EVALUATION OF REINFORCED CONCRETE BRIDGE JOINTS

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SUMMARY

Damage in recent earthquakes has resulted in increasingly conservative design of reinforced concrete beam column bridge joints. Current recommendations produce joint detailing which result in high levels of congestion of steel reinforcement and extreme difficulties in construction. Currently, a research project at the University of California, Berkeley is focusing on the development of a rational model to describe joint response to earthquake loading, a general design procedure for bridge joints, and a method of incorporating headed reinforcement into the design to improve joint constructability.

In order to accomplish the project goals, experimental investigations into the response of bridge joints to earthquake loading are being conducted. The investigation consists of quasi-static laboratory testing of eight reduced scale models of bridge joint components. The primary goal of this phase is to improve the understanding of joint behavior and to determine how conventional and headed reinforcement can be better utilized in improving joint response. Results from the first phase of the experimental study have found that California design strategies produce joints that are capable of supporting the formation of a column hinge mechanism, although at the expense of constructability. Headed joint transverse reinforcement proves to be a viable means of reducing construction difficulties without any decrease in joint performance. Some of the secondary issues being investigated are the effectiveness of strut and tie modeling and the effect of controlling slip of joint reinforcement. A parallel computational study of the first phase is being conducted; three-dimensional finite element models are being developed for further understanding of the joint behavior. Preliminary results and techniques for developing an effective three-dimensional finite element model are presented.

INTRODUCTION

Due to the catastrophic failure of bridge systems in the recent earthquakes of Loma Prieta, Northridge, and Kobe, there has been a great effort directed towards safer civil infrastructure in the United States and Japan. This has taken the form of retrofitting or strengthening existing bridges and increasing the design requirements for new bridge systems. While strengthening techniques and design requirements for beams and columns are well established [Park 1975], designing or evaluating the connection between the two is still in contention. The current methods of joint design are based on either a two-dimensional evaluation of the flow of stresses within the joint or through strut and tie methods which often neglect compatibility in their formulation. In general, reinforced concrete bridges are subjected to multi-directional ground motion. Therefore, response of bridge beam-column joints is predominantly three-dimensional (3D). Accurate evaluation of existing generic bridges and development of general design requirements for beam-column joints may require the use of 3D-models which take into account compatibility, equilibrium and the constitutive properties of the system. Many computational methods exist for modeling systems in three-dimensions, however, how well these models reflect the actual behavior of reinforced concrete bridges, particularly systems subjected to seismic loading, is not clear.

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This study evaluates current methods of design of bridge beam-column joints and investigates 3D finite element modeling methods for application on reinforced concrete bridges using available techniques and solution strategies. An effective procedure for modeling these systems in three-dimensions is presented. The results of these modeling techniques are compared to the response of reduced scale experimental bridge subassembly tests. The experimental specimens model the center portion of a three column bridge bent system with pinned column-to-footing connections (Figure 1). The sub-assembly consists of half the beam span on either side of the center column and the full column height. In summary, the present research aims toward fully understanding the structural response of reinforced concrete bridge beam-column joints through reduced scale experiments and 3D finite element analysis.

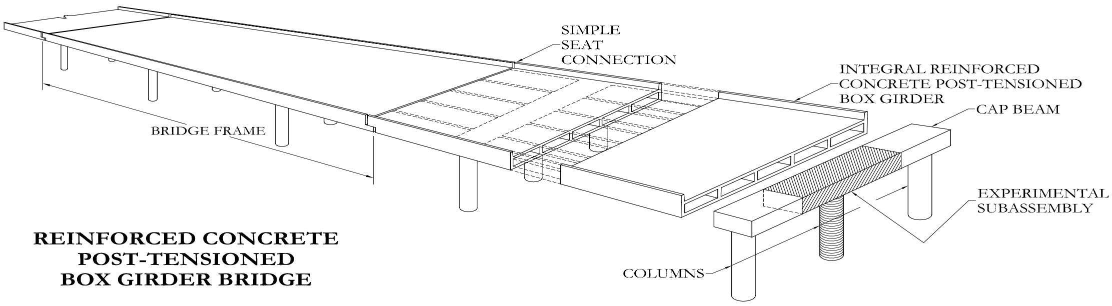


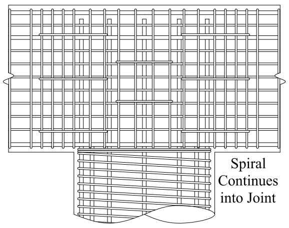
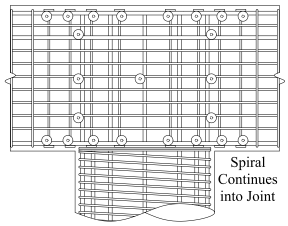
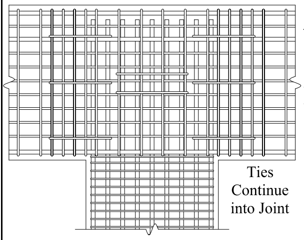
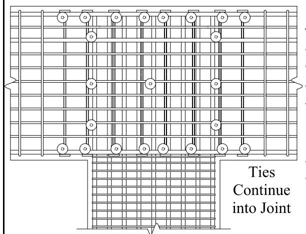
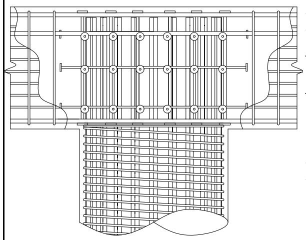
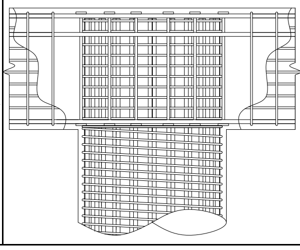
Figure 1: Bridge system under investigation

EXPERIMENTAL INVESTIGATION

The experimental facet of the program is divided into three groups: A, B and C. Groups A and B represent the first phase of the experimental study which will be presented in this paper (Table 1). Group A investigates current California bridge design methodologies as represented in the Bridge Design Specifications [Caltrans BDS 1995] and Memos to Designers [Zelinski 1995]. This test series consists of four test specimens, designated A1 through A4. Specimen A1 and A2 are of a round column configuration (typical of California construction) and A3 and A4 of a square configuration. As required by Caltrans standards, the specimens are all designed to have an ultimate capacity reliant upon the flexural strength of the column. The focus of this series is three-fold: First, the effectiveness of joint design requirements is evaluated in specimen A1 and A3; Second, the application of headed reinforcement for use as transverse joint reinforcement is quantified in specimens A2 and A4; and third, the behavior of square column – rectangular beam configurations is evaluated for the development of simplified analytical models.

Upon the completion of group A, evaluation of the joint region under elevated demands was conducted and two additional specimens were constructed and tested. Both specimens consisted of a circular column and rectangular beam. The specimens were chosen to model tall bridge structures which typically have a beam depth to column depth ratio on the order of 0.8. As a result, the beam depth is reduced from that used for group A. In addition, the quantity of longitudinal reinforcement is increased and the quantity of transverse joint reinforcement is decreased. As a result, the joint demands from shear and bond development are highly elevated (max vertical joint shear stress = $7\sqrt{f'c}$ to $8\sqrt{f'c}$ for group A versus $11\sqrt{f'c}$ for group B where stress is computed from the applied load and the resulting tensile column force on the joint). The specimens are constructed using normal weight concrete and grade 60 reinforcement, the measured properties are included in Figure 9. The second phase of this experimental research includes group C, where the box girder is included and bi-directional loading is applied. Use of 3D models will be essential for investigating group C specimens. Direct application of the developed 3D FEM models will be conducted and documented in future publications.

Table 1: Phase 1 - Experimental program matrix

 A1 – Caltrans Design Circular column ρ=2.3% Beam / column depth ~ 1.0 ρ_{jv}=0.17%, ρ_{jh}=0.22%	 A2 – Caltrans Design w/ Headed joint & column longitudinal reinforcement. Circular column ρ=2.3% Beam / column depth ~ 1.0 ρ_{jv}=0.15%, ρ_{jh}=0.21%
 A3 – Caltrans Design Square column ρ=2.3% Beam / column depth ~ 1.1 ρ_{jv}=0.17%, ρ_{jh}=0.26%	 A4 – Caltrans Design w/ Headed joint & column longitudinal reinforcement. Square column ρ=2.3% Beam / column depth ~ 1.1 ρ_{jv}=0.16%, ρ_{jh}=0.27%
 B1- Lower Bound Design w/ headed horizontal & vertical joint reinforcement. Circular column ρ=3.1% Beam / column depth ~ 0.8 ρ_{jv}=0.38%, ρ_{jh}=0.36%	 B2 – Lower Bound Design w/ spiral & headed vertical joint reinforcement. Circular column ρ=3.1% Beam / column depth ~ 0.8 ρ_{jv}=0.38%, ρ_{jh}=0.29%
Note: ρ is defined as the column longitudinal reinforcement ratio, ρ_{jv} is the joint vertical volumetric reinforcement ratio and ρ_{jh} is the joint horizontal volumetric reinforcement ratio.	

Testing of the sub-assemblies was performed upside-down (Figure 2). The dead load of the bridge system was modeled by applying an axial load to the column base (5% of axial capacity) and reacting against the top of the cap beam. The locations of the cap beam reaction were chosen to provide the same shear and flexure at the joint face as that produced by the uniform dead load in the real bridge system. Transverse loading was applied to the system at the column base under displacement control. The load was cyclically applied in a quasi-static manner to increasing levels of peak displacement: 0.1, 0.25, 0.50, 1.00, 2.00, 4.00, 7.00, and 10.00 inches. The subassembly was modeled with pin-roller boundary conditions as illustrated by the insert in Figure 2. A roller was chosen to allow for free dilation of the joint.

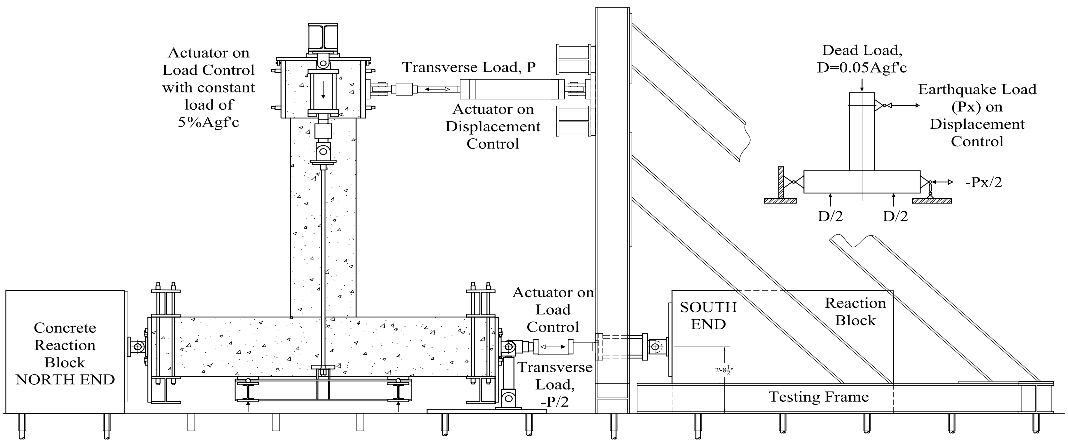


Figure 2: Test elevation and structural model

Experimental Results

Brief discussion of results from phase 1 of the experimental program is presented in this section. Figure 3 presents comparison of the force – displacement relationships obtained from the six specimens tested in phase 1. From this comparison one can infer the following:

1) The current Caltrans designs (presented by specimens A1 and A3) behaved well as far as adequate energy dissipation (full hysteretic loops up to maximum applied lateral displacement), ductility and strength degradation.

2) Square columns tend to show more pinching than circular ones. This may be attributed to less effectiveness of confinement and less efficient distribution of longitudinal reinforcement.

3) Use of headed bars is very beneficial where almost exactly similar behavior is obtained with improved constructability (Although the same volume of steel is used, constructing the specimen with headed bars was more efficient); compare the behavior of A1 to A2 and A3 to A4.

4) Joints with much higher shear demand (B1 and B2) showed significant pinching with strength degradation at small displacements (less ductility).

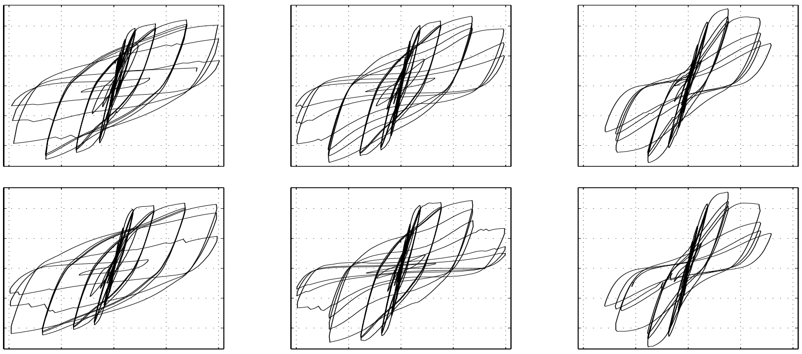


Figure 3: Force - displacement hysteretic behavior

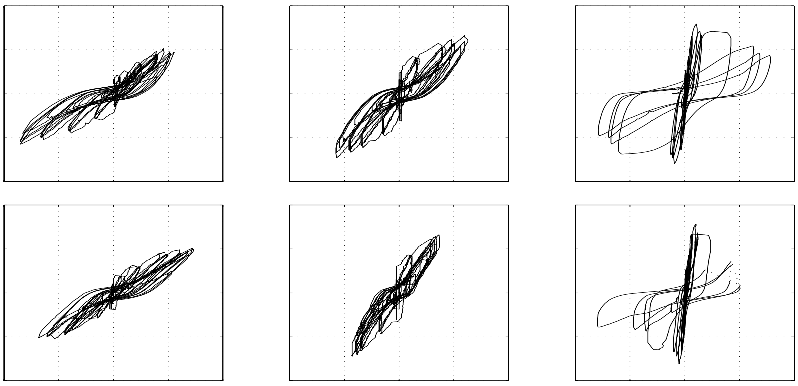


Figure 4: Joint shear stress-strain relationship (Note the strain scale difference between A and B)

Closer investigation of the shear deformation of the joint is illustrated by Figure 4. This figure gives results of shear stresses versus shear strain obtained from external measurements within the joint region. Group A showed relatively low shear strain within the joint region. The use of headed bars for group A lead to: 1) reduction of shear strain for the square column specimens, and (2) better behavior of the circular column specimens in terms of less pinching. On the other hand for large joint shear demand cases (specimen B1 and B2), permanent shear strains were observable with the spiral reinforced joint having significantly lower energy dissipation (for joint shear deformation) before failure than the joint confined with headed reinforcement. It should be noted that the joint horizontal volumetric ratio for the specimen with headed bars (B1) was 25% higher than that of the spiral (B2). This may be attributed to the less confinement provided by the spirally reinforced joint. It is expected that the larger the width of the joint region, the less effective the spiral is in confining the joint. In the case of large joint geometry, as is common in bridges, confinement is better introduced by a horizontal grid of headed bars.

The experimental study has shed some light on the complexity of the spiral behavior of bridge beam-column joints. For further understanding of the behavior, computational models calibrated with the test results were conducted. This effort has been undertaken using three dimensional (3D) finite element analysis as discussed in the following section.

COMPUTATIONAL INVESTIGATION

The finite element models using tri-linear brick elements are discretized to match the boundary conditions and geometry of the tested specimens. The finite element discretization and boundary conditions are shown in Figure 5 and 6. The gross dimensions and material properties are chosen to agree with that of the as-built sub-assembly. Dead load is applied as a uniform load on the top of the column and reacted on the bottom of the beam. The location and magnitude of the reactions are chosen to match the loads applied during the tests. Transverse loading is applied as a discrete nodal displacement at the center of the top reaction block in the x – direction (parallel to the beam axis). Both ends of the beam are modeled as pin supports. It should be noted that the pin supports of the beam are modeled using space truss stiff elements similar to the test setup. In the experiment the axial load of the beam is maintained under load control to be P/2 in each span, where P is the lateral load corresponding to the applied lateral displacement. This loading arrangement is not accounted for in the current analysis.

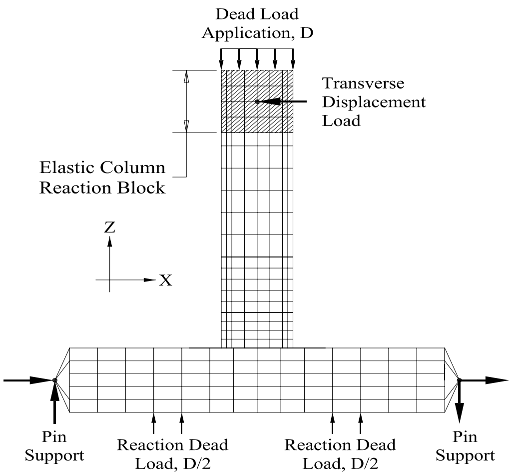


Figure 5: F.E. model boundary conditions and loading

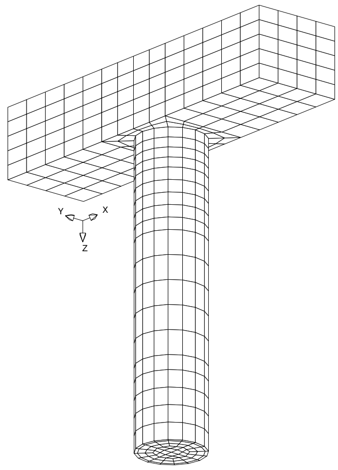
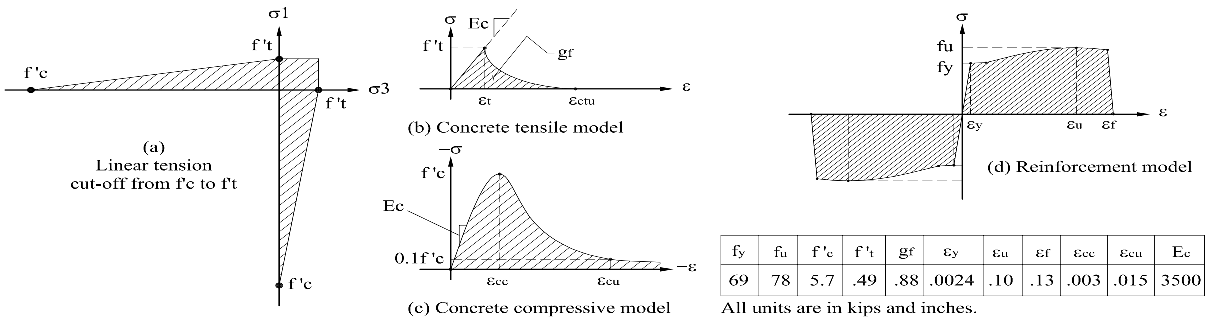


Figure 6: Model discretization

In general, reinforcement can be modeled by one of two methods. The first method, which is less computationally demanding, involves the use of embedded or smeared reinforcement. The second method, more computationally expensive, involves separate discretization of the reinforcement. The second model allows for the investigation of bond-slip behavior of reinforcement with respect to the surrounding concrete. In this case, reinforcing bar can be modeled as a truss element attached to the adjacent concrete element through a series of bond-slip elements. This technique carried out over the entire system becomes computationally expensive. The discussion presented in the remaining sections focuses on the use of the embedded reinforcement formulation. Combination of the two techniques where discrete modeling is only used in regions of potential slip and the rest of the reinforcement is embedded is currently under investigation.

Material Models

The constitutive relationships used for the finite element models are shown in Figure 7. The reinforcement uses Von Mises yield hardening criteria with constitutive models matching the behavior determined from testing (Figure 7d). Concrete is modeled using either Von Mises or Drucker Prager yield criteria for compression and a tension cut-off from the concrete compressive strength, f'c, to the concrete tensile strength, f't (Figure 7a) for tension. The concrete compressive behavior models the behavior obtained from displacement controlled testing of concrete cylinders (Figure 7c). The concrete tensile behavior uses the Hordijk model [Hordijk 1992] (Figure 7b). It consists of elastic response to the tensile capacity followed by a nonlinear unloading branch.   Cracking is modeled using both multiple fixed crack and rotating crack formulations [Rots 1988]. The results presented in this paper are limited to the fixed crack model.



fy	fu	f 'c	f 't	gf	εy	εu	εf	εcc	εcu	Ec
69	78	5.7	.49	.88	.0024	.10	.13	.003	.015	3500

All units are in kips and inches.

Figure 7: Constitutive concrete and reinforcement models (Note: graphs are to different scales)

Computational Results

The presented finite element results are obtained from the program DIANA* [Witte 1998]. The comparison of the global load-displacement behavior is shown in Figure 8. The finite element model envelopes the experimental response. As discussed earlier, the experimental tests are performed cyclically while the finite element analyses are monotonic. When concrete systems are subjected to cyclic loading the entire system undergoes tension - compression reversals. As a result, cracks open and close leading to a greater rate of system stiffness degradation than that observed under monotonic loading. Load reversal requires more sophisticated constitutive modeling. This issue is being investigated for better representation of the loading conditions.

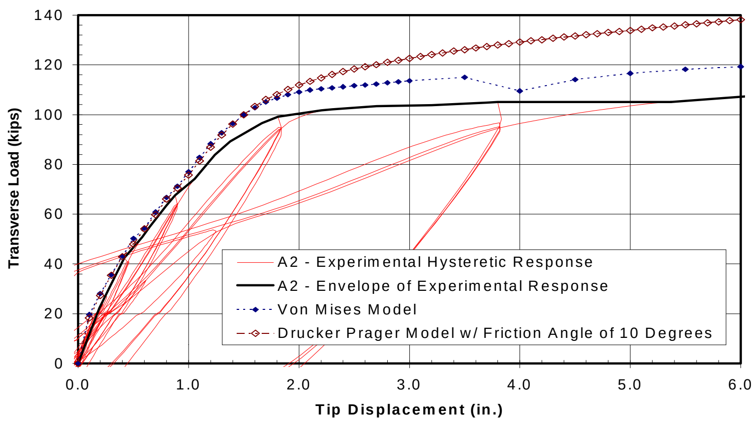
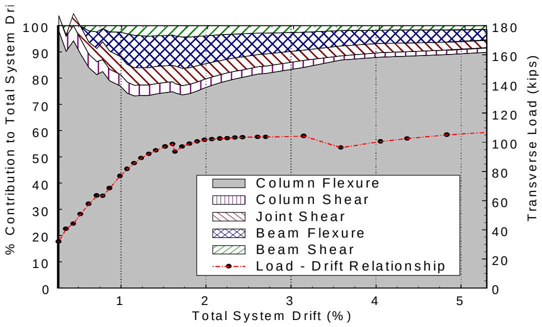


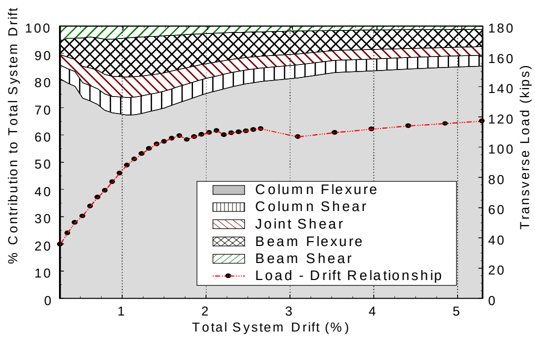
Figure 8: Comparative load displacement behavior

To evaluate the accuracy of the finite element model using Von Mises yield criterion for the concrete for specimen A2, the contribution to the total system drift is evaluated at each level of applied total drift. In the experimental evaluation, the contributions are computed using an array of external displacement transducers. For small levels of drift to 0.5%, inherent errors in the transducer measurements lead to an overestimation of the total displacement (Figure 9a). Nevertheless the overall trend of the displacement contributions agrees with the observed behavior. Initially, the column flexure is the predominant contributor to the displacement as initial cracks form in the column. This is followed by cracking of the beam section at 1% drift. Following this level, the inelastic column yielding and spalling takes place, leading again to a predominance of column flexural response. Using the computed nodal displacements from the finite element analysis, the component contributions to the total drift from the analytical model can be determined. As shown in Figure 9b the analytical model appropriately replicates the behavior observed in the experimental study. The largest error being the under-estimation of column flexure. As discussed earlier, to create a model which is computationally simple but accurate, tri-linear brick elements were used. Linear elements are inherently stiff in modeling flexure. As a result, the column flexure is underestimated on the order of 5%.

* *DIANA 7 (DISplacement ANAlyzer version 7) is a finite element code developed at TNO Building and Construction Research in the Netherlands.*



(a) Experimental sub-assembly



(b) Finite element model

Figure 9: Contributions to total system drift (Specimen A2)

The analytical model shows that the use of embedded reinforcement provides a good estimation of reinforcement strains for the analyzed experimental model (A2). As shown in Figure 10, strain values at levels beyond yield are properly estimated both in the column section as well as within the beam-column joint region. Note that, due to loss of a few strain gages during the experiment, only four measured strains are presented.

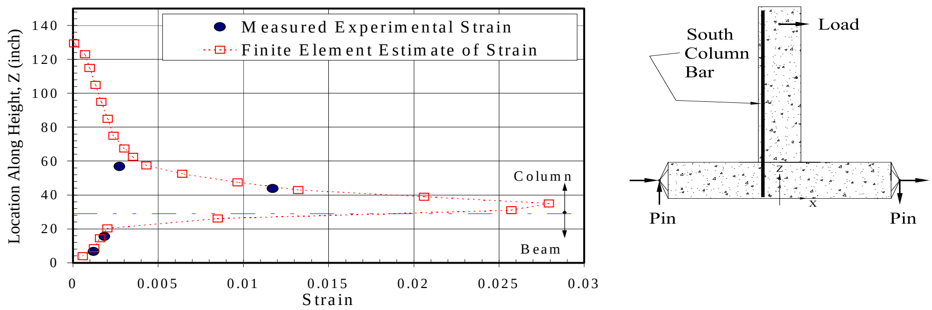
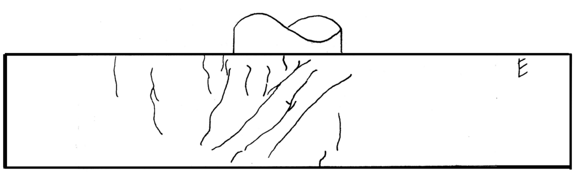


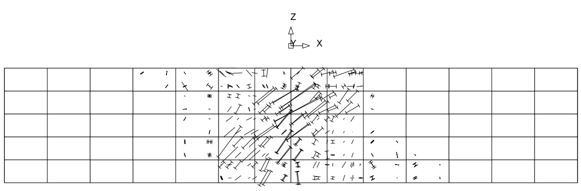
Figure 10: Strain in southern exterior column longitudinal reinforcement (3.6% drift - Specimen A2)

A comparison of experimental and analytical results shows that the multi-directional fixed crack formulation provides a good estimation of crack formation in a three-dimensional model. Figure 11 illustrates the crack behavior in the beam at 4.0-in. displacement (3.6% Drift) for specimen A2. This level corresponds to a displacement ductility of approximately 4.0, half of the ultimate displacement capacity. Figure 11(a) represents the extent of beam cracking present on the East face of the experimental sub-assembly. The estimated orientation of cracks in the analytical model is shown in Figure 11(b). As shown, the diagonal joint cracking pattern observed in the experiment is captured by the analytical model. Furthermore, the crack strain is largest along the central diagonal joint crack, as observed in the experimental investigation.

Figure 13 demonstrates the importance of 3D finite element modeling. In this figure, contour plots of the calculated transverse strains within the joint region are shown. These results are given at an applied lateral drift of 4%. It is observed that regions of high tension occur close to the cap beam longitudinal faces. These regions of high transverse tension indicate potential location of splitting cracks in the longitudinal direction of the cap beam; this behavior was observed in group B specimens where larger demand is placed on the joint region. From these preliminary results, one can infer the importance of 3D modeling for further understanding and accurate modeling of the models of failure of beam column joints.



(a) Experimental



(b) Analytical

Figure 11: Crack modeling of beam response at 3.6% drift

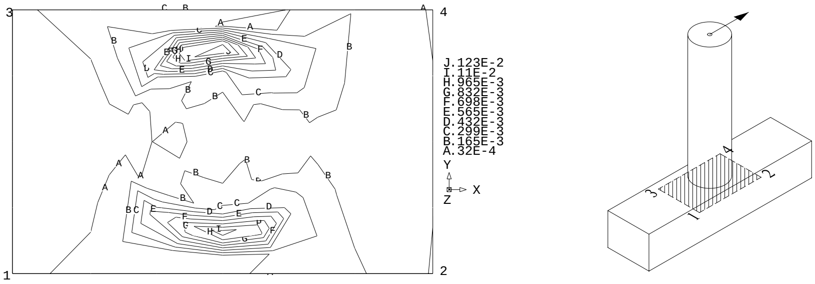


Figure 12: Contour plots of strains in the transverse direction (Y-axis) within the joint region

CONCLUDING REMARKS AND FUTURE EXTENSION

Current design standards in the state of California produce joints which have adequate ductility and serviceability. This, however, is done at the expense of constructability resulting in highly congested bridge joint reinforcement details. Test results show that the use of headed transverse reinforcement as opposed to conventional transverse reinforcement provides effective confinement of the joint region, allowing for less congestion with comparable levels of performance. Investigations have also shown that confinement of the joint may be better achieved through the use of headed lateral joint transverse reinforcement than the continuation of the column spiral into the joint region.

The used finite element models are effective in capturing the global as well as local behavior of the experimental models. The developed finite element model will be extended to allow for bond – slip and cyclic loading for more detailed evaluation of response. Results from group C (specimens with integrated box girder) will be used to further refine the model. Once the finite element model is fully verified, investigation of effect of several parameters (not covered in the experimental study), e.g. effect of prestressing, arrangement of joint reinforcement, loading history and rate, will be conducted. Finally the combination of the experimental results and the finite element investigation will be used to develop effective simplified tools and guidelines for use in practical and efficient design of reinforced concrete bridge joints.

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