MULTIDIRECTIONAL SEISMIC RESPONSE OF A CURVED HIGHWAY BRIDGE MODEL

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ABSTRACT

A continuing experimental model study relating to the seismic resistance of large multi-span curved overcrossings is reported.

A brief description of the microconcrete model, a 1/30 true-scale version of a hypothetical prototype, which was tested on the 20 ft x 20 ft (6.1 m) Shaking Table at the University of California, is given. The response of the model is described for a series of simulated seismic excitations applied

(i) horizontally in the asymmetric longitudinal direction, and
(ii) horizontally in the symmetric direction, both alone and also with simultaneous vertical excitation.

The influence of expansion joint design on the seismic behaviour is the main parameter studied, with emphasis on the need for joint restrainers of adequate ductility, to effectively tie adjacent girders together. However, because of the severe damage caused by impacting at the expansion joints during moderately strong excitation, it is concluded that expansion joints should ideally be omitted from such structures.

INTRODUCTION

A study of seismic damage has shown that bridge structures have proved particularly vulnerable to the action of strong motion earthquakes(1). However, unlike the numerous older structures consisting of single or multiple simple truss or girder spans supported on massive piers and abutments in which extensive damage was generally caused by foundation failure and often resulted in progressive collapse, certain types of modern bridge structures may also be quite susceptible to damage from structural dynamic effects.

Following the San Fernando earthquake of February 9, 1971(1,2) in which the then just completed South Connector Overcrossing at the Golden State-Antelope Valley 5/14 freeway interchange suffered damage including partial collapse (Fig. 1), a comprehensive multi-phase research project sponsored by the U.S. Department of Transportation, Federal Highway Administration, was initiated at the Earthquake Engineering Research Center, University of California, Berkeley, with the aim of investigating the effectiveness of existing bridge design methodology in providing adequate structural resistance to seismic disturbances. Although interim measures to correct certain design deficiencies were quickly enacted after the earthquake(3), such structures are still largely designed using a static seismic coefficient method similar to that formerly used and often still adopted for building design(4).

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Due to the nonlinear, discontinuous behaviour of the expansion joints the dynamic response characteristics of long multiple-span reinforced concrete bridge structures are very complex and unique, deserving full and separate investigation by analytical and experimental means.

In the short stiffer bridges, foundation interaction effects become increasingly dominant, and this causes their seismic response to be equally complex.

Final reports covering Phases I and II have recently been published(1,5). The first stage of Phase III and Phase IV, the subject of this paper, are completed and will be reported in detail at a later date (6,7).

The major objectives of Phase IV were to perform detailed model experiments on a shaking table in order to identify and examine the important parameters affecting the seismic response of large curved multi-span overcrossings, to generate experimental dynamic response data, and to use that data to verify the validity of theoretical response predictions. In particular it was intended that concepts which would be difficult to model analytically should be investigated experimentally and that the model should be constructed as an assemblage of prefabricated components with sufficient versatility to allow a variety of tests to be undertaken with the same basic model.

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

Results from the first phase of testing, together with a description of model design and the overall experimental program, have recently been reported[8]. This paper reports on further tests aimed at providing data for response in the prime symmetric mode and determining the effect of simultaneous vertical and horizontal excitations. The design parameters studied were concerned with the detailing at the expansion joints and included the effects of hinge restrainers of varying strength and the inclusion of collapsible expendable buffers.

MODEL ANALYSIS

Considerations involved in the development of a model to satisfy the necessary similitude requirements of such a complex structure, and also capable of being tested on the 20 ft x 20 ft (6.1 m) Shake Table [9,10] at the University of California were described in reference 8.

The selected prototype was the South Connector Overcrossing of Figs. 1 and 3. This structure is typical of modern large curved highway bridges and because of its partial collapse it has been the subject of considerable interest following the San Fernando earthquake. It was essentially five curved reinforced concrete box girders with a total span of 1349 ft (410 m) separated by expansion joints, and supported on a single line of massive columns varying in height from 15 ft (4.5 m) to 140 ft (43 m).

The model as adopted (Fig. 2) was a 1/30 true-scale weight-distorted model of a simplified version of the prototype made in prototype material. A schematic is shown in Fig. 4 and its resemblance to the central portion of the prototype bridge (piers 2 through 6) is apparent. Reproducing the self weight effects of the prototype necessitated increasing the self weight of the model by a factor of 30. This was accomplished by using lead weights and the resulting model ratios were

time ratio $T_r = L_r^{1/2} = 1/5.5$ and
acceleration ratio $a_r = 1$.

Analytical studies confirmed that this model adequately reproduced the lower natural frequencies and mode shapes of a scaled version of the original bridge.

Table 1 gives the first seven natural frequencies as computed by the linear dynamic analysis program BSAP[5] for the following systems: The South Connector Overcrossing for the two joint conditions of zero friction in the axial direction and infinite friction, and these values scaled to a frequency ratio of 5.5; the model as built.

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

EXPERIMENTAL DETAILS

Model Description:

The model (Fig. 2) was made as an assemblage of replaceable components; deck, expansion joint, and column components. This facilitated gaining as much experimental data as possible from the same basic model.

The microconcrete superstructure is designed to remain elastic and has the correctly scaled torsional and bending stiffnesses, although for simplicity it has been constructed as a rectangular solid section instead of the normal multi-cell box girder.

The columns also have the correctly scaled stiffnesses and, further, inelastic cyclic bending tests of column elements confirm that the nonlinearity of well designed prototype reinforced concrete columns is adequately simulated.

In the original expansion joint (Fig. 4) the 1/2 in. (13 mm) long hinge seat was heavily reinforced for shear and moment using 7 1/8 in. (3.2 mm) diameter bent bars at 1 1/4 in. (32 mm) centres. A 1/16 in. (1.6 mm) thick rubber pad extends over the entire hinge seat simulating the normal elastomeric bearing pads. The shear key was 2 in. (51 mm) long.

Following analysis of the results from the first series of testing, modifications were made to the design of the expansion joint components with the aim of increasing the shear and moment capacity of the hinge seat and increasing the shear capacity of the shear key. Otherwise the model was essentially the same for all series of tests.

Joint Restrainers:

The girders were tied together at the expansion joints by means of 5 1/2 in. (140 mm) long joint restrainers mounted on each side of the deck and parallel to the bridge axis (Fig. 7). Two types were constructed by annealing 3/32 in. (2.4 mm) and 1/8 in. (3.2 mm) diameter steel rods at approximately 1000°F (550°C) for two hours. Properties of the restrainers are given in Table 2. The original Type A restrainer of 420 lb (190 kg) yield strength was considered to represent large ductile restraint. In later tests heavier and equally ductile Type B joint restrainers of 650 lb (294 kg) yield strength were used. In prototype terms Type B represents a 14 ft (4.3 m) long restrainer with a yield strength of 600 kip (270×10^3 kg) i.e. 4 in. (100 mm) diameter at 50 ksi (345 N/mm^2), and capable of plastic extension for almost 3 ft (1 m) before fracture.

Test Procedure:

After determining the small amplitude dynamic characteristics of the bridge model and satisfactorily correlating them with the analytically determined values for this elastic range[8], the model was subjected to a series of simulated seismic motions.

The command excitations were independently generated by a filtered white noise process[11] in which the filter frequency for the horizontal and vertical components was chosen as 2.5 Hz and 4 Hz respectively and the damping 60% of critical, typifying the usual California earthquake records including San Fernando, 1971. The filtered signals were multiplied by an intensity function of time consisting of a 2-second parabolic build-up to 8 seconds of strong motion followed by exponential decay over the next 5 seconds. The resulting simulated ground motion records were considered appropriate for the site of the South Connector Overcrossing. The horizontal component was the same as that used for post-earthquake studies of the Olive View Hospital(12). The vertical component was scaled so that peak accelerations were approximately 60% of those for the horizontal motion. These motions were scaled in time to give 3 seconds of excitation (Figs. 8 and 13) and the actual horizontal and vertical table motions were effectively filtered at 14 Hz and 18 Hz respectively by the hydraulic system. The real time-scaled accelerograms included high frequency components up to about 25 Hz but judging by response characteristics this difference should be insignificant.

During each test, data in the form of digitized LVDT (displacement transducer) output was collected at 100 samples per second per channel on the data acquisition system (10). The LVDTs were mounted on a very stiff instrumentation framework so as to measure the X and Y components of the relative global displacements near the points corresponding to nodes 9, 14 and 22 of the mathematical idealization (Fig. 5). The relative movement on each side of both expansion joints was similarly measured.

The sequence of tests is given in Table 3. In a typical designation, X1A,

X refers to the direction of horizontal excitation,
1 refers to the sequential test number,
A refers to the joint restrainer type.

The first series of tests, X1A through X7A, was confined to excitation applied horizontally in the longitudinal X direction.

The second series, Y1A through Y5B, with the strengthened expansion joint and a new set of identical columns was concerned with symmetrically directed excitation. The model was subjected to the progressively more intense sequence of excitations shown in Table 3. After severe damage at test Y5A the expansion joints were repaired and the Type A restrainers were replaced with the heavier Type B restrainers and the testing program proceeded with tests Y1B through Y3B. At this stage the joint restrainers were replaced with new Type B restrainers and anchored to have a zero tie gap for the next test, Y4B. For the final test, Y5B, the elastomeric bearing pads on the vertical surfaces of the expansion joints were removed. Also 1 1/8" x 1" x 0.15" (28 mm x 25 mm x 3.8 mm) thick Hexcel honeycomb pads precrushed to a crushing load of 650 lb (294 kg) were placed on the upper vertical faces of each joint at the inside and outside edges (Fig. 14). The anchor blocks were also readjusted to give an initial tie gap of 0.03 in. (0.8 mm).

DISCUSSION OF RESULTS

1. Seismic Tests:

In the first series of tests, concerned with horizontal excitation in the X direction, the response was essentially oscillatory motion in the first mode. This fundamental mode, which is basically longitudinal motion of the central girder/pier subsystem combined with antisymmetric motion in the horizontal plane (Fig. 6), has a response frequency of 5 Hz for small amplitude free vibrations although this was reduced to approximately 3.5 Hz for test X7A where peak table accelerations reached 0.87 g.

Typical response maxima for the X-component displacement at the top of column 2 (node 14, Fig. 5) and the inside opening of expansion joint 2 are given in Table 3 together with the peak base accelerations of the progressively more intense excitations.

Associated with the increasing response and expansion joint opening was the progressive damage suffered by the expansion joints, the first signs of which appeared in test X5A. This test series culminated with severe expansion joint damage in the form of impact spalling of adjacent surfaces, major cracking of the hinge seat and failure of the shear key (Fig. 7). At this stage the joint restrainers had yield displacements of up to 0.3 in. (8 mm) representing ductility factor requirements of up to 30. However, the only damage to the columns was minor flexural cracking at the bases. Response histories for longitudinal motion of the central column and relative movement of expansion joint 1 at the outside edge during test X7A are given in Fig. 8. Major damage was confined to the expansion joint zone and although the joint restrainers were subjected to heavy ductility demands no catastrophic collapse occurred or appeared likely.

Based on these tests alone it appeared that provided adjacent girders were effectively tied together, with due regard to ductility demand and design of the expansion joint details, long high curved bridge structures should be capable of withstanding large seismic disturbances without major inelastic drift and without total collapse. However, subsequent testing in Series II and III have revealed the structure's greater vulnerability to excitation in the symmetric Y direction.

The horizontal base motions directed along the symmetric axis of the bridge primarily excite mode 2 which is essentially symmetric rigid body motion of the superstructure in the horizontal plane (Fig. 9). The vertically directed motions cause response in mode 5, predominantly symmetric bending of the deck in the vertical plane (Fig. 10). Typical response maxima for the Y component displacement at the top of column 2 and the outside opening of expansion joint 2 are shown in Table 3.

Comparing responses from tests Y1A and Y2B indicates that the vertical component has very little effect for low levels of excitation. For stronger excitation, tests Y3A and Y4A, response was greater in the case of simultaneous horizontal and vertical motions although this may be due to progressive stiffness degradation of the overall model. However at this stage no signs of damage were apparent.

In test Y5A with peak accelerations less than 0.5 g the model underwent large inelastic lurches and, unexpectedly, the expansion joint suffered extensive damage. Despite the increased shear and bending strength of the hinge seat and increased shear key strength, multiple impacting in both torsional and translational modes caused extensive damage (Fig. 11). The gap opening of 0.41 in. (11 mm) indicates that the supported girder was practically separated from the hinge seat and this was confirmed by a film of the test which showed the model to be on the verge of collapse. The lateral offset displacement of 0.3 in. (8 mm) was due to the failure of the shear key.

Despite the large global response, the only other damage was very minor flexural cracking sustained at the column bases, and then predominantly the central column (Fig. 12).

In series III with the repaired expansion joint and heavier Type B restrainers, these same trends were apparent. As expected, the stiffer restrainers were effective in reducing overall response for moderate excitations as shown by comparing results from tests Y3A and Y2B and also Y4A and Y3B. But again, at the stronger excitation with peak accelerations about 0.5 g, the expansion joint suffered extensive damage in the form of multiple lateral impacting on the shear key and torsional impacting causing failure of the hinge seat (Fig. 14). Response histories for the lateral motion of the central column during test Y4B and the relative movement at the outside edge of expansion joint 1 during tests Y4B and Y5B are given in Fig. 13. This was for the model in a pseudo virgin state having new joint restrainers fitted with a zero tie gap. Average joint gap opening was of the order of 0.3 in. (8 mm). This is equivalent to an extension of approximately 10 in. (254 mm) when translated into prototype terms and could easily cause collapse for the 15 in. (380 mm) long hinge seat after allowance for a typical thermal movement rating of 2 in. (5 mm) has been made.

As shown by the results of test Y5B (Fig. 13), the inclusion of collapsible energy absorbing buffers in the joint to soften longitudinal impact had little beneficial effect for this symmetric mode of response.

2. Dynamic Analyses:

As part of the process of development of theoretical analyses, dynamic analyses of this complex 3-dimensional nonlinear structure are being undertaken and the results compared with those obtained experimentally. Problems with modelling concepts, quantifying certain model parameters and numerical procedures are all encounted and it is relevant to note that rarely have nonlinear dynamic analyses of complex systems been subjected to the close scrutiny offered by this complementary experimental study.

3. General:

From the above discussion it is clear that expansion joints create design difficulties particularly in curved bridges. An obvious remedy would be the elimination of these discontinuities altogether and it is possible that high curved overcrossings could be designed so that thermal deformations would be absorbed by flexing in the long columns without detrimental consequences. It is recommended that this possibility be further investigated.

In California it is common for expansion joints to be provided every 300 to 400 ft whereas current European practice often dictates continuous lengths several times that.

If expansion joints prove necessary it is suggested that particular attention should be paid to the following:

i) the dynamic transverse shear forces that the shear keys have to sustain are very large and likely to damage the key unless adequately designed.

ii) the longitudinal impact forces at the expansion joints can be very large and measures to reduce them should be considered.

iii) some form of tie is necessary at the joint and preferably these should be very ductile energy absorbing restrainers.

iv) the combined effects of vertical shear and dynamic torsional impacting produce severe vertical forces on the supporting ledge at both edges of the deck. This may be the most critical design problem as it can cause shear failure of the ledge.

CONCLUDING REMARKS

The dynamics of large curved bridge structures are very complex. In particular, the discontinuities provided by the expansion joints create a very undesirable situation - both physically and analytically - being very prone to damage caused by multiple impacting in both torsional and translational modes of response.

It is apparent that the structure is more susceptible to damage when responding in the prime symmetric mode i.e. when the excitation is orientated principally in the translational direction. For curved bridge structures torsional deformations are typically more pronounced in this mode of response. Although strong ductile joint restrainers proved very effective in maintaining structural integrity by preventing excessive separation at the expansion joints in the case of the longitudinal asymmetric mode of vibration, their effectiveness in providing adequate structural resistance to seismic disturbances which excite the prime symmetric mode of response was not so pronounced. In this latter case torsional impacting is the predominant damaging influence. The use of heavy ductile restrainers is not sufficient to prevent the localized damage at the expansion joints as the damaging level of excitation is more dependent on the shear and bending strength of the hinge seat and shear key strength.

The difficulty of designing the expansion joint zone to withstand the damaging impact forces is such that the possibility of eliminating expansion joints altogether should be seriously investigated for certain types of bridges.

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Table 1. Computed Natural Frequencies (Hz)

	Prototype		Prototype to 1/30 Linear Scale		Model
Mode \ Joint Friction	0	∞	0	∞	As Built
1	0.20	0.72	1.1	4.0	5.0
2	0.36	0.99	2.0	5.5	5.7
3	0.50	1.45	2.8	8.0	7.7
4	0.89	1.54	4.9	8.5	8.5
5	1.26	1.80	6.9	9.9	8.6
6	1.44	2.15	7.9	11.8	11.1
7	1.45	2.25	8.0	12.4	14.3

Table 2. Joint Restrainer Properties

Type	Diameter (in.)	Strengths (lb)		Deformation (in.) [Strain (%)] Limits		
		Yield	Ultimate	Yield	Yield Plateau	Ultimate
A	3/32	420	475	0.01 [.2]	0.2 [4]	1 [20]
B	1/8	650	750	0.01 [.2]	0.2 [4]	1 [20]

1 in. = 25.4 mm
1 lb = 0.45 kg

Table 3. Response Maxima

Test No.		Peak Table Acceleration (g)		Displacement (in.)	
		Horizontal	Vertical	Column 2	Expansion Joint
Series I		$\ddot{X}$	$\ddot{Z}$	X-Component	EJ 2 (Inside)
X1A	Longitudinal	-.06		.03	-.01
X2A		-.13		.11	-.03
X3A		-.23		.18	-.06
X4A		-.38		.23	.07
X5A		-.60		.32	.22
X6A		-.75		.29	.25
X7A		-.87		.31	.30
Series II		$\ddot{Y}$		Y-Component	EJ 2 (Outside)
Y1A	Symmetric	.17		-.10	-.06
Y2A		.16	.09	-.09	-.05
Y3A		.32		.20	.15
Y4A		.33	.18	.26	.25
Y5A		.48		.41	.41
Series III					EJ 1 (Outside)
Y1B		.18		-.10	-.04
Y2B		.35		.18	.07
Y3B		.34	.19	.23	.13
Y4B		.54		.35	.25
Y5B		.51		.46	.31

(1 in.= 25.4 mm)

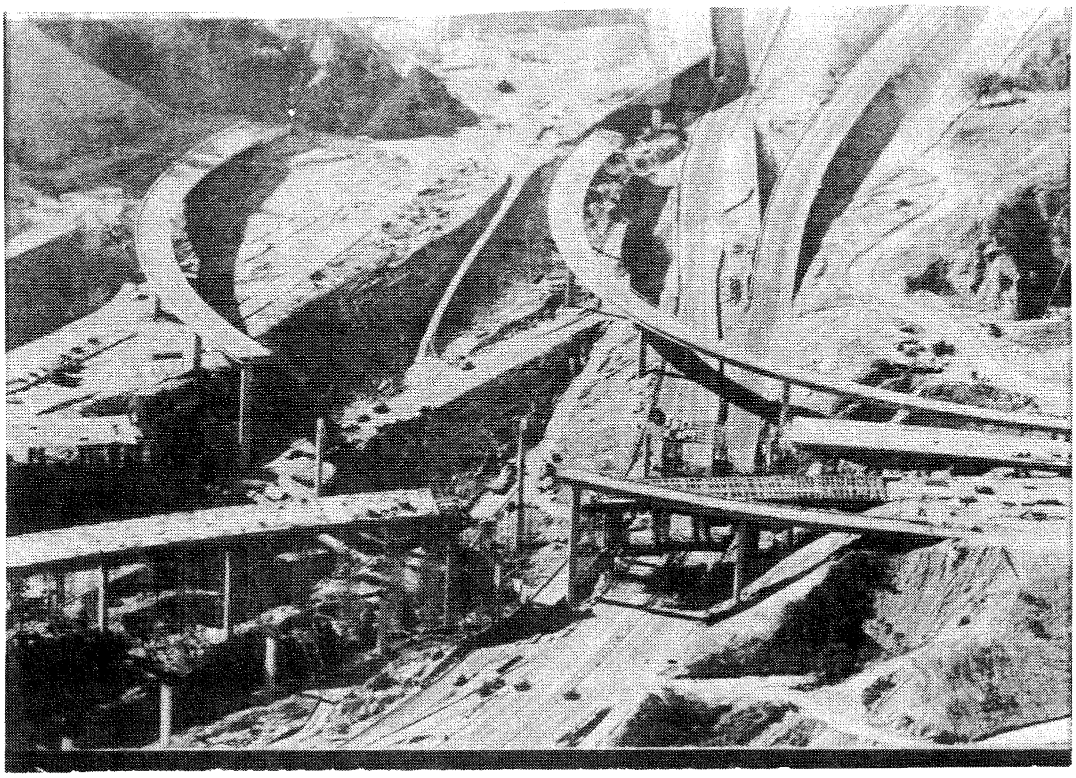

FIGURE 1: POST-EARTHQUAKE VIEW LOOKING WEST OF THE GOLDEN STATE-ANTELOPE VALLEY FREEWAY INTERCHANGE SHOWING PARTIAL COLLAPSE OF THE SOUTH CONNECTOR OVERCROSSING (RALPH SAMUELS PHOTO).

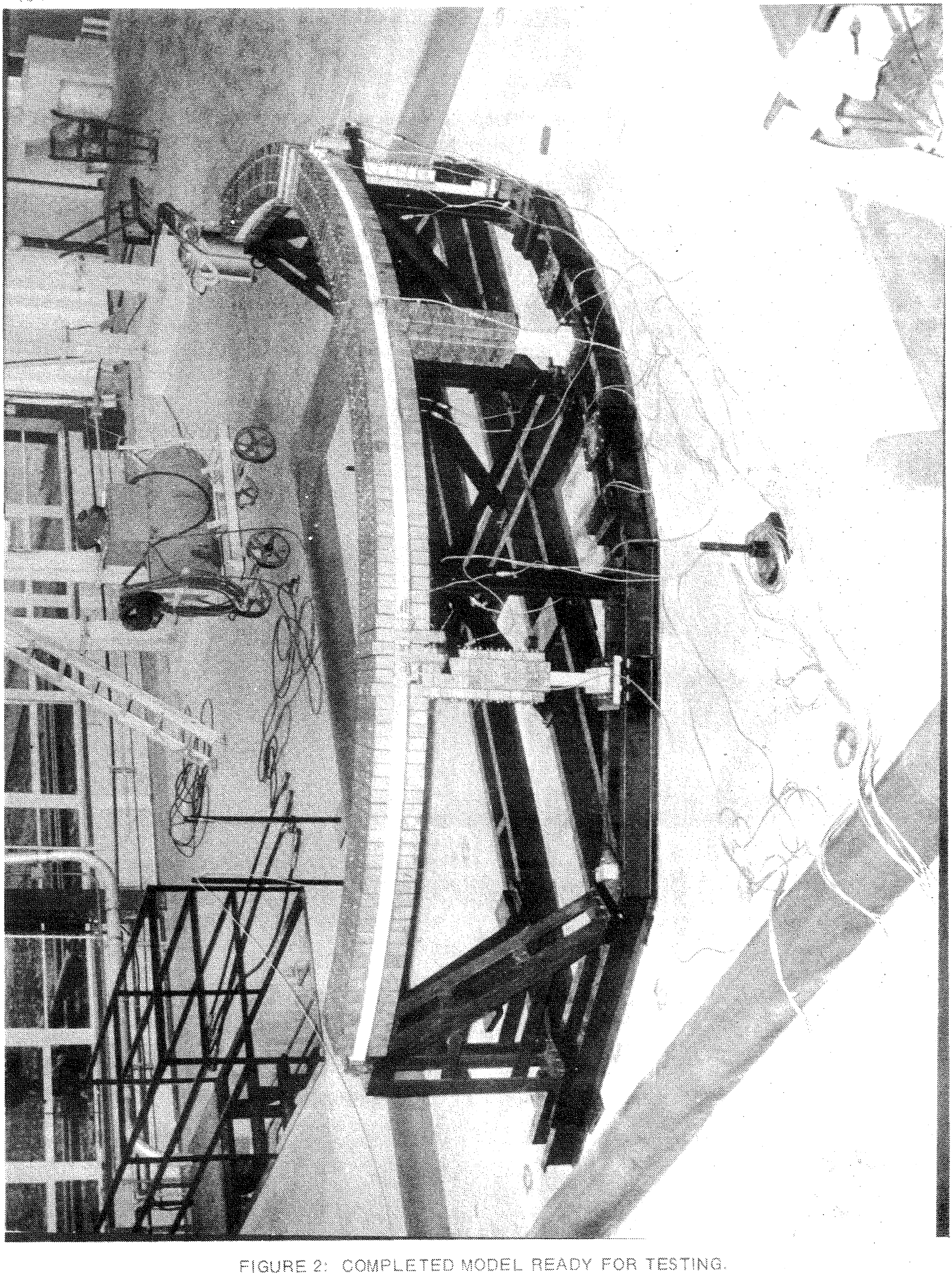

FIGURE 2: COMPLETED MODEL READY FOR TESTING.

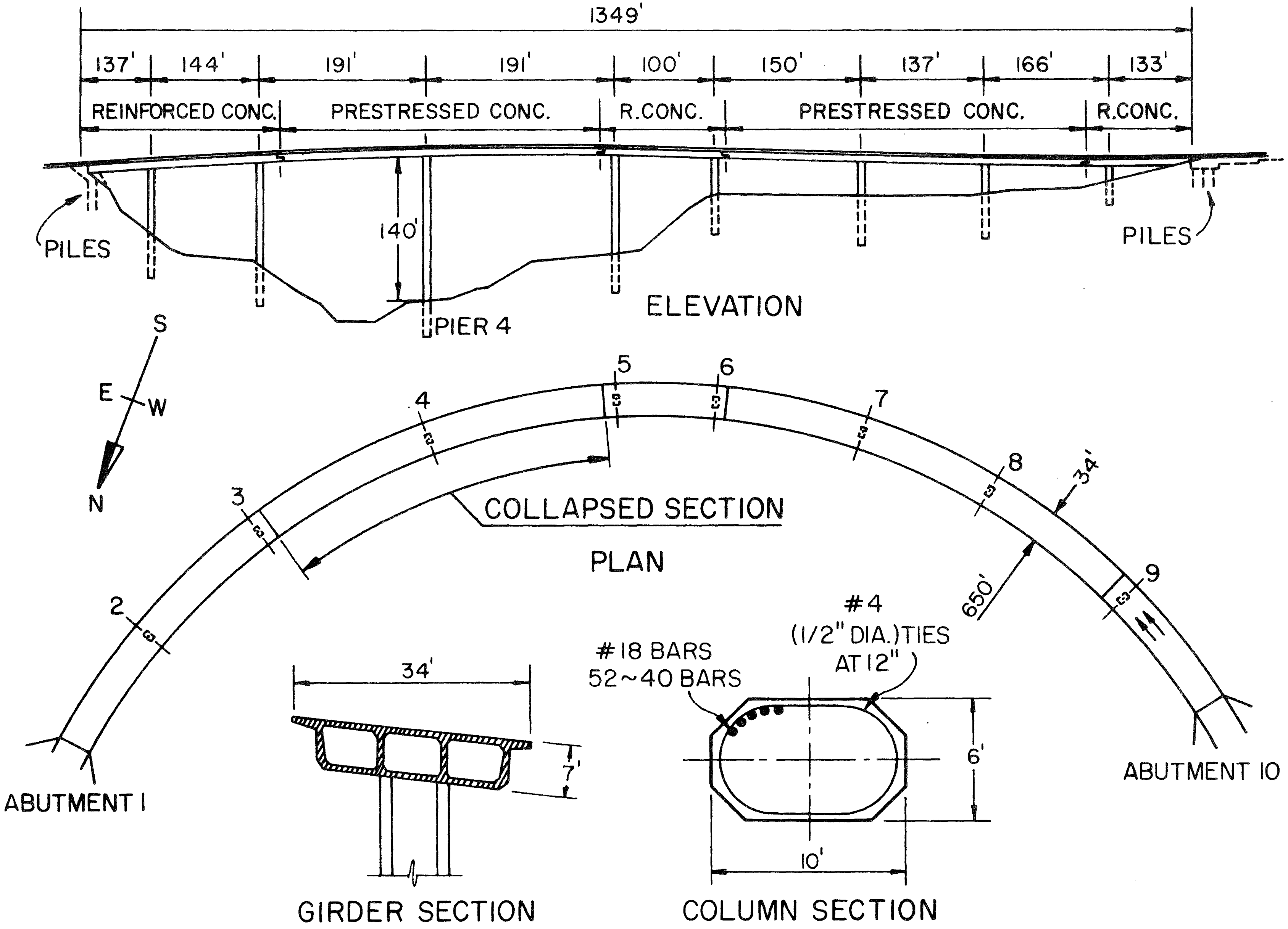

FIGURE 3: STRUCTURAL SYSTEM OF THE SOUTH CONNECTOR OVERCROSSING.

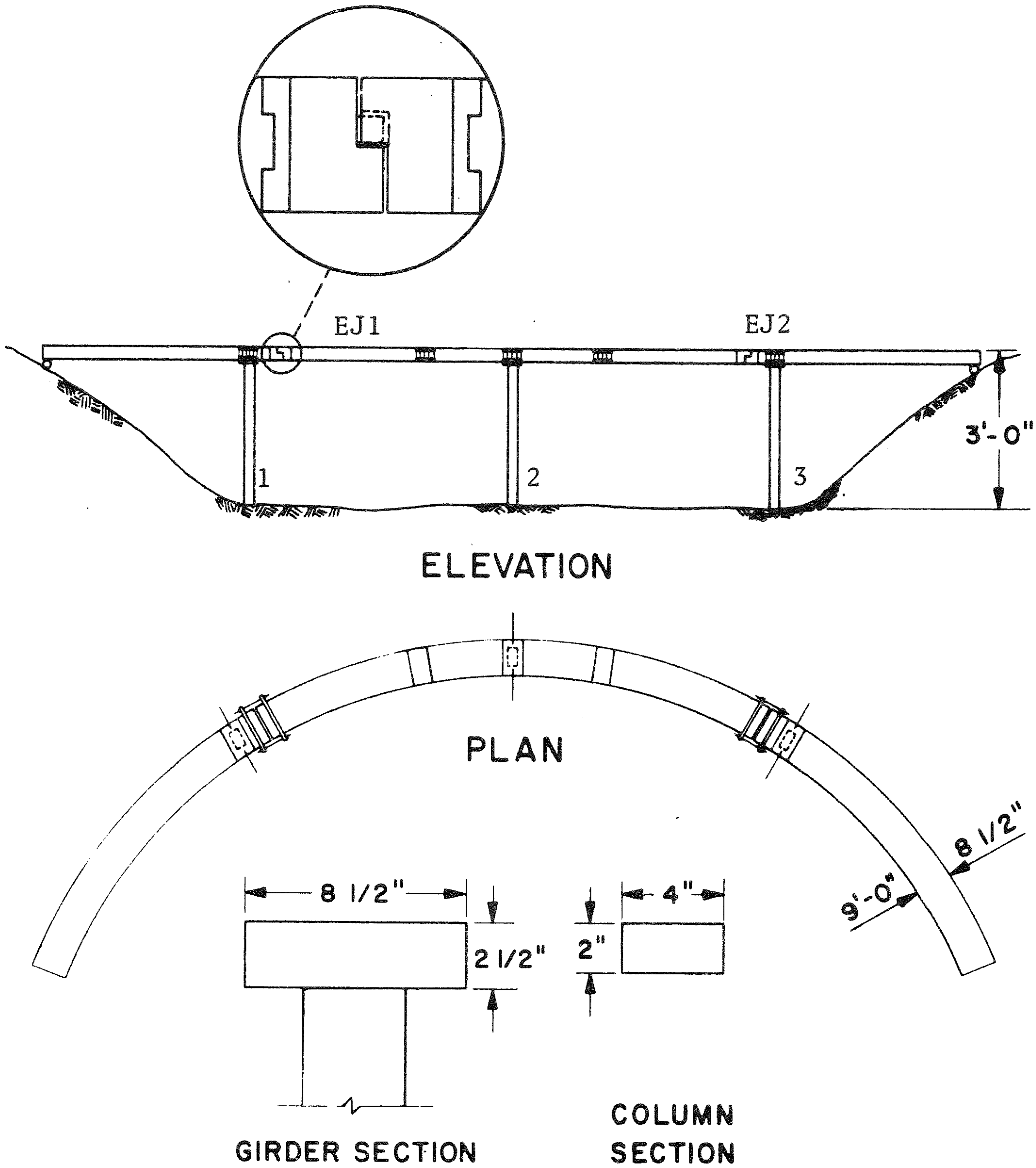

FIGURE 4: SCHEMATIC OF TEST MODEL.

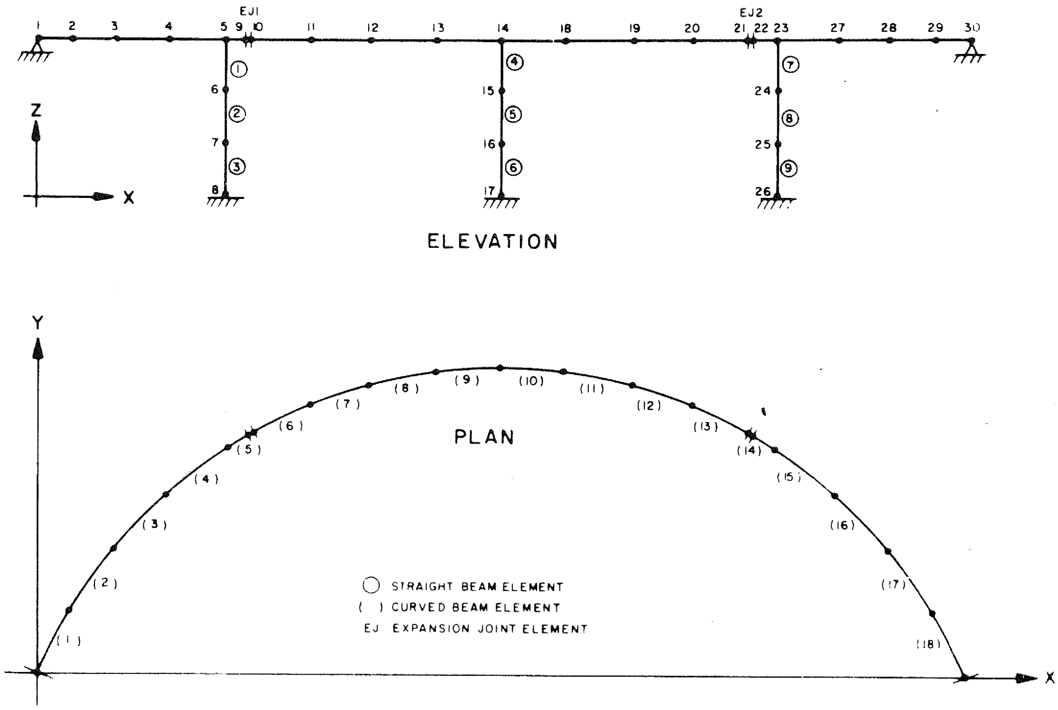

FIGURE 5: LUMPED PARAMETER ANALYTICAL MODEL.

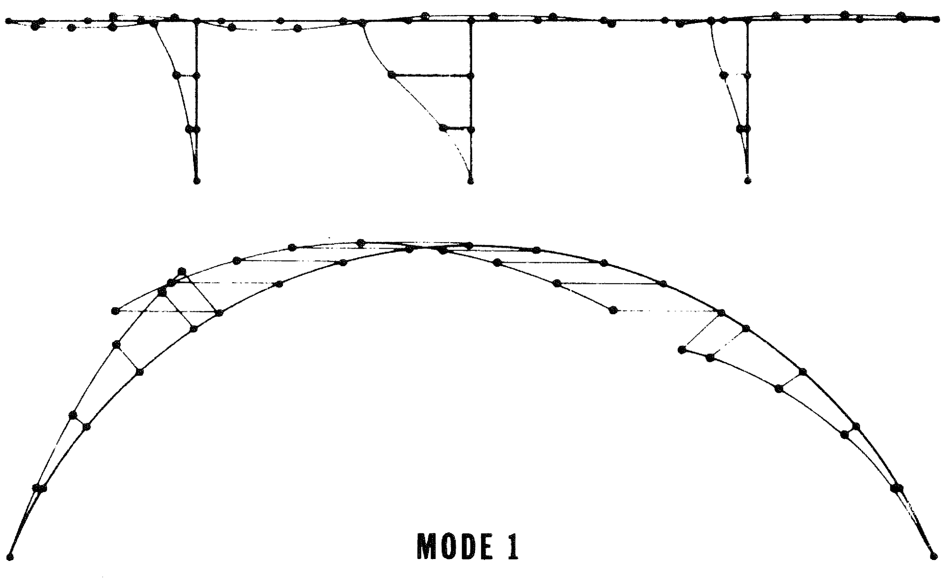

FIGURE 6: FUNDAMENTAL MODE SHAPE OF THE TEST STRUCTURE

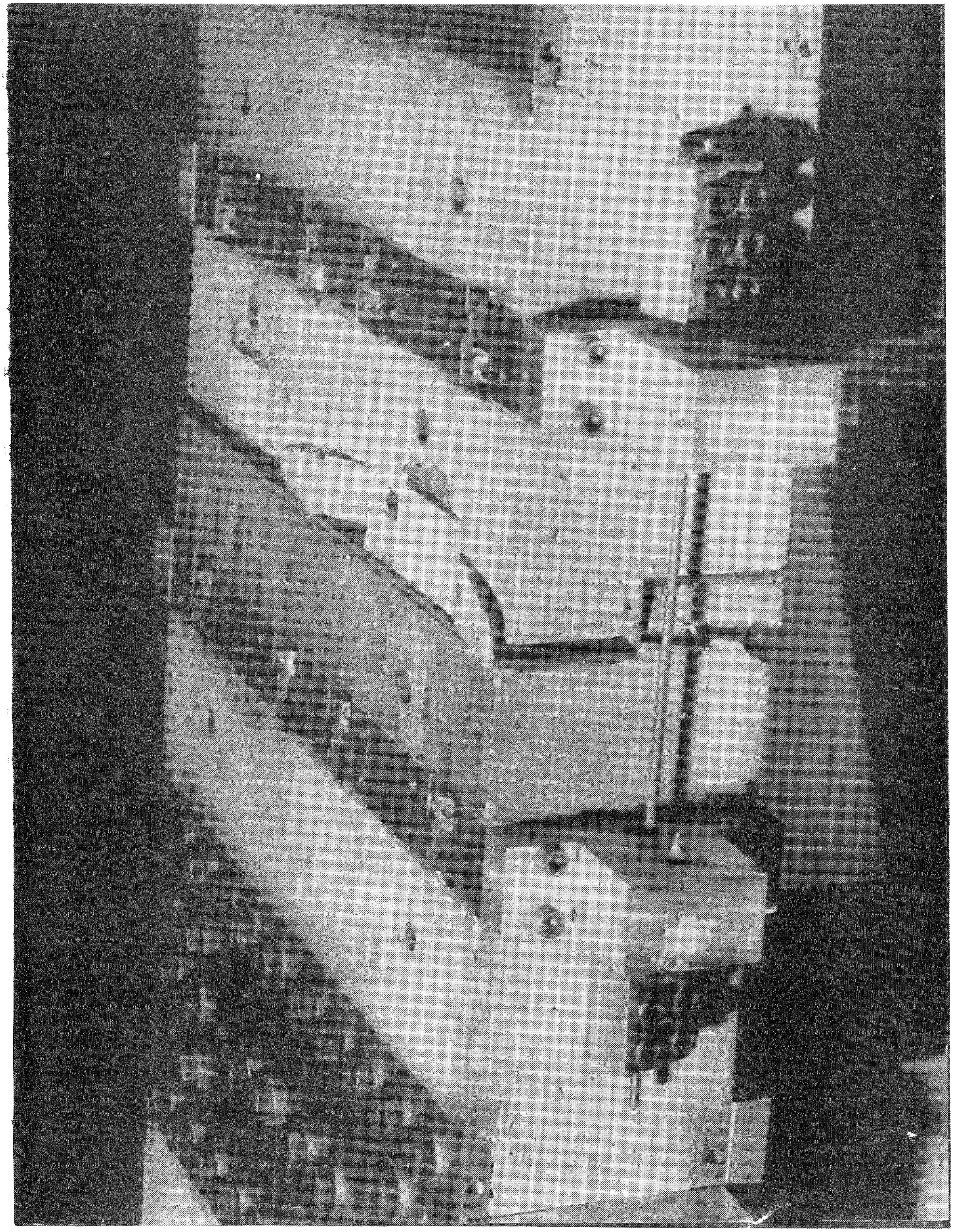

FIGURE 7: DAMAGED EXPANSION JOINT. TEST X7A.

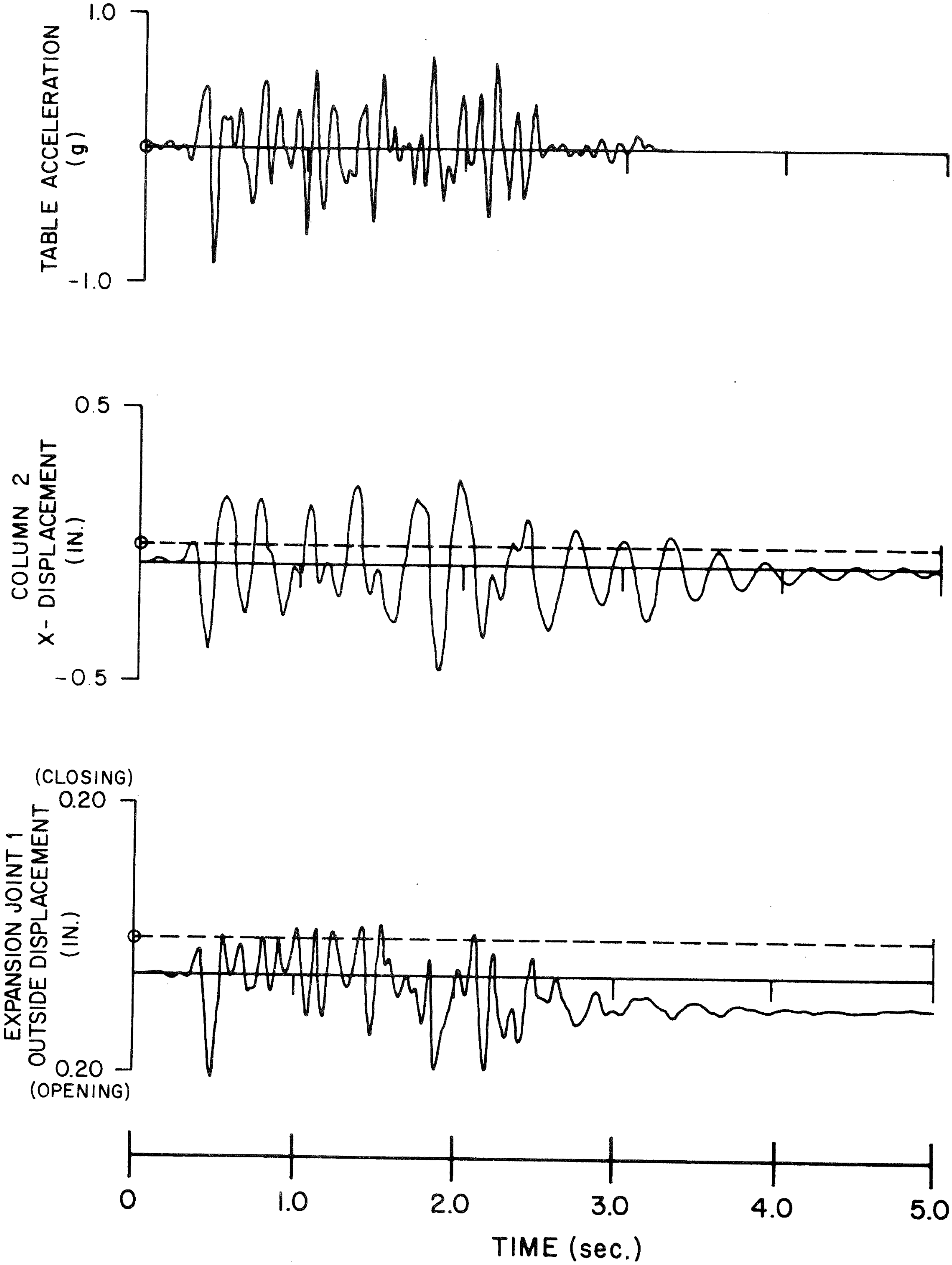

FIGURE 8: RESPONSE HISTORIES FOR TEST X7A.

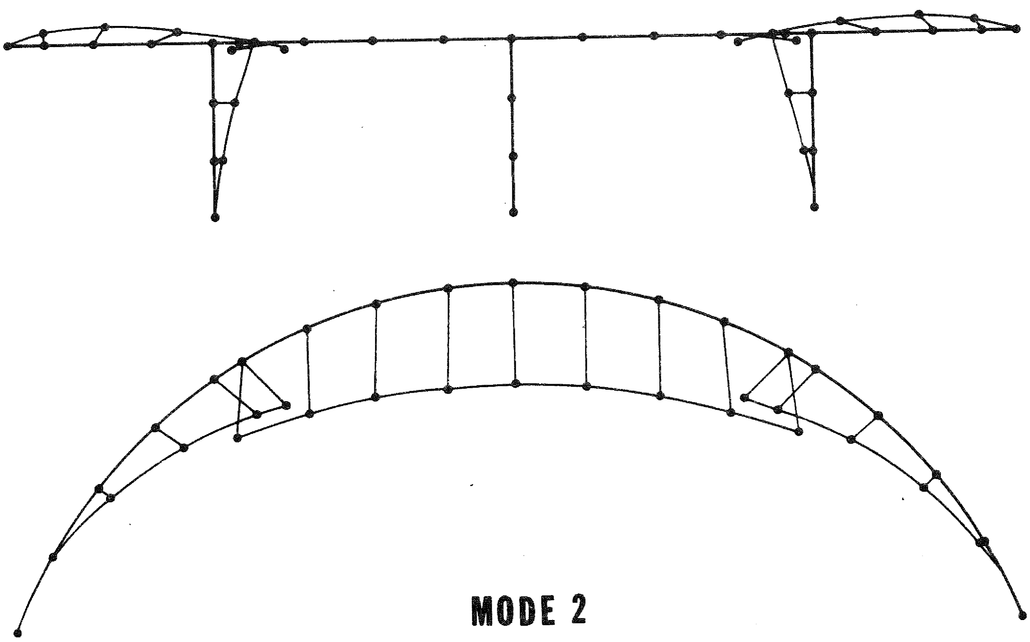

FIGURE 9: CHARACTERISTIC SHAPE MODE 2.

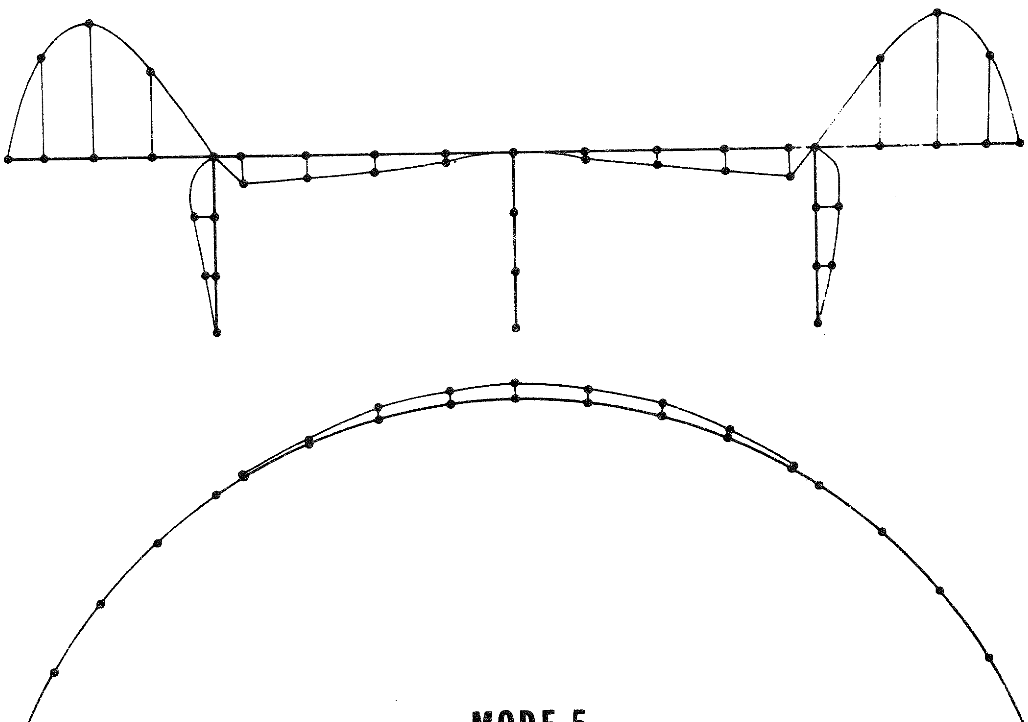

FIGURE 10: CHARACTERISTIC SHAPE MODE 5.

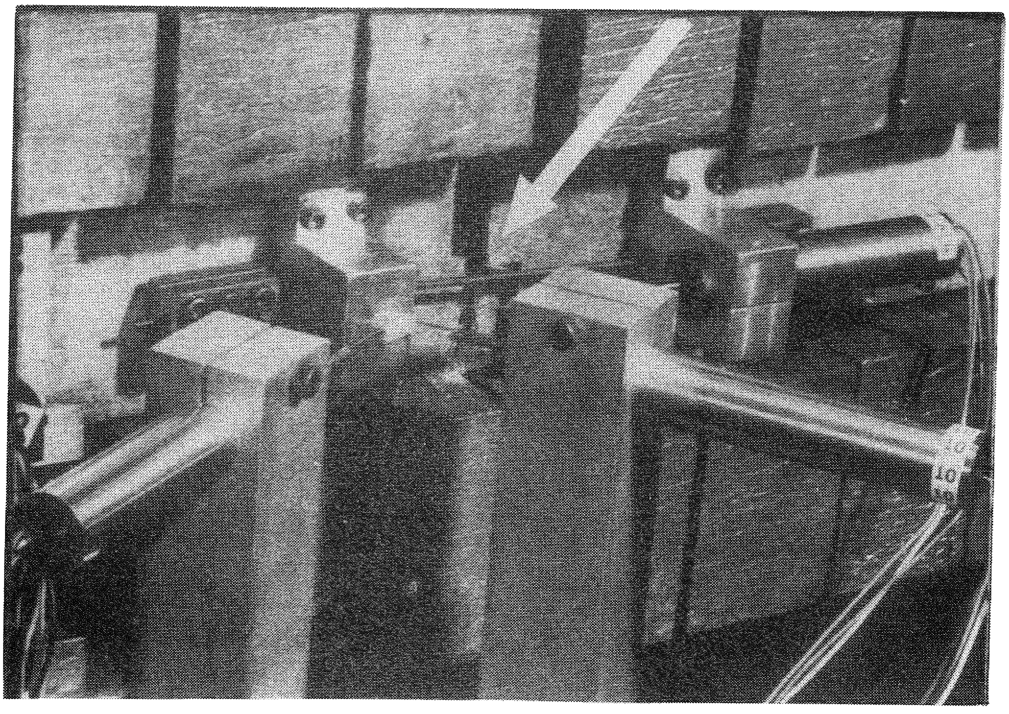

FIGURE 11: DAMAGED EXPANSION JOINT,
TEST Y5A.

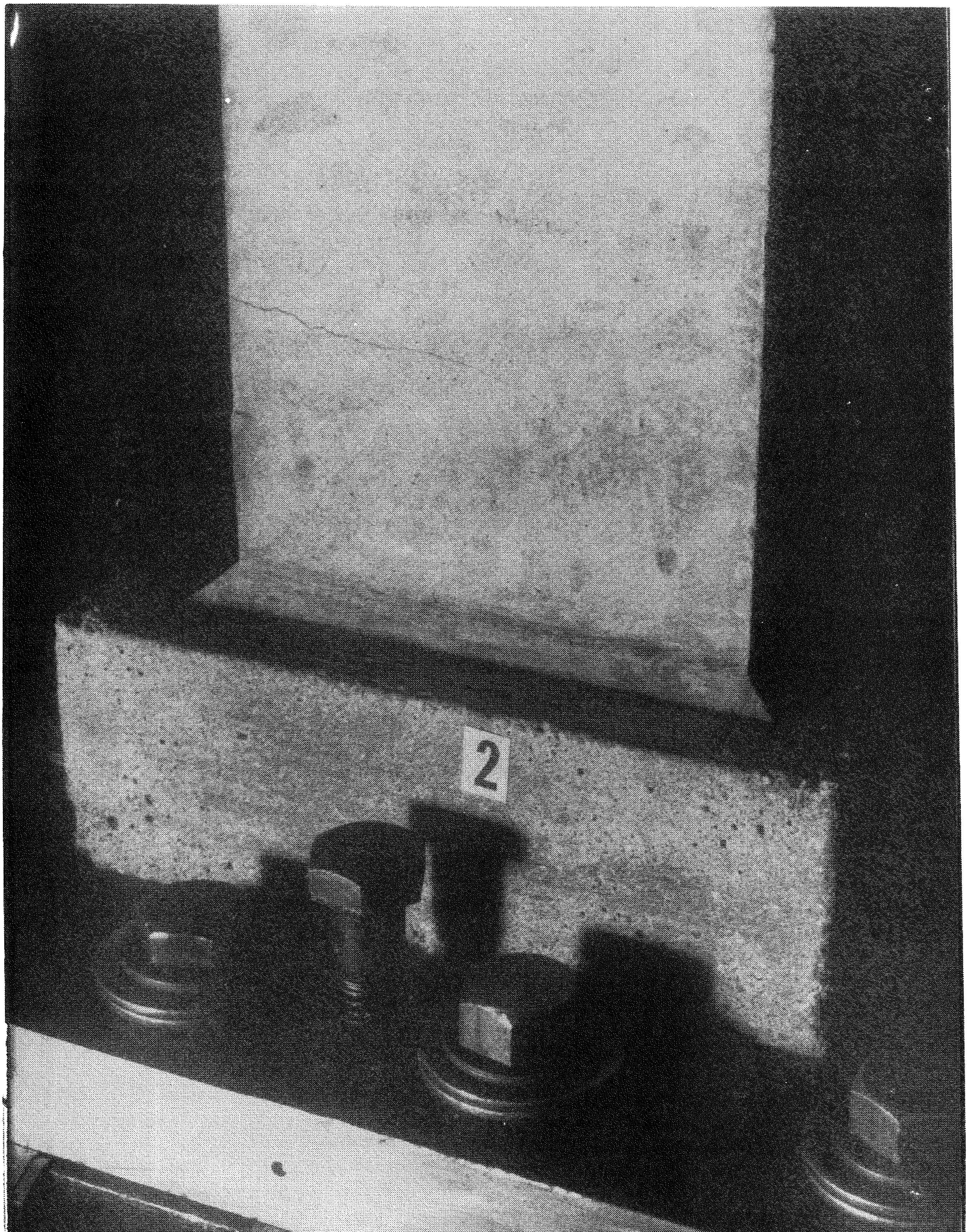

FIGURE 12: CENTRAL COLUMN DAMAGE, TEXT Y5A.

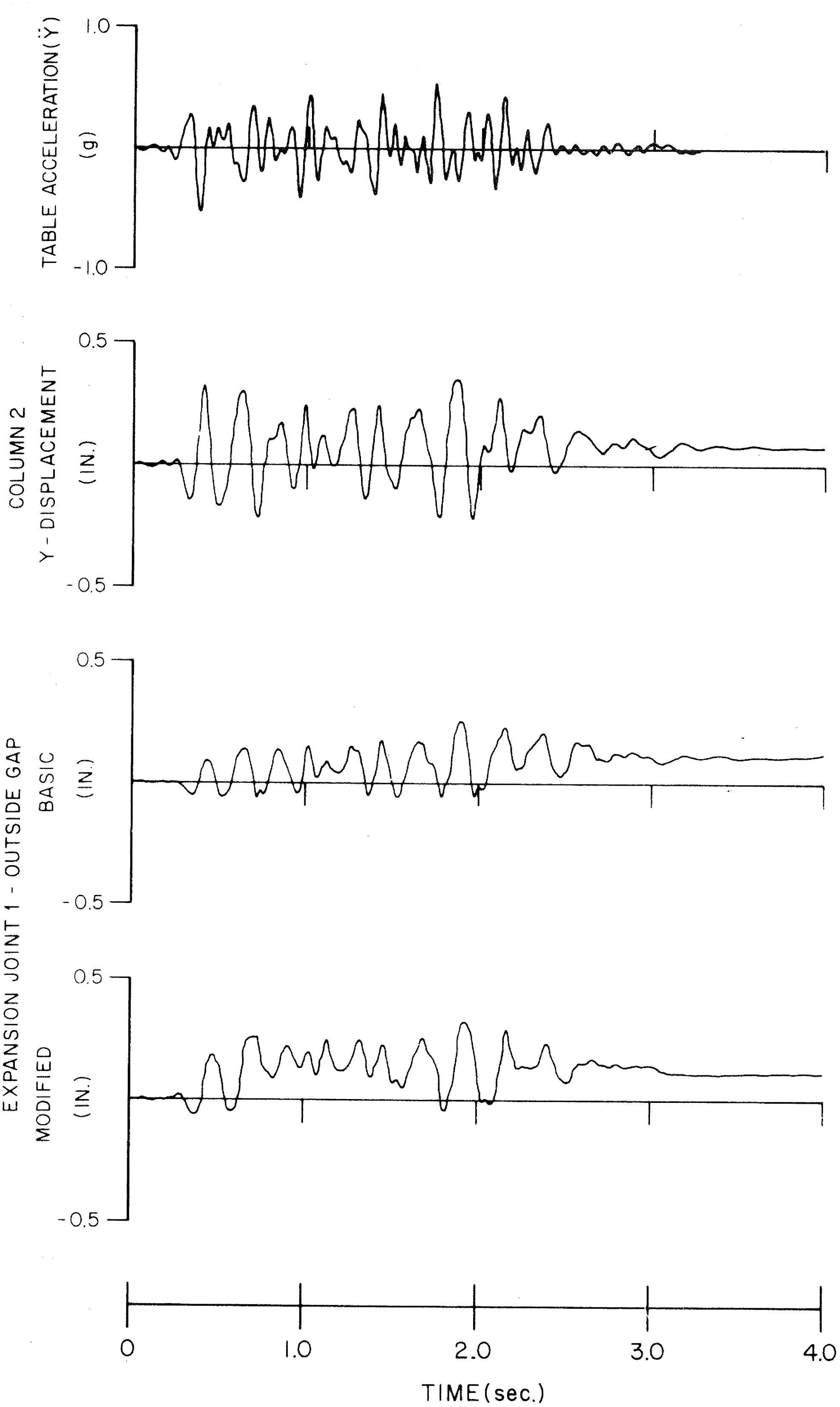

FIGURE 13: RESPONSE HISTORIES FOR TESTS Y4B AND Y5B.

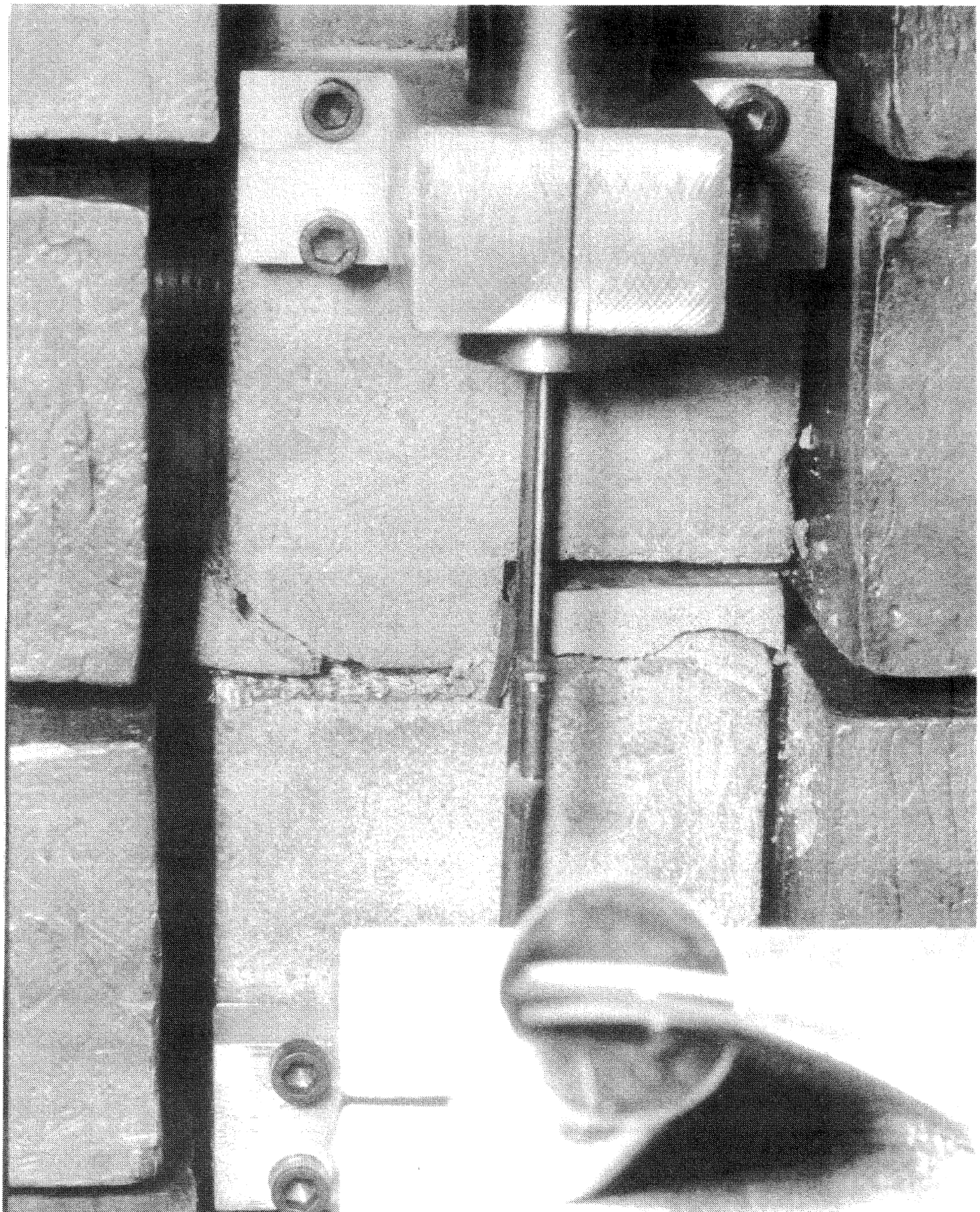

FIGURE 14: DAMAGED EXPANSION JOINT, TEST Y5B.