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SHAKE TABLE TESTING AND ANALYSIS OF TWO-COLUMN BENTS

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SUMMARY

The seismic performance of two-column bridge bents with different aspect ratios was investigated. Three models with aspect ratios of 6.64, 4.5 and 2.5 and scale ratio of 0.3 were designed according to the updated CALTRANS design criteria [1]. The three specimens were tested dynamically using a shake table. The shake table was able to exert various amplitudes of the Sylmar record from the 1994 Northridge Earthquake on the three specimens. All deformations, rebar strains and mass accelerations were recorded during shaking. The three specimens behaved very well and resisted high levels of excitations. The two taller specimens had similar flexural behavior with high levels of ductility and drift, whereas the short specimen had a flexural/shear behavior with lower levels of ductility and drift.

INTRODUCTION

The design of structural bridge elements like columns, beam-column joints, and cap beams has evolved in the past 20 years. Many experimental tests have been done in order to determine the behavior of bridge bents under seismic loading. Most of those tests were static performed by monotonic cyclic loadings, whereas few of those tests were dynamic performed by shake tables to simulate the actual earthquakes. Based on the literature review, few studies were concerned about testing the seismic behavior of the newly designed models, in particular, the models representing two-column, hinged base, bridge bents with box superstructure. Therefore, the present study investigated the seismic behavior of the models of the two-column bridge bents by subjecting them to actual time earthquakes. The study also aimed to model the actual bridge mass to have more realistic representation of dynamic mass and the P-δ effect.

TEST MODELS

Based on the 0.3-scale model developed from a previous study [2], three specimens were designed using current CALTRANS specifications [1] and recommendations. The current design focused on column confinement, column shear capacity, and the seismic details of the longitudinal and transverse reinforcement in the cap beam and in the beam-column joint. The

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three specimens were identical except for the column aspect ratios, which were 2.5, 4.5, and 6.64. The new design configuration for the three specimens is shown in Fig. 1. Several computer programs were used to predict the seismic behavior of the test specimens and to ensure that all specimens would reach total failure before exceeding the maximum shake table capacity. Using the expected seismic behavior obtained from computer analysis, an instrumentation layout was designed to measure specimen acceleration and displacement and to record concrete and steel strains generated during testing.

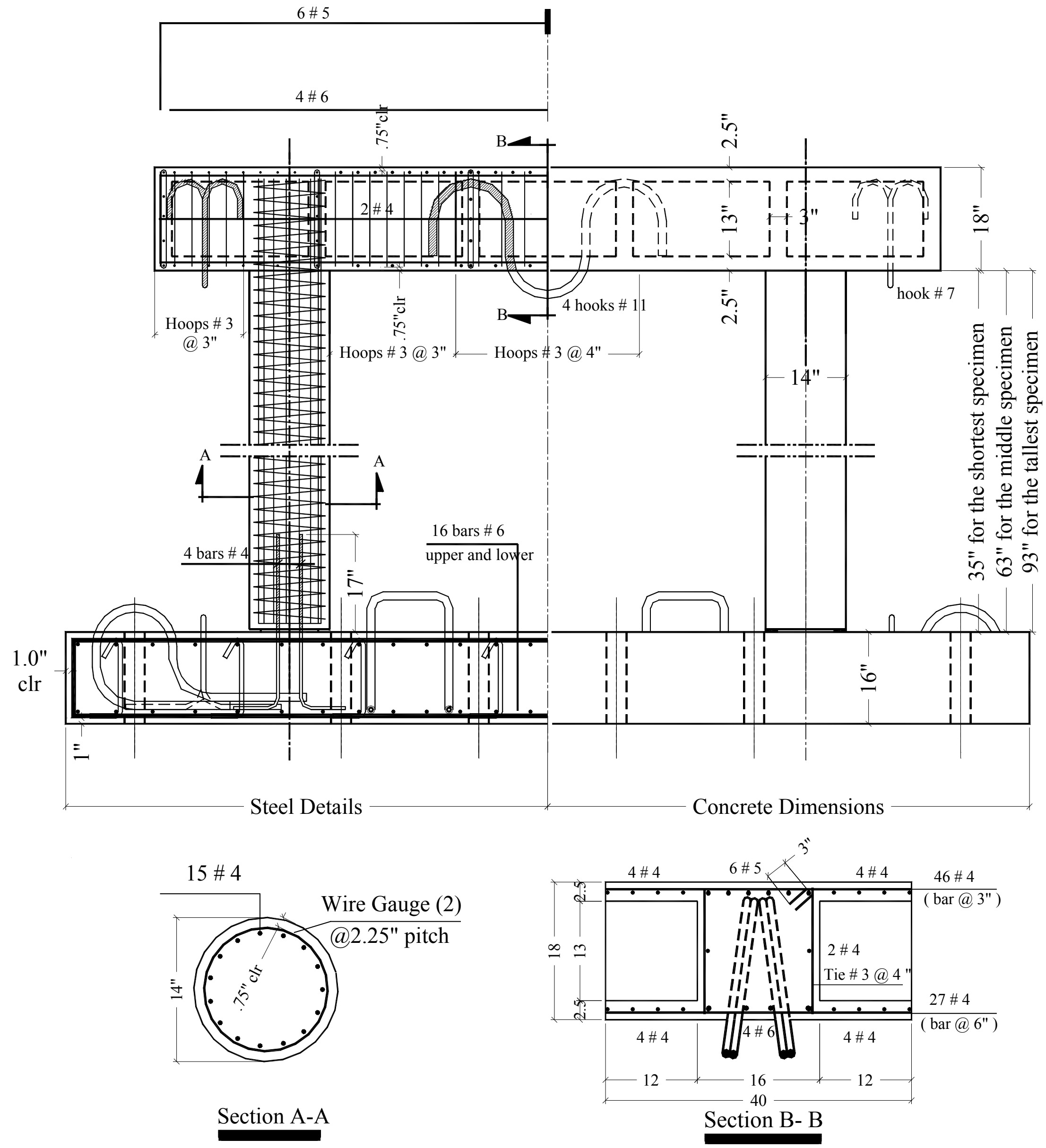


Fig. 1a: Model Configuration (English Units)

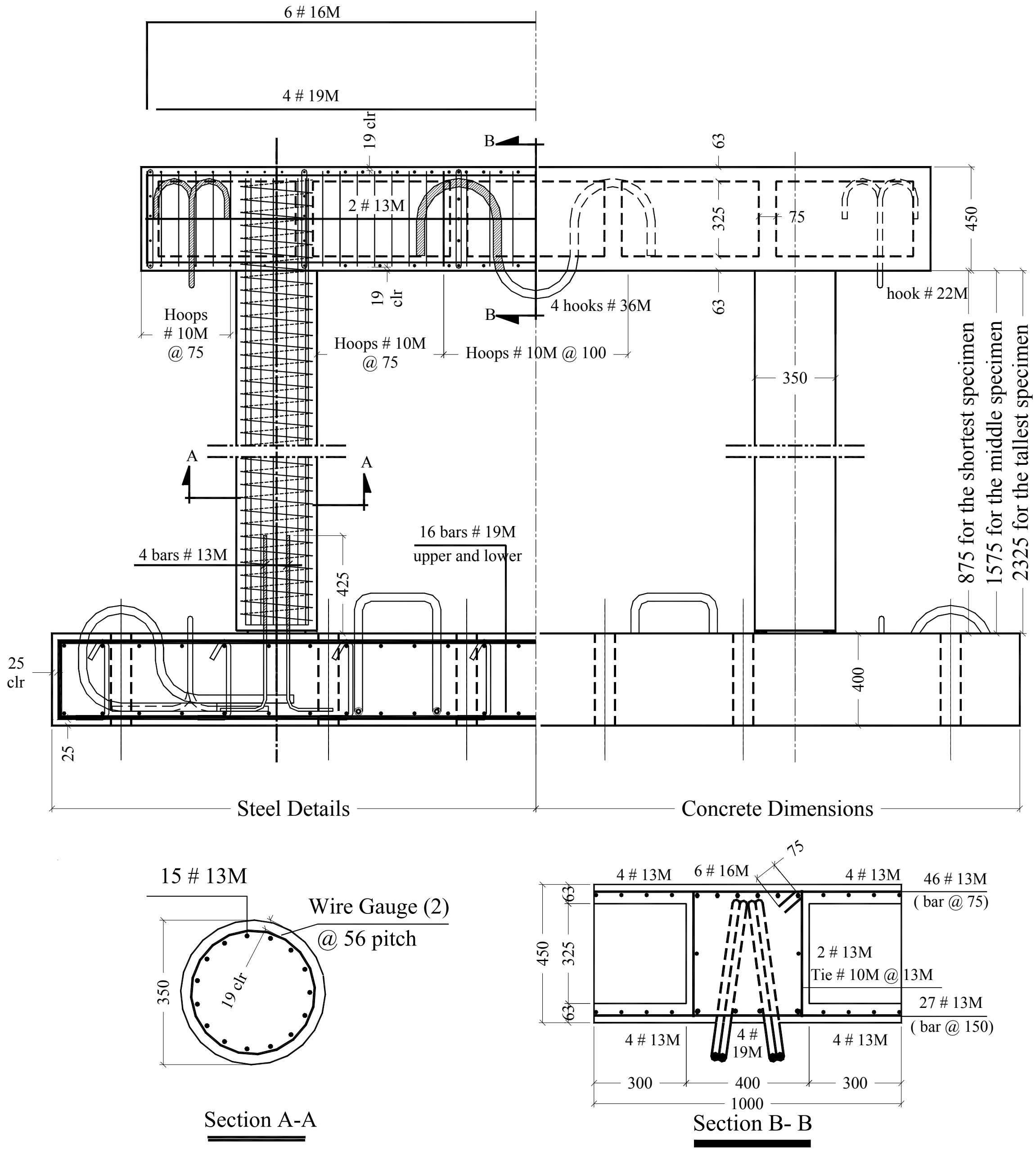


Fig. 1b: Model Configuration (Metric Units)

LOADING SYSTEM

To simulate the structure weight, lead weight was placed on the structure. The value of lead mass was calculated based on the scale ratio between the model and prototype. To produce realistic stresses, the weight causes an axial load in each column of 0.05 f'_cA_g for the target f'_c of 5.0 ksi (35 MPa). Steel buckets were designed to contain the lead blocks and to be loaded on the cap beam of each specimen. Fig. 2 shows the lead buckets placed on middle height specimen.

Fig. 2: Loading and Stability System

STABILITY SYSTEM

The loading system allowed the mass on each bent to move freely back and forth during shaking, and it was expected for the mass to cause large displacements after forming the mechanism. Therefore, a stability system was developed to stop the moving mass in the event of total collapse. The stability system was designed to resist the inertia force generated from the bent mass motion after reaching total failure. It consisted of cables connected between cap beams and footings by means of hooks attached to the specimen body. The stability system was also designed to limit any large displacements in the transverse direction. An additional steel frame was designed and attached to the shake table to secure the area around the table and to prevent any damage to the shake table. Fig. 2 shows the stability system for the medium height specimen. The height of the horizontal beam was adjusted for each specimen.

SPECIMEN CONSTRUCTION

The construction of the three specimens took three stages: footings, columns and decks. It was decided to do each stage for the three specimens simultaneously. For the footings, all hinge dowels were prepared for strain gages before concrete casting. Fig. 3a shows part of the footing reinforcement before casting. Fig. 3a also shows the hinge dowels of one of the columns after covering the dowel strain gages. After footing casting, special care was taken to prepare the column-footing construction joint. Fig. 3b shows the column-footing construction joint before removing the concrete debris.

For the columns, each column cage was constructed and all strain gages were placed in their locations after surface preparation. After this process, all strain gages were labeled and put in plastic tubes to be collected from one outlet (Figs. 3c and 3d). For the decks, each steel cage was constructed in its place and one side of each form was left open for placing strain gages. The construction of the three specimens was completed after casting the bent decks. Fig. 3e shows the casting process of the specimen with the short columns. During the casting of each stage, concrete cylinders were taken to make sure of the concrete quality on the casting day and to know the concrete strength on the day of the test.

Fig. 3a: Footing Reinforcement

Fig. 3b: Column-Footing Construction Joint

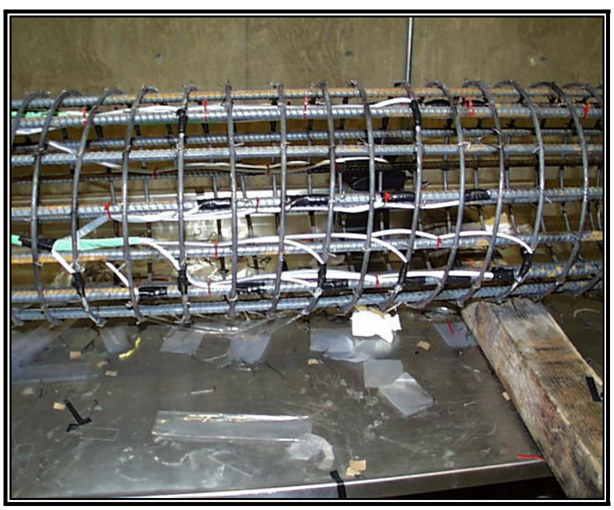


Fig. 3c: Gaging Process

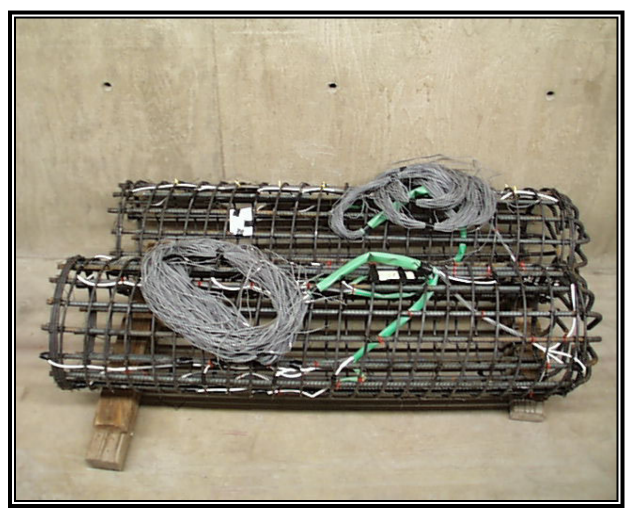


Fig. 3d: Steel Cages after Gaging Process

Fig. 3e: Concrete Casting for Specimen Beam

EXPERIMENTAL RESULTS

Each specimen was loaded with increasing values of the Sylmar record from the 1994 Northridge Earthquake in California. Table 1 shows the sequence of loadings for each specimen. The input acceleration for 1.0 x Sylmar was 0.61g. The load-displacement cumulative hysteresis curves for the three specimens are shown in Fig. 4. The increase in specimen maximum displacement and the reduction in the specimen lateral capacity are evident as the specimen aspect ratio increases from 2.5 to 6.64 (from short to tall specimens, respectively). Table 2 provides a summary of the experimental results. In the short specimen, the failure mode was a mixture of flexural and shear. This was clearly shown in column flexural and shear cracks. Fig 5 shows the short specimen columns after the maximum load (3.25 x Sylmar). In the plastic hinge zones of both columns, concrete spalled till exposing the column spiral. The concrete started to spall at the loading of 2.5 x Sylmar and the column longitudinal reinforcement started to yield during the loading of 1.75 x Sylmar. The test was stopped due to failure at the column base. During the maximum loading, the maximum reported base displacement was 0.5" at the east column base. Fig. 5c shows the residual slippage at the east column base at the end of the loading. There was no spalling in the east column base because of the low column compression since the predominant motion of Sylmar record is in the west direction. Fig. 5d shows the failure of the west column base. This compression failure was as a result of the lack of confinement in this region and the large compression force. For beam-column joints, limited cracks were observed (Figs. 5e and 5f) during the maximum loadings.

In the medium and tall specimens, the failure mode was primarily flexural. As shown in Fig. 6, the maximum loading caused deep concrete spalling till exposing the longitudinal and transverse reinforcement in the expected plastic hinge zones. In the tall and medium specimens, a large piece of concrete spalled from the hinge base on the west side of the west column base. This failure was also observed in the short specimen at the same location (as mentioned earlier).

Similar to the short specimen, the beam-column joints in the tall and medium specimens experienced only limited cracking.

Table 1: Testing Sequence for Three Specimens

Short Specimen		Medium Specimen		Tall Specimen	
Run	Motion (x Sylmar)	Run	Motion (x Sylmar)	Run	Motion (x Sylmar)
1	Snap	1	Snap	1	Snap
2	0.20	2	0.10	2	0.10
13	Snap	3	0.20	3	0.20
4	0.25	4	0.25	4	0.25
5	0.50	5	0.50	5	0.50
6	0.75	6	0.75	6	0.75
7	1.00	7	1.00	7	0.85
8	Snap	8	Snap	8	1.00
9	1.25	9	1.25	9	1.25
10	1.40	10	1.40	10	Snap
11	1.75	11	1.50	11	1.50
12	2.00	12	1.75	12	1.75
13	Snap	13	2.00	13	2.00
14	2.125	14	Snap	14	Snap
15	2.25	15	2.25	15	2.25
16	2.375	16	2.50	16	2.50
17	2.50	17	2.75	17	2.75
18	2.625	18	3.00	18	Snap
19	2.75				
20	3.00				
21	3.25				

Table 2: Summary

Specimen	Column Concrete Strength, ksi (MPa)	Input Accel. of 1st Spalling	Displace. at 1st Spalling, in. (mm.)	Max. Input Accel.*	Peak Force, Kips (kN)	Max. Displace. in* (mm.)	Displace. Ductility*
Short	5.86 (41)	1.5g	1.3 (33)	1.95g	87.0 (387)	1.71 (43)	4.0
Medium	4.1 (29)	.75g	1.9 (48)	1.8g	50.0 (223)	6.35 (159)	6.0
Tall	4.11 (29)	0.9g	3.75 (94)	1.65g	33.35 (148)	10.0 (250)	8.0

* The maximum input acceleration, maximum displacement and displacement ductility are all based on a capacity equal to 80% of the peak capacity. Displacements are based on total system displacements.

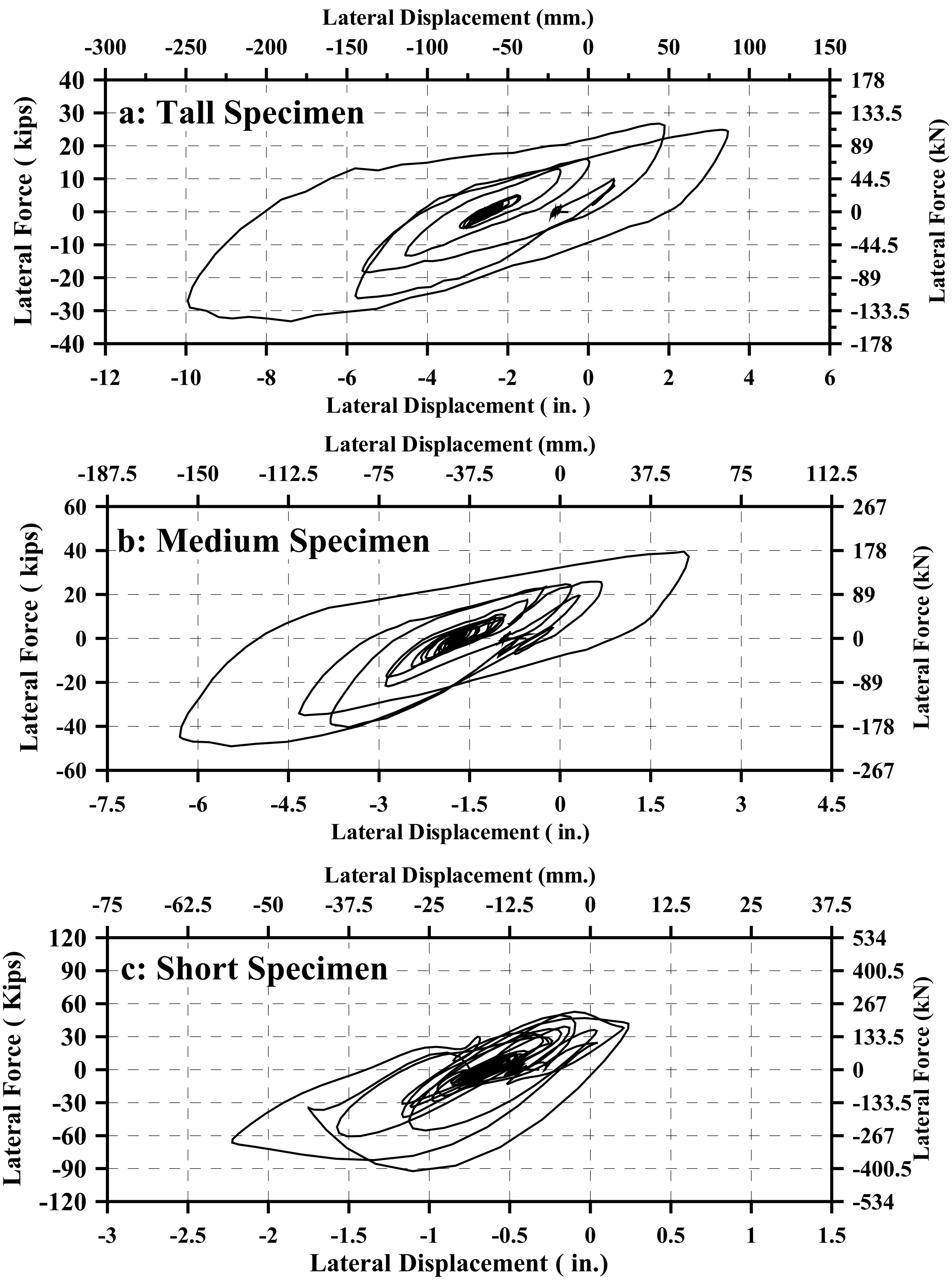


Fig. 4: Cumulative Experimental Hysteresis Responses for the Three Specimens

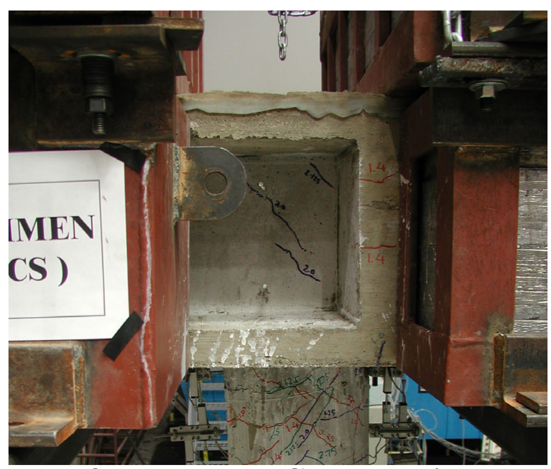
a: East Column (South-East View)

b: West Column (South View)

c: East Column (North View)

d: West Column Base (South view)

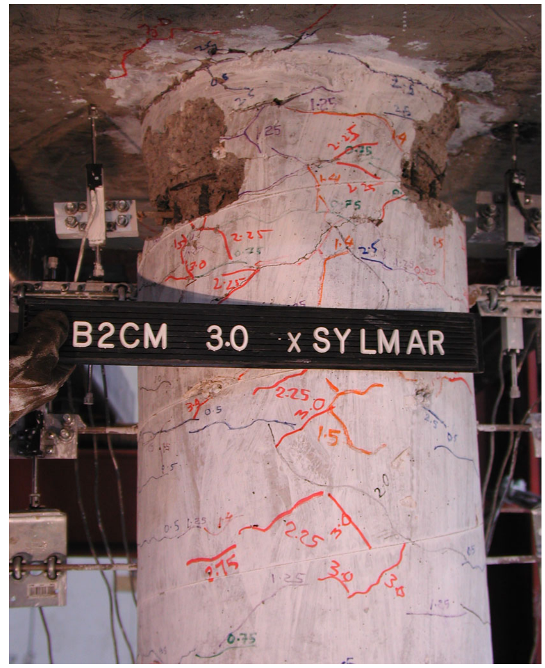
e: East Beam-Column Joint



f: West Beam-Column Joint

Fig. 5: Observed Behavior in Short Specimen

a: East Column –North View



b: West Column –South View

c: East Column –West View

d: West Column –West View

e: Hinge Base at West Column

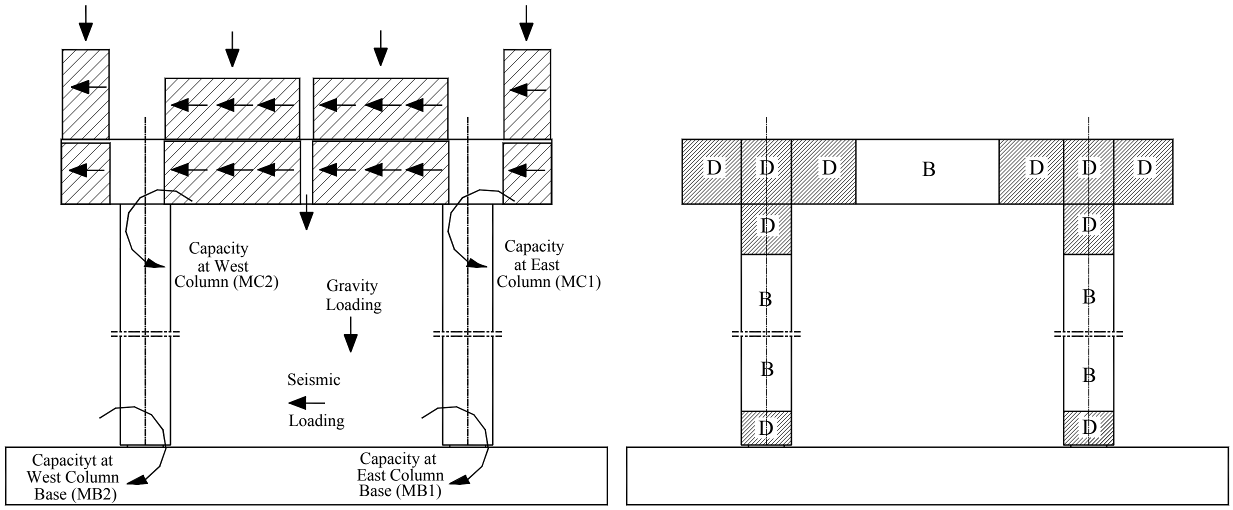
f: Hinge Base at West Column

Fig. 6: Observed Behavior in Medium (a, b & e) and Tall (c, d & f) Specimens

SPECIMEN MODELING AND ANALYTICAL RESULTS

Two analytical models were used to analyze the three specimens statically and dynamically. First, each specimen was modeled as 2D-beam, and the system nonlinearity was represented by a lumped plasticity model. This representation was used to perform the nonlinear static (push-over) and dynamic analyses on the three specimens. SAP 2000 program [3] was used to perform the push-over analysis while DRAIN-3DX program [4] was used to perform the dynamic analysis. The lumped plasticity model required the calculation of moment-rotation at expected plastic hinges. Reinforcement slippage was calculated and included in the model in the form of additional rotation at the expected plastic hinges. The maximum shear deformations were also included by using the column shear area after cracking. The concrete shear area for the circular columns was developed based on the truss mechanism. It was also successfully used in predicting the column shear deformation in a previous study by Laplace [5]. The push-over analysis proved to have a good prediction for the specimen peak displacement and capacity. The details of the analytical work are illustrated in a report by Moustafa, et al [6].

Second, the strut-and-tie model was used to predict the specimen capacity and to interpret the behavior at the specimen D-regions, e.g., beam-column joints, hinge bases and plastic hinge zones. In both models, the effect of dynamic loading on the concrete and steel properties, so called the strain rate effect, was included. In the strut-and-tie model, equilibrium was maintained between external and internal forces. As shown in Fig. 7 the external loading carried by each specimen was distributed along the specimen cap beam since the cap beam carries the dynamic mass. As a first trial, the value of the external force was taken as the total shear demand of specimen columns when the column critical sections reach their flexural capacities. The yielding flexural capacities of the column top and bottom sections were calculated using the RCMC program [7]. The ACI- 318 code method [8] was followed in developing the strut-and-tie model. The specimen B- and D-regions were determined as shown in Fig. 7. The flexural forces at the end of the B-regions in specimen columns were determined using the RCMC program. The axial forces in each column were calculated from the specimen equilibrium when column top and bottom sections reach their flexural yielding. After several trials, the strut-and-tie model of each specimen was selected and the final member forces were calculated. The use of strut-and-tie model showed good agreement with the experimental results [6] and provided an understanding of the behavior of the beam-column joints and hinge base regions.



a: Modeling of Gravity and Seismic Loadings

b: Specimen D- and B-Regions

Fig. 7: Model Loading and Boundaries

CONCLUSIONS

Based on the experimental observations and analytical studies for the three specimens, the following conclusions were drawn:

- The tall and medium specimens behaved satisfactorily as their behavior was controlled by flexure whereas the short specimen behaved with combined flexure/shear mode.
- Sliding failure at the short column bases precluded the columns from reaching their maximum flexural capacity.
- In all specimens, flexural concrete spalling was well contained and the column-confined core was almost intact during high levels of loadings.
- In all specimens, the cap beam experienced limited cracking.
- Despite the simplicity of the beam-column joint details, they were sufficient to protect the joints from failure. In the three specimens, measured and observed results assured that the joint strength was significantly higher than the adjoining columns.
- Using simple analytical models (2D-beam with a lumped plasticity model) in SAP2000 program showed good correlation with the experimental results.
- Using the Takeda model in DRAIN-3DX program accurately predicted the nonlinear response of the flexurally dominated specimens. For the short specimen, however, shear models need to be included in the DRAIN-3DX models to produce good behavior prediction.

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