

PRACTICAL BRIDGE RETROFIT CONCEPTS TO REDUCE DAMAGE PRODUCED BY SEISMIC MOTIONS

A. Longinow and R. R. Robinson, IIT Research Institute, Chicago, Illinois
W. Podolny, Federal Highway Administration, Washington, D.C.
K. H. Chu, Illinois Institute of Technology, Chicago, Illinois
D. S. Albert, Hazalet & Erdal, Consulting Engineers, Chicago, Illinois

This paper presents design criteria and design details on eight bridge retrofit concepts which were developed for implementation on existing highway bridges so as to minimize damage and hazard to life in the event of an earthquake. Retrofit categories were selected on the basis of observed damage experienced by highway bridges in previous earthquakes. The eight retrofit concepts are:

1. Concrete box girder hinge longitudinal restrainer.
2. Girder longitudinal displacement stopper at abutment.
3. Steel girder vertical displacement restrainer.
4. Steel girder hinge expansion joint longitudinal restrainer.
5. Pier footing strengthening.
6. Reinforced concrete bent column strengthening.
7. Girder bearing area widening.
8. Steel girder pin bearing vertical and lateral displacement restrainer.

This narrative describes the bridge retrofit process and illustrates the individual concepts in terms of intended function and structural details. The design method employed is illustrated by including the step by step design of one of the eight retrofit concepts.

The work described herein is at least in part motivated by the damage that was sustained by highway bridges during the February 9, 1971 San Fernando earthquake. This earthquake clearly pointed out a number of deficiencies in bridge design specifications. It also focused on the fact that numerous existing bridges may be expected to fail in some major way during their remaining life if subjected to strong motion seismic loads. Bridge failures are clearly undesirable since a bridge may be a vital link in a road network. When a portion of the road network has been disrupted by the collapse of a bridge, vital services to the surrounding communities are disrupted for the time required to find an alternate route or for the bridge to be repaired or re-

placed. Depending on the extent of other physical damage and casualties produced by the earthquake, the loss of vital bridges, i.e., those that provide access to hospitals for example, can magnify the effects of the disaster.

Following the February 9, 1971 San Fernando earthquake, Federal Highway Administration (FHWA) launched a study (1) whose objective was to identify and define practical techniques and criteria for retrofitting existing highway bridges so as to increase their resistance to seismic forces. This was followed by another effort which produced a design reference manual (2). The objective was to illustrate retrofit concepts that can be applied to existing bridges, which will enhance the probability of survival of the structure when it is subjected to postulated seismic motions. This narrative is based on the results of this effort.

Bridge Retrofit Decision Process

In determining whether a given bridge warrants retrofitting these three steps (as a minimum) should be considered:

1. Will the bridge suffer a critical failure (i.e., so extensive that the bridge could not remain in even emergency use) if subjected to the probable earthquake ground motions for the bridge site? If the structural analysis produces a negative answer to this question one need go no further. If the answer is affirmative the second step is:

2. Determine the level of importance of the bridge to the given locality by considering the type of highway, traffic volume, accessibility of other crossings, etc. A recommended procedure for deciding on the importance of the bridge is given in (3). If it is determined that the bridge is unimportant to the locality, it may be decided that retrofitting is not feasible even though the answer to step 1 was affirmative. If however, it is decided that the bridge is important to the area, the third step is:

3. Determine the type or types of retrofit measure(s) to employ. This decision is based on the following considerations: (a) probable mode of failure; (b) how will the chosen retrofit measure(s) influence the performance of other parts of the bridge under seismic as well as normal service loading;

(c) a comparison of expected interference with traffic flow on and under the bridge for different retrofit measures; (d) a comparison of expected costs of fabrication and installation of different retrofit measures. This comparison is based on, but not necessarily limited to: accessibility of the area to be retrofitted (e.g., it would be more costly to enlarge a footing if the existing footing were under water as opposed to normal backfill on dry land); the type and quantity of construction materials (e.g., type of steel, concrete); type and quantity needed for installation; length of time needed for installation; the number and qualifications of men needed to do the work; number of bridges; if a large number of bridges in a given area are identified for retrofitting, there are practical considerations in contracting and inspection. A greater degree of efficiency is achieved if a number of bridges in one area can be included under a single contract. It is more efficient to prepare plans and let contracts for several large jobs than a large number of single bridge contracts. A large contract can also be inspected more efficiently by a single inspector. Bridges in a single contract should be reasonably close together (4).

Two of the key items in the bridge retrofit decision process that need emphasizing are (a) the determination of probable ground motions and (b) selection of appropriate structural analysis method and a sufficiently accurate structural model of the bridge.

The probable "earthquake" at a given bridge site can be determined on a statistical basis by the use of historic earthquake data previously compiled for the given geographic area. The earthquake at the site can be expressed in terms of maximum ground acceleration, an "effective" acceleration, a response spectrum or a motion history. A number of sources for such information are currently available and several levels of "accuracy" are possible (5,6). An approximate procedure for selecting a probable ground acceleration at a bridge site is included in (7).

The unique aspect of earthquakes is the fact that motions generally persist for a "long time" relative to the natural periods of bridge components. A component can therefore experience many load cycles of varying magnitude during the passage of the principal portion of an earthquake. Many cycles of "low" magnitude can be more effective in producing damage than a single cycle of "high" magnitude and short duration. Analysis methods capable of considering such random motions and predicting the corresponding behavior of a structure in whatever phase (elastic, nonelastic) it chooses to respond, are still very much in the research phase. Presently the problem is in part hampered by lack of reliable data on the cyclic behavior of reinforced concrete past the linear range. However, the major problem is still the quantity of computer time required to solve a reasonably sized model of the structure. To overcome some of these difficulties, a simplified analysis method was developed and verified as part of the effort (7) described. The method is a practical engineering approach which considers only the dominant modes of response. It is an iterative procedure in the sense that several models of varying complexity may be required to zero-in on the answer. This method has the advantage of providing reliable answers in a relatively short period of time without reliance on any specific computer program. However, it places a great deal of reliance on the basic engineering intuition and practical experience of the user.

Bridge Retrofit Measures

Bridge retrofit measures considered were selected

on the basis of the type of failure modes and damage experienced by highway bridges in previous earthquakes. Observed failure modes can be grouped into two categories, i.e., substructure (pier or abutment) failures and hence loss of superstructure capacity, and superstructure collapse due to excessive relative motion of the support bearings. Both of these types of failure occurred during the San Fernando earthquake (8).

Structural failures and damage to bridges are also caused by inadequate foundation strength or load-bearing degradation during the course of seismic loading. Soil liquefaction is an example of this failure mode.

Severe motion of the soil supporting the foundation can cause large horizontal and vertical deformations of the support point. These transient motions create relative movement between the support points which can lead to the failures described.

Based on highway bridge damage observed in previous earthquakes, eight retrofit measures were identified:

1. Concrete box girder hinge longitudinal restrainer.
2. Girder longitudinal displacement stopper at abutment.
3. Steel girder vertical displacement restrainer.
4. Steel girder hinge expansion joint longitudinal restrainer.
5. Pier footing strengthening.
6. Reinforced concrete bent column strengthening.
7. Girder bearing area widening.
8. Steel girder pin bearing vertical and lateral displacement restrainer.

Each of these retrofits is addressed to increasing either the rigid body stability of the superstructure or the strength of the substructure. Thus the retrofit measures, if appropriately designed, will enhance bridge resistance to the dominant failure modes that have been observed from severe seismic loading.

Since the emphasis of the study described (2) was on retrofit concepts rather than on bridge analysis, seismic forces that the various retrofit measures were designed to resist were not determined by analyzing each given bridge when subjected to a probable earthquake. Rather, these forces were determined as follows.

Horizontal motion restrainers were designed for a force of 0.25 times the contributing dead load. For a simply supported span fixed at one end and free to translate at the other, the contributing dead load is the total superstructure weight at the fixed end for longitudinal seismic loading and one-half of the superstructure weight at each end for transverse loading. Other examples of contributing dead load are given in (9).

Vertical motion restrainers connected between the superstructure and the substructure across the bearings were designed to withstand a separation force equal to 0.10 times the bearing dead load reaction (10).

In actual bridge retrofit efforts such forces would be determined from the bridge response analysis if a detailed analysis method is used, or from criteria given in (7) if the IITRI simplified analysis method is used.

To keep the illustrations simple, only one component of earthquake motion was considered with each retrofit concept. Obviously in the actual case of bridge retrofit analysis, all three components (lateral, longitudinal and vertical) would need to be

considered.

In general, the retrofit design concepts are based on earthquake loading considerations consistent with AASHTO Load Factor Design - Group VII loading (11). Therefore the earthquake load is multiplied by 1.3 and yield stresses are generally permitted for the materials. If a working stress design is used for a particular retrofit, the basic unit allowable stress can be increased by 33 percent as indicated in the 1975 Interim (11).

The new materials that are employed in the retrofit measures are listed in Table 1. It is noted that conventional materials and moderately high strength concrete have been employed for the retrofit measures.

It is not considered practical to design bridges that will economically serve normal transportation needs and also will not be damaged to some extent when subjected to severe seismic motions. Thus the basis of retrofitting a given bridge should not be intended to allow no damage whatsoever, but should be such as to limit the damage to the extent that the bridge does not collapse and that traffic may be maintained at least in the immediate period after an earthquake, with minimum emergency repairs to the bridge.

Seven of the eight retrofit measures are described in general term in the following paragraphs. Retrofit 8, Steel Girder Pin Bearing Vertical and Lateral Displacement Restrainer is described in the last section along with its step by step design.

Concrete Box Girder Hinge Longitudinal Restrainer

A four-span grade separation bridge (Figure 1) of multiple concrete box girder construction is cited to illustrate this retrofit concept. The original design employs a single transverse expansion joint hinge such that no longitudinal force can be transmitted across the joint.

The purpose of the hinge longitudinal restrainer is to restrict relative longitudinal motion across the hinge during an earthquake. The device consists

of 12 restrainer rods (Figures 1 and 2) used to tie the bridge together. Excess openings (see Figure 2) are provided in the lower part of the box girder for implementing the device. With this device, the portion of the superstructure supported at the hinge will not fall off the hinge seat due to excessive relative motion. In the design, a certain amount of free thermal expansion travel is permitted to take place before the motion restrainers exert resistance.

Girder Longitudinal Displacement Stopper at Abutment

The objective of the longitudinal girder stopper is to restrict the relative longitudinal motion between the superstructure and the abutment at the expansion bearings during an earthquake. Using this retrofit measure will limit the bearing motion and eliminate bearing instability from toppling or falling off the abutment due to excessive motion. A certain amount of free travel from thermal expansion effects and allowable earthquake motion is permitted to take place before the stopper exerts resistance to the motion. To illustrate the stopper retrofit concept, a 27.43 m (90 ft) simple span I-beam bridge, adapted from (1) was chosen with the stopper applied at the expansion bearing (Figure 3). Stopper details which resulted from the design effort are shown in Figure 4.

Steel Girder Vertical Displacement Restrainer

The objective of the vertical displacement restrainer is to restrict the relative vertical motion between the superstructure and the pier or abutment seat during an earthquake with a strong vertical component. The use of this retrofit will limit the vertical separation that can occur at the support bearings and eliminate bearing instability and hence loss of superstructure support.

To illustrate the design of this concept, a bridge was considered with longitudinal girders supported by bearings which do not provide a positive

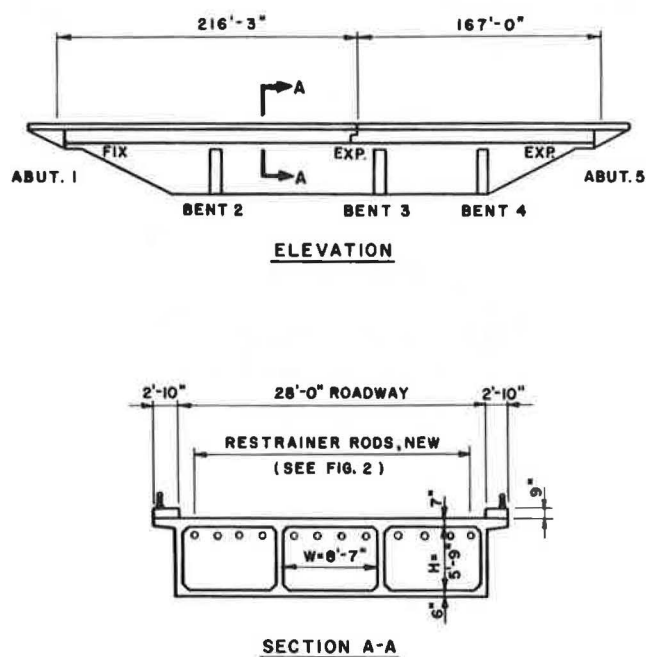
Table 1. New Materials for retrofit design.

Material	AASHTO (ASTM)	F_y ksi	F_u ksi	Comments
	Spc.			
1. Structural steel	M183 (A36)	36	58-80	
2. Low-carbon steel	(A307)		60-100	Machine bolts, grade B
3. High strength bolts	M164 (A325)	92	120	1/2 inch to 1 inch diameter
		81	105	1-1/8 inch to 1-1/2 inch diameter
4. Reinforcing steel	M31 (A615*)	60	90	Billet steel, grade 60
5. Sponge rubber	M153 (D1752)			Type I density ≥ 30 pcf
6. Fabric pads	AASHTO 1973 Std. Specification, Art. 2.10.3 (L)			
7. Carbon steel bars	M227 (A306)	30	60-72	Anchor bars, grade 60
8. Concrete	AASHTO 1973 Std. Specification, Art. 2.4, $f'_c = 5$ ksi			
9. Grout	PCI 1971, Part B, Guide Specifications for Posttensioning Materials, Art. 4.3 (page B-27), $f'_c = 6$ ksi			

Note: 1 in. = 2.54 cm. 1 ksi = 6.89 MPa. 1 pcf = 16.02 Kg/m³.

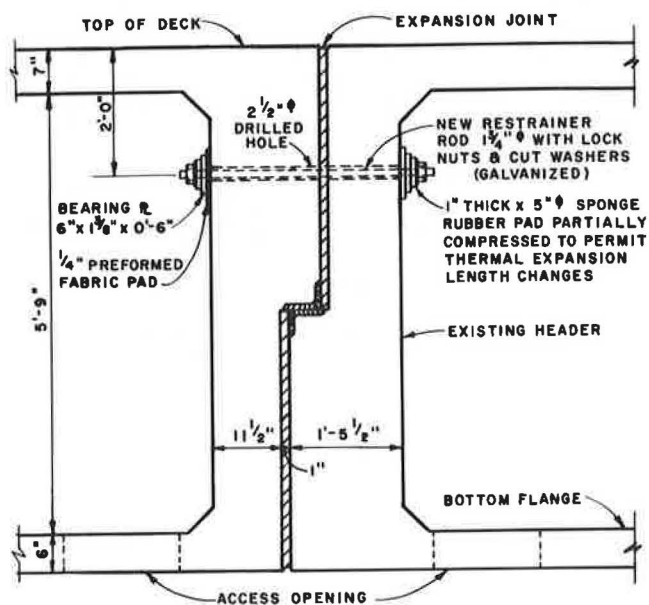
restraint to uplift forces. The piers are reinforced concrete frames with sufficient open space under the cap beam to accommodate the restrainer details. Figure 5 illustrates the resulting concept.

Figure 1. Box girder hinge longitudinal restrainer retrofit concept.



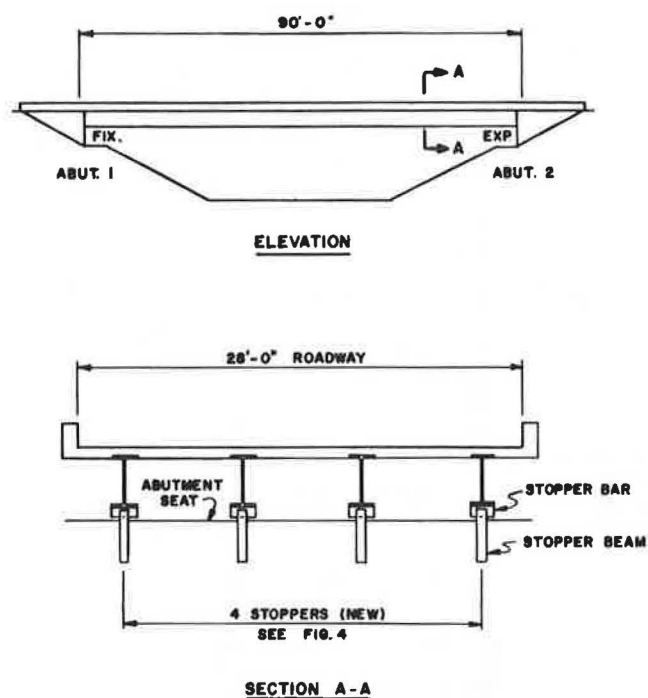
Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Figure 2. Restrainer detail.



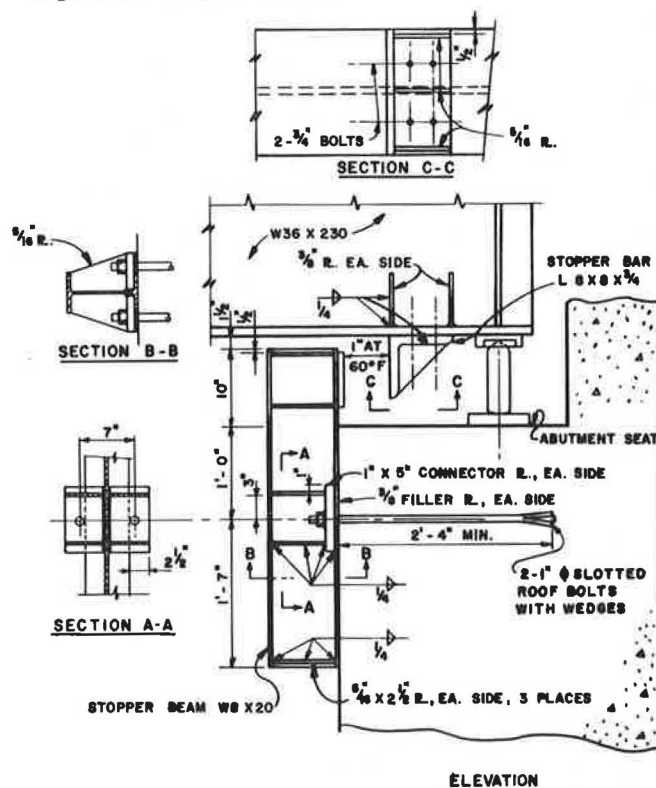
Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Figure 3. Girder longitudinal displacement stopper at abutment retrofit concept.



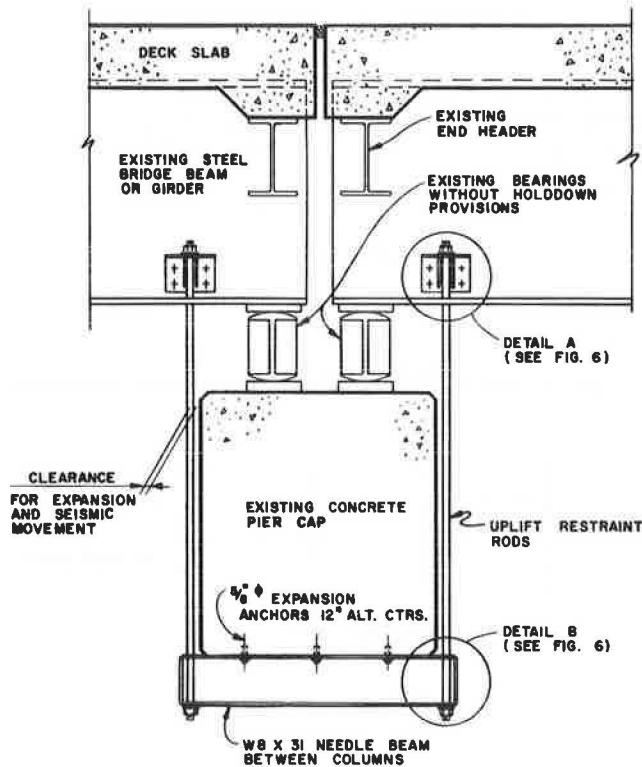
Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Figure 4. Stopper detail.



Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Figure 5. Steel girder vertical restrainer retrofit concept.



Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Steel Girder Hinge Expansion Joint Longitudinal Restrainer

As with the box girder discussed previously, the purpose of the expansion joint longitudinal restrainer is to restrict the relative longitudinal motion across the expansion joint during an earthquake. Using this retrofit concept, excessive separation displacements across the hinge are reduced and hinge failures created by this effect are thus eliminated. A certain amount of free thermal expansion is permitted at the hinge before resistance is encountered.

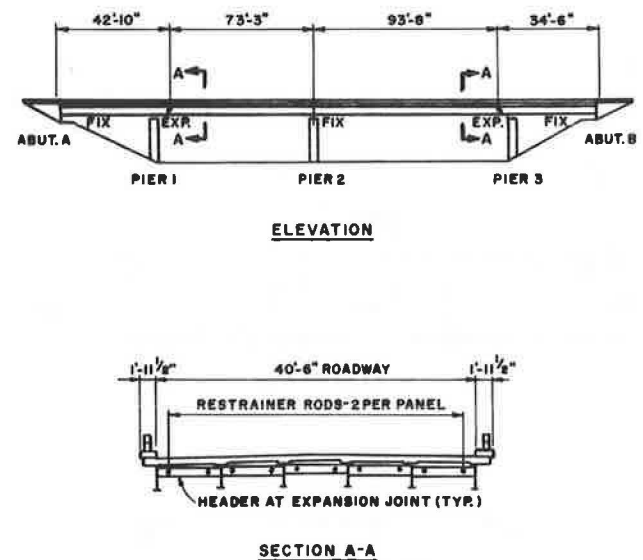
A typical four-span grade separation of cantilever and suspended span construction illustrates the use of this retrofit concept. The original design of the expansion joint is such that no longitudinal force can be transmitted across the joint.

The retrofit concept makes use of existing headers (see Figure 6) located at either side of the expansion joints. Restrainer rods located close to the bridge girders are used to tie the bridge together. Since in this particular case the headers are not by themselves sufficiently strong, steel channels (see Figure 7) provide a diagonal brace to transfer the design load from the restrainer rods to the girder web.

Pier Footing Strengthening

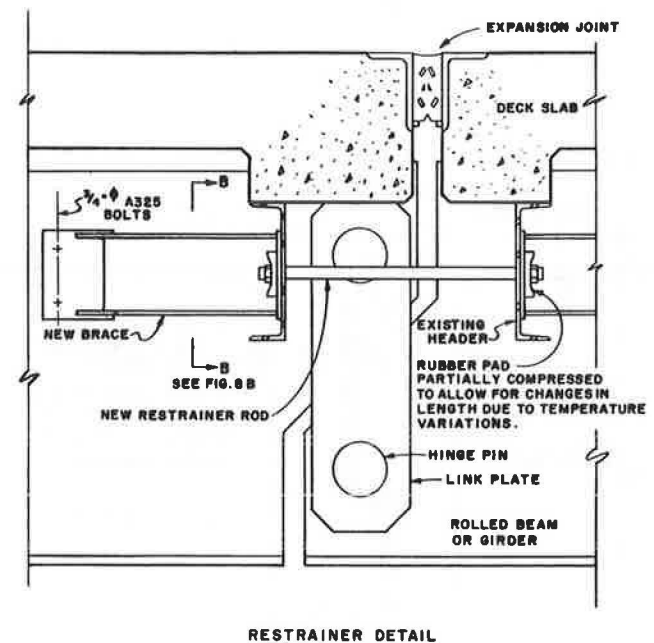
The objective of this retrofit is to increase the longitudinal load resistance of a pier footing so that substructure failure will not occur during a high intensity seismic disturbance. The technique employed involves the addition of new piles around the perimeter of the footing and widening and deepening the footing so as to tie-in the new piles.

Figure 6. Steel girder hinge expansion joint longitudinal restrainer retrofit concept.



Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Figure 7. Hinge expansion joint longitudinal restrainer details.

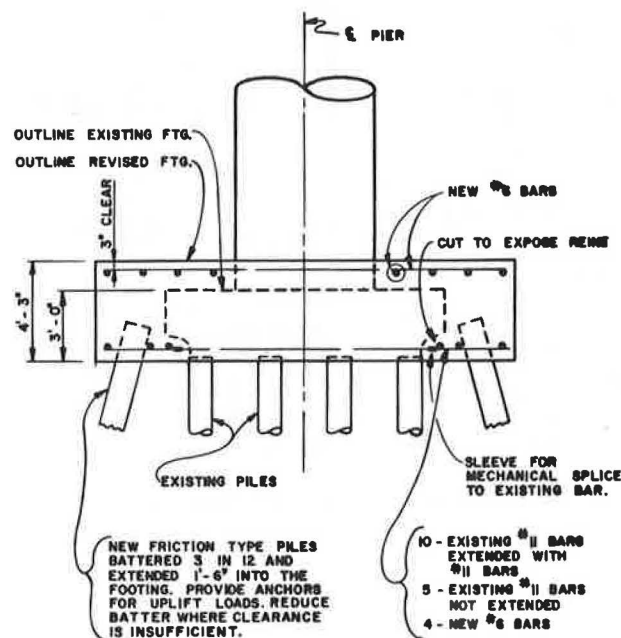


Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

The added strength is such that the footing will be capable of resisting the bending moment and shear forces induced by the earthquake loading.

To illustrate the retrofit concept, a 3.66 m x 4.57 m x 0.91 m (12 ft x 15 ft x 3 ft) footing with 20 reinforced concrete piles is modified to withstand the longitudinal seismic shear and moment forces that significantly exceed the existing pile capacity. The footing supports a single 1.83 m (6-ft) diam reinforced concrete column. Modifications determined as the result of the analysis performed are shown in Figures 8 and 9.

Figure 8. Pier footing strengthening retrofit - elevation.



Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Reinforced Concrete Bent Column Strengthening

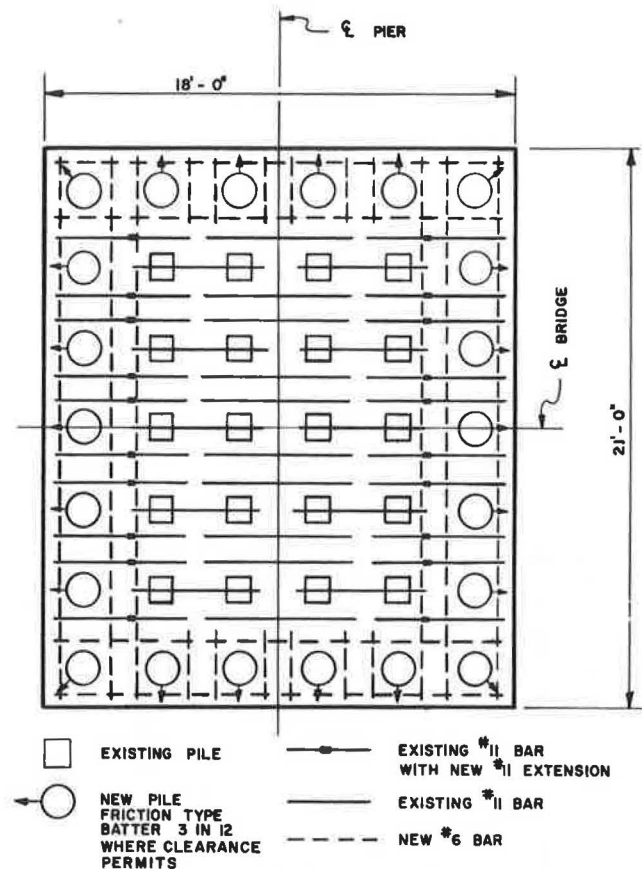
The objective of this retrofit is to increase the flexural capacity of a bent column so that bent failure will not occur during a strong motion earthquake. The method used provided additional longitudinal reinforcement to the exterior of the column which is connected to the bent cap and footing by grouting the new bars in drilled holes. Lateral dowels are also introduced to enhance the monolithic behavior of the new addition to the parent column.

A representative two-span reinforced concrete box girder bridge is used to demonstrate the retrofit measure. The original design employed a single column pier with the cap monolithic with the superstructure and a pile spread footing. The structural characteristics of the retrofit are shown in Figure 10.

Girder Bearing Area Widening

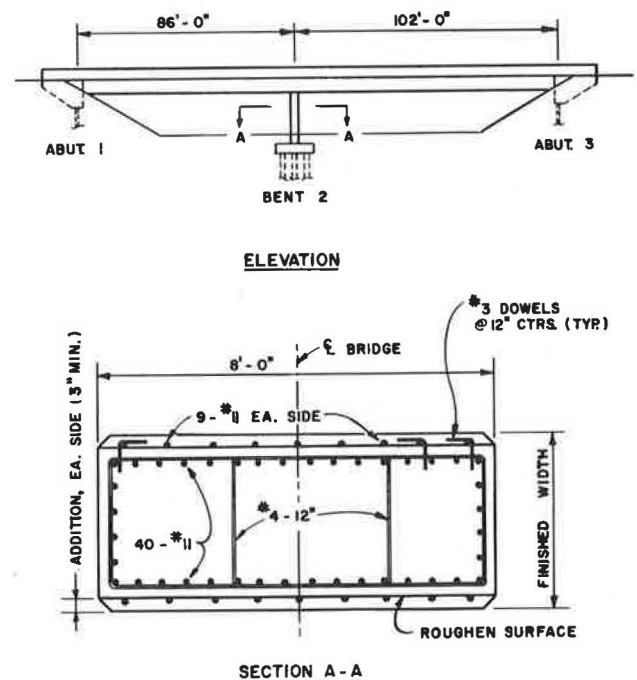
The purpose of widening a bearing area is to provide additional width at the girder support points in the event that strong motion seismic loading pulls the superstructure off the bearings. Using this retrofit the displaced girders are expected to impact on the widened bearing area thus averting

Figure 9. Pier footing strengthening retrofit - pile and bottom reinforcement details.



Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Figure 10. Reinforced concrete bent column strengthening retrofit concept.

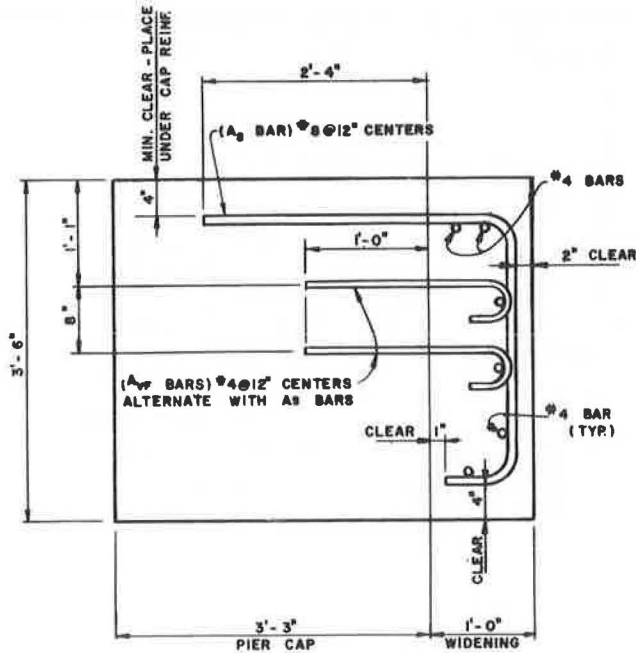


Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

collapse of the superstructure.

To illustrate this retrofit a 0.99 m (3 ft-3 inch) wide reinforced concrete pier cap is postulated and a 0.30 m (12 inch) width addition, on one side of the cap, is designed to withstand the loading associated with the impact of the girders on the widened area (Figure 11). The superstructure girders are spaced at 1.83 m (6 ft) centers and the load on the widened area is assumed to be centered at 0.15 m (6 inches) from the existing pier cap face. The addition to the pier cap is designed using the shear friction design method (12).

Figure 11. Girder bearing area widening retrofit details.



Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Design of Bridge Retrofit Concepts

This section contains the step-by-step design of the Steel Girder Pin Bearing Vertical and Lateral Displacement Restrainer bridge retrofit concept. The purpose is to demonstrate the procedure employed in design.

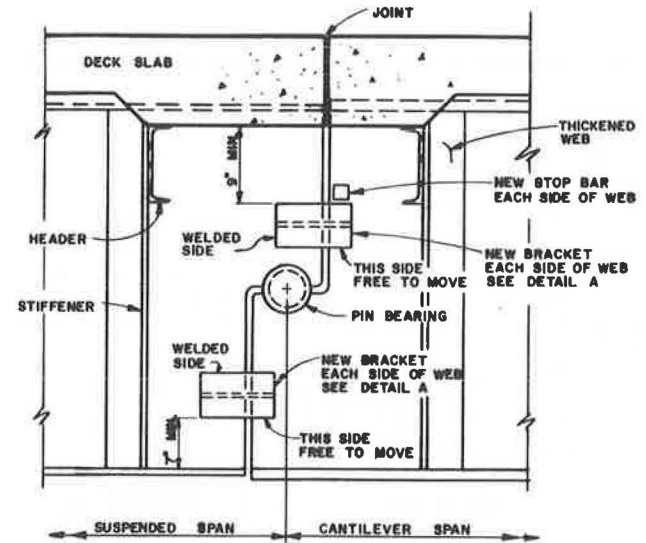
Steel Girder Pin Bearing Vertical and Lateral Displacement Restrainer

The objective of the pin bearing displacement restrainer is to inhibit essentially all of the relative vertical and lateral motion across the bearing that could take place during an earthquake. With this retrofit, potential vertical and lateral motions during an earthquake are arrested by the addition of a bracket and stopper bar arrangement welded to the webs of the girders. The joint is also effectively restrained against relative longitudinal motion during an earthquake by the new vertical restraint which prevents the suspended span from rolling over the pin.

The retrofit method is applicable to any bridge with longitudinal girders supported by hinged bear-

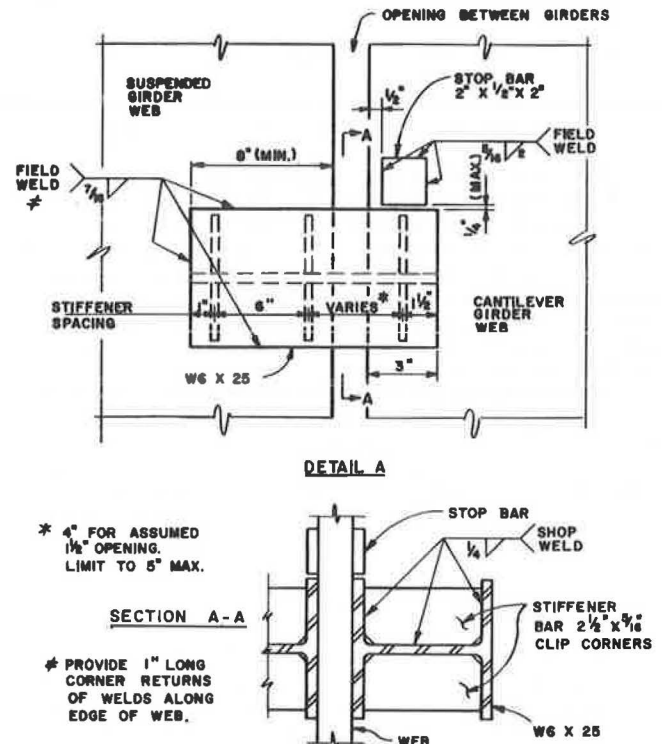
ings which do not provide positive restraint against uplift or lateral motion. A 200 kip (889.64 kN) dead load vertical reaction at the pin bearing was assumed to illustrate the concept. The brackets are fabricated from stiffened W6 x 25 beams that are positioned horizontally across the bearing joint opening with one of the bracket flanges welded to the suspended girder web only. A 2 inch (50.8 mm) square, 0.5 inch (12.7 mm) thick stop bar is welded to the girder web on the other side of the bearing joint above the bracket (Figures 12 and 13).

Figure 12. Steel girder pin-type bearing vertical and lateral displacement restrainer.



Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Figure 13. Steel girder restrainer details.



Note: 1 ft = 0.305 m. 1 in. = 2.54 cm.

Earthquake Loading. Suspended span girder reaction = 200 kips/girder (889.64 kN/girder)

Lateral earthquake load

$$EQ = 0.25 \times \text{contributing dead load} = 50 \text{ kips} \\ (222.41 \text{ kN})$$

Vertical earthquake load

$$0.1 \times 200 = 20 \text{ kips} (88.96 \text{ kN})$$

AASHTO 1975
Art. 1.2.20(D)

Lateral Restrainer Design

Pin bearings may have a rim projection which provides resistance to lateral loads; however, no lateral force will be assumed as transferred through the bearing. Cantilever brackets (W6 x 25) welded to each side of the suspended span web plate above and below the bearing pin will provide adequate lateral load transfer. Each pair of brackets will be assumed to transfer one-half of the vertical load. The top pair of brackets will also function as a vertical up-lift restrainer.

Use load factor design

$$\text{Design load per unit} = 1.3 \times 50/2 = 32.5 \text{ kips} \\ (144.57 \text{ kN})$$

AASHTO 1975
Art. 1.2.22

This lateral force must be resisted in either direction.

Size of Cantilever Brackets. Opening between girders assumed to be 1.5 in. (38.10 mm).

$$\text{Cantilever arm} = 3.0 + \bar{X} = 7.67 \text{ in.} (194.82 \text{ mm}), \\ \text{where } \bar{X} = 4.67 \text{ in.} (118.62 \text{ mm}) \\ (\text{see bracket connection calculation})$$

$$\text{Moment} = 32.5 \times 7.67 = 249.3 \text{ inch-kips} \\ (28.2 \text{ kN}\cdot\text{m})$$

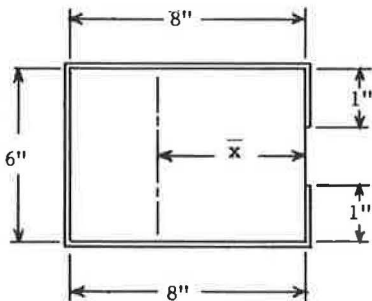
Use W6 x 25 brackets

$$S_{xx} = 16.7 \text{ inches}^3 (273664.6 \text{ mm}^3) \\ f = 249.3/16.7 = 14.9 \text{ ksi} < 36 \text{ ksi} \\ (102.7 \text{ MPa} < 248.2 \text{ MPa}) \text{ o.k.}$$

Attachment of Lateral Restrainers to Girders.

Field weld cantilever brackets to suspended span web with 7/16 inch (11.11 mm) fillet weld. Provide proper corner returns (1 inch, 25.4 mm) so that welds are capable of withstanding loads normal to the web.

Properties of Bracket Weld



	A	x	Ax	I_{oo}	Ax^2
8 x 2	16	4	64	85.3	256
6 x 1	6	8	48	-	384
1 x 2	2	0	0	-	0
	24		112	85.3	640.0
					85.3
	$\bar{x} = 112/24 = 4.67 \text{ in.}$			$-Ax^2 = -522.7$	
	(118.62 mm)				
	$S = I/\bar{x} = 43.4 (\text{in.})^3$			$I = 202.6 (\text{in.})^4$	
	(711200.1 mm ³)			(84328400.6 mm ⁴)	
				AASHTO 1973	
				Art. 1.7.28(B)	

Allowable Weld Stress

$$0.45 \times 58 = 26.1 \text{ ksi} (178. \text{ MPa})$$

$$7/16 \text{ in. weld strength} = 26.1 (7/16) 0.707 \\ = 8.1 \text{ kips/inch} \\ (1418.5 \text{ kN/m})$$

AASHTO 1973
Art. 1.7.135(A)

Bracket Weld Stress from Lateral Load

$$\frac{32.5}{24} + \frac{249.3}{43.4} = 1.35 + 5.74 = 7.1 \text{ kips/in.} < 8.1 \\ \text{kips/in.} (1243.4 \text{ kN/m}) < 1418.5 \text{ kN/m} \quad \text{o.k.}$$

Girder Web Stiffness. The webs are assumed to be at least 2 inches (50.8 mm) thick and this thickness is adequate to withstand the lateral loads from the brackets without additional stiffeners.

Vertical Restrainer Design

Use a pair of restrainer units at each girder. Each unit to consist of two stop bars and cantilever brackets field welded to the girder webs.

Use load factor design

$$\text{Design load per unit} = 20 \times 1.3/2 = 13 \text{ kips} \\ (57.83 \text{ kN})$$

AASHTO 1975
Art. 1.2.22

Size of Stop Bars

$$A = 13/36 = 0.36 \text{ sq in.} (232.26 \text{ mm}^2) \\ \text{Use } 2 \times 1/2 \text{ bar } (A = 1 \text{ sq in., } 645.16 \text{ mm}^2)$$

Size of Cantilever Brackets

$$\text{Cantilever arm} = 7.67 \text{ in.} (194.82 \text{ mm}) \\ M = 13 \times 7.67 = 99.7 \text{ inch-kips} (11.26 \text{ kN}\cdot\text{m})$$

Use the pair of upper W6 x 25 brackets that were provided for the lateral restrainer. These cantilever brackets will be employed to resist both the vertical and lateral restraint loads. Assume vertical load is resisted only by the connected flange.

$$S = 0.456 (6)^2/6 = 2.74 \text{ inches}^3 (44900.65 \text{ mm}^3)$$

$$f_b = 99.7/2.74 = 36.4 \text{ ksi (250.97 MPa) (allow. 36 ksi, 248.21 MPa o.k.)}$$

Connection of Uplift Restrainer to Girder

Stop Bar. Field weld to cantilever span web with 5/16 inch (7.94 mm) fillet weld.

Allowable weld stress = $0.45 \times 58 = 26.1 \text{ ksi}$ (179.95 MPa)

5/16 inch weld strength = $26.1 (5/16) 0.707$
 $= 5.77 \text{ kips/inch}$
 (1010.48 kN/m)
 AASHTO 1973
 Art. 1.7.135(A)

Required length = $26.1/5.77 = 4.5 \text{ inches}$ (114.3 mm)

Weld around three sides = 6 inches (152.4 mm)

No weld along bottom to permit full bearing of bracket flange.

Cantilever bracket. Check 7/16 inch (11.11 mm) weld stress (see lateral restrainer calculations for weld properties) 7/16"

Weld strength = 8.1 kips/inch (1418.54 kN/m)

Loads $P = 13 \text{ kips (57.83 kN)}$; $M = 99.7 \text{ inch-kips}$ (11.26 kN·m);

Min. Sec. Modulus = 43.4 inch^3 (711200.1 mm³)

Bracket weld stress from vertical load

$$\frac{13}{24} \pm \frac{99.7}{43.4} = 0.54 \pm 2.30 = 2.8 \text{ kips/inch}$$

(490.36 kN/m) < 8.1 kips/inch (1418.54 kN/m) o.k.

Conclusion

Bridges that are located on high risk seismic zones that were designed for earthquake loading according to the pre-1975 AASHTO Design Criteria may suffer substantial structural damage, and in some cases, collapse can be anticipated. This conclusion is based on analyses performed for several bridges and reported in (1). Each bridge was subjected to a postulated seismic load of the highest severity that will occur during the life of the structure at the bridge site.

This set of bridge retrofit concepts and the simplified analysis method given in (7) are intended to provide the practicing bridge engineer with basic guidelines, information and examples that may be used in planning and executing a bridge retrofit program.

It is believed that most bridges can be modified, if required, to dramatically increase their seismic strength. A number of the retrofit concepts can be relatively economically implemented especially when compared with the cost of structural failure.

The philosophy to be employed for determining the type and need for a retrofit measure is one of a balanced risk concept. Retrofitting should not be based on a need for eliminating all damage, but to limit the damage such that collapse does not occur and traffic can be maintained or restored after minimal repairs. A good deal of yielding and damage can be absorbed by the piers and other ductile components before collapse of the structure.

Acknowledgements

This work was supported by the Federal Highway Administration under Contract DOT-FH-11-8847. The period of performance was June 30, 1975 to May 31, 1977.

References

1. Robinson, R. R., et al, "Structural Analysis and Retrofitting of Existing Highway Bridges Subjected to Strong Motion Seismic Loading", Report FHWA-RD-75-94, Federal Highway Administration, Office of Research and Development, Washington, D.C. 20590, May 1975
2. Robinson, R. R., Longinow, A., and Albert, D. S. "Seismic Retrofit Measures for Highway Bridges - Design Reference Manual for Retrofitting Bridges to Withstand Earthquakes", for Federal Highway Administration Contract DOT-FH-11-8847, Final Report - Volume 2, June 1977 (in draft form)
3. Longinow, A., Bergman, E. and Cooper, J. D., "Bridge Retrofitting - Selection of Critical Bridges in a Road Network", Proceedings Technical Council on Lifeline Earthquake Engineering, Specialty Conference, University of California, Los Angeles, California, August 30-31, 1977
4. Degenkolb, O. H., "Retrofitting Highway Structures to Increase Seismic Resistance", Proceedings Technical Council on Lifeline Earthquake Engineering, Specialty Conference, University of California, Los Angeles, California, August 30-31, 1977
5. Newmark, N. M., Blume, J. A., and Kapur, K. K., "Seismic Design Spectra for Nuclear Power Division, American Society of Civil Engineers, November 1973
6. Algermissen, S. T. and Perkins, D. M., "A Probabilistic Estimate of Maximum Acceleration in Rock in the Contiguous United States", U.S. Geological Survey, Open File Report 76-416, 1976
7. Longinow, A., Robinson, R. R. and Chu, K. H., "Seismic Retrofit Measures for Highway Bridges - Training Course Material", for Federal Highway Administration Contract DOT-FH-11-8847, Final Report, Volume 1, June 1977 (in draft form)
8. "San Fernando, California, Earthquake of February 9, 1971", Volume II, Utilities, Transportation, and Sociological Aspects, U.S. Department of Commerce, 1973
9. 1975 Interim Specifications, Interim 2, Section 2 - Loads (Earthquake Stresses), Standard Specifications for Highway Bridges, AASHTO
10. Japan Road Association, "Specifications for Earthquake Resistance Design of Highway Bridges" quoted in, Iwasaki, T., Penzien, J., Clough, R., "An Investigation of the Effectiveness of Existing Bridge Design Methodology in Providing Adequate Structures Resistance to Seismic Disturbance. Phase 1: Literature Survey", FHWA-RD-73-13, November 1973
11. 1975 Interim Specifications, Interim 3, Loading Combinations, Standard Specifications for Highway Bridges, AASHTO.
12. 1974 Interim Revision, Section 5 - Reinforced Concrete Design, Standard Specifications for Highway Bridges, AASHTO