# RESPONSE OF BASE-ISOLATED STRUCTURES IN RECENT CALIFORNIAN EARTHQUAKES

James M. Kelly, Ian D. Aiken, and Peter W. Clark

## ABSTRACT

The October 1989 Loma Prieta and February 1990 Upland earthquakes both affected base-isolated structures and provided the first significant earthquake response data from base-isolated structures in the U.S. In the Loma Prieta earthquake, California Strong Motion Instrumentation Program (CSMIP) sensors on the Sierra Point Overhead were triggered, and in the Upland earthquake, the Foothill Communities Law and Justice Center (FCLJC) in Rancho Cucamonga experienced significant ground accelerations, CSMIP instruments again recording the motions.

The responses of the two structures are investigated in a study of the strong-motion data. Isolation system characteristics are determined from the data and these are used in analytical studies of the structure responses. Implications of the results on current design approaches and code requirements for base-isolated structures are addressed.

## 1. INTRODUCTION

### 1.1 Description of Structures

**FCLJC** - The FCLJC is a four story plus basement braced steel frame. The building was designed in 1983 and construction was completed in 1985. The building is 414 ft by 110 ft in plan, and the height of the main roof above the isolation bearings is 76.5 ft (Fig. 1). At the basement level, 14-inch thick concrete shear walls extend the full height of the basement. These walls serve to spread overturning actions in the braced frames onto the bearings. The foundation system consists of individual spread footings for each isolation bearing. The site is underlain by sand, silty sands, gravel, and a coarse-to-fine alluvial sequence, with basement rock at a depth of about 800 ft.

The FCLJC was the first base-isolated structure in the United States, and as such, a number of conservative assumptions were made in the design of the building. The County of San Bernardino requested that the building would experience only minor structural damage in the Maximum Probable Earthquake — the maximum event that could be expected during a period of 100 years — and would not suffer permanent damage to the basic structure in a Maximum Credible Earthquake — the worst seismic event that is postulated within the geotechnical framework of the site [1]. The Maximum Credible Earthquake was defined as a Richter magnitude 8.3 event on the San Andreas Fault, 13.5 miles from the site. This event corresponds to a motion with PGA = 0.6 g, a spectrum with constant velocity of 50 in/s in the period range beyond 0.8 second, and a duration longer than 35—40 seconds. The extremely long (414 ft) plan dimension of the building resulted in very large torsional effects when the 5% eccentricity code requirement was included in the analyses. The FCLJC isolation system is discussed in detail in section 2.

**Sierra Point Overhead** - The Sierra Point Overhead is an 8-lane, 616 ft long bridge on U.S. Route 101 north of San Francisco International Airport. The 117 ft wide, 10-span bridge is skew in plan, with the north and south abutments skewed 59° and 72°, respectively, to the radial line.

The bridge was designed in 1955, constructed in 1957, widened in 1969, and base-isolated as part of a seismic upgrade project in 1984-85 [2]. Plan and typical section views of the structure are shown in Fig. 2. The superstructure consists of a concrete deck slab on steel girders supported on lead-rubber isolation bearings. The bent columns are cantilevered from individual spread footings on hard sandstone. There are a total of 27 concrete columns, 36 inches in diameter, and they are generally arranged in 4-column bents. The bents at the ends of the bridge have only one, two, or three columns because of the skew alignment. The columns vary in length from 11 ft, 19 ft, and 27-30 ft, with the shorter columns being nearer the abutments. The bridge carries traffic over two railroad tracks. Three pre-cast, reinforced-concrete collision walls, 13.75 ft high and 3 ft thick extend between bent columns parallel to the railroad alignment as shown in Fig. 3.

The bridge was originally designed under 1953 AASHO specifications and in a 1982 structural review was found to be deficient in column strength. Base isolation was chosen as an alternate solution to reconstruction, and was able to reduce the horizontal earthquake loads subjected to the vulnerable columns. The structure was isolated by replacing the spherical steel pin-type bearings with lead-rubber isolation bearings. The design earthquake was a Richter magnitude 8.3 occurring on the San Andreas fault 4½ miles west of the site. The isolation system is discussed in detail in section 2.

### 1.2 Earthquake Motions

The epicenter of the $M_L$ = 5.5 February 28, 1990 Upland earthquake was located about 8 miles east of the FCLJC building. The maximum free-field acceleration recorded at the site was 236 gals, and 40 seconds of data were acquired. The free-field accelerations were significantly larger than those experienced by the building. Instrumentation on the FCLJC consists of 16 accelerometers on the structure itself, and an additional 3 free-field accelerometers located approximately 350 ft SE of the building. The instrumentation locations are shown in Fig. 1.

The $M_L$ = 7.1, October 17, 1989 Loma Prieta earthquake was centered about 50 miles south of the Sierra Point Overhead bridge. The maximum site free-field acceleration was 104 gals, and 38 seconds of data were acquired. Instrumentation on the bridge consists of 13 accelerometers located on two adjacent columns (Fig. 2). Accelerations are measured above and below two isolators and at the base of one of the columns. Three additional sensors are located in the free-field, on a sidehill bench on sandstone bedrock 200 ft NW of the bridge, at approximately the same elevation as the bridge deck.

No damage occurred in either of the structures as a result of the earthquakes.

## 2. DESCRIPTION OF ISOLATION SYSTEMS

**FCLJC** - The building is supported on 98 high-damping natural rubber bearings and has a design period of approximately 2.0 seconds under the Maximum Credible Earthquake. There are eight different bearing types incorporated in the design. All of the bearings are 20 inches in diameter and the total height of rubber in each case is 11.97 inches. A view of one of the bearings during construction is shown in Fig. 4. Four different rubber compounds were used in the manufacture of the isolators, aspects of which are discussed in more detail in section 3. The maximum displacement in the isolators under the Maximum Credible Earthquake was calculated to be 15 inches. To accommodate this displacement, a seismic gap of 16 inches is provided on all sides of the building. Ball joints were added to piping and utility connections across the isolation interface to allow the large horizontal displacements. The isolation bearings possess substantial inherent damping, which varies from about 9 to 12% depending on the level of deformation in the isolators. The first two fixed-base translational periods were calculated to be 0.62 second and 0.72 second in the N/S and E/W directions, respectively. Connection of the bearings to the foundation and superstructure is by steel pin shear dowels into the bearing end plates.

Each isolator used was tested at the time of manufacture at shear strain levels of 2%, 10%, 25%, and 50% [1]. First cycle (unscragged) and tenth cycle (fully scragged) stiffnesses were obtained. The composite stiffnesses of the entire isolation system (unscragged and scragged) are given in Table 2. In addition to these tests, four bearings were tested to the full design displacement of 15 inches [3].

**Sierra Point Overhead** - The Sierra Point Overhead isolation system consists of 33 lead-rubber and natural rubber bearings. There are a total of six different bearing designs in the system. There are two bearings sizes: 18 inches square or 22 inches in diameter; the square bearings have either no lead plug or a lead plug of 3 inches or 6 inches in diameter. The circular bearings have 4 inch diameter lead plugs. The total height of rubber in the bearings is either 5.625 inches or 6.375 inches. The natural rubber used in the bearings is an unfilled compound with a relatively low inherent damping and a moderately nonlinear low-strain stiffness behavior. The abutment details were not changed in the isolation retrofit, and remain the same as in the original 1957 construction, being 1 inch gaps filled with expansion joint material. The maximum displacement of the deck with respect to the footings in the design earthquake was calculated to be 7-9 inches, involving about 1½ inches in the columns and the remaining displacement in the isolators themselves. The maximum isolator displacement was calculated to be 7½ inches.

Clearly, the required seismic gap has not been provided. The reason for not doing so at the time of the upgrade was given as economic. In the event of a major earthquake and consequent damage to the haunch at the top of the abutment backwalls, the required seismic gap would be provided at the time of repair.

The overall design of the bearings was influenced by several dimensional constraints. The removal of the spherical steel pin type bearings provided a clear gap of only 9 or 10 inches between the top of the columns and the underside of the bent cap girders. The plan size of the bearings was limited by the overall size (36 inches diameter) of the columns. Because of the height restrictions, steel dowels were not used for the connection between the upper and lower bearing plates and the isolators themselves. The bearings are kept in place by keeper plates welded to the sole plates.

## 3. RUBBER COMPOUNDS

**FCLJC** - Four distinct compounds were used in the FCLJC isolators, all identified as high-damping natural rubber. The compounds are designated by the supplier, LTV Energy Products, Arlington, Texas (in order of decreasing shear modulus), as 246-70, 243-65, 2x-69, 2x-71.

All of these high-damping compounds have a highly non-linear stress-strain behavior that is particularly advantageous for base-isolated systems. The materials are very stiff for cycles of shear at small strain levels. The effective modulus decreases as the strain level increases up to about 100% and for cycles of shear strain beyond this level it stiffens up again. Thus, the system is stiff for wind loads, environmental disturbance, and small earthquake input. It softens for large earthquake input providing a long period isolation effect and if the motion is much greater than that assumed in the design, the stiffness increases and the displacements are controlled.

Another aspect of the mechanical behavior of these compounds that is important in interpreting isolation system performance is the fact that the material undergoes a reversible process known as scragging. When tested to a specific level of cyclic shear strain, the stiffness in the first cycle is higher than the subsequent cycles. The material rapidly reaches a steady-state and all cycles after about the third are identical. When the compounds are tested under standard test conditions, the accepted procedure is to quote only the fully-scragged properties. The properties associated with the first few cycles are generally ignored. After testing the material will revert to its unscragged state in a matter of hours or sometimes days. Thus, an isolation system that has been unshaken for several years will respond with higher initial stiffness than would be anticipated for fully-scragged properties.

The combined effects of non-linearity due to low strain level and of scragging will mean that the period of the building is much less than the period specified in the design.

The shear moduli of the various compounds were evaluated at the time of manufacture of the isolators and the values obtained are listed in Table 1. The average ratio of the unscragged 2% modulus to the scragged 50% modulus (the design level) is 6.24, and this would predict that the 1.50 second period (if the superstructure were taken to be rigid) would be reduced to a period of 0.8 second. This result agrees with the period identified from the earthquake response data.

**Sierra Point Overhead** - The elastomer used in the bearings is a lightly-filled rubber designated by the manufacturer, LTV Energy Products, as 247-55. It is a high-strength, low-damping natural rubber compound which is less nonlinear than the high-damping compounds. The scragged 2% shear modulus is about 250 psi compared to the scragged 50% modulus of about 100 psi. No information is available on the unscragged moduli for this compound.

## 4. EARTHQUAKE RESPONSE OF THE STRUCTURES

CSMIP processed data was used in the response studies of both structures. Typical CSMIP accelerogram processing involves instrument correction, baseline correction, high and lowpass (bandpass) filtering, and then numerical integration to obtain velocity and displacement. The FCLJC accelerograms were bandpass filtered with ramps at 0.3—0.6 Hz and 23—25 Hz, while the Sierra Point Overhead filter ramps were 0.15—0.30 Hz and 23—25 Hz. This means that all long-period content greater than 1.67 seconds and 3.33 seconds was removed from the FCLJC and Sierra Point Overhead records, respectively.

**FCLJC** - The peak building response and free-field accelerations are given in Table 3. Plots of the N/S foundation, basement, 2nd floor, and roof accelerations are shown in Fig. 5. Linear-elastic 5%-damped response spectra for these responses are shown in Fig. 6. The two figures indicate that the isolation system reduced the high-frequency content of the input ground motion to the superstructure. The structure response period in the N/S direction is 0.7 second from the response spectra (Fig. 6). The maximum displacements of the isolation system were 0.25 inch (2% shear strain) and 0.10 inch (0.8% shear strain) in the N/S and E/W directions, respectively. Frequency-domain analyses of the data indicate that the structure responded with a period of approximately 0.7 second in both the N/S and E/W directions. A single-degree-of-freedom analysis of the isolation system using 2% unscragged properties for the bearings gives a period of approximately 0.7 second. Information from the original design calculations and computer analyses of the structure indicate that the first translational periods of the superstructure are approximately 0.7 second in both the N/S and E/W directions. These results suggest that the FCLJC response in the Upland earthquake involved coupled response of the closely-spaced first isolated and superstructure modes. Time-history analyses of the FCLJC subjected to the foundation motions recorded in the Upland earthquake have been performed with good agreement between analytical and measured responses.

The torsional response was investigated by evaluation of the torsional displacements of the isolation system and of the superstructure above the isolators. While found not to be significant, about one-half of the total torsional displacement occurring at the roof level was due to the isolation system. Vertical deformations in the isolators on the north and south sides of the building were calculated. The deformations were found to be extremely small ( 0.02 inch) and did not cause any appreciable rocking motion in the superstructure.

**Sierra Point Overhead** - The peak structural response and free-field accelerations are given in Table 4. Plots of the longitudinal (N/S) west column above-isolator, below-isolator (top of column), base of column, and free-field accelerations are shown in Fig. 7. The above-isolator peak acceleration (264.5 gals) is slightly less than that below the isolator (304.5 gals) and some reduction of high-frequency content is evident. Linear-elastic 5%-damped acceleration response spectra for these responses (excluding the free-field) are shown in Fig. 8. For the high-frequency peak at about 0.1 second there is some reduction in the spectral ordinate from below the isolator to above the isolator, but for periods greater than about 0.2 second the accelerations above the isolator are equal to or greater than those below it.

The maximum displacements in the isolation system were 0.20 inch (3.6% shear strain) and 0.12 inch (2% shear strain) in the longitudinal (N/S) and transverse (E/W) directions, respectively. These maxima both occurred in the west column isolator. Calculation of the force-deformation relationship for the west column isolator indicates that this level of deformation corresponds approximately to the point of first yield of the lead plug. The force-deformation relationships for all of the six different types of bearings were calculated and the composite system stiffness at pre-yield deformation levels determined. The isolated period was calculated to be approximately 0.75 second. It was not possible to identify a structural period from frequency-domain analyses or response spectra, and thus this calculated period could not be confirmed. However, because of the lack of clearance at the abutments (a nominal 1-inch gap filled with expansion joint material) the deck is not free to move as if it were properly isolated.

As put forward in [4] the 3 to 4-fold increase in superstructure accelerations over those in the footing indicates that either

(a) the composite backfill-abutment-superstructure system is almost rigid and responded to a spike in the site response spectrum (which is not evident in the spectra), or

(b) the conglomeration of loosely-connected superstructure members (the longitudinal stringers that span between the bent caps are fixed at one end and sit on a rocker bearing with additional restrainer-bar seismic restraint at the other) responded dynamically by moving with respect to each other and transmitted frequencies throughout the steel superstructure which were recorded by the instruments. If this were the case, then the accelerations recorded would not be representative of the inertial forces subjected to the columns.

Because of the restricted clearance and restraint existing at the abutments and the segmental nature of the bridge superstructure, it is not possible to make a more detailed interpretation of the records.

## 5. CONCLUSIONS

It is clear that both of these base-isolated structures did not act under recent earthquake loading in the manner intended for a base-isolated structure. In the case of the FCLJC the response is satisfactorily explained by the fact that the effective isolation period at the level of deformation induced in the elastomer is about the same as that of the fixed-base superstructure. Isolation as a concept is based on there being a large separation between the isolation period and the fixed-base structure period. The advantages of isolation derive from the low participation factors of the higher modes. The small amplification in accelerations seen in the FCLJC is a consequence of the fact that the building did not fulfill this requirement at that level of input. The degree of attenuation of the free-field motion under the building was also surprising but due to the proximity of the epicenter and the depth of the earthquake the motion was most probably generated by vertically-propagating shear waves. The waves striking the free surface reflect in a different way than waves impacting the covered surface under the foundation and this rather than the degree of embedment could explain the attenuation. Although the isolators were about 8 times stiffer than assumed in design, the damping was very high assuring a substantial degree of energy absorption. The structure did perform with smaller drifts and smaller forces than would the fixed-base structure subjected to the same input motion.

In the case of the Sierra Point Overhead the fact that the abutments prevented movement explains the unusual response. This, combined with the high stiffness of the bearings at low levels of strain combine to prevent isolation action.

The conclusion to be drawn from this study is that over-conservatism in the design of base isolation systems is not necessarily a correct approach. Since the FCLJC building was the first base-isolated building in the United States, a great deal of conservatism was incorporated into the design. In recent years extensive tests to failure of isolators for nuclear power plant applications have shown that properly designed and manufactured isolators are capable of very large strains under load and have remarkable margins of safety beyond the design level. This implies that designers should be careful with their conservatism in designing base isolation systems. The structure should be designed on the same basis as a conventional structure with the confidence that if the design-level earthquake is exceeded, the isolators will not fail but the failure will be in the superstructure with exactly the same mechanisms as in a conventional structure. In this way, when the structure is struck by the type of moderate earthquake which is highly likely during the lifetime of the structure, its performance will include those aspects for which isolation is used, namely, reduction of acceleration in the superstructure, no participation of the higher modes and reduced input to building contents.

Analysis of the Sierra Point Overhead Loma Prieta data revealed that the bridge instrumentation layout involves unnecessary redundancy in some channels. It is recommended that the layout be evaluated and redesigned to provide a more informative response data-set for future earthquakes.

## REFERENCES

[1] Tarics, A.G., Way, D., and Kelly, J.M., "The Implementation of Base Isolation for the Foothill Communities Law and Justice Center," A Report to the National Science Foundation and the County of San Bernardino, Reid and Tarics Associates, San Francisco (November 1984).

[2] Pomeroy, E.G., "Sierra Point Overhead Seismic Isolation Retrofit," *ATC-17*, Applied Technology Council, pp. 123-132 (March 1986).

[3] Kelly, J.M. and Celebi, M., "Verification Testing of Prototype Bearings for a Base-Isolated Building," *Report No. UCB/SESM-84-01*, University of California, Berkeley (March 1984).

[4] Buckle, I.G., "Recent Progress in Base Isolation for Bridges in the United States," *Proceedings*, 6th U.S.-Japan Bridge Engineering Workshop, Lake Tahoe, Nevada (May 1990).

Table 1 Shear Moduli for FCLJC Rubber Compounds

| Compound | Shear Modulus (psi) | | | | | |
|---|---|---|---|---|---|---|
| | 2% | | 50% | | 100% | |
| | unsc. | scr. | unsc. | scr. | unscr. | scr. |
| 246-70 | 1083 | 500 | 240 | 163 | 176 | 156 |
| 243-65 | 1083 | 417 | 205 | 143 | 146 | 130 |
| 2x-69 | 617 | 402 | 150 | 114 | 111 | 100 |
| 2x-71 | 542 | 259 | 125 | 100 | 96 | 89 |

Table 2 Isolation System Stiffnesses

| | Shear Stiffness (kips/in) | | | |
|---|---|---|---|---|
| | 2% | 10% | 25% | 50% |
| unscragged | 4949 | 2288 | 1412 | 955 |
| scragged | 1700 | 1000 | 800 | 640 |

Table 3 Summary of Peak Accelerations, FCLJC

| Component | Accelerations (gals) | |
|---|---|---|
| | N/S | E/W |
| roof | 152.8 | 85.7 |
| 2nd floor | 69.4 | - |
| basement[1] | 52.6 | 37.8 |
| foundation[2] | 138.9 | 106.2 |
| free-field | 235.6 | 228.5 |

Superstructure N/S channels are at center of building
1 - above isolators
2 - below isolators

Table 4 Summary of Peak Accelerations, Sierra Pt. Overhead

| Component | Accelerations (gals) | | |
|---|---|---|---|
| | N/S | E/W | Vertical |
| above W. col isolator | 264.5 | 190.3 | 85.2 |
| below W col. isolator | 305.4 | 204.2 | - |
| above E col. isolator | 283.3 | 179.7 | - |
| below E col. isolator | 307.5 | 188.3 | - |
| base of W column | 88.0 | 44.4 | 28.9 |
| free-field | 103.6 | 56.6 | 31.2 |

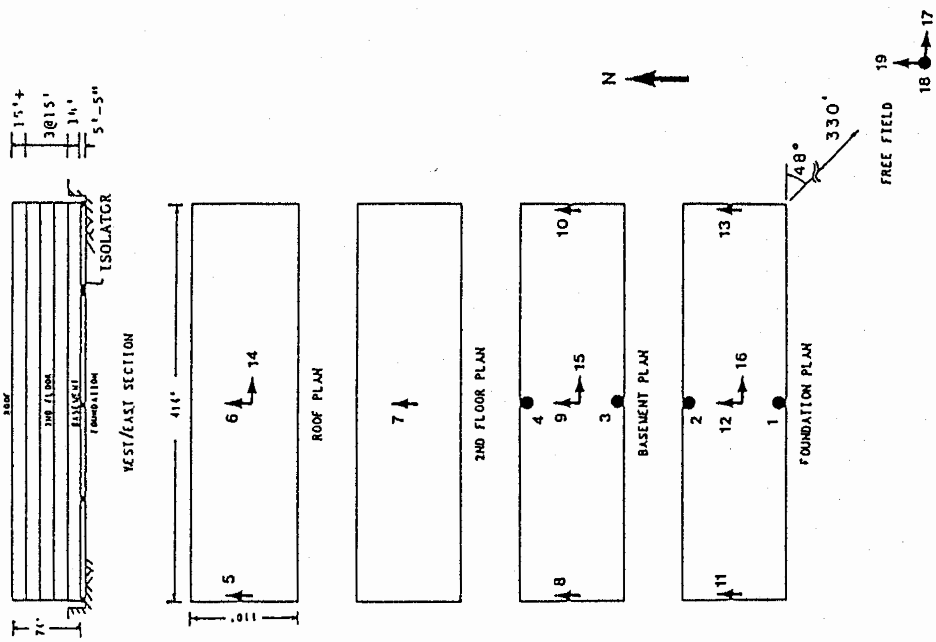


Fig. 1 FCLJC Plan and Elevation with Instrumentation Layout

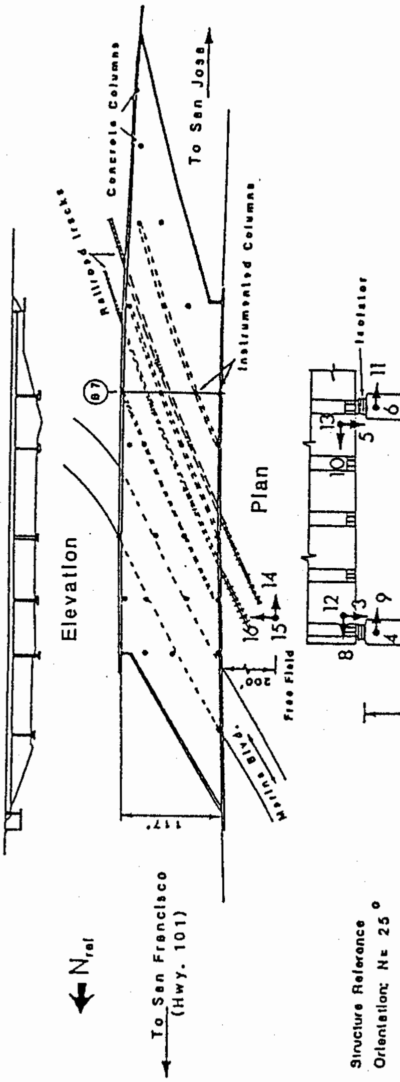


Fig. 2 Sierra Pt Overhead Plan and Elevation with Instrumentation Layout

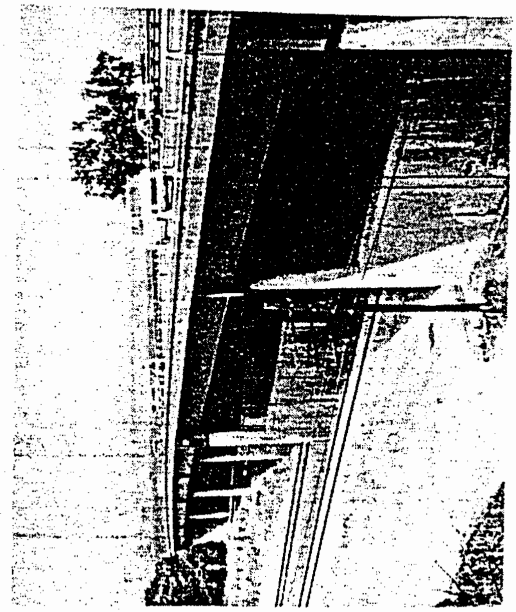

Fig. 3 General View of West Side of Sierra Pt Overhead

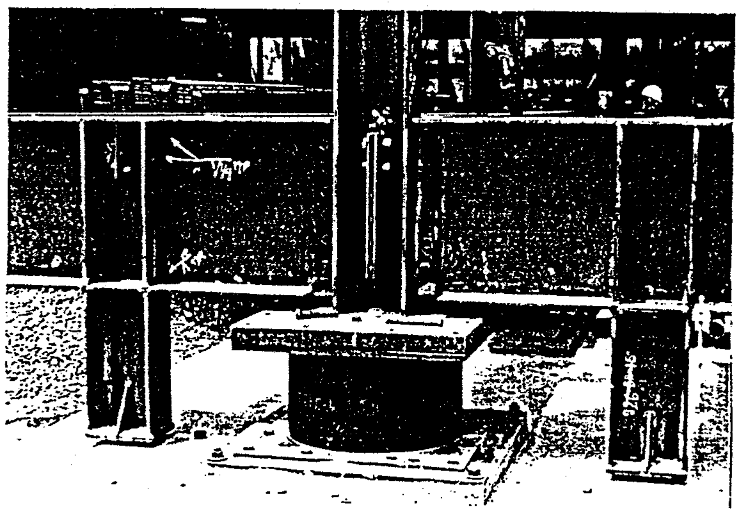

Fig. 4 FCLJC Bearing Installed During Construction

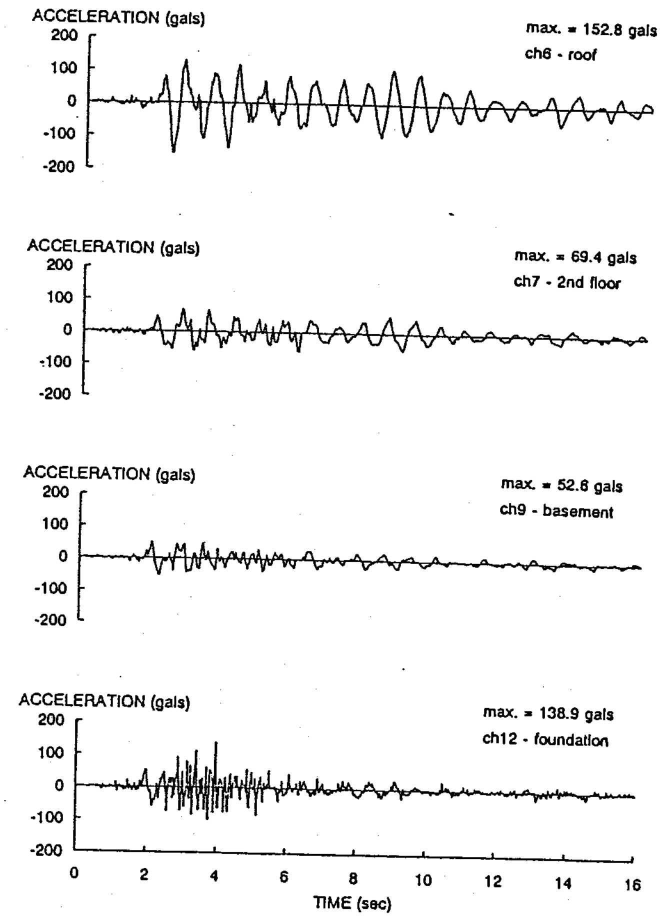


Fig. 5 FCLJC Transverse (N/S) Response Accelerations

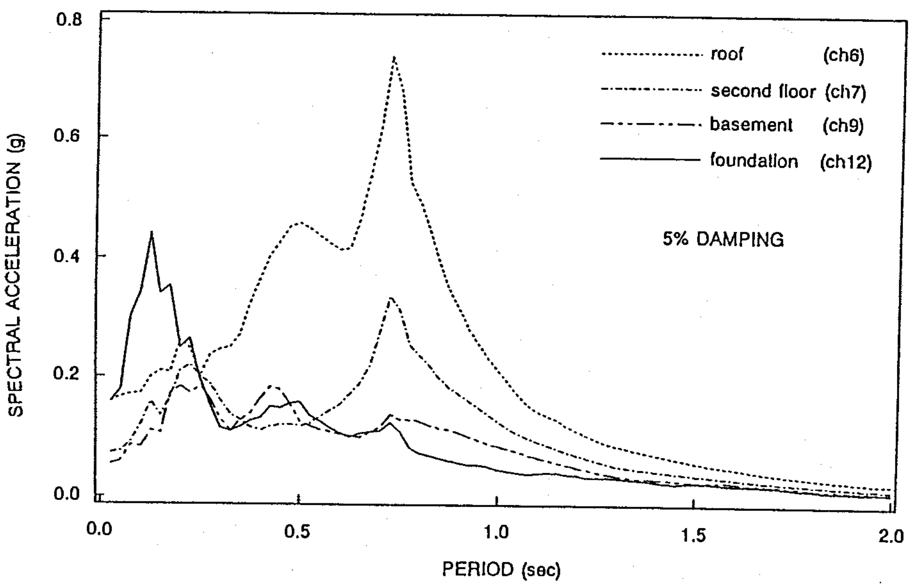


Fig. 6 FCLJC 5%-Damped Response Spectra, Transverse (N/S) Accelerations

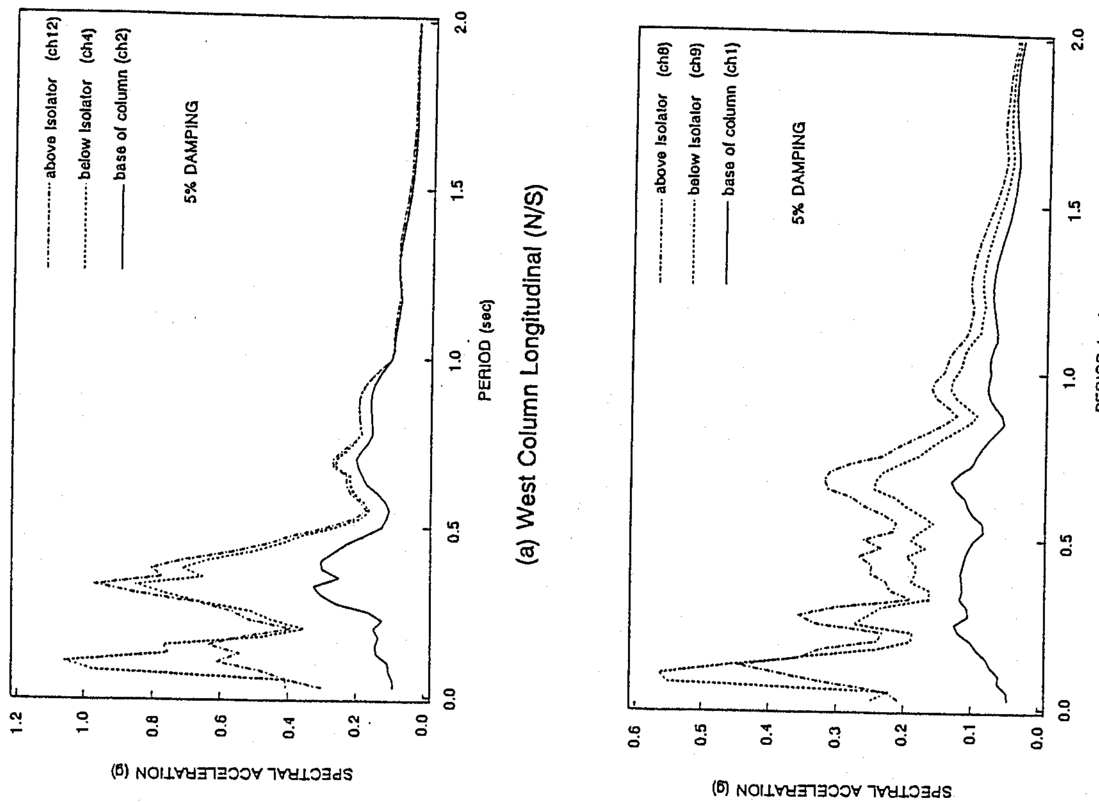


Fig. 8 Sierra Pt Overhead 5%-Damped Response Spectra

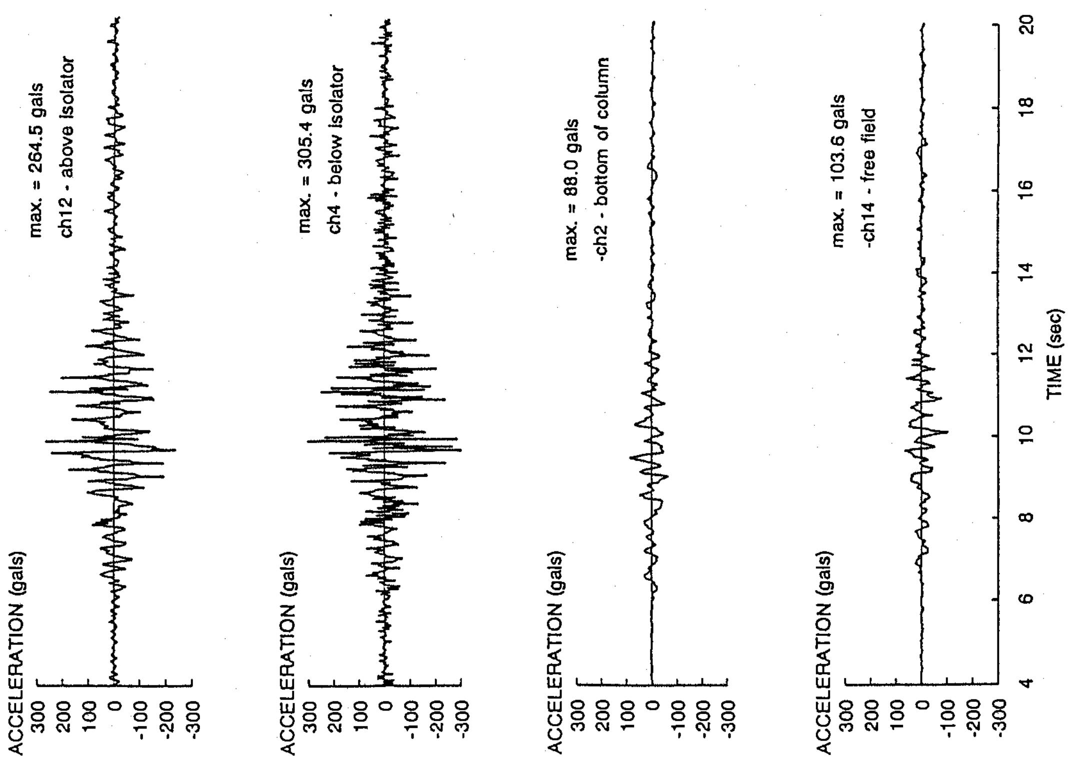


Fig. 7 Sierra Pt Overhead Longitudinal (N/S) Response Accelerations