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Paper No. 668. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

BEHAVIOR AND REHABILITATION OF BEAM-COLUMN T-JOINTS IN OLDER REINFORCED CONCRETE BRIDGE STRUCTURES

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ABSTRACT

A series of experimental tests investigating the seismic behavior of reinforced concrete beam-column T-joints was recently completed at the University of California, Berkeley. The evaluated connection was representative of interior beam-column joints in multi-column bridge frames constructed in the 1950's and 1960's. Three one-third scale models, representing the as-built joint and two retrofit joints, were tested. The results of this research project are an improved understanding of the seismic behavior of lightly reinforced bridge T-joints, as well as verification of a design procedure for retrofitting this type of connection.

KEYWORDS

reinforced concrete; earthquake loading; retrofit; bridge design; seismic design; beam-column joint; t-joint; experimental testing; cyclic loading; bond.

INTRODUCTION

Recent earthquakes have exposed the weaknesses of older reinforced concrete beam-column bridge joints. The collapse of the upper deck of the Cypress Street Viaduct in Oakland, California during the Loma Prieta earthquake was attributed to the failure of exterior beam-column joints (Bollo, 1990). The Viaduct was constructed in the late 1950's when standard practice for bridge design did not specifically address joint design. At that time, design details for longitudinal reinforcement passing through or terminating in the joint were commonly based solely on anchorage requirements under gravity loading. Additionally, for ease of construction, transverse reinforcement was frequently discontinued in the joint region.

Researchers began a dedicated investigation of the design of beam-column joints for earthquake loading in the 1960's. Since then, the majority of this research has focused on new design of building joints and has not addressed the issues critical to retrofit design for existing bridge joints. Based on this research, current standard practice for the design of new building joints requires that transverse reinforcement be provided in the joint to transfer load and to confine the joint core concrete. However, it is often impractical to retrofit older, lightly reinforced joints with the volume of transverse reinforcement and reinforcement details recommended for design of new joints. Additionally, with older, lightly reinforced bridge joints, there are the concerns of deterioration of load transfer mechanisms due to the yielding of the members that frame into the joint, as well as development of adequate bar anchorage for member longitudinal steel passing through or terminating in the joint.

In California there is a significant number of older reinforced concrete bridges. Many of these bridges require retrofit of beam-column connections in order to ensure a ductile response to earthquake loading. With the goal of verifying a design procedure for retrofitting these joints, scaled models of an as-built beam-column T-joint (Model One) and two retrofit T-joints (Model Two and Model Three) were tested.

EXPERIMENTAL TEST MODELS AND TEST PROCEDURE

Engineering drawings of single level, multi-column reinforced concrete bridge frames designed and constructed in California in the late 1950's and early 1960's were reviewed to determine typical dimensions and reinforcement details. Based on this review, a prototype frame was designed (Figures 1 and 2). This frame included the following typical reinforcing details that were considered to be critical to the behavior of interior beam-column joints:

- Grade 40 (f_y = 317 MPa) column longitudinal reinforcing bars were anchored in the joint with straight development lengths of 20 d_b.
- Neither beam nor column transverse reinforcement was continuous through the joint region.
- The majority of beam bottom longitudinal reinforcement was not continuous through the joint.
- Column axial load was approximately three percent of nominal concrete capacity.
- Beams were designed so that maximum flexural reinforcement strain under gravity loads was approximately 25 percent of the yield strain.

During the 1950's and 1960's common practice did not dictate consideration of the relative flexural capacity of beams and columns framing into the joint, consideration of the force transfer through the joint region, or consideration of member shear capacity as a function of member flexural capacity. For this test program, the flexural reinforcement ratios of the prototype frame were adjusted to result in a frame that included the following target design features:

- The sum of the cap beam nominal flexural strengths exceeded the column nominal flexural strength (both projected to the joint center), although yield of the beam in positive flexure was expected prior to flexural yielding of the column. This target yield path was expected to result in particularly critical anchorage conditions for the column reinforcement.
- The maximum nominal joint shear stress was expected to be $22\sqrt{f_c}$ kPa, with f_c equal to the concrete compressive strength in kPa ($8.5\sqrt{f_c}$ psi, f_c in psi).
- To prevent shear failure of either the beams or the column, transverse reinforcement for both the beams and the column was designed to support the shear force corresponding to the member ultimate flexural strength. This resulted in the use of significantly more transverse reinforcement than is usually found in existing bridges from this time period.

Because of the size and strength of actual bridges it was not feasible to experimentally test in the laboratory an entire bridge frame. Therefore scaled sub-assemblages from the prototype frame were tested (Figures 1 and 2). By using a scaling factor of one-third, the test specimens were sufficiently large that it was possible to use the same construction materials in the models as were found in the actual structures. Sub-assemblages consisted of a single interior column, the beam-column joint and a portion of the cap-beam extending on either side of the joint. To model the internal force distribution, determined from an analytical model of the prototype frame, appropriate boundary conditions included the following (Figures 1 and 2):

- The moment and shear distribution at the beam-joint interface due to gravity loading was simulated by point loads acting on the top of beam. Gravity loads were reacted at the base of the column.
- Simulated earthquake loads were applied to the base of the column through an idealized pin. This loading was equilibrated by concentrated reactions acting perpendicular to the beam at the ends of the beam and by a compressive reaction acting along the axis of the beam at one end of the beam.

ANALYSIS OF EXPERIMENTAL DATA

To ensure that the results of this study could be compared with the results of previous research, nominal joint shear stress was computed as a measure of joint load. The ACI-ASCE Committee 352 (ACI-ASCE 352, 1985) procedures for calculation of joint shear force are based on the assumption that beams develop flexural strength at the joint face and that columns remain essentially elastic. The nominal joint shear stress calculated by this procedure acts on a horizontal plane at the mid-height of the joint. In bridge construction, where column yielding often is the desired mode of inelastic response, revised procedures for calculating the nominal joint shear stress are required, as described below.

To determine a nominal joint shear, the actions from the members framing into the joint were assumed to comprise member shear and tension and compression forces, with tension and compression forces representing the member moment and axial load, see Figure 3. Tension and compression forces at both the beam and column critical sections were assumed to act at the centroid of the extreme longitudinal reinforcement in the member (see Figure 3). The effective depth of the joint was defined to be equal to the depth of the member tension-compression couple (see Figure 3). Effective joint width was taken equal to the width of the beam, including retrofit if present. It is also noted that the member critical sections were located at the perimeter of the effective joint. Nominal horizontal joint shear was computed at mid-height of the joint and nominal vertical joint shear was computed at mid-span of the joint. Nominal joint shear stresses were computed as the nominal joint shear force divided by the appropriate effective joint area. Because in this study joint forces and joint areas were computed in a consistent manner, the resulting nominal vertical and horizontal joint shear stresses were equal (see Figure 3).

The effect on the joint of cyclic loading was evaluated through analysis of the load-displacement and nominal joint shear stress-strain relationships. The experimental load-displacement relationship was compared with that computed from a simple analytical model. Applied load corresponded to the simulated earthquake load applied transverse to the column axis by the actuator at the column base. Model displacement was defined as the displacement of the base of the column parallel to the beam axis. Analytical model strength was computed based on the flexural strength of the members. Member nominal flexural strengths were computed in accordance with ACI Committee 318 (ACI 318, 1989), with the exception that for Model One, beam positive nominal flexural strength was computed based on beam bottom longitudinal reinforcement being at the yield point. For Model One it was expected that beams would develop nominal positive flexural strength prior to the column; beam post-yield strength was computed based on the assumption that the beam bottom reinforcement carried the maximum measured tensile strength. Analytical model stiffness was computed assuming a rigid joint zone and constant stiffness for each member cross-section. For all members, with the exception of Model Three beams, cross-section stiffness was computed as the secant stiffness to the cross-section moment-curvature relationship at 75 percent of the nominal flexural strength. For Model Three beams, the cross-sectional stiffness was taken as the secant stiffness to the moment-curvature relationship at 75 percent of the maximum flexural load carried by the beams.

Nominal joint shear stress-strain relationships were also evaluated through comparison of the experimental and computed relationships. Nominal joint shear stress was computed from the measured load as previously discussed and joint shear strain was computed from deformation at the surface of the joint as measured by direct current differential transformers (DCDT's) and from deformation as measured by concrete strain gages embedded in the joint core. The experimental nominal joint shear stress-strain relationships were compared with that computed from the material constitutive relation.

Slip of column longitudinal reinforcement was measured as a means of evaluating the anchorage of this reinforcement in the joint. PVC tubing was used to leave a hole in the concrete extending from the top of the joint to the end of two column longitudinal bars anchored in the joint. DCDT's were used to measure the movement of the bar ends with respect to the top of the joint.

EXPERIMENTAL TEST MODEL ONE

Model One was the as-built model previously discussed (see Figure 2). When subjected to the simulated earthquake loading, the model did not perform in a ductile manner. Although the model carried the simulated gravity forces throughout the test, peak simulated earthquake load was only 66 percent of the nominal strength. The model exhibited a large reduction in stiffness and load capacity when cycled to relatively small maximum displacements and this degradation continued as testing progressed. Also during small displacement cycles, diagonal cracks formed in the cover concrete in the joint area; these cracks appeared to initiate in the concrete over the anchored column reinforcement and propagate towards the column compression zone. At the peak load, significant cracking occurred in the joint. These cracks appeared to delineate the anchorage zone of the column longitudinal reinforcement. During subsequent displacement cycles, damage continued to accumulate in the joint and the model began to lose simulated earthquake load carrying capacity. Testing of Model One was concluded when the model exhibited only minimal earthquake load capacity.

Figure 4 shows the experimental and computed load-displacement relationships for Model One, Figure 5 shows the experimental and computed nominal joint shear stress-strain relationships and Figure 6 and Figure 7 show the relationship between model displacement and slip of column longitudinal reinforcement. Model One carried a peak nominal joint shear stress of $14\sqrt{f_c}$ kPa, f_c in kPa ($5.4\sqrt{f_c}$ psi, f_c in psi). At the maximum load, column bar slip of 1.5 mm (0.06 inches) was measured. Bar slip accumulated as testing continued and the instrumentation maximum of 31 mm (1.2 inches) was measured prior to the final displacement cycles.

EXPERIMENTAL TEST MODEL TWO

Model Two was a retrofit as-built connection. The retrofit was designed following the testing of Model One and consisted of casting reinforced concrete bolsters on both sides of the existing as-built cap beam (see Figure 2). Bolsters were designed to increase beam flexural capacity so that under earthquake loading the column would develop nominal flexural strength while beams would remain essentially elastic. It was expected that the additional concrete volume in the joint region would result in a maximum joint shear stress comparable to the maximum stress developed in Model One and that additional concrete cover would increase bond stress capacity in the joint for the column reinforcement.

When subjected to simulated earthquake and gravity loads, Model Two maintained gravity load throughout the test and earthquake load carrying capacity to a nominal ductility of four. Model Two carried a peak nominal joint shear stress of $13.7\sqrt{f_c}$ kPa, f_c in kPa ($5.2\sqrt{f_c}$ psi, f_c in psi). However, damage accumulated in the joint. Also, measured maximum column bar slip was equal to the instrumentation maximum of 31 mm (1.2 inches).

EXPERIMENTAL TEST MODEL THREE

Following the testing of Models One and Two, a second retrofit design was implemented in Model Three. This retrofit consisted of the addition of post-tensioned concrete bolsters on both sides of the as-built beam (see Figure 2). As was the case for Model Two, the addition of bolsters was anticipated to increase beam flexural strength so that under simulated earthquake loading, inelastic deformation would be isolated in the column. The bolsters and post-tension force were designed to reduce the maximum nominal joint tensile stress to below the nominal concrete tensile strength. Additionally, it was expected that the addition of the post-tension force would improve anchorage of column longitudinal reinforcement in the joint by widening the zone of compression in the joint. Qualitative evaluation of the joint load transfer mechanism for Model One indicated that very little of the column reinforcement was anchored in the joint compression zone. Addition of the post-tension force in Model Three increased the zone of compression in the joint, thereby increasing the length of column longitudinal reinforcement developed in this zone and improving anchorage of this reinforcement.

Only the retrofit bolsters were post-tensioned, and force transfer between the existing beam section and the bolsters was achieved through shear friction at the concrete interface. Steel reinforcing dowels were epoxied into holes drilled in the existing beam, the reinforcing steel in the retrofit bolsters was placed around these dowels and the bolster concrete was cast. At the ends of the beam segments, these dowels were designed to ensure that the post-tension force would be evenly distributed between the bolsters and the existing beam at a distance of 1.25 times the beam depth from the beam-joint interface. Elsewhere along the length of the beam, these dowel bars were placed at a nominal spacing of 102 mm (4 inches) for the one-third scale model.

With the added post-tension force, a consistent method for calculating nominal joint shear stress was not entirely obvious. It was decided that at the beam critical section the post-tension force would be modeled as a single point force acting at the mid-height of the member. Based on this model, the nominal joint shear stress did not depend on the post-tension force. Nominal joint principal stresses were computed from the nominal joint shear stress and the nominal joint compressive stress, that was considered to be equal to the post-tension force distributed across the entire beam area.

Model Three behaved in a ductile manner when subjected to simulated earthquake and gravity loading. Throughout the laboratory test, the joint remained essentially elastic and flexural yielding was isolated in the column. The model carried a maximum simulated earthquake load of approximately 1.1 times the calculated strength at a displacement of approximately nine times the nominal yield displacement. Figure 8 shows the computed and experimental load-displacement relationships, Figure 9 shows the computed and experimental nominal joint shear stress-strain relationships and
Figure 10 and Figure 11 show the relationship between model displacement and slip of column longitudinal reinforcement. For this model, the maximum load corresponded to a nominal joint shear stress of $13\sqrt{f_c}$ kPa and a maximum nominal joint tensile stress of $7.9\sqrt{f_c}$ kPa, f_c in kPa ($4.8\sqrt{f_c}$ psi and $3.0\sqrt{f_c}$ psi, f_c in psi). For this model, maximum column bar slip was less than 0.5 mm (0.02 in.).

CONCLUSIONS

Consideration of nominal joint shear stress is not sufficient for evaluation or retrofit of beam-column T-joints in older reinforced concrete bridge frames. Evaluation and retrofit must include appraisal of load transfer mechanisms and anchorage of member longitudinal reinforcement in the joint. Nominal joint shear stress can be used in evaluation and retrofit design; however, because older reinforced concrete bridge joints are usually only very lightly confined by joint transverse reinforcement and by members framing into the joint, the nominal joint shear strength of these joints is relatively low. For post-tensioned joints, evaluation of nominal joint principal tensile stress provides a useful criterion for design. The distribution of stresses within the joint should be evaluated qualitatively to verify that yielding of members framing into the joint does not deteriorate joint load transfer mechanism and to verify that member longitudinal reinforcement terminating in the joint is anchored in compression zones. Retrofit design should focus on reducing bond stress demand, increasing bond stress capacity and reducing concrete nominal principal tensile stress in the joint as well as development of desirable frame yield mechanisms.

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ACKNOWLEDGEMENT

The views expressed in this paper were developed in part through research activities sponsored by the California Department of Transportation (Caltrans), but do not necessrily represent the views of the sponsor. The authors would like to recognize the participation of Li-Hong Sheng, Caltrans, Division of Structures, as well as fomer and current UC Berkeley graduate students Abe Lynn, Mark Aschheim, Graham Archer, Scott Campbell, Stephen Jirsa, Dawn Lehman, Silvia Mazzoni, Boza Stojadinovic and Wendy Taniwangsa.

FIGURES

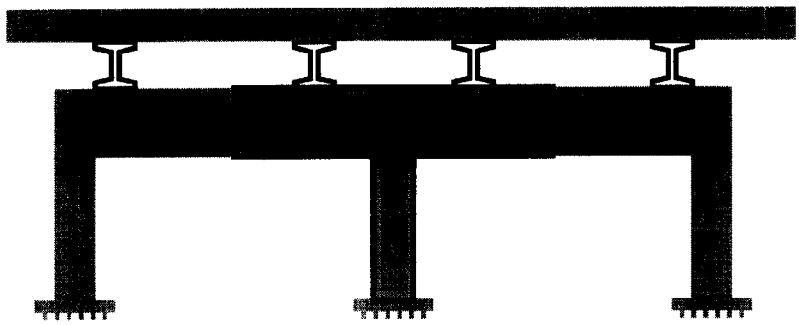


Figure 1: Prototype As-Built Bridge Frame and Experimental Test Sub-Assemblage

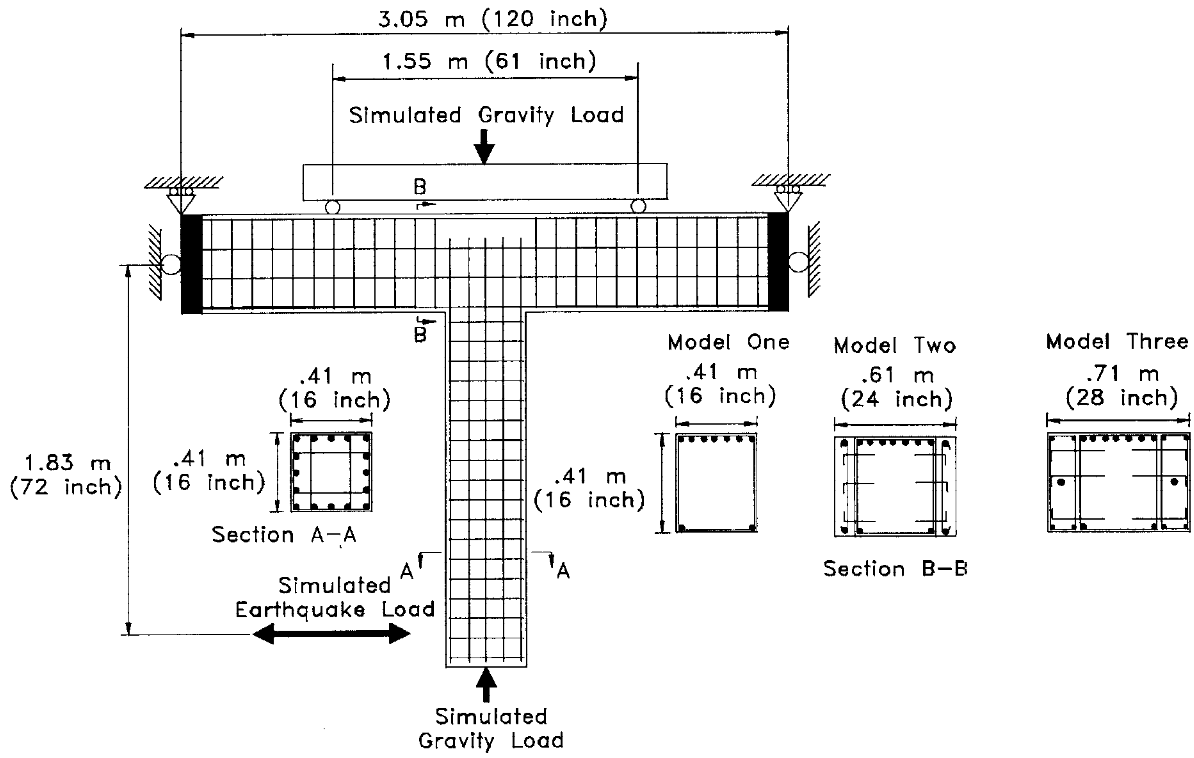
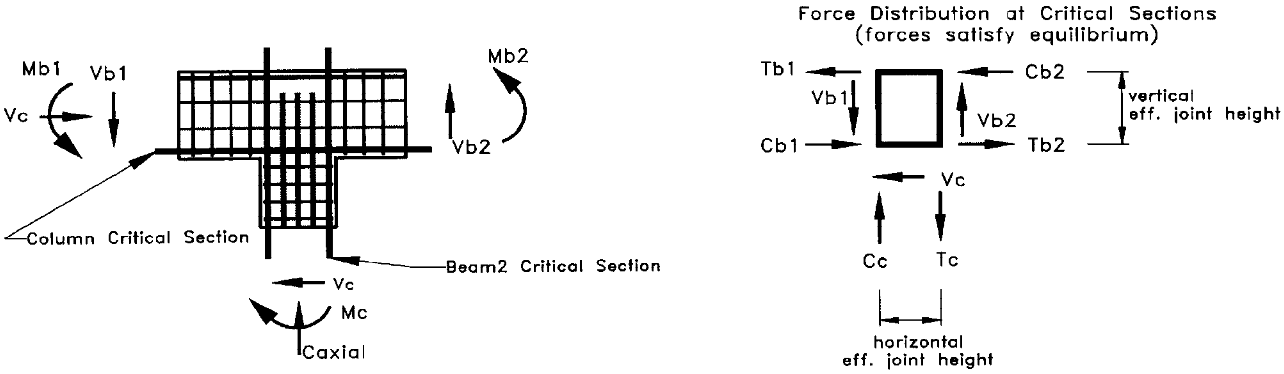


Figure 2: Idealized Loading of Experimental Test Models One, Two and Three



$$\text{nominal joint shear stress} = \frac{Tb1 + Cb2}{\text{horz. height} * \text{width}}$$

$$= \frac{Cc - Vb1}{\text{vert. height} * \text{width}}$$

Figure 3: Idealization of Joint Loads and Area Used in Computation of Nominal Joint Shear Stress

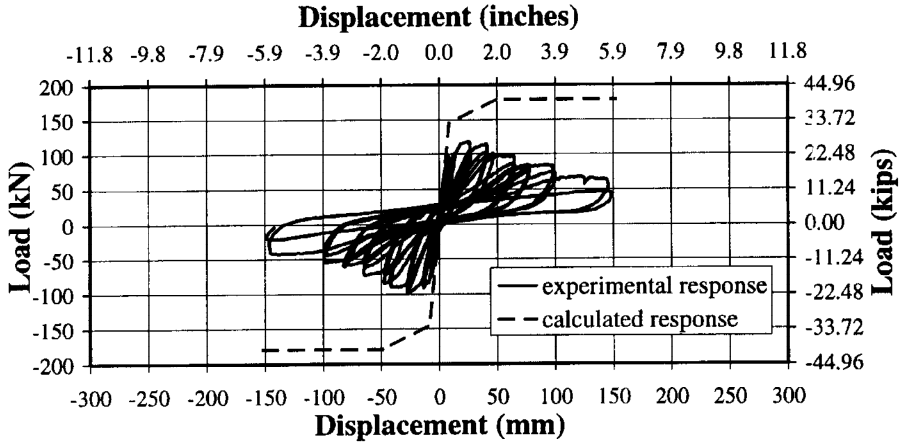


Figure 4: Model One Load-Displacement Relationship

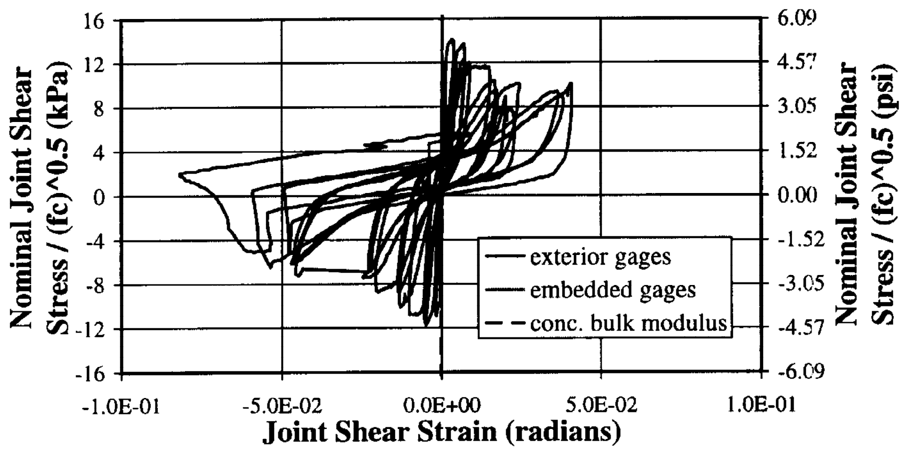


Figure 5: Model One Nominal Joint Shear Stress-Strain Relationship

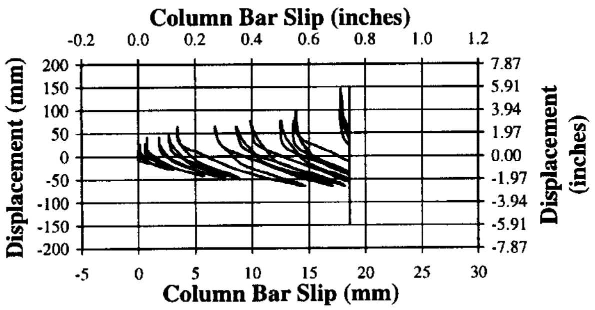


Figure 6: Model One Displacement Versus Column Bar Slip (Top Bar)

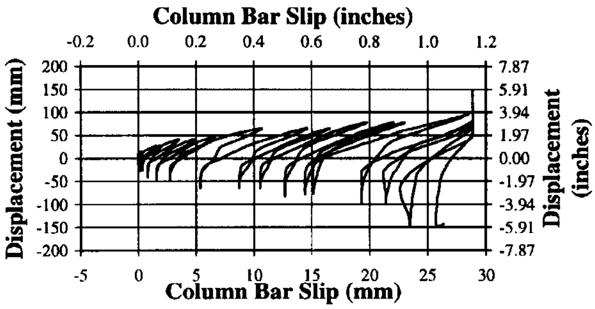


Figure 7: Model One Displacement Versus Column Bar Slip (Bottom Bar

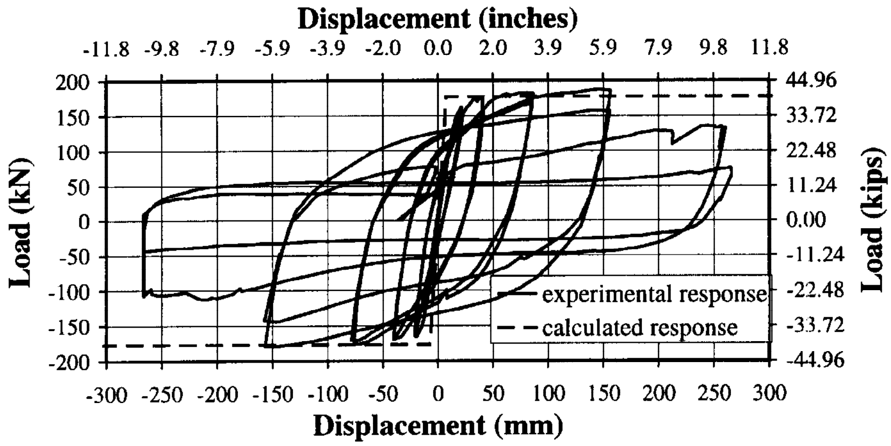


Figure 8: Model Three Load-Displacement Relationship

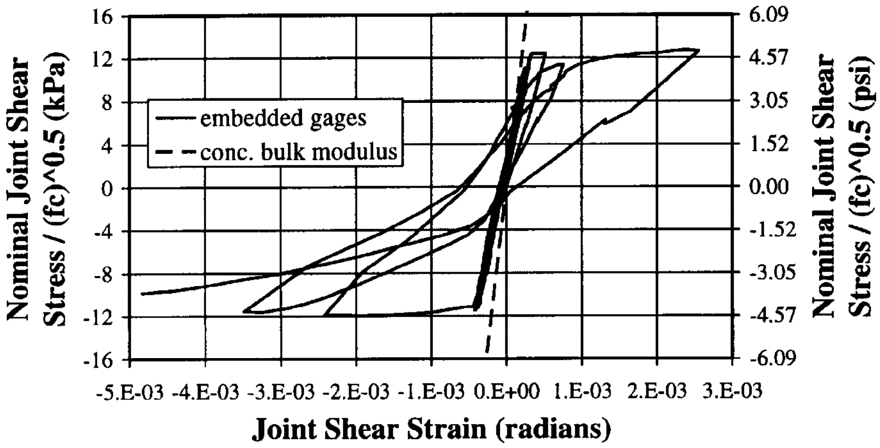


Figure 9: Model Three Nominal Joint Shear Stress-Strain Relationship

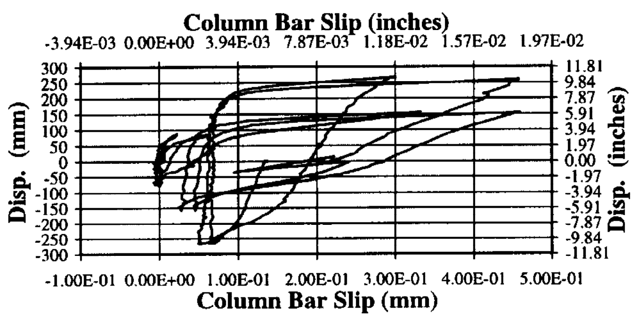


Figure 10: Model Three Displacement Versus Column Bar Slip (Top Bar)

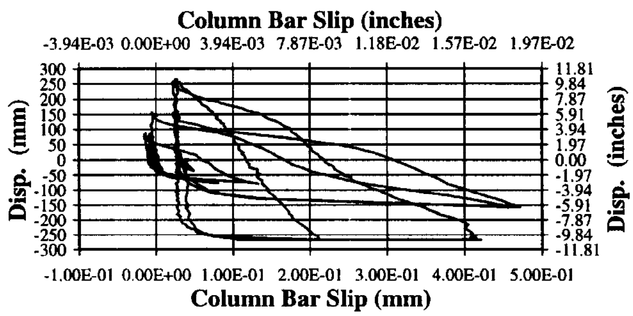


Figure 11: Model Three Displacement Versus Column Bar Slip (Bottom Bar)