Earthquake Retrofit of California Bridge: Route 242/680 Separation

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The California Department of Transportation is currently implementing a statewide earthquake retrofit program. The goals of the program are to improve structural ductility and to provide corrective measures to the factors that contributed to the major damage and collapse of bridges during the San Fernando and Loma Prieta earthquakes. A case study of a retrofit design project of a bridge located in the San Francisco Bay area, in proximity of the Concord fault, is described. The structure type, geometry, traffic volumes, and maximum credible earthquake led to many difficulties in the analysis and resulted in solutions that were complex and unique for a bridge structure. The bridge investigated consists of two very different structure types joined by a voided pier that serves as a central abutment for both. The focus is primarily on the south structure, where base isolation was used, in contrast to the more conventional retrofit techniques used for the north structure.

The Route 242/680 separation carries Route 242 traffic over six lanes of Route 680 and Walnut Creek. This facility was designed in 1960, constructed in 1964, and seismically retrofitted in 1984 (prior to the Loma Prieta earthquake). The bridge is 12 spans, totaling 373 m (1,222 ft), with the longest span of 65 m (214 ft) crossing over Route 680. The structure is made up of two different types of bridges joined at Bent 4, which acts as a common seat abutment.

The south structure has a width of 12 m (39 ft 8 in.) and a depth of 2.44 m (8 ft) and is a continuous three-span prestressed concrete box girder supported by steel rocker bearings at Abutment 1 and Bent 4. At Abutment 1 and Bent 4 there is a gallery that is 0.92 m (3 ft) wide. Spanning this gallery and supporting vehicle traffic is a short flat concrete deck slab with a 0.15-m (6-in.) seat. At Bents 2 and 3 the superstructure rests on fixed steel bearings. These bents have five square columns, each with external bent caps. The abutment and bents of this section of the structure are on a 69-degree skew.

The north structure has a width of 12 m (39 ft 8 in.) and a depth of 1.22 m (4 ft) and is a continuous concrete box girder. Bent 4 is on a 69-degree skew, and Bent 5 through Abutment 13 are on a 54-degree skew. Steel rocker bearings support the superstructure at Bent 4 and Abutment 13. The bridge is composed of three frames with hinges located in Spans 6 and 9, near Bents 7 and 10, respectively. Bents 5 through 12 are two square-column bents, with the bent caps cast monolithically with the superstructure. The bridge spans Walnut Creek between Bents 9 and 10. To protect the columns within the channel during flood stages, concrete debris walls were constructed at Bents 7, 8, 9, 10, and 11 and were slightly slotted into column sides for support.

The abutments are of the seat type and are supported by two rows of piles. Bent 4 is a 9.14-m (30-ft)-high voided pier that acts as a seat abutment for both structures and is also supported by two rows of piles (Figure 1). The other bents are supported on pile footings. All piles are concrete with a compressive design load of 400 kN (45 tons).

The bridge had previously been retrofitted during the initial phase of the earthquake retrofit program of the California Department of Transportation (Caltrans) (Phase I). The goals of this phase were to provide continuity to the superstructure by connecting all narrow hinge seats with longitudinal cable restrainers and to place additional supports (concrete catcher blocks) at bearing locations to eliminate the chance of the superstructure losing elevation if the bearings fail. Bents 2 and 3 additionally have 19-mm (3/4-in.) cables connecting the soffit to the external bent caps to restrain longitudinal movement, also in case of bearing failure.

The closest fault is the Concord fault, which is 1.77 km (1.1 mi) from the bridge site. The maximum credible earthquake (MCE), as determined by a site-specific study, is a magnitude 6.5 on the Richter scale. The depth to rock-like material is about 45 m (150 ft), with a maximum horizontal bedrock acceleration of 0.53 *g*.

Analysis with As-Built Computer Model

A global three-dimensional linear response spectrum analysis was performed on the structure by using STRUDL. The following assumptions and methods were used in modeling the structure:

1. The superstructure was modeled by using a single line of elements.
2. A lumped mass model was used. Mass was placed at quarter points along the superstructure spans.
3. The bases of the footings were assumed to be fixed points in the model. The effects of the soil stiffness were accounted for only through application of the chosen response spectrum curve (see item 10).
4. For aged concrete a value of 34.47 MPa (5,000 lb/in.^{2}) for compressive strength was used according to the recommendation of Caltrans.
5. Gross (uncracked) section properties were used for the columns to obtain maximum force demand.
6. Cracked section properties were used for the columns to obtain maximum displacements.
7. Steel rocker and fixed bearings were assumed to fail during the design seismic event for a subsequent analysis. To represent friction contact between the soffit and catcher blocks, bearing locations were modeled with relatively soft spring elements. The spring stiffness was adjusted after the initial run to equal the dead load times the friction coefficient.
8. The hinges were modeled as short, rigid elements to allow for the proper member end force and moment releases to simulate actual hinge movement. Longitudinal cable restrainers across the hinges were modeled as short-space truss members of equivalent stiffness, parallel to and connecting to the same joints as the rigid hinge elements. To model the nonlinearity of the hinges with cable restrainers, two models were created. The first, a tension model, allowed relative longitudinal movement between adjacent frames by releasing the longitudinal force in the rigid hinge element and activating the cable restrainer elements. The second, a compression model, locked the longitudinal hinge force and allowed only moments about the vertical and horizontal centerlines of a hinge to be released.
9. Each cable restrainer unit (1984 seismic retrofit) at Bents 2 and 3 was modeled as an individual space truss element. Since only one side of the cable is in tension at any given point in time, each side is modeled with half of its equivalent stiffness. This resulted in the correct stiffness for longitudinal movements.
10. The response spectrum used was a standard Caltrans' elastic site spectrum (*ARS*) curve (*1*) with a depth of alluvium of 25 to 45 m (80 to 150 ft), acceleration of 0.6 *g*, and 5 percent damping, where *A* represents the base rock acceleration of 0.6 *g*, *R* represents the structure damping, which is assumed to be 5 percent, and *S* represents the effects of the depth of alluvium over the bedrock of from 25 to 45 m (80 to 150 ft).
11. Earthquake forces were applied independently along the centerline of the superstructure and perpendicular to the centerline. Forces were combined by using the standard Caltrans method (100 percent longitudinal plus 30 percent transverse and 100 percent transverse plus 30 percent longitudinal).

Summary of Structural Deficiencies

The plans were reviewed, and the results of the dynamic analysis obtained by using STRUDL, displacement ductility analysis (*2*), and hand calculation were evaluated. The following structural deficiencies were found:

1. All columns were designed with very little transverse steel. The columns in Bents 2 and 3 have No. 5 hoops at 0.3 m (12 in.). The other columns have No. 4 hoops at 0.3 m (12 in.). Because of this minimal transverse reinforcing of the ultimate curvature of the potential, plastic hinge zones are limited by the strain that causes the concrete cover to spall.
2. At each column ductility demands exceeded the allowable flexural ductility ratio of 1.5 (*2*), and the rotational ductility in the plastic hinge zone was also exceeded.

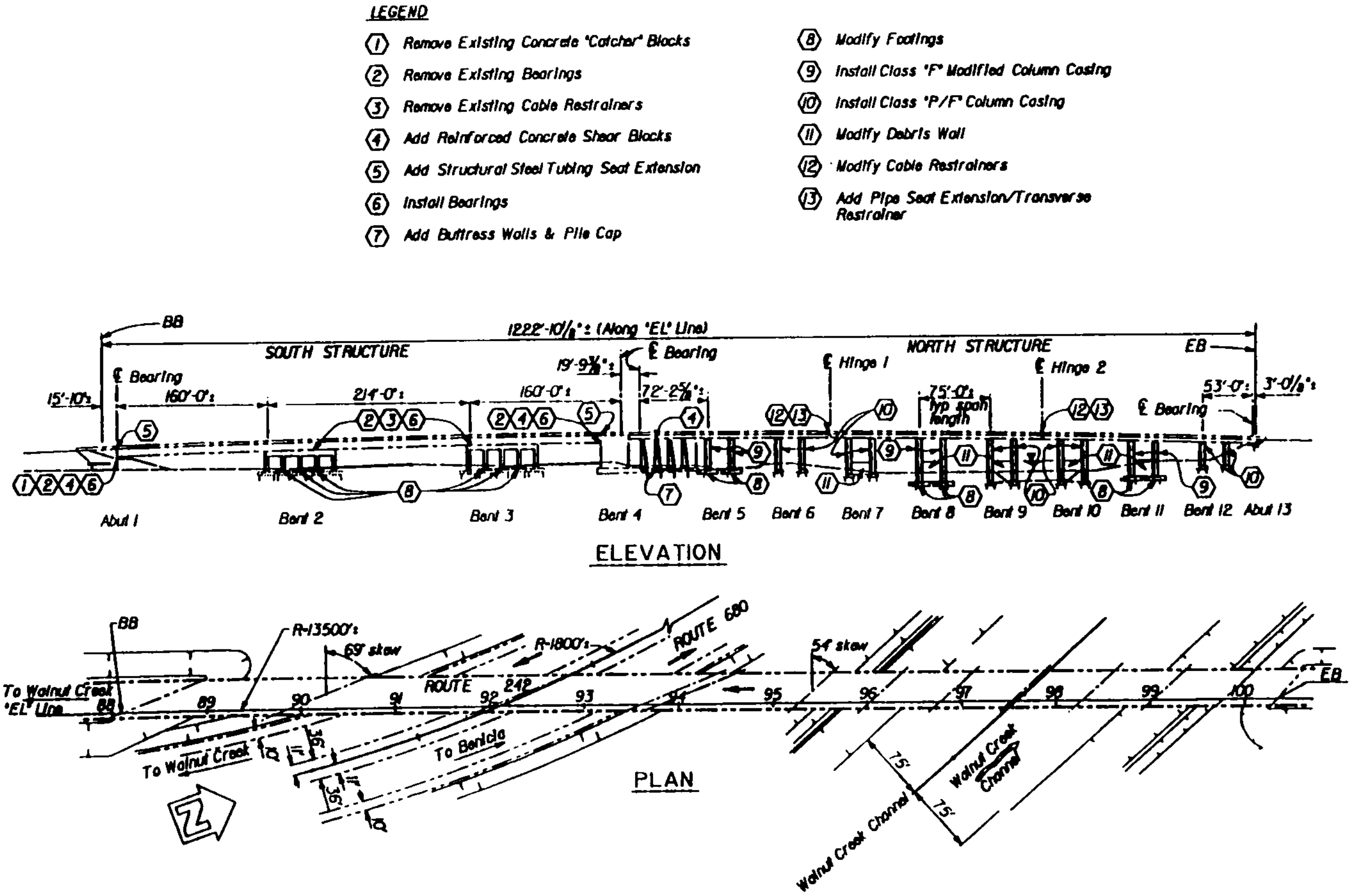


FIGURE 1 Elevation and plan of south and north structures.

3. Footings had insufficient moment capacity to resist the column's plastic hinging moments. This deficiency was due to the lack of top mat steel in the footing and insufficient pile tension capacity.

4. As assumed previously, rocker and fixed steel bearings would fail under the large lateral inertial forces.

5. The debris walls resulted in an unintended change in stiffness along the centerline of the bent that could force plastic hinging in the columns near the top of the wall. This would create an unstable situation if plastic hinges also occurred at the bases and tops of the columns.

6. The south structure exhibited unacceptably large displacements in both transverse and longitudinal directions that could cause the short concrete deck slabs spanning the gallery area to collapse, leaving a large gap in the roadway surface.

Retrofit Strategy Alternatives

The condition of the north structure posed no major obstacles to the use of a conventional bridge retrofit. The as-built condition of the structure was analyzed, and the minimum possible numbers of columns and footings were selected to be modified. The selection process was based on Caltrans criteria (2) to provide the minimum amount of modifications required to prevent collapse of the structure.

The retrofit of the north structure (Figure 2) consisted mainly of providing 9.5-mm (3/8-in.)-thick steel casings, to be installed around the columns, which are pressure grouted to provide confinement for increased ductility, and modifying the footings to withstand the plastic moments of these modified columns. Additional retrofit measures were to install casings at other columns ungrouted for twice the column width above the footing or below the soffit to protect against shear failure (2). The intent at these locations is to ensure a flexural failure at the tops or bottoms of the columns at a seismic demand level below that required for a shear failure. At these columns rotational capacity is lost either by failure in flexure of the column or failure of the footings or piles. Therefore, it is assumed that these columns have no lateral restraint and are only capable of carrying vertical loads after the seismic event. To reat-

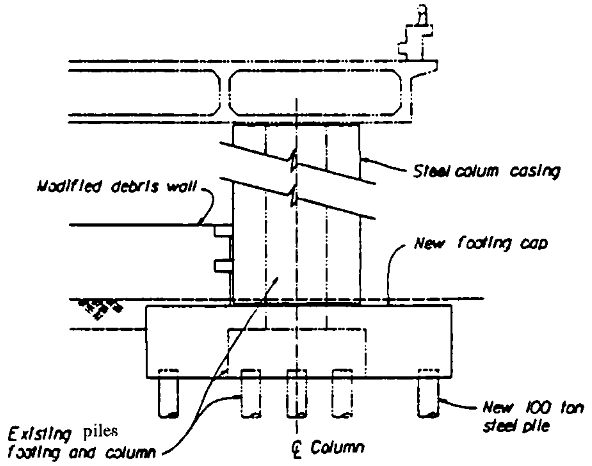


FIGURE 2 Column and footing retrofit.

tach the debris walls that were vertically saw cut and partially removed adjacent to each column for installation of the casing, a slotted steel debris wall-to-column casing connection with steel plates was designed. A small gap was left to prevent the columns from impacting the wall. This eliminated the potential for plastic hinging to occur in the column at the top of the wall height. In addition, the superstructure hinges adjacent to Bents 7 and 10 received pipe seat extensions (2). These 216-mm (8-in.)-steel pipes are installed in holes cored through the hinge diaphragms, parallel to the superstructure, to provide additional seat length and transfer transverse forces across the hinge.

The south structure provided some interesting challenges. Four possible retrofit strategies were investigated. The first two, Alternatives A and B, are termed the "resistive" solutions. The other two, Alternatives C and D, are termed the "isolated" solutions.

The resistive solutions involved standard modifications to the columns, footings, and bent caps of Bents 2 and 3. Both required steel column casings and complete footing modifications, including a reinforced concrete footing cap and additional 890-kN (100-ton) piles. In addition, Alternative A required removal of the bearings at Bents 2 and 3 and extension of the bent cap up into the superstructure to be monolithic with the internal diaphragm of the superstructure. Alternative B left the bearings in place, but it relied on modifications at Abutment 1 and Bent 4 to restrict the superstructure movement.

Both of these strategies met the goals of the design criteria and were therefore acceptable. However, several concerns arose. The cost (including that of the north structure, but not that of traffic control) of either of these strategies is approximately \$3.5 million or \$840/m^2 (\$78/ft^2) of deck. This is nearing the cost of replacement of the structure of about \$915/m^2 (\$85/ft^2) of deck. Also, with the major work required at Bent 3 (which lies in the median of Route 680), traffic handling would be a major concern. The closure of one lane on each side of the bent would be required for approximately 60 days. With high peak-hour traffic volumes (average daily traffic, 183,000; peak-hour traffic, 20,100) at this location, any lane closures would have a significant negative impact on the public.

The isolated solutions were then investigated. The first isolation strategy, Alternative C, simply involved allowing the steel bearings to fail at Abutment 1 and Bents 2, 3, and 4 (all points of support for the south structure). This would result in the superstructure dropping approximately 1 in. onto the existing concrete catcher blocks of the previous retrofit (Caltrans Phase I seismic retrofit program, 1984). It was anticipated that the friction that would develop between the concrete surfaces of the superstructure soffit and catcher blocks would result in forces and displacements at the tops of Bents 2 and 3 below their capacity. However, this did not hold to be true. The frictional force was assumed to be approximately 50 percent of the dead load, according to Caltrans recommendations. A static analysis of the bents by using the frictional forces applied at the top of the catcher blocks and a dynamic analysis by using STRUDL, with spring elements (similar to those used in the as-built model) used to approximate this frictional force, were performed. Both of these analyses indicated that the frictional force resulted in flexural stresses and displacements that exceeded the ultimate capacities of both the columns and the bent caps. The large horizontal displacements would also allow the short concrete spans to drop into the galleries. Furthermore, the locations of the existing catcher blocks are such that upon bearing failure the redistribution of dead load stresses in the bent caps and internal superstructure diaphragms exceeded their ultimate capacities. Because of these predicted failures and the unpredictability of the maximum lateral displacement of the superstructure, this alternative was not pursued further.

Final South Structure Strategy

The second isolated, and final, strategy, Alternative D, was developed. This strategy uses lead-core rubber seismic isolation bearings (3) (Figure 3) at Abutment 1 and Bent 4, and polytetrafluoroethylene (PTFE) spherical bearings (4) (Figure 4) at Bents 2 and 3.

This strategy provides some very distinct advantages over the other alternatives:

1. Increased system damping and, therefore, lower force demands. The lead-core rubber bearings act in

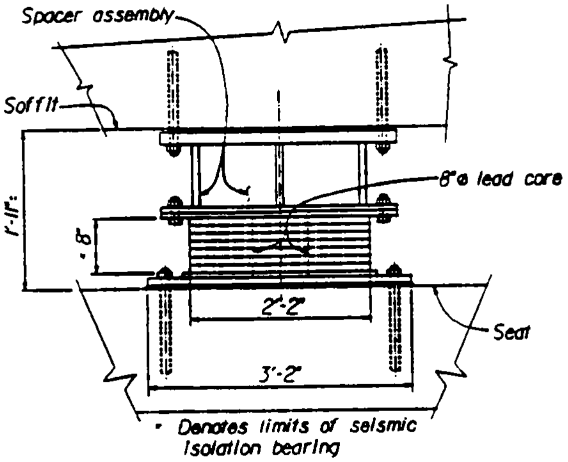


FIGURE 3 Seismic isolation bearing detail.

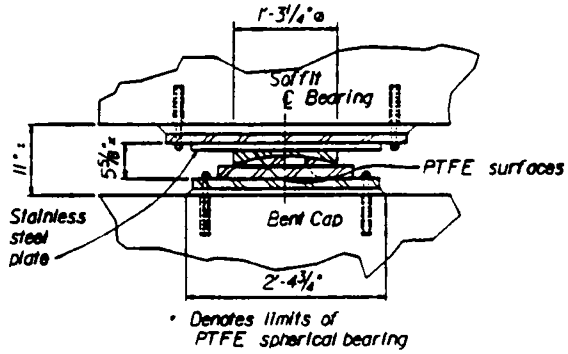


FIGURE 4 Elevation of PTFE spherical bearings.

two important ways to achieve this. First, the bearings add a soft stiffness component to the support points. This, combined in series with the more rigid stiffness of the existing substructure, results in a lower overall system stiffness and, therefore, a period shift to a region of lower demand on the response spectrum curve (Figure 5).

Second, the lead-core component of the bearing acts to raise the damping from the commonly assumed 5 percent to nearly 20 percent, based on the bearing size used and the configuration of the bridge structure. This is predicted to occur for modes with periods greater than 1.33 sec, at which point the lead core is predicted to begin plastic shear deformation.

Although a lead-core elastomeric isolation bearing was used for this project because of specific energy dissipation and displacement limitation requirements, other types of isolation bearings (such as friction pendulum, nonlinear elastomeric, etc.) are available. One or more types of isolation bearings may be acceptable for use in a particular project.

2. Dramatically reduced force demand at Bents 2 and 3. In addition to the beneficial effects of the lead-core rubber bearings given above, the PTFE-to-stainless steel contact of the PTFE spherical bearings reduces the frictional forces to less than 10 percent of the concrete-to-concrete friction forces from the as-built seismic forces (Figure 6).

3. Predictable displacement of the superstructure. Isolation bearings, including the lead-core rubber bearings used for this project, are designed to produce a hysteresis loop (Figure 7), which results in predictable displacement behavior. This was particularly critical for retrofitting the seat length of the short [length, 0.92 m (3 ft)] slab spans crossing the galleries at the ends of the south superstructure. Alternatives A and B also offered predictable displacement; however, Alternative C, which relied on concrete-to-concrete friction to resist lateral movement, did not.

4. Construction cost. The total construction cost, including that of the north structure, is estimated to be \$1.97 million, or \$474/m^2 (\$44/ft^2) of deck. This is a 43.6 percent reduction (\$1.53 million savings) from the typical resistive alternatives (Figure 8).

5. Traffic handling. This structure carries Route 242 over six lanes of Route 680. For approximately 6 hr per day the commuter traffic on Route 680, as well as Route 242, is stop-and-go, with an emphasis on stop. Any lane closures other than temporary night closures would have a significant adverse impact on this already poor traffic service level. The retrofit work required in the median of Route 680 at Bent 3, including the footing caps, can be accomplished from within the shoulder limits of the median. The result is a minor impact on traffic.

Additional retrofits required for this alternative are as follows:

- Remove existing cable restrainers at Bents 2 and 3 to allow for freer superstructure movement and less

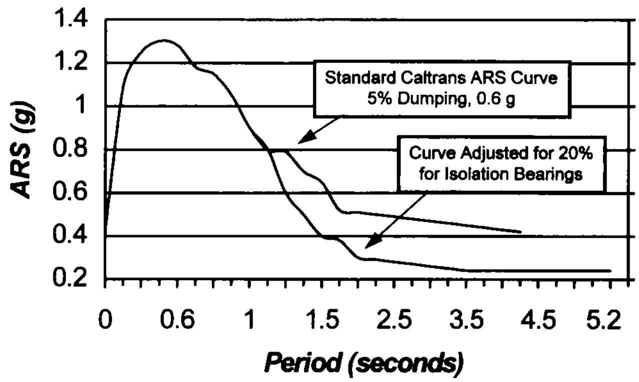


FIGURE 5 Composite response spectrum.

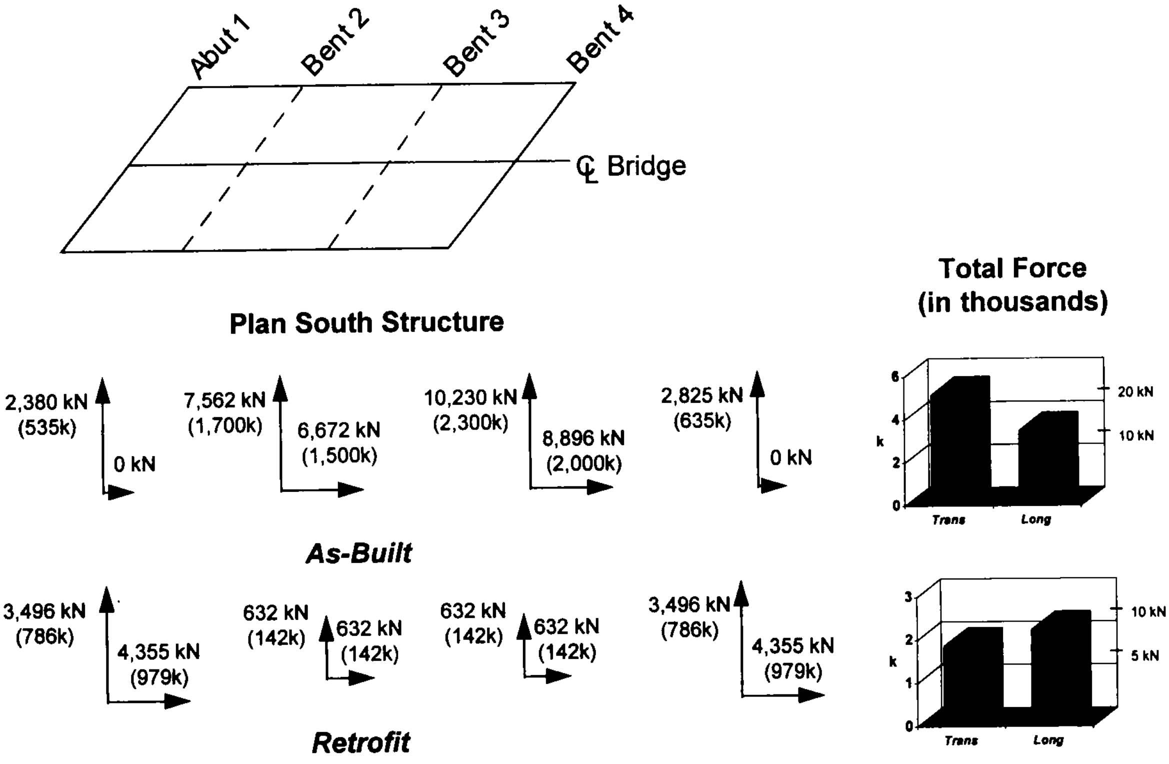
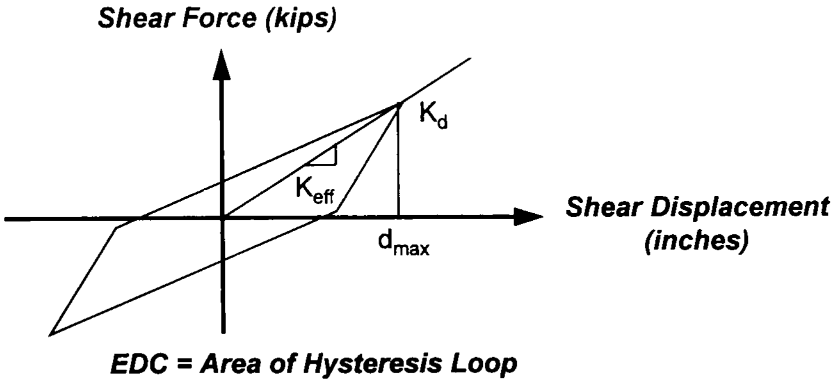


FIGURE 6 Redistribution of force.



Location		d_{max}	K_{eff}	K_d	EDC
Abut or Bent	Bearing Location	mm (inches)	kN/mm (kip/inch)	kN/mm (kip/inch)	kN/mm (kip-inch)
Abut I and Bent 4	All	275 (10.83)	3.22 (18.4)	2.22 (12.7)	289 (2560)

Force - Deflection characteristics, per isolator

FIGURE 7 Hysteresis loop.

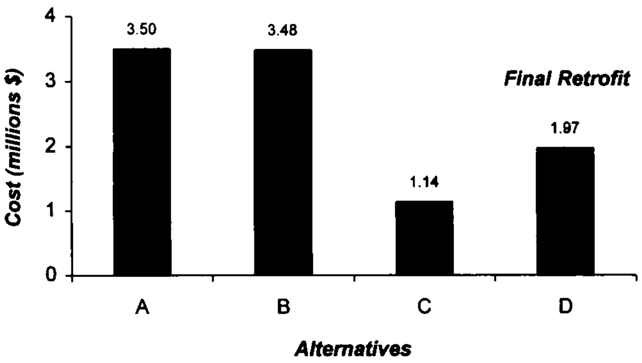


FIGURE 8 Cost of each alternative.

force delivered to the bents after installation of the new PTFE bearings.

- Install reinforced concrete shear or thrust blocks behind each lead-core isolation bearing at Abutment 1 and Bent 4 to prevent the superstructure from moving into the gallery area and crushing the short concrete slabs.
- Install a structural tube steel seat extension on the end diaphragms at both ends of the superstructure (Abutment 1 and Bent 4) to prevent the short concrete slab from dropping into the gallery.
- Reinforce Bent 4 (serving as a raised abutment) with a series of four buttress walls and pile-supported grade beam to prevent overturning of the bent and failure of the existing piles.

CONCLUSION

Before 1971 (San Fernando earthquake) bridge and building design codes required minimal attention to earthquake forces and seismic detailing. Demand forces were often only a small fraction of that required under current codes. Therefore, many structures of that era are potentially dangerous to the public and provide an economic liability to the owner. In most cases the cost of replacement is prohibitive, and thus, replacement is not a viable solution.

With any structure there may be many viable retrofit alternatives. It is the engineers' responsibility to fully understand the behavior of the structure, the impacts of the proposed retrofit, and the requirements of the owner. As a minimum all retrofits must prevent loss of life. Beyond this the owner's requirements must be fully understood, such as the level of structural damage an owner is willing to accept. On the basis of minimum life and costly safety criteria, this can vary from developing a retrofit that may leave the client with a condemned structure after an earthquake to developing one that results in little or no damage and no interruption to the use of the structure.

The case study presented here shows the benefit of investigating various solutions. For the south structure, Alternatives A and B met the required design criteria and may have been acceptable to the client. However, both would have been very costly to construct and the damage to the retrofitted structure during a large earthquake could still be very extensive. Alternative C, on the other hand, was the least expensive but was also the strategy that would most likely endanger the public and therefore was eliminated.

The selected alternative, Alternative D, not only provided a significant reduction in construction costs over those of resistive alternatives and would have a minimal impact on traffic, but also only minor damage, such as concrete spalling at thrust blocks, should result from an MCE.

In contrast, the north structure resulted in a very routine retrofit by using standard Caltrans (2) retrofit practice. For this more conventional (in California) portion of the structure, these standard retrofit procedures also resulted in the most economical solution

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