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INFLUENCE OF PRESTRESS ON COLUMN/CAP BEAM TEE JOINTS

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ABSTRACT

Current design methods of bridge joints can result in unacceptable congestion of reinforcing steel within the joint. As a part of an ongoing research program on column/cap beam connections, three tee joints with circular columns were designed and tested under simulated transverse seismic loading at the University of California, San Diego (UCSD). Reinforcement requirements within the joints of the test units were significantly reduced compared to the current methods by (a) detailing them using force transfer mechanisms rather than designing for the maximum joint shear forces, and (b) introducing prestress in the cap beams. Maintaining a similar steel content in the columns, the amount of cap beam prestress was varied between the units from zero to a fully prestressed design. The test unit with a fully prestressed cap beam was also used to demonstrate a precast fabrication of a concrete multi-column bent. The design details of the test units and their performance with respect to a poorly detailed as-built specimen are presented in the paper.

1. INTRODUCTION

In the pre-1960 era, concrete bridge joints which were designed in California were provided with no shear reinforcement. A lack of understanding of the joint behavior under seismic loading led to a poor estimate of the joint shear forces. Recognizing that the joint shear forces are typically 4-5 times higher than the shear in members framing into the joint, bridge joint detailing has been gradually improved over the past decades. The probable consequence of poor joint detailing was, however, not appreciated until the 1989 Loma Prieta earthquake [5] which resulted in complete failure or distress to a number of cap beam/column connections. Given that not all of the damaged joints were from pre-1960 structures, a vigorous research program was initiated by the California Department of Transportation (Caltrans). The focus of this research was to (a) examine the competence of as-built concrete bridge joints, (b) seek retrofit measures for poorly designed connections, and (c) establish efficient alternative reinforcement details for the cap beam/column connections. The emphasis on investigating alternative reinforcement detailing was also placed because the conventional methods require a considerable amount of reinforcement within the joint, causing congestion problems.

This paper describes an ongoing investigation conducted at UCSD, in which alternative reinforcement details are sought for seismic response of cap beam/column tee connections. Of particular interest in this study is to investigate the influence of cap beam prestress on the performance of tee joints under seismic loading.

2. DESIGN PROCEDURE

2.1 Current Practice

In current building design practice, the reinforcement content of the joint region is determined based upon the maximum shear forces developed within the joint at the ultimate strength of the structure. In addition, several different constraints such as limiting maximum joint shear stress, satisfying minimum confining reinforcement provisions, and requiring minimum development length for longitudinal steel are imposed to force transfer mechanisms and cap beam prestressing are presented for bridge tee joints with circular columns in this paper. Large scale laboratory tests on three tee joints showed that the proposed details are efficient alternatives to the current design practice. The test units which were designed with reinforced concrete, partially prestressed and precast fully prestressed cap beams, contained significantly less joint reinforcement than is recommended by conventional design procedures.

The test models were redesigns of a pre-1960 interior cap beam/column joint from the Santa Monica Viaduct in Los Angeles. The redesigned joints produced significant improvement in performance with respect to the as-built joint. A balanced failure of the column and joint was observed for the first unit with a fully reinforced concrete cap beam. Better performance of the joint and complete failure in the column are anticipated in 'real' situations where additional vertical stirrups are placed on the sides of the cap beam near the column axis to cope with longitudinal response. These stirrups were deliberately omitted in all three redesigned units. The presence of these vertical stirrups would provide further resistance to dilation of the joint. Also, if the vertical reinforcement is provided in the form of closed ties, additional confinement to the joint would be obtained, further enhancing the joint performance.

Since the longitudinal column reinforcement was directly anchored into the diagonal strut for the two prestressed joints, there was no external vertical reinforcement provided, thus resulting in further reduction in the horizontal joint reinforcement when compared to the fully reinforced concrete unit. Despite the reduced level of joint reinforcement, it was apparent that the use of cap beam prestress greatly improved the joint performance. The construction procedure adopted for the third unit demonstrated that precast construction is a viable option for building multi-column bents.

ACKNOWLEDGMENT

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ensure satisfactory performance of the joints. These guidelines were developed empirically based on laboratory tests on beam/column joints from multi-story building frames. Further, the joint performance of these tests was interpreted by isolating the joint shear force, rather than examining the complete rational mechanism responsible for force transfer through the joint.

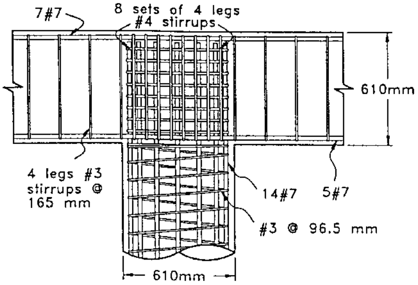


Figure 1: Joint detailing based on the maximum joint shear forces (1 inch = 25.4 mm).

The current ACI code [1] and the regulatory document governing bridge design [7] do not provide any specific guidelines for seismic design of bridge cap beam/column connections. When a design philosophy based on the maximum joint shear forces is considered for a bridge joint, unacceptable amounts of joint reinforcement often occur. This is expected because overstrength moment capacity of a bridge column, which dictates the maximum joint shear forces in bridge joints, is typically higher than that of a beam in a building frame due to high axial load and large longitudinal steel content.

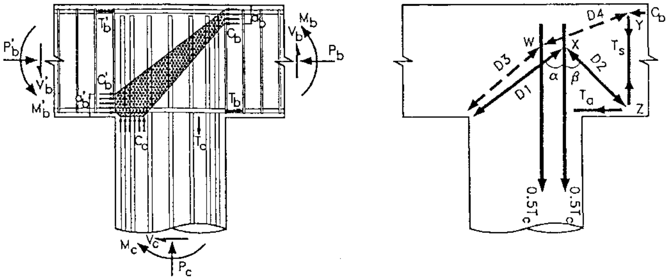
An example of steel congestion in a bridge joint is depicted in Figure 1, where the tee joint shown in Figure 5b was redesigned for the maximum joint shear forces. Since the vertical and horizontal joint shear forces are equal, similar reinforcement was provided in both directions. As seen in the figure, the amount of reinforcement required within the joint is markedly high, posing construction difficulties. Further, the embedment length provided for the column bars into the joint does not satisfy the ACI code recommendation [1]. For straight bar anchorage, the code requires that the cap beam depth be increased by 40 percent. For the design of the joints in the test units of the current study, a procedure outlined in the following section was used. This resulted in considerable reduction in the joint reinforcement and demanded no increase of the cap beam depth. The performance of the test units is detailed in Section 4.

2.2 Proposed Method

Considering various failure mechanisms, Priestley [3] presented several alternative force transfer mechanisms and appropriate reinforcement details for knee and tee joints of single-level bridge structures. In each mechanism, it was ensured that longitudinal column reinforcement was adequately anchored into the joint such that the overstrength moment capacity of the column could be developed. This allows plastic hinges to form, as typically required, in the columns adjacent to the joints, hence developing a desirable inelastic mode of response for bridge structures. Detailing the joint region by identifying a force-transfer mechanism generally requires less amount of joint reinforcement than that recommended by the current design procedures. When a design is based on a force transfer mechanism, a nominal joint reinforcement should be placed to limit any significant joint damage.

A mechanism requiring the least amount of shear reinforcement within the joint was chosen as the basis for the investigation reported in this paper. In order to understand the force transfer through the joint, consider the compression and tension forces induced in members framing into a tee joint with a circular column as shown in Figure 2a. The transfer of the compression in the joint is represented with a single diagonal strut with straight boundaries. As noted previously, a desirable inelastic response of the structure can be ensured if all the longitudinal column bars are adequately anchored into the joint. Because of large strut depth in the center and bottom left corner of the joint, if it is assumed that the column bars in compression, and tension bars carrying 50 percent of the total tension force ($0.5T_c$), located furthest from the tension side of the column, are directly anchored into the joint diagonal strut, then a mechanism is only required to transfer the remaining 50 percent column tension force. The extreme tension bars are embedded into the diagonal strut with insufficient anchorage length, and hence the mechanism should be aimed at providing resistance against these reinforcing bars pulling out of the joint when subjected to high tensile forces.

A force transfer mechanism which requires external vertical shear reinforcement outside the joint, as shown in Figure 2b, is considered to be appropriate for providing adequate clamping of the extreme column tension bars while requiring little reinforcement within the joint region. The external reinforcement provided outside the joint assists in developing a compression strut (D2) outside the joint. It is intended that the extreme column tension bars can be anchored into the joint by a clamping effect, resulting from struts forming both inside (D1) and outside (D2) the joint. Note that, in detailing the joint, it was preferred that the column longitudinal reinforcement be anchored into the joint with straight bars, as bends or hooks at the top of the column bars can also cause congestion of steel in the joint region.



(a) Idealized joint strut (b) A force transfer mechanism.

Figure 2: A reinforced concrete tee joint with a circular column.

The amount of reinforcement required to support the above mechanism is obtained assuming that the vertical components of struts D1 and D2 are equal, and that each of these components supports $0.25T_c$. By approximating the tension force of a circular section at its ultimate strength to:

$$T_c \approx 0.5\lambda_0 A_{sc} f_{yc} \tag{1}$$

where λ_0 ($\approx$1.3) is the overstrength factor of the column longitudinal steel, A_{sc} is the total area of the column bars and f_{yc} is the yield strength of longitudinal column reinforcement, the following reinforcement detailing is recommended [3,4]:

- Area of external vertical stirrups required outside the joint is 16 percent of A_{sc}. It is suggested that the external joint reinforcement be placed over a distance equal to half the cap beam depth. The reinforcement provided to resist shear in the cap beam is not likely to be fully utilized under positive moment, and thus the reserve capacity of the cap beam shear reinforcement can be supplemented towards the area of steel required outside the joint.

- Although no vertical joint reinforcement is required by the mechanism, vertical reinforcement amounting to 8 percent of A_{sc} is placed within the joint primarily to avoid severe joint cracking and to provide lateral resistance against buckling of the beam bars.

- Volumetric ratio of the joint horizontal reinforcement, ρ_s, is obtained as follows:

$$\rho_s = \frac{0.3 A_{sc} \lambda_o f_{yc}}{f_{yh} l_a^2} \tag{2}$$

where l_a is the anchorage length of the column bars in the joint, and f_{yh} is the yield strength of joint spiral. This provision accounts for the unbalanced horizontal force induced at node X by struts D1 and D2.

- A minimum ρ_s value as given by eq. (3) is required to ensure some tensile resistance when cracking occurs in the joint region;

$$\rho_s = \frac{0.29\sqrt{f_c'}}{f_{yh}} \tag{3}$$

where f_c' is the compressive strength of concrete in MPa. This minimum joint spiral requirement is about 50 percent of that recommended in the ACI code [1] for building joints if a compressive strength of 34.5 MPa (5 ksi) is considered.

- Additional bottom longitudinal beam reinforcement equivalent to 8 percent of A_{sc} should be provided for the stability of strut D2 at node Z (Figure 2b).

- Column bars should be extended into the joint as close as possible to the top of the beam with a minimum embedment length, l_a, as given by the following equation:

$$l_a \geq 0.30\, d_b f_{yc} \big/ \sqrt{f_c'} \tag{4}$$

where d_b is the diameter of longitudinal column reinforcement in mm.

Considering equilibrium at node Y (Figure 2b), it is obvious that the compression force C_b no longer acts horizontally at the cap beam joint interface. Instead, the tension force T_s redirects C_b towards node W. The change of direction of the strut creates an additional clamping effect for the longitudinal tension reinforcement, particularly for those located furthest from the tensile side of the column.

If the cap beam is prestressed, a broader compression strut develops within the joint even at the ultimate strength of the column as shown in Figure 3. The depth of the compression strut is generally large enough to directly anchor all the column longitudinal reinforcement into the joint. This can be ensured by comparing one third of the reinforcement development length (see eq. 4) with the estimated strut depth at the section where the most extreme tension bar is located. In equation 4, a conservative value for the average bond stress was considered, but the maximum average bond stress is typically about three times the value assumed in this equation [4,6]. If the column bars can be directly anchored into the joint, the overstrength capacity of the column can be developed without any significant strength degradation of the system, and no additional

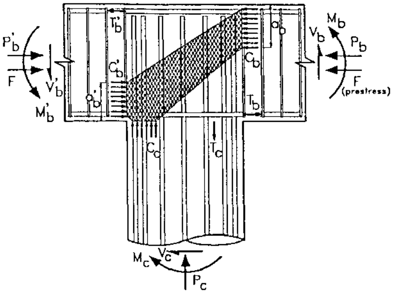


Figure 3: Idealized joint strut for a tee joint with cap beam prestressing.

stirrups are required outside the joint in the cap beam. If there is no external joint strut developed, an unbalanced horizontal joint force does not exist and only nominal joint steel consistent with equation 3 is required. When an external joint strut is developed for a prestressed tee joint, a part of the unbalanced force can be equilibrated by the cap beam prestress and the required horizontal joint steel is obtained as follows:

$$\rho_s = \frac{3.3}{D f_{yh}\, l_a}\left[\frac{0.09 A_{sc} \lambda_o f_{yc} D'}{l_a} - F\right] \qquad (5)$$

where D is the core diameter of the column and F is the cap beam prestress.

The application of prestressing also reduces the joint principal tensile stress, implying less damage to the joint than would be obtained in an equivalent reinforced concrete joint. However, it is important to note that prestressing increases the stress level in the joint diagonal strut, which may lead to crushing of concrete, and hence a brittle failure of the system. Therefore, it should be ensured that the stress demand in the joint strut is within the allowable limit.

3. EXPERIMENTAL STUDY

Three tee joints incorporating detailing based on the proposed design approach were tested in the Charles Lee Powell laboratory at UCSD [6]. The test units, namely IC1, IC2 and IC3, were chosen as redesigns of a typical column/cap beam tee joint from the Santa Monica Viaduct in Los Angeles, California. The behavior of an as-built joint (referred to as SM3) from Bent 793+57 of the Santa Monica Viaduct was previously established from large scale testing at UCSD [2]. The 3/4 scale model unit SM3 was built and tested inverted in the transverse direction.

Overall dimensions of the structural members in the prototype structure were retained in IC1, IC2 and IC3. The ideal capacity and transverse reinforcement content of 0.5 percent of the prototype column were duplicated in IC1 and IC2. In unit IC3, the volumetric ratio of the column spiral content was increased by 60 percent as it was felt, based on the performance of IC1 and IC2, that an increase in the restraining force may be appropriate against buckling of the longitudinal reinforcement bars and to enhance ultimate compression strain. Consequently, an increase in the displacement capacity and no significant modification in the ideal capacity of the column were expected. The cap beams and joints of IC1, IC2 and IC3 were redesigned using the procedure outlined in Section 2.2, forcing plastic hinging in the column adjacent to the joint interface to comply with the capacity design philosophy. These test units were built at 1/2 scale and tested inverted in the transverse direction under reverse cyclic loading as shown in Figure 4.

Joint reinforcement details of all four test units are shown in Figure 5. The joint detail of SM3 shows (Figure 5a) two obvious deficiencies; (a) no joint reinforcement was provided as typically detailed in the 1960s, and (b) the longitudinal column bars were prematurely terminated within the joint. A reinforced concrete cap beam with no prestressing was designed for unit IC1, whereas a partially prestressed and a fully prestressed bent caps were chosen for IC2 and IC3 respectively. External vertical joint reinforcement was provided only in the first unit (Figure 5b), and it was anticipated that all the column longitudinal bars could be directly anchored into the joint diagonal strut in the two prestressed joints. The amount of prestressing applied for IC2 (Figure 5c) was based on the prestressing sustaining 50 percent of the cap beam negative moment resulting from seismic and gravity loads. Because there was no continuous mild steel required in the cap beam of IC3, this unit was fabricated using precast modules. The column and joint were constructed as a single module whereas the cap beam was built as two separate modules. The test unit was formed by connecting the two segments of the cap beam to either side of the joint solely by prestress.

Prestressing in IC2 and IC3 was applied with zero eccentricity using Dywidag bars. The bars were grouted a few days prior to the day of testing. Unlike the longitudinal direction, the gravity moments are generally small in the transverse direction of multi-column bents. Therefore, straight tendons were preferred to draped tendons as the primary objective of prestressing was to enhance the performance of the joints.

Shear reinforcement required in the cap beam of each unit was nominal. Horizontal joint shear reinforcement provided in the first unit and the two prestressed units was based on eq. (2) and eq. (3) respectively. Vertical joint reinforcement was provided in the form of hairpins in units IC1 and IC2. As observed, particularly in the response of the first unit, anchorage of this type of reinforcement is likely to deteriorate when cracks develop and spall off the cover concrete in the joint region. Therefore, closed ties were preferred as the vertical joint reinforcement in the joint of IC3.

4. PERFORMANCE OF THE TEST UNITS

The hysteretic force-displacement behavior of all four units is shown in Figure 6. The performance of the as-built joint SM3 was unsatisfactory. Severe cracking at ductility 1.0, crushing of concrete in the joint region at ductility 1.5 and large diagonal cracking within the joint at ductility 2.0 were observed. As seen in Figure 6a, the yield strength of the column was developed for both loading directions, but not the ideal flexural strength. Starting from displacement ductility 2.0, the damage was concentrated in the joint region and the influence of joint rotation on the horizontal displacement measured at the top of the column gradually became prominent. Consequently, the force resisting ability of the system was continuously deteriorated.

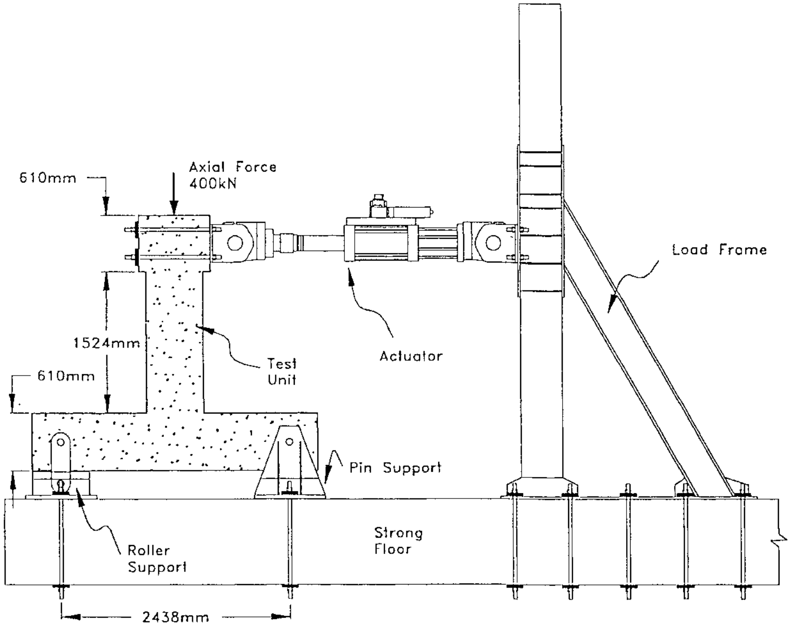
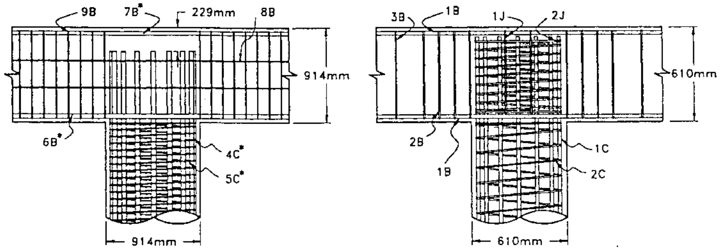
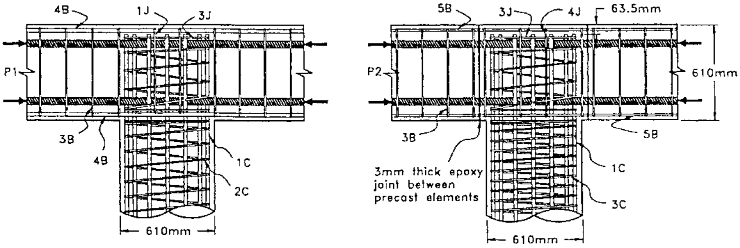


Figure 4: Overall test set-up (1 in. = 25.4 mm).



(a) SM3 – as-built joint from pre-1960

(b) IC1 – reinforced concrete bent cap



(c) IC2 – partially prestressed bent cap

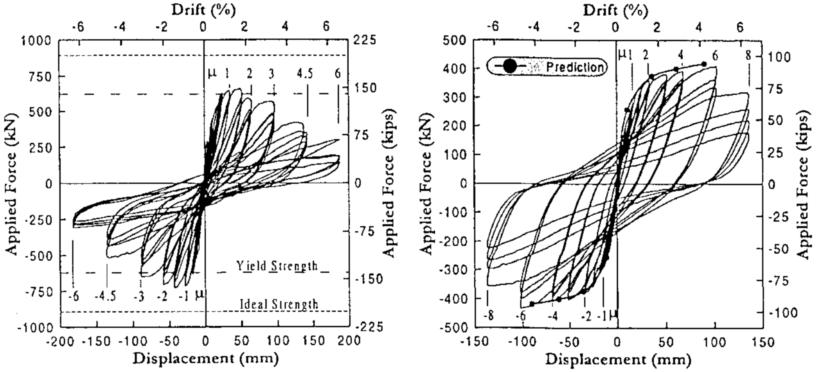
(d) IC3 – precast fully prestressed bent cap

1C - 14#7 (22 mm dia.)	1J - 2 sets of 4 legs #3 (9.5 mm dia.) hairpins
2C - #3 (9.5 mm dia.) spiral @ 96.5 mm	2J - #3 (9.5 mm dia.) spiral @ 57 mm
3C - #3 (9.5 mm dia.) spiral @ 63.5 mm	3J - #3 (9.5 mm dia.) spiral @ 121 mm
4C - 16#14 (43.0 mm dia.)	4J - 2 sets of 4 legs #3 (9.5 mm dia.) stirrups
5C- #3 (9.5 mm dia) spiral @ 66.7 mm	
	P1 = 1670 (4x417.5) kN prestressing force
1B - 7#7 (22 mm dia.)	P2 = 2940 (6x490) kN prestressing force
2B - 4 legs # 3 (9.5 mm dia.) stirrups @ 102 mm	
3B - 4 legs # 3 (9.5 mm dia.) stirrups @ 165 mm	**DIMENSIONS:**
4B - 4#6 (19 mm dia.)	
5B - 4#4 (12.5 mm dia.)	IC1, IC2 & IC3 - Column : 610 mm dia.
6B - 7#14 (43.0 mm dia.)	Beam : 610x686 mm^2
7B - 7#10 (32.3 mm dia.)	
8B - 2#3 (9.5 mm dia) outside the column cage	SM3 - Column : 914 mm dia.
9B - 4 legs # 3 (9.5 mm dia.) stirrups @ 114 mm	Beam : 914x1029 mm^2

*Grade 40 (276 MPa) steel , 1 in. = 25.4 mm

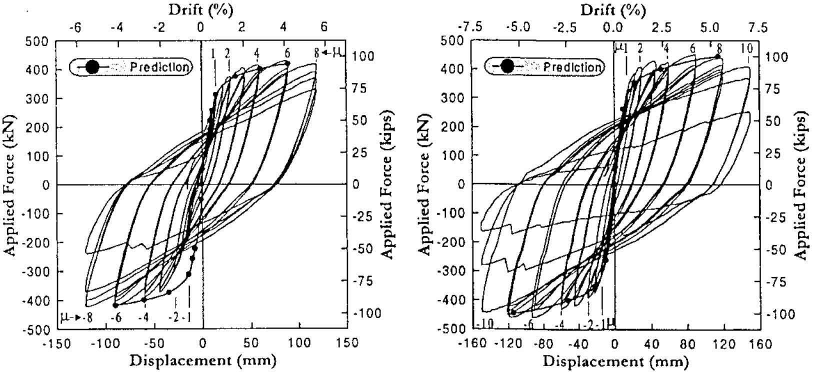
Figure 5: Joint reinforcement details of the test units.

Behavior of unit IC1 was satisfactory under cyclic loading (Figure 6b). Diagonal tension cracks developed in the joint at a horizontal force of 200 kN as predicted prior to the test based on joint principal tensile cracking strength of $\mathbf{0.29\sqrt{f_c^{'}}\,(MPa)}$ $\left(\mathbf{= 3.5\sqrt{f_c^{'}}\,(psi)}\right)$. The inelastic energy absorption of the system was found to be dependable up to displacement ductility $\mu_\Delta = 6$, following which strain softening of the system occurred due to the formation of major diagonal cracks in the joint. The joint deteriorated gradually during cyclic response at $\mu_\Delta = 8$, leading to what appeared to be a diagonal strut failure in the joint at the end of the test. Nevertheless, it was interesting to observe that buckling of longitudinal column reinforcement and fracture of the column spiral were imminent in the plastic hinge region at ductility 8, implying a balanced failure of the joint as well as the column.



(a) SM3 - as-built joint from pre-1960.

(b) IC1 - fully reinforced concrete joint.



(c) IC2 - partially prestressed joint.

(d) IC3 - precast fully prestressed joint.

Figure 6: Force displacement response of the test units.

The loading sequence planned for test unit IC2 was identical to that used for IC1. As a result of actuator signal imbalance, when the hydraulics were turned on, the actuator applied an impulsive force to the specimen in the pull direction. Peak horizontal displacement of the column at the center of the load stub corresponding to the applied force was found to be approximately equal to a system ductility of 3. The horizontal displacement of the column was subsequently brought back to zero, and the loading sequence which was originally planned for the test was applied starting with a horizontal load of 250 kN. Overall response characteristics of IC2 and its energy absorbing ability were extremely good (Figure 6c). There was no significant strength degradation observed prior to $\mu_\Delta = 8$. The strength of the column deteriorated during cyclic loading corresponding to $\mu_\Delta = 8$ due to buckling in compression and fracturing under tension of some of the longitudinal column bars. This was a consequence of low cycle fatigue resulting from alternative buckling and straightening of the longitudinal reinforcement.

Energy absorption capacity of test unit IC3, as indicated by the shape and stability of the lateral force-displacement hysteresis loops (Figure 6d), was excellent. The test unit exhibited no significant strength degradation up to ductility 10. Buckling of the column longitudinal bars was observed during the first cycle in the pull direction and during the following cycle in the push direction at $\mu_\Delta = 10$. A few column bars and spirals fractured in the plastic hinge region in subsequent cycles, as reflected in the force-displacement response.

Damage to the joint region of the two prestressed units was insignificant and limited to uniformly distributed fine cracks, despite the lack of special joint reinforcement. Cracking initiated in the joint of the second unit when an initial horizontal load of 250 kN was applied. For the third unit, joint cracking occurred at $\mu_\Delta = 2$. Shear deformations in the prestressed joints were not sufficiently large to cause any concrete spalling, nor significant joint dilation in the longitudinal direction.

Force-displacement characteristics of IC1, IC2 and IC3 were predicted prior to the test using the measured reinforcement properties and estimated concrete compressive strength. Each unit was modeled using three beam-column elements, considering cracked section properties and flexible end regions. As seen in Figure 6b-d, the predicted and observed envelopes were in good agreement for the first two units except for IC2 in the region where the column suffered some initial damage and no experimental data was recorded. The predicted response for the third unit slightly underestimated the observed force-displacement characteristics, particularly in the early stages of testing (Figure 6d). The predicted ultimate drifts of the three units, considering the influence of transverse reinforcement content, was smaller than that observed in the test.

Figure 7 compares experimentally observed envelopes of force-displacement characteristics for all four units. The actuator force applied to SM3 was scaled to obtain a force equivalent to that required for a 1/2 scale model. Identical initial stiffness was obtained for all four units with the maximum displacements ductilities of 6, 8, 8 and 10 for SM3, IC1, IC2 and IC3 respectively. Joint shear deformation of units IC2 and IC3 was considerably smaller than that observed for the other two joints, and this resulted in smaller maximum displacement for IC2 than for IC1. Comparison of the hysteresis loops of Figures 6 b-d shows that the energy absorbed increased as the amount of prestress increased, presumably as a result of reduced joint deformation.

As noted previously, the strength deterioration seen for IC2 and IC3 at the maximum ductility was primarily due to the damage which occurred to the columns in the hinge region. The strength degradation associated with IC1 was due to damage occurring to both the column and joint. The response of SM3, which never attained its theoretical flexural strength, was mainly influenced by joint deformation initiating at a low ductility level of 1.5. The negative stiffness of the system, which was introduced by the joint damage, is also similar for IC1 and SM3 for both loading directions.

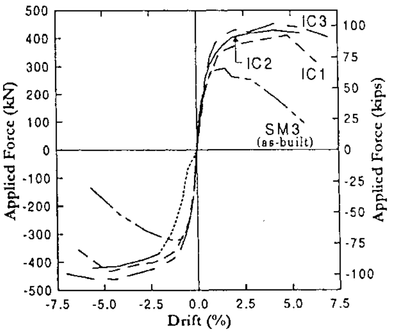


Figure 7: Comparison of force-displacement envelopes of all four joints.

5. DISCUSSION AND CONCLUSIONS

Seismic design of bridge joints using concepts developed for building frames demands large reinforcement content within the joint, leading to congestion problems. Alternative joint detailing using