DYNAMIC RESPONSE OF REINFORCED CONCRETE COLUMNS TO MULTIDIRECTIONAL EXCITATIONS

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SUMMARY

The results are presented for a preliminary series of analytical and experimental studies of circular reinforced concrete bridge columns subjected to one and two horizontal components of excitation. The purpose of these studies is to assess the effects of multi-direction seismic input, the influence of different ground motion characteristics, and the reliability of various analysis methods. Four identical columns were constructed and tested in pairs using two different ground motion records. Results show that the bi-directional response is for the columns considered similar to that observed under only one component of excitation. Bi-directional response also resulted in a slower accumulation of visible damage and deterioration in performance. Analysis results indicated that elastic and inelastic idealisations were able to predict peak response well, but details of response were difficult to predict with conventional models.

INTRODUCTION

Damage to bridges during recent earthquakes has raised questions regarding the adequacy of design and analysis methods. The intense motions associated with near-fault sites or with unusually large earthquakes can impose special demands requiring careful consideration in earthquake hazard evaluation and bridge design. Structural response to such motions may include numerous, multi-direcional displacement excursions into the inelastic range. The ability of current details to withstand such demands and reliability of current analysis methods in predicting response under such severe loading conditions needs to be carefully addressed.

One issue raising concern regarding performance under these intense ground motions is the ability of analytical models to simulate the effects of degradation due to spalling, bar buckling, fracture of reinforcement, and loss of confinement. Cyclic displacement controlled tests indicate the change in capacity due to such deterioration, but do not indicate their effect on demands. Similarly, concern has also been raised about the effects of bi-directional response during such events. Only limited static [Wong et al, 1993; Zayati and Mahin, 1996] and dynamic [Kitajima et al, 1994] bi-directional testing has been performed to date. Previous analytical [Pecknold and Sozen, 1975] and experimental results suggest that differences between one and two components of horizontal excitation may be significant. Research is under way using the earthquake simulator (shaking table) at the Pacific Earthquake Engineering Research Center, University of California, Berkeley, to study the effects of near-fault and other intense ground motions on bridge systems. The main research objectives are to investigate: (1) the effects of bidirectional loading on the response of bridge columns and simple bridge systems, (2) the effects of short-duration near-fault ground motions as opposed to long-duration motions, and (3) the reliability of linear and nonlinear analysis techniques.

The initial focus of the experimental and analytical studies is on a set of four identical circular columns having spiral transverse reinforcement. In this investigation, the column is idealised as being fixed at the bottom and free to translate and rotate at the top. The experimental component of the study is carried out using an earthquake simulator. Gravity and inertial masses are simulated using concrete blocks supported at the top of the column. Two ground motions are used in the experimental study, one representative of a record obtained near the

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causative fault rupture, and the other representative of a larger magnitude event recorded at moderate epicentral distance. For each ground motion, one specimen is subjected to a single component of motion, while the second specimen is subjected to both components of motion.

SPECIMENS

Specimen Design

The specimens were designed according to the CALTRANS Bridge Design Specifications [California Department of Transportation, 1990], assuming a scale factor of 4.5. Each specimen consisted of a circular column with a diameter of 400 mm and a clear height of 1600 mm. The height to the center of the supported mass was 2400 mm, resulting in an aspect ratio of 6. The base and top of the column were anchored to slabs having a depth of 400 mm, and dimensions of 2440 mm x 2440 mm (Figure 1). The bottom slab was postensioned to the earthquake simulator and the top slab was attached to three additional weight blocks. The weight blocks weighed 77.5 kN each while the top slab weighed 57 kN. The cumulative total axial load on the column was about 290 kN, or 5.7% of the nominal compressive strength of the concrete section.

The column section had a 13-mm clear cover, and was reinforced with twelve No. 4 (13-mm diameter) longitudinal bars. The spiral reinforcement consisted of W2.5 plain spiral with a diameter of 4.5 mm, spaced at 32 mm. Hence, the longitudinal and transverse reinforcement ratios were 1.20 % and 0.54 % respectively. The longitudinal reinforcement was selected to provide the required capacity obtained from Caltrans' ARS spectra for soil type B, taking into consideration the scale effect which requires shifting the period by $1/(4.5)^{1/2}$, and using a ductility and importance factor of 4. This resulted in a transverse design acceleration of 0.25g or 72.5 kN, and a design moment capacity of 175 kN-m.

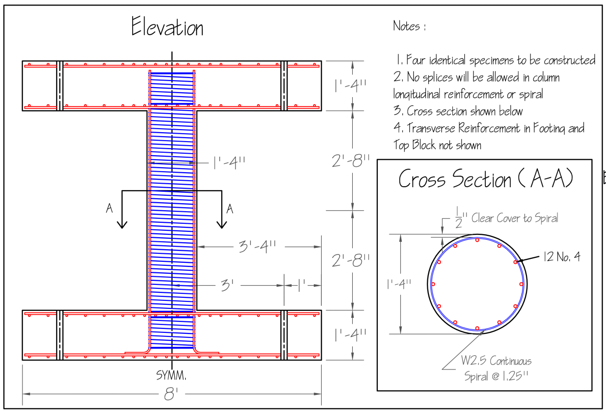


Figure 1: Specimen details and dimensions

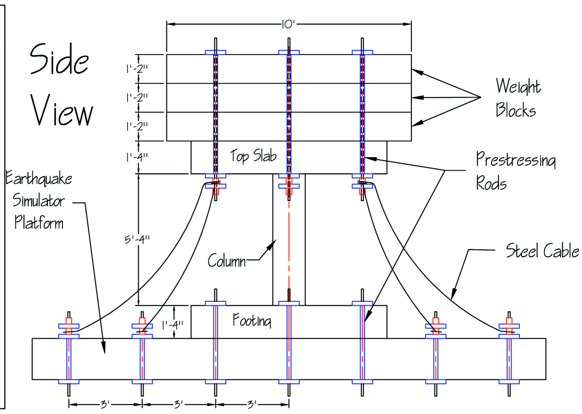


Figure 2: Test Setup

Material Properties

The longitudinal steel used was grade 60 ASTM 706 steel. Tension tests showed yield strength of about 520 MPa and an ultimate stress of about 720 MPa. The W2.5 plain wire used for the spiral was grade 80 ASTM 82. Tensile tests showed a yield strength of about 620 MPa.

The concrete had a specified 28-day strength within the range 28-35 MPa. The obtained 28-day strength was 30 MPa for the first two specimens (specimens A1 and A2), and 34 MPa for the remaining two (specimens B1 and B2). At the time of testing, the actual compressive strength was about 39.3 MPa for all specimens, while the measured splitting tensile strength averaged about 2.9 MPa.

EXPERIMENTAL PROGRAM

Test Setup

The top of the column was unrestrained simulating a cantilever configuration (Figure 2). This set-up allowed P-delta effects to be simulated realistically. Loose cables connecting the top slab to the platform were installed to prevent the top mass displacement from exceeding about 250 mm.

Input Motions

The four specimens were divided into two sets (A and B), with a different earthquake employed for each set. Within each set, the first column was subjected to one horizontal component of the earthquake (columns A1 and B1), while the second column was subjected to two horizontal components of the same earthquake (columns A2 and B2). The two ground motions used were selected after performing a wide array of dynamic analyses using different ground motion records. The first pair of columns was subjected to a near-fault motion recorded at the Olive View Hospital during the 1994 Northridge, California, earthquake (magnitude 6.7). Several older bridges were damaged near this site. The second pair was subjected to the Llolleo record from the 1985 Chile earthquake (magnitude 7.8). This record is nearly 120 seconds in duration. The time scale factor selected for the Olive View motion was $1/(4.5)^{1/2}$, while the duration of the Llolleo record was scaled by a factor of $1/(2)^{1/2}$.

The testing program consisted of a series of tests performed in the following order: (1) small amplitude snap-back tests to characterize natural frequencies and damping ratios; (2) a single test near the yield level to characterise the response of the specimens under earthquakes during which elastic response is expected; (3) a single excitation at the design-level earthquake amplitude; (4) a larger amplitude event to simulate a reasonable maximum credible event; followed by (5) a repetition of the design-basis event to assess the deterioration in the response characteristics due to accumulating damage (and possible response characteristics in a significant aftershock). Specimens within each pair were then subjected to the same series of repetitions of the maximum credible and design-basis events to study the effect of accumulating damage on performance.

Instrumentation

The shaking table and the specimen were extensively instrumented to measure the displacement and acceleration histories and the strain variations in the steel reinforcement bars. For each of the tests, 144 channels of data were recorded. Sixteen channels were used for monitoring the displacements and accelerations of the shaking table, while the remaining 128 channels included: 48 channels for strain gages in both the longitudinal and spiral steel reinforcement, 19 channels for accelerometers placed at different locations on the specimen to measure accelerations, 32 channels for Direct Current Displacement Transducers (DCDT's) which measure relative displacements between different sections along the height of the column for subsequent curvature calculations, and 29 channels for linear potentiometers measuring total external displacements at various points on the specimen. The external measurements allowed the monitoring of the deflected shape of the column during the test. The data was sampled at a rate of 0.01 seconds.

RESULTS

Some representative results from the Northridge and Llolleo records are presented below.

Northridge Olive View Records

Column A1 was subjected to the fault normal component of the Olive View record, applied in the nominal longitudinal direction of the bridge. Column A2 was subjected to the fault normal and parallel components of the Olive View record applied in the longitudinal and lateral components, respectively. The ground motion was scaled identically for columns A1 and A2 for the different runs. Figure 3 presents the displacement orbits for the first application of one or two components of the maximum credible earthquake. The measured table accelerations were 0.88g and zero, and 0.93g and 1.02g in the fault normal and parallel directions. In spite of intense fault-parallel motions, the observed response is dominated by the fault-normal component. The peak projected displacement is about 10% more during the unidirectional test. Note that the yield displacement of the column is about 25.4 mm. Time histories of the absolute and relative column displacements are shown in Figure 4. The response is dominated by a single large displacement cycle.

Specimens A1 and A2 showed no permanent damage for the *yield level* event (run 1). Permanent cracks were observed following the *design level* event (Run 2), accompanied by some minor spalling of the cover. Specimen A1 reached a peak displacement of 125 mm, while specimen A2 reached peak displacements of 121 mm and 41 mm in the longitudinal and lateral directions, respectively. At the first application of the *maximum level* event (run 3), spalling extended to about 200 mm above the base, and permanent cracks were observed to about 300 mm above the base. Both specimens showed very similar response, except that maximum spalling occurred in Specimen A2 at an angle of about 30 degrees from the longitudinal axis. Response was still dominated by the fault normal component. Maximum displacements were 142 mm for A1, and 132 mm and 58 mm for A2. No rebar buckling or severe yielding of the transverse reinforcement was observed at this stage.

The *design level* was applied again in run 4 and run 7 to simulate the effect of a large aftershock. The response to this level was similar to the first application of the design level event, although the specimens had gone through a number of runs at the maximum level. This behavior suggests that good behavior might be expected during aftershocks.

The *maximum level* was applied again in runs 5 and 6. Column A1 experienced two longitudinal rebar fractures during the fourth application of the maximum level (run 8), and the test was stopped. Column A2 was able to sustain six applications of the maximum level (up to run 10). One longitudinal bar fractured in run 9. In run 10, the specimen reached a maximum displacement of 245 mm corresponding to a displacement ductility of about 10, with a residual displacement of 155 mm; at this point, the test was stopped. Displacements increased gradually with repetitions of the maximum credible earthquake, and in the case of Specimen A2, the displacement im the last maximum event was 100% larger than during its first application.

Figure 5 and figure 6 show the increase in displacement during the various excitations imposed for specimens A1 and A2, while figure 7 compares the increase for Columns A1 and A2. The displacement response spectrum increases with increasing period near the natural period of the test column (Figure 8).

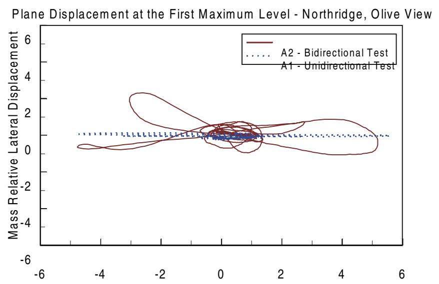


Figure 3: In-plane displacement orbits for specimens A1 and A2

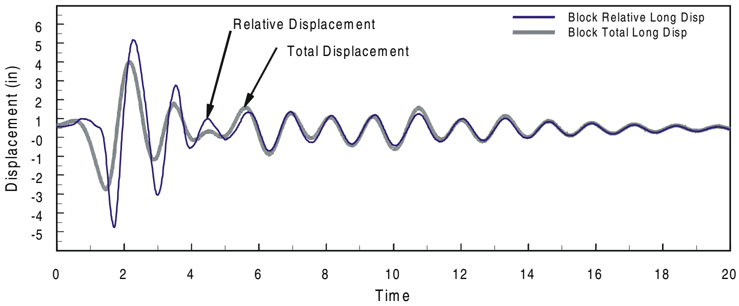


Figure 4: Displacement history for specimen A2

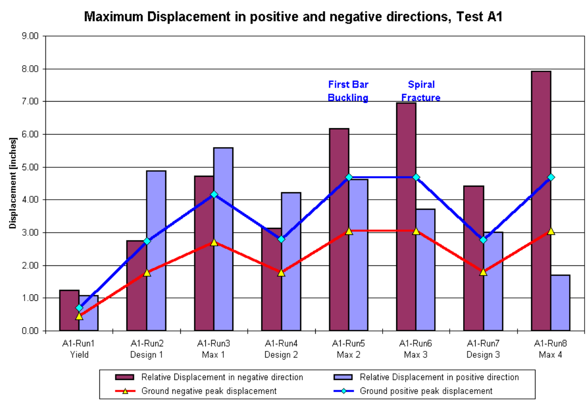


Figure 5: Peak displacements for specimen A1

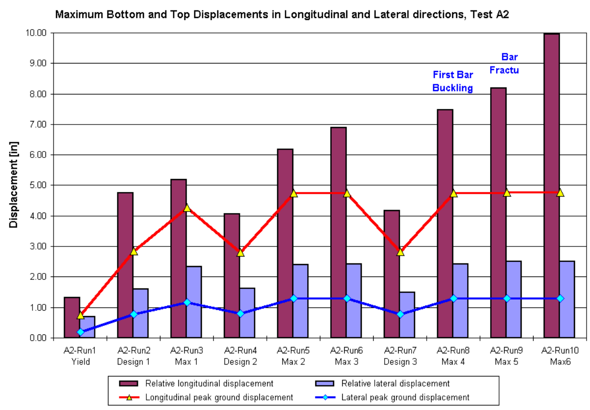
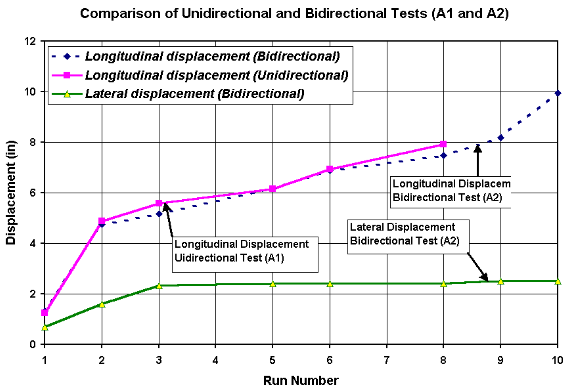


Figure 6: Peak displacements for specimen A2



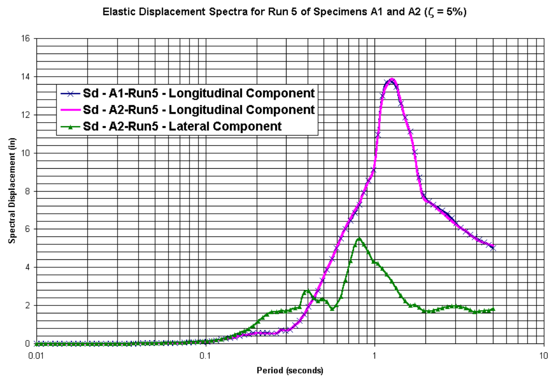


Figure 7: Comparison of peak displacements obtained for Specimens A1 and A2

Figure 8: Displacement response Spectrum of the Olive View Record

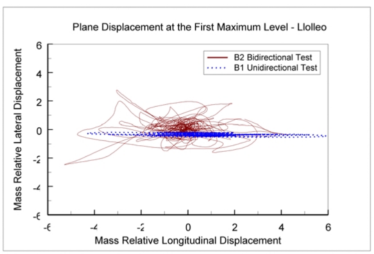
Llolleo Records

Specimen B1 was subjected to the larger component of the Llolleo record while specimen B2 was subjected to both components. The records were scaled similarly for both specimens to allow the two tests to be compared.

The loading scheme was very similar to that of the first set. The first four runs brought the specimen gradually to a level slightly above *yield*. No permanent damage was seen up to this point in either of the two specimens, although some minor cracking was seen at run 4.

The *design level* event was applied in runs 5 and 7. Following run 5, some spalling was observed at the base of the column, with major permanent flexural cracks up to a height of 500 mm. There was less cracking in Specimen B2 compared to B1. The maximum relative displacement reached was 86 mm for Specimen B1, and 78 mm in the longitudinal direction and 43 mm in the lateral direction for Specimen B2. The displacement observed at the second application of the design level earthquake was very similar in magnitude, and little additional damage was observed.

The *maximum earthquake level* event was applied three times to Specimen B1 in runs 6, 8 and 9. Following run 6, significant spalling was observed, accompanied by the buckling of one longitudinal bar. This bar fractured in run 8 along with another spiral reinforcing bar on the opposite side. In the ninth, another longitudinal bar fractured, and a few other spirals fracture; the test was stopped at this stage. The peak displacement reached was about 150 mm, and this displacement was almost the same for all three repetitions of the maximum level event. As for Specimen B2, the maximum level was applied seven times in runs 6 and 8 through 13. Damage progressed at a slower rate compared to Specimen B1, and the peak displacements reached were about 150 mm and 70 mm in the longitudinal and lateral directions, respectively. The peak displacements were almost constant through all repetitions of the maximum level, mainly due to the flat nature of the displacement spectrum near the vibration period of the column. Buckling was delayed until the 11th run, and the test was terminated after a longitudinal bar fractured in run 12 and spiral fractured in run 13.



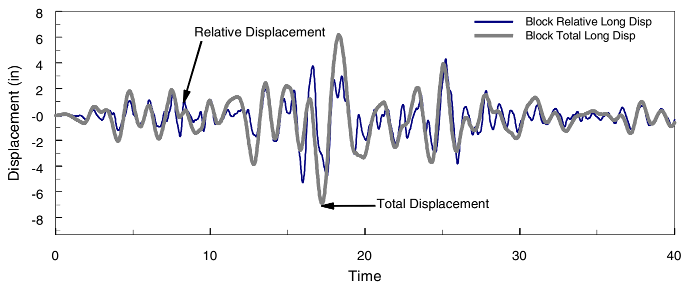


Figure 9: In-plane displacement orbits for specimens B1 and B2

Figure 10: Displacement history for Specimen B2

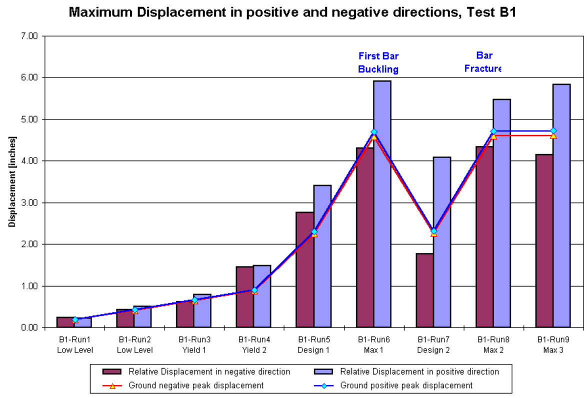


Figure 11: Peak displacements for Specimen B1

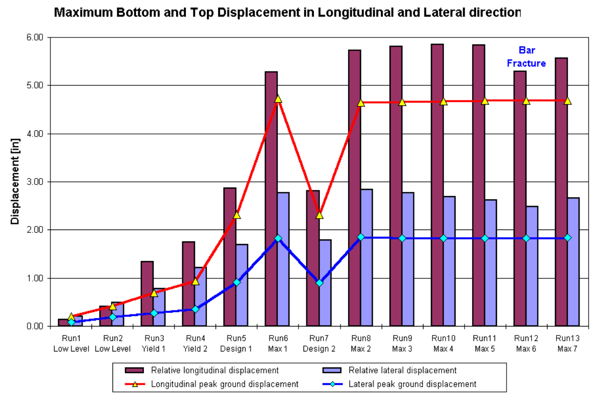


Figure 12: Peak displacements for Specimen B2

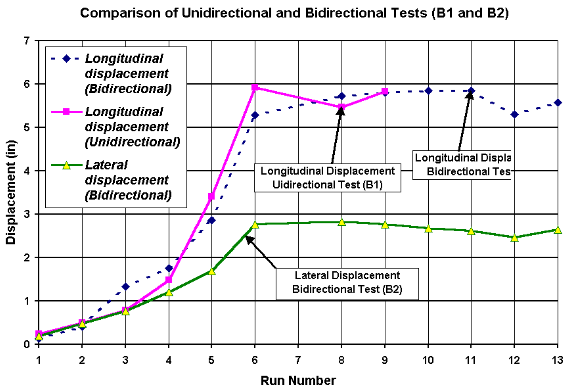


Figure 13: Comparison of displacements obtained in Specimens A1 and A2

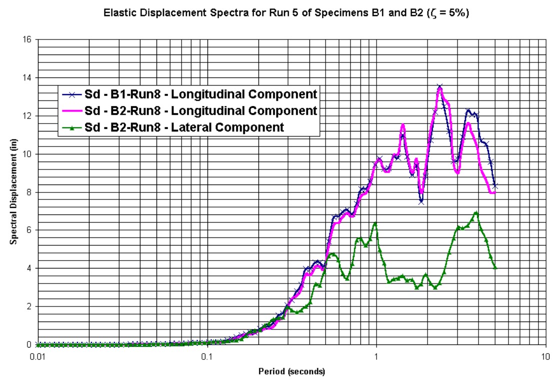


Figure 14: Response Spectrum of Llolleo record

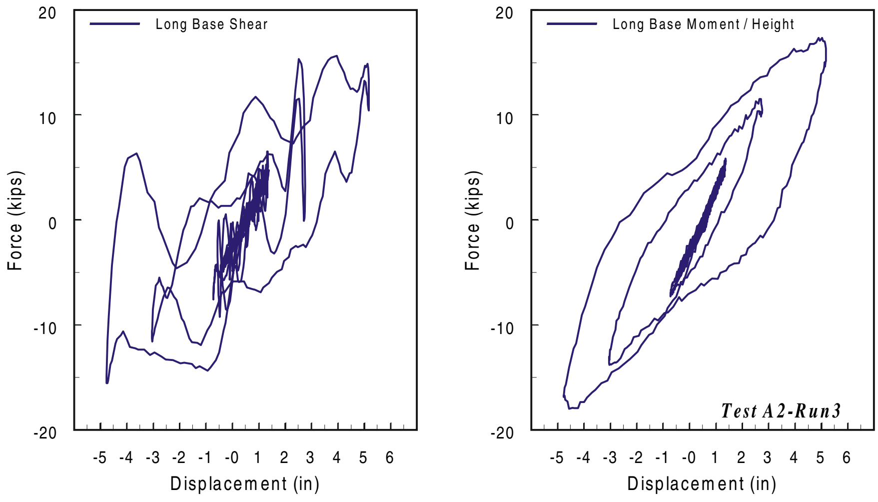


Figure 15: Force-displacement plot for longitudinal direction under two components of the Live View records: (a) shear forces and (b) normalised overturning moment

General Observations

Interestingly, all specimens began to develop flexural cracks near their tops during early cycles, with some of them lightly spalling during the later runs. This observation is attributed to the significant higher mode effects introduced by the ground motions. The higher mode effects are associated with the rotational mass moment of inertia of the weight used to represent the bridge deck. During large displacement excursions, the top of the column does not immediately translate or rotate due to inertia forces, which results in significant moments at the top of the column. This also results in higher shear forces, which fluctuate in magnitude with a high frequency. As a result, lateral displacement-shear hysteretic loops are more complicated than base moment - chord rotation (time height) (See figure 15).

Interestingly, it was observed that all specimens exhibited nearly constant maximum flexural capacity, and this did not vary significantly for the unidirectional and bidirectional tests, until one or more bars had fractured. In most cases, however, the column displacement did not increase significantly, even after bar fracture occurred.

ANALYSIS

An extensive analytical investigation is being carried away to assess various analysis methods appropriate for design and more refined evaluation, and to study the accumulation of damage during the tests.

Figure 16 compares experimental and analytical results for Specimen A2 for the first design level event. A fiber model is used to represent the specimen. Very good correlation was obtained for the initial near-field pulse portion of the record. The correlation at the end of the record, where the response should be essentially elastic, is not as good. This result is attributed to the lack of a model for rebar pullout at the foundation of the column. Hence, the estimated natural period of the column may be underestimated, which might lead to erroneous results when a simple linear or nonlinear dynamic analysis is performed.

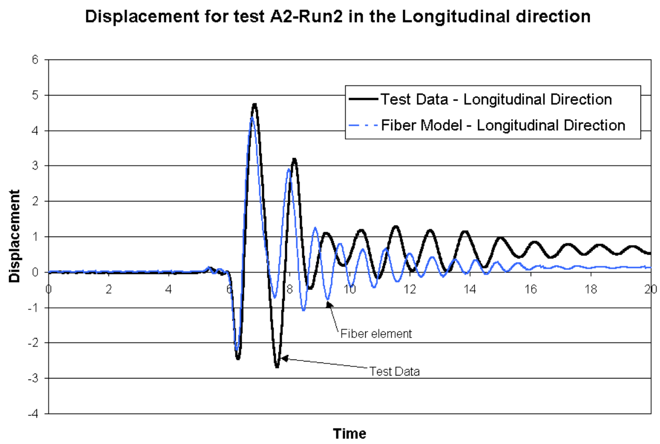


Figure 16: Computed and recorded fault normal response

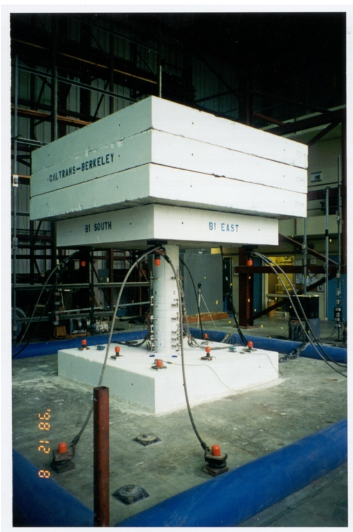


Figure 17: Undamaged specimen

Figure 18: Specimen A1 after the 5th maximum level

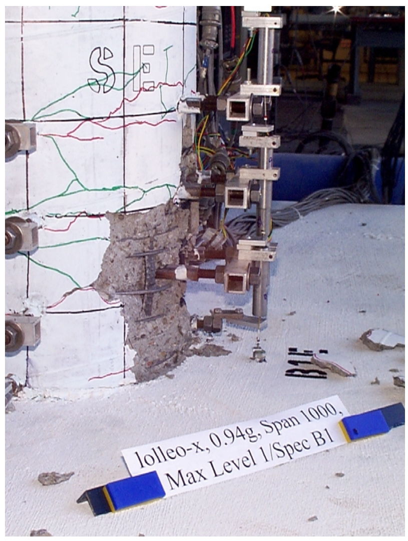


Figure 19: Specimen B1 after the 1st maximum level

CONCLUSIONS

The results of parametric analytical studies, shaking table tests, and verification analyses are being used to assess the adequacy of existing linear and nonlinear analysis procedures for bridge columns, and the reliability of design methods for near-fault and multi-component excitations. Results obtained to date indicate that current design methods are reliable for moderately long period bridges provided they are subjected to ground motions consistent with those used in their design. The results clearly indicate that reinforced concrete columns, even with significant permanent damage, have considerable reserve capacity enabling them (in the absence of significant P-delta effects) to withstand substantial numbers of aftershocks or even additional earthquakes. The results suggest that bi-directional response is dominated by one component of ground shaking, that response for one- and two- components of motion are similar, that permanent damage is less when two components of motion are considered than when only one is used. Analytical results suggest that elastic methods can predict peak displacement demands relatively well so long as the period of the structure is long compared to the duration of the pulse or predominant period of the structure. Analyses are continuing to validate and where necessary improve nonlinear dynamic analysis models as well as design methods and idealizations. Parametric studies are underway to assess the effects of various ground motion and structural characteristics on performance.

In the near future, additional tests will be conducted using noncircular columns with interlocking spirals. As part of this new phase of the experimental investigation, two columns will be connected to form a portion of single column viaduct structure. In this case, the periods will be different in the longitudinal and transverse directions of the roadway, and the columns will develop plastic hinges at the top and bottom under longitudinal frame action and at their bottom under transverse cantilever action.

ACKNOWLEDGEMENTS

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