Bridge Superstructure Rehabilitation and Strengthening

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Increased demand for higher legal loads and permit overloads on the nation's highway system necessitates the strengthening of a large number of existing short and medium span bridge structures. Frequently, the strengthening of bridge superstructures goes hand in hand with rehabilitation measures of the bridge deck and the road surface, such as structural concrete overlays or widening measures to accommodate additional traffic lanes. As part of an ongoing research project with the California State Department of Transportation, a 12 ft. wide, 60 ft. long section of an existing 25 year old cast-in-place reinforced concrete T-girder bridge was brought from near Fresno, California to the Charles Lee Powell Structural Systems Laboratory at the University of California, San Diego, for full scale investigation of different repair and strengthening measures.

The bridge deck was rehabilitated by a full depth structural concrete overlay and tested under simulated service and overloads. Upon completion of the overlay tests to the flexural yield limit state, the flexural cracks of the bridge girder were repaired by epoxy injection. Two types of strengthening measures were subsequently investigated, namely the more popular external post- tensioning and a new innovative approach in which a prestressed high strength precast concrete panel was added to the existing bridge girder in the form of a thin bottom soffit. Prior and subsequent to the implementation of the repair and strengthening measures, forced vibration tests as well as centric and eccentric working load level tests were conducted to monitor the change in the structural response characteristics. A brief description of the implementation of the repair and strengthening measures is provided together with experimental behavioral data and a discussion on the effectiveness of the various strengthening measures.

INTRODUCTION

General

There is a growing need for rehabilitation, repair and strengthening of a large number of aging short and medium span bridge structures on the nation's highway system. Increased demand for higher legal loads and permit overloads, changes in design specifications, corrosion of existing reinforcement and the need for significant deck resurfacing or overlays necessitates strengthening of existing reinforced concrete bridge structures. Frequently, the strengthening of bridge superstructures goes hand in hand with rehabilitation measures of the bridge deck and the road surface or widening measures to accommodate additional traffic lanes.

The design of rehabilitation measures is commonly based on design experience or available empirical field data. Frequently, repair or strengthening measures replace or add structural components in an existing bridge structure in a composite fashion. However, few design guidelines or specifications based on comprehensive research exist to ensure monolithic composite behavior. With a large number of bridge structures in need of rehabilitation, a systematic and research oriented approach to the design of the rehabilitation measures is essential to ensure an effective and reliable structural systems behavior.

Small-scale laboratory experiments on repair and strengthening measures are only of limited use, since the connection detail between the existing bridge and the rehabilitation measure does not allow for scaling without loss of accuracy for various behavior limit states. Thus, basic data on the effectiveness of repair and strengthening measures can either be obtained from low level nondestructive field tests, destructive field tests or full scale laboratory experiments.

As part of an ongoing research project with the California State Department of Transportation, an 11 ft-6 in. wide, 56 ft long section of an existing 25 year old cast-in-place reinforced concrete T-girder bridge was brought from Fresno, California, to the Charles Lee Powell Structural Systems Laboratory at the University of California, San Diego, for full scale repair and testing of a structural concrete overlay [1]. A basic overview of the bridge geometry is given in Fig. 1(a), together with an outline of the section which was removed and brought to San Diego for full scale laboratory testing. The principal dimensions of the test section are shown in Fig. 1(b). After 25 years of service, the Gepford Overhead became obsolete since the railroad line it crossed was abandoned and Caltrans decided to remove the bridge structure eliminating continuous maintenance. Upon completion of the overlay tests which are summarized in the following section and reported in detail in [2], the fully instrumented bridge section presented a unique opportunity to investigate the effectiveness of strengthening measures for existing bridge superstructures under controlled laboratory conditions. First, the flexural cracks in the bridge girders were sealed by epoxy injection. Subsequently, two types of strengthening measures were investigated, namely, the more popular external post-tensioning and a new innovative approach in which prestressed high strength precast concrete panels were added to the existing bridge girder in the form of a thin bottom soffit. The bottom soffit panel was positioned with galvanized dowel bolts and epoxy grouted to the girder bottom. The panel not only increases the longitudinal flexural capacity, but also improves the transverse load distribution characteristics and bridge aesthetics through a closed bottom soffit. Prior and subsequent to the implementation of the repair and strengthening measures, forced vibration tests as well as centric and eccentric working load level tests were conducted to monitor the change in the structural response characteristics.

A brief overview of the overlay research and a description of the implementation and effectiveness of the

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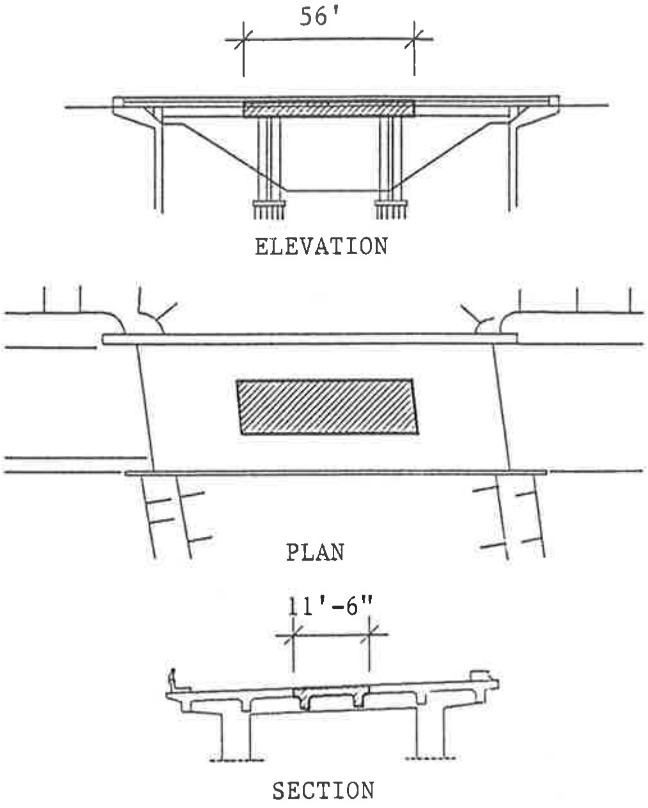
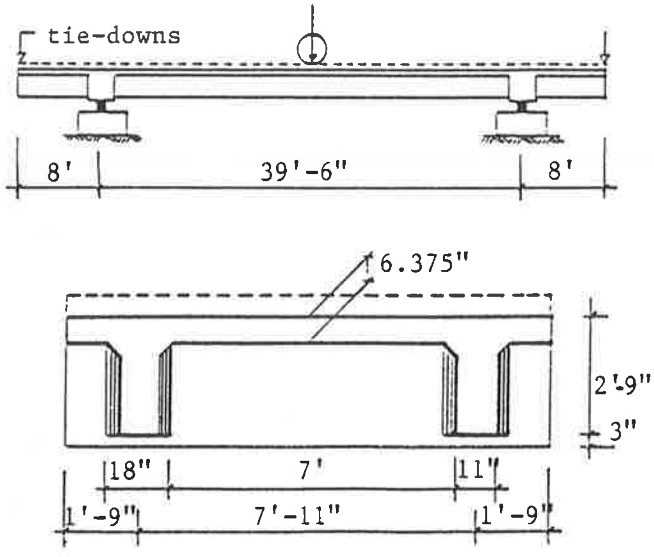


(a) Prototype from Calif. Hwy. 41, Fresno



(b) Full-Scale Test Section Dimensions

FIGURE 1 Gepford Overhead bridge section

repair and strengthening measures together with behavioral data is presented in the ensuing sections.

Structural Concrete Overlay

Repair of individual potholes or scaled-off concrete bridge deck areas typically has limited longevity, and hence prompted the use of full depth structural concrete overlays placed on top of the existing damaged bridge deck in the State of California. Since the contribution of the existing damaged deck is difficult to assess, the full depth overlay is typically fully reinforced. However, this fully reinforced overlay can only be considered effective if horizontal shear transfer between the old deck and the overlay ensures monolithic flexural action. Reinforcing dowels placed across the horizontal construction joint to provide monolithic flexural action are very costly and labor intensive. Moreover, their effectiveness to provide horizontal shear transfer in conjunction with various interface surface preparations is questionable.

The Gepford Overhead bridge section was repaired with a full depth (6 in.) structural (reinforced) concrete overlay in accordance with the design recommendations developed in [1,3] which are based on horizontal shear transfer without dowels [4] for most practical applications. The full depth overlay was placed on a dry and clean surface without any interface dowels for horizontal shear transfer. The surface was sandblasted over one-half the span and scarified over the other half of the span. The overlaid test section is depicted in Fig. 2, together with the vertical loading arrangement and the inflection point tie-downs to effectuate continuous bridge action. The overlay implementation was substantially simplified with the omission of interface dowels and no interface delamination was observed during the overload tests to the flexural yield limit state or after 200,000 dynamic cyclic (5 Hz frequency) wheel load tests at the quarter span points.

A detailed account of the overlay application and the associated test results are presented in [1] and [2]. In the following, the repair and strengthening measures are described first in general, followed by a section on specific behavioral results.

STRENGTHENING MEASURES

Epoxy Injection of the Flexural Cracks

The Gepford Overhead bridge section upon removal from State Highway 41, already exhibited substantial flexural cracking in the positive moment region. The midspan flexural cracks extended through the girders almost to the top deck slab. The yield limit state tests, subsequent to the full depth overlay application clearly amplified and propagated this flexural crack pattern. The presence of nearly forty flexural full penetration cracks in each girder presented an unique opportunity to investigate the effectiveness of epoxy injection of flexural cracks under controlled laboratory conditions.

The tie-downs at the inflection points were released to investigate the grouting of fully open flexural cracks. In preparation for epoxy injection, loose material was blown from the cracks with compressed air. Injection ports were

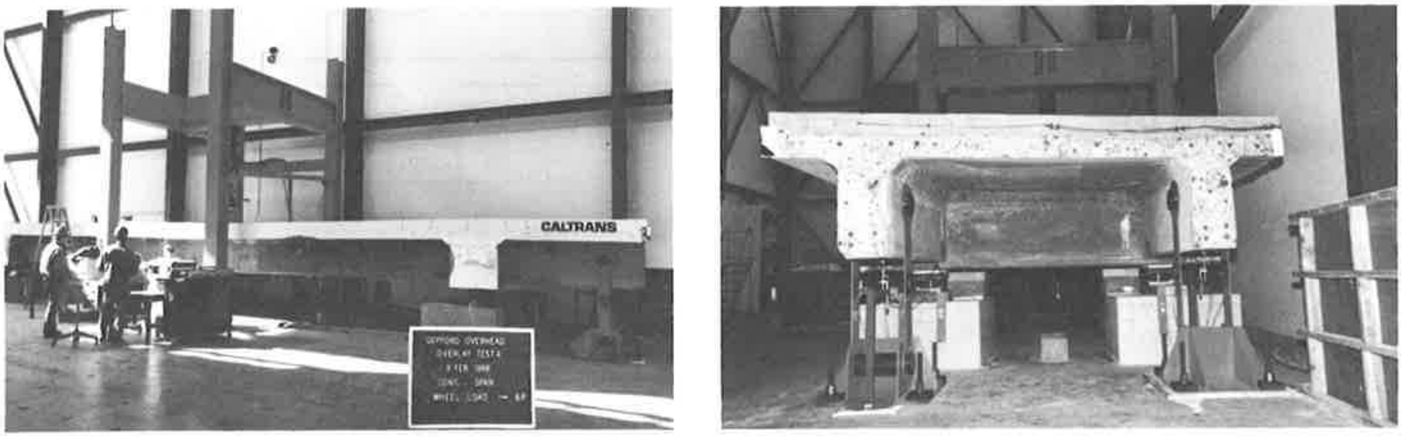


FIGURE 2 Gepford overhead bridge section laboratory setup

placed every 10 to 12 inches along the length of the crack, as shown in Fig. 3(a). The port spacing corresponds approximately to the web thickness to ensure adequate epoxy penetration through the cracks. A two component, rapid curing, smooth paste epoxy gel was used to seal the cracks and secure the injection ports to the cracks prior to pressure grouting. Upon curing of the epoxy seal (approximately two hours), a two component, low viscosity, high strength epoxy adhesive was used for pressure injection of the flexural cracks. Injection started at the lowermost injection port on each crack and progressed from port to port, as soon as epoxy emerged from the next higher port along the crack or a pressure of 100-150 psi was sustained at one port for 30 seconds without the flow of epoxy. Injection was not essential at all ports; if the resin flow was good, the interim ports were only used to monitor penetration. Upon curing of the epoxy adhesive (approximately 1-2 days), the epoxy seal around the cracks was burnt off with a gas torch and the residues scraped off leaving a smooth but discolored surface as indicated in Figs. 3(b) and 3(c).

Low level forced vibration tests performed before and after the epoxy injection of the flexural cracks, showed a 15% increase in the first natural frequency, from 8.8 Hz to 10.42 Hz. This 15% fundamental frequency increase corresponds to a 40% stiffness increase in an equivalent generalized single degree of freedom bridge deck model. Subsequent overload tests showed the development of new flexural cracks in the bridge girders, independent of the existing epoxy sealed cracks.

External Prestressing

The bridge section was tied to the test floor after epoxy injection of the flexural cracks to effectuate continuous bridge action. External prestressing tendons were added at the bottom of the webs to increase the flexural midspan capacity. The external post-tensioning was designed to balance the increased negative moment capacity at the bents due to the full depth structural concrete overlay. Two #8 Grade 150 thread bars were placed along the sides of each girder, see Fig. 4(a), and anchored to the bridge section through the anchor blocks in the webs on one end and through the intermediate diaphragm at the other end of the bridge span.

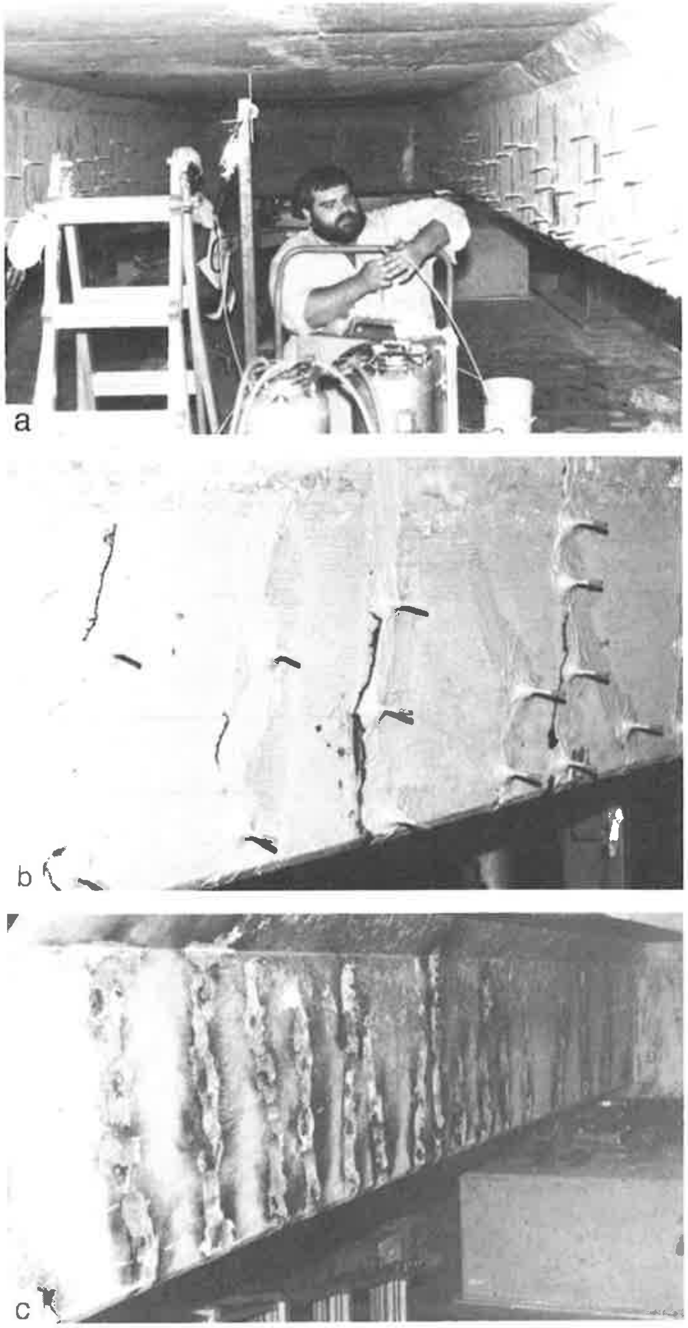
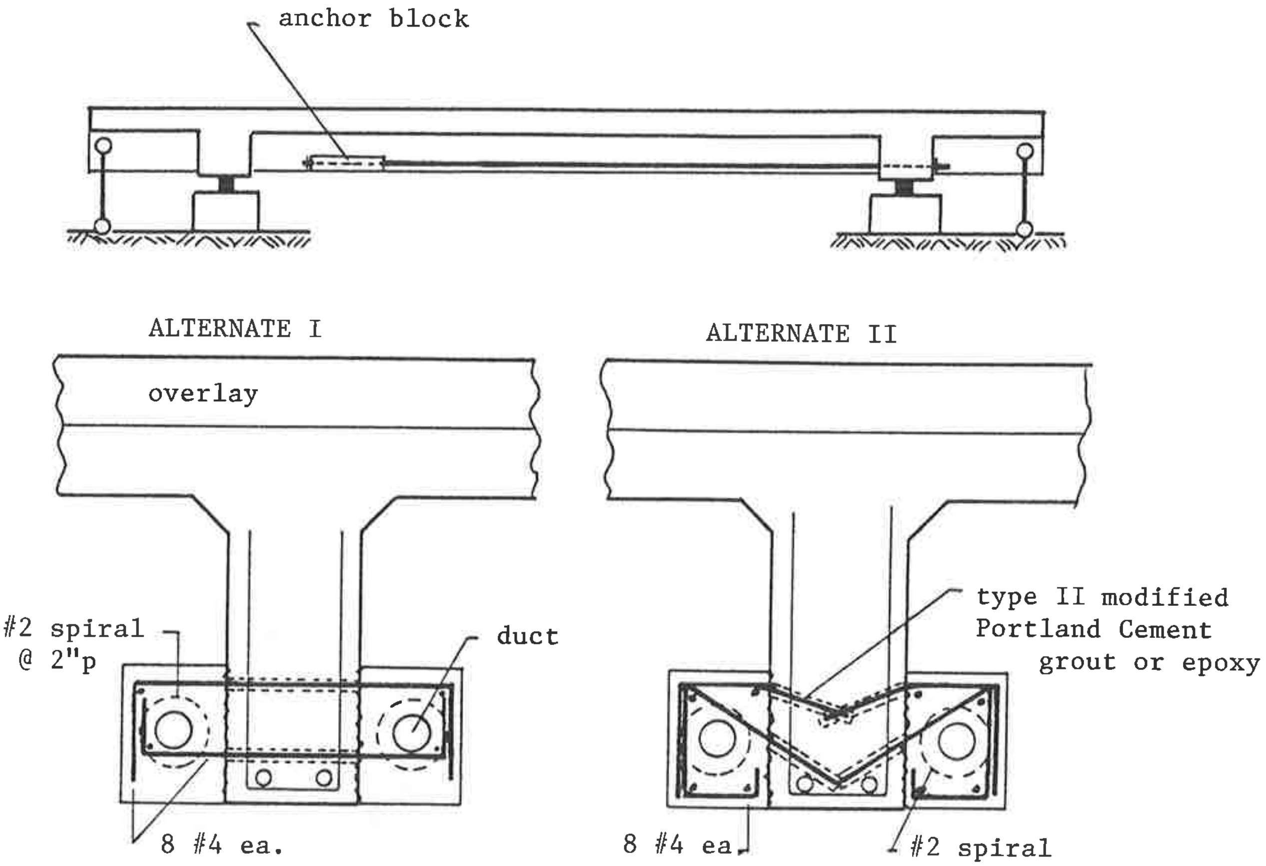
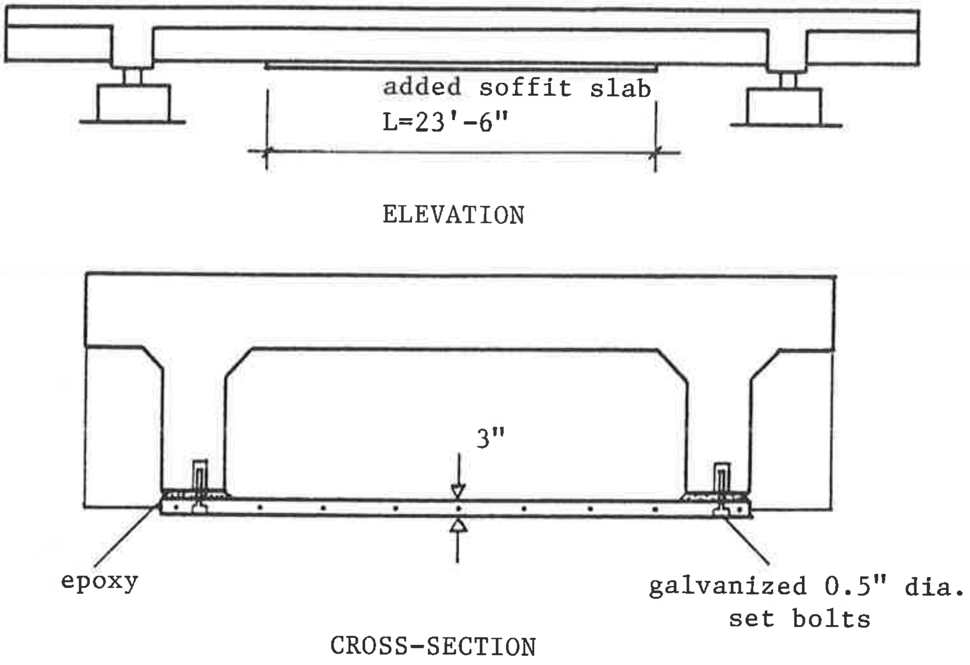


FIGURE 3 Epoxy injection of flexural cracks

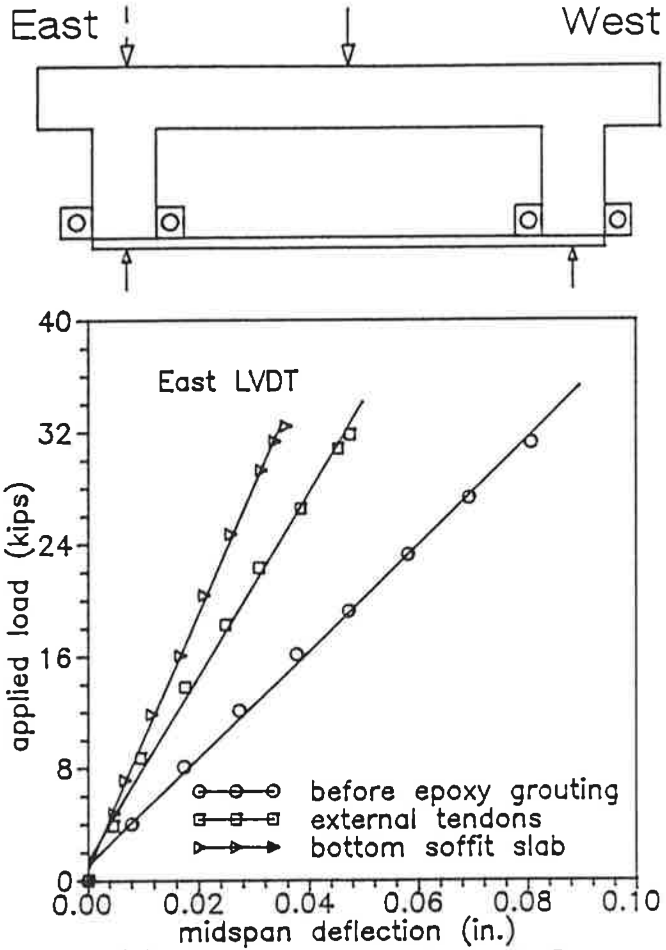


(a) External Tendon Strengthening Measure

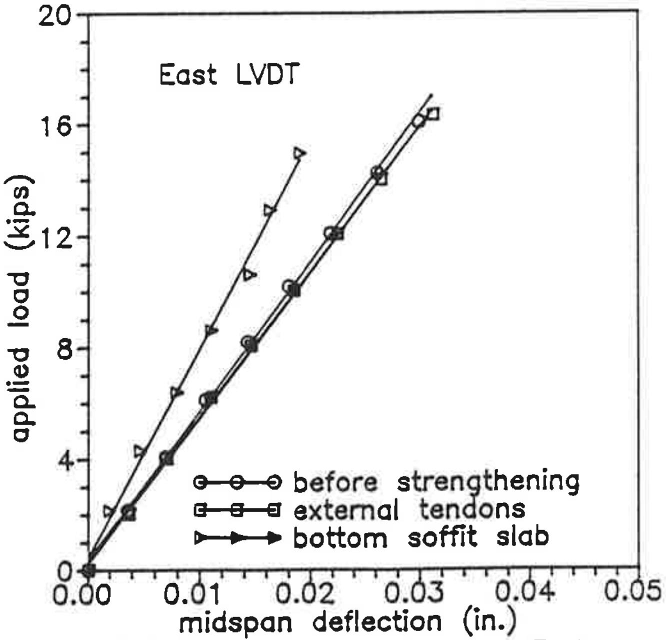


(b) Bottom Soffit Slab Panel Addition

FIGURE 4 External strengthening measures



(a) Midspan Concentric Load Tests



(b) Midspan Eccentric Load Tests

FIGURE 5 Stiffness characteristics of the test section

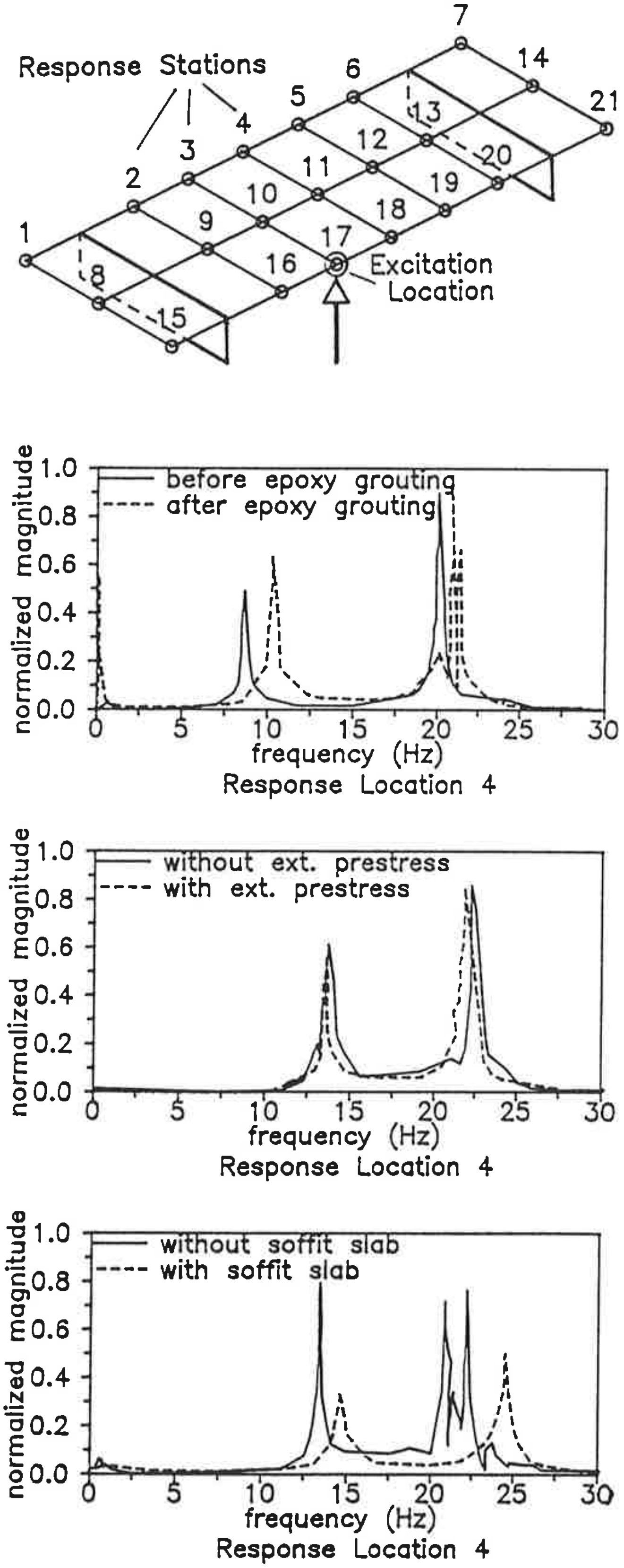


FIGURE 6 Change in dynamic response characteristics

The amount of anchor block reinforcement was based on ACI 318-83, shear friction criteria. Three different ways to anchor and develop the shear friction reinforcement were investigated, namely, through dowels grouted with epoxy in horizontally drilled holes, using the previously described epoxy injection process, and alternatively, individual dowels set in 6 in. deep drilled inclined holes with Type II modified Portland cement grout on one side and epoxy gel on the other side of the girder, respectively, as depicted in Fig. 4(a). All dowel reinforcement consisted of #4 Grade 60 bars pre-bent or bent-in-place depending on the application. The contact surface between the girder face and the anchor block was scarified. The anchor blocks were cast with 5,000 psi concrete and the #8 post-tensioning bars were placed in galvanized pipes for protection. The bars were tensioned to a force level of about 80 kips per bar after the anchor block concrete had reached 80% of its design strength. The galvanized pipes were grouted using Portland cement grout with an expansive grout aid. For post-tensioning, all the anchor blocks were instrumented with Linear Variation Displacement Transducers (LVDT's) in addition to strain gage instrumentation on selected shear friction dowels. No separation or sliding between the anchor block and the bridge girder was observed, which indicates that all three dowel bonding methods were effective.

Service load tests and forced vibration tests indicated virtually no increase in stiffness due to the external post-tensioning. This is again an indication that all flexural cracks were effectively sealed during the epoxy injection process. The bridge section behaved as a typical prestressed concrete structure with no significant flexural cracking even at a load level of 208 kips applied centric between the girder lines at midspan.

Bottom Soffit Panel

The final strengthening measure investigated consisted of the addition of a thin prestressed, high strength precast concrete soffit slab, see Fig. 4(b), after the external tendons were cut. The prestress level in the panel was designed to match the midspan flexural capacity of the external tendon strengthening measure. The panel was manufactured in a 200 ft. long casting bed where nine 0.5 in. ϕ Grade 270 strands were pre-tensioned to 60% f_{pu} prior to the casting of the high strength concrete.

The horizontal load transfer between the new bottom soffit panel and the existing bridge section was designed to be achieved through epoxy bonding, while the self weight of the panel was supported by ten 1/2 in. ϕ threaded rods which were also used to position and clamp the soffit panel in place during installation and the curing of the epoxy. Two different epoxy bonding procedures were employed, namely the gel process where the soffit slab was buttered with 1/2 in. thick epoxy gel along one girder, and the epoxy injection process, where the joint between the soffit panel and the bridge girder was first sealed with epoxy gel and subsequently injected with a low viscosity, high strength epoxy adhesive. In both cases, the bottom of the girder was roughened with a jack hammer and cleaned with compressed air and the surface of the precast soffit slab panel was sandblasted prior to installation.

The addition of the bottom soffit panel resulted in a significant increase in flexural stiffness and improved transverse load distribution characteristics. Overload tests with a midspan point load between the girder lines initiated horizontal shear failure of the girder concrete cover adjacent to the epoxy joint at the south-western corner of the precast bottom soffit panel, at a load level of 176 kips and an estimated horizontal shear stress level of 350 psi.

SYSTEMS EVALUATION

Load Tests

Each structural repair or strengthening measure was evaluated by a series of tests which were performed prior and subsequent to the structural modification. First, forced vibration tests were conducted to evaluate the effectiveness of low level experimental modal analysis techniques to predict the change in the dynamic response characteristics due to various repair and strengthening measures. Subsequent to the forced vibration tests, the test program consisted of midspan point loads applied both centrically between and eccentrically over the girder lines. These loads represented wheel loads at various service load limit states. Finally, these same point loads were used to investigate overload limit states, in some cases, up to the flexural yield limit state in the midspan girder reinforcement.

Point loads were applied at midspan with a MTS servo controlled actuator reacted against a four column load frame depicted in Fig. 2. The load was applied in increments of the AASHTO-83 [5] HS-20 basic maximum wheel load of $P_{20} = 16{,}000$ lbs. without impact allowance. The 8 inch by 20 inch loading area was based on AASHTO-83, Section 3.3 tire contact area of $A = 0.01\ P_{20}$ [in^2] or 160 in^2 with a 1:2.5 aspect ratio. At full multiples of the P_{20} load, three cycles of unloading and reloading were performed to simulate three traffic cycles at that particular load. The point load tests were primarily used to establish comparative load deformation or stiffness characteristics between adjacent girders.

The longitudinal stiffness increases due to the overlay application and continuous support conditions were expected but the most striking result was obtained from the epoxy injection of the flexural cracks, where a substantial stiffness increase was noticed, see Fig. 5(a). This stiffness increase can be completely attributed to the epoxy injection since no additional stiffness increase resulted from the external post-tensioning as is evident from Fig. 5(b).

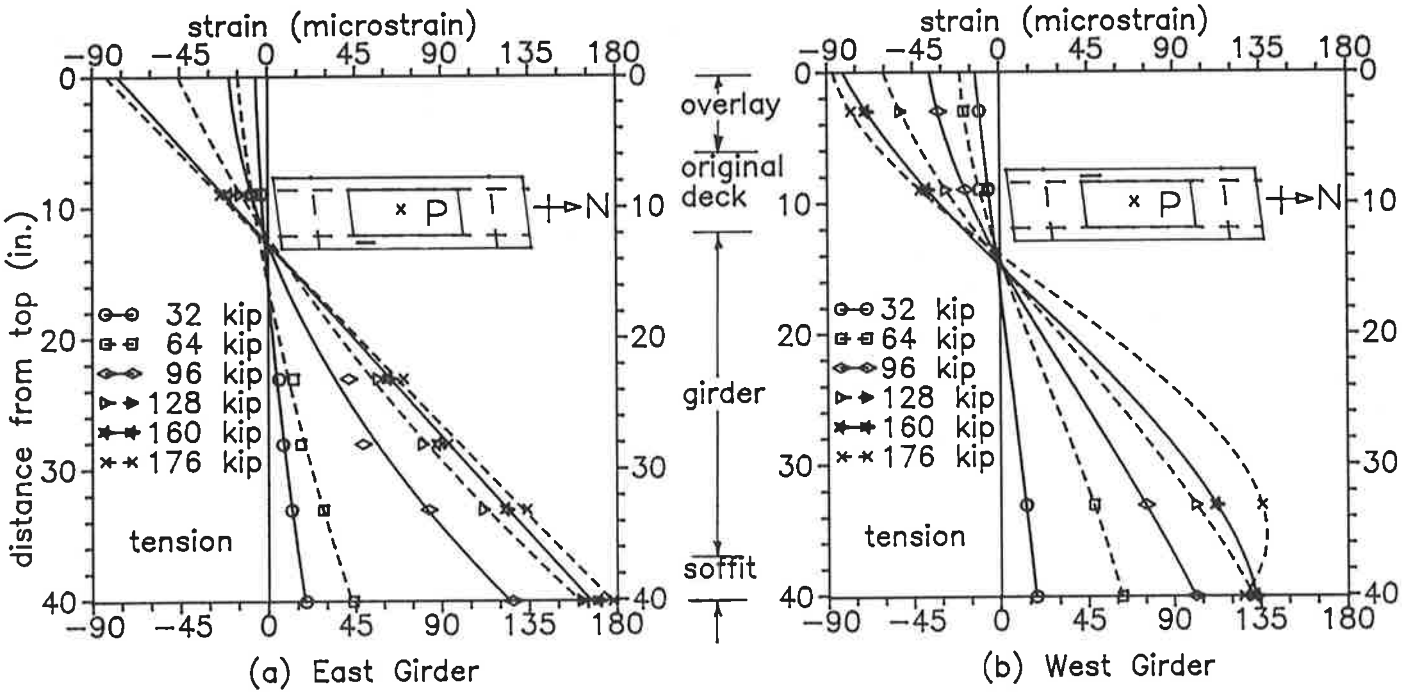
Forced Vibration Tests

Full scale field tests with known service loads are very involved and often prohibited by continued service requirements. Moreover, overload load tests can introduce additional cracking and with it progressive and cumulative structural deterioration. Thus, there is a need for nondestructive field testing to identify the current state of the bridge structure and assess the changes in the structural response due to the various repair or strengthening measures.

Experimental modal analysis with a low level random excitation from a seismic shaker is one such nondestructive testing technique used for systems identification of the bridge section at various stages of the research program. The frequency response was obtained at a number of discrete,

Table 1 - Summary of Response Characteristics of Gepford Overhead Section

Test No.	Bridge Condition	Support Condition	Mode	Frequency (Hz)	Mode Shape Characteristic
(1)	Simply supported	simple	1 2	8.80 20.24	Main span & overhang flexure Main span torsion
(2)	(1) + epoxy grouted cracks	simple	1 2	10.42 20.90	Main span & overhang flexure Main span torsion
(3)	(2) + continuous support	continuos	1 2	14.04 22.70	Main span flexure Main span torsion
(4)	(3) + external post-tensioning	continuous	1 2	13.85 22.00	Main span flexure Main span torsion
(5)	(4) + after traffic load tests	continuous	1 2	13.56 21.66	Main span flexure Main span torsion
(6)	(5) + removal of external tendons	continuous	1 2	13.23 20.90	Main span flexure Main span torsion
(7)	(6) + bottom soffit panel	continuous	1 2	14.50 24.40	Main span flexure Main span torsion
(8)	(7) - continuous support	simple	1 2	10.65 21.80	Main span & overhang flexure Main span torsion



(a) East Girder

(b) West Girder

FIGURE 7 Strain profile on the girder at the south quarter span

strategic locations in the structure to an excitation from a 30 lb. Acoustic Power System vertical shaker. An analysis band of 50 Hz was chosen to capture changes in the fundamental modes of the bridge structure due to the various strengthening procedures.

A numerical evaluation of the first two natural frequencies obtained from the forced vibration tests conducted and the various support conditions are presented in Table 1. The most striking result can be found in a 15% frequency increase in the first mode of the simply supported bridge structure due to epoxy grouting of the flexural cracks, from 8.80 Hz to 10.42 Hz, whereas virtually no additional stiffness increase was observed due to external prestressing. Gluing a thin precast slab panel to the bottom of the double-Tee bridge section increased the fundamental frequency from 13.23 Hz to 14.5 Hz, thereby indicating an increased longitudinal stiffness. A significant increase in the transverse load distribution characteristic was also noted by the change in the second mode frequency from 20.9 Hz to 24.4 Hz on the addition of the precast bottom soffit panel, since the transverse flexible T-girder deck was transformed into a stiff box girder with highly improved torsional characteristics. However, no transverse stiffness increase was noticed due to the epoxy injection of the flexural girder cracks or external prestressing. The shift in the natural frequency is graphically depicted for a midspan response station in Fig. 6. These low level dynamic systems identification results are currently correlated to service and overload test results as well as comprehensive analytical modeling to quantify the effectiveness of rehabilitation measures based on nondestructive systems evaluation.

Overload and Ultimate Load Tests

Midspan point load overload tests were performed subsequent to each strengthening measure up to the flexural yield limit state to correlate low level nondestructive test characteristics with subsequent limit state behavior and to investigate the onset and development of possible failure modes. One such failure mode characteristic for the precast bottom soffit strengthening measure is graphically depicted in Fig. 7 where the strain profiles at a section 10 ft. from the left (South) support are plotted at subsequent overload levels. The nonlinear strain profile in the west girder at higher load levels, see Fig. 7, indicates a force reduction in the bottom soffit panel. This force reduction on the west side of the soffit panel was caused by a horizontal shear failure, see Fig. 8, in the cover concrete of the bridge girder parallel to the epoxy joint at a calculated horizontal shear stress level of 350 psi. Thus, while the epoxy bonding of the bottom soffit panel proved adequate for the overload tests, the necessity to anchor the added soffit panel mechanically through dowels into the core of the bridge girder became obvious for the ultimate limit state. The overload tests were repeated after the horizontal failure crack was sealed again by epoxy injection and the tie-downs detensioned to provide a simple support condition for the bridge section. In the subsequent final ultimate limit state test with an increasing midspan point load the bridge section exhibited a similar horizontal shear failure of the girder concrete cover parallel to the epoxy joint.

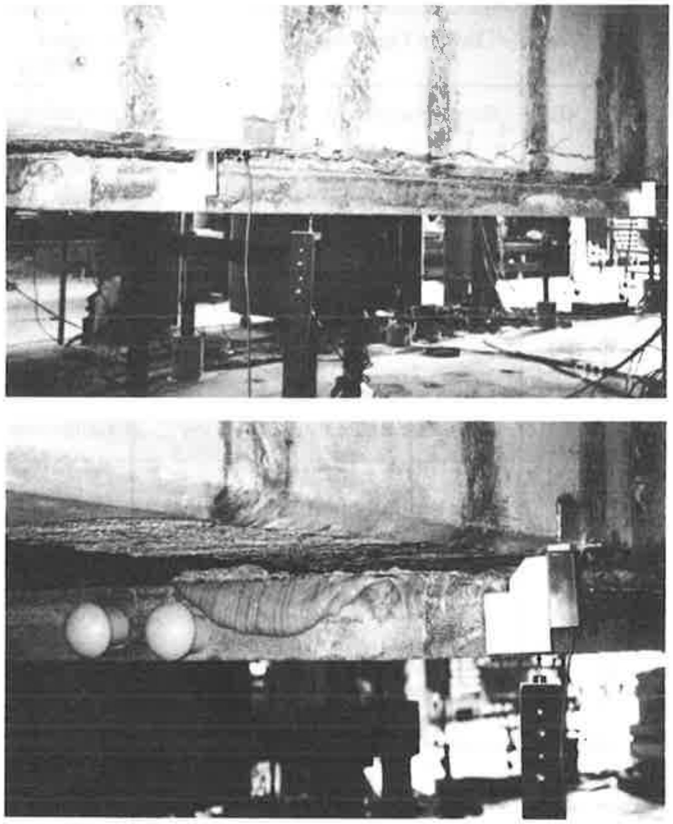


FIGURE 8 Shear failure on the west girder

Finally, ultimate load tests were conducted on the west and east exterior anchor blocks added for the external post-tensioning strengthening measure, see Fig. 4(a), to investigate ultimate limit state characteristics for different anchoring procedures. The west exterior anchor block had 16 # 4 dowels anchored in 6 in. deep inclined holes and grouted with Type II modified Portland cement grout. The east exterior anchor block had 16 # 4 through dowels anchored in horizontally drilled holes, grouted by epoxy injection. The anchor blocks were sheared off with calibrated hollow core jacks and the horizontal anchor block movement as well as the dowel strains were monitored. Fig. 9 depicts the load versus horizontal displacement relationship obtained for the two different anchoring procedures together with the ACI shear friction design level. It is evident that the anchor blocks performed adequate with respect to the shear friction design value. However, the through dowels exhibited a ductile behavior while the set dowels resulted in anchor block slip with significant capacity loss, see Fig. 9, due to slip in the dowel anchorage.

CONCLUSIONS

The repair and strengthening measures investigated were: (1) bridge deck repair with a full depth structural concrete overlay; (2) repair of flexurally cracked bridge superstructures by epoxy injection; (3) strengthening with external post-tensioning tendons; and (4) strengthening with the addition of a thin high strength prestressed bottom soffit panel to the

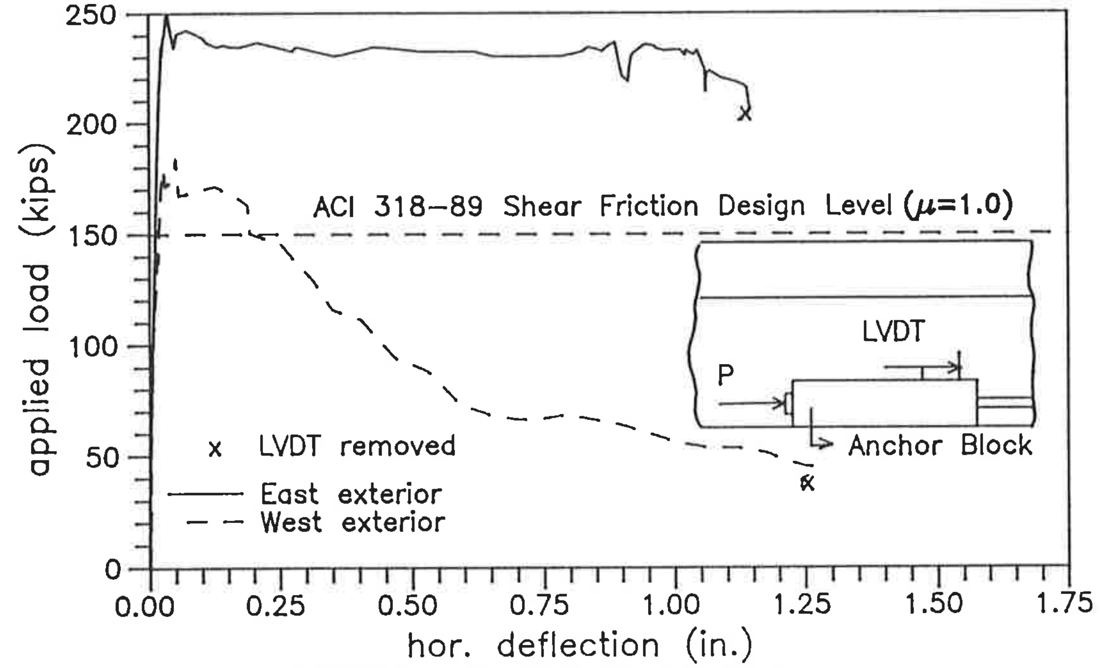


FIGURE 9 Anchor block-ultimate load test response

implementation of the other repair and strengthening measures was furnished together with experimental behavioral data.

The dowel reinforcement was found to be ineffective for full depth structural concrete overlays in conjunction with a rough and clean interface if horizontal shear stress levels did not exceed $2\sqrt{f'_c}$. Forced Vibration systems identification of the Gepford Overhead test section at various stages of the research program showed a significant change in the structural response characteristics due to epoxy injection of the flexural cracks and the addition of the bottom soffit panel, whereas external post-tensioning had virtually no impact on the structural stiffness. Both centric and eccentric working load level tests confirmed the above inference thereby proving low level forced vibration testing as an effective nondestructive systems identification tool. Overload tests to the onset of flexural yield in the main longitudinal rebars, with the external post- tensioning exhibited the excellent performance of the anchor blocks and the bridge structure with virtually no new flexural cracks. Ultimate limit state tests on the anchor blocks investigated the differences in the failure characteristics due to the different anchorage procedures. The two different epoxy bonding procedures for the bottom soffit panel performed satisfactorily and the bridge section behaved as a monolithic unit. The horizontal shear failure of the girder concrete demonstrates the need for additional edge bonding (e.g. dowels) by mechanical means into the core of the bridge girder.

ACKNOWLEDGEMENTS

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Epoxy injection of the flexural cracks and epoxy bonding of the anchor block dowels and the precast bottom soffit panel was performed and donated to the project by Slater Waterproofing Inc., Montclair, California, in cooperation with Sika Corporation. Post-tensioning hardware equipment was provided by Dywidag Systems International, U.S.A. Inc., Long Beach, California. The precast prestressed bottom soffit panel was manufactured at Tru Span, San Diego, a subsidiary of Spancrete.

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