want to compute a hydrograph for the rural peak calculated? (Y/N) N
Do you want to do more flood frequency calculations in Florida? (Y/N) N
Do you want to do flood frequency calculations in another state? (Y/N) N
Program terminated.
NFF Log session ended 07/03/1990 09:05
```

Figure 2.-- Summary of input data, questions and responses during an interactive session with the National Flood Frequency Program

FLOOD-HYDROGRAPH ESTIMATION

Procedures also will be provided in NFF for computing a flood hydrograph associated with the 500-year flood-peak discharge and floods of other return periods. The flood hydrographs produced by these procedures may be applicable only in certain parts of the country. These computations are based on procedures described by Inman *(8)* and are only briefly summarized here. Using these procedures, a dimensionless hydrograph can be converted to a typical T-year hydrograph by estimating the T-year flood-peak discharge and watershed lagtime (time from centroid of rainfall excess to centroid of runoff). The flood hydrograph that results from this procedure is an average or typical hydrograph that corresponds to a given T-year flood-peak discharge. The volume under the hydrograph may, or may not, have the same recurrence interval as the peak discharge.

Briefly, the procedures for estimating flood hydrographs, as described by Inman *(8)*, involves the following steps. Rainfall-runoff data are used to determine unit hydrographs and watershed lagtimes for selected watersheds in a region. These unit hydrographs are averaged to obtain an average unit hydrograph for the region or State. This average unit hydrograph is converted to an average dimensionless hydrograph by dividing the ordinates by the flood-peak discharge and the abscissa by the watershed lagtime. A typical T-year hydrograph is then obtained by multiplying the ordinates of the dimensionless hydrograph by the T-year peak discharge and multiplying the abscissa by the watershed lagtime. The flood-peak discharge will be estimated by regression equations in the NFF program. The watershed lagtime for urban watersheds will be estimated by regression equations in the NFF program (Sauer and others *(3)*). For rural watersheds, the values for watershed lagtime will be input by the user. The computed T-year flood hydrograph can be used along with the peak discharge to determine bridge scour, flood-hazard risk, and other environmental impacts.

Figure 3 is an example of a typical flood hydrograph for a 500-year peak discharge of 12,800 ft^3/s for the Fenholloway River near Foley, Florida. Flood hydrographs, such as the one shown in figure 3, can be used to determine the duration that flow is above a given discharge. This information should be useful in evaluating scour at bridges.

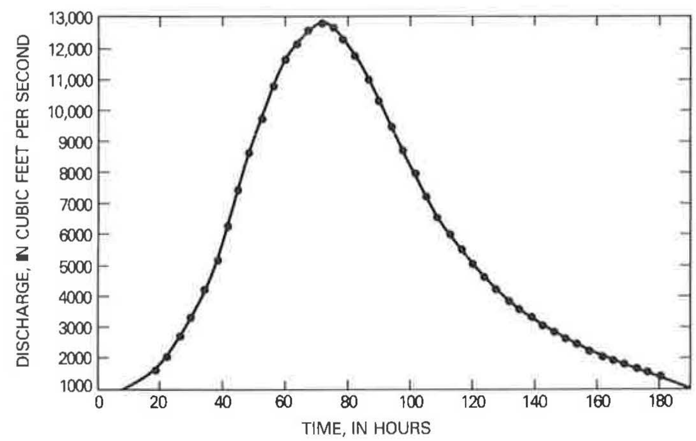

Figure 3. Typical flood hydrograph associated with a 500-year flood-peak discharge for the Fenholloway River near Foley, Florida.

SUMMARY

The procedures to be provided in the NFF microcomputer program give the highway engineer relatively easy and straightforward methods of estimating the flood-peak discharge and flood hydrograph of extreme floods, such as the 500-year flood. The rural flood discharge estimates are based on regression equations that have been developed by the USGS over the last 15 years or so. The equations to be given in the NFF program will be updated periodically as new equations are developed. Procedures will also be available for estimating flood-peak discharges for urban watersheds. If Statewide urban flood-frequency procedures are not available, then nationwide techniques, described by Sauer and others *(3)*, can be used. Finally, a flood hydrograph, associated with the T-year rural or urban flood discharge, can also be computed to aid in the determination of scour during extreme floods.

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Upper Confidence Limit of Local Pier-Scour Predictions

DAVID C. FROEHLICH

Eighty-three on-site measurements of local scour at bridge piers were gathered from both published and unpublished sources and analyzed using multiple regression analysis to obtain local scour prediction equations. This investigation differs from most previous studies of scour at bridge piers because only on-site measurements were used. The regression analysis provides a probability statement about the distribution of the prediction, from which an upper confidence limit of the depth of local scour can be determined. An upper confidence limit having an appropriate exceedance probability can be used for design of bridge pier foundations to provide a desired factor of safety.

INTRODUCTION

Exposure or undermining of bridge-pier foundations from the erosive action of flowing water can result in structural failure of a bridge, requiring a major expenditure for repair or replacement (see Davis, 1984 for case histories of scour problems at several bridges). Therefore, bridge-pier foundations need to be designed for the maximum depth of scour that is likely to occur for a specified design condition. Spread footings, drilled pier foundations, and caisson foundations need to be placed well below the estimated depth of scour; and piles piers need to be long enough to support the structure after scour has occurred.

The term "scour" is used here to mean a lowering by erosion of a stream-bed below an assumed natural level or other appropriate datum. "Depth of scour", or "scour depth", refers to the depth of material removed below the selected datum. Scour is of concern primarily at bridge crossings on alluvial streams, but stream-bed erosion occurs in nearly all waterways.

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Scour at a bridge waterway is usually separated into three components: (1) General scour, (2) constriction scour, and (3) local scour. General scour is the progressive aggradation or degradation of the stream channel caused by changes in channel controls (such as a dam construction or gravel mining), changes in sediment supply, or changes in stream form (such as a change from a meandering to a braided stream). Constriction scour is caused by the reduction of waterway area by bridge embankments, piers, and other obstructions, that increase flow velocities within and near a stream channel generally for a short distance upstream of and downstream of the bridge crossing. Local scour is caused by flow disturbances created by piers and abutments, resulting in an abrupt decrease in stream-bed elevation in the immediate vicinity of these structure.

The phenomena of local scour at a bridge pier has been studied often, and numerous mathematical and graphical relations for predicting depth of local scour at a bridge pier have been developed. Most of these relations are based entirely on experimental data collected in small-scale laboratory flumes and provide widely varying predictions for a given set of pier, approach flow, and bed material characteristics (see Jones, 1984, for a comparison of several local pier scour prediction methods).

Departing from the experimental approach, 83 on-site measurements of local scour at bridge piers were assembled from published sources, and local pier-scour prediction equations for live-bed scour conditions were developed from multiple regression analysis of these data. The regression analysis also provides a probability statement about the distribution of the prediction, from which an upper confidence limit of the predicted depth of local scour can be determined. An upper confidence limit having an appropriate exceedance probability can be used for design of bridge pier foundations to provide a desired factor of safety.

MECHANISM OF LOCAL SCOUR

Local scour is classified as clear-water or live-bed depending on whether or not sediment is supplied to the scour hole by the approaching flow. Clear-water scour occurs when the approach flow is not capable of transporting sediment to the scour hole. The maximum depth of local scour for clear-water flow is attained when the capacity for transport of material out of the scour hole is reduced to zero (that is, when an equilibrium condition is reached). Live-bed scour occurs when material is continuously supplied to a scour hole. Because of the movement of bed forms in a stream channel when live-bed flow exists, the lowest point in a scour hole will oscillate periodically about an average value. The equilibrium scour depth for live-bed conditions is defined as the distance between the temporal averages of the ambient stream-bed elevation and the elevation of the lowest point in the scour hole.

Piers are classified as blunt-nosed or sharp-nosed depending on their geometry. Blunt-nosed piers have square or rounded upstream ends; sharp-nosed piers have a tapered upstream end that terminates at a sharp edge. Scour around a blunt-nosed pier is initiated by a downward flow along the upstream face of the pier caused by a vertical pressure gradient that develops as approaching flow impinges on the nose of the pier. Weaker downward flows will occur along the tapered sides of a sharp-nosed pier. As flow is deflected downward and around a pier, velocities near the bed are increased and large-scale eddy structures (vortices) are formed (Laursen and Toch, 1956; Roper and others, 1967; Melville, 1975; and Dargahi, 1987). Local scour at a pier is caused by the combined action of vortices and accelerated flow around the pier.

ON-SITE MEASUREMENTS

On-site measurements of local scour at bridge piers were assembled from available published and unpublished sources and are summarized in Tables 1 and 2. The depth of local scour, d_s, is defined as the distance from the lowest point in the scour hole to the estimated elevation that the stream-bed would have assumed if the pier was removed. Scour holes were measured using a variety of devices including sonic depth meters, sounding weights, and sounding rods. Each measurement was made during a sustained high flow, and is assumed to represent an equilibrium condition.

Variables governing local scour at a pier consist of those that describe the pier, the approach flow, and the streambed material. Pier data consist of a pier shape code (1 = a square-nosed pier, 2 = a round-nosed pier, and 3 = a sharp-nosed pier); the pier width, b; and the pier length, ℓ. Approach flow data consist of the flow depth, y_a; the depth-averaged flow velocity, V_a; and the angle at which the flow approaches the pier, α (α equals zero if the pier is aligned with the approaching flow). Bed material variables consist of the median particle diameter, d_{50}; and the geometric standard deviation, $\sigma_g = \sqrt{d_{84}/d_{16}}$; where d_i is the intermediate diameter of a particle for which i percent (by weight) of the material is finer. Reported bed material characteristics are assumed to represent the material forming the bed of the approaching channel and the bed surrounding the pier.

ANALYSIS OF DATA

Using dimensional analysis, the variables governing local scour at a bridge pier were combined to form the following functional relation for relative depth of local scour:

$$\frac{d_s}{b} = f\left\{\phi, \frac{b'}{b}, \frac{y_a}{b}, F_a, R_p, S_s, \frac{b}{d_{50}}, \sigma_g\right\} \quad (1)$$

where ϕ is a dimensionless pier-shape factor, $b' = b \cos \alpha + \ell \sin \alpha$ is the pier width projected normal to the approach flow, $F_a = V_a/\sqrt{gy_a}$ is the Froude number of the approach flow, $R_p = V_a b/\nu$ is the Reynolds number based on approach-flow velocity and pier width, and $S_s = \rho_s/\rho$ is the specific gravity of the bed material.

Because viscosity (or water temperature), specific gravity of bed material, and bed material gradation were rarely reported for the on-site measurements, the variables R_p, S_s, and σ_g were eliminated from the analysis. However, the influence of water viscosity, bed material density, or bed material gradation on local scour at a bridge pier may be significant. Shen and others (1969)

TABLE 1. SUMMARY OF ON-SITE MEASUREMENTS OF LOCAL SCOUR AT BRIDGE PIERS: REFERENCES, LOCATION, MEASUREMENT NUMBER, DATE, AND COMMENTS

Reference(s) and Location of Measurement	Measurement Number	Date of Measurement[a]	Comments
Bata and Todorovic (1960): Danube River Bridge at Novi Sad, Yugoslavia	1	--/--/26[b]	
	2	--/--/58	
Neill (1965): Beaver River LaCorey Bridge, Alberta, Canada	3	16/19/62	center pier
Breusers (1970), and Breusers and others (1977): Niger River Onitsha Bridge	4	--/--/--	
Melville (1975): Waikato River Tuakau Bridge, New Zealand	5	08/15/58	
Norman (1975): Susitna River Anchorage-Fairbanks Hwy. Bridge near Sunshine, Alaska	6	07/02/71	pier 1
	7	"	pier 2
	8	"	pier 3
	9	08/11/71	pier 1
	10	"	pier 2
	11	"	pier 3
Knik River Bridge near Palmer, Alaska	12	07/11/65	pier 5
Knik River Bridge near Eklutna, Alaska	13	06/17/66	pier 3
	14	"	pier 4
	15	"	pier 5
	16	"	pier 6
	17	"	pier 7
	18	06/24/66	pier 1
	19	"	pier 2
	20	"	pier 3
	21	"	pier 4, exposed foundation
	22	"	pier 5
	23	"	pier 6
	24	"	pier 7
	25	06/28/66	pier 1
	26	"	pier 2
	27	"	pier 3
	28	"	pier 4, exposed foundation
	29	"	pier 5
	30	"	pier 6
	31	"	pier 7

TABLE 1. SUMMARY OF ON-SITE MEASUREMENTS OF LOCAL SCOUR AT BRIDGE PIERS: REFERENCES, LOCATION, MEASUREMENT NUMBER, DATE, AND COMMENTS (continued)

Reference(s) and Location of Measurement	Measurement Number	Date of Measurement[a]	Comments
Tazlina River	32	04/22/69	
Richardson Hwy. Bridge	33	09/02/71	
near Glennallen, Alaska	34	09/04/71	
	35	10/01/71	
Tanana River	36	07/16/71	pier 1
Richardson Hwy. Bridge	37	"	pier 2
at Big Delta, Alaska	38	"	pier 3
	39	"	pier 4
Tanana River Anchorage-Fairbanks Hwy. Bridge at Nenana, Alaska	40	07/30/67	
Snow River Seward Hwy. Bridge near Seward, Alaska	41	09/22/70	
Chang (1980):			
Red River Hwy. 6 Bridge	42	12/27/77	
near Grand Encore, La.	43	06/16/78	
Red River Hwy. 8 Bridge	44	06/28/77	
near Boyce, La.	45	06/06/78	
Red River Hwy. 3026 Bridge	46	06/06/77	
near Alexandria, La.	47	11/21/77	
	48	06/19/78	
Atchafalaya River	49	07/14/77	
Hwy. 190 Bridge near	50	"	
Krotz Springs, La.	51	02/29/77	
	52	"	
	53	07/12/78	
	54	"	
Mississippi River Hwy. 65 Bridge at Natchez, Miss.	55	09/07/77	
Mississippi River Hwy. 190 Bridge at Baton Rouge, La.	56	06/14/77	
Hopkins and others (1980):			
Red River Texas St. Bridge Bridge at Shreveport, La.	57	12/18/72	

TABLE 1. SUMMARY OF ON-SITE MEASUREMENTS OF LOCAL SCOUR AT BRIDGE PIERS: REFERENCES, LOCATION, MEASUREMENT NUMBER, DATE, AND COMMENTS (continued)

Reference(s) and Location of Measurement	Measurement Number	Date of Measurement[a]	Comments
Davoren (1985):			
Oahu River near	58	09/20/82	Cylindrical pier
Twizel, New Zealand	59	09/22/82	constructed specifically
	60	06/10/82	to study local scour
	61	01/19/82	
	62	01/20/82	
	63	08/09/82	
	64	07/04/82	
	65	07/05/82	
	66	08/11/82	
	67	07/15/82	
	68	04/28/82	
Jarrett and Boyle (1986), and Jarrett (written commun., 1986):			
South Platte River	69	10/02/84	pier 5
County Rd. 87 Bridge	70	"	pier 6
at Masters, Colo.	71	"	pier 7
	72	06/25/84	pier 5
	73	"	pier 6
	74	"	pier 7
	75	05/18/84	pier 5
Arkansas River	76	05/23/84	pier 1
County Rd. 613 Bridge	77	"	pier 3
near Nepasta, Colo.	78	06/05/84	pier 1
	79	07/27/84	pier 1
Rio Grande Hwy. 285 Bridge	80	05/22/84	
at Monte Vista, Colo.	81	06/01/84	
South Platte River Hwy. 37	82	05/21/84	pier 1
Bridge near Kersey, Colo.	83	06/26/84	pier 1

[a] Date given as mm/dd/yy.
[b] Information not available.

TABLE 2. SUMMARY OF ON-SITE MEASUREMENTS OF LOCAL SCOUR AT BRIDGE PIERS: PIER DATA, APPROACH FLOW DATA, AND BED MATERIAL DATA

Measurement number	Depth of local scour, in meters	Pier data			Approach flow data			Bed material data	
		Shape code[a]	Width, in meters	Length, in meters	Depth, in meters	Velocity, meters per second	Angle, in degrees	Median diameter, in millimeters	Geometric standard deviation
1	4.30	2	4.50	14.00	18.80	1.84	0	0.25	2.24
2	3.00	2	4.50	14.00	17.40	2.28	0	0.25	2.24
3	1.74	2	1.92	17.37	5.39	1.80	5	0.50	--[b]
4	7.80	2	8.50	8.50	9.00	0.65	12	0.67	--
5	2.75	1	2.40	8.85	3.45	0.96	10	0.78	2.30
6	0.76	3	1.52	6.10	5.80	1.98	0	70	--
7	0.76	3	1.52	6.10	4.10	2.59	0	70	--
8	0.61	3	1.52	6.10	3.40	2.13	0	70	--
9	0.61	3	1.52	6.10	5.30	3.05	0	70	--
10	0.61	3	1.52	6.10	6.60	2.90	0	70	--
11	0.61	3	1.52	6.10	5.20	3.51	0	70	--
12	0.82	3	1.80	9.60	5.50	3.67	0	1.5	--
13	0.30	2	1.52	11.58	1.20	0.49	0	0.5	--
14	0.30	2	1.52	11.58	1.50	0.76	0	0.5	--
15	0.30	2	1.52	11.58	1.20	0.88	0	0.5	--
16	0.76	2	1.52	11.58	0.50	0.27	0	0.5	--
17	1.22	2	1.52	11.58	0.60	0.15	0	0.5	--
18	0.61	2	1.52	11.58	2.10	1.52	0	1.6	--
19	0.61	2	1.52	11.58	2.00	1.55	0	1.6	--
20	0.91	2	1.52	11.58	3.00	1.58	0	1.6	--
21	1.22	2	1.52	11.58	3.20	1.98	0	1.6	--
22	1.37	2	1.52	11.58	3.00	1.80	0	1.6	--
23	1.07	2	1.52	11.58	2.60	2.07	0	1.6	--
24	1.83	2	1.52	11.58	3.00	1.83	0	1.6	--
25	0.46	2	1.52	11.58	0.90	0.94	0	1.6	--
26	0.61	2	1.52	11.58	0.90	0.98	0	1.6	--
27	0.46	2	1.52	11.58	1.80	1.10	0	1.6	--
28	0.61	2	1.52	11.58	2.40	1.16	0	1.6	--
29	0.76	2	1.52	11.58	2.30	1.13	0	1.6	--
30	0.46	2	1.52	11.58	1.50	1.13	0	1.6	--
31	0.76	2	1.52	11.58	2.00	0.98	0	1.6	--
32	0.60	3	4.60	--	0.60	--	0	90	--
33	1.50	3	4.60	--	3.70	2.90	0	90	--
34	1.70	3	4.60	--	4.60	3.51	0	90	--
35	0.90	3	4.60	--	1.50	0.61	0	90	--
36	1.80	2	1.52	10.36	3.70	2.16	35	14	--
37	2.10	2	1.52	10.36	3.70	2.22	35	14	--
38	1.80	2	1.52	10.36	4.60	2.07	35	14	--
39	2.40	2	1.52	13.56	4.30	1.74	35	14	--
40	1.80	2	3.05	17.60	6.70	2.59	0	15	--
41	0.90	2	0.98	0.98	1.70	1.61	0	8.0	--
42	4.00	2	4.90	12.80	1.80	--	0	0.053	--
43	4.60	2	4.90	12.80	4.60	--	0	0.053	--
44	3.70	2	8.20	8.20	4.90	0.46	0	0.060	--
45	4.30	2	8.20	8.20	4.30	0.61	0	0.060	--

TABLE 2. SUMMARY OF ON-SITE MEASUREMENTS OF LOCAL SCOUR AT BRIDGE PIERS: PIER DATA, APPROACH FLOW DATA, AND BED MATERIAL DATA (continued)

Measurement number	Depth of local scour, in meters	Pier data			Approach flow data			Bed material data	
		Shape code[a]	Width, in meters	Length, in meters	Depth, in meters	Velocity, meters per second	Angle, in degrees	Median diameter, in millimeters	Geometric standard deviation
46	7.30	2	13.00	38.00	4.10	0.55	5	0.027	--
47	6.80	2	13.00	38.00	3.40	0.66	15	0.027	--
48	8.50	2	13.00	38.00	5.40	1.16	20	0.027	--
49	4.30	3	9.80	12.50	11.00	0.73	5	0.008	--
50	8.20	3	9.80	12.50	12.80	0.81	30	0.008	--
51	4.60	3	9.80	12.50	13.60	1.08	15	0.008	--
52	7.90	3	9.80	12.50	16.30	1.22	25	0.008	--
53	4.00	3	9.80	12.50	11.60	0.82	15	0.008	--
54	7.60	3	9.80	12.50	13.40	0.91	25	0.008	--
55	6.10	1	9.40	19.50	19.50	1.80	0	0.036	--
56	10.40	2	19.50	38.00	11.30	0.66	15	0.036	--
57	2.80	2	3.66	17.30	3.60	0.64	0	0.100	--
58	1.30	2	1.50	1.50	3.10	2.38	0	20	--
59	1.30	2	1.50	1.50	3.00	2.69	0	20	--
60	0.80	2	1.50	1.50	2.50	2.54	0	20	--
61	0.90	2	1.50	1.50	1.40	2.65	0	20	--
62	0.90	2	1.50	1.50	1.30	2.43	0	20	--
63	0.40	2	1.50	1.50	1.30	2.68	0	20	--
64	0.40	2	1.50	1.50	1.00	2.39	0	20	--
65	0.50	2	1.50	1.50	0.90	2.33	0	20	--
66	0.40	2	1.50	1.50	0.90	2.56	0	20	--
67	0.40	2	1.50	1.50	0.70	2.24	0	20	--
68	0.40	2	1.50	1.50	0.60	--	0	20	--
69	0.61	1	0.29	3.66	0.76	1.04	15	1.5	--
70	0.61	1	0.29	3.66	0.61	1.36	15	1.5	--
71	0.52	1	0.29	3.66	0.73	1.17	15	1.5	--
72	0.58	1	0.29	3.66	0.43	1.13	10	2.3	--
73	0.46	1	0.29	3.66	0.58	1.02	10	2.3	--
74	0.49	1	0.29	3.66	0.70	1.12	10	2.3	--
75	0.66	1	0.29	3.66	1.81	1.22	15	2.3	--
76	0.64	2	1.22	6.40	2.13	1.17	0	0.6	--
77	0.40	2	1.22	6.40	0.55	0.69	0	0.6	--
78	1.22	2	1.22	6.40	2.32	1.70	0	0.6	--
79	0.61	2	1.22	6.40	0.70	0.66	0	0.6	--
80	0.37	3	0.94	27.43	1.40	1.54	0	7.9	--
81	0.15	3	0.94	27.43	1.22	1.35	0	4.3	--
82	0.98	3	0.52	8.29	3.21	1.68	10	1.2	--
83	0.65	3	0.52	8.29	2.14	1.17	10	1.8	--

[a]Pier shape code defined as follows: 1 = square-nosed pier, 2 = round-nosed pier, 3 = sharp-nosed pier.
[b]Information not available.

present a prediction equation that contains R_p, and conclude that the influence of viscosity on relative depth of local scour is significant. Raudkivi (1986) and Chiew (1989), on the basis of analyses of experimental data, conclude that bed material gradation has a significant influence on local scour depth because of the ability of a graded sediment to form an armour layer in the scour hole and on the surrounding streambed. Future on-site measurements of local scour at bridge piers need to include both temperature and bed-material gradation data.

Multiple linear regression was used to develop prediction equations for relative depth of local scour, based on the proposed functional relation in equation 1. A multiple regression model is written in matrix notation as

$$\mathbf{y} = \mathbf{X}\beta + \varepsilon \tag{2}$$

where $\mathbf{y}$ is an (n×1) vector of the observations, $\mathbf{X}$ is an (n×p) matrix of (p - 1) independent variables augmented by a column of ones, β is a (p×1) vector of the regression coefficients to be determined, and ε is an (n×1) vector of random errors. The least-squares estimator of β is

$$\hat{\beta} = (\mathbf{X}'\mathbf{X})^{-1}\mathbf{X}'\mathbf{y}. \tag{3}$$

The vector of fitted values corresponding to a vector of observed values is computed as

$$\begin{aligned}\hat{\mathbf{y}} &= \mathbf{X}\hat{\beta} \\ &= \mathbf{X}(\mathbf{X}'\mathbf{X})^{-1}\mathbf{X}'\mathbf{y} \\ &= \mathbf{H}\mathbf{y}\end{aligned} \tag{4}$$

where the (n×n) matrix $\mathbf{H} = \mathbf{X}(\mathbf{X}'\mathbf{X})^{-1}\mathbf{X}'$ is called the "hat" matrix. The hat matrix and its properties play a central role in regression analysis.

The effect of pier shape was evaluated by defining two indicator variables (Draper and Smith, 1981, p. 241), Z_1 and Z_2, as follows:

$$Z_1 = \begin{cases} 1, & \text{if the pier has a square-nose} \\ 0, & \text{otherwise} \end{cases}$$

$$Z_2 = \begin{cases} 1, & \text{if the pier has a sharp-nose} \\ 0, & \text{otherwise} \end{cases}$$

Hence, $(Z_1, Z_2) = (1, 0)$ for a square-nosed pier; $(Z_1, Z_2) = (0, 1)$ for a sharp-nosed pier; or $(Z_1, Z_2) = (0, 0)$ for a round-nosed pier.

Only live-bed scour measurements were considered in the analysis. Measurements were classified as live-bed scour if the approach flow velocity was greater than a critical velocity, V_c, calculated using the relation developed by Neill (1968) for coarse uniform bed material,

$$V_c = 1.58\sqrt{(S_s - 1)gd_{50}}\left(\frac{y_a}{d_{50}}\right)^{1/6}, \tag{5}$$

where S_s was assumed to equal 2.65 for all measurements. Sixty-six measurements were classified as live-bed scour.

Logarithmic transformation of all variables except the indicator variables was found to provide the best linear relations. The logarithm of relative depth of local scour was regressed against every possible combination of the seven independent variables. The best two, three, four, five, and six-variable regression equations, chosen on the basis of Mallow's C_p statistic, are summarized in Table 3. The two indicator variables, Z_1 and Z_2, were retained in all of the models because experiments have shown a definite dependence of local scour depth on pier shape (Breusers and others, 1977). During preliminary analyses, measurement number 55 was found to exert a large influence on the regression models and was determined to be an "outlier". It was deleted from the data set for final analyses. The best six-variable equation differs slightly from an equation presented in a previous analysis of the data set (Froehlich, 1988) because of the deletion of the outlier measurement. The six-variable equation presented here is preferred.

The five-variable regression equation minimizes the standard error of estimate of the logarithm of relative depth of local scour and contains parameters that are all significant at the 0.10 confidence level. The prediction equation written in terms of untransformed values is

$$\frac{\hat{d}_s}{b} = 0.40\,\phi\left(\frac{b'}{b}\right)^{0.53}\left(\frac{y_a}{b}\right)^{0.46}\left(\frac{b}{d_{50}}\right)^{0.052} \tag{6}$$

where

$$\phi = \begin{cases} 1.4, & \text{for square-nosed piers} \\ 1.0, & \text{for round-nosed piers} \\ 0.5, & \text{for sharp-nosed piers} \end{cases}$$

PREDICTING NEW OBSERVATIONS

Relative depth of local scour predicted by equation 6 is the expected response to a specified set of regressors, and is not known with certainty. However, a local scour depth that has a small probability of being exceeded needs to be used for designing pier foundations to insure against the failure of a bridge. A probability statement about the distribution of $\log(d_s/b)$ provides a means of evaluating the accuracy of the estimate and of determining an appropriate design value. Letting

$$\mathbf{x}_0' = \left[1,\ Z_1,\ Z_2,\ \log\left(\frac{b'}{b}\right),\ \log\left(\frac{y_a}{b}\right),\ \log\left(\frac{b}{d_{50}}\right)\right] \tag{7}$$

be the transpose of the vector of predictor variables at a particular site, a $100(1 - \alpha)$ percent upper confidence limit for $\log(d_s/b)$ is computed as

$$\log(\hat{d}_s/b)_0 + t_{\alpha,n-p}\sqrt{\mathrm{var}(\mathbf{x}_0\hat{\beta})} \tag{8}$$

(Montgomery and Peck, 1982, p. 141), where

$$\log(\hat{d}_s/b)_0 = \mathbf{x}_0'\hat{\beta}() \tag{9}$$

is the predicted value of $\log_e(d_s/b)$,

$$\hat{\beta}' = \left[-1.150,\ 0.412,\ -0.422,\ 0.534,\ 0.464,\ 0.052\right] \tag{10}$$

is the transpose of the vector of regression model parameters, $t_{\alpha,n-p}$ is the one-sided t-distribution statistic corresponding to an exceedance probability α and n-p degrees of freedom, n is the number of observations used in the regression, p is the number of regression model parameters,

$$\mathrm{var}(\mathbf{x}_0\hat{\beta}) = \hat{\sigma}^2\left(1 + \mathbf{x}_0'(\mathbf{X}'\mathbf{X})^{-1}\mathbf{x}_0\right) \tag{11}$$

is the estimated variance of prediction, and $\hat{\sigma}^2$ is the residual mean square error (estimated model error). For the 5-variable regression model, n = 65, p = 6, $\hat{\sigma}^2$ = 0.1056, and

$$(\mathbf{X}'\mathbf{X})^{-1} = \begin{bmatrix} 0.1433 & -0.0196 & 0.0375 & -0.0056 & -0.0360 & -0.0160 \\ -0.0196 & 0.2566 & 0.0410 & -0.0962 & -0.0059 & 0.0018 \\ 0.0375 & 0.0410 & 0.1436 & -0.0077 & -0.0340 & -0.0075 \\ -0.0056 & -0.0962 & -0.0077 & 0.1039 & -0.0220 & -0.0011 \\ -0.0360 & -0.0059 & -0.0340 & -0.0220 & 0.0523 & 0.0045 \\ -0.0160 & 0.0018 & -0.0075 & -0.0011 & 0.0045 & 0.0022 \end{bmatrix} \tag{12}$$

One-sided t-distribution statistics for n - p = 59 degrees of freedom and several confidence levels are given in Table 4. An upper estimate of $\log(d_s/b)$ is obtained from equation 8 by selecting an appropriate confidence level, say 99 percent, and obtaining the corresponding t statistic from Table 4. Use of an upper limit of scour depth for design of pier foundations will provide a factor of safety needed to insure against failure of the structure.

Hidden Extrapolation

Extrapolation beyond the region defined by the data used to develop a regression equation may provide an unreliable prediction. It is possible that an equation that predicts accurately in the region spanned by the data will perform poorly outside that region. Because the levels of the regressor variables jointly define the region containing the data, it is easy to extrapolate inadvertently. Although each member of a given set of regressors may lie within the range of that particular variable, it is possible that the point defined by the set lies outside the region defined by the original data.

To detect a hidden extrapolation, the smallest convex set containing all of the original data points is considered and

TABLE 3. SUMMARY OF LOCAL PIER-SCOUR REGRESSION EQUATIONS

Number of vari-ables	Constant	Coefficient of Z_1	Z_2	$\log \frac{b'}{b}$	$\log \frac{y_a}{b}$	$\log F_a$	$\log \frac{b}{d_{50}}$	Coeffi-cient of determ-ination	Stan-dard error of estimate
2	-0.625	1.214	0.064[a]	---[b]	---	---	---	0.388	0.508
3	-0.739	0.447	-0.089[a]	0.724	---	---	---	0.601	0.414
4	-0.761	0.367	-0.241	0.560	0.355	---	---	0.713	0.353
5	-1.150	0.412	-0.422	0.534	0.464	---	0.052	0.762	0.325
6	-1.220	0.396	-0.450	0.549	0.497	0.190[a]	0.094	0.767	0.325

[a]The hypothesis that the coefficient equals zero cannot be rejected at the 0.10 significance level.

[b]Variable is not included in the regression model.

TABLE 4. ONE-SIDED t STATISTICS FOR SEVERAL CONFIDENCE LEVELS AND 59 DEGREES OF FREEDOM

Confidence level, in percent	One-sided t-distribution statistic
50.0	0.000
60.0	0.254
70.0	0.527
80.0	0.848
90.0	1.30
95.0	1.67
97.5	2.00
99.0	2.39
99.5	2.66

referred to as the regressor variable hull (RVH). If a point defined by a set of regressors lies inside or on the boundary of the RVH the prediction involves interpolation, while extrapolation is needed if the point lies outside the boundary. The diagonal elements h_{ii} of the hat matrix **H** can be used to determine the position of a point $\mathbf{x}_i$ (Montgomery and Peck, 1982, p. 143). The values of h_{ii} are dependent on the Euclidean distance of a point $\mathbf{x}_i$ from the centroid of the RVH, and on the density of the points in the RVH. Usually the largest value of h_{ii}, say h_{max}, corresponds to a point that lies on the boundary of the RVH in a region of the x-space where the density of observations is relatively low.

The location of a point $\mathbf{x}_0$ relative to the RVH is reflected by the quantity

$$h_{00} = \mathbf{x}_0'(\mathbf{X}'\mathbf{X})^{-1}\mathbf{x}_0. \tag{13}$$

A point $\mathbf{x}_0$ for which $h_{00} \geq h_{max}$ lies outside an ellipsoid enclosing the RVH, and is a point for which extrapolation would be needed. If $h_{00} \leq h_{max}$, the point $\mathbf{x}_0$ is inside the ellipsoid and possibly within the RVH, and would be considered an interpolation point. Generally, the smaller the value of h_{00}, the closer the point $\mathbf{x}_0$ lies to the centroid of the RVH. For the 5-variable prediction equation listed in Table 3, $h_{max} = 0.214$. When using equation 5 to predict a local scour depth, h_{00} needs to be calculated using the $(\mathbf{X}'\mathbf{X})^{-1}$matrix (equation 12) and the $\mathbf{x}_0$ vector of regressor variables, then compared to h_{max} to determine whether or not the prediction is an extrapolation.

EXAMPLE APPLICATION

To demonstrate use of the local scour prediction equation and calculation of an upper confidence limit to be used for design, consider the following values for a hypothetical square-nosed bridge pier:

$$b = 1.0 \text{ m}$$
$$\ell = 6.0 \text{ m}$$
$$\theta = 5 \text{ deg}$$
$$y_a = 4.0 \text{ m}$$
$$V_a = 2.0 \text{ m/s}$$
$$d_{50} = 2 \text{ mm}$$
$$S_s = 2.65$$

Dimensionless variables for the example are $\frac{b'}{b} = 1.52$, $\frac{y_a}{b} = 4.0$, $\frac{b}{d_{50}} = 500$; and $V_c = 1.0$ m/s (from equation 5). Critical velocity exceeds the approach flow velocity; hence, live-bed conditions exist and equation 6 is applicable. The transpose of the vector of predictor variables is

$$\mathbf{x}_0' = [1.0,\ 1,\ 0,\ 0.418,\ 1.386,\ 6.215]$$

and $h_{00} = \mathbf{x}_0'(\mathbf{X}'\mathbf{X})^{-1}\mathbf{x}_0 = 0.232$. The predicted relative depth of local scour $\hat{d}_s/b = 1.58$ from equation 6, and the 99 percent upper confidence limit of $\log(d_s/b)$ yields $d_s/b = 3.73$. Because $h_{00} \geq h_{max}$, the point $\mathbf{x}_0$ is outside the region defined by the variables used to fit the regression model parameters. Hence, the prediction is an extrapolation and needs to be evaluated carefully before being accepted.

SUMMARY AND CONCLUSIONS

On-site measurements of local scour at bridge piers obtained from both published and unpublished sources were assembled, and prediction equations for relative depth of local live-bed scour were developed using multiple regression analysis. The best prediction equation obtained was a 5-variable model that requires a description of pier shape, and values of pier width, pier length, angle at which flow approaches a pier, approach flow depth, and the median size of bed material. In addition, an estimate of approach flow velocity is needed to determine if live-bed scour conditions exist at a site. The regression analysis provides a probability statement about the accuracy of a prediction, based on particular values of the regressor variables, which allows an upper confidence limit of relative depth of local scour to be calculated. The upper confidence limit provides a factor of safety for design that is based on sound statistical theory.

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Chasing Floods and Measuring Scour

Roy E. Trent and Mark Landers

ABSTRACT

Field measurements of local scour at bridges during floods, and measurement of sedimentation processes in general, has been a frustrating and unrewarding activity in the past. Surprisingly, little scour data has been collected in that time and only a small portion of the data have been taken during floods at bridges crossing natural streams. Recent efforts of U.S. Geological Survey (USGS) field crews to measure scour during floods for the Federal Highway Administration and several State highway agencies, however, demonstrate that it can be done. This paper outlines procedures used, insights and experiences gained, and some preliminary results obtained while chasing floods and measuring scour during the May 1990 floods in the Southwest.

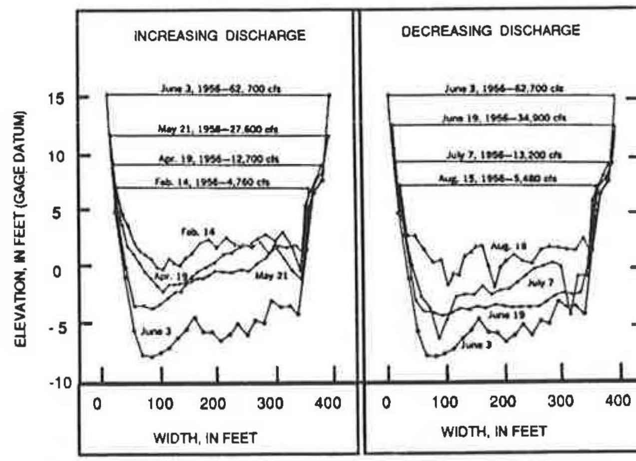

Figure 1. Scour and deposition with flood flow for Colorado River at Lees Ferry, Arizona.

BACKGROUND

Scour data collected during flood conditions reinforces our knowledge that fluvial processes are dynamic. The high flow velocities and turbulence that occurs during a flood causes increased shear stress on the streambed and results in scour, erosion, and increased sediment transport capacity of streams. As flood flows recede, the bed load is redeposited and streambed elevations typically increase and scour holes in the streambed and near structures refill. Figure 1 shows an example of bed scour and fill for a 1956 flood on the Colorado River at Lees Ferry, Arizona. Figure 2 shows another example of this process for the May 1990 flood on the Red River at the U.S. Highway 71 bridge in Arkansas. This process is characteristic of alluvial streams having noncohesive sand or silt bed materials. This process is less typical of rivers with cobbles or boulders or cohesive clay bed materials; however, if finer materials are also present, they will fill local scour holes in the cohesive or large grain size material during the flow recession. The process of channel scour and fill is not linear with hydraulic factors because of the effects of bed load and other deterministic factors on scour. Bed load is itself a temporally and spatially dynamic process, so that generalizations of the relation between scour and hydraulic factors must be applied with care. The relation of bed load to discharge will generally follow a clockwise, hysteretic curve for a given flow event. For live bed streams, this may mean initial filling of the general (and thus local scour) sections for a flow event. This is illustrated in figure 3 for data collected on the South Altamaha River at I-95 near Brunswick, Georgia, and in figure 4 for San Juan River near Bluff, Utah. These and other data emphasize the importance of measuring scour near the peak of the flood to improve our understanding of these processes. This is particularly important when conducting scour inspections of bridges.

Obviously, it cannot be safely concluded that a depression or deep hole measured around a pier after the flood represents the maximum scour experienced in recent time. **Accordingly, inspection of bridges for scour, even shortly after a flood, may result in misleading and even dangerous conclusions.** Perhaps, coring or using geophysical subbottom profiling techniques, and looking for undisturbed sediments, marles, or other parent materials, may provide conclusions about the scour history of a particular pier. Every pier, however, requires separate evaluation because every pier has different scour potential and scour resistance. Scour potential also varies laterally across the channel as a function of channel curvature, flow concentration due to contraction or flow redirection, and many other site specific parameters. Many variables affect the rate and depth of scour. The actual scour performance of a bridge during any given flood event is best determined by real time measurements by field crews.

DATA REQUIREMENTS

Different levels of data are required for different purposes. Currently, a primary concern of bridge owners and major purpose

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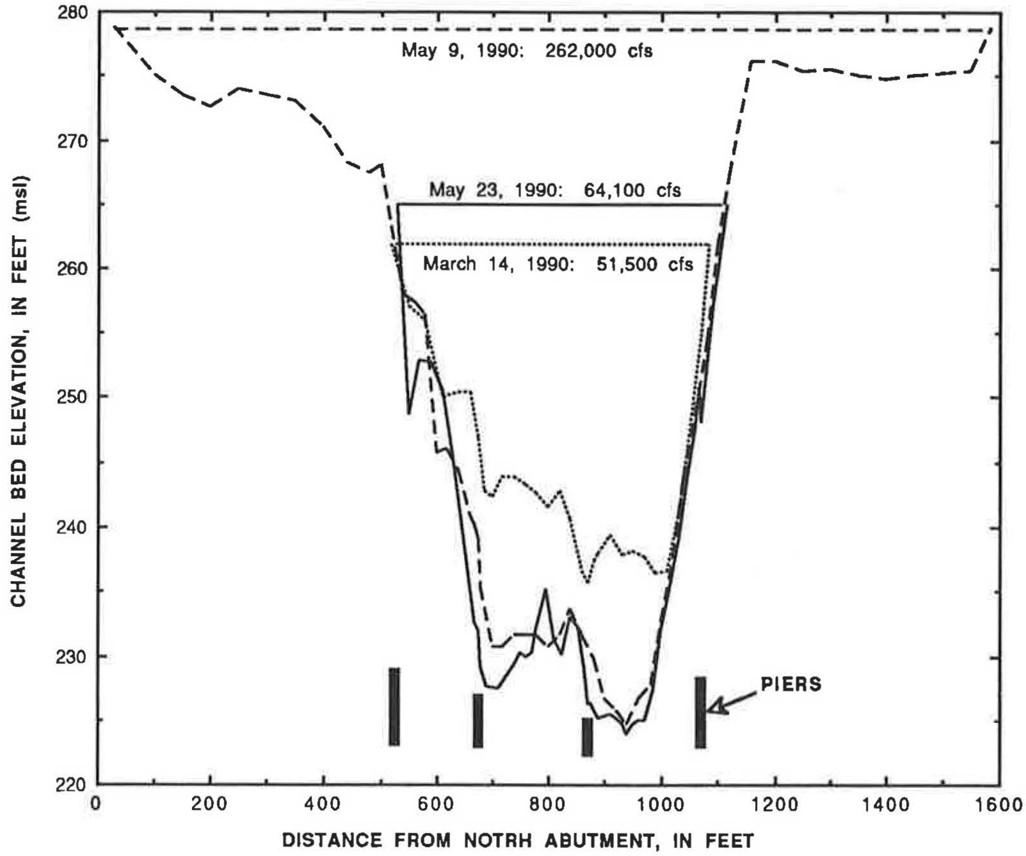

Figure 2. Changes in channel geometry with flood discharge for the Red River at U. S. Highway 71 near Ibex, Arkansas.

for measuring bridge scour is to verify local scour formulas or to put practical constraints on the resulting scour estimates. A **limited data set** is required for these objectives. This paper discusses procedures for obtaining the limited data set. Such data includes and is an extension of data (stage, velocities, depths) obtained from standard flood discharge measurement procedures used by USGS. An important distinction exists between flood discharge measurements and scour measurements: **the limited data set focuses measurements on and around the bridge features that are typically avoided in standard discharge measurements.** Scour measurement adds details on geometry of the river bottom near the bridge piers and abutments, including cross sections on both sides of the bridge, photographs and observations of flow conditions, and a judgement about the status of bedload movement. USGS scour survey teams use a checklist as a reminder of items to be included in a limited data set (figure 5). Other critical data are site records on foundations, bed material, channel and floodplain characteristics, and bridge hydraulics. These data are preferably obtained prior to measured flood events, but at many sites they can also be collected afterwards.

Another reason to collect data is to develop, calibrate, and verify sediment transport models and methods in order to simulate fluvial processes--a ***detailed data set*** is required. Data required for this purpose would include flow, sediment, and bedload data in addition to scour depths and velocity data. This data must be measured at the bridge and at sections well upstream and downstream several times over the flood hydrograph. In addition to collecting real time data during the flood, surveys of section bedforms and channel conditions before (during preliminary field surveys of study sites) and soon after the flood should be made. Uses for this level of data includes modeling the fluvial processes for long-term performance of a river system, as well as the response of the channel to a single flow event. Simulation of channel response to measures like channelization, dredging, aggregate mining, channel shortening, flow control, flood proofing, and width constrictions are now possible. These data will support improved technology and simulation models that are easier to use and more reliable. Ultimately this and even more detailed data are needed to establish cause and effect relationships, for example, to relate the flow event to the sedimentation processes and relate that to river responses.

For selected high risk bridges, where scour potential is known to be a serious threat, scour measurements taken during major flood events could become a part of the standard bridge inspection reports. Only a very ***restricted inspection data set*** (local geometry near piers and abutments) would usually be required for safety inspections of bridges during floods. The methods prescribed here would be suitable for adoption and adaptation by bridge inspection teams or highway maintenance crews for making scour inspections during floods. The primary use for such data would be to document conditions of foundation support rather than to quantify or describe scour processes.

PROCEDURES

Procedures for collecting scour data are governed by the requirements for the data set being collected, be that an inspection, limited, or detailed data set. The procedures

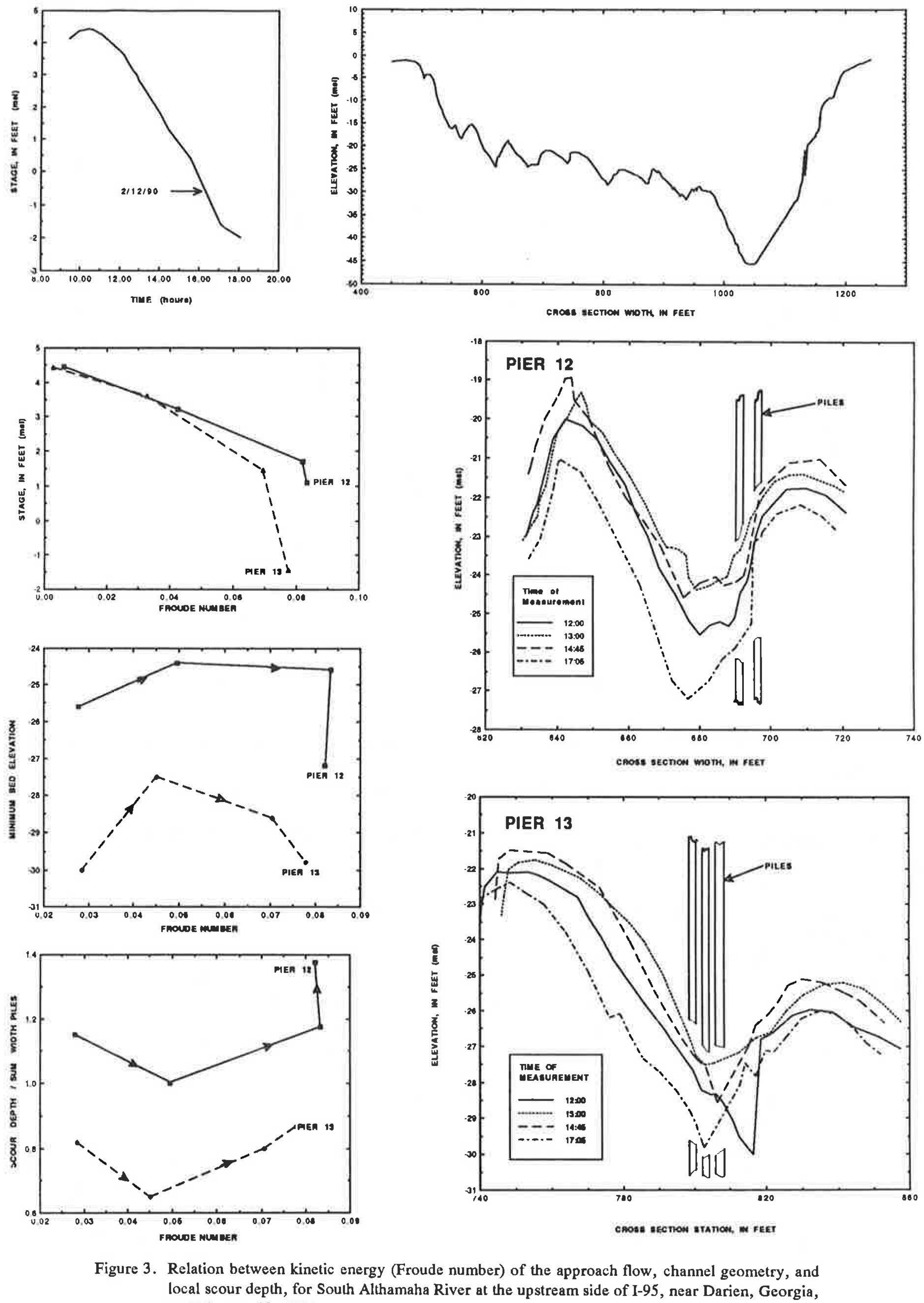

Figure 3. Relation between kinetic energy (Froude number) of the approach flow, channel geometry, and local scour depth, for South Althamaha River at the upstream side of I-95, near Darien, Georgia, on February 12, 1990.

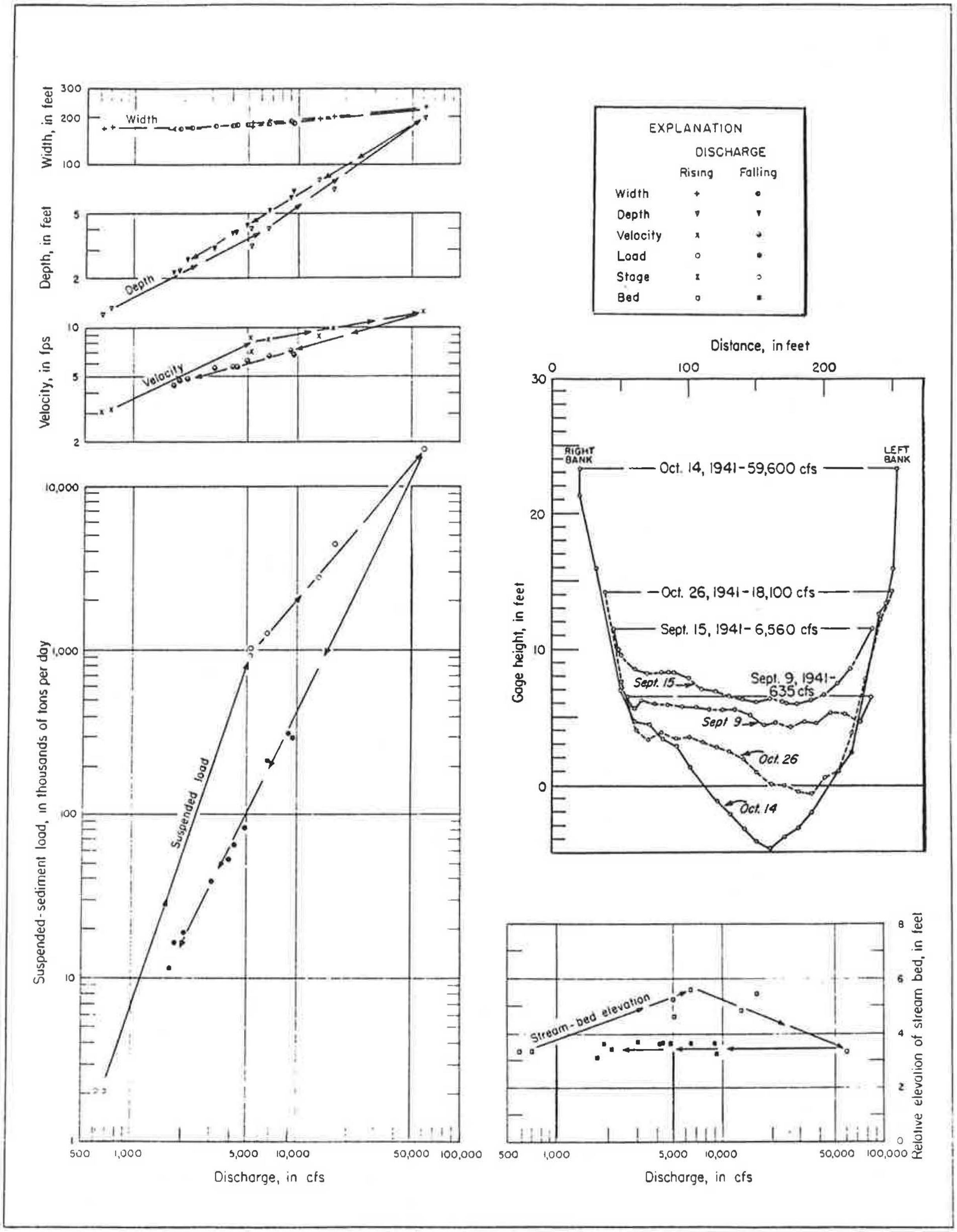

Figure 4. Changes in channel geometry, velocity, and sediment load during the flood of September-December 1941, San Juan river near Bluff, Utah (from Leopold and Maddock, Jr., 1953).

prescribed here apply to the limited data set used to collect local scour data. An adaptation of these procedures may be particularly useful for bridge owners to use as a standard inspection procedure for bridges suspected to have the potential for scour.

Preparation

Preparation for collection of scour data may be broken down into three phases: reconnaissance, site selection, and site establishment. Note that the field methods described may also be used where sites have not been reconnoitered or established; however, it is very useful to have advanced knowledge of bridge and hydraulic conditions.

- Reconnaissance: A thorough review of local, university, State, and Federal agency data files should be examined for previous records (like past discharge measurements made from bridges or similar records) that can be used to evaluate constriction scour and/or local scour. Figures 1, 4, and 6 are based on the results of past discharge measurement or bridge inspection reports. Research papers, journal articles, and other technical publications related to bridge scour may contain additional information on the past scour performance of bridges. A major source of information on scour problems and bridge performance is available from the bridge owner, particularly the bridge inspection, hydraulics, and maintenance organizations. Types of information to gather include repair and maintenance reports, soils data, and design plans.

- Site selection: This is a very critical step toward a successful program. Sites should be selected to support the purpose of the program and to obtain the required data. There are many criteria to consider for targeting a potential site for inclusion in a data collection effort. Obviously, bridge sites must be accessible during floods. Physically, the bridge must be safe and effective for survey work. Foundation geometry and bridge configuration should be simple and typical. The channel, floodplain, and river system should be uncomplicated and representative. The river bed should have potential to scour and be free of scour limitations like a shallow hardpan layer that will not erode or even a protective layer of riprap around the pier or abutment. In brief, sites should be representative of the primary concerns and reasons for collecting scour data--including geographical distribution, bridge and foundation type and size, hydrology and watershed factors, fluvial geomorphology and sediment types--and of course for special problem resolution. Finding suitable, representative sites may at first seem impossible--ideal conditions can only be found in the laboratory. Consider that the goal of site selection is to **maximize the opportunity and potential** to collect data types required to support established program purposes.

- Site establishment: Activities required to properly establish a site include gathering more thorough information on a site as an extension to the reconnaissance performed earlier, and preparation of the sites and also survey teams for easy and effective field work. Initial preparation would include: equip and train teams, establish procedures to "watch" the sites for measurable events, assigning sites to each team to watch, and possibly hire local flood observers who give an alert at a predetermined stage or precipitation amount. Site preparations would include preliminary surveys and soundings, scouting boat launch sites when applicable, marking the bridge for measurement stations, taking field notes and documenting potential problems, and preparing advanced data entry forms and summary sheets.

BRIDGE SCOUR MEASUREMENT CHECKLIST

Sta. no:________ Name:________
Date:________ Party:________

Get all the information once. If the peak is long enough, measure it all again. **Detail of scour hole geometry (~1 ft. intervals) is critical.** Try to sound directly at face and sides of piers.

__ Photographs and notes of observations of turbulence around piers, flow conditions in channel, overbank, and at abutments, constriction of floodplain flow at bridge, debris, angels, etc.

__ Determine whether bed is moving.

__ Cross section measurements on upstream side of bridge, including detailed soundings around and as close as possible to piers, abutments. and other scour holes.

__ Cross section measurements on downstream side of bridge including detailed soundings around and as close as possiblo to to piers, abutments. and other scour holes.

__ Standard flood discharge measurement, also if not at gage, regular tapedowns over duration of measurement.

__ Water temperature.

NOTE: If the high-stage rating is fairly well defined, measure the cross section and scour hole data first, before the discharge measurement.

Figure 5. Checklist form used by USGS scour survey teams.

Methods

Methods for collecting scour data are not unlike those used in typical discharge measurements, except for the method of sounding to collect channel geometry. The traditional method for gathering scour and bedform data has been to sound the depth with a Columbus-type lead sounding weight (figure 7). For scour data collection, a 100- to 300-pound weight is generally used. Sounding with the weight may be the best method to collect channel geometry data where velocities are high, average channel depths are less than about 15 feet, and the channel bed is relatively hard. For example, on May 10, 1990, flooding on the Red River at I-30 resulted in flow for the first time through a relief opening which also serves as an overpass for a local paved road. This bridge nearly failed during that flood because of contraction and abutment scour. Large fill material was dumped at the upstream abutments almost constantly over a 24-hour period in an attempt to arrest local scour. USGS personnel measured a velocity of 14.8 feet per second in the opening. The fathometer could not obtain a depth reading due to excessive turbulence and entrained air, so the transducer was stripped from the weight. The 200-pound weight was allowed to free-fall to make a single contact sounding of the bottom before being swept downstream. The average depth was about 8 feet and the maximum was about 9 feet. These conditions might also prevail in mountainous regions with steep sloped streams.

For more commonly occurring alluvial channels, the lead weight sounding method has several potential problems. As channels get deeper and or velocities get higher, the weight is swept much farther downstream from the point of suspension. Vertical angle corrections may be applied in this case; however the depth measured may not be very close to the desired cross section location on the upstream or downstream sides of the bridge. At velocities above about 10 feet per second, it may not be possible to sound depths at all with a weight on deeper streams. Debris near the bottom, and especially around piers, can easily hang up a weight, break a cable, and create a hazard to the field crew.

Sonic depth finders have been used as an alternative to sounding with weights. The systems used at most of the sites on the spring 1990 floods are graphical chart-recording fathometers with narrow (8-degree) beam transducers to improve accuracy close to piers. These units were tuned by the manufacturer for 100 feet of transducer cable. The transducer is mounted on the bottom of a weight (figure 7) and suspended from an instrument crane mounted on a truck or dolly device mounted on a bridge (figure 8). Accurate soundings can usually be obtained running the transducer 2 or more feet below the surface of the water, and moving slowly across the bridge at a fixed rate. The elevation of the transducer is referenced to the measured stage of the water surface. The depth of the transducer below the water surface must be checked often to provide an accurate record of depth to the channel bottom. Where piers are inset from the edge of a bridge, the sounding weight may be lowered further to increase the drag allowing the flow of water to carry the sounding weight closer to the pier. Another successful method has been to attach flotation devices to the weight, which can increase drag. The flotation device shown in figure 9 worked reasonably well in velocities up to about 6 feet per second. Depth to the transducer varies with velocity and was measured using tags to the suspension cable. More streamlined floats are being tested.

Use of this equipment under flood conditions by USGS hydrologists has not required special training. Bridge maintenance or inspection teams would be able to take valid soundings during floods after some initial training and experience in deployment methods. Table 1 provides a list of equipment typically used be the USGS for a detailed data set survey. Prices are approximate and less expensive components are available.

TABLE 1. LIST OF EQUIPMENT USED IN SCOUR DATA COLLECTION SURVEYS.

Crane, truck mounted	$2,000
Power wench for crane	2,500
Sounding weight, 600 pounds	600
Fathometer, recording	700
Floats for sounding weight	150
Velocity meter	2,000

USGS scour teams typically contact State highway agencies during large floods to assist in checking bridges of particular concern. Problem bridges, however, do not often provide useful data for the limited data set. Also large regional floods, such as those of May 1990 in the southwest, endanger many bridges and USGS teams or personnel for this activity are limited. Flood scour inspection teams need to make repeated checks at bridges where a relatively small change in scour depths would endanger the bridge and where flood hydrographs may last for several days. This was the case for two bridges on the Red River in the May 1990 flood. Because a crew could not stay at these sites and make repeated measurements over the hydrograph, it was assumed that data collected was the maximum scour. On major bridges or where the streambed is not alluvial, this assumption could result in misleading information or dangerous conclusions.

The use of a boat and recording fathometer can dramatically increase the information on channel geometry and bedform under

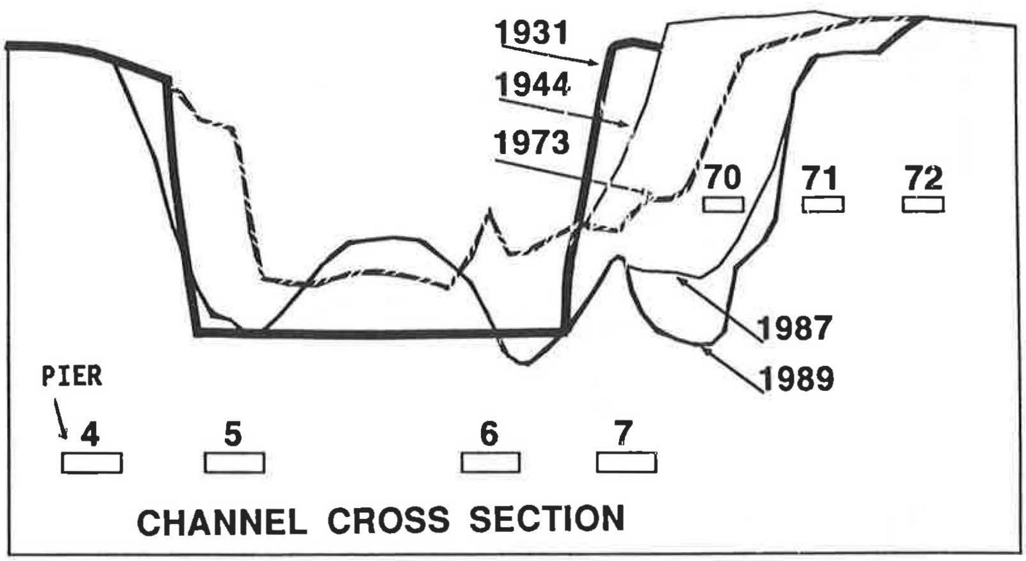

Figure 6. Bridge inspection records document the history of channel migration and bedform changes over several years.

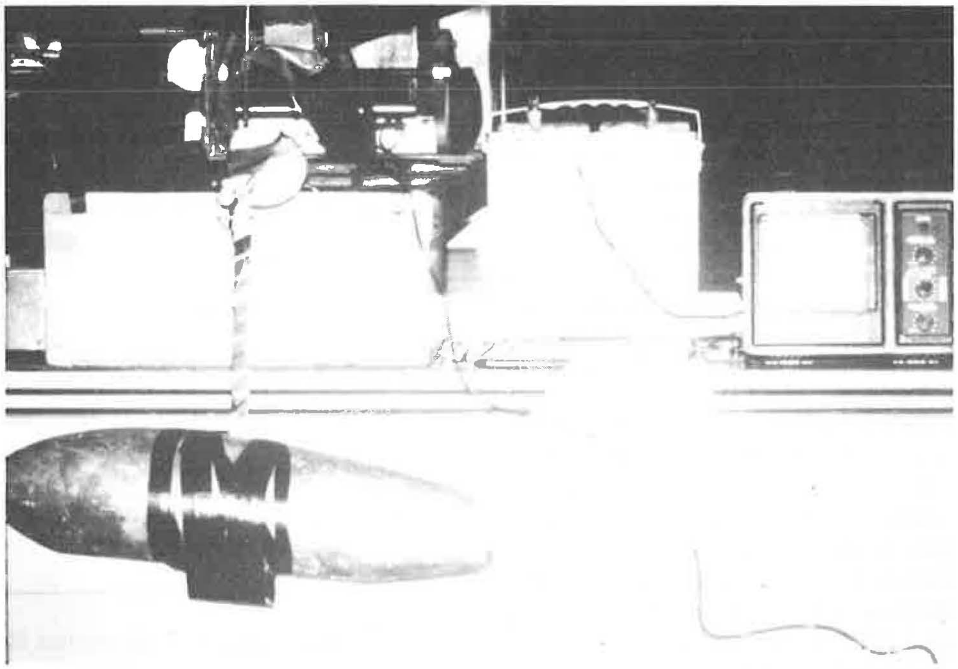

Figure 7. Columbus-type sounding weight, used by USGS with a fathometer transducer attached to the bottom.

the bridge and at approach and exit cross sections. It is not always possible nor safe, however, to launch and operate a boat during a measurable flow event, and less likely during a large flood. Where detailed data sets are the goal, boats provide invaluable data on channel movement at considerable distances from the bridge. Management of information becomes a considerable problem because large quantities of data can be generated in a very short time. Use of a graphic recording fathometer will help control bed elevation records. Position of the bridge and bank with regard to all observations and records must be estimated and carefully logged. Automatic positioning systems are now available. When these records are tied to fathometer records and other location dependent data, more accurate and easier to interpret information results. A simple range finder, hand compass, stop watch, and clinometer or hand level, when combined with encoded notes that can be deciphered later, work together to document fathometer and sounding records taken from a boat.

Figure 8. Truck-mounted crane used to suspend and manipulate the weight over the side of the bridge.

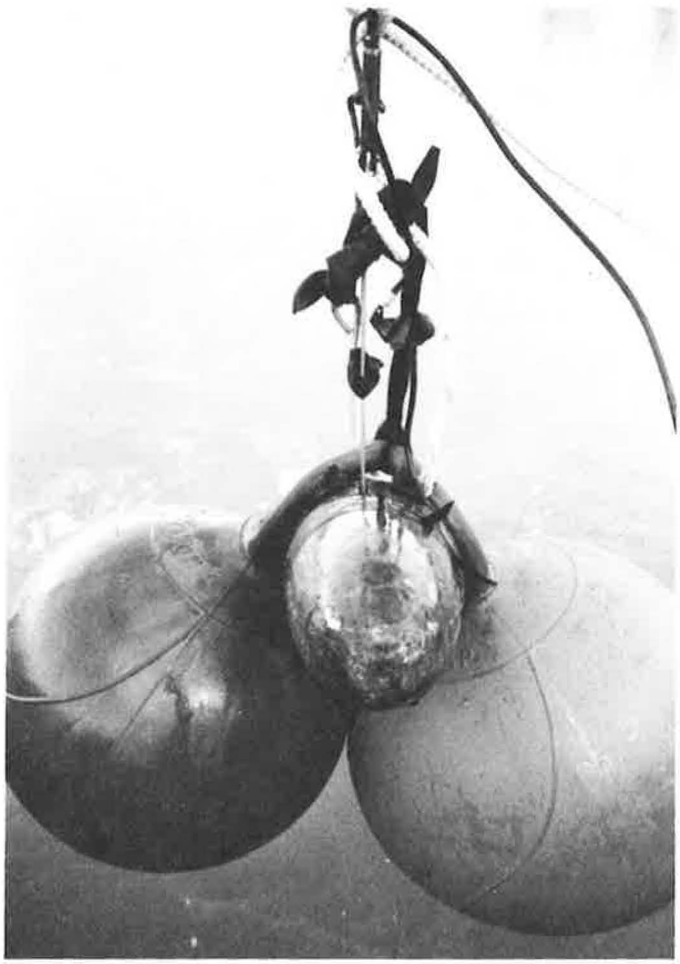

Figure 9. Balloons are added to a sounding weight to float it under the bridge deck for data collection near the pier.

Manpower

The key ingredient to successful collection of scour data during a flood lies with the motivation, and to a lesser extent, the training and experience of the field crew. Chasing floods to collect scour data requires a willingness, if not eagerness, to work weekends, holidays, long hours and usually in adverse weather, and sometimes with uncertain opportunities to sleep and eat. Besides, it might not be entirely safe to work on a bridge with a scourable river bottom--and sometimes a known scour problem. And yet, hydrologists are uncommonly willing to work in such conditions if they have the full support of management and are provided proper equipment, resources, and preparation.

A minimum of two qualified crew members are required to collect the limited detail scour data at a site during a flood. A procedural element, which may be more important than the number of people, is an advanced agreement to assign personnel and equipment to collect scour data during a flood event. This requires special consideration because there can be no advanced schedule to collect scour data. It takes a lot of coordination and planning to put a fast response team in the field in time to take records at critical times during the flood. It may not be logistically possible for a headquarters based team to survey remote sites or to arrive in time to take measurements where the time to flood peak is quick or where the flood duration is short. Teams need the freedom and authority to respond to rapidly changing conditions, requirements, and opportunities for these procedures to work. Under even the most favorable circumstances, this activity needs high administrative priority and managerial support.

LIMITATIONS AND PROBLEMS

Scour data collection is limited by a variety of factors, including the logistics of assembling a team with field equipment and getting to a site in time to measure the actual event. Other adverse working conditions are and will continue to limit collecting scour data while its occurring. Accurately locating and recording horizontal and vertical transducer or sounding weight positions is the principal limitation to accuracy using this technique.

Bridges themselves pose one of the greatest problems--they come in all sizes and shapes. Trying to use the procedures outlined here on a through-truss bridge consumes much more time than a deck structure (truss or girder) supported entirely from the bottom. Bridges with walkways, pedestrian barriers, fences, or other obstructions to the upstream edge will not allow access with the crane currently being used.

Bridges and entire road systems may not be accessible due to high water or road failures. Sanitation, safe drinking water, adequate rest, and fit food for the field crew, though usually accounted for, cannot be overlooked. Traffic control of course can be a problem and an added hazard. Debris around a pier can block fathometer readings and other sounding devices. Moving debris can also rip the sounding weight and transducer away or damage the rig and crane transport. Experienced teams actually have few problems with debris or other obstacles.

CONCLUSIONS

Local scour, contraction scour, and degradation and aggradation of alluvial streambeds are dynamic processes that occur during floods or high flow conditions. The data collected in the Spring of 1990 show that these various forms of scour are difficult to separate, except in the laboratory or an ideal case. Chasing floods provides the best chance of collecting data for proper interpretation and separation of the different contributing scour processes.

Maximum scour occurs during floods and high flow events. Scour holes usually start to infill or refill and bedload starts to redeposit after the flood passes or before the flood fully recedes. Accordingly, remnant scour holes (ones that remain after the flood peak has passed or high flow recedes) can be a misleading and dangerous indication of the actual scour performance of a bridge during the flood.

Key ingredients to a successful program to collect scour data during floods includes:

- Properly prepared and motivated survey teams.
- Management support of the scour data collection program.
- Practical and flexible operating procedures.
- Innovative approach to instrumentation and its use.

One dependable method of measuring scour is to use fast response field crews to mobilize when floods are in progress and take real time scour and flow measurements at bridges using the procedures prescribed in this paper. Other candidate methods to

measure scour have limitations that make portable instrumentation, mobile teams, and chasing floods attractive for certain classes of problems and sites.

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Estimating Scour at Bridges

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Methods of estimating scour at bridges and abutments based on recommendations in the FHWA 1988 Technical Advisory on Scour [1] are presented. Estimating abutment scour is discussed based on specific recommendations in FHWA's publication HEC-18. Additionally, research results on pressure flow developed at Colorado State University by Lila Abed, and on the effect of wide footings on scour depths developed at the Fairbanks Turner Hydraulics Laboratory by Sterling Jones are presented. Also discussed are the effect of angle of attack on shape factor of a pier and on rows of columns and the top width of scour holes. Knowledge of the top width of scour holes is important in determining if scour at piers and abutments are independent of each other, and for designing riprap protection.

Total Scour at a highway crossing is comprised of three components. These components are:

1. Aggradation and Degradation. These are long-term streambed elevation changes due to natural or man induced causes within the reach of the river on which the bridge is located.
2. Contraction Scour. Contraction scour results from a contraction of the flow, a change in downstream control of the water surface elevation, or the location of the bridge in relation to a bend. Contraction of the flow by the bridge approach embankments encroaching onto the floodplain and/or into the main channel is the most common form of contraction scour.
3. Local Scour. Local scour occurs around piers, abutments, spurs, and embankments, and is caused by the acceleration of the flow and the subsequent development of vortex systems induced by these obstructions to the flow.

In addition to the types of scour mentioned above, *lateral movement or shifting of the stream* may also erode the approach roadway to the bridge or change the total scour by changing the alignment of the flow in the waterway at the bridge crossing.

In 1988, the Federal Highway Administration issued a Technical Advisory on Scour [1]. In the advisory, interim procedures for evaluating scour at bridges were given. These procedures were based on a detailed evaluation of the best available knowledge of scour. Since issuance of the TA, researchers, as well as state and federal agencies, have been active in the improvement and verification of predictive methods to determine scour depths. Some of this activity is in the form of fundamental research in the laboratory and field. One of the most promising programs is being conducted by various state agencies who have been actively evaluating existing bridges and assessing their vulnerability to scour.

From this resurgence of interest in scour at bridges has come a wealth of new literature concerning bridge scour. These studies have sought to improve upon the equations in the TA, as well as shed new light on factors which could have an aggravating or mitigating influence on the total scour at bridges. In particular, these efforts have:

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1. questioned whether the scour depths predicted by the recommended equations in the TA for abutments and piers are too deep;
2. sought to determine whether, and to what extent, larger bed material sizes (cobbles) decrease scour;
3. sought to determine the effect of footings or multiple piles on scour depths when they are exposed by the flow;
4. sought to determine the width of the scour hole around piers and abutments; and to determine to what extent pressure flow influences total scour.

For example, research by Raudkivi and Ettema [2], Raudkivi [3], Copp et. al. [4], and, Melville and Sutherland [5] indicates that large particles in the bed-material decrease scour depths. However, field verification is insufficient to incorporate these studies into a recommended procedure that takes bed-material size into account at this time (FHWA [6] and Richardson and Richardson [7]).

Concerning abutment and pier scour, new studies indicate that the equations for abutment scour are conservative, and can be used to estimate the worst-case scenario. For pier scour, the CSU equation still appears to give the most likely estimate of scour depths. Research by Jones [8] reveals a method to evaluate the effect of wide footings and pile groups, while very recent research by Lila Abed shows that contraction, abutment and pier scour are significantly increased with pressure flow. In some tests, these increases are 3 times what clear water, free surface flow scour would be.

It should be noted that most of these questions still do not have definitive answers. However, the recent research, highlighted above, needs to be brought forward and placed in the context of the original TA. It is for this reason that the discussions of the above topics will be included, where relevant, in a general review of methods and equations for determining total scour at a bridge. This review will begin by outlining the basic design approach, followed by a more detailed discussion of the methods and equations to be used for each of the design steps.

DESIGN APPROACH

Before the various scour forecasting methods for contraction and local scour can be applied, it is first necessary to 1) obtain the fixed-bed channel hydraulics; 2) estimate the long-term profile degradation or aggradation; 3) adjust the fixed-bed hydraulics to reflect these changes and 4) compute the bridge hydraulics. If contraction scour is large (larger than 4 ft.), it may be necessary to adjust the fixed-bed hydraulics before computing local scour.

The steps recommended for estimating scour at bridges as given in the Technical Advisory [1], are summarized below and discussed in greater detail after this general summary.

STEP 1. Determine scour analysis variables.

STEP 2. Analyze long-term bed elevation change.

STEP 3. Compute the magnitude of contraction scour.

STEP 4. Compute the magnitude of local scour at abutments.

STEP 5. Compute the magnitude of local scour at piers.

STEP 6. Plot the total scour depths.
STEP 7. Evaluate the total scour depths.
STEP 8. Reevaluate the bridge design as necessary.

DETAILED PROCEDURES

Determine Scour Analysis Variables

1. From a flood frequency study, determine the magnitude of the discharges for the floods to be used for design, including the overtopping flood, when applicable. If the magnitude of the 500 year flood is not available, multiply the Q_{100} by 1.7. This multiplier may be modified if hydrology studies indicate another value may be more appropriate. Experience has shown that the incipient overtopping discharge often puts the most stress on a bridge. However, special conditions such as ice jams, debris, and other factors may cause a more severe condition for scour with a flow smaller than the overtopping flood or, floods with a return period of 100 or 500 years.
2. Determine the water surface profiles for the design discharge using the FHWA/USGS program WSPRO or other existing methods that employ the FHWA bridge analysis procedure. Various conditions and discharges should be used to determine the worst combinations of scour conditions that could occur at the bridge. The engineer should anticipate future conditions at the bridge, in the stream's watershed, and with various downstream water surface elevation controls.
3. Determine if there are existing or potential future factors that will produce a combination of high discharge and low tail water downstream. Check for bedrock or other controls (old diversion structures, erosion control checks, other bridges, etc.) that might be removed in the future. In some cases, dams or locks downstream can control the tailwater elevation on a seasonal basis. Dams upstream or downstream can directly influence the water surface elevation at the bridge. As a rule, the lowest reasonable downstream water surface elevation and the largest discharge should be used to estimate the greatest scour potential. Assess the distribution of the velocity and discharge per foot of width for the design flow and other flows through the bridge opening. Consider also the approach flow and the flow distribution downstream (the contraction and expansion of the flow). The designer must also consider conditions and anticipated future changes in the river.
4. From the output of the above computer programs and from other hydraulic studies, determine the critical discharges, velocities, and other hydraulic parameters which are most critical to the structural stability of the bridge. These parameters are needed for computation of the scour depth.
5. Any additional pertinent information concerning the bridge crossing and the stream requires collection and assimilation prior to assessing the scour at the bridge. This information is essential to the proper estimation of scour depth and includes, but is not limited to, the following types of information:
   - Bore hole logs;
   - Bed material size distribution in the bridge reach;
   - Existing stream and floodplain cross section;
   - Stream geomorphic plan form;
   - Watershed characteristics;
   - Scour data on other bridges in the area;
   - Slope of energy grade line upstream and downstream of the bridge;
   - Estimation of the bed material sediment discharge for flood discharges (flood discharges are mean annual, and 5, 10, 20, 50, 100 and 500 year frequencies). Use Colby's method for sand-bed streams and the Meyer-Peter, Muller equation for coarse-bed-material streams [10];
   - History of flooding;
   - Location of bridge site with respect to other bridges in the area, tributaries to the stream close to the site, bed-rock controls, man-made controls (dams, old check structures, river training works, etc.), and downstream confluences with another stream;
   - Character of the stream (perennial, flashy, intermittent, gradual peaks, etc.);
   - Geomorphology of the site (floodplain stream; crossing of a delta, youthful, mature or old age stream; crossing of an alluvial fan, etc.);
   - Erosion history of the stream;
   - Development history (past, present and future) of the stream and watershed. Collect maps, photographs, areal photographs, interviews of local residences, and planned or contemplated water research projects;
   - Sand and gravel mining in the streambed up or downstream from site;
   - The potential for stream movement and its effect on the bridge, and;
   - Any other factors that could affect the bridge.

Analysis Of Long-Term Bed Elevation Change

Using the information collected above, determine qualitatively the long-term trend in the streambed elevation. Where conditions indicate that significant aggradation or degradation is likely, estimate the change in bed elevation over the next 100 years using one or more of the following techniques:

1. Available computer programs such as HEC 6 from The Corps of Engineers;
2. Straight line extrapolation of present trends;
3. Engineering judgment; and
4. Using the worst-case scenarios, i.e., in the case of a confluence with another stream immediately downstream of the bridge, assume the design flood would occur with a low downstream water surface elevation using a qualitative assessment of the joint probability of flood magnitudes and river conditions on the main stream and its tributary.

If the stream is aggrading, and this condition can be expected to affect the crossing, consider countermeasures including a relocation of the bridge. This should be done taking into account contraction scour. If the stream is degrading, then use the change in bed elevation in the calculations of total scour.

Scour Analysis Method

The recommended method to estimate scour is as follows:

- estimate the natural channel's hydraulics for a fixed-bed condition based on existing conditions;
- assess the expected profile and plan form changes;
- adjust the fixed-bed hydraulics to reflect any expected long-term profile or plan-form changes;
- estimate contraction scour using the empirical contraction formula and the adjusted fixed-bed hydraulics;
- estimate local scour using the adjusted fixed-bed channel and bridge hydraulics; and
- add the local scour to the contraction scour to obtain the total scour.

Contraction Scour

Types of Contraction Scour

Contraction scour can be caused by different bridge site conditions. There are four main conditions (cases) which are as follows:

Case 1.Overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge.

1a.The river channel width becoming narrower either because of the bridge abutments projecting into the channel or the bridge being on a narrower reach of the river.

1b.Abutments set at the stream bank.

1c.Abutments set back from the stream channel.

Case 2.The normal river channel width becoming narrower either because of the bridge itself or by the bridge site being on a narrower reach of the river.

Case 3.A relief bridge in the overbank area with little or no bed material transport in the overbank area.

Case 4.A relief bridge over a secondary stream in the overbank area.

Estimating Contraction Scour

Case 1. Contraction Scour, Stream With Over Bank Flow. Laursen's [9] equation (Equation 1) is recommended to predict the depth of scour in the contracted section for case 1a and 1b. Note that the average contracted depth is the difference between the flow depth upstream, y_1 and the depth in the contracted section, y_2.

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{K_1} \left(\frac{n_2}{n_1}\right)^{K_2} \qquad (1)$$

where

y_1 =Average depth in the main channel,

y_2 =Average depth in the contracted section,

W_1 =Width of the main channel,

W_2 =Width of the contracted section,

Q_c =Discharge in the main channel upstream of the bridge,

Q_t = Discharge in the contracted section,

n_1 = Manning n in the main channel,

n_2 = Manning n in the contracted section

The powers K_1 and K_2 can be related to the transport factor, e, which can be related to the ratio of the shear velocity, V_{*c}, to the fall velocity of the bed material, w.

For most cases, Table 1 can be used as a guide to determining the appropriate values of K_1 and K_2.

TABLE 1 K_1 and K_2 FOR CONTRACTION SCOUR

$\frac{V_{*c}}{w}$	e	K_1	K_2	Mode of Bed Material Transport
< 0.5	0.25	0.59	0.066	Mostly Contact Bed Material
1.0	1.0	0.64	0.21	Some Suspended Bed Material
> 2.0	2.25	0.69	0.37	Mostly Suspended Bed Material

For values of e not listed in the table, interpolation to find the appropriate transport factor provides a method of determining K_1 and K_2. This entails computation of the shear velocity using:

$$V_{*c} = (g y_1 S_1)^{0.5}$$

where

S_1=Slope of the Energy Grade Line in the Main Channel,

g = Acceleration of Gravity (32.2 Ft/s^2).

The exponents K_1 and K_2 can be determined by division of V_{*c} by w and interpolation to find the proper value of e. Using this value of the transport factor, K_1 and K_2 can be computed using the following relationships.

$$K_1 = \frac{6(2+e)}{7(3+e)}$$

$$K_2 = \frac{6e}{7(3+e)}$$

It should be noted that for this case (Case 1a and 1b):

1.The Manning n ratio can be significant when the main channel is composed of dunes with the contracted channel being plain bed, washed out dunes, or antidunes [10].

2.The average width of the bridge opening is normally taken as the bottom width with the width of the piers subtracted.

3.Laursen's equation for a long contraction will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of the contraction or if the contraction is the result of the bridge abutments and piers. At this time however, this equation is the best available.

4.The fall velocity for sand-sized material can be estimated given the grain size of the material by using Figure 1.

Case 1c: is complex, and there is no single equation available to determine contraction scour for this case. The depth of contraction scour depends on many factors such as: how far back from the bank line the abutment is set, the condition of the bank (erodibility, vegetation, bank height, etc), width at the bridge in relationship to the upstream section, the amount of the overbank flow returning to the bridge opening and other influences. Therefore, it is recommended that variables to be used in Equation 1 be carefully selected, coupled with engineering judgement, when estimating contraction scour.

For example, contraction scour in the stream bed of the main channel underneath the bridge could be estimated using Equation 1 by substituting the increase in discharge in the main channel under the bridge for Q_t which can be obtained using WSPRO. Furthermore, the width under the bridge in the main channel would be used for W_2.

The contraction scour in the overbank area under the bridge can be estimated using Equation 2 by substituting the ratio of the overbank flow upstream of the bridge to the overbank flow at the bridge for W_1/W_2. These discharges can also be obtained using WSPRO. It should be noted that Laursen's abutment scour equation for the case of the abutment set back from the main channel gives the sum of the contraction and local scour.

Case 2. Contraction Scour, No Overbank Flow. This case applies where there is no overbank flow, but the stream channel narrows either naturally or by the bridge abutments encroaching on the channel. Laursen's equation described for Case 1 using $Q_t = Q_c$ is applicable in this case. If the decrease in width W_2 is less than 10 percent, then neglect any width change.

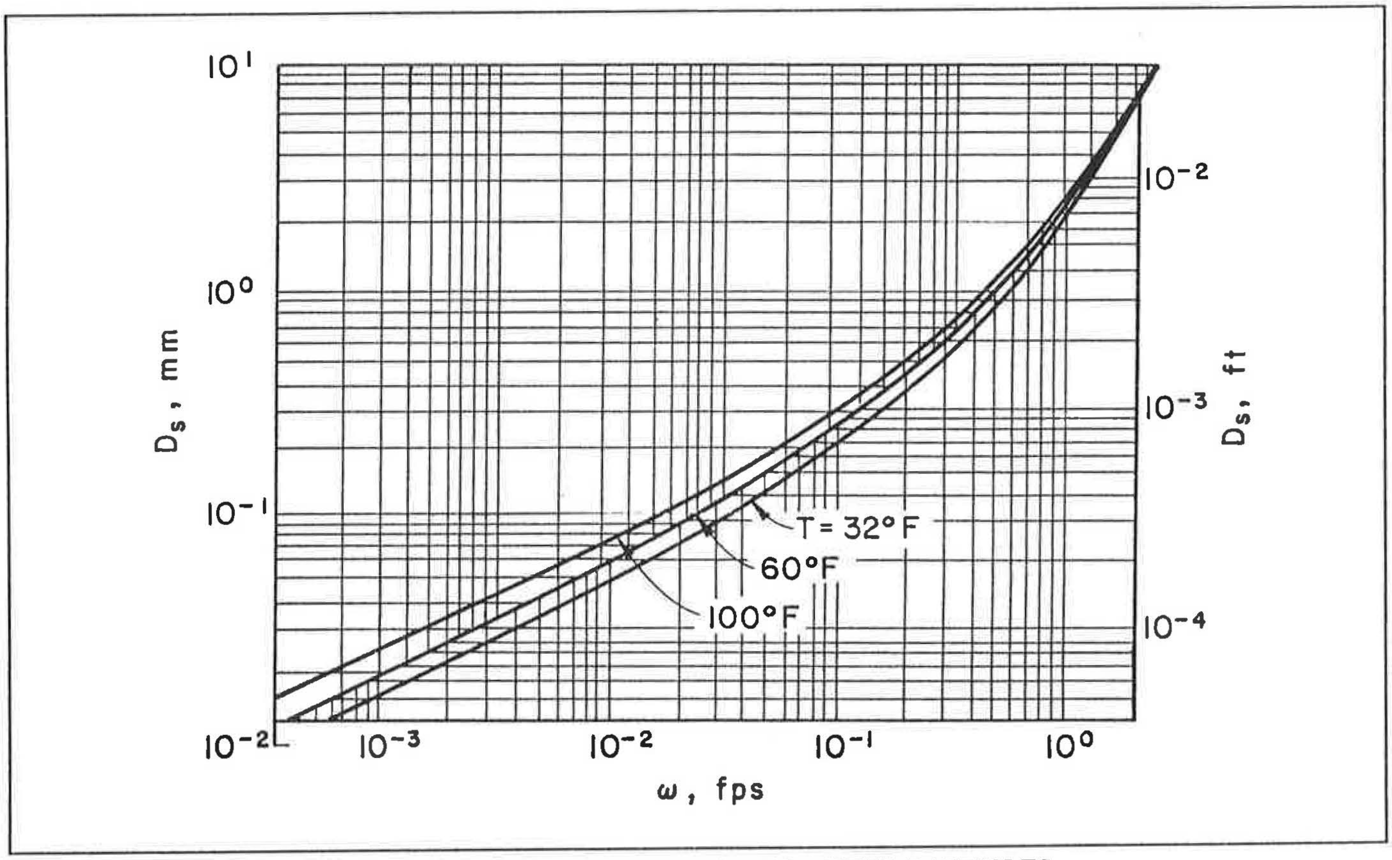

FIGURE 1. FALL VELOCITY OF SAND-SIZED PARTICLES

Case 3. Contraction Scour, Relief Bridge With No Bed-Material Transport. Case 3 applies to a relief bridge on a floodplain where there is no bed-material transport (i.e., clear-water scour). Laursen's [11] equation given below is applicable for this case.

$$\frac{y_2}{y_1} = \left(\frac{W_1}{W_2}\right)^{\frac{6}{7}} \left[\frac{V_1^{\,2}}{120 y_1^{\frac{1}{3}} D_{50}^{\frac{2}{3}}}\right]^{\frac{3}{7}} \qquad (2)$$

In Equation 2, most of the variables were defined previously. However, for this equation, the subscripts 1 and 2 refer to upstream conditions and to the width and depth at the relief bridge respectively. Furthermore:

W_1 = Width upstream of the relief bridge in feet. This can be estimated by assuming a point of stagnation between the main and relief bridge,

V_1 = Average velocity on the floodplain, in ft/s, one bridge length upstream of the river crossing,

D_{50} = Median diameter of the bed material, in feet, at the relief bridge.

Case 4. Contraction Scour, Relief Bridge With Bed-Material Transport. Case 4 applies to a relief bridge with bed-material transport. This case can occur when a relief bridge is over a secondary channel on the floodplain. It is recommended that Equation 1, which was described for Case 1 be used with appropriate adjustments of the variables.

Other Contraction Scour Considerations

Contraction scour resulting from variable water surface downstream of the bridge is analyzed by determining the lowest potential water surface elevation downstream of the bridge in so far as scour processes are concerned. It is recommended that a universally accepted water surface program such as the U.S. Corps of Engineers HEC 2 or the FHWA/USGS WSPRO (WSPRO) program be used to determine the flow variables such as velocity and depth through the bridge. With these variables, determine contraction and local scour depths.

Contraction scour resulting from the flow through the bridge being concentrated in one area is analyzed by determining the superelevation of the water surface on the outside of the bend and estimating the resulting velocities and depths through the bridge. The maximum velocity in the outer part of the bend can be 1.5 to 2 times the mean velocity. A physical model study can also be used to determine the velocity and scour depth distribution through the bridge for this case.

Estimating contraction scour for unusual situations involves particular skills in the application of principles of river mechanics to the specific site conditions, and such studies should be undertaken by engineers experienced in the fields of hydraulics and river mechanics.

Local Scour At Abutments

General

Most equations for predicting local scour at abutments, including those by Liu, et al [12], Laursen [11] and Froehlich [13] are based, almost entirely, on laboratory data. The lack of field data for derivation or verification of abutment

scour inhibits the accuracy and reliability of these equations. Still, these equations represent the best available technology at the present time.

Liu et al's [12], equations were developed by dimensional analysis of the variables, and a best-fit line was drawn through the laboratory data. Laursen's equations [11] are based on inductive reasoning of the change in transport relations due to the acceleration of the flow caused by the abutment. Froehlich's equations [13], are derived from regression analysis of the available laboratory data. Because of the manner that these equations were developed, and their inherent uncertainties, they are conservative and should be used to consider the worst-case condition of abutment scour.

Because of the methods by which experiments were conducted to obtain these equations, they will only predict the maximum scour that could occur for an abutment projecting out in a stream if the velocities and depth upstream of the abutment are similar to those in the main channel. However, the way the experiments were conducted do not represent many of the conditions typically encountered in the field. Field conditions may have tree lined or vegetated banks with low velocities and shallow depths upstream of the abutment. With overland flow, depths are often shallow in the overbank, with low velocities and little to no bed-material transport.

Because of the discrepancies between the laboratory data and field conditions, engineering judgement is required. In many cases, foundations can be designed with shallower depths than given by the equations provided the foundations are protected with riprap placed below the streambed. Alternatively, a spur dike (guide bank) placed upstream of the abutment can be used instead of riprap.

Abutment Site Conditions

Abutments can be set back from the natural streambank or can project out into the flow. They can have various shapes, and can be set at an angle to the flow. Scour at abutments can be live-bed or clear-water scour. Finally, there can be varying amounts of overbank flow that are intercepted by the approaches to the bridge and returned to the stream at the abutment. All of these factors affect abutment scour.

Scour at abutments can be divided into seven cases. These cases are given in Table 2. However only Laursen [11] has developed equations for all these cases. His equations are based on transport relations, but have not as yet been fully verified in the field. Laursen's equations, as well as Liu, et al's [12], Froehlich's [13] and one presented in Highways in the River Environment (HIRE [10]) for case 6, are given in FHWA [10] and HEC-18 [6].

Abutment Shape

There are two general shapes for abutments. These are vertical wall abutments, with or without wing walls, and spill-through abutments. The depth of scour is about double for vertical wall abutments without wing walls, as compared with spill-through abutments (see Liu et al's equation in HIRE [10] or Froehlich's correction term for abutment shape given later in this section).

Design for Scour at Abutments

It is recommended that abutment foundation depths be set by AASHTO [14] and [15] standards, and that they be protected with riprap designed according to HEC-11 [16] or by the methods given in HIRE [10], and/or be protected by spur dikes designed as per FHWA instructions. The reason for this recommendation is as stated at the start of this section. That is, the available equations give the worst-case scour, and there little to no field verification of these equations.

As a check on the potential depth of scour to aid in the design of the foundation and placement of riprap or spur dikes, the equation for live-bed scour developed by Froehlich [13] (Equation 3) is recommended. This equation was developed after analysis of 170 live-bed scour measurements in laboratory flumes.

$$\frac{y_s}{y_a} = 2.27 K_1 K_2 \left(\frac{a'}{y_a}\right)^{0.43} Fr_e^{\,0.61} + FS \qquad (3)$$

where

y_a = Depth of Flow at the Abutment

y_s = Abutment Scour Depth

K_1 = Coefficient for Abutment Shape. For vertical abutments, $K_1 = 1.0$, for vertical abutments with wing walls, $K_1 = 0.82$, for spill-through abutments, $K_1 = 0.55$.

$K_2 = \left(\frac{\theta}{90}\right)^{0.13}$ Which is a Coefficient for angle of embankment to flow. Where Θ is equal to the angle between the approach embankment and a line drawn normal to the main flow. As a convention, Θ is less than 90 degrees if the embankment points downstream and greater than 90 degrees if the embankment points upstream.

$a' = \frac{A_e}{y_1}$ = Projected length of Abutment normal to the flow. For this variable, A_e is the flow area of the cross-section which is obstructed by the embankment.

$Fr_e = V_e/(g y_1)^{0.5}$ = Froude number of the approach flow upstream of the abutment; $V_e = Q_e/A_e$ and;

Q_e = The flow obstructed by the abutment and approach embankment.

The factor of safety, FS, is given in Table 3 and is derived based on the measured laboratory abutment scour. As an illustrative example, a factor of safety of 1.0, 98 percent of the measured laboratory abutment scour depths were less than that predicted using Froehlich's equation.

TABLE 2 SUMMARY OF ABUTMENT SCOUR CASES

CASE	ABUTMENT LOCATION	OVERBANK FLOW	$\frac{a}{y_1}$	BED LOAD	ABUTMENT TYPE
1	Projects Into Channel	No	< 25	Live Bed Clear Water	Vertical Wall or Spill through
2	Projects Into Channel	Yes	< 25	Live Bed Clear Water	Vertical Wall or Spill through
3	Set Back from Main Channel	Yes	< 25	Clear Water	Vertical Wall or Spill through
4	Relief Bridge on Floodplain	Yes	< 25	Clear Water	Vertical Wall or Spill through
5	At Edge of Main Channel	Yes	< 25	Live Bed	Vertical Wall or Spill through
6	Not Designated	Yes	> 25	Not Designated	Spill through
7	Skewed to Stream	---	---	---	---

From personal contact with Froehlich, he has suggested that the abutment scour depth be increased by one sixth of y_a if there are dunes in the main channel upstream of the abutment. From the authors experience and from Jain and Fischer's [19] research we recommended that no increase in the abutment scour depth for either plain-bed or antidune regimes.

TABLE 3 FACTOR OF SAFETY FOR ABUTMENT SCOUR

FS	PERCENT OF LABORATORY SCOUR DATA LESS THAN PREDICTED USING EQUATION
0.00	44.6%
0.25	72.4%
0.50	91.4%
0.75	95.2%
1.00	98.0%
1.50	99.4%
2.00	99.4%
2.50	100.0%

Clear-water Scour at an Abutment Froehlich [13] also developed a clear-water abutment scour equation based on dimensional analysis and multiple regression analysis of 164 clear-water scour measurements in laboratory flumes. However, it is not recommended for use because the potential decrease in scour depth at abutments resulting from coarser material is not known. Although not recommended, this equation is given in HIRE [10] AND HEC-18 [6], and may be useful for use by researchers.

Local Scour At Piers

General

Local scour at piers is a function of bed-material size, flow characteristics, fluid properties and the geometry of the pier. There is also some evidence that pile caps, pile groups, or exposed footings influence local scour at piers. The subject has been studied extensively in the laboratory, but there is only limited field data. As a result of these many studies, there are an equal amount of equations. In general, the equations are for live-bed scour in cohesionless, sand-bed streams.

For the determination of pier scour, HEC-18 [6] and the Technical Advisory [1] recommends the use of the Colorado State University equation for both live-bed and clear-water scour. With a dune-bed configuration, the equation predicts equilibrium scour depth. Therefore, when dunes are present, the maximum scour will be 30% greater than that predicted by the CSU equation. For flow with plane-bed configuration or antidunes, from analysis of Jain and Fischer's [19] research, CSU's equation will predict the maximum scour. Furthermore, from a study of Jain and Fischer's [19] research and from the author's field and laboratory experience with plane-bed and antidune flow, the trough of the antidune is not a factor when considering local scour at piers.

CSU's equation, as with most equations, does not take into account the possibility that larger sizes in the bed material could armor the scour hole. Raudkivi and others [2,3,4 and 5] developed equations that take into consideration large particles armoring the bed. However, the significance of armoring of the scour hole over a long time frame and over many floods is not known. Therefore, equations which consider large bed-material armoring are not recommended for use at this time. However, for the researchers needs an equation based on Raudkivi's work is given in HIRE [10], HEC 18 [6] and Richardson, and Richardson [7].

Computing Pier Scour

The Colorado State University's equation (Richardson et al) [10] is as follows:

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{\;0.43} \qquad (4)$$

where

y_1 = Depth of Flow directly upstream of the pier,

y_s = Scour Depth at the pier,

K_1 = Correction coefficient for pier type (Table 4),

K_2 = Correction coefficient for angle of piers skewed to the flow (Table 5),

a = Pier width.

$Fr_1 = V_1/(gy_1)^{0.5}$ = Froude number of the approach flow upstream of the pier using, the velocity in front of the pier for V_1.

The subscript 1 used for variables in the CSU equation refers to the depth and velocity in the flow zone in front of the pier. These values will be larger than the average stream velocity in most cases. In a straight, uniform channel, V_1 would probably be about 20% greater than the average velocity in the thalweg. In bends, V_1 can be as much as 50% to 100% greater than the average velocity in the thalweg.

When piers are not skewed to the flow, $K_2 = 1$ and K_1 can be determined using values tabulated in Table 4. For skewed piers, no correction for pier shape should be attempted. Therefore, for skewed piers set $K_1 = 1$ and select K_2 from values tabulated in Table 5. Note that the value of K_2 depends on the ratio of the pier length, L, to the width of the pier, a.

TABLE 4 K_1, FOR PIER TYPE

Pier Type	K_1
Square Nose	1.1
Round Nose	1.0
Cylinder	1.0
Sharp Nose	0.9
Group of Cylinders	1.0

TABLE 5 K_2, FOR ANGLE OF ATTACK

Angle	$\frac{L}{a} = 4$	$\frac{L}{a} = 8$	$\frac{L}{a} = 12$
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.5	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

For a group of cylinders skewed to the flow, the scour depth depends on the spacing of the piers. If the spacing was zero, meaning the piers were touching, the scour would resemble a single solid pier. However, for piers spaced further apart, there is no good method to estimate the scour depth. Reference is made here to Raudkivi [3], who in discussing effects of alignment states *...the use of cylindrical columns would produce a shallower scour; for example, with five-diameter spacing the local scour can be limited to about 1.2 times the local scour at a single cylinder...* It is clear from this discussion that when groups of columns are skewed to the flow, engineering judgement must be employed in determining the maximum pier scour.

For quick estimation of scour depths, the Technical Advisory [1] recommends equations based upon the CSU equation. Equation 5 is valid for Froude numbers less than or equal to 0.9, and Equation 6 is recommended for Froude numbers larger than 0.9. Note that these equations are identical, except for the coefficient.

$$y_{s_{max}} = 2.3 K_2 a \tag{5}$$

$$y_{s_{max}} = 2.9 K_2 a \tag{6}$$

Pier Scour for Exposed Footings

Pier footings or pile caps can become exposed to the flow by scour. This may occur either from long-term degradation, contraction scour, local scour, or lateral shifting of the stream. In practice, computations of local pier scour depths for exposed footings uses the footing width instead of the pier width to determine the scour depth. This practice is usually too conservative. For example, calculations of scour depths for the Schoharie Cr. bridge failure were closer to the measured model and prototype scour depths when pier width was used rather than footing width [17]. In this case, where the footing top was at the elevation of the bed surface, the calculated depths using the footing width were 47% larger than the measured scour depths.

Furthermore, a recent model study of scour at the Acosta bridge at Jacksonville, Florida, by Jones [8] found that when the top of the footing was flush with the stream, bed local scour was 20 percent less than when; 1) the bottom of the footing was at the bed surface; 2) when the top of the footing was at the water surface with the pile group exposed; and 3) the top of the footing protruded vertically up into the flow approximately half the flow depth. In a generalized study, he found that a footing with a lip extending upstream of the pier, reduced pier scour when the top of the footing was located flush or below the bed. However scour depths were deeper and larger in proportion to the extent that this footing projected up into the flow field. It is clear that stream degradation can expose footings if they are not placed deep enough below the streambed. For new bridges, it is recommended that, as a minimum, the top of footings be set a minimum of three feet below the sum of the long-term bed degradation and the contraction scour.

Based on this study, Jones states:

"It is recommended that the pier width be used for the value of ' a ' in the pier scour equations if the top of the footing is at or below the streambed (taking into account contraction scour). If the pier footing extends above the streambed, make a second computation using the width of the footing for the value of a and the depth and average velocity in the flow zone obstructed by the footing for the ' y ' and ' V ' respectively in the scour equation. Use the larger of the two scour computations"

To determine ' V ' obstructed by the flow, Jones recommends Equation 7. Using this approach, the values of V_f and y_f would be used in the CSU equation presented previously.

$$\frac{V_f}{V_1} = \frac{\ln\left(\frac{10.93 y_f}{k_s} + 1\right)}{\ln\left(\frac{10.93 y_1}{k_s} + 1\right)} \tag{7}$$

Where:

V_f = Average velocity in the flow zone below the top of the footing.

y_f = Distance from the bed to the top of the footing.

k_s = The grain roughness of the bed (usually the D_{84} of the bed material).

Pier Scour for Exposed Pile Groups

Jones [8] also conducted experiments to determine guidelines for specifying the characteristic width of a pile group that is or may be exposed to the flow when the cylinders are spaced laterally as well as longitudinally in the stream flow. He concluded the following:

Pile groups that project above the stream bed can be analyzed conservatively by representing them as a single pier width equal to the projected area of the piles ignoring the clear spaces between piles. Good judgment needs to be used in accounting for debris because pile groups tend to collect debris that could effectively clog the clear spaces between piles and cause the pile group to act as a much larger mass."

Pressure Flow Scour

Pressure flow at a bridge occurs when bridge decks intersect the flow or are submerged. Flume studies at Colorado State University were conducted in the spring of 1990 with a bridge deck partly submerged and a single pier in the flume. For this study, the independent parameters were the distance from the streambed to the bridge deck and the flow velocity. There was no sediment transport upstream of the bridge (clear-water scour). Without the deck submerged, there was no contraction scour and only local scour occurred. With the deck submerged, there was contraction scour and scour depths at the pier were increased by a factor of two or three. The magnitude of the contraction and local scour was as expected, and depended on the velocity of the approach flow and the distance from the deck to the bed. For the same approach velocity, contraction scour and local pier scour increased as the distance from the bed to the deck decreased. These results are preliminary and are only now being analyzed. This research is the subject of a dissertation by Ms. Lila Abed, which should be completed in the near future.

Width of Scour Holes

The top width of a scour hole in cohesionless bed material from one side of a pier or footing can be estimated from the angle of repose of the material (Equation 8).

$$W = y_s(K + Cotan\theta) \tag{8}$$

where

W = The width of the scour hole measured from the side of the pier to the lateral extent of the hole.

y_s = Scour depth.

K = Bottom width of the scour hole as a fraction of the scour depth.

θ = Angle of repose of the bed material (ranges between 30 to 44 degrees)

From the above equation, using K equal to 0, for the range of repose angles, the width of the scour hole will be between 1.1 to 1.8 times the depth of scour. If the bottom width of the scour hole was equal to the scour depth, the range of widths would be between 2.1 to 2.8 times the depth of scour. These extremes will encompass most of the scour widths encountered at bridge piers.

Total Scour Depth And Design Evaluation

Total Scour Depth Plots

Abutment and foundation design parameters and limits can be determined after determining the various scours, which are discussed in previous sections. For this step in the design, it is recommended that the estimate of 1) long-term bed elevation change; 2) contraction scour; and 3) local scour at the piers and abutments be plotted on a cross-section of the stream channel and floodplain at the bridge crossing. It is also recommended that a distorted scale be used so that the total scour can be easily evaluated. Make a sketch of any plan form changes (lateral stream channel movement due to meander migration, etc) that might also be reasonably expected to occur. The plotting of the individual scour depths can be envisioned in the following recommended order.

- Long-term elevation. Note that these may involve aggradation or degradation.

- Contraction scour. Plot these from the aggradation or degradation lines which were determined in the previous step.

- Local scour is then plotted from and below the combined aggradation and contraction scour lines.

- It is important to plot the scour-hole width at abutments and piers, as well as the total depth of scour.

Evaluate The Total Scour Depth

After construction of the total scour depth plots, it is essential to evaluate them. In some cases the results of this step will require the designer to go back and reevaluate the estimations of one or more scour-depth computations discussed previously in this text. The following is a list of many aspects of this evaluation, which may cause to designer to go back and reevaluate various aspects of the design.

- Are the scour depths reasonable and consistent with previous experience?

- Do the local scour holes from the piers or abutments intersect or come close to each other between spans? If so, Method 1 should be considered for the scour analysis. The length of the bridge opening may need to be reevaluated, increasing the opening size, or decreasing the number of piers as necessary.

- Are there additional factors (lateral movement of the stream, scour-hole armoring, stream-flow hydrograph, velocity and discharge distribution, moving of the thalweg, shifting of the flow direction, channel changes, type of stream, etc.) to be considered?

- Do the calculated scour depths appear too deep for the conditions in the field relative to the laboratory conditions? Remember, the abutment scour equations were for the worst-case conditions. Would riprap or spur dikes (guide bank) be a better, more cost-effective solution?

- Evaluate cost, safety, and any other aspect relating to the design. Also, has the influence of debris on scour been adequately accounted for?

- Consider whether countermeasures may be needed. If they are needed, determine whether their use is a cost-effective method of alleviating hazards to the bridge. In some cases they can be incorporated into the design at a later date to prevent total failure of the support system in the future.

Reevaluate The Bridge Design

Reevaluate the bridge design on the basis of the foregoing scour analysis. Revise the design as necessary. This evaluation should consider:

- If the contraction scour is too large, the waterway area may need to be increased.

- If the scour holes from the piers overlap with each other, or with the abutment scour holes, then the number of piers may need to be decreased. Also the alignment of the piers may be modified to minimize pier scour.

- If abutment scour or contraction scour is excessive, relief bridges may be needed to pass the flood flows and to decrease the amount of overland flow returning to the main channel. Alternatively, and in some cases, the main bridge opening can be widened. In some cases, bridge widening and relief bridges may be necessary.

- Check whether the bridge abutments and piers are properly aligned in regards to the stream channel and the flood plain. Adjust as necessary.

- In some cases the bridge crossing of the stream and the flood plain may not be in a desirable location. If the location presents problems, see if it can be realigned or relocated. If not, ascertain whether river-training works, guide banks or relief bridges could be used to develop an acceptable flow pattern at the bridges.

- Determine whether the hydraulic study is adequate to provide the necessary information for foundation design. If the flow patterns are complex, a two-dimensional water surface profile model may be required to adequately describe the flow conditions and provide parameters for the estimation of scour. In some cases, based partly on the costs of an acceptably safe foundation, a physical model study may be required as well.

SUMMARY AND CONCLUSIONS

Equations and methods for predicting and evaluating total scour at bridges for design purposes are presented in this paper. These procedures and recommendations are based on the 1988 Technical Advisory on Bridge Scour and on HEC-18, titled "Evaluating Scour at Bridges". Both publications have been funded and published by the Federal Highway Administration.

In summary, equations derived by Laursen [9 &11], CSU [10], and Froehlich [13] are recommended for computation of contraction, pier, and abutment scour, respectively. It should be noted that the recommended abutment scour equations were developed with little to no field data. Therefore, the estimations of abutment scour represent the maximum scour that might occur at abutments. Because of this, it is recommended that abutments be set at minimal depths and protected with riprap and/or spurs.

The influence of pier footings of local scour at piers was also discussed in this paper. For piers with footings at or below the bed level, local scour depths are decreased. For footings above the bed level, the local scour is increased and depends on the degree of footing penetration into the flow. A method originally determined by Jones [8] for determining local scour for this case was reiterated in this report. Jones [8] also has shown that pile groups that are exposed to the flow can be also be analyzed.

Recent research at Colorado State University on the effect of pressure flow caused by submerged bridge decks show a significant increase in local and contraction scour. Preliminary results indicate that this increase can be as high as two to three times the contracted or local scour depth. In general the degree of increase depends on the approach velocity and the distance from the deck to the bed of the channel.

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Stepwise Procedure for Evaluating Stream Stability

JAMES D. SCHALL AND PETER F. LAGASSE

ABSTRACT

Systematic analytical procedures for evaluation of stream stability typically are not readily available to the highway engineer. To remedy this, the purpose of this paper is to outline a stepwise analytical procedure that may be utilized to evaluate stream stability. The procedure incorporates three levels of analysis of increasing complexity and is somewhat similar to a three-part approach that was used in the early 1980's when the authors were employed at Simons, Li and Associates. Through the efforts of many, that procedure evolved into a valuable tool for stream stability analysis. For the FHWA Hydraulic Engineering Circular No. 20, "Stream Stability at Highway Structures," the three level approach was further refined and organized to include development of flow charts to assist in evaluation of stream stability at bridge crossings.

The completion of each level of analysis results in an assessment of stream stability. Then, depending on the complexities of stream response or bridge design, the decision to proceed to the next level of analysis is made. The specific steps considered typical of Level 1 and Level 2 type analyses are outlined in this paper. The overall procedure provides a systematic approach to evaluation of stream stability at highway crossings, and will facilitate completion of such analyses by highway engineers.

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INTRODUCTION

A stable channel does not change in size, form, or position with time; however, all alluvial channels change to some extent and are somewhat unstable. For highway engineering purposes, a stream channel can be considered unstable if the rate or magnitude of change is great enough that the planning, location, design, or maintenance considerations for a highway encroachment are significantly affected.

Local instability caused by the construction of a highway crossing or encroachment on a stream is also of concern. This includes general scour caused by contraction of the flow, and local scour due to the disturbance of streamlines at an object in the flow, such as at a pier or an abutment.

Systematic, analytical procedures for evaluation of stream stability are not readily available to the highway engineer. The purpose of this paper is to outline a stepwise analytical procedure for evaluation of stream stability that will assist highway engineers responsible for completing such analyses.

GENERAL SOLUTION PROCEDURE

The inherent complexities of stream stability, further complicated by highway stream crossings, requires a multilevel solution procedure. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternative solutions. This analysis should be followed with quantitative analyses using basic

hydrologic, hydraulic, and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including water surface profile analysis), and basic sediment transport analyses such as evaluation of watershed sediment yield, incipient motion analysis, and scour calculations. These analyses can be considered adequate for many locations if the problems are resolved and the relationships between different factors affecting stability are adequately explained. If not, a more complex quantitative analysis based on detailed mathematical modeling and/or physical hydraulic models should be considered.

In summary, the general solution procedure for analyzing stream stability could involve the following three levels of analysis:

Level 1: Application of Simple Geomorphic Concepts and other Qualitative Analyses
Level 2: Application of Basic Hydrologic, Hydraulic, and Sediment Transport Engineering Concepts
Level 3: Application of Mathematical or Physical Modeling Studies

Level 1: Qualitative and Other Geomorphic Analyses

A flow chart of the typical steps in qualitative and other geomorphic analyses is provided in Figure 1. The six identified steps are generally applicable to most stream stability problems. These steps are discussed in more detail in the following paragraphs. As shown in Figure 1, the qualitative evaluation leads to a conclusion regarding the need for a more detailed Level 2 analysis or a decision to proceed directly to selection and design of countermeasures based only on qualitative and other geomorphic analyses.

Step 1. Define Stream Characteristics

The first step in stability analysis is to identify stream characteristics. Figure 2 illustrates a number of geomorphic factors that may assist in stream classification. Defining the various characteristics of the stream according to this scheme provides insight into stream behavior and response, and information on impacting activities in the watershed. A good way to utilize Figure 2 is to simply circle the descriptions that best describe the stream characteristics. This can be completed with very little information, as in an office review of mapping or aerial photography, or after a complete field reconnaissance.

Step 2. Evaluate Land Use Changes

Water and sediment yield from a watershed is a function of land-use practices. Thus, knowledge of the land use and historical changes in land use is essential to understanding conditions of stream stability and potential stream response to natural and man-induced changes.

The presence or absence of vegetative growth can have a significant influence on the runoff and erosional response of a fluvial system. Large scale changes in vegetation resulting from fire, logging, land conversion and urbanization can either increase or decrease the total water and sediment yield from a watershed. For example, fire and logging tend to increase water and sediment yield, while urbanization promotes increased water yield and peak flows, but decreased sediment yield from the watershed. Urbanization may increase sediment yield from the channel.

Information on land use history and trends can be found in Federal, State and Local government documents and reports (i.e., census information, zoning maps, future development plans, etc.). Additionally, analysis of historical aerial photographs can provide significant insight on land use changes. Land use change due to urbanization can be classified based on estimated changes in pervious and impervious cover. Changes in vegetative cover can be classified simply as no change, vegetation increasing, vegetation damaged, and vegetation destroyed. The relationship or correlation between changes in channel stability and land use can contribute to a qualitative understanding of system response mechanisms.

LEVEL 1: QUALITATIVE ANALYSES

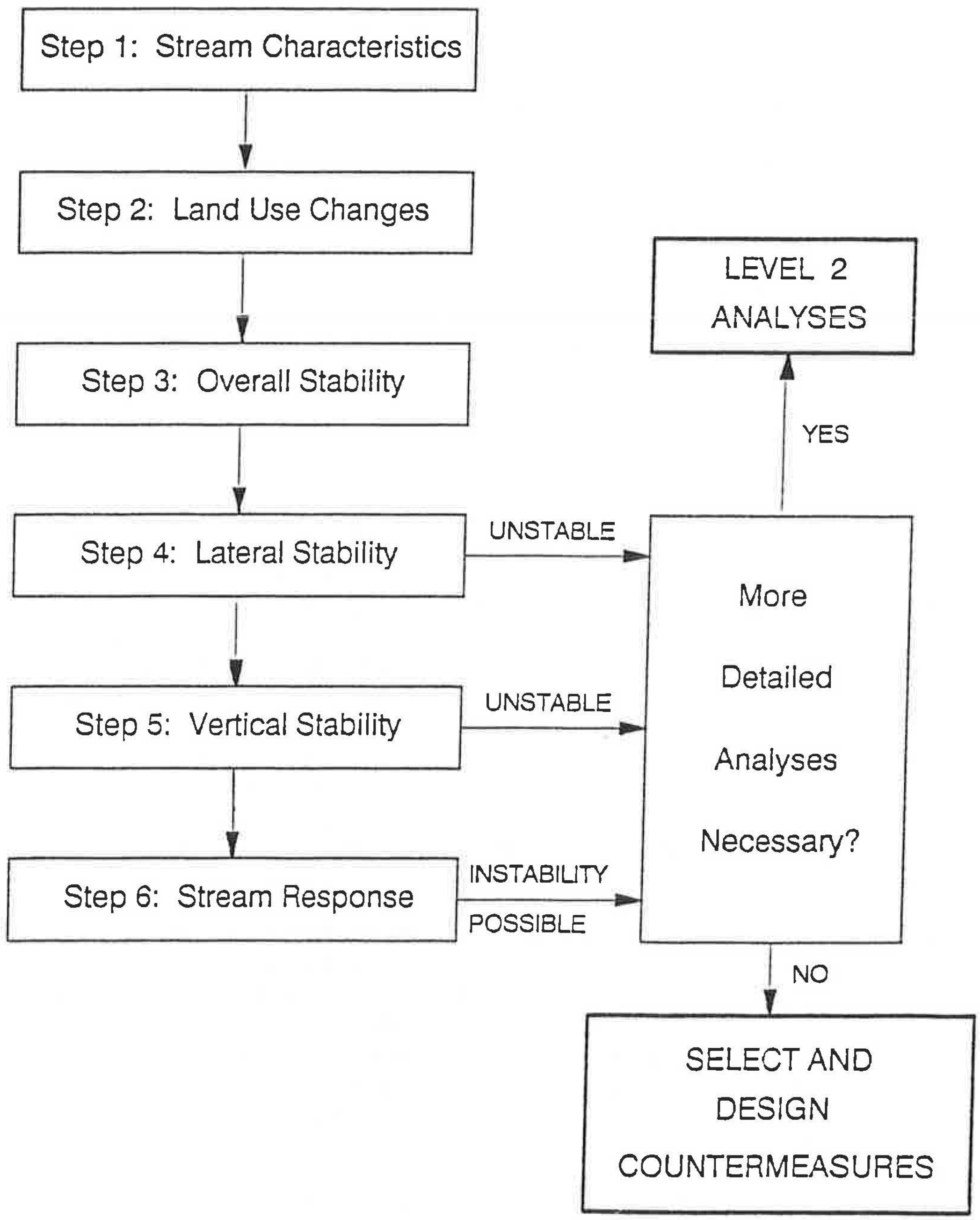

Figure 1. Flow chart for Level 1: Qualitative Analyses.

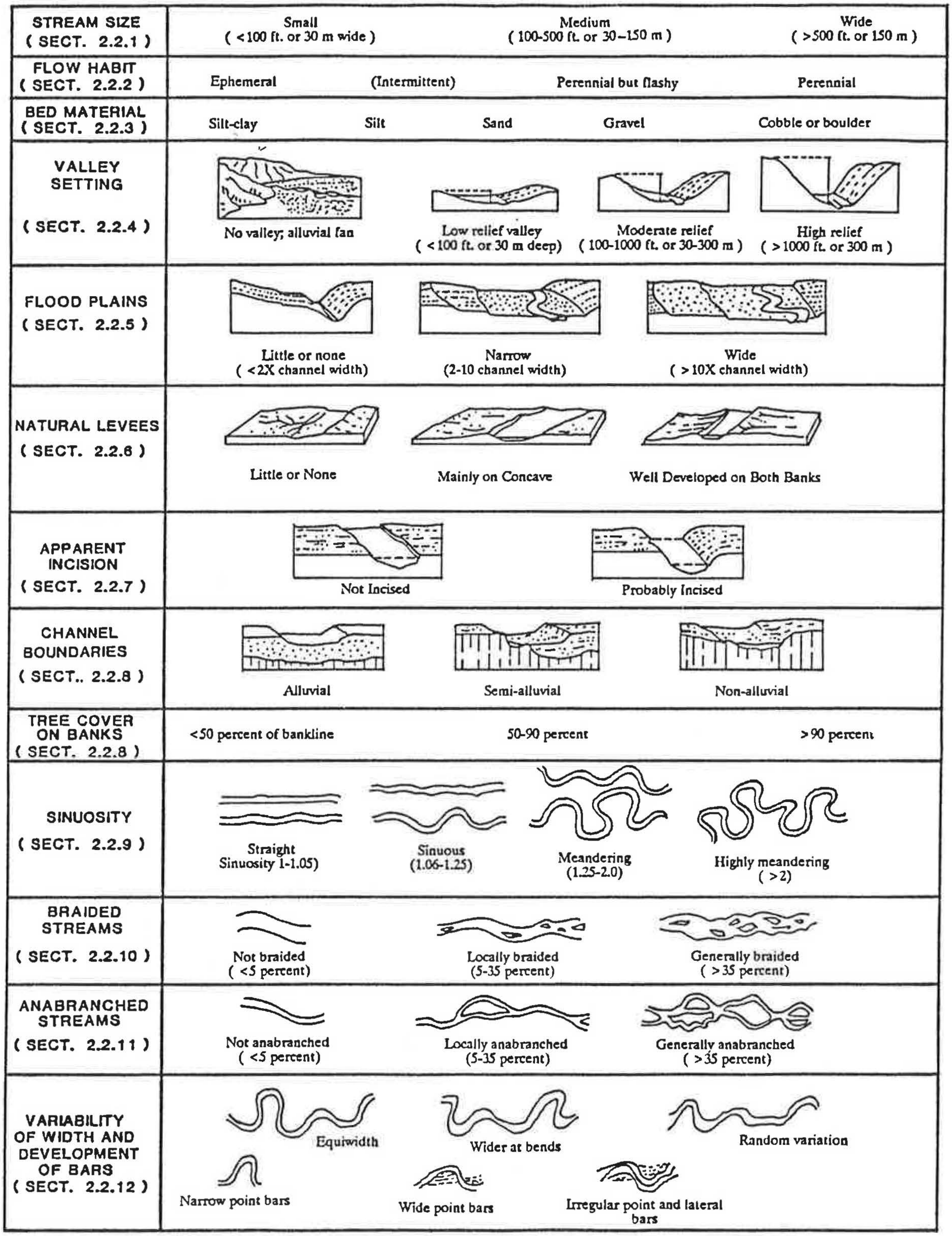

Figure 2. Geomorphic factors that affect stream stability

Step 3. Assess Overall Stream Stability

Table 1 summarizes possible channel stability interpretations according to the geomorphic factors identified in Figure 2, as well as additional factors that commonly influence stream stability. Figure 3 is also useful in making a qualitative assessment of stream stability based on stream characteristics. It shows that straight channels are relatively stable only where flow velocities and sediment load are low. As these variables increase, flow meanders in the channel, causing the formation of alternate bars and the initiation of a meandering channel pattern. Similarly, meandering channels are progressively less stable with increasing velocity and bed load. At high values of these variables, the channel becomes braided. The presence and size of point bars and middle bars are indications of the relative lateral stability of a stream channel.

Step 4. Evaluate Lateral Stability

The effects of the lateral instability of a stream at a bridge are dependent on the extent of the bank erosion and the design of the bridge. Bank erosion can undermine piers and abutments located outside the channel and erode abutment spill slopes or breach approach fills. Where bank failure is by a rotational slip, lateral pressures on piers located within the slip zone may cause cracks in piers or piling or may displace pier foundations. Migration of a bend through a bridge opening changes the direction of flow through the opening so that a pier designed and constructed with a round-nose acts as a blunt-nosed, enlarged obstruction in the flow, thus accentuating local and general scour. Also, the development of a point bar on the inside of the migrating bend can increase contraction at the bridge if the outside bank is constrained from eroding.

A field inspection is a critical component of a qualitative assessment of lateral stability. A comparison of observed field conditions with descriptions of stable and unstable channel banks helps qualify bank stability. Similarly, field observations of bank material, composition, and existing failure modes can provide insight on bank stability.

Lateral stability assessment can also be completed from existing records of the position of a bend at two or more different times. Aerial photographs or maps are usually the only records available. Surveyed cross sections are extremely useful although rarely available. Some progress is being made on the numerical prediction of loop deformation and bend migration (Level 3 type analyses). At present however, the best available estimates are based on past rates of lateral migration at a particular reach. In using these estimates, it should be recognized that erosion rates may fluctuate substantially from one years-long period to the next.

Step 5. Evaluate Vertical Stability

The typical effects associated with gradation (bed elevation) changes at highway bridges are erosion at abutments and the exposure and undermining of foundations with degradation. Aggradation, a reduction in flow area under bridges resulting in more frequent flow over the highway, may also occur. Bank caving associated with degradation poses the same problems at bridges as lateral erosion from bend migration, but the problems may be more severe because of the lower elevation of the streambed. Aggrading stream channels also tend to become wider as aggradation progresses, eroding floodplain areas and highway embankments on the floodplain. The location of the bridge crossing upstream, downstream, or on tributaries may cause gradation problems.

Brown, et al. reported that serious problems at degradation sites are about three times more common than at aggradation sites.(5) This is a reflection of the fact that degradation occurs more frequently than aggradation, and also that aggradation does not endanger the bridge foundation. It does not, however, indicate that aggradation is not a serious problem in some areas of the United States.

Problems other than those most commonly associated with degrading channels include the undermining of cutoff walls, other flow-control structures, and bank protection. Bank sloughing resulting

Table 1. Interpretation of observed data.

OBSERVED CONDITION	CHANNEL RESPONSE STABLE	UNSTABLE	DEGRADING	AGGRADING
Alluvial Fan \1				
Upstream		X		X
Downstream		X	X	
Dam and Reservoir				
Upstream		X		X
Downstream		X	X	
River Form				
Meandering	X	X	Unknown	Unknown
Straight		X	Unknown	Unknown
Braided		X	Unknown	Unknown
Bank Erosion		X	Unknown	Unknown
Vegetated Banks	X		Unknown	Unknown
Head Cuts		X	X	
Diversion				
Clear water diversion		X		X
Overloaded w/sediment		X	X	
Channel Straightened		X	X	
Deforest Watershed		X		X
Drought Period	X			X
Wet Period		X	X	
Bed Material Size				
Increase		X		X
Decrease		X	Unknown	X

\1The observed condition refers to location of the bridge on the alluvial fan, i.e., on the upstream or downstream portion of the fan.

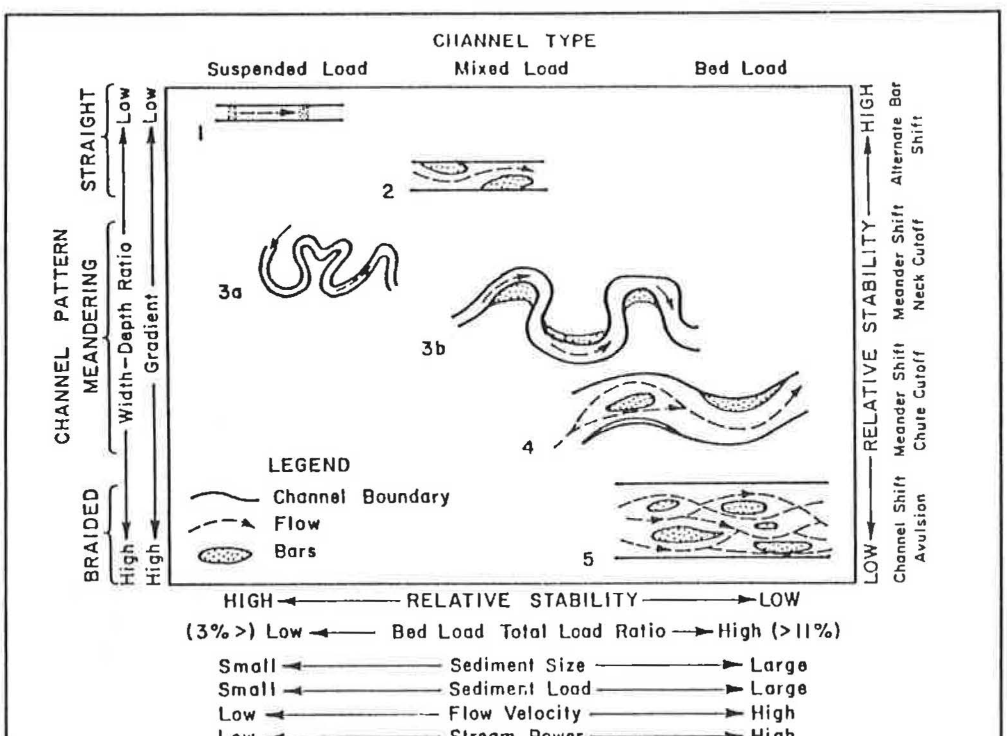

Figure 3. Channel classification and relative stability as hydraulic factors are varied

from degradation often greatly increases the amount of debris carried by the stream and increases the hazard of clogged waterway openings and increased scour at bridges. The hazard of local scour becomes greater in a degrading stream because of the lower streambed elevation.

Aggradation in a stream channel increases the frequency of backwater that can cause damage. Bridge decks and approach roadways become inundated more frequently, disrupting traffic, subjecting the superstructure of the bridge to hydraulic forces that can cause failure. In addition, approach roadways become subject to overflow that can erode and cause failure of the embankment. Where lateral erosion or increased flood stages accompanying aggradation increases the debris load in a stream, the hazards of clogged bridge waterways and hydraulic forces on bridge superstructures are increased.

Data records for at least several years are usually needed to detect gradation problems. This is due to the fact that the channel bottom often is not visible and changes in flow depth may indicate changes in the rate of flow rather than changes in gradation. Gradation changes normally develop over long periods of time even though rapid change can occur during an extreme flood event. The data needed to assess gradation changes include historic streambed profiles and long-term trends in stage-discharge relationships. Occasionally, information on bed elevation changes can be gained from a series of maps prepared at different times. Bed elevations at railroad, highway and pipeline crossings monitored over time may also be useful. On many large streams, the long-term trends have been analyzed and documented by agencies such as the U.S. Geological Survey and the U. S. Army Corps of Engineers.

Step 6. Evaluate Channel Response to Change

The knowledge and insight developed from evaluation of present and historical channel and watershed conditions, as developed above through Steps 1-5, provides an understanding of potential channel response to previous impacts and/or proposed changes, such as construction of a bridge. Additionally, the application of simple, predictive geomorphic relationships, such as the Lane Relationship can assist in evaluating channel response mechanisms.(4)

Level 2: Basic Engineering Analyses

A flow chart of the typical steps in basic engineering analyses is provided in Figure 4. The flow chart illustrates the typical steps to be followed if a Level 1 qualitative analysis results in a decision that Level 2 analyses are required (Figure 1). The eight basic engineering steps are generally applicable to most stream stability problems and are discussed in more detail in the paragraphs which follow. The basic engineering analysis steps lead to a conclusion regarding the need for more detailed Level 3 analysis or a decision to proceed to selection and design of countermeasures without more complex studies.

Step 1. Evaluate Flood History and Rainfall-Runoff Relations

A detailed discussion of hydrologic analysis techniques, and the analysis of flood magnitude and frequency in particular, is presented in HEC-19 (6) and will not be repeated here. However, several hydrologic concepts of particular significance to evaluation of stream stability are summarized.

Consideration of flood history is an integral step in attempting to characterize watershed response and morphologic evolution. Analysis of flood history is of particular importance to understanding arid region stream characteristics. Many dryland streams flow only during the spring and immediately after major storms. For example, Leopold, et al. (7) found that arroyos near Santa Fe, New Mexico flow only about three times a year. As a consequence, dryland stream response can be considered to be more hydrologically dependent than the response of streams located in a humid environment. Whereas the passage of time may be sufficient to cause change in a stream located in a humid environment, time alone, at least in

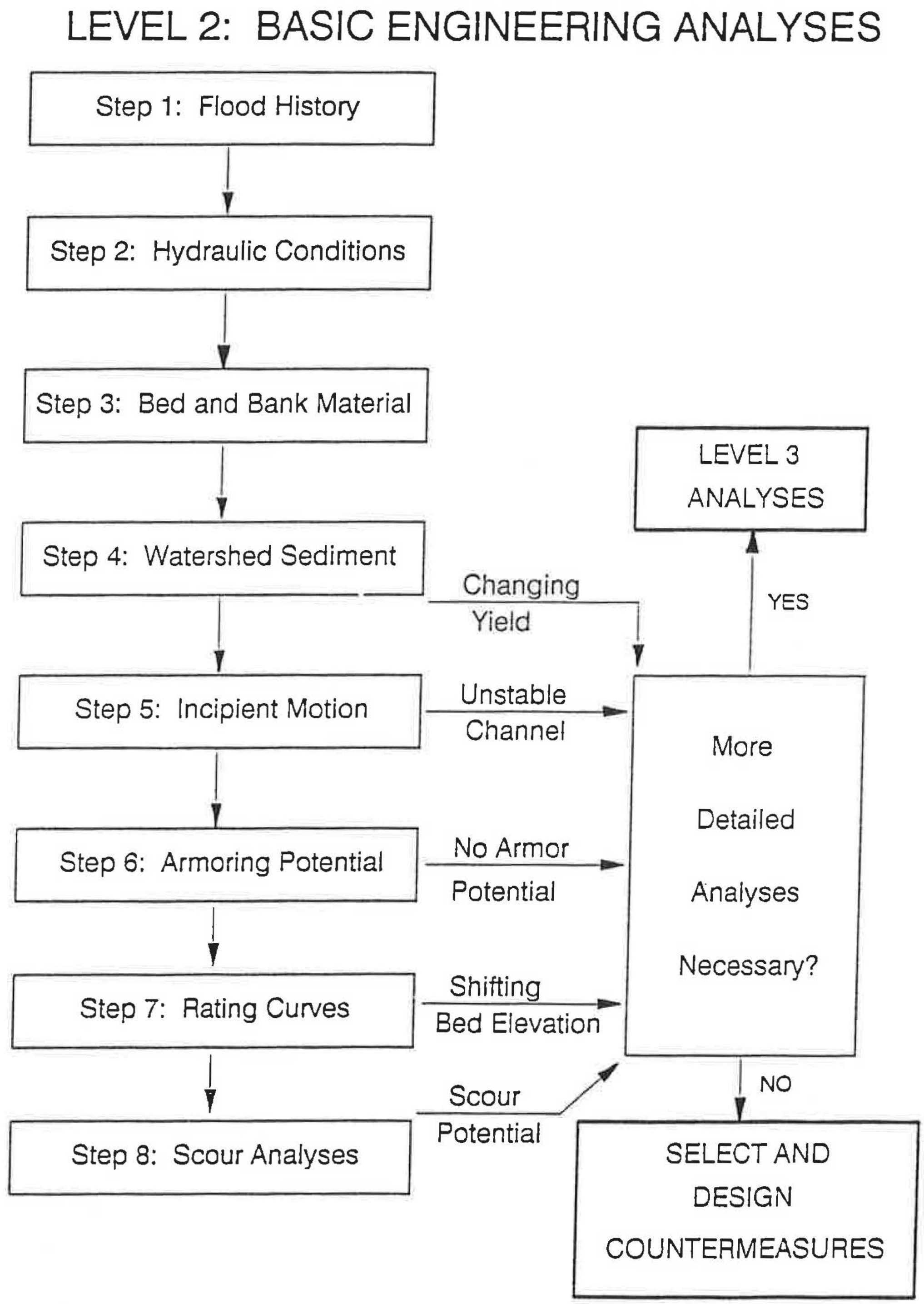

Figure 4. Flow chart for Level 2: Basic Engineering Analyses

the short term, may not necessarily cause change in a dryland system due to the infrequency of hydrologically significant events. Thus, the absence of significant morphological changes in a dryland stream or river, even over a period of years, should not necessarily be construed as being indicative of system stability.

Although the occurrence of single large storms can often be directly related to system change in any region of the country, this is not always the case. In particular, the succession of morphologic change may be linked to the concept of geomorphic thresholds as proposed by Schumm (8). Under this concept, although a single major storm may trigger an erosional event in a system, the occurrence of such an event may be the result of a cumulative process leading to an unstable geomorphic condition.

When flood records are available, the study of these records and corresponding system responses may help determine the relationship between morphological change and flood magnitude and frequency. Time-sequenced aerial photography or other physical information may provide such information. Evaluation of wet-dry cycles can also be beneficial to an understanding of historical system response. Observable historical change may be better correlated with the occurrence of events during a period of above-average rainfall and runoff than with a single large event. In addition, the study of historical wet-dry trends may explain certain aspects of system response. For example, a large storm preceded by a period of above-average precipitation may result in less erosion due to better vegetative cover than a comparable storm occurring under dry antecedent conditions; however, runoff volumes might be greater due to saturated soil conditions.

A good method for evaluating wet-dry cycles is the plotting of annual rainfall amounts, runoff volumes and maximum annual mean daily discharge for the period of record. A comparison of these graphs will provide insight into wet-dry cycles and flood occurrences. Additionally, a plot of the ratio of rainfall to runoff is a good indicator of watershed characteristics and historical changes in watershed condition.

Step 2. Evaluate Hydraulic Conditions

Knowledge of such basic hydraulic conditions as velocity, flow depth and top width for given flood events is essential for completion of a Level 2 stream stability analysis. Incipient motion analysis, scour analysis, and assessment of sediment transport capacity all require basic hydraulic information. Hydraulic information is sometimes required for both the main channel and overbank areas, as in the analysis of contraction scour.

For many river systems, particularly those near urban areas, hydraulic information may be readily available from previous studies such as flood insurance studies, channel improvement projects, and so on. In such cases, a complete re-analysis may not be necessary. In other areas however, hydraulic analysis based on appropriate analytical techniques will be required prior to completing other quantitative analyses in a Level 2 stream stability assessment. The most common computer models for analysis of water surface profiles and hydraulic conditions are the Corps of Engineers HEC-2 and the Federal Highway Administration WSPRO. For the analysis and design of bridge crossings, WSPRO is generally considered a better model. The computational procedure in WSPRO for evaluating bridge loss is superior to that utilized in other models, and the input structure of the model has been specifically developed to facilitate bridge design.

Step 3. Bed and Bank Material Analysis

Bed material is the sediment mixture of which the streambed is composed. Bed material ranges in size from huge boulders to fine clay particles. The erodibility or stability of a channel largely depends on the size of the particles in the bed. Additionally, knowledge of bed sediment is necessary for most sediment transport analyses, including evaluation of incipient motion, armoring potential, sediment transport capacity, and scour calculations. Many of these analyses require knowledge of particle size gradation, not just the median (D_{50}) sediment size.

Bank material usually consists of particles the same size as, or smaller

than, bed particles. Thus, banks are often more easily eroded than the bed, unless protected by vegetation, cohesion, or some type of man-made protection.

Of the various sediment properties, size is of the greatest significance to the hydraulic engineer, not only because size is the most readily measured property, but also because other properties, such as shape and fall velocity, tend to vary with particle size. A comprehensive discussion of sediment characteristics, including sediment size and its measurement, is provided in reference (9). The following information briefly discusses sediment sampling considerations.

Important factors to consider in determining where and how many bed and bank material samples to collect include:

1. Size and complexity of the study area
2. Number, lengths and drainage areas of tributaries
3. Evidence of or potential for armoring
4. Structural features that can impact or be significantly impacted by sediment transport
5. Bank failure areas
6. High bank areas
7. Areas exhibiting significant sediment movement or deposition (i.e., bars in channel)

Tributary sediment characteristics can be very important to channel stability, since a single major tributary or tributary source area could be the predominant supplier of sediment to a system.

The depth of bed material sampling depends on the homogeneity of surface and subsurface materials. Where possible, it is desirable to dig down some distance to establish bed-material characteristics. For example, in sand/gravel bed systems the potential existence of a thin surface layer of coarser sediments (armor layer) on top of relatively undisturbed subsurface material must be considered in any sediment sampling. Samples containing material from both layers would contain materials from two populations in unknown proportions, and thus it is typically more appropriate to sample each layer separately. If the purpose of the sampling is to evaluate hydraulic friction or initiation of bed movement, then the surface sample will be of most interest. Conversely, if bed-material transport during a large flood (i.e., large enough to disturb the surface layer) is important, then the underlying layer may be more significant. Methods of analysis are given in reference (9).

Step 4. Evaluate Watershed Sediment Yield

Evaluation of watershed sediment yield, particularly of the relative increase in yield as a result of some disturbance, can be an important factor in stream stability assessment. Sediment eroded from the land surface can cause silting problems in stream channels resulting in increased flood stage and damage. Conversely, a reduction in sediment supply can also cause adverse impacts to river systems by reducing the supply of incoming sediment, thus promoting channel degradation and headcutting. A radical change in sediment yield as a result of some disturbance, such as a recent fire or long term land use changes, would suggest that stream instability conditions either already exist or might readily develop.

Assessment of watershed sediment yield first requires understanding the sediment sources in the watershed and the types of erosion that are most prevalent. The physical processes causing erosion can be classified as sheet erosion, rilling, gullying and stream channel erosion. Other types of erosional processes are classified under the category of mass movement, e.g., soil creep, mudflows, landslides, etc. Data from publications and maps produced by the Soil Conservation Service and the Geological Survey can be used along with field observations to evaluate the area of interest.

Actual quantification of sediment yield is at best an imprecise science. The most useful information is typically obtained not from analysis of absolute magnitude of sediment yield, but rather from the relative changes in yield as a result of a given disturbance. One useful approach to evaluating sediment yield from a watershed was developed by the Pacific Southwest Interagency Committee (10). This method, which was designed only to aid in broad planning, consists of a numerical rating of nine factors affecting sediment production in a watershed. This

then defines ranges of annual sediment yield in acre-feet per square mile. The nine factors are surficial geology, soil climate, runoff, topography, ground cover, land use, upland erosion, channel erosion and channel transport.

Other approaches to quantifying sediment yield are based on regression equations, as typified by the Universal Soil Loss Equation (USLE). The USLE is an empirical formula for predicting annual soil loss due to sheet and rill erosion, and is perhaps the most widely recognized method for predicting soil erosion. Wischmeier and Smith (11) provide detailed descriptions of this equation and its terms.

Step 5. Incipient Motion Analysis

An evaluation of relative channel stability can be made by evaluating incipient motion parameters. The definition of incipient motion is based on critical or threshold conditions where hydrodynamic forces acting on one grain of sediment have reached a value that, if increased even slightly, will move the grain. Under critical conditions, or at the point of incipient motion, the hydrodynamic forces acting on the grain are balanced by the resisting forces of the particle.

The Shields diagram may be used to evaluate the particle size at incipient motion for a given discharge (9). For most river flow conditions, the following equation, derived from the Shields diagram, is appropriate for evaluation of incipient motion:

$$\boxed{D_c = \frac{\tau}{0.047(\gamma_s - \gamma)}}$$

where D_c is the diameter of the sediment particle at incipient motion conditions, τ is the boundary shear stress (see (9) for equations defining the boundary shear stress), γ_s and γ are the specific weights of sediment and water, respectively, and 0.047 is a dimensionless coefficient often referred to as the Shields parameter.

As originally proposed, the Shields parameter was 0.06 for flow conditions in the turbulent range. The value of 0.047 was suggested by Meyer-Peter and Muller (12) and was further supported by Gessler (13). Recent research has indicated that this coefficient is not constant (values range from 0.02 to 0.10), and equations have been derived as a function of surface and subsurface particle size. As a first estimate, however, the use of 0.047 should provide reasonable results in most situations.

Evaluation of the incipient motion size for various discharge conditions provides insight on channel stability and how a flood might disrupt channel stability. The results of such an analysis are generally more useful for analysis of gravel or cobble-bed systems. When applied to a sand bed channel, incipient motion results usually indicate that all particles in the bed material are capable of being moved for even very small discharges, a physically realistic result.

Step 6. Evaluate Armoring Potential

The armoring process begins as the non-moving coarser particles segregate from the finer material in transport. The coarser particles are gradually worked down into the bed, where they accumulate in a sublayer. Fine bed material is leached up through this coarse sublayer to augment the material in transport. As sediment movement continues and degradation progresses, an increasing number of non-moving particles accumulate in the sublayer. Eventually, enough coarse particles can accumulate to shield, or "armor" the entire bed surface.

An armor layer sufficient to protect the bed against moderate discharges can be disrupted during high flow, but may be restored as flows diminish. Therefore, as in any hydraulic design, the analysis must be based on a certain design event. If the armor layer is stable for that design event, it is reasonable to conclude that no degradation will occur under design conditions. However, flows exceeding the design event may disrupt the armor layer, resulting in degradation.

The potential for development of an armor layer can be assessed using incipient motion analysis and a representative bed-material composition. In this case the representative bed-material composition is that which is

typical of the depth of anticipated degradation. For given hydraulic conditions the incipient motion particle size can be computed as discussed above in Step 5. If no sediment of the computed size or larger is present in significant quantities in the bed, armoring will not occur.

The D_{90} or D_{95} size of the representative bed material is frequently found to be the size "paving the channel" when degradation is arrested. Within practical limits of planning and design, the D_{95} size is considered to be about the maximum size for pavement formation (12). Therefore, armoring is probable when the computed incipient motion size is equal to or smaller than the D_{95} size in the bed material.

By observing the percentage of the bed material equal to or larger than the armor particle size (D_a), the depth of scour necessary to establish an armor layer can be calculated (15):

$$\boxed{Y_s = y_a\left(\frac{1}{P_c} - 1\right)}$$

where y_a is the thickness of the armoring layer and P_c is the decimal fraction of material coarser than the armoring size. The thickness of the armoring layer (y_a) ranges from one to three times the armor particle size (D_a), depending on the value of D_a. Field observations suggest that a relatively stable armoring condition requires a minimum of two layers of armoring particles.

Step 7. Evaluation of Rating Curve Shifts

When stream gage data is available, such as that collected by the U.S. Geological Survey, an analysis of the stage-discharge rating curve over time can provide insight on stream stability. For example, a rating curve that was very stable for many years but shifted suddenly might indicate a change in watershed conditions causing increased channel erosion or sedimentation, or some other change related to channel stability. Similarly, a rating curve that shifts continually would be a good indicator that channel instability exists. However, it is important to note that not all rating curve shifts are the result of channel instability. Other factors promoting a shift in a rating curve include changes in resistance to flow due to bed forms (or lack thereof), channel vegetation, ice conditions, beaver activity, etc.

The most common cause of rating curve shifts in natural channel control sections is generally scour and fill.(16) A positive shift in the rating curve results from scour, and the depth (and hence the discharge) are increased for a given stage. Conversely, a negative shift results from fill, and the depth and discharge will be less for a give stage.

Shifts may also be the result of changes in channel width. Channel width may increase due to bank-cutting, or decrease due to undercutting of steep streambanks. In meandering streams, changes in channel width can occur as point bars are created or destroyed.

Analysis of rating curve shifts is typically available from the agency responsible for the stream gage. If such information is not available, field inspection combined with the methods described by reference (16) can be utilized to analyze observed rating curve shifts. If the shifts can be traced to scour, fill, or channel width changes, such information will be a reliable indicator of potential channel instability.

Gaging stations where continuous sediment data are collected may also provide clues to the existence of gradation problems. Any changes in the long-term sediment load may indicate lateral movement of the channel, gradation changes, or a change in sediment supply from the watershed.

Step 8. Evaluate Scour Conditions

HEC-20 (4) provides an overview of scour at bridge crossings; HEC-18 (17) provides detailed computational procedures. These references should be consulted for bridge scour analysis procedures.

CONCLUSIONS

A systematic, analytical procedure for the evaluation of stream stability has been outlined in this paper. The procedure incorporates three increasingly complex levels of analysis. It was developed for the FHWA Hydraulic Engineering Circular No. 20, "Stream Stability at Highway Structures."(4) The completion of each level of analysis results in an assessment of stream stability, and then, depending on the complexities of stream response or bridge design, the decision to proceed to the next level of analysis is made.

The specific steps typical of Levels 1 and 2 analysis have been outlined. Level 3 analysis, involving detailed evaluation and assessment of stream stability, is accomplished with mathematical and/or physical model studies. However, the need for the detailed information and accuracy available from either of these model studies must be balanced by the time and money available. As accounting for more factors makes the analysis more complicated, the level of effort necessary becomes proportionally larger.

The decision to proceed with a Level 3 analysis has historically been made only for high risk locations, extraordinarily complex problems, and for forensic analysis where losses and liability costs are high; however, the importance of stream stability to the safety and integrity of all bridges suggests that Level 3 analyses should be completed routinely. The widespread use of personal computers and the continuing development of more sophisticated software have greatly facilitated completion of Level 3 investigations and have reduced the level of effort and cost required.

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Countermeasures for Scour and Stream Instability at Bridges

J. R. RICHARDSON AND E. V. RICHARDSON

This paper presents short case histories where stream instability has led to the failure or partial failure of a bridge. In this context, stream instability can be characterized by either lateral instability or by vertical instability. In general lateral instability is caused by lateral migration of the stream or river, while vertical instability can be caused by a combination of contraction scour, long term aggradation or degradation, and local scour. In this paper, the design of three principal common countermeasures will be discussed. These include spurs, check dams and guide banks.

On April 5, 1987, span 3 and 4 of the New York State Throughway bridge over Schoharie Creek collapsed causing loss of life, destruction of property and severing of an important transportation artery. Subsequent investigations [1] of this failure indicated that pier scour during a near record flood was the cause of the failure of this bridge. The pier scour which caused this failure is one of several types of vertical instabilities in rivers and streams which endanger bridge structures.

Lateral instability can also endanger highway crossings over rivers and streams. For example, consider the failure of U.S. 51 highway bridge over the Hatchie River, in Tennessee. On April 1, 1989 portions of this bridge collapsed, also causing loss of life. Lateral shifting of the channel which undermined a bent, was identified as the cause for this disastrous failure.

These failures of bridges due stream instabilities are not isolated cases. There are many documented cases where either lateral and vertical stream instability have caused damage and/or failure of bridges throughout the United States. For example Highways in the River Environment [2] (HIRE) documents cases in Oklahoma, where the bridge crossing of State Highway 51 over the Cimarron river, and the crossing of U.S. Highway 62 over the Arkansas river, have sustained damaged by lateral shifting. HIRE [2] and also FHWA reports [3] and [4] also document other cases, such as SR-16 crossing over Lawerence Creek, Louisiana and on the Russian River in California, where vertical and/or lateral instabilities have endangered bridge crossings.

Recently, FHWA has produced two reports concerning the impact of stream stability on bridges over streams and rivers. The first, HEC-18 [5], discusses methods to evaluate scour at bridges. The second HEC-20 [6] focuses on the evaluation of stream instability as well as the selection and design of countermeasures to control lateral and vertical instabilities.

Countermeasures for the control of scour and stream instabilities at bridges and abutments are revetments, spurs, guide banks and check dams. Additionally, riprap can be considered a countermeasure for local scour at piers and abutments. Monitoring and inspection is considered to be a non-structural countermeasure for scour and instability problems at highway crossings.

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Revetments are rigid or flexible armor placed on or parallel to a bank or embankment as a protection against scour and lateral erosion. Revetments can be constructed from a wide variety of construction materials, including rock (riprap), soil cement, sheet piles, rock-filled wire baskets (gabions), crib dikes, concrete mattresses, jacks, tetrahedrons, and used tires.

Spurs are either pervious or impervious structures which extent from the banks of a stream at an angle to the flow. The slowing of the flow caused by the spur protects bank lines and encourages deposition of transported material. Spurs countermeasures can be constructed of embankments with riprap, sheet piles, rock, rock-filled wire baskets (gabions), crib dikes, concrete mattresses, jacks, tetrahedrons, or fences made of treated wood or steel.

Guide banks are structures constructed perpendicular to the approach of a highway crossing at the bride opening to guide flood flows through the bridge opening. These structures are usually constructed of embankment material and protected with riprap to impede erosion of the structure.

Check dams are constructed in the channel to provide protection from a difference in the bed elevation of the stream. These structures can be constructed of rock, concrete, sheet piles, gabions, or treated timber piles.

Although riprap can be used as a construction material for the countermeasures discussed above, it also can be considered as a countermeasure in and of itself for the control of local scour at piers and abutments.

Selection of countermeasures for these stream instabilities are presented in HEC-11 [7], 18 [5] and 20 [6] as well as in HIRE [2]. This paper will focus on three countermeasures for stream instability. These are; 1) spurs, for the control of lateral instabilities; 2) check dams for the control of vertical instabilities; and 3) guide banks for the control of abutment scour. Other countermeasures will be briefly discussed.

COUNTERMEASURES FOR STREAM INSTABILITY

General

Streams and rivers continuously adjust their slope, alignment, and channel shape based upon the volume of water and sediment they must transport [2, 3, 4, 5, and 6]. The extent and rate of these adjustments depends on a complex interaction of the rivers flow, and the physical constraints such as bed, banks and structures in the rivers channel or on the floodplain. These movements of river channels either laterally or vertically constitutes instability from the highway departments standpoint.

The selection of countermeasures for stream instability depends on the type of instability (lateral or vertical), the characteristics of the stream, construction and maintenance requirements, and costs. Characteristics of the stream include the channel width, bank heights, bank vegetation, sediment transport, velocities and the extent of ice and in debris the river. The reader is referred to HEC-20 [6] for a detailed discussion of the selection of countermeasures for stream instabilities.

Three primary countermeasures recommended in HEC-20 [6] are spurs, guide banks, and check dams. Spurs are used primarily for controlling lateral instabilities, while check dams are used to control vertical instabilities (specifically channel degradation). Guide banks are recommended to control abutment scour and to provide a more uniform flow through the bridge opening. A fourth countermeasure for scour and stream instability is monitoring and inspection of bridges. These countermeasures are discussed in detail in subsequent sections of this text. Other countermeasures will be discussed in more general terms.

Monitoring and Inspection

One of the most important countermeasure for scour and stream instability at bridges is monitoring and inspection. Monitoring and inspection should evaluate the long term aggradation, degradation, contraction scour, and local scour. Additionally, monitoring and inspection should assess the degree, location and rate of lateral shifting or widening of the river channel both upstream and downstream of the river crossing.

The frequency of monitoring and inspection depends on the vulnerability of the bridge to scour and stream instability. This frequency depends on whether or not the bridge is considered to be *scour critical* or not. A scour critical bridge is defined as a bridge whose foundation material and/or depth is unknown or insufficient to protect the bridge from failure caused by scour or stream instability.

At a minimum, for a non scour critical bridge, inspection frequency should be FHWA's mandatory two year cycle with a five year underwater inspection cycle for bridges with foundations which cannot be inspected from the surface. For scour critical bridges, as an interim measure, monitoring during floods and inspection shortly thereafter can be used as a countermeasure. If monitoring a scour critical bridge during a flood is selected as an interim measure instead of closing the bridge, then piers and abutments must have interim protection such as riprap, or a method to determine that scour is not undermining the foundation must be provided. At this time the sizing of riprap to resist scour is not *fail-safe*. Therefore, even if riprap is placed around piers or abutments, the bridge must be monitored during floods and inspected afterwards.

Spurs

A spur is a previous or impervious structure projecting from the stream bank into the channel. Spurs are used to deflect flowing water away from, or to reduce flow velocities in critical zones near the stream bank, to prevent erosion of the bank, and to establish a more desirable channel alignment or width. The main function of spurs is to reduce flow velocities near the bank, and encourage sediment deposition. As sediment is deposited behind the spurs the stability of the banks is enhanced. Since the presence of spurs moves the location of scour away from the bank, partial failure of the spur can often be repaired before bridges and structures are damaged.

Spurs can be used to arrest meander migration and to channelize wide, poorly defined streams into better defined channels. The use of spurs to establish and maintain a well-defined channel location, cross-section, and alignment in braided streams can decrease required bridge lengths, decreasing the cost of bridge construction and maintenance.

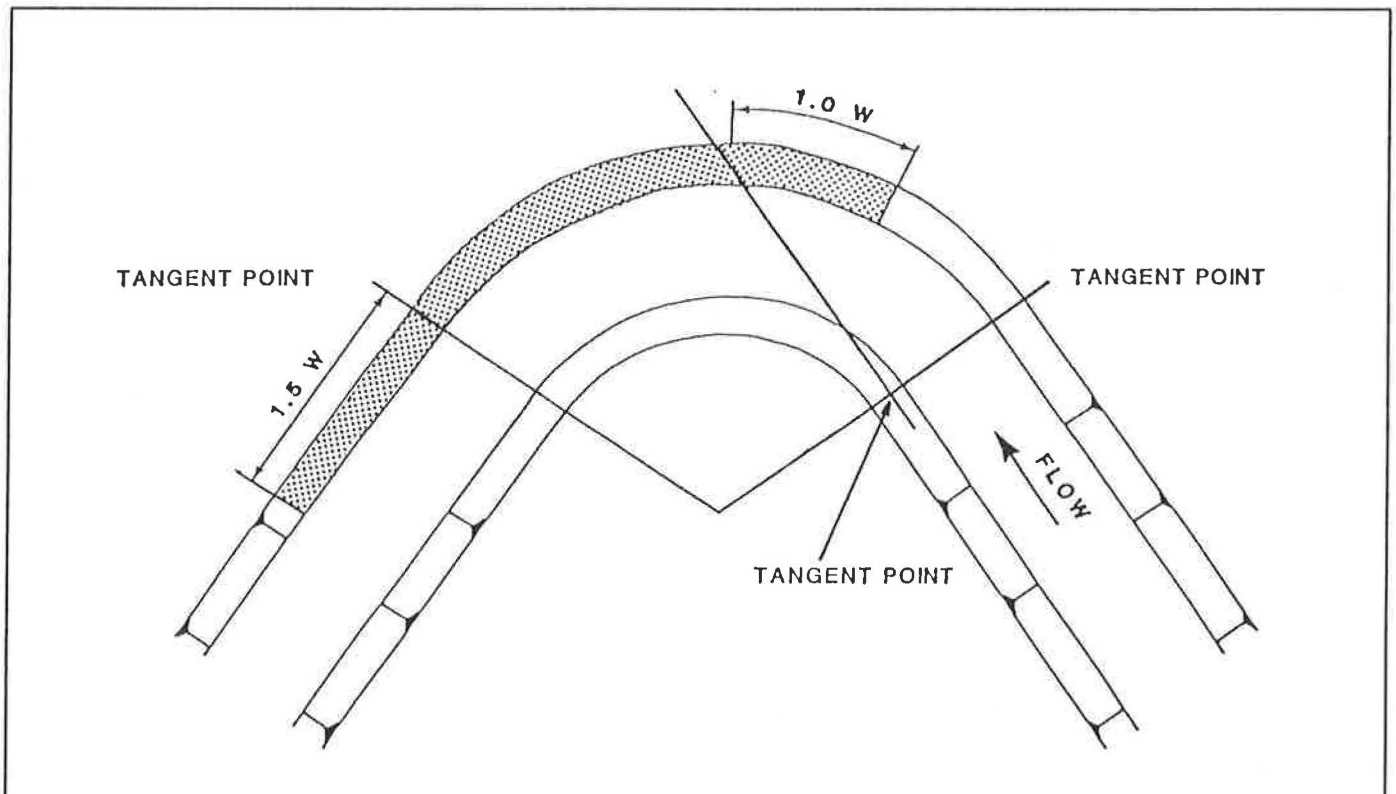

Figure 1 Extent of Protection Required at a Channel Bend (After [6]).

Spur design includes setting the limits of bank protection required; selection of the spur type to be used; and design of the spur installation including spur orientation, spacing, length, and height. Care must be exercised to carefully evaluate local scour at spurs and to protect them with riprap as necessary.

Limits of Spur Field

Spurs are constructed to protect a length of channel bank. As such a length of bank will require several spurs. The longitudinal extent of channel bank requiring protection is discussed in HEC-20 [6]. The minimum extent of bank protection determined from Figure 1 should be adjusted according to field inspections to determine the limits of active scour. In general, surveys of field installations of bank protection have found that protection commonly extends farther upstream than necessary and not far enough downstream.

Spur Type

Spur types are classified based upon their permeability as deflector spurs, retarder/deflector spurs, and retarder spurs. The permeability of spurs is defined simply as the percentage of the spur surface area facing the stream flow that is open. Deflector spurs are impermeable spurs which function by diverting the primary flow currents away from the bank. Retarder/deflector spurs are semi-permeable and function by retarding flow velocities at the bank and diverting flow away from the bank. Retarder spurs are more permeable and function by retarding flow velocities near the bank.

The primary factors influencing the selection of a specific spur type are the function, erosion mechanism of the stream, the sediment transport of the stream, flow velocities and depths, and whether ice or debris is present in the river. The selection of spur type for design purposes is discussed in detail in HEC-20 [6] and HIRE [2].

Spur Orientation

It is recommended that the spurs be oriented at an angle of approximately 90 degrees to the desired bank line. The only exception is for the spur furthest upstream which should be angled downstream to provide a smoother transition of the flow lines near the bank and to minimize scour at the nose of the leading pier. Subsequent spurs downstream should all be set normal to the desired bank line to minimize construction costs.

Spur Spacing

Spur spacing is a function of spur length, spur angle, permeability, and the degree of curvature of the bend. The flow expansion angle, the angle at which flow expands toward the bank downstream of a spur, is a function of spur permeability and the ratio of spur length to channel width. Figure 2 indicates that the expansion angle for impermeable spurs is almost constant at 17 degrees. Spurs with 35 percent permeability have almost the same expansion angle except where the spur length is greater than about 18 percent of the channel width.

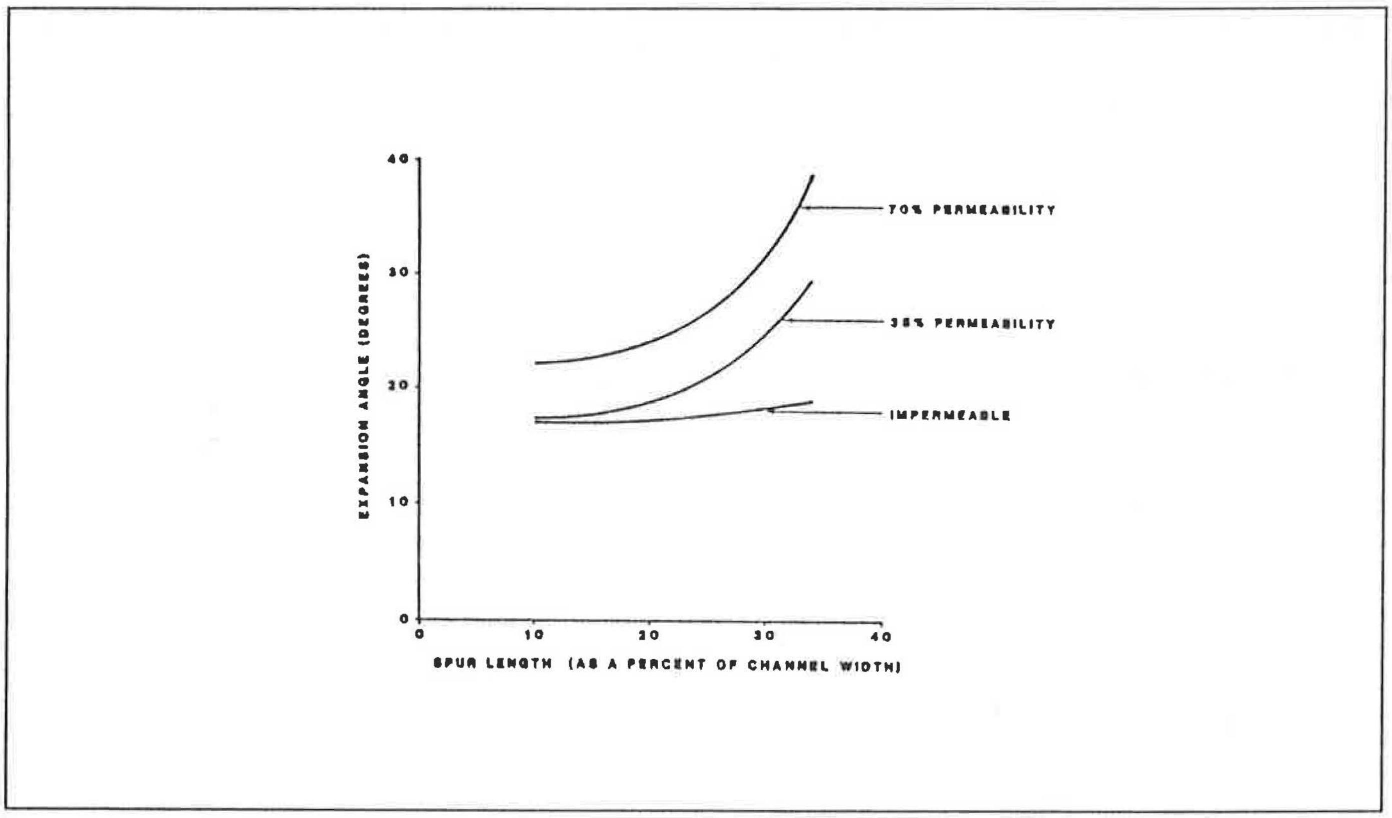

Figure 2 Relationship between spur length and expansion angle for several spur permeabilities. (From [6]).

The expansion angle for spurs increases as the permeability of the spur and as the ratio of spur length to channel width increases. The angle the spur makes with the flow (spur orientation) also influences the expansion angle but to a lesser degree.

Spurs should be spaced using Equation 1. This insures that the flow as it expands from the nose of the upstream spur will not intersect the bank to be protected before intersecting the next spur downstream.

$$S = L \cot\theta \tag{1}$$

where

S = is the spacing between spur toes. In the case of the furthest downstream spur this is the distance between the spur and the bank downstream. (ft),

L = is the effective length of spur, or the distance between the toe of spurs and the desired bank line, (ft) and

θ = is the expansion angle downstream of spur tips (degrees)

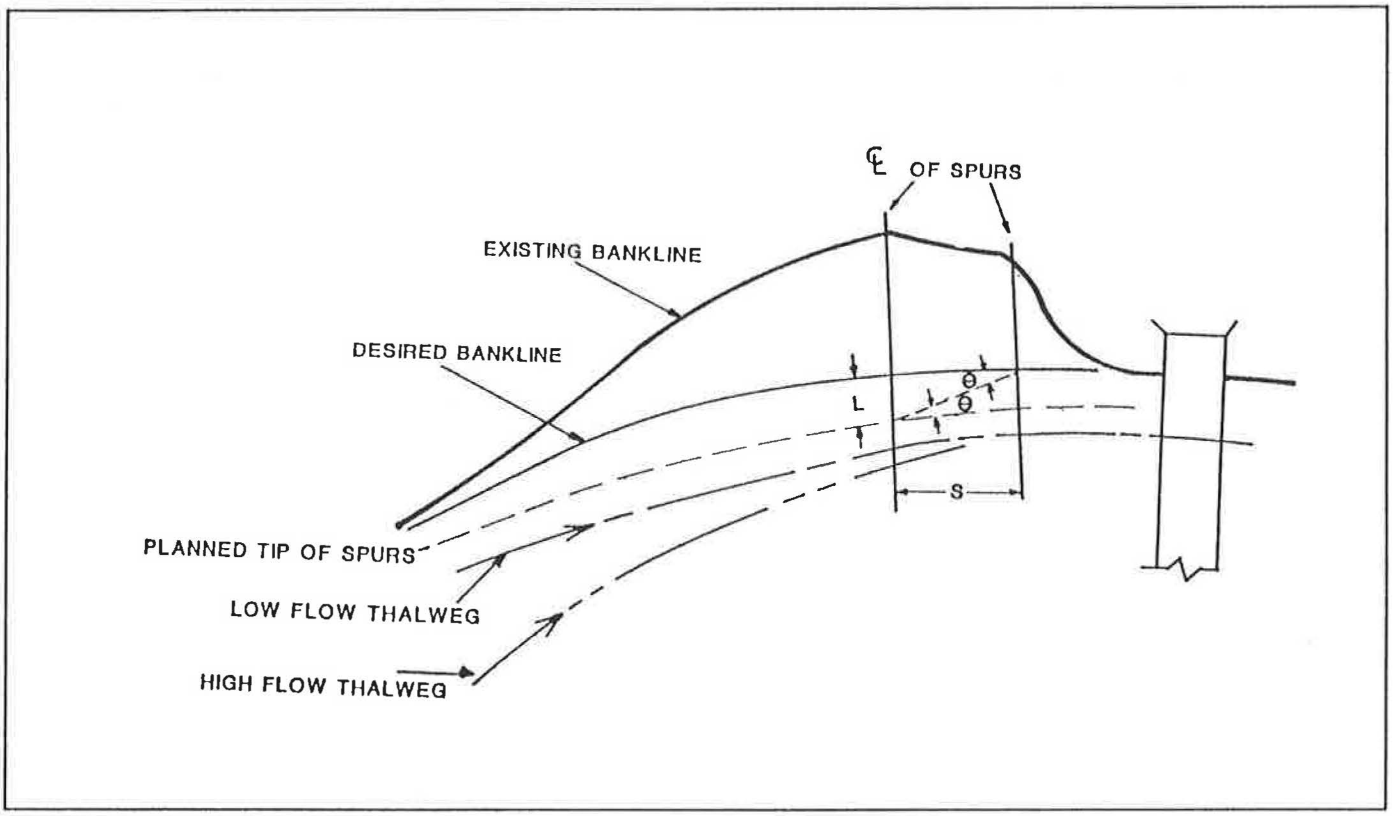

Figure 3 Illustration of spur spacing in a meander bend.

Spur spacing in a bend can be established on a scale plan view of the stream by drawing a line or arc describing the desired flow alignment as shown in Figure 3. This location should represent the desired location of the thalweg along the outside bank. The desired flow alignment may differ from existing conditions or represent no change in conditions, depending on whether there is a need to arrest erosion or reverse erosion that has already occurred. Base upon the desired flow alignment, a second line, describing the desired bank location can be established.

As a third step, a line which will connect the toe of the spurs in the installation. The distance from this line to the line describing the desired bank line, along with the expansion angle, will set the spacing between spurs. The line or arc describing the ends of spurs projecting into the channel will be essentially parallel or concentric with the arc describing the desired flow alignment.

Length, Height

Impermeable spurs are generally designed not to exceed the bank height because erosion at the end of the spur in the over bank area could increase the probability of outflanking at high stream stages. Where stream stages are greater than or equal to the bank height, impermeable spurs should be equal to the bank height. If flood stages are lower than the bank height, impermeable spurs should be designed so that overtopping will not occur at the bank.

The crest of impermeable spurs should slope downward away from the bank line. Use of a sloping crest will avoid the possibility of overtopping at the bank, which could cause damage to the spur or to the stream bank.

Permeable spurs, and in particular those constructed of light wire fence, should be designed to a height that will allow heavy debris to pass over the top. However, highly permeable spurs consisting of jacks or tetrahedrons are dependent on light debris collecting on the spur to make them less permeable. The crest profile of permeable spurs is generally level except where bank height requires the use of a sloping profile.

In general the length of the spur is dictated by the location and spacing discussed earlier. The length must be sufficient to extent from the nose of the spurs to, or beyond the bank line. The bank side of the spurs should be anchored and tied into the existing bank to prevent outflanking by the stream.

Scour and Protection

Scour around spurs must be checked. Scour will be less for permeable spurs than for impermeable spurs. The determination of the extent of local scour at spurs is discussed

in HEC-20 [6] and also in HIRE [2]. For impermeable spurs, rock riprap should be placed on the upstream and downstream faces as well as on the nose of the spur to inhibit erosion of the spur. Depending on the embankment material being used, a gravel sand or fabric filter may be required.

It is recommended that riprap be extended below the bed elevation to a depth of five feet. Riprap should also extend to the crest of the spur, in cases where the spur would be submerged at design flow, or to two feet above the design flow, if the spur crest is higher than the design flow depth. Riprap should be placed on the upstream side of the spur and along the downstream side if the spur will be overtopped.

Guide Banks

When embankments encroach on wide flood plains, the flows from these areas must flow parallel to the approach embankment, to the bridge opening. These flows, can erode the approach embankment. At the abutment the severity of the contraction is increased by the returning flow, and can reduce the effective bridge opening, thus increasing the severity of scour at the abutments of the bridge. Guide banks can be used effectively in these cases to prevent erosion of the approach embankments by cutting off the flow adjacent to the embankment, guide stream flow through a bridge opening, and to transfer scour away from abutments to prevent damage caused by abutment scour. Guide banks are often in other literature referred to as *Spurs* or *Spur Dikes*. However, this terminology is confused. Therefore the term *Guide Banks* is used, because it is more descriptive and will not be confused with Spurs.

Figure 4 presents a typical guide bank plan view. Without this guide bank, over bank flows would return to the channel at the bridge opening, which can increase the severity of contraction at the abutment. Note, that the scour hole which normally would occur at the abutment of the bridge is moved upstream away from the abutment.

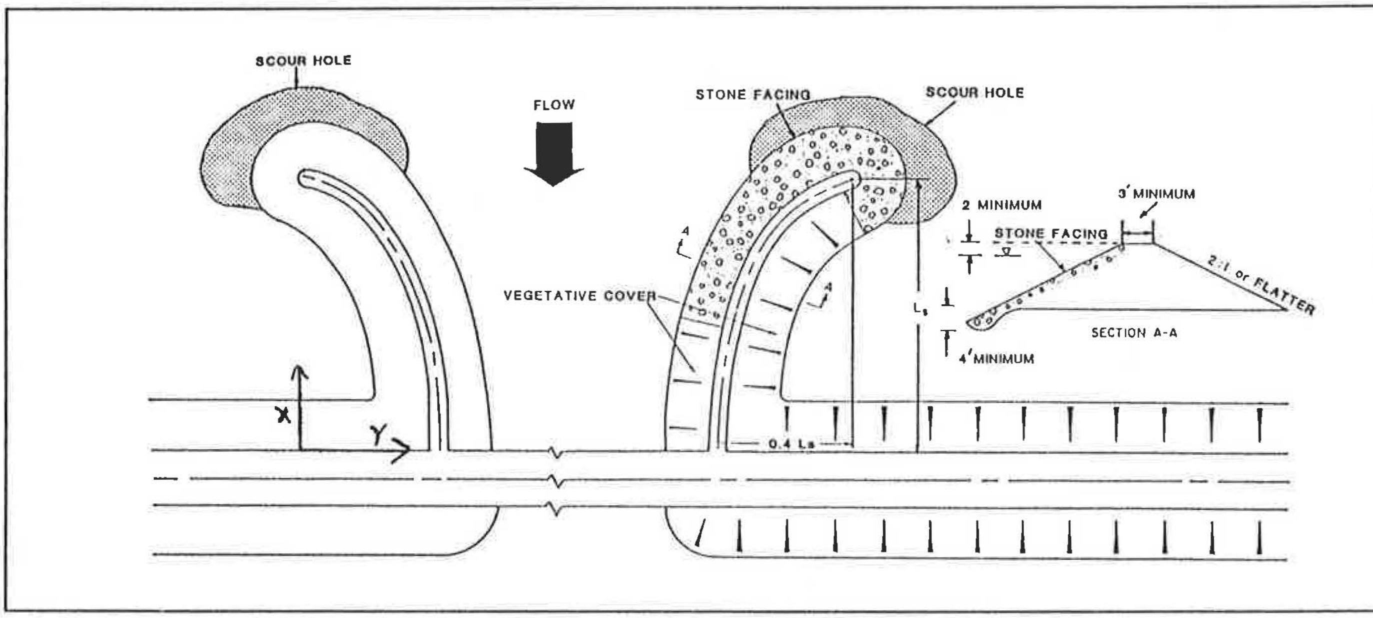

Figure 4 Typical Guide Bank

Guide banks can be used on both sand and gravel bed streams. Principal factors to be considered when designing guide banks, are their orientation to the bridge opening, upstream and downstream length, plan shape, cross-sectional shape, and crest elevation.

Orientation and Plan Shape

Guide banks begin and extend upstream from the abutments of the bridge opening. If guide banks are use on both sides of the stream, the distance between the guide banks at the bridge opening should be set equal to the distance between bridge abutments. Best results are obtained by using guide banks with a plan form shape in the form of a quarter of an ellipse, with the ratio of the major axis (length L_s) to the minor axis (offset) equal to 2.5 : 1. This allows for a gradual constriction of the flow. For design purposes, the length of the guide bank, measured perpendicularly from the approach embankment to the upstream nose of the guide bank, is denoted as L_S. The amount of expansion of each guide bank (offset), measured from the abutment, parallel to the approach, should be 0.4 L_S.

The plan view orientation can be determined using Equation 2 which is the equation of an ellipse with origin at the nose of the guide bank. For this equation, the orientation of the orthogonal axies for X and Y are shown on Figure 4.

$$\frac{X^2}{L_s^2} + \frac{Y^2}{(0.4L_s)^2} = 1 \qquad (2)$$

Length

For design of guide banks, the length of the guide bank, L_s can be determined using a nomograph which was developed from laboratory tests performed at Colorado State University and from field data compiled by the U.S. Geological Survey [6]. For design purposes the utilization of the nomograph which is presented in Figure 5 involves the following parameters:

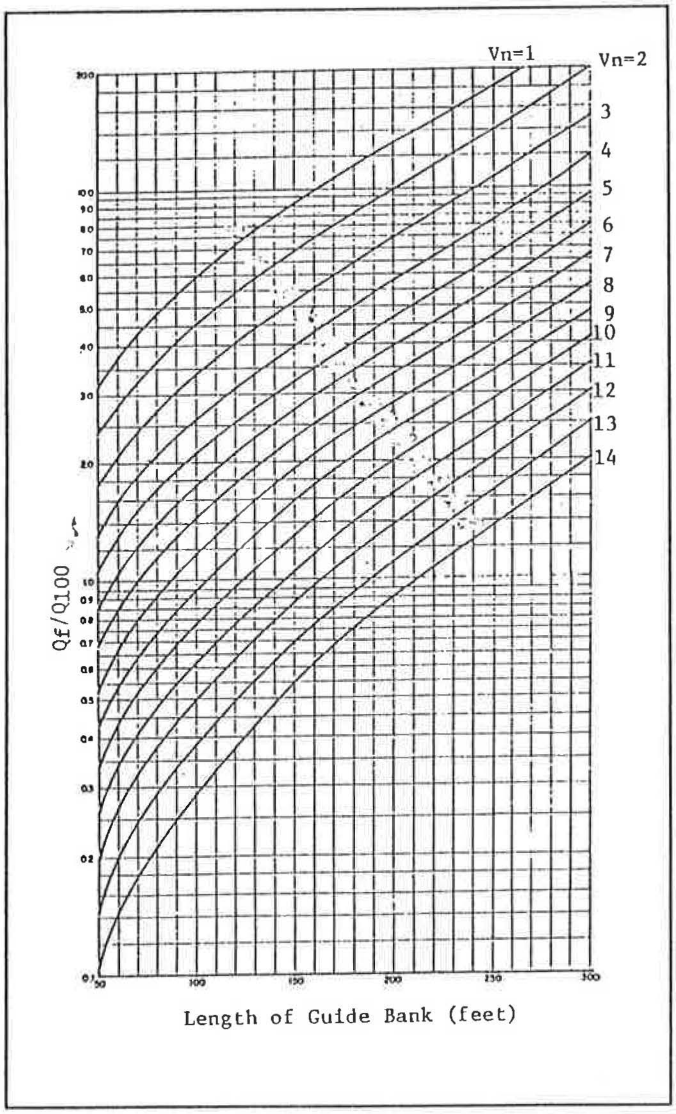

Figure 5 Nomograph to Determine Guide Bank Length (After [6])

Q = Total discharge of the stream (cfs),

Q_f = Lateral or flood plain discharge of either flood plain (cfs),

Q_{100} = Discharge in 100 feet of flood plain adjacent to the main channel which is cut off by the approach embankment (cfs),

b = Length of the bridge opening (ft),

A_{n2} = Cross-sectional flow area at the bridge opening at normal stage (ft^2)

$V_{n2} = \frac{Q}{A_{n2}}$ = Average velocity through the bridge opening (cfs),

$\frac{Q_f}{Q_{100}}$ = Guide bank discharge ratio

L_s = Projected length of guide bank.

The nomograph should be used to determine the guide bank length for designs greater than 50 feet and less than 250 feet. If the nomograph indicates the length required to be greater than 250 feet the design should be set at 250 feet. For guide banks less than 50 feet, no guide bank is required.

Cross-Section And Crest Elevation

In contrast to spurs, guide banks should be designed so that they will not be overtopped at the design discharge. If this were allowed to occur, unpredictable cross flows and eddies could scour and undermine abutments. In general a minimum of 2 feet of freeboard, above the design water surface elevation should be maintained.

The cross-sectional shape and size of guide banks should be similar to spurs discussed previously. That is to say that the minimum crest width should be 3 feet, however widths of 10 to 12 feet are more common due to construction methods used. The upstream end of the guide bank should be round nosed. Side slopes should be 2:1 or flatter.

Some states construct short guide banks downstream of the abutments to minimize scour due to rapid expansion of the flow at the downstream end of the abutments. These shorter guide banks are usually less than 50 feet long and are sometimes referred to as *heels*.

Check Dams

Check dams or channel drops are used downstream of highway crossings to control vertical instabilities by arresting head cutting, general channel degradation or the progression of nickpoints upstream. Typically check dams are usually built of rock riprap, concrete, sheet piles, gabions, or treated timber piles.

Check dams can initiate erosion of banks and the channel bed downstream of the structure as a result of energy dissipation and turbulence at the drop. This local scour can undermine the check dam and cause failure. The use of energy dissipaters downstream of check dams can reduce the energy available to erode the channel bed and banks. In some cases it may be less costly to construct several consecutive drops of shorter height to minimize extensive erosion control.

Lateral erosion of channel banks just downstream of drop structures is another adverse result of check dams and is caused by turbulence produced by energy dissipation at the drop, bank slumping from local channel bed erosion, and eddy action at the banks. The solution to this problem is to place revetment or riprap on the stream bank.

Design Considerations

A typical vertical drop is diagramed in Figure 6, For lack of any better equation, the Veronese equation (Equation 3) is recommended for all conditions to estimate the depth of scour downstream of a vertical drop for submerged and unsubmerged conditions [6].

$$d_s = K\, H_t^{0.225} q^{0.54} - d_m \qquad (3)$$

where

d_s = Local scour depth for a free overfall, measured from the stream bed downstream of the drop, (ft),

q = Discharge per unit width, (cfs per foot of width),

H_t = Total drop in head, measured from the upstream to the downstream energy grade line, (ft).,

d_m = Tail water depth, (ft).

K = 1.32

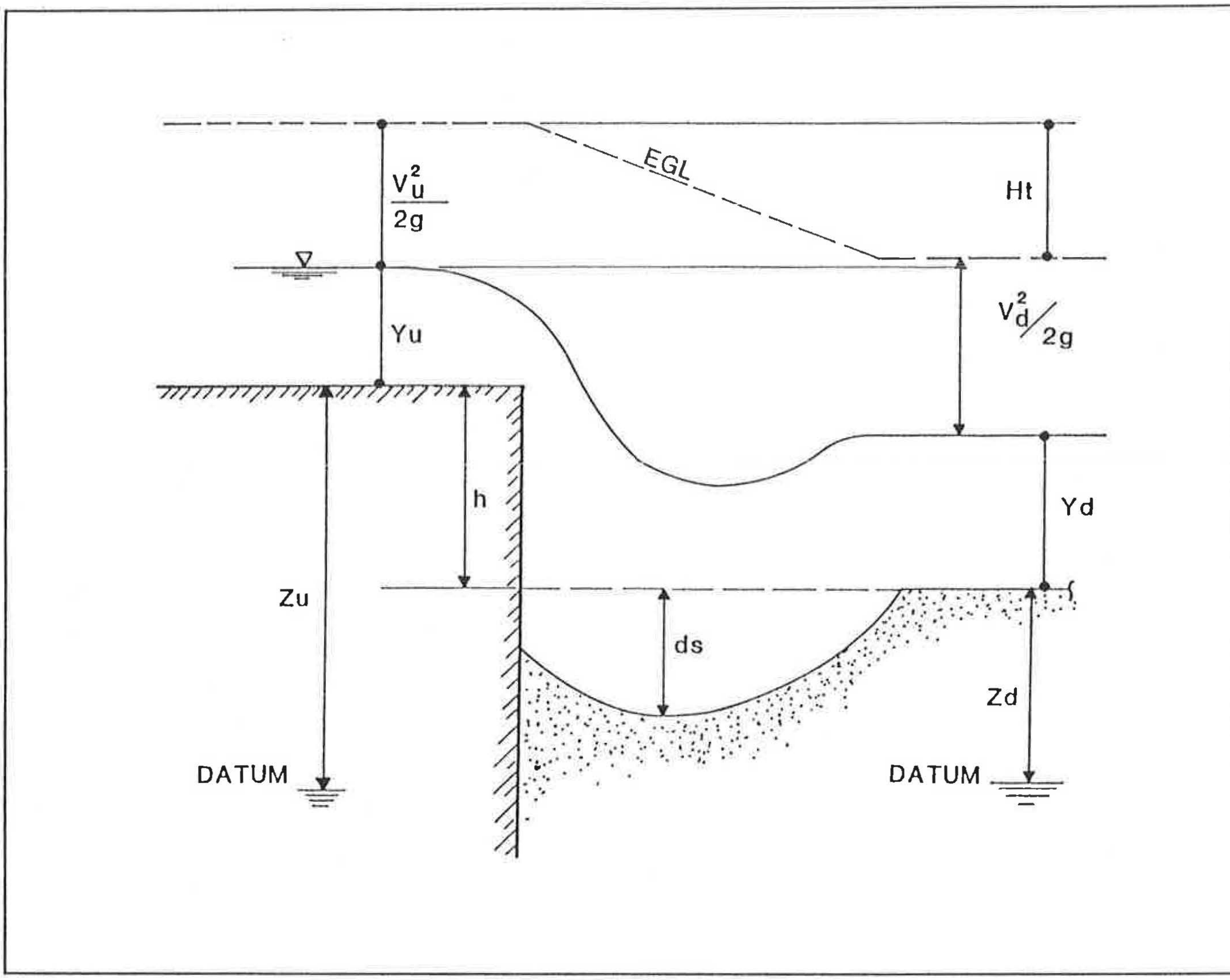

Figure 6 Schematic of a Vertical Drop Caused by a Check Dam (After [6])

It should be noted that H_t is the difference in the total head from upstream to downstream. This can be computed using Bernouli's equation for steady uniform flow Equation 4.

$$H_t = \left\{ Y_u + \frac{V_u^2}{2g} + Z_u \right\} - \left\{ Y_d + \frac{V_d^2}{2g} + Z_d \right\} \tag{4}$$

where

Y = Depth (ft)

V = Velocity (ft/s)

Z = Bed elevation referenced to a common datum (ft)

g = Acceleration due to gravity 32.2 f/s^2

The subscripts u and d refer to upstream and downstream of the channel drop respectively.

It should be noted that the depth of scour as estimated by the above equation is independent of the grain size of the bed material. This concept acknowledges that the bed will scour regardless of the type of material composing the bed but the rate of scour depends on the composition of the bed. In some cases, with large or resistive material, it may take years or decades to develop the maximum scour hole. In these cases the design life of the bridge may need to be considered when designing of the check dam.

The check dam must be designed structurally to withstand the forces of water and soil assuming that the scour hole is as deep as estimated using the equation above. Therefore, the designer should consult soils and structural engineers so that the check dam will be stable under these scoured conditions. In some case a series of check dams may be employed to minimize construction costs of foundations.

Other Countermeasures

Other countermeasures for the control of lateral and vertical instabilities include revetments, riprap, hardpoints, jacks, tetrahedrons, longitudinal, rock dikes, vane dikes, crib dikes, bulkheads, and soil cement. The selection and design of these countermeasures are discussed in FHWA publications HEC-11[7], HEC-18 [5], HEC-20 [6] and HIRE [2].

Many of these countermeasures can be used to enhance the function of one of the countermeasures discussed in this paper. In some cases, the use of these other countermeasures can be used as temporary methods for immediate protection of a bridge or highway encroachment.

One of the most promising methods of stabilizing banks when large rock is not available is the use of soil cement. This method mixes cement with native soils which is placed along

river banks. This strengthens the banks and helps to resist the erosive action of the flow. The use of soil cement has been shown to be an effective means to protect banks.

SUMMARY AND DISCUSSION

Recognizing that the failure or damage to bridges and highway crossings due to lateral and vertical instabilities of the stream and rivers, the Federal Highways Administration has produce a series of Hydraulic Engineering Circulars (HEC) to aid highway engineers. These publications provide for the selection, evaluation and design of countermeasures to protect highway bridges and highway crossings from scour and instabilities of the river.

Three principal countermeasures for the protections of bridges are spurs, for lateral instabilities, check dams, for vertical instabilities, and guide banks for the improvement of the flow hydraulics through bridge openings and as a countermeasure to control abutment scour. These design of these three countermeasures are described in this paper.

An important distinction in terminology between guide banks and spur dikes was made in this paper. That is, that the term guide bank is a more descriptive term for structures which align and guide the flow through the bridge opening. The use of the term *spur dike* which guide banks are often referred to as, is too often confused with *spurs*. The function of guide banks are to align the flow while the function of spurs are to divert or impede the flow.

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Sizing Riprap to Protect Bridge Piers from Scour

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ABSTRACT

Equations recommended for determining riprap size to protect bridge piers are compared to experimental results from small-scale model studies conducted in laboratory flumes. Adjustments to the equations are recommended from the analysis of laboratory data, as are uses of the equations to protect circular and rectangular bridge piers from scour.

INTRODUCTION

The leading cause of bridge failure over waterways has been the scouring of foundation material by floodwaters (Makowski, Thompson, and Yew 1989). Bridge piers obstruct flow and induce local secondary currents that take the form of strong eddy systems which have a much higher capacity for eroding bed material than unobstructed flow. As a result, local scour holes tend to form in unprotected alluvial streambeds surrounding bridge piers. Often local scour holes develop to the extent that the pier foundation is undermined causing settlement and, in some cases, the collapse of supported bridge spans.

One of the most common methods for protecting piers is the placement of a riprap apron to armour the streambed influenced by the secondary currents. The rock sizes required to protect the streambed tend to be much larger than normally required for unobstructed flow conditions. Small-scale laboratory experiments have been conducted to relate the rock sizes required to protect bridge piers to local flow conditions (Quazi and Peterson 1973, Parola 1990). This paper compares the data of those small-scale experiments to the current equations for sizing riprap to protect bridge piers suggested by the Federal Highway Administration (FHWA), and proposes equations based upon experimental data.

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CURRENT FHWA METHOD

FHWA published a Technical Advisory to provide guidelines for evaluating and designing bridge foundations in waterways (FHWA 1989). The advisory includes a method for sizing rock to protect the streambed surrounding bridge piers. The method involves determining the local average velocity just upstream of the pier such that the pier does not directly influence the flow. This velocity, V, is multiplied by a factor, f, which can range from 1.5 to 2.0. The increased velocity is then used in the equation proposed by Isbash (1935) for determining the stability of rocks dumped into flowing water. The equation recommended for sizing rock protection at piers can be written as

$$D_{50} = 0.347 \frac{(f\ V)^2}{g\ (S_g - 1)} \tag{1}$$

where

D_{50} = the nominal sieve size for which 50 percent of the rock is finer by weight (ft)

S_g = specific gravity of rock material

V = local average velocity (ft/s)

g = gravitational acceleration (ft^2/s)

f = 1.5 to 2.0, factor to account for pier turbulence

The value of f used in the equation is left to the designer's discretion.

Equation 1 was recommended based on the suggestions of Breusers, Nicollet, and Shen (1977), Neill (1973), and unpublished velocity measurements collected in the vicinity of model bridge piers at the Turner-Fairbank Highway Research Center in McLean, Virginia.

Through comparison of the rock sizes recommended to protect streambeds conveying unobstructed uniform flow to the rock sizes recommended to protect piers in similar flow conditions, the dramatic increase in rock size required to protect bridge piers is apparent. The equation (US Department of Transportation 1989) recommended for determining rock sizes to protect a streambed conveying uniform flow is

$$D_{50} = 0.387 \frac{V^3}{Y^{1/2}\ g^{3/2}\ (Sg-1)^{3/2}} \tag{2}$$

where

V = average velocity (ft/s)

Y = flow depth (ft)

An important difference between Equation 1 and Equation 2 is that the rock sizes in Equation 1 are much larger than those predicted by Equation 2 for the same flow conditions. In addition, the rocks sizes predicted by Equation 2 are dependent on flow depth, Y, unlike the rock sizes predicted in Equation 1.

Both equations can be written in non-dimensional form by dividing each side of each equation by the flow depth, Y, and by the substitution of Froude number, $F = V/(gY)^{1/2}$. Equation 1 can be rewritten as

$$\frac{D_{50}}{Y} = 0.347 \frac{(f\ F)^2}{(S_g-1)} \tag{3}$$

and Equation 2 can be rewritten as

$$\frac{D_{50}}{Y} = 0.387 \frac{F^3}{(S_g-1)^{3/2}} \tag{4}$$

Equation 3 with f = 1.5 and f = 2.0 and Equation 4 are compared graphically in Figure 1 with the assumption that the specific gravity of the rock material is 2.65. The difference in the slope of each line is a result of the dependence of D_{50} on flow depth in Equation 4 and the non-influence of flow depth in Equation 3. The relative rock sizes, D_{50}/Y, predicted by each equation can be compared for a given flow condition (Froude number). Note also that the size differences between the uniform flow equation and the pier equations decrease with increases in Froude number.

MODEL STUDY RESULTS AND COMPARISONS

Small-scale model studies were conducted in laboratory flumes to relate the rock sizes required to protect pier foundations to local flow conditions (Quazi and Peterson 1973, Parola 1990). Quazi and Peterson (1973) conducted experiments in a 4.0 ft wide flume using a 0.13 ft round-nosed pier model with rock placed flush with the surrounding streambed elevation. Parola (1990) conducted experiments in a 6.0 ft wide flume using a 0.375 ft rectangular-nosed pier model where rock was mounded around the pier in some

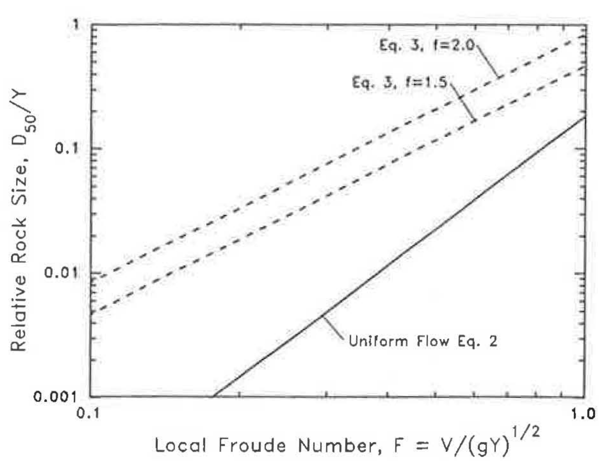

FIGURE 1 Comparison of Currently Recommended Uniform Flow and Pier Protection Rock Sizing Equations

test runs, and was placed within preformed scour holes of various depths in other runs. A cylindrical 0.375 ft diameter pier model with rock mounded around the pier was also tested. All of the experiments were conducted using uniform gravel to model riprap under subcritical flow conditions with a Froude number range from 0.2 to 0.6. Quazi and Peterson (1973) defined failure as the first displacement of a single rock particle, while Parola (1990) defined failure as the removal of the top layer of gravel resulting in the exposure of the second gravel layer.

Based on the data from the rectangular-nosed pier experiments, the following conservative equation is proposed

$$\frac{D_{50}}{Y} = 1.0 \frac{F^2}{(S_g-1)} \tag{5}$$

The data from the rectangular-nosed pier experiment, Equation 5, and Equation 3 with f = 1.5 and f = 2.0 are plotted in Figure 2. A specific gravity of 2.65 is assumed.

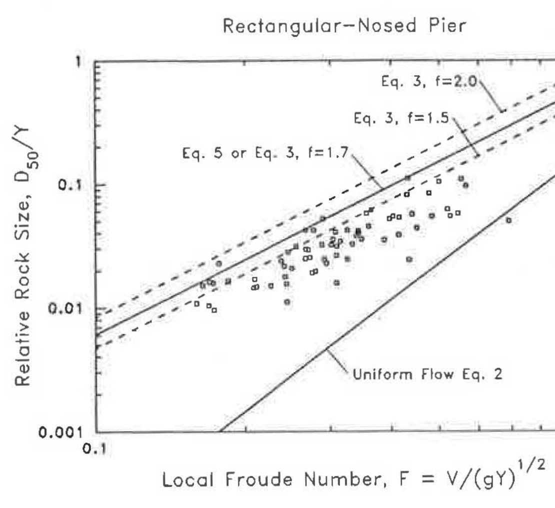

FIGURE 2 Comparison of Rectangular-Nosed Pier Data With the Proposed Eq. 5 and the Currently Recommended Equations for Pier and Uniform Flow Protection

Equation 5 is equivalent to Equation 1 or Equation 3 with f = 1.7. The data scatter is large because data were included from several different experiments where the level of the riprap was varied from well-below the streambed to a mounded condition above the streambed. Analysis of data collected under laboratory conditions in which the streambed was fixed showed that a reduction of approximately 50 percent in rock size could be realized if the rock was placed to a depth of 0.7 pier widths below the streambed. However, in an actual riprap placement, a general lowering of the streambed may occur that would reduce the depth of the protection with respect to the streambed, creating a highly unstable situation; therefore, the size rock as predicted in Equation 5 is recommended without reduction for the depth of placement.

Based on the data from the round-nosed pier (Quazi and Peterson 1973) and cylindrical pier (Parola 1990) experiments, the following conservative equation is proposed

$$\frac{D_{50}}{Y} = 0.61 \frac{F^2}{(S_g-1)} \tag{6}$$

The data from the round-nosed pier experiment, cylindrical pier experiment, Equation 6, and Equation 3 with f = 1.5 and f = 2.0 are plotted in Figure 3. A specific gravity of 2.65 is assumed. Equation 6 is equivalent to Equation 1 or Equation 3 with f = 1.3. Direct comparison of Equation 5 and Equation 6 shows that the rock sizes required to protect round-nosed and cylindrical piers are 40% smaller than those required to protect rectangular-nosed piers.

The results of the model studies indicate that the form of Equation 1, with slight adjustment to the values of f to compensate for the increased turbulence at the piers, is useful in determining the rock size necessary to protect bridge piers from scour.

As is implied by Equation 1, the rock size required to protect piers was not found to be dependent on flow depth, Y, unlike the rock sizes predicted by the equation for uniform flow conditions.

RECOMMENDED APPLICATION

The small-scale model studies were conducted using model piers of simple

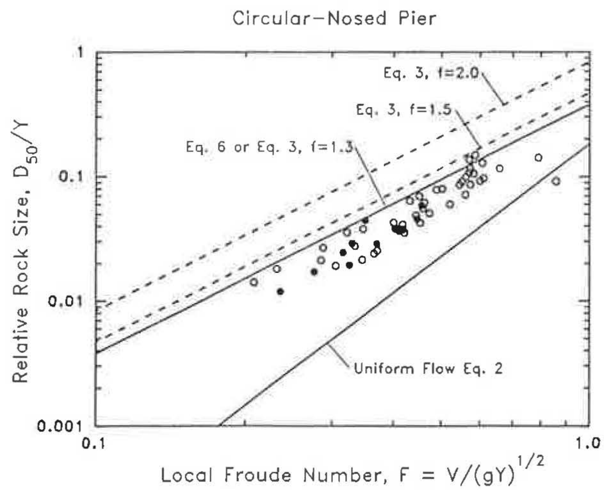

FIGURE 3 Comparison of Round-Nosed Pier Data and Cylindrical Pier Data With Proposed Eq. 6 and the Currently Recommended Equations for Pier and Uniform Flow Protection

geometry and uniform rock sizes. Although some older bridge piers may have rectangular-shaped noses, most modern piers have rounded or pointed noses and are supported on rectangular spread foundations or rectangular pile caps. Equation 1 with f = 1.3 is applicable for use with round-nosed piers that are fairly aligned with flow, that are not subjected to ice jams or debris accumulation, and that are located in bridge openings where the general bed elevation is expected to remain above the rectangular pile cap or spread footing. Equation 1 with f = 1.7 is recommended for piers in which the square foundation is expected to be above the general bed elevation, or for piers that are founded on square footings which are likely to be exposed by general scour throughout the bridge opening. Experiments on pointed-nosed piers or piers skewed to the flow have not been performed, although Equation 1 with f = 1.7 should provide a conservative estimate of rock protection size for these conditions. A safety factor should be applied to the rock sizes obtained.

The laboratory experiments were conducted using uniform gravels in at least three layers. Ettema (1976) has shown that for clear-water local scour, well-graded bed material tends to scour less than uniform material. Based on this research, a speculation is made that well-graded riprap with D_{50} equal to the uniform rock sizes recommended for pier protection in this paper will be conservative. As a design criteria for failure of a protective mat, the layer thickness of the protection is recommended to be at least three times D_{50}, as was used in the model experiments. Further research should be conducted to determine the effects of riprap gradation and layer thickness.

CONCLUSIONS

A comparison of the results of small-scale laboratory studies indicates that the equation recommended by FHWA with slight adjustment to the factors used to accommodate pier turbulence are well suited for determining rock sizes to protect rectangular-nosed, cylindrical, and round-nosed bridge piers from scour. Factors to account for pier turbulence were recommended based on the results of the small-scale model studies. Recommendations for the application of the equations were provided.

ACKNOWLEDGEMENT

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Scour Monitoring Devices for Bridges

P. F. LAGASSE, C. F. NORDIN, J. D. SCHALL, AND G. V. SABOL

ABSTRACT

Research efforts have developed a large body of knowledge on bridge scour, mostly from laboratory model studies. However, field data and measurements of scour at bridges, necessary to better understand the problem of scour and to evaluate analytical methods for scour prediction, are extremely limited. This deficiency results largely from the difficulties of field data collection under flood flow conditions when scour conditions are typically most severe. These same adverse conditions have inhibited development of a reliable scour monitoring device that can automatically or even semi-automatically collect scour data.

This paper presents a status report on the ongoing National Cooperative Highway Research Program (NCHRP) study being conducted for the Transportation Research Board (TRB). The objective of this research study is to develop, test, and evaluate instrumentation that would be both technically and economically feasible for use in monitoring maximum scour depth at bridge piers and abutments. An Interim Report was submitted to the NCHRP panel and TRB in October 1990 providing identification of potential devices, evaluation of devices, and recommendations for the laboratory phase of the research.

INTRODUCTION

Highway bridge failures cost millions of dollars each year as a result of both direct costs necessary to replace and restore bridges, and indirect costs related to disruption of transportation facilities. However, of even greater consequence is loss of life from bridge failures. Stream instability, long term stream aggradation or degradation, general scour, local scour and lateral scour or erosion cause most of these failures.

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Research efforts have developed a large body of knowledge on bridge scour, mostly from laboratory model studies. However, field data and measurements of scour at bridges, necessary to better understand the problem of scour and to evaluate analytical methods for scour prediction, are extremely limited. This deficiency results largely from the difficulties of field data collection under flood flow conditions when scour conditions are typically most severe. These same adverse conditions have inhibited development of a reliable scour monitoring device that can automatically or even semi-automatically collect scour data.

There are many scour vulnerable bridges on spread footings or shallow piles in the United States. With limited funds available, these bridges cannot all be replaced or repaired. Therefore, they must be monitored during and inspected following high flows. During a flood, scour is generally not visible and during the falling stage of a flood scour holes generally fill in. Therefore, visual monitoring during a flood and inspection after a flood cannot fully determine that a bridge is safe. A reliable device to measure maximum scour would resolve this uncertainty.

RESEARCH OBJECTIVES

In December, 1989, Resource Consultants, Inc. (RCI) was selected by the National Cooperative Highway Research Program (NCHRP) to conduct Research Project 21-3. RCI is supported by Colorado State University (CSU) and Electronic Techniques, Inc. (ETI) on this research program. The objective of the research is to develop, test and evaluate instrumentation that would be both technically and economically feasible for use in monitoring maximum scour depth at bridge piers and abutments. The scour monitoring device(s) should be low cost, reliable, and capable of installation on or near a bridge pier or abutment.

Required criteria for the scour monitoring device(s) include:

* capability for installation on or near a bridge pier or abutment;
* ability to measure maximum scour depth within an accuracy of +/- 1 foot;
* ability to obtain scour depth readings from above the water or from a remote site; and
* operable during storm and flood conditions.

Desirable criteria for the device(s) are:

* capability to be installed on most existing bridges or during construction of new bridges;
* capability to operate in a range of flow conditions;
* capability to withstand ice and debris;
* relatively low cost;
* vandal resistant; and
* operable and maintainable by highway maintenance personnel.

By March 1991, the first phase of this 2-1/2 year study will have been completed and laboratory testing of selected devices will be underway. This paper summarizes (as of November 1990) the results of comprehensive literature search, identification of devices for consideration, and initial evaluation of devices. The recommendations of the Research Team to the NCHRP panel are also summarized.

SCOUR MEASUREMENT - AN OVERVIEW

Historical Background

Early Observations

The earliest observations of bridge scour probably were carried out by railroad engineers during the first half of the 19th century. By the turn of the century, hydraulic engineers were involved in laboratory model studies of pier scour, but the main responsibilities for design and field studies were borne largely by the railways. The tradition continues today. In the People's Republic of China, bridge scour studies are carried out by the Ministry of Communications and Academy of Railway Sciences; in India field observations apparently are carried out by the Ministry of Railways (1); and in the USSR, the Ministry of Transport Construction has these responsibilities (2).

Equipment used for scour observations was simple: sounding rods for shallow flows and lead sounding weights on a line for deeper flows. Both of these devices were developed to sound for navigation depths hundreds of years ago, and were adapted for depth soundings in connection with stream flow measurements during the 19th century. The main adaptations involved streamlining the sounding weights and using stay lines or vertically supported sounding rods so that the weights or rods would not be swept downstream in high velocities. Early streamflow measurements were often made at ferry crossings, and special supports were designed to mount on the ferry cable so that the sounding rod with the meter could be held vertically approximately three feet in front of the bow of the ferry. The history of these developments is not clear, but they were standard operations in stream gaging early in the 20th century (3). The earliest record we could find of a vertically supported sounding rod permanently mounted on a bridge pier to measure scour was one designed in 1921 by Thomas Maddock for a bridge in Arizona.

Developments in the 1950's

Major advances in instrumentation occurred during the Second World War. By the mid-1950's, many devices became commercially available and were introduced into scientific studies of rivers. The main advances were in sonar, sonic sounders, electronic positioning equipment, and radar. Very often, equipment developed for one special purpose was modified and adapted to studies in rivers. For example, the dual channel stream monitor described by Karaki and others (4) was an ultrasonic sounder that had its origins in a device designed for ultrasound imagery of the human body developed by the Colorado Medical School and used to measure the thickness of fat layers on cattle. This same device was used to study alluvial channel bed configurations and the scour and fill associated with migrating sand waves. The equipment was highly accurate, but could not operate in depths greater than about 6 or 8 feet. Richardson and others (5) developed a sonic sounder for use in the laboratory and in shallow flows in the field. The sounder would work in flows from 1 to 6 feet. It had a very narrow cone and in 2 feet of water would only sound an area of the bed of 1/4 inch in diameter.

Commercial sounders such as the Bludworth and Raetheon (use of trade names is for identification purposes only, and does not imply endorsement by the authors

or sponsors) became available in the 1950's and soon were used extensively in hydrographic surveys. These also were used extensively to monitor scour and fill associated with migrating sand waves (6). This equipment was fairly accurate and could cover a great range of depth, but it could not operate in depths below about 3 feet, so it did not find much application in laboratory studies or in studies in small shallow streams.

Much of the impetus for bridge scour studies in the United States came from the pioneering work of Emmett Laursen and his co-workers at the University of Iowa. One extremely important piece of equipment, a scour meter, was developed for the model-prototype studies of the Skunk River. It was mounted in the streambed upstream of the pier and could sense the water-sediment interface at the streambed. This device, which operated on the electrode-impedence principle, was developed and designed by Philip G. Hubbard, and was described by him in an appendix to the report on scour around bridge piers and abutments by Laursen and Toch (7). For field application, the electrodes were mounted at six-inch spacing in a one-foot square concrete pile that was driven into the streambed upstream of the pier.

Current Practices

Current practices refer to those developed during the past 25 or 30 years and to techniques and equipment that are in use today (1990) in a variety of ongoing field studies. For these latter cases, not much information has been published, so we have drawn heavily on personal communications with individuals who are carrying out the studies.

There are no standard methods or equipment for collecting scour data in the United States. In part, this is because there has been no coordinated long-term effort to study scour processes. Also, most scour studies are site-specific and the equipment and techniques that are used have to be tailored to the geometry of the site and the peculiarities of the existing hydrology and hydraulic conditions. However, a few trends can be identified. First, sonic sounders are standard equipment in most scour studies, but in many cases they will not work and more traditional methods are employed. Second, mobile teams are generally more effective in collecting scour data than relying on fixed installations. Third, there is a growing interest in the use of fixed installations, largely because improvements in data loggers, data transmission, and computers allow the collection and processing of large amounts of real time data.

Earlier models of sonic sounders were heavy, cumbersome to use, and not too reliable. Nonetheless, their potential was recognized, and a number of special investigations were undertaken to develop the equipment and techniques for scour studies (4) (8) (9) (10) (11). Fixed installations were tried in a number of instances, but these met with limited success because of ice, debris, and mechanical problems related to the installations (Norman, 1990, personal communication). Pandy (1) reported that scour measurements were made with a sounding weight on a line before and during construction of Mokameh bridge across the Ganges; after the bridge was completed, a depth sounder was mounted from a sounding weight on a short cable and later on an L-shaped bracket that could monitor scour depth at 16 points around the bridge pier. Later, this system was abandoned and data are now being collected from a boat.

Scour studies are currently carried out with a great variety of equipment and techniques. Since pier and abutment scour are major concerns for operation and maintenance, many bridges are now inspected on a regular basis. Techniques for determining the extent of local scour include the use of divers and visual inspection, direct measures of scour with mechanical and electronic devices and indirect observations using ground-penetrating radar and other geophysical techniques (12). Only a few of the techniques currently employed are suitable for applications in fixed installations.

Selected Examples

Although sonic depth sounders are clearly the most versatile and widely used piece of equipment for detecting the water-sediment interface at the bed of a stream, there are some conditions under which they do not work. For example, along the Yellow River, sediment concentrations near the bed are so high that the standard sonic sounders cannot distinguish between the moving sediment and the non-moving bed (Long Yuqian, Chief Engineer, Yellow River Conservancy Commission, 1983, personal communication). At the Old River Control structure along the Mississippi River, scour measurements are made with a lead weight on a line because the highly turbulent flows entrain so much air that sonic devices will not work. In general, sonic devices do not perform satisfactorily if there are high sediment concentrations, debris, or air entrained

in the flow. For these kinds of flows, simple mechanical devices may still be the most satisfactory instruments for measuring local scour.

Vertically supported sounding rods have been used in depth soundings for decades and for measuring scour at bridge piers at least since 1921. They presently are being used to monitor scour depth at bridge piers at a number of locations. A telescoping rod was mounted on an experimental bridge pier in New Zealand about 1980, several simple devices have been mounted on bridge piers in Arkansas, and a commercially available device, the Brisco Monitor, is being used in the State of New York. So far as we can tell, all of these devices work satisfactorily although only recently, in Arkansas, have there been floods of any magnitude since installation of the sounding rods. There is no evidence that these have been used in sand bed streams or that they have been tested to determine if the base plate of the sounding rod resting on the stream bed introduces scour in addition to that caused by the presence of the pier.

During recent years, many scour studies have been undertaken in New Zealand. One of the developments now being used in the field to measure maximum scour depth at bridge piers is called the "Scuba Mouse." The device consists of a pipe mounted to the bridge pier around which is placed a horseshoe shaped collar that rests on the streambed and sinks to the bottom of the scour hole during the flood. On falling flood stages the collar is covered with sediments as the scour hole refills. Its position is determined by sending a detector down the inside of the pipe after the flood. Earlier models involved a metal detector inside a PVC pipe, but the pipe was sometimes damaged by debris, so the current models use a steel pipe, a radioactive collar, and a radiation detector inside the pipe. This device is operational and has been installed on several bridges in New Zealand (Robert Ettema, University of Iowa, 1989, personal communication) (13).

At present, the U. S. Geological Survey Hydrologic Instrumentation Facility (HIF) has made a small effort in developing equipment for bridge scour studies. They have designed several conductance probes. One was used about 15 years ago in Arizona, and a more recent model was used in Arkansas. These devices apparently worked, but there were difficulties in collecting ground truth, and installation of the meters was difficult. HIF also has evaluated data loggers and developed the component circuitry so that signals from sounders could be entered directly to the loggers (James Ficken, USGS, 1990, personal communication).

One of the more ambitious and successful investigations using fixed installations is underway at a new bridge site on Federal Highway 101 across Alsea Bay near Walport, Oregon. Stage, velocity, and scour depth are monitored every fifteen minutes and stored in a data logger that can be accessed by phone to enter the data directly into a computer. Velocities are sensed with Marsh-McBurney and Montedoro-Whitney electromagnetic flow meters, and depth soundings are made using Lowrance and Eagle sounders. The transducers for sounding are mounted on angles attached to the piers and pointed out slightly to avoid interference from detecting the side of the pier. The system has worked well, but the installation is not subject to much debris, ice, or air entrainment from highly turbulent flows. (Milo D. Crumrine, USGS, 1990, personal communication)

The USGS, in a cooperative scour study with the New York State Department of Transportation, has installed Brisco Monitors and Datasonics sonar altimeters to measure scour (Gerard K. Butch, USGS, 1989, personal communication). In this USGS/NYSDOT study remote telemetry of data is also used. Also, the New York Thruway Authority has installed Brisco Monitors to monitor their scour critical bridges (Keith Giles, NYSTA, 1989, personal communication). In addition, the Virginia Transportation Research Council has plans to install Brisco Monitors and sonic devices on the I-95 bridge at the Virginia/North Carolina state line (Daniel D. McGeehan, Virginia Transportation Research Council, 1990, personal communication). These field investigations should provide valuable information on sonic sounders and the use of rods to measure scour or provide a warning when scour has occurred.

Equipment used in geophysical surveys is finding increased applications in scour studies (12). Impulse radar, lasers, and multi-channel sounders appear to be the most promising techniques, but these devices are expensive and often the results are difficult to interpret. It may be unlikely that they could be adapted to fixed installations within reasonable costs at this time, but such devices will be considered and evaluated.

RESEARCH APPROACH

Overview

The following sections provide a brief discussion of our planned approach for the performance of each task of the research work plan for NCHRP Project 21-3. These tasks are:

1. Comprehensive Information Gathering
2. Identification of Potentially Feasible Measurement Devices
3. Evaluation of Devices Identified
4. Preparation of Interim Report and Test Plan
5. Development of Prototype Devices
6. Testing and Evaluation of Prototype Devices
7. Cost Analysis of Each Prototype Device
8. Preparation of Final Report

Task 1: Comprehensive Information Gathering

The first requirement for the project was to complete comprehensive information gathering. Specifically, a review of all relevant domestic and foreign practice, performance data, and research findings directly related to field measurement of scour at bridge crossings, or indirectly related research that could be transferable to the highway industry, was completed. A survey of equipment that has been proposed previously for bridge scour monitoring was also completed.

Many of these activities will continue through the research project. Any information obtained through these activities will be used to update our annotated bibliography periodically. Appropriate findings are included in the Interim Report, Task 4, and will also be included in the Final Report, Task 8.

Task 2: Identification of Potentially Feasible Measurement Devices

Task 2 identified electrical, mechanical, or other devices that have been or could be adapted to measure maximum scour at bridge piers and abutments. The identification of potentially feasible measurement devices was completed based on the criteria of significance for the research project. Specifically, this analysis was completed without any evaluation of a given device's feasibility, to insure that no possible device was eliminated from consideration at this stage. With the rapid advances made in instrumentation technology in recent years, a technique that as little as 6 months ago was not feasible, based on cost or other reasons, may now be a practical alternative.

During Task 1 it was found that the techniques and devices for measuring scour depth fall into three general categories: (1) direct soundings to determine the bed elevation; (2) devices attached to the piers or abutments that can indirectly sense the difference between flowing water and the non-moving sediment of the bed; and (3) various devices such as scour collars, buried chains, and tracers. A list of the more common devices and techniques and some of their advantages and disadvantages are given in Table 1.

Direct soundings can be made with mechanical devices, sounding rods and sounding weights on a line, or by acoustic devices, impulse radar, or pulsed laser. Sonic sounders are standard tools in scour studies today. Impulse radar is being investigated for applications in scour studies, and pulsed laser is developed for bathymetric surveys in fairly clear water to depths of about 50 m.

The various devices that indirectly sense the difference between flowing water and non-moving sediment are based on differences in conductance, heat dissipation, or light transmission. The common problems of these devices are how to install and secure them on existing piers and abutments, anticipating the expected depth of scour, and reliability under conditions encountered during a flood.

The use of scour collars, scour chains, and various tracers are generally limited because they have to be recovered after the flood. These are used mostly in arid regions where the stream beds are dry much of the time. However, miniature transmitters such as those used to track fish and game show considerable promise. For example, transmitters of various frequencies that are motion-activated could be buried in a grid around a bridge pier and detected by a downstream receiver when the depth of scour reaches them and they float out.

Task 3: Evaluation of Feasible Devices

The evaluation of feasible devices was based on a comprehensive analysis and screening program. For each device this analysis considered the advantages, limitations, purchase and installation costs, potential problems and failure modes, and other important features or considerations (see additional discussion under Task 5). Based on this evaluation, those devices that showed promise for

TABLE 1. SUMMARY OF SOME TECHNIQUES OR DEVICES FOR MEASURING LOCAL SCOUR AT PIERS AND ABUTMENTS

TECHNIQUE OR DEVICE	ADVANTAGES	DISADVANTAGES	REMARKS
Direct Bed Elevation Soundings			
Mechanical			
Sounding Rods	Inexpensive, simple, rugged, accurate	Cannot be used in debris-laden flow, great depths, high velocities	Could be used unattended by mounting in a pipe on the pier or abutment, lowering and raising with an electro-mechanical system, from the bridge deck or the bank. Need to develop a tensiometer to sense the contact with the bed, reversing switch method to measure distance traversed and to transmit this information to a remote location.
Sounding weight on a line	Same as sounding rod	Same as sounding rod	Probably cannot be used unattended except for almost-ideal conditions.
Electrical			
Acoustic	Inexpensive, accurate	Problems with debris, air entrainment under transducer, and extremely high sediment concentrations	This is standard equipment for most scour studies today. Most work is done from boats or the bridge deck. A complete system for this costs about $700.00. Needs development for weatherproofing, transmitting signal to the bank or a remote site, etc.
Impulse Radar	Can "see" through air, ice, water, sediment	Expensive, cannot "see" through organic debris	USGS-FHWA are conducting research on applications of Ground-Penetrating Radar to bridge scour studies.
Pulsed Laser	Can "see" through air, water	Clay particles in water cause problems. Expensive.	Presently operational for airborn bathemetry to about 50 m. in coastal waters and clear rivers. Possible applications to scour studies have not been investigated.

TECHNIQUE OR DEVICE	ADVANTAGES	DISADVANTAGES	REMARKS
Indirect Bed Elevation Determinations			
Conductivity	Designed to operate unattended.	May be difficult to mount on existing bridges.	Field tested by University of Iowa in 1954.
Heat Dissipation Gage	Same as above	Same as above	Same preliminary design by Federal InterAgency Sedimentation project. May be possible to fabricate using casting resins.
Photo-Electric Cells	Unknown	Same as above	Photo-cells mounted in a plastic pipe have been driven into the stream bank and used to monitor bank erosion. Applications to scour studies have not been investigated.
Pressure Transducers	Unknown	Must move up and down with the stream bed	Can measure depth to plus or minus 2% with relatively inexpensive system.
Tracers and Other Methods			
Scour Collars	Simple, inexpensive	Must be installed and recovered	Have been used on small circular piers. These tend to induce some additional scour in sand-bed streams.
Buried chains, tapes, etc.	Same as above	Same as above	Have been used to obtain general scour in arid-region streams.
Tracers			
Colored, fluorescent	Same as above	Same as above	Have been used to a limited extent in arid regions.
Magnetic	Same as above	Same as above	Have been used to a limited extent. Can be detected through about 60 cm of sediment.
Radioactive	Easy to detect	Hazardous to use	Environmental concerns limit use today.
Miniature Radio Transmitters	Easy to detect	Expensive unless recovered	Can be designed to float out of or tumble into scour hole.

further development and testing were identified. The screening of equipment measures and procedures resulted in an identification matrix summarizing all pertinent information on each device and an evaluation matrix. It was recognized at the outset of this research that it is not likely that any single device will meet the development objectives on all bridges or under all hydraulic and geomorphic conditions.

Task 4: Preparation of Interim Report and Test Plan

Upon completion of Tasks 1-3, an Interim Report was prepared documenting the findings of these tasks and proposing a Test Plan for evaluation of those devices selected during Task 3. Pertinent information documenting results of Tasks 1-3 included complete identification of all sources of information and a detailed discussion of the evaluation process. These were provided so that NCHRP reviewers have adequate information to independently evaluate potentially feasible devices. A cost estimate for developing and testing identified devices was also provided.

Development of the test plan was centered around the Hydraulics Laboratory at Colorado State University, which can provide prototype testing under controlled laboratory conditions. The Task 6 discussion highlights the specific prototype test conditions possible at the Hydraulics Laboratory for the testing, including the range of approach depth, velocity, pier size, etc.

Task 5: Development of Prototype Devices

This task will not be initiated until NCHRP has reviewed and approved the Interim Report. Development of prototype devices will be a team effort, involving both river engineering expertise and instrumentation expertise. Development of prototype sensors/electronics will be based on information obtained from Tasks 1 through 4 - the review, identification, and evaluation of existing, proposed, or conceptual devices which meet certain criteria.

Throughout the development, design and fabrication, the following factors and criteria will be given strong consideration:

* Hydraulic factors, flow patterns and possible complications from turbulence generated by the device itself.
* Mounting arrangements that protect the device from debris, ice and vandals such that it is operable during storms and flooding conditions when scour is most likely to occur.
* Ability to measure maximum scour depth to +/- 1.0 ft. and to operate over a broad range of flow conditions. Measurement and readout electronics must ultimately provide these readings at a safe location, such as the bank or possibly a remote site.
* Installation on most existing bridges and during construction of new bridges.
* Long term reliability, low maintenance and serviceability by highway maintenance personnel.
* A design that produces a relatively uncomplicated device that is simple to build, low in cost, and easy to ship.

Given all these factors, it is important to recognize that there may not be one single device that applies to all bridges and/or stream conditions.

Task 6: Testing and Evaluation of Prototype Devices

Laboratory Test Program

Once the prototype scour measurement devices/instruments have been developed and fabricated, each device will be laboratory tested and its performance evaluated at the Hydraulics Laboratory of Colorado State University. The prototype devices will be tested in two stages using the following facilities:

1. The prototype scour measurement devices will first be tested using a 1:50 scale pier model. The model will be placed in a tilting recirculating flume with dimensions of 200 feet long, 8 feet wide and 4 feet deep. The pier will be placed in the simulated stream channel of erodible bed material in which scour depths of over 2 feet can potentially develop. The prototype device will be mounted to the model pier and activated. The flume is capable of flows of over 100 cfs at bed slopes of up to 2 percent. Each device will be tested in clear water under controlled laboratory conditions.

2. When the scour measurement device proves to be reliable in the first stage, the device will be tested near full scale, under prototype conditions. An existing concrete flume 180 feet long, 20 feet wide and 8-13 feet deep will be modified and a stream channel will be simulated. A pier will be constructed and installed in erodible bed material on the stream bottom. The facility is capable of discharging over 100 cfs resulting in average channel velocities of 6-9 fps. The test set-up will allow for scour depths around the pier of up to 5 feet. In addition, sediment and/or debris can be injected into the flow to evaluate how debris impact and debris buildup may potentially affect device durability and operation.

The laboratory testing program will allow the Research Team to redesign and/or refine each device based upon laboratory performance. The two-stage laboratory testing at near prototype conditions will allow for a thorough evaluation of each scour measurement device before conducting an expensive, time consuming comprehensive field testing program.

Field Test Program

Field evaluation of the installation and operational characteristics of the most promising device(s) would support the cost analysis (Task 7) and recommendations for the comprehensive field evaluation program (Task 8). A limited field evaluation would provide information not available from the laboratory study regarding operation in the field, installation problems, cost of installation, and practicality of using state or county personnel to accomplish installation. Consequently, a limited field evaluation program of the most promising device(s) applicable to the widest range of geometric and geomorphic conditions will be conducted.

An important element of such a pilot field program will be availability of a suitable bridge site subject to high discharges on a predictable basis, i.e., below a controlled release site. If such a site can be located, the field test could be accomplished during the eight months currently scheduled for Task 6.

Task 7: Cost Analysis of Each Device Tested

Cost will be an important factor in the ultimate selection of a scour measuring device, particularly if widespread implementation is to be possible. Cost analysis will include the cost of fabrication, installation, and delivery, as well as operation and maintenance costs. An estimate of costs and time for performing comprehensive field evaluations will be included in the final report (Task 8).

Task 8: Final Report

The Final Report (scheduled for January 1992) will document all research findings, including recommendations for a comprehensive field evaluation program for prototypes that show promise, and an estimate of the related time and cost requirements. As part of the Final Report for this project, it is expected that several prototype devices will be recommended for further comprehensive field studies and testing. For each recommended device, including its measurement, interrogation, and recording component, a summary of expected acceptable operating conditions (ranges) will be provided. This information will include, but not be limited to, the temperature range at which the device will operate properly, maximum sediment transport rate at which the device will provide accurate readings, vulnerability to debris loading and hydraulic forces, etc. These operating conditions/ranges will be considered and evaluated during Task 6 testing.

Expected Results

The results of this research will include a comprehensive survey and evaluation of possible devices for measuring maximum scour at bridge crossings. Detailed testing of the most feasible devices will be completed at near full scale, prototype conditions. Limited field testing of the most promising device(s) will also be conducted. Results will be presented in the final report, with appropriate data presented in graphical and tabular form. Plans and specifications will be included as appropriate. Laboratory testing will also include still photography and video of observed performance.

The research will rank the devices based on completed testing and provide detailed recommendations and criteria for

a comprehensive field evaluation program to include identification of potential sites (Task 8). It is anticipated that the final recommendation of the best device(s) for widespread implementation will incorporate the results of this research effort and the comprehensive field evaluation program. Completion of the comprehensive field evaluation is not part of the scope of work of this research effort; however, involvement of state representatives on the State Advisory Panel will provide the transition from this research program to a comprehensive field testing phase.

EVALUATION OF FEASIBLE DEVICES

Discussion

Devices considered feasible for scour measurement at bridge piers and abutments can be grouped into four broad categories: Sonar, Sounding Rods, Buried (Driven) Rods, and Other Buried Devices (see Table 2). The evaluations in Table 2 are qualitative - Good, Fair, Poor or Unknown. When one of the criteria is marked U (unknown), it generally means that there is insufficient information or that the device has never been used under circumstances to test and establish its performance for that criteria.

While Item 1 (mandatory) rates the capability for installation on or near a bridge pier or abutment, Item 5 (desirable) evaluates the ease of installation. A device may be capable of being installed and rate higher under Item 1, but if it would be difficult to install the device would rate low under Item 5. Moreover, a device that would be easy to install on both existing and new bridges would rate higher under Item 5 than one that could be easily installed on only a new bridge.

The appraisal of Item 8 (desirable) is arbitrary and is based on the estimated cost of procurement, but not installation costs. For purposes here, costs under $2,000 are rated good and costs over about $20,000 are rated poor. In practice, costs need to be evaluated relative to the cost of the structure or relative to the cost of failure of the structure.

A significant number of bridges exist in the United States for which bridge plans are not available, i.e., the foundation and soil properties are unknown. Item 12 (desirable) evaluates the ability of the device to perform successfully and provide scour data where foundation and soil conditions are unknown. Specifically, this criteria rates as less desirable those devices requiring a particular pier or foundation geometry for installation or requiring detailed knowledge of soil and substrate conditions underlying the bridge. In addition, a device which would provide additional information on the below-ground structural foundation of the bridge would be desirable. Item 12 of the evaluation matrix rates such a device higher than one that provides no data on unknown foundation conditions.

Sonar

The first category (see Table 2), Sonar, is a clear favorite for most scour studies, but it is also clear that it cannot operate under all conditions. For example, floods rising over dry beds in desert streams usually pile debris against a pier from the bed up, and it is unlikely that sonar would work under those conditions. Similar conditions might exist at bridge sites that are subjected to flooding from ice jams. Some problems remain to be resolved with sonar, but these relate mostly to the mechanics of mounting the transducer on the structure in such a way that it can be easily replaced during flooding conditions, and to problems associated with loss of signal in flows with very high sediment concentrations. This latter problem can be studied in the laboratory. Laboratory studies also need to evaluate what region(s) of a scour hole is detected with various cone angles and mounting angles.

Sounding Rods

Vertically Supported Sounding Rods also rate fairly high in the evaluation matrix, but they clearly are not suitable for all rivers. For example, it would be very difficult to install a sounding rod for the great flow depth of the lower Mississippi River. A number of questions remain to be resolved regarding these devices: Do they induce scour around their baseplate when installed in sand-bed streams? Will the unsupported length of rod vibrate due to vortex shedding and induce additional scour? In flows with high sand concentration, can sediment entering between the rod and its vertical support members hinder the free sliding of the rod or cause it to bind? Some of these questions can be answered by laboratory testing. Cayuga Industries has provided the Research Team with a prototype of their patented Brisco Monitor for testing.

Table 2. Evaluation Matrix

	Mandatory Criteria				Desirable Criteria							
Device	1	2	3	4	5	6	7	8	9	10	11	12
	Install on Pier	Accuracy ± 1 foot	Read Above Water or Remote	Operate During Floods	Ease of Installation	Range of Discharge	Withstand Ice/Debris	Low Cost	Vandal Resistant	O & M	Reliability	Subsurface Detection
1. Sonar	G	G	G	G	G	G	G	G	G	F	G	G\1
2. Sounding Rod	G	G	G	U	F	P	U	F	F	G	G	P
3. Buried Rod												
Conductance	G	G	G	U	P	G	G	U	G	U	P	P
Optical\2	G	G	G	U	P	G	G	P	G	U	U	P
Motion Activated\3	G	G	G	U	P	G	G	U	G	U	U	P
Mechanical\4	G	G	P	U	P	G	G	U	G	G	G	P
4. Other Buried Devices\5	G	G	G	G	P	G	G	F	G	P	U	P

Legend:

G ood
F air
P oor
U nknown

Notes:

\1 Item 12 - Tuned transducer only (see Table 2).
\2 e.g., Sanyo device
\3 e.g., Wallingford type device, piezoelectric film or mechanical trip transducer
\4 e.g., Scubamouse
\5 e.g., Radio transmitters

Buried (Driven) Rods

The next category of devices in the matrix is Buried (Driven) Rods. These are devices that need to be mounted or driven below the bed on the pier or abutment to the expected depth of scour. Conductance meters, mechanical meters, and optical meters all fall into this category. Due to the below grade installation requirements, all of these devices are rated poor for ease of installation, particularly at existing bridges. The USGS has provided the Research Team with the conductance meter that was installed last year in Arkansas, and some tests on this device in the outdoor flume are recommended. Mechanical devices are presently best illustrated by the New Zealand Scubamouse (see Selected Examples section); but for this particular device remote readout capability is not currently available. Environmental concerns relative to the use of a radioactive collar may also restrict the use of a Scubamouse type device in the United States. One of the problems with the optical device is determining the upper limits of sediment concentrations beyond which it does not function. The only available commercial device is too expensive (about $10,000) to procure for this project, but if it were possible to obtain just the sensor, it could be tested in the laboratory to resolve the sediment concentration issue.

Motion activated sensors are generally described as those devices that have some type of motion sensor at various intervals along the rod, such that when unburied by scour the sensor is free to move and motion is detected. For +/- one foot accuracy, such devices would have sensors at one foot intervals. The Wallingford "Tell Tail" device, which has a patent pending, falls into this category. The Research Team has developed several alternate concepts for a motion detector device, which could be tested in the laboratory. These include a piezoelectric polymer film transducer and a mechanical trip switch probe. Concerns with such a device would include installation, adequate assurance of free motion once unburied, possible erratic indications due to turbulence in the scour hole, and operation under potentially high sediment concentrations.

Other Buried Devices

The final category in Table 2 is "Other Buried Devices" which are considered to be those devices that either float out of a hole or drop into a scour hole. Such devices could include radio transmitters, pressure transducers, or a tethered target. Devices that drop into the hole would likely have to be tethered at an upstream location. Questions about the use of such devices for scour monitoring relate primarily to the physical motion of the device and its interaction with the flow and stream bed. Limited model study testing on the mechanics of this physical motion, without regard to the type of device or detection scheme, would contribute to evaluation of the applicability of these devices as a scour monitoring device.

RECOMMENDATIONS

The Research Team recommendation to the NCHRP Panel is that at least one device from each of the four categories of Table 2 be procured or developed under Task 5 and tested under Task 6. Testing would be done in two phases in either the indoor flume (pre-testing - Phase 1) or the outdoor flume (near-prototype testing - Phase 2) at the Hydraulics Laboratory at Colorado State University. The recommended objectives of the testing for each of the categories of devices are discussed below.

Sonar

Testing is not necessary to prove the technology employed in sonar devices. This has been established through numerous applications to the scour monitoring problem and through adaptation of sonar devices to such related activities as river and reservoir cross section and bottom profiling. Results and performance data are available in the literature and from active field installations for the bridge scour application. Testing of sonar should concentrate on:

* Effects of debris (or ice) on performance and accuracy.
* Effects of high sediment concentrations on performance and accuracy.
* Mechanics of mounting the transducer on the bridge structure (pier or abutment) so that it is accessible and easy to replace.
* Identifying effects of cone angle and mounting angle on the accuracy of the instrument (i.e., how does the device "see" the scour hole, and will it "see" the maximum depth of scour?)

Sounding Rods

While field installations provide some performance data on vertically supported sounding rods, a number of questions regarding the interaction of the rod with the flow and with the stream bed remain. The Brisco Monitor, for example, has been installed on several bridge abutments and piers in New York, but the streams have been of coarser or cohesive bed material. Thus, these field installations have not provided data or experience on performance of a vertical rod on a sand bed stream. A prototype Brisco Monitor is available to the Research Team and should be included in both the pre-testing and near-prototype testing phases to determine:

* The interaction of the rod baseplate with a sand bed, i.e., does the baseplate itself induce scour?
* Will the unsupported length of rod vibrate due to vortex shedding and induce additional scour?
* Is the rod system mechanically reliable with high concentrations of sediment which could cause the rod to bind?

Buried (Driven) Rods

Buried (driven) rods include conductance, optical, motion activated, and mechanical devices. It is recommended that up to three types of driven rods be tested during the near-prototype phase of laboratory testing.

* The USGS conductance probe that had been installed in the field in Arkansas (supplied by the USGS to the Research Team) should be tested to gain experience with a conductance type device, supplement data from the field installation with laboratory data under controlled conditions, and, if possible, determine why the device did not perform satisfactorily in the field.
* A motion activated buried rod device developed by the Research Team should be tested during the near-prototype phase. This could be a rod with either piezoelectric polymer film transducers or mechanical trip switch transducers. Test objectives would include evaluation of installation techniques, performance, reliability, and data acquisition techniques.
* One testing "slot" should be held open to conduct tests on a device to be developed by the Research Team or to be acquired from a manufacturer (e.g., optical sensor, or Wallingford "Tell Tail" device). This device will be selected after consultation between the NCHRP Panel and the Research Team.

Other Buried Devices

Testing of the physical response of a typical buried device (e.g., radio transmitter, pressure transducer, or tethered target) should be conducted during the near-prototype phase with the following objectives:

* Evaluate the response and physical fate of a "dummy" buried device to ascertain the mechanics of physical motion and interaction of the device with the flow and with a scour hole as it develops.
* Testing of detection and data acquisition capabilities of buried type devices should be deferred until the mechanics of motion tests provide better data on the physical fate of these devices on or in the stream bed and scour hole.

SUMMARY

As discussed in the Introduction, highway bridge failures caused by scour or erosion-related processes account for most of the bridge failures in this country, yet our knowledge of scour under field conditions is extremely limited. The hazards of data collection during flood events and the lack of automated instrumentation presently inhibit collection of field data. There are many scour vulnerable bridges on spread footings or shallow piles in the United States. With limited funds available, these bridges cannot all be replaced or repaired. Therefore, they must be monitored and inspected following high flows. During a flood, scour is generally not visible and during the falling stage of a flood scour holes generally fill in. Therefore, visual monitoring during a flood and inspection after a flood cannot fully determine that a bridge is safe. A reliable device to measure maximum scour would resolve this uncertainty.

The ability to measure scour in the field would allow monitoring scour critical bridges so that countermeasures

can be taken before the problem becomes severe, or could provide a possible long term alternative countermeasure for scour in some circumstances. These actions would increase the safety of the traveling public and would reduce the costs of bridge inspection, operation, and maintenance. Thus, the results of this research will be of immediate value to state highway departments, authorities, county and city roadway and street departments, and private bridge owners. In addition, several states have undertaken or will initiate cooperative scour research programs with the U. S. Geological Survey. Instrumentation to measure maximum scour depths at bridge piers and abutments would provide invaluable support to such cooperative research efforts.

The lack of field data on scour requires researchers investigating bridge scour, and those developing analytical tools to predict scour, to rely on laboratory data, usually collected from scale models. However, problems in scaling sediment sizes and erosional processes limit the value of laboratory data acquired from scale models and such data cannot provide reliable confirmation of analytical procedures. Therefore, there is real and immediate need for field measurement of scour and erosional processes at bridge crossings. The development of a reliable, cost effective device(s) for measurement of maximum scour would support efforts to acquire a field data base on scour that would be of great value in development of reliable analytical procedures for scour prediction. Such data would be welcomed by researchers worldwide. Finally, this research could lead to development of early warning systems for impending bridge failures. Such systems could contribute to reducing the tragic loss of life associated with catastrophic bridge failures.

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