INCREASING THE SEISMIC RESISTANCE OF EXISTING HIGHWAY BRIDGES

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Prior to the 1971 San Fernando Earthquake, bridges in California experienced only minor seismic related damage. The San Fernando event demonstrated that structures designed by AASHTO Specifications in use at that time are vulnerable to seismic shaking. Failure of these bridges during an earthquake could be hazardous to highway users and block vital transportation lifelines. The State of California initiated a bridge retrofitting program in 1971 in order to increase the seismic resistance of bridges built before that time. The most prevalent deficiency of pre-1971 bridges is a lack of longitudinal restraint of girders at hinges and end supports. California has developed devices which will have been used to retrofit more than 649 bridges at a cost of $22 million by 1980. An evaluation of all state owned bridges is currently being made in order to complete the program. Many of the bridge columns which were designed according to specifications prior to 1971 were proven to be seismically deficient because they had too few ties to adequately confine the concrete. This paper and presentation will cover a brief background, philosophy, magnitude of the problem, design criteria, details and costs.

Introduction

Prior to 1971, very little earthquake damage was experienced by bridges in the mainland 48 states. The little damage that did occur was generally limited to minor cracking and spalling of concrete, damaged bearings and grout pads, and slight displacements of spans. The bridges involved were generally quite low with rather short spans.

The 1971 San Fernando earthquake provided a test of modern bridges which had spans as long as 191 feet and heights of 150 feet. Much of the damage to these bridges was caused by vibration induced by ground motion.

The San Fernando earthquake occurred at 6:00 a.m., before peak morning traffic, and before the completion of two major highway interchanges which were under construction in the area. As a consequence, there was relatively little inconvenience to the travelling public and only two fatal injuries. If the same quake had occurred a few months later and a few hours later in the day, the inconvenience to the public and number of fatalities could have been dramatically different.

Deficiencies In Existing Bridges

The 1971 earthquake pointed out a number of deficiencies in bridge design specifications and detailing practices. Although the level of seismic design forces and methods of applying those forces proved to be inadequate, the most serious deficiencies were attributed to details. Segments of superstructures were not properly tied together. Some concrete columns were seismically inadequate because they had an insufficient amount of spiral and tie reinforcement, the ties and spirals were inadequately detailed, and main column reinforcement frequently had insufficient splice length and end anchorage. The column deficiencies are especially critical in bridges with single column bents.

Retrofitting Philosophy

It is not practical or economical to design new bridges or retrofit existing bridges that will serve normal transportation needs but not be damaged to some extent if subjected to severe seismic shaking. The aim is to make structures seismically resistant to the extent that they may sustain damage but not collapse completely. It is also desirable that they be capable of carrying at least a limited amount of emergency traffic even though they may be damaged. Although retrofitting existing structures will increase their seismic resistance considerably, a designer is limited by the capabilities and features of the existing facilities and economics. Portions of some existing structures have to be strengthened to accommodate the anchorage forces which restrainers require. In some cases restrainers which would develop the forces required to hold the segments of a bridge together would pull the ends out of the spans or pull over the columns. When hinges are not restrained, segments of a bridge can act independently and forces in the columns can be significantly greater than if hinge movements are limited. Thus, retrofitting hinges with restrainers can significantly reduce the probability of column failures.

Prioritizing Retrofitting Work

It was realized immediately after the 1971 earthquake that existing bridges should be retrofitted in order to increase their seismic resistance. A prioritizing system was devised which assigned weighted values to:

1. Type of bearings
2. Width of hinge or bearing seat
3. Restraint of supports
4. Height of structure
5. Type of supports
6. Flexibility of supports
7. Curvature in alignment
8. Probable earthquake intensity
9. Hazard to public on and under structure
10. Disruption to traffic and utilities
11. Danger to buildings or facilities under the structure.

This system worked well for identifying candidate structures for immediate retrofitting. However, the prioritizing numbers obtained did not always reflect the true relative importance of some structures. The input is largely a matter of judgment, but under certain circumstances a single factor might be important enough to justify a high priority regardless of all other factors. A less important structure could rate lower in a number of less important categories but get a higher overall rating. The results from any prioritizing system should be subject to adjustment by good judgment.

There are also practical considerations which can, to some extent, override the strict adherence to a prioritizing system. If a large number of bridges spread over a very large area are identified for retrofitting, there are considerations in contracting and inspection which should not be overlooked. Although there are not any definitive rules which can be followed, there are general guidelines which should be considered. A greater degree of efficiency can be achieved if a number of bridges in one area can be included in a single contract. It is more efficient to prepare plans and let contracts for a few large jobs than a great number of single bridge contracts. A contractor's mobilization costs can be spread out and personnel can be trained and used more efficiently on a contract with a number of bridges. A large contract can be inspected efficiently, but a single inspector on too small a job will have time to waste unless he can be given other work to do. For efficiency, it is obvious that bridges in a contract should be located reasonably close together. It is generally true that groups of bridges in different contracts should not ordinarily overlap. If individual structures are prioritized by an inflexible system, it is highly unlikely that structures with nearly equal priorities will be geographically located to form logical contracts.

Hinge And Bearing Restrainers

Restrainers should be capable of developing a minimum force equal to 25% of the weight of the lighter segment of superstructure connected, based on working strength design. This rule of thumb is satisfactory for relatively short structures where the influence of the abutment backfill on the superstructure is uncertain. However, dynamic analysis should be made for larger and more complex structures and provisions made for larger forces, if required. All of California's seismic dynamic analyses are based on load factor methods and a ductility factor of one is used for restrainers.

California has used 3/4-inch pre-formed 6x19 galvanized cables (ASTM Designation A-603) with a minimum breaking strength of 23 tons (205 kN) as the basic unit for its restraining devices. Swaged end fittings are used which are required to develop the minimum breaking strength of the cable. This type of cable and end anchorages have been used in highway barrier systems for many years. They are being tested on a regular basis and have an excellent performance record. 1¼-inch diameter galvanized ASTM A-722 (with supplementary requirements) steel bars which have a specified minimum elongation of 7% measured in 10-bar diameters are also being used.

The ideal restrainer should absorb and dissipate energy. Although a number of such devices have been considered, they have not been regarded as being economically practical for routine retrofitting work. The steel cables and rods can store energy, but transfer it back into the structure as they pull the segments of superstructure back together. Much of the energy is probably dissipated by the pounding of the superstructure elements when they come together. The damage caused by this action is repairable and should not cause the bridge to collapse.

When the hinge and bearing retrofitting program was started, most of the designs were done by working strength methods. A working load of 50% of the ultimate strength for galvanized cables plus an overstress of 33% permitted for seismic conditions gives a total allowable load of 30.6 kips (136 kN) per cable. For load factor design methods, a yield strength of 85% of ultimate load, or 39.1 kips (174 kN) per 3/4-inch cable is assumed. The design yield stress for 1¼-inch high strength bars is 120 k.s.i. (827 k Pa) or 150 kips (667 kN) per bar. These bars are particularly useful in cases where it is impractical or undesirable to use the number of 3/4-inch cables required to obtain the necessary resisting force. Many older bridges which are being retrofitted have shear keys which are inadequate for keeping the two sides of the hinge aligned longitudinally if the structure is subjected to seismic shaking. Since a transverse shearing action at the hinge could cause the rods to fail and become ineffective in tension, supplemental solid mild steel rods are installed through the

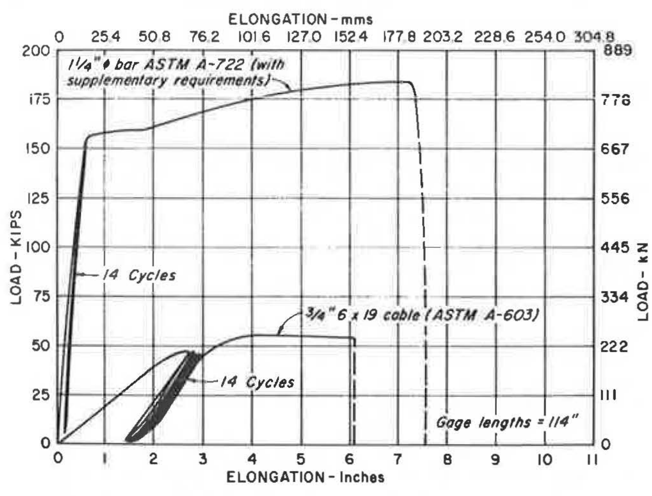


Figure 1

hinges in order to provide additional shear resistance.

California has conducted a number of tests of 3/4" cables and $1\frac{1}{4}$" ϕ bars to compare their qualities as restrainers. Figure 1 shows the stress-strain relationship of specimens tensioned from near zero stress to specified minimum yield stress (assumed to be 0.85 Fy for cables) for 14 cycles and then to failure.

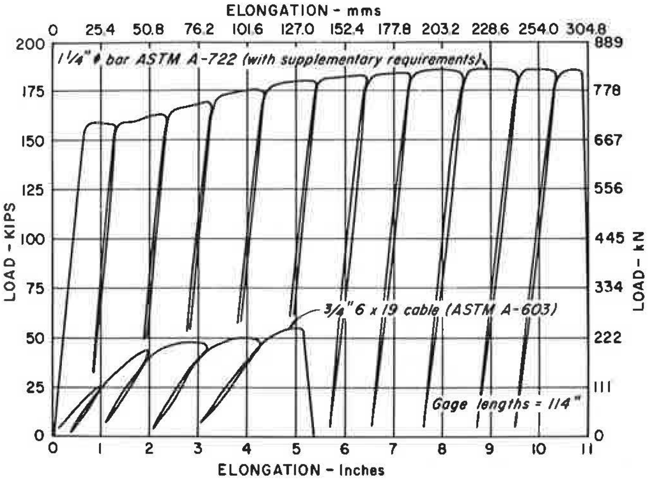


Figure 2

Figure 2 shows stress-strain relationships for cables and bars tensioned to failure but releasing the load to nearly zero at approximately one inch increments of stretching.

Cycling 3/4" cables within the elastic range required more than twice the amount of energy than cycling an equivalent number of $1\frac{1}{4}$ ϕ bars of the same length for the same number of cycles. This is due to the fact that bars have a greater modulus of elasticity and the elongation within the elastic limit is less than for cables. Within this range the cables and bars store energy but do not dissipate any significant amount.

The bars stretched and cycled beyond the elastic limit dissipated approximately 3 times as much energy as the equivalent number of the same length cables.

If restrainers are permitted to yield, greater joint openings and column deflections will be realized. Once either type of restrainer is stretched beyond its elastic limit it obviously will not assist in closing the joint to its normal position. Although bars will dissipate more energy than cables when failure occurs, the elongation will also be much greater. This could be an extra factor of safety in some structures but could be disastrous in structures with relatively short, stiff columns. When a restrainer is stretched to its ultimate limit, the structure is vulnerable to any additional shocks.

Considering the impreciseness of predicting a bridge's response to a possible future earthquake, it is generally not prudent to depend on restrainers acting beyond their elastic limit.

Restrainer Details

Figure 3 shows the most commonly used detail for retrofitting hinges of existing concrete box girder bridges. The concrete bolsters are generally required to spread out the concentrated forces of the restrainers so that they don't destroy the hinge

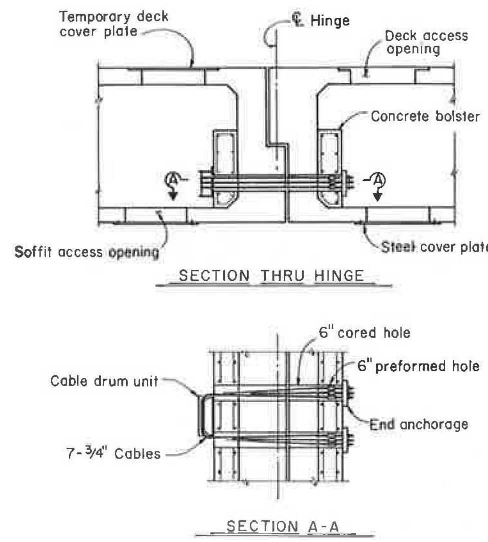


Figure 3

diaphragms. A minimum of one 7-cable (428 kip, 1,900 kN) unit placed in each exterior cell at each hinge is generally considered to be a minimum requirement in order to provide maximum resistance to transverse bending of the entire superstructure. Access to the cells is made through the soffit whenever possible in order to avoid interfering with traffic on the bridge. If access through the soffit is not possible or desirable due to conflicts with traffic under the structure, or other reasons, work is done through deck openings. In this case, traffic handling may become critical and work limited to off-peak hours. Steel plates set flush with the roadway surface are used to carry traffic across the access holes between working periods. Deck access holes must be permanently closed when work is completed. Holes in the soffit are covered with galvanized steel plates which can be readily removed for future inspections.

Figure 4 is a modification of the concept shown in Figure 3. It is generally restricted to hinges and end supports of shorter span T-beam bridges where the restraining force requirements are considerably lower.

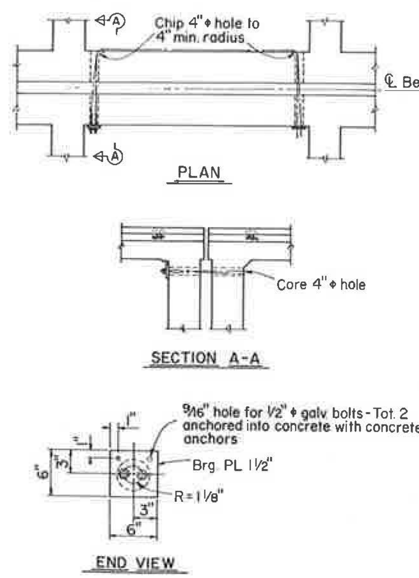


Figure 4

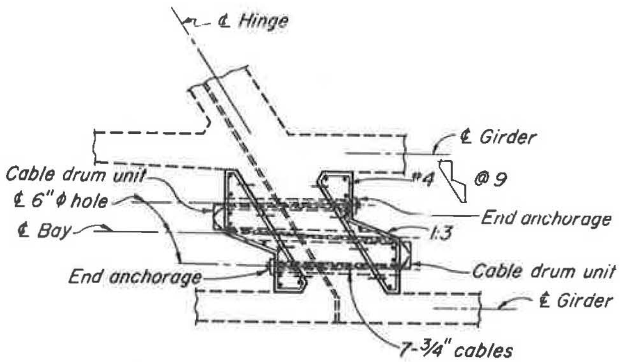


Figure 5

Figure 5 is another modification of Figure 3 and has been used in a few situations where the existing diaphragms are capable of resisting the greater force provided by the seven cables which pass through the joint three times.

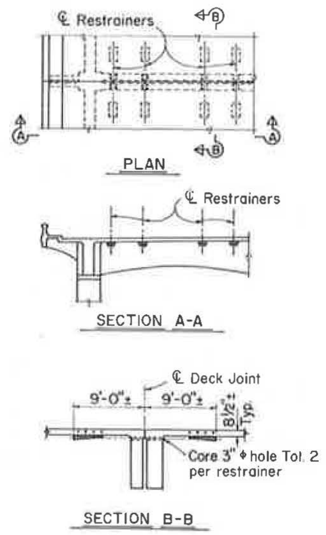


Figure 6

The detail shown in Figure 6 has been used on a limited basis where the diaphragms are not capable of being adequately strengthened and it would have been less desirable to attach restrainers directly to the girder stems. In this particular case it was necessary to place the cable anchorages far enough from the ends of the deck slab so that they would not pull the ends out of the spans.

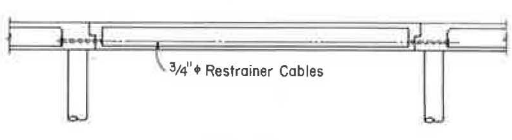
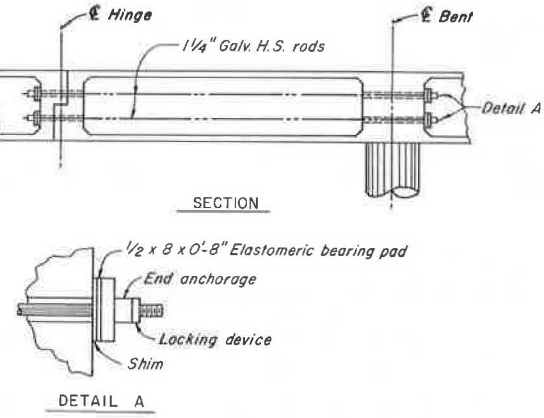


Figure 7

Variations of Figure 7 have been used in a number of instances where drop-in spans could be expected to fall if the structure were shaken in an earthquake. If the hinge seats are very narrow and the cables very long, additional cables might be required in order to limit the amount of stretching under seismic loading. This method is uneconomical in very long span.

An installation using high strength rods is illustrated in Figure 8. Cables could also be used



HIGH STRENGTH ROD RESTRAINER

Figure 8

in this scheme.

Figure 9 shows a commonly used detail for restraining steel girders which are in line with each other. When girders in adjacent spans are offset, transverse beams are attached to the bottom girder flanges which are used for anchoring the restrainer cables, as shown in Figure 10.

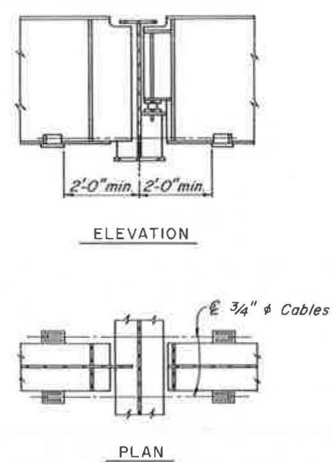


Figure 9

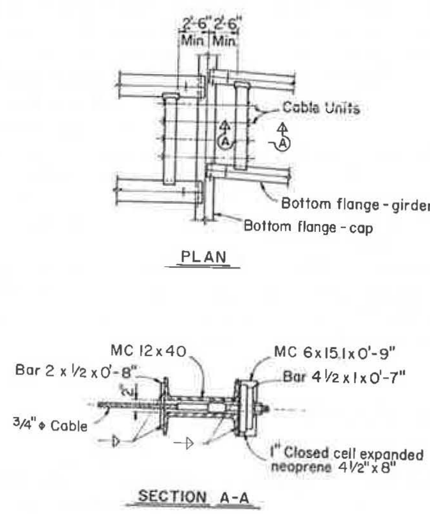


Figure 10

Figure 11 illustrates a method of attaching the ends of steel girders directly to the supporting concrete bents.

The restrainers illustrated above are only a few of the many types we have used to date. Each bridge has its own peculiarities and requires special attention and details.

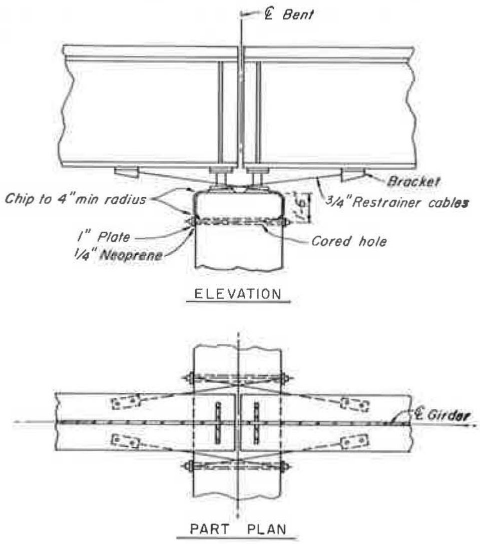


Figure 11

Costs

The following contract unit prices are taken from a large number of recent contracts which were bid competitively:

	Low	Avg.	High	
Deck access openings	$200.	$230.	$300.	/each
Soffit " "	200.	228.	300.	/each
Miscellaneous metal (cables, fittings, brackets, etc.)	1.50	1.75	5.00	/pound
Core 6" holes	38.	42.	62.	/lin.ft.
Core 4" holes	26.	33.	55.	/lin.ft.
Core 2" holes	18.	23.	30.	/lin.ft.
Diaphragm bolsters	200.	253.	300.	/each
Close deck access openings.	200.	251.	350.	/each

Installation of Restrainers

One of the main problems in connection with retrofitting existing bridges is minimizing interference with existing traffic. It is frequently necessary to limit work to off-peak hours. When retrofitting box girder bridges, the designer is given the option of specifying access to the girders through either the deck or soffit. Deck and soffit openings are generally made quite close to the hinges where tensile stresses in the girder reinforcement and compressive stresses in the concrete are relatively low, but far enough away so that the openings are not an inconvenience to the workmen.

Steel cover plates are generally required over the deck openings to provide for traffic during non-working hours. The 5/8-inch thick cover plates were placed on top of the deck in earlier contracts but were found to be hazardous to certain vehicles. Plates are now required to be recessed into the deck so they provide a flush riding surface. After work inside the girder cells is completed, extensions are welded to the ends of the cut reinforcing steel in the deck, to provide lap splices, and the opening is filled with concrete.

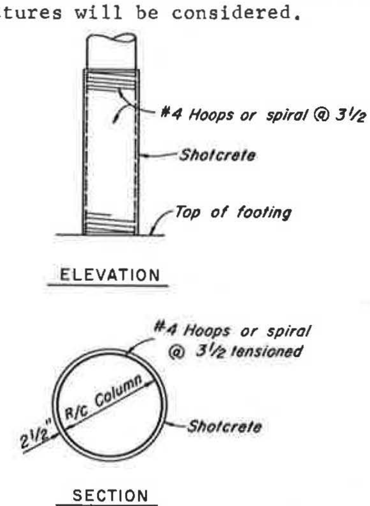
It is not considered necessary to replace reinforcement and concrete in soffit openings. Exposed ends of the reinforcing steel are painted with zinc-rich paint and a galvanized steel plate bolted over the opening.

Some contractors have expressed a preference for doing all of their work through the soffits whenever possible, in order to avoid conflicts with traffic on the bridge deck. Present equipment allows them to work as much as 100 feet from ground underneath a structure. A preference has also been expressed for gaining access to a temporary platform suspended underneath narrower structures from the bridge deck.

Retrofitting Columns

The second greatest weakness of Pre-1971 structures pointed out by the San Fernando earthquake was that the reinforcing steel ties in columns did not provide adequate confinement of the concrete. Bridges with single column bents are particularly vulnerable. Since the restraining of the superstructure at hinges and bearings was judged to be a more serious problem, and providing that restraint alleviated the seriousness of the column deficiency, more can be obtained for the money by retrofitting the hinges and bearings first. Methods of retrofitting columns to make them more earthquake resistant are being investigated and a developmental contract will be let in the near future for trying out some of the schemes.

All bridges which might require column retrofitting are currently being identified. When the developmental contract is completed a program to retrofit the columns of some of the state's more critical structures will be considered.

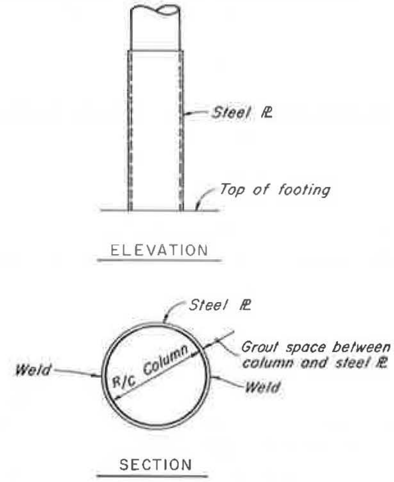


COLUMN RETROFITTING

Figure 12

Figure 13

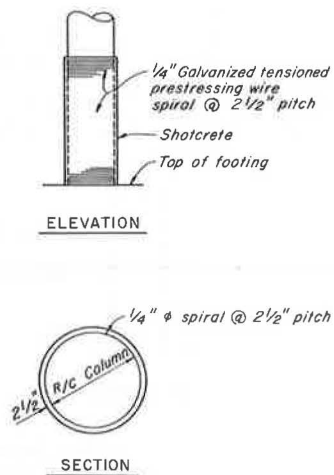
Figure 12 illustrates reinforcing steel hoops that are prestressed on the outer face of the column which is then covered with shotcrete. The device shown in Figure 13 was especially designed for this purpose. It is basically a turnbuckle which develops the strength of the reinforcing steel and places an initial pre-stress in the hoop.



COLUMN RETROFITTING

Figure 14

The column retrofitting method shown in Figure 14 consists of wrapping a column with tensioned prestressing wire and applying a protective coat of shotcrete.



COLUMN RETROFITTING

Figure 15

Figure 15 illustrates a method which consists of welding a steel shell around an existing column and filling the space between the shell and column with grout. "Weathered" steel can be used for achieving an architectural effect, if desired, or ordinary steel can be used and painted.

Conclusions

Many bridges which were designed by pre-1971 specifications and standards have serious seismic deficiencies. California has a program for retrofitting many of those bridges to make them more seismically resistant.

During an earthquake, each bridge abutment and pier can rotate in any direction independently, in phase or out of phase with any other pier or abutment. Ground between piers can distort elastically and in some cases may rupture or liquify. The seismic analysis of bridges and criteria for retrofitting bridges to increase their seismic resistance is, at the present time, a developing state-of-the-art process. Engineering judgment is an important factor in retrofitting bridges to make them more seismically resistant.