Application of Base Isolation to Single-Span Bridge in a Zone with High Seismicity

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The Carlson Boulevard Overcrossing is part of the 23rd Street Grade Separation Project, a major reconstruction of a complex intersection in the city of Richmond, California. The design of this bridge was based on a recommended, site-specific response spectrum because the project site is 3 km from the Hayward fault. The response spectrum values are very high over a broad range of periods. Also, because of the proximity of the structure to the fault, large displacements occur at the longer periods. The initial design approach was to use a two-span concrete structure with a center bent. It was then determined that the entire grade separation could be spanned with a single-span structure and the cost of the center bent could be saved. The single-span, multicell concrete box girder structure is 153 ft long, varies in width from 49 to 57 ft, and is on a skew of approximately 40 degrees. With the single-span structure, the lateral earthquake forces are required to be resisted by only the abutments. However, the tall seat-type abutments used for this project were not able to resist the large accelerations required by the response spectrum. Therefore, isolation bearings were selected to reduce the lateral seismic forces and to accommodate the large displacements at the abutments. The joints between the abutments and superstructure consist of 2-in. joint seals for everyday service use and short knock-off walls that give way for the large seismic movements. The analysis, design, and construction of the Carlson Boulevard Overcrossing are described in detail.

The 23rd Street Grade Separation Project (Figure 1) is designed to carry 23rd Street under the Southern Pacific Railroad mainline and the adjacent Carlson Boulevard. The completed project maintains—with much better geometrics—the traffic pattern that existed before underpass construction, but it eliminates a complicated six-point intersection crossing the railroad's main route to the Pacific Northwest, Central Valley, and eastern connections. Twenty-third Street is a major thoroughfare that currently carries an average of more than 25,000 vehicles per day, with traffic volume expected to increase to 40,000 vehicles per day once the work on nearby Interstate Highways 580 and 80 is completed.

Other project features include the removal of an abandoned Santa Fe Railway bridge, the relocation of utilities and petroleum project pipelines, and the design of a storm water pumping station, traffic signals, street lighting, landscaping, and a double-track railroad shoofly. The completed project will conduct traffic through

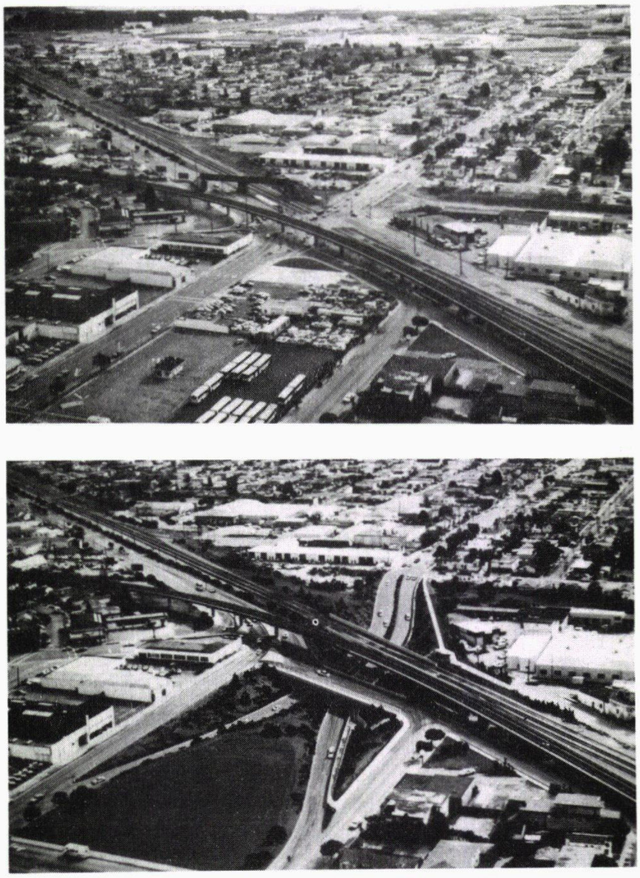


FIGURE 1 Aerial photographs of project site: (*top*) project site before construction, looking south; (*bottom*) artist's conception of finished project, looking South (Carlson Boulevard Overcrossing in foreground).

the area more efficiently and will create developable land that should invigorate existing businesses and enhance the neighborhood overall.

History and Design of Carlson Boulevard Overcrossing

The 23rd Street Grade Separation Project was planned and designed in the mid-1960s, but it was shelved before the project went out for bid. The Carlson Boulevard Overcrossing was originally designed as a four-span, reinforced concrete box girder bridge. However, in the late 1980s, when the project was resurrected, the geometrics had to be modified and retaining walls had to be added. To take advantage of these changes to the project, the Carlson Boulevard Overcrossing was shortened from 239 to 153 ft. With the shortened length, it became feasible to cross over 23rd Street with a single span.

In the period between the 1960s and the 1980s, the Bay Area Rapid Transit (BART) built an aerial structure between the Southern Pacific Railroad mainline and Carlson Boulevard. Subsequently, the design of the Carlson Boulevard Overcrossing in the 1980s had to address issues related to the construction of the new overcrossing structure in the vicinity of the existing BART aerial structure. One such site constraint imposed by BART was a vibration limit for the existing aerial structure throughout the construction period. Therefore, to reduce or eliminate vibration, the pile-supported substructure components of the project were changed from driven 12-in. precast, prestressed concrete piles to 30-in.-diameter, cast-in-drilled-hole (CIDH) concrete piles.

The Carlson Boulevard Overcrossing is a single-span, cast-in-place, prestressed concrete box girder, at-grade structure on an approximate 40-degree skew. At Abutment 1 one of the exterior girders is curved to parallel the horizontal curve of the roadway. The abutments are of the pile-supported, tall-seat type. The overcrossing is designed according to the State of California Department of Transportation (Caltrans) Bridge Design Specifications and 1983 AASHTO guidelines with revisions by Caltrans (*1*). The design concrete strength, f'_c, is 4,500 lb/in.^{2} for the superstructure and 4,000 lb/in.^{2} for the abutments. The total jacking force, with low relaxation strands, is 11,900 kips, or 1,983 kips per girder. All of the preceding design values are fairly typical of a regular Caltrans-type bridge. What is not regular, however, is the seismic design input and the subsequent method of design to accommodate the seismicity criteria.

Seismicity Criteria

The project site is located approximately 3 km from the active Hayward fault. Because of its proximity to this fault, Geospectra (in Richmond, California), the geotechnical engineer for this project, developed site-specific spectra for the project. To establish design earthquake ground motions at the site, Geospectra performed an in-depth seismic risk analysis (2). The seismic risk for the site was evaluated by using probabilistic assessments of ground motions. In general, it involved the use of a model for the seismic activities of pertinent seismic sources, the Hayward, Calaveras, San Andreas, Seal Cove-San Gregorio, Rodgers Creek-Healdsburg, and Green Valley-Concord faults, and a model for the attenuation of the ground motion, from source to site, to evaluate exposure to seismic activity and to estimate the probability that peak ground motion parameters would be exceeded during the estimated life of the structure. A more detailed description of the approach taken by the geotechnical engineer to conduct the seismic risk analysis and develop the site-specific spectra is beyond the scope of this paper.

For large return periods, the seismic risk analysis yielded a peak ground acceleration value of 0.7 *g* for the site. See Figure 2 for the site-specific elastic response spectra for both 5 and 10 percent dampings.

Seismic Design

The seismic design for a single-span bridge is not complicated. Normally, an equivalent static analysis is performed to determine the seismic load to be applied to the system. However, the results from an equivalent static analysis for the Carlson Boulevard Overcrossing yielded lateral loads greater than 1.6 times the weight of the superstructure, because the period placed the structure only slightly off the peak on the downslope side of the response spectrum.

A dynamic analysis was then done, modeling the abutment stiffness to see if the structure response could be brought down farther from the peak. Because of the nature of the response spectrum, somewhat flat at and near the peak, the dynamic loads were not much less than the previously calculated equivalent static load.

For a single-span structure, the seismic load due to the inertia of the superstructure is resisted only by the two abutments. The longitudinal forces (in the direction of traffic) are resisted by one abutment at a time. The transverse forces are shared by the two abutments. Caltrans seismic design criteria restrict longitudinal seismic forces above the abutment seat by allowing the abutment back wall (from the top of bridge deck to the soffit) to fail and mobilize the soil behind (Figure 3) to an ultimate pressure of 7.7 ksf. Transverse seismic forces are resisted by abutment shear keys (limited to 75 percent of the lateral capacity of the abutment piles) plus the shear capacity of one abutment wing wall (Figure 4). The design criteria for resisting longitudinal and transverse seismic loads is intended to keep all damage above the footing level.

The seismic design forces, as calculated by equivalent static or dynamic analysis, exceeded the ultimate longitudinal pressure of 7.7 ksf and indicated the need for additional CIDH concrete piles to resist the transverse forces. Adding these piles was considered only a partial solution, because it did not address the longitudinal force problem.

Ideas to modify the abutments to accommodate the seismic forces were numerous and varied. One idea was to design a friction slab under the approaches and behind the back walls (Figure 5) to use the weight of overburden to resist the longitudinal forces. A friction slab, however, would have had such substantial dimensions that use of a friction slab would not have been economical or feasible. To provide additional transverse resistance, interior shear walls located behind the abutment wall and below the seat were considered (Figure 6). Again, this led to another problem: the abutment dimensions did not allow for the required number and size of shear walls to be constructed.

Another idea was to provide restrainer cables (Figure 5) at each abutment so that both abutments could resist the seismic forces simultaneously in the longitudinal direction. This mechanism was determined to be undependable, however. Even the idea of reintroducing an intermediate bent was entertained. This bent would have provided another lateral load-resisting system, but it also would have cluttered the grade separation by adding columns to the median of the separation (although this was the original concept in the 1960s). The bottom line, however, was that an intermediate bent was not cost-effective. At a meeting with Geospectra to discuss the development and consequences of the site-specific response spectrum, the question of how to design for the large load magnitudes generated by the response spectrum was considered. At this point the geotechnical engineer suggested investigating the use of base isolation to absorb the forces produced by the superstructure impacting the abutments. He further explained that for structures in close proximity to a main causative fault, displacement is as important a design consideration as ground acceleration due to ground motion, and base isolation would be a good candidate that could be used to accommodate both considerations. Preliminary calculations were run to determine if lower levels of superstructure acceleration could be achieved without increasing superstructure displacement to unacceptable values. The calculations showed that it was attainable. Dynamic Isolation Systems, Inc. (DIS), was then contacted for support and to provide a detailed analysis and design of the base isolators for the superstructure.

Isolation System Design

A successful seismic activity isolation system must provide a horizontal plane of flexibility to lengthen the period of response, substantial amounts of energy dissipation to control displacements, restoring force for centering after an earthquake, and high initial stiffness to resist lateral service loads such as wind and traffic loads. The lead-core rubber isolation bearing designed by DIS (Figure 7) provides all of these features in a single component.

The first step in the design process was to conduct a feasibility study to assess the benefits of introducing isolation bearings at the abutment seats. DIS was presented

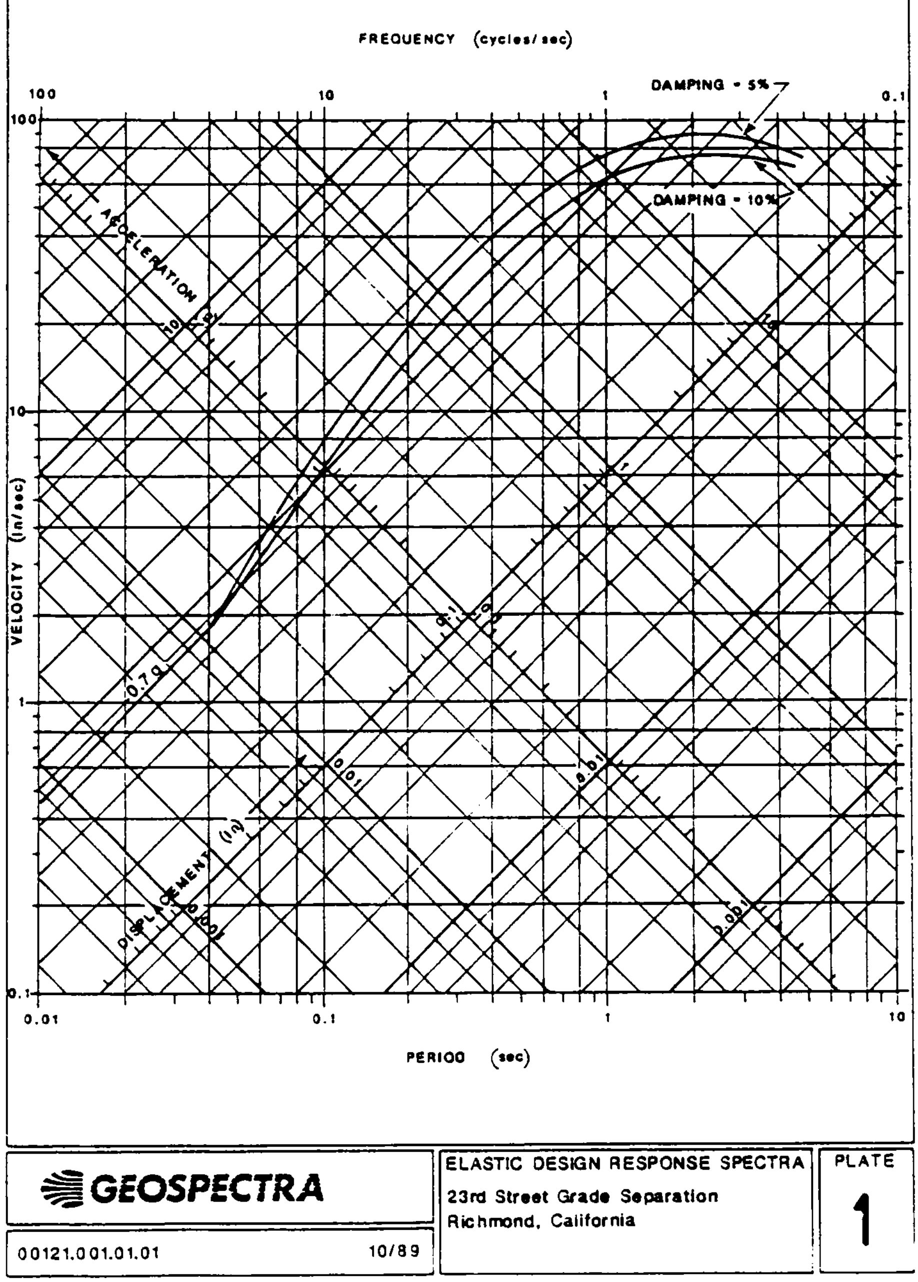


FIGURE 2 Response spectra (*continued on next page*).

with the necessary information, including a general plan, site-specific design response spectra, and dead and live load reactions for each abutment. Usually, it would be important to provide information regarding wind and traffic loads and thermal and creep movements. This information is needed to ensure that the design of the seismic isolators can also accommodate these service load conditions. However, it was clear from the site-specific spectrum that seismic criteria would govern the design, which could be checked for adequacy against service loads.

The DIS software program LEADeR was used to design the lead-core rubber isolators. LEADeR performs the two interrelated parts of the design process: sizing the isolators to ensure adequate factors of safety under all seismic and nonseismic load combinations and evaluating the seismic performance of the isolators so designed. The Caltrans curve for 0.7 *g* and 80 to 150 ft of alluvium was used to design the isolators, because this provided a good match with the site-specific spectrum for isolated periods greater than 1.25 sec (Figure 2).

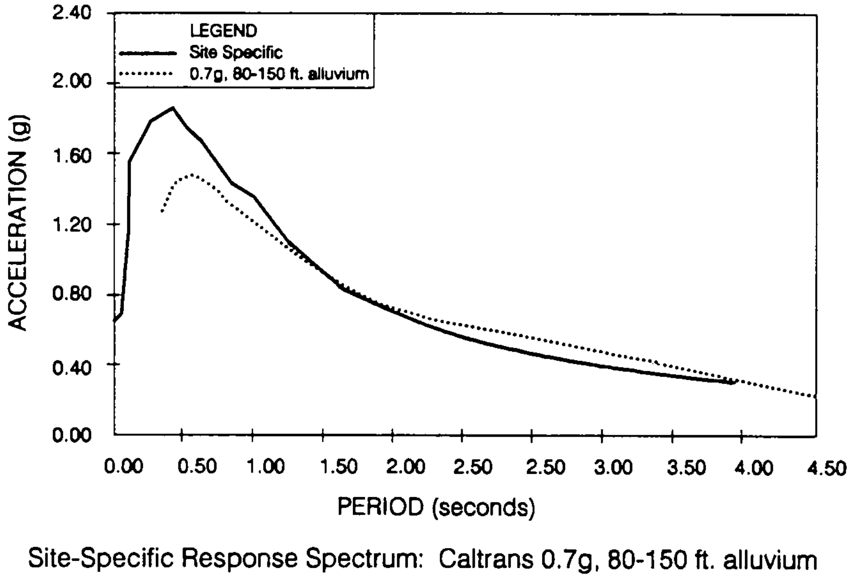


FIGURE 2 *(continued)*.

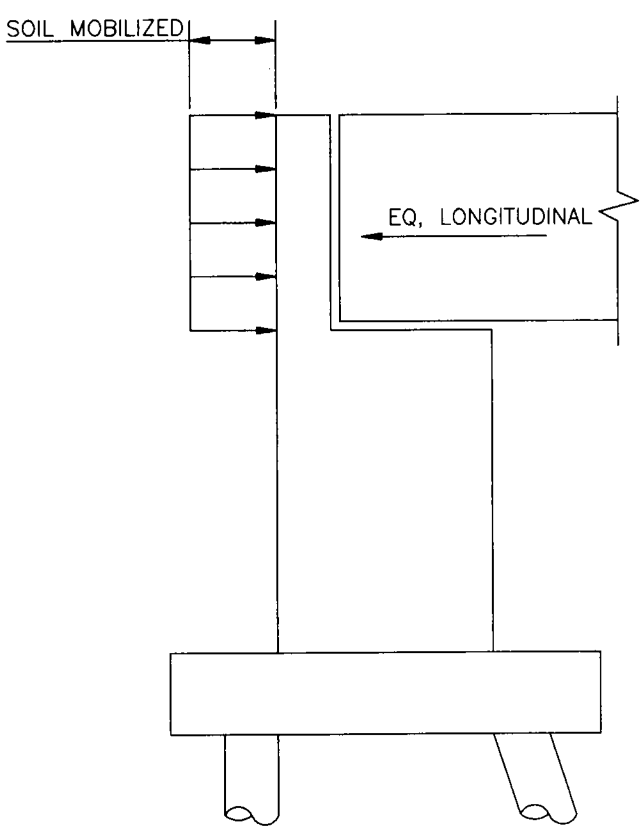


FIGURE 3 Abutment section.

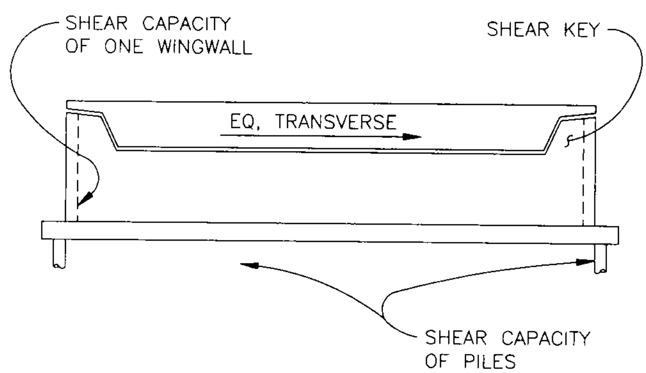


FIGURE 4 Abutment front section (*3*).

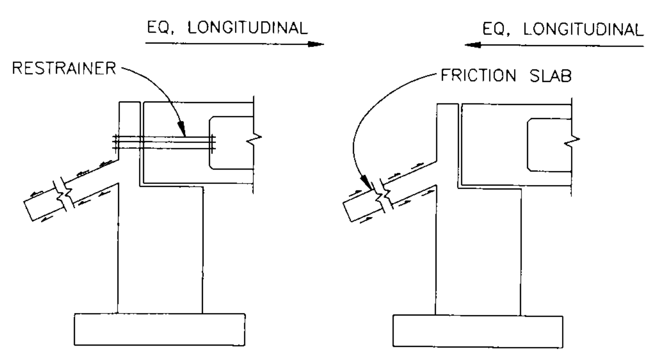


FIGURE 5 Abutment sections with friction slab and restrainers (*3*).

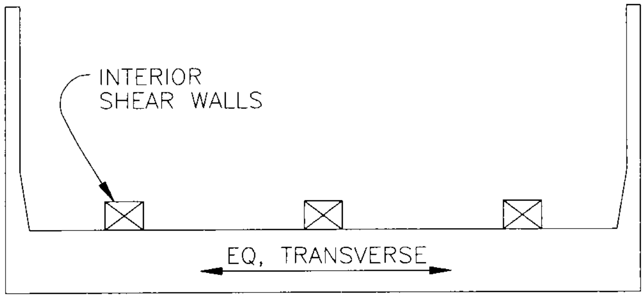


FIGURE 6 Abutment plan with interior shear walls (3).

The study assumed four isolators at each abutment, requiring the end diaphragm to span approximately 15 ft between isolators. For the total vertical load at each abutment, the maximum dead and live loads per isolator were 325 and 48 kips, respectively. By using the LEADeR program, the design called for isolators 29 in. square in plan, nominally 12 in. high, with a 7.25-in.-diameter lead core. A plan size of only 21 in. square was adequate to support dead and live loads; however, in this case, the plan size was controlled by the requirement to carry the dead load safely at the seismic displacement under the maximum credible earthquake. The lead cores, as dimensioned, provided a yield level of $0.15W$ (where W is the weight of structure above the isolators), which resulted in an elastic lateral load capacity per isolator of 43 kips, which was more than sufficient to resist the applied wind and traffic loads. The isolator height was also more than that required to accommodate creep and thermal movements.

The square isolators were to be oriented parallel to the skewed superstructure alignment. Because of the oversized masonry plate required to anchor the isolators, the abutment seat was inordinately wide. It was decided to use circular isolators to reduce this width. The final design produced a 31-in.-diameter isolator, nominally 13 in. high, with the same 7.25-in.-diameter lead core. The effective period is approximately 1.6 sec, and the elastic force coefficient is 0.46, which represents a reduction factor of approximately 3.5 from the conventional coefficient of 1.6. The maximum displacement is 11.6 in.

Isolator Specification and Testing

The final design was implemented with a performance specification that included tabulation of the required performance characteristics on the isolator plan sheet (Table 1). The essence of the performance specification is embodied in the testing requirements, which are designed to ensure that the properties of the isolators installed in the structure are those on which the engineer's global design is based. Prototype tests of isolators are required to confirm the design properties used for the analysis and performance evaluation of the structure. Quality control tests are performed on isolators intended for installation and are a means of verifying the consistency of properties over a large number of units, as well as allowing visual inspection of each unit under compressive and shearing load conditions.

Four prototype tests were performed. The first checked nonseismic displacement, under repeated service loading, against the specified maximum of 0.13 in. The two intermediate tests checked isolator performance at various increments of the specified maximum seismic displacement of 12 in. The final test subjected the isolators to three fully reversed cycles of loading at 1.5 times the seismic displacement, or 18 in. Throughout these tests, the isolators remained stable and exhibited the desired force-deflection and energy-dissipation characteristics.

The quality control tests included those performed in accordance with relevant ASTM standards for the re-

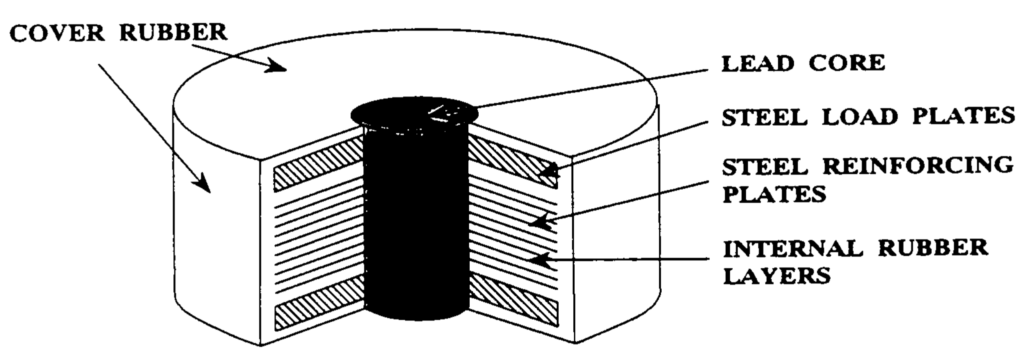


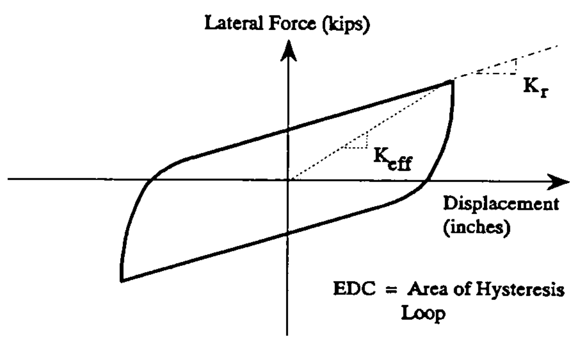
FIGURE 7 Seismic isolation bearing.

TABLE 1 Isolation Performance Specifications

Seismic Isolation Bearing Performance and Test Data, Based on Site-Specific Spectra		
Acceleration Level	0.7g	
Effective Period (secs)	1.6	
Maximum Displacement (inches)	12	
Maximum Lateral Non-Seismic Displacement (inches)	0.13	
Elastic Force Coefficient	0.45	
Effective Damping, %, at 12 inches Displacement	20	
Shape	circular	
Maximum Baseplate Size (circular or square)	3'-0"	
Dead Load	1300^{k} (1)	325^{k} (2)
Live Load	220^{k}	55^{k}
Maximum Lateral Non-Seismic Force (kips - total at each abutment)	50^{k}	12.5^{k}

Notes: (1) Kips - Total load at each abutment
(2) Kips - Load per bearing; with 4 bearings per abutment

Required Hysteretic Behavior of the Seismic Isolation System at 1.6 Seconds per Cycle Period of Vibration			
Each Bridge			Tolerance (Prototype Test)
K_{eff}	K_r	EDC	$K_{eff} \pm$ 10% K_{eff} (Table)
kips/inch	kips/inch	kip-inch	$K_r \geq$ 90% K_r (Table)
12.5	8.5	2095	EDC $\geq$ 90% EDC (Table)



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quired material properties of the isolator's components (rubber, steel, and lead). The completed isolators were tested under sustained compression, compression stiffness, and combined compression and shear conditions. In addition, a sample taken from a prototype isolator was tested in accordance with the provisions of California Test 663, which requires a minimum fatigue life of the rubber bond of 10,000 cycles at ±50 percent shear strain.

Practical Aspects

The use of isolation bearings led to some special design considerations. Because the lead-core rubber isolation bearings were designed to safely provide up to 12 in. of displacement in any lateral direction, a detail allowing this amount of movement at the superstructure-abutment interface had to be developed. The final design is shown in Figure 8. The knock-off concrete block at the top of

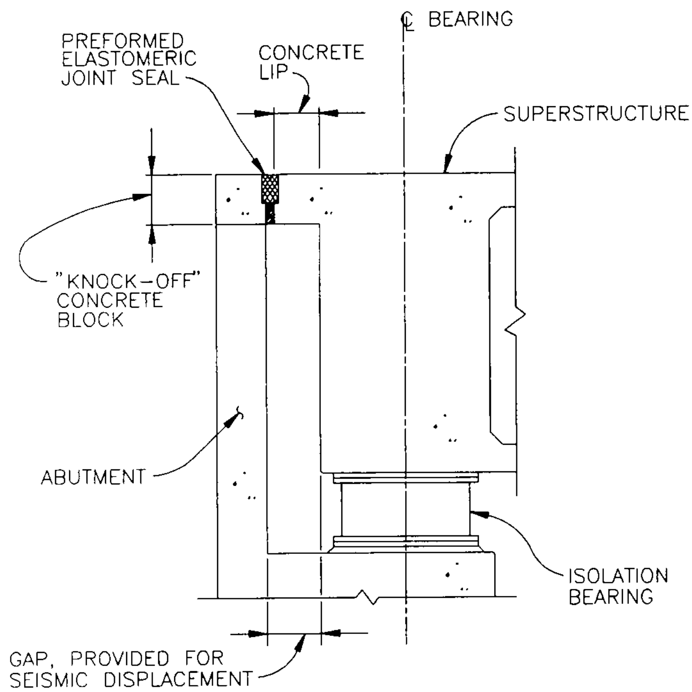

FIGURE 8 Superstructure-abutment interface.

the abutment back wall is intended to break away when it is struck by the concrete lip hanging off the end diaphragm; this will provide for the anticipated seismic movement. Everyday movements are accommodated by the preformed elastomeric joint seal detail. At the barriers and sidewalk, a 12-in. gap was detailed into the superstructure-abutment interface for seismic displacements. Steel plates were used at the barriers and tread plates were used at the sidewalk to produce a continuous surface across the gap. Finally, the abutment seat had to be widened to support the rather large circular isolation bearings, but not as much as would have been needed to accommodate the square bearings.

As for any special construction aspects, the use of isolation bearings had no impact on the schedule other than that they be manufactured and tested early in the project—earlier than use of regular elastomeric bearings would have required.

The postseismic aspect of the isolation bearing design compares favorably with the only possible and workable alternative discussed previously in the Seismic Design section: the addition of an intermediate bent. The base isolated design would require the replacement of the knock-off wall and, on either side of this wall, the repair of the roadway and possibly the overhanging concrete lip. During reconstruction, traffic access across the overcrossing may still be maintained by placing steel plates over the damaged sections of the approaches.

Conclusions

The single-span Carlson Boulevard Overcrossing presented many different interesting design facets to the project: from being a redesign of a project completed 20 years earlier and having to be constructed close to an existing aerial structure to having to be designed for site-specific spectra because of its proximity to an active fault. The seismic design required investigating several provocative alternatives, the most provoking being a return to the original design concept of using multiple spans. Fortunately, for the cleaner design concept of the late 1980s, the use of isolation bearings prevented the design direction from being reversed to the use of multiple spans visualized in the 1960s. Because isolation bearings are specified to be tested individually, they can be relied on to perform their intended function for everyday service loads and the infrequent seismic loads over the life of the overcrossing. The examination of numerous seismic design strategies led to a simple, cost-effective solution that uses seismic isolation. This retained the open, single-span design and provided a simple solution to an otherwise frustrating and complicated problem.

Acknowledgment

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