# SEISMIC RESPONSE OF A CURVED HIGHWAY BRIDGE MODEL

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This paper reports research on a model relating to the seismic resistance of large multispan curved overcrossings. The feasibility of developing a model that satisfies the necessary similitude requirements of such a complex structure and that is also capable of being tested on the 20 by 20-ft (6.1 by 6.1-m) shaking table is outlined. The small-amplitude dynamic characteristics of the 1:30 scale model of a hypothetical prototype are examined, and for this elastic range the experimental results compared satisfactorily with those obtained analytically. The response of the microconcrete model to a progressively more intense simulated seismic excitation applied horizontally in the asymmetric direction is described. The severe damage at the expansion joints during strong excitation is outlined, and ways of reducing such damage are suggested. The influence of expansion joint design on the seismic behavior points to the need for joint restrainers, of adequate ductility, to tie adjacent girders together.

•HISTORICALLY, highway bridges have proved particularly vulnerable to the action of strong-motion earthquakes (1). Many of these structures consisted of single- or multiple-span simple trusses or girders supported on massive piers and abutments. Damage was primarily due to foundation failure and often resulted in progressive collapse.

Even the more continuous designs of some modern bridge structures may be equally vulnerable to damage in earthquakes. In long, multiple-span, reinforced concrete bridge structures, the design of the expansion joints and the long columns has a profound influence on the structural integrity of the system under dynamic loading, and, because of the nonlinear discontinuous behavior of the deck, the dynamic characteristics are very complex. In short, stiffer bridges, foundation interaction effects become increasingly dominant; therefore, their seismic response is equally complex.

After the San Fernando earthquake on February 9, 1971 (1, 2), in which the just-completed South Connector Overcrossing at the interchange of I-5 and Calif-14 suffered damage and partial collapse (Figure 1), a comprehensive, multiphase research project sponsored by the Federal Highway Administration was initiated at the Earthquake Engineering Research Center, University of California, Berkeley. The aim of the project was to investigate the effectiveness of existing bridge design methodology in providing adequate structural resistance to seismic disturbances. Interim measures to correct certain design deficiencies were quickly undertaken after the earthquake (3); however, such structures are still designed mainly by using a static seismic coefficient method similar to that formerly used (4).

Phases 1 and 2 of the project were a literature survey of seismic effects on highway bridges and an analysis of the seismic response of long, multispan highway bridges. The results of these phases have recently been published (1, 5). Phase 3, an analysis of short, stiff highway bridges, will be published shortly (6). Phase 4, which supplements the analytical investigation of phase 2, is the subject of this paper, and a detailed report on this subject is in preparation (7).

The major objectives of phase 4 were to perform detailed model experiments on a shaking table to identify and examine the parameters affecting the seismic response of large, curved multispan overcrossings so that experimental dynamic response data can be generated with which to verify the validity of response predictions.

Other objectives of phase 4 were to investigate those concepts that were difficult to model analytically and to construct a model from prefabricated components with sufficient versatility to allow a variety of tests to be undertaken with the same basic model.

One limitation of this model is that the base excitation represents rigid ground motion over the entire table; hence, there is no means of simulating the spatial effects of ground motion.

## FEASIBILITY STUDY

Initially the feasibility of undertaking model studies of the complete South Connector Overcrossing on the shaking table at Berkeley was examined. This structure (Figure 2) is typical of modern large curved highway bridges and, because of its partial collapse, has been the subject of considerable interest following the San Fernando earthquake. Essentially, it was four curved reinforced concrete or prestressed concrete box girders with a total span of 1,349 ft (411 m), separated by expansion joints, and supported on a single line of massive columns varying in height from 15 to 140 ft (4.6 to 43 m). Table 1 gives the first seven natural frequencies of the prototype for two expansion joint conditions: zero friction in the axial direction and fully locked joint.

The 20 by 20-ft (6.1 by 6.1-m) shaking table made from reinforced and posttensioned concrete weighs 100 kips (45 Mg) and is driven by hydraulic actuators in the vertical direction and one in the horizontal direction (8, 9). The acceleration response spectra for harmonic motion of the unloaded table are shown in Figure 3. The typical operating frequency range is 0 to 10 Hz. For relatively light test structures of less than 10 kips (4540 kg), these response values are reduced very little.

Because of the large geometric and time scale factors that would be required to model the complex dynamic behavior of this particular overcrossing, modeling the complete structure on the shaking table was not considered feasible. However, an analytical study of the natural frequencies and mode shapes of the prototype (5) suggested that the west half of the bridge, which was supported on short columns, contributed very little to the dynamic response of the complete bridge. Accordingly, a smaller hypothetical prototype that incorporated the important features of the east half was modeled. Hence, this structure, which terminated in an abutment at each end, included the expansion joints, the deck curvature, and the long columns. It was designed in such a way that the lower mode shapes and frequencies were similar to those of the prototype bridge. Moreover, because scaling effects in the dynamic response of small concrete models are unknown, the model was considered primarily as a small structure, and its behavior was correlated with analytical studies. For this reason, the system was idealized further by making it symmetrical and by making the columns of equal length (Figure 4). Ideally the model should be subjected to a realistic seismic excitation; and, because of limitations in the performance of the shaking table, a frequency ratio of about 1 should be adopted. Unfortunately in this nonlinear study, scale effects made this impracticable.

Another major problem in modeling a system of this kind is satisfying the requirements of bringing all inertial forces, including gravitational forces, into the same force scale. Given the complexity of the structure and the nonlinear response characteristics due to sliding, impacting, cracking, and yielding, a true-scale weight-distorted model made in prototype material was considered the best compromise solution.

The model adopted had a geometric scale of $L_r = 1/30$. To maintain a force ratio of $F_r = L_r^2$ required that a substantial amount of lead weight be added. The resulting time ratio was $T_r = \sqrt{L_r} = 1/5.5$; the acceleration ratio between model and prototype was then $a_r = 1$. This model closely resembled the central portion of the prototype bridge (piers 2 through 6). Analytical studies confirmed that this model adequately reproduced the lower natural frequencies and mode shapes of a scaled version of the prototype.

Table 1 gives frequencies computed by the linear dynamic analysis program BSAP (5) for the following systems: the prototype of zero friction in the axial direction and infinite friction, the prototype for these two values scaled to a frequency ratio of 5.5, the model as designed, and the model with revised boundary and expansion joint conditions. The differences between the last two sets of values are discussed later.

**Figure 1. South Connector Overcrossing after San Fernando earthquake.**

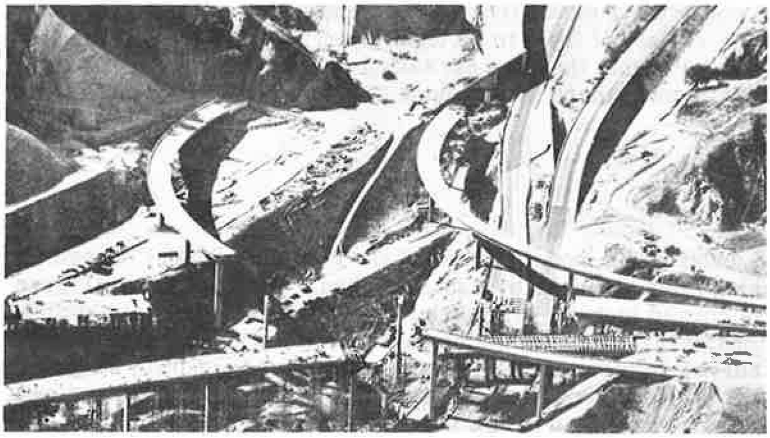


**Figure 2. Structural system of South Connector Overcrossing.**

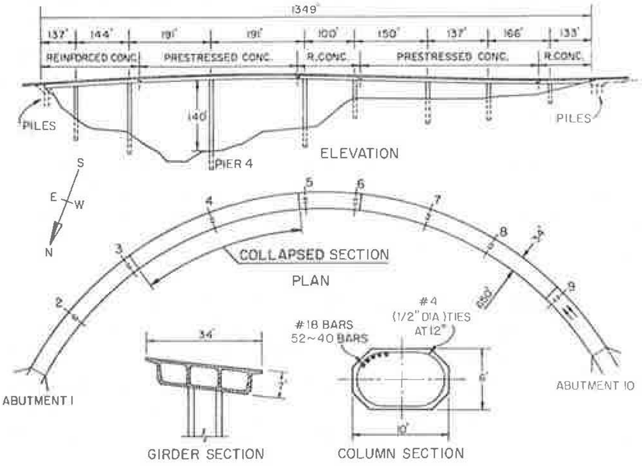



**Table 1. Computed natural frequencies (in hertz) at joint friction of zero and infinity.**

| | Prototype | | Scaled Prototype | | Model | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | As Designed | | | |
| | | | | | 0 | | | |
| Mode | 0 | ∞ | 0 | ∞ | Roller[a] | Pinning[b] | ∞ | As Built |
| 1 | 0.20 | 0.72 | 1.1 | 4.0 | 2.0 | 2.0 | 3.04 | 5.0 |
| 2 | 0.36 | 0.99 | 2.0 | 5.5 | 2.48 | 3.0 | 6.95 | 5.7 |
| 3 | 0.50 | 1.45 | 2.8 | 8.0 | 2.49 | 3.2 | 7.0 | 7.7 |
| 4 | 0.89 | 1.54 | 4.9 | 8.5 | 3.0 | 7.0 | 8.9 | 8.5 |
| 5 | 1.26 | 1.80 | 6.9 | 9.9 | 3.26 | 7.02 | 9.3 | 8.6 |
| 6 | 1.44 | 2.15 | | | | | | 11.1 |
| 7 | 1.45 | 2.25 | | | | | | 14.3 |

[a]Roller at abutments. [b]Pinned at abutments.

The earthquake excitation applied to the model was scaled to a frequency ratio $f_r$ = 5.5. This required that the high-frequency component be filtered out, but the shaking table response spectra indicated that sufficient capability was available to ensure that all significant vibration modes of the structure could be adequately excited.

## MODEL DESIGN AND CONSTRUCTION

In view of the time and expense of fabricating such a complex model in concrete and the desirability of gaining as much experimental data as possible from the same basic model, the model was assembled from replaceable components: deck, expansion joint, and column components. In this way damage could be isolated, and the damaged components could be either repaired or replaced. A view of the completed model on the shaking table is shown in Figure 5.

### Superstructure

Except in the vicinity of the expansion joint where impact and large shear forces make damage unavoidable, it was desirable that the superstructure behave elastically for the large number of tests planned. To achieve this the section was overdesigned by a factor of approximately 2; the aim was to maintain steel stresses below 20 ksi (138 MPa). Although the prototype would normally be a multicell box girder section, a rectangular solid section with similarly scaled bending and torsional stiffnesses was adopted to avoid the difficulties associated with constructing a box girder on such a small scale. The dynamic response should not be sensitive to this difference.

The reinforcement and formwork are shown in Figure 6. The concrete used was a high-strength (8,000 psi or 55 MPa), shrinkage-resistant microconcrete to eliminate shrinkage cracking, which would adversely affect the damping characteristics of the model. The 10 by 3 by 2-in.-wide (254 by 76 by 51-mm) lead weights were attached by means of two bolts running through the deck to increase the weight of the superstructure to the required scale value. In this way the added weight contributed very little to deck stiffness, and extraneous damping was minimized.

### Columns

The prototype column reinforcement is shown in Figure 2. If correctly modeled the test column should contain between 40 and 52 deformed bars of $^1/_{16}$-in. (1.5-mm) diameter, which should have the same stress-strain characteristics as the No. 18 (57-mm) prototype rebars. In an effort to accomplish this, suitable wires were deformed by both indentation and protrusion and then were subjected to a range of annealing conditions, but the resulting stress-strain properties were not completely satisfactory. To simplify matters, a column section of equivalent strength was designed by using four No. 2 (6-mm) rebars with a yield strength of 50 ksi (345 MPa). The steel was butt welded to the column end plates, and extra shear keys were provided to prevent premature failure by shear in these regions.

The columns were made by using a 4,500-psi (31-MPa) normal portland cement microconcrete. It has essentially the same stress-strain characteristics as plain prototype concrete except for the more gradual falling branch, which more closely resembles confined prototype concrete and probably reflects the greater tensile strength associated with microconcrete.

Inelastic cyclic bending tests of the column elements (Figure 7) confirm the validity of using this procedure to simulate the nonlinearity of well-designed prototype reinforced concrete columns. Reasonably stable behavior with acceptable stiffness deterioration is evident at a displacement ductility factor of 3, which could have been increased by more closely spaced stirrups. As designed, the lower 9 in. (229 mm) of the model columns had 16 gauge mild steel stirrups at $^1/_2$-in. (12.7-mm) centers.

Figure 3. Shaking table performance limits.

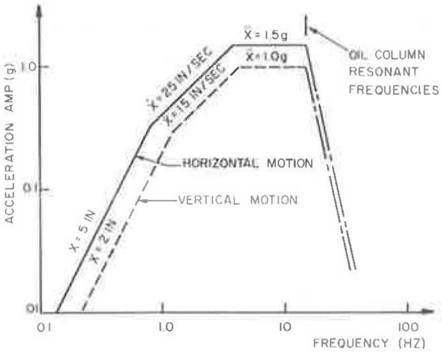



Figure 4. Schematic of test model.

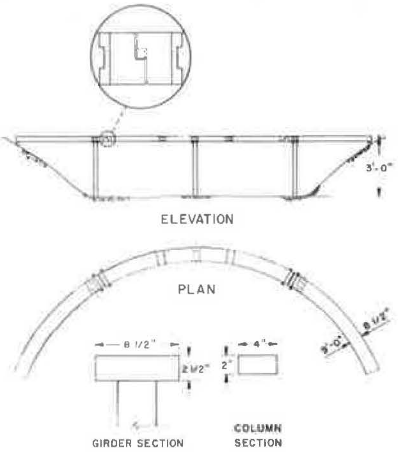



Figure 5. Completed model ready for testing.

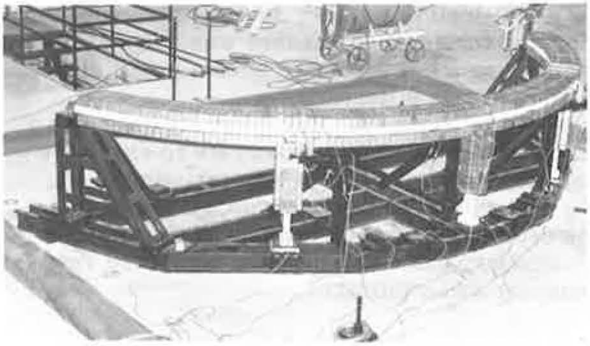
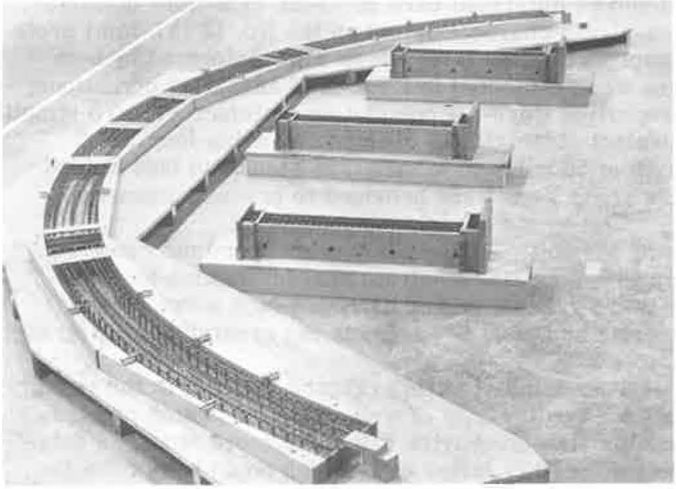


Figure 6. Model before casting.



When the bridge system was assembled, the deck girders were clamped to the column heads, the close-fitting machined connection detail simulating a rigid joint.

### Joint Restrainers

For the first test series the girders were tied together at the expansion joints by means of $5\frac{1}{2}$-in.-long by $\frac{3}{32}$-in.-diameter (140 by 2.4-mm) mild steel joint restrainers mounted on each side and parallel to the bridge axis. These restrainers had respective yield and ultimate strengths of approximately 420 and 475 lb (1870 and 2115 N) and yield and ultimate strain limits of 0.2 and 20 percent. This represents large ductile restraint.

## DISCUSSION OF TEST RESULTS

### Small-Amplitude Tests

Before the completed model was subjected to simulated seismic excitation, the dynamic characteristics of the model were studied under small-amplitude vibration. Mode shapes, frequencies, and damping ratios were ascertained from accelerometer measurements during free vibration.

The first mode (Figure 8), which is essentially longitudinal motion of the central girder-pier subsystem combined with antisymmetric motion in the horizontal plane at 5 Hz, was easily excited by hand. However, the second mode, which is essentially symmetric rigid body motion of the superstructure in the horizontal plane at 6.7 Hz, was difficult to excite manually and invariably reverted to the first mode, even in those cases where the symmetry of the mode was enforced by snapping tensioned cables attached to three points.

Based on free vibration decay tests of these two modes, a viscous damping of approximately 3 percent critical was obtained. Given the complexity of the system with the expansion joints and the large amount of added weight, this value was acceptable.

Knocking the head of the central column sharply by hand in the transverse direction excited mode 7, a symmetrical bending mode of the central girder in the horizontal plane at a frequency of 15 Hz (Figure 9). It was, in fact, excited more readily than mode 2.

Another symmetrical mode (mode 5), predominantly bending of the deck in the vertical plane (Figure 9) with a frequency of 11 Hz and a damping ratio of 2 percent critical, was excited by knocking the deck in the vertical direction. No effort was made to obtain the antisymmetric vertical bending modes inasmuch as these would not be excited with rigid base excitation.

Forced harmonic motion techniques in which the shaking table was used to provide a base motion were also applied to determine the small displacement dynamic response characteristics of the model. For the very small amplitude required, adequate control of the table is difficult to maintain in such a test, and this method is not considered very suitable for small models. Ideally, to determine mode shapes and frequencies for such a complex structure requires use of a multiple-channel shaker system in which several shakers can be placed on the structure and adjusted to produce predominant response in any natural mode.

When the model was assembled, the stiffness characteristics of the free-standing weighted columns after they had been securely attached to the base structure were checked. Results from free vibration tests in both principal directions on all three columns indicated that the measured stiffness was consistently 25 percent of that calculated by assuming full fixity at the base. This was confirmed by static load-deflection tests on the columns. Inasmuch as the calculated value of EI was substantiated by results from load-deflection tests on identical beam specimens, it was concluded that, in spite of the stiffness of the base system, base flexibility had a large effect and consequently would have to be taken into account for a more appropriate analysis. However, for the special purpose program, a reduced value of EI, which is more convenient and

Figure 7. Inelastic cyclic bending behavior of column element.

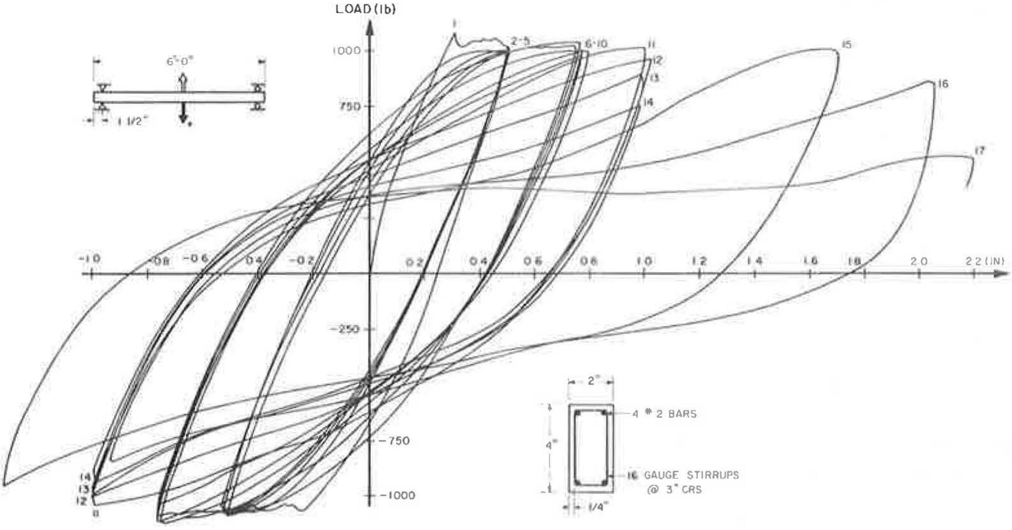



Figure 8. Fundamental mode shape of test structure.

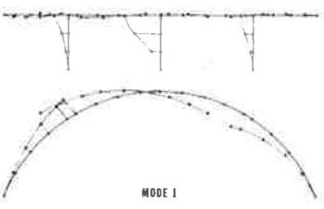
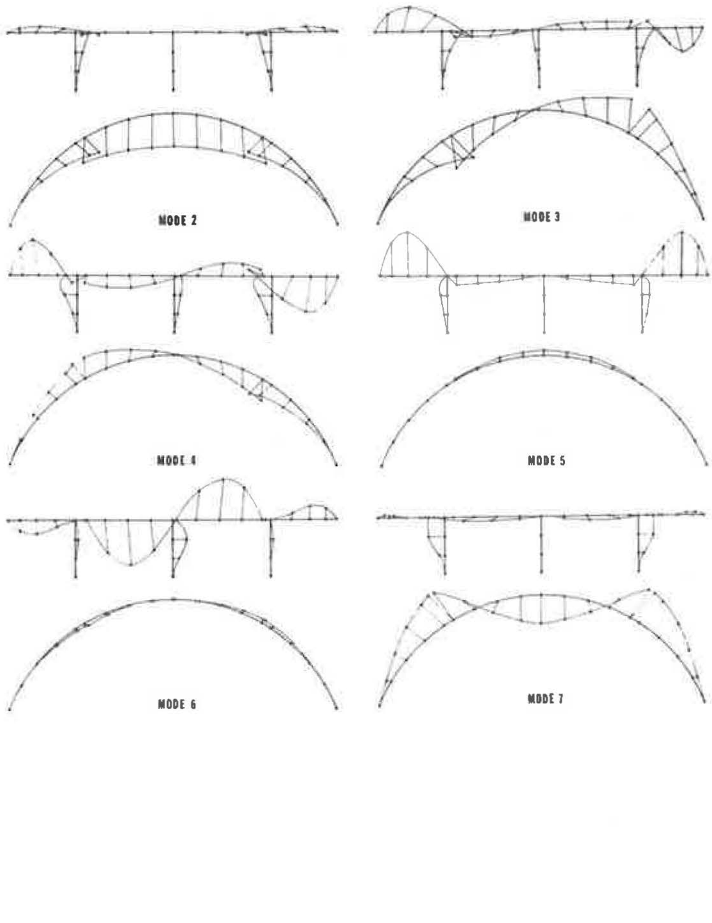



Figure 9. Characteristic shapes of modes 2 through 7.



has essentially the same effect in the significant modes, was used.

## Linear Dynamic Analyses

This information was used for the lumped parameter system and finite element idealization of the model shown in Figure 10; then, linear dynamic analyses were undertaken by using the computer program BSAP to determine the theoretical dynamic characteristics of the test structure.

When the experimental values were correlated with those calculated from the linear analysis, the importance of the spring stiffness provided by the elastomeric bearing pad at the expansion joint became evident. Initially only two joint conditions were provided for: zero friction in the axial direction and finite friction. During preliminary small-amplitude testing it was observed that, although motion took place at the expansion joints, sliding did not occur inasmuch as shear strain deformations in the elastomeric bearing pad at the hinge allowed relative rotation and translation to take place. The longitudinal shear and torsional stiffnesses of the pad were determined, and appropriate springs were incorporated into the mathematical model to allow for this behavior.

With this revision, model analyses were performed, and the frequencies (Table 1) and mode shapes (Figures 8 and 9) agreed closely with the experimental results.

## Seismic Tests

The model was subsequently subjected to progressively more intense simulated seismic excitation applied horizontally in the asymmetric X-direction. The command excitation was artificially generated motion, the same as that used for postearthquake studies of the Olive View Hospital (10). It consists of a 2-sec parabolic buildup to 8 sec of strong motion, followed by exponential decay during the next 5 sec. Maximum spectral acceleration values occur for frequencies of 2.5 Hz. This motion was scaled in time to give 3 sec of excitation, and the actual table motion shown in Figure 11 is effectively filtered at 14 Hz by the hydraulic system. The real time-scaled accelerogram includes high-frequency components up to about 25 Hz, but based on response characteristics this difference should be insignificant.

During each test, data in the form of digitized linear variable differential transformer (LVDT) output were collected at 100 samples per second per channel on the data acquisition system (9). The LVDTs were mounted on a very stiff instrumentation framework so that the X- and Y-components of the relative global displacements near the points corresponding to nodes 9, 14, and 23 of the mathematical idealization (Figure 10) could be measured. The relative movement on each side of both expansion joints was similarly measured.

The sequence of tests showing peak base accelerations is given in Table 2. Typical response maxima for the X- and Y-components at the top of column 1 (node 9) and the X-component at the top of column 2 (node 14) are also included.

The response was essentially oscillatory motion in the first mode (Figure 8). Even in the final test where peak table accelerations reached 0.87 *g* this is still true, as shown by the response histories in Figure 11, although the response frequency was reduced to approximately 3.5 Hz.

In this test series the expansion joint suffered severe damage in the form of impact spalling of adjacent surfaces, major cracking of the hinge seat, and failure of the shear key (Figure 12). The joint restrainers had displacement ductility factor requirements of approximately 20.

The only damage to the columns was minor flexural cracking at the bases. Major damage was confined to the expansion joint zone, and although the joint restrainers were subjected to heavy ductility demands no catastrophic collapse occurred.

Figure 10. Lumped parameter analytical model.

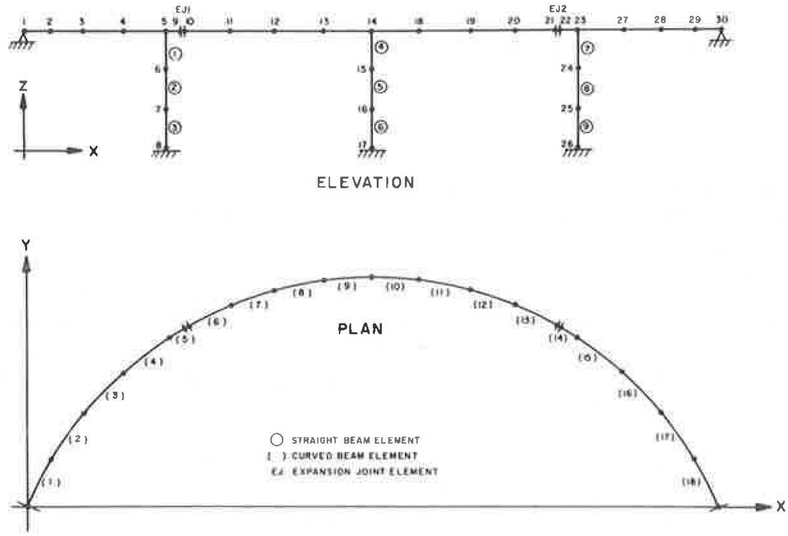



Figure 11. Response histories for final test.

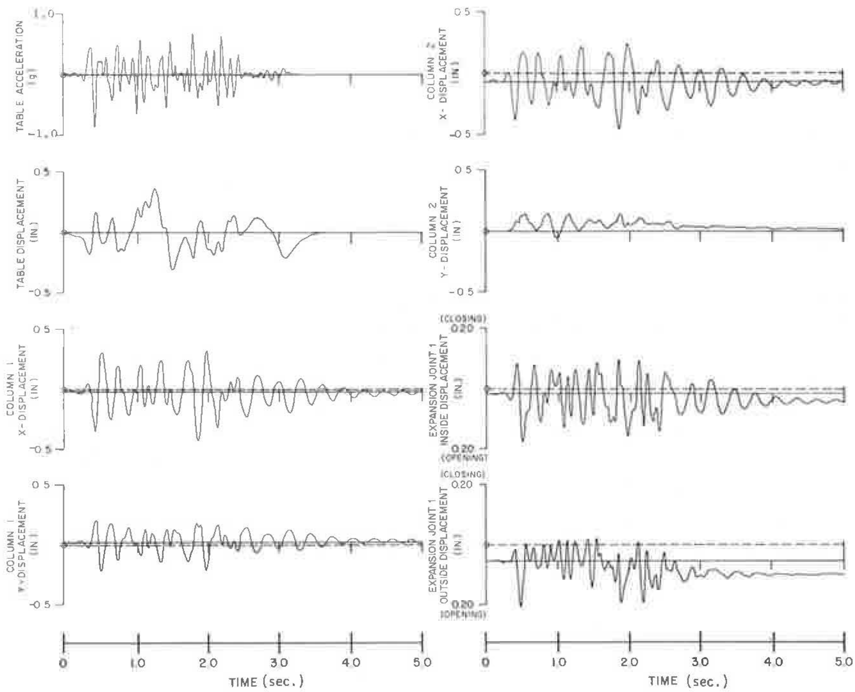



Table 2. Response maxima.

| Test Number | Peak Table Acceleration (g) | Displacement (in.) | | |
|---|---|---|---|---|
| | | Column 1 | | Column 2, |
| | | X-Component | Y-Component | X-Component |
| 1 | -0.06 | 0.03 | 0.02 | 0.03 |
| 2 | -0.13 | 0.11 | 0.08 | 0.11 |
| 3 | -0.23 | 0.17 | 0.12 | 0.18 |
| 4 | -0.38 | 0.23 | 0.16 | 0.23 |
| 5 | -0.60 | 0.37 | 0.20 | 0.32 |
| 6 | -0.75 | 0.40 | 0.19 | 0.29 |
| 7 | -0.87 | 0.42 | 0.21 | 0.31 |

Note: 1 in. = 25.4 mm.

Figure 12. Damaged expansion joint.

## CONCLUSIONS

The results of the first series of tests indicated that, if adjacent girders are effectively tied together at the expansion joint with due regard to ductility demand, a bridge of this type should be capable of withstanding large seismic disturbances without major inelastic drift and without total collapse. However, inasmuch as the major damage in the bridge consistently occurred at the expansion joints because of impact and the high values of vertical and horizontal shears and torsion in this vicinity, special attention must be paid to the detailed design of the deck in this location. Because dynamic analyses of three-dimensional nonlinear systems are extremely difficult, further response data of both a qualitative and quantitative nature are essential to a better understanding of the dynamics of such complex structures and to the development and verification of theoretical analyses.

Continuing experimental work planned for this model includes the following series of tests.

1. The repaired model will be subjected to the simulated seismic excitation applied horizontally in the symmetric direction, both alone and also with simultaneous vertical excitation.
2. A parameter study of expansion joint design will be undertaken. Models incorporating hinge restrainers of various strength and expansion joints with collapsible sacrificial buffers will be tested.
3. The model will be retested in a four-column design, with the central girder supported on two columns.

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