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MULTIPLE COLUMN BRIDGE BENTS
– Seismic Design Recommendations

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SUMMARY

Following widespread damage to bridge structures in recent earthquakes in California, several research investigations have been conducted at the University of California towards improving seismic design of concrete bridges. Considering the results and findings of various research projects on structural components, two large-scale multi-column bridge bents, one with a prestressed and the other with a reinforced concrete cap beam, were designed and tested under simulated seismic loading. When detailing the test units, (a) the current design specifications were satisfied wherever possible, (b) congestion of the reinforcement was minimized particularly in the cap beam/column connections, and (c) it was ensured that the cap beam/column joints would be subjected to the maximum feasible shear demand. Based on the performance of the test units and analysis of data, recommendations are given in this paper for seismic design of multi-column bents with reinforced, partially prestressed or fully prestressed cap beams.

1. INTRODUCTION

The 1987 Whittier Narrows earthquake and 1989 Loma Prieta earthquake were mainly responsible for initiating vigorous research investigations on bridge structural systems in California. Following observed damaged to various components, flexural and shear performance of columns, column/cap beam connections, column/footing connections and influence of cap beam prestressing have been studied at the University of California, San Diego, (UCSD) with financial support from the California Department of Transportation (Caltrans). A summary of the research studies conducted up to 1995 is included in reference [4]. These studies focused initially on establishing design deficiencies in existing structures, followed by investigations on repair of structural members damaged in earthquakes and retrofit techniques to ensure adequate seismic performance of existing bridge structures. In the next phase of the research, detailing of structural members in modern bridges has been studied with emphasis on reducing steel congestion to improve constructability and ensuring adequate dependable ductility capacity for the structure when subjected to seismic loading. Incorporating research findings from various studies, two half scale test units representative of multiple column bridge bents were designed and tested under simulated seismic loading. With a brief summary of the performance of test units, design recommendations established following the analysis of test data are presented for detailing bridge bents for seismic actions.

2. SEISMIC DESIGN PHILOSOPHY

The current seismic design of bridges in California is based on the capacity design philosophy [4]. Accordingly, plastic hinges are preselected in the columns and detailed adequately to ensure sufficient ductile response under seismic attack. In addition, undesirable failure modes such as anchorage and shear failures are precluded because of their poor hysteretic behavior.

3. EXPERIMENTAL STUDIES

Establishing that a test unit having two columns, representing an exterior and an interior column, would sufficiently model a multiple column bent (see Fig. 1), a three-column prototype bent was modeled at half scale [6,7]. With overall dimensions as shown in Fig. 2, two test units were designed with the objectives of examining the following seismic design details:

- Column confinement and shear requirements
- Pin connections at the column/footing interface
- Cap beam shear design
- Straight bar anchorage of column bars into the joint
- Joint design based on the external strut force transfer model
- Joint design utilizing cap beam prestressing

The major difference between the two test units was that the first unit, MCB1, was designed with a fully prestressed cap beam whereas in the second unit, MCB2, the cap beam was detailed with zero prestressing. For the design of joints in the test units, the external strut force transfer model [3-5] shown in Fig. 3 was used as the design tool, where $\mathbf{0.5T_c}$ resulting from the extreme tension bar is anchored by a claming mechanism at node X and the remainder is supported at node W. The variable $\mathbf{T_c}$ represents the total column tension force and is approximated to $\mathbf{0.5\lambda_0 A_{sc} f_{yc}}$, where λ_0 is the overstrength factor, $\mathbf{A_{sc}}$ and $\mathbf{f_{yc}}$ are respectively the total area and yield strength of column bars. The reinforcement requirements consistent with the design model were obtained assuming that the external joint strut would transfer $\mathbf{0.25T_c}$ to the beam stirrups outside the joint [3,4]. As a result of the cap beam prestressing, the joints of MCB1 were designed with only nominal reinforcement as recommended in reference [5], following testing on bridge tee joints with varying amounts of cap beam prestressing.

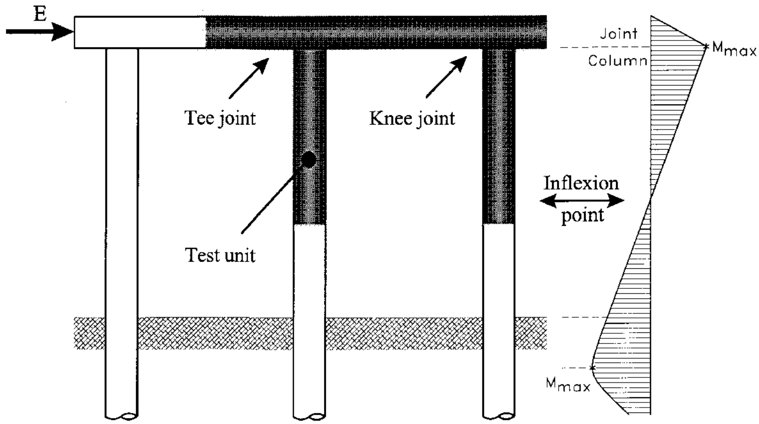


Figure 1 Prototype multi-column bent and its representative test model unit.

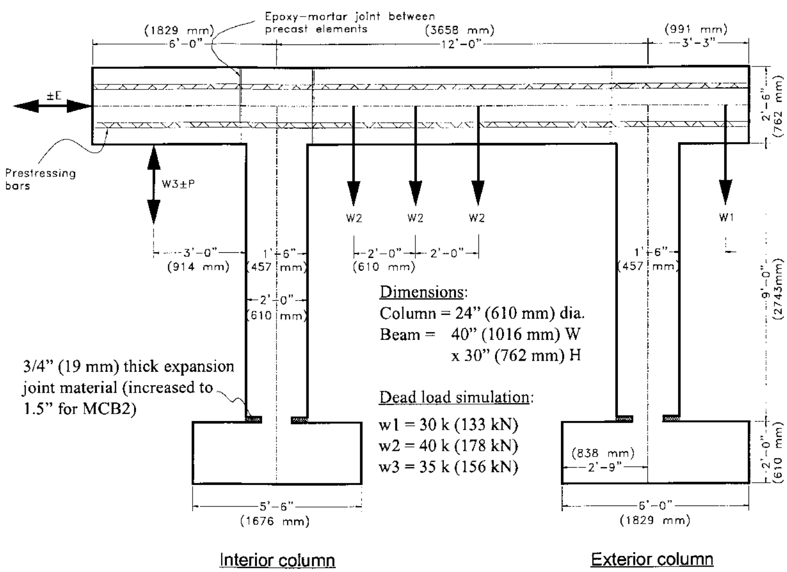


Figure 2 Overall dimensions and test set-up of the first multi-column unit MCB1 (Note: Beam width was reduced to 36" (914 mm) in the second unit MCB2).

In the joints of both multi-column test units, high shear demand was imposed by providing the columns with longitudinal reinforcement content in the range of 3 - 4%.

4. TEST RESULTS

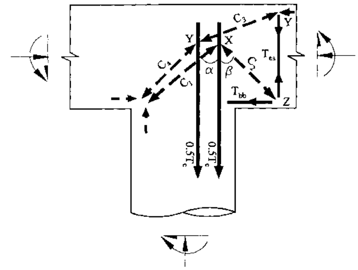


Fig. 3 External strut joint force transfer mechanisms [3-5].

Both test units exhibited satisfactory performance when subjected to simulated seismic loading (see Fig. 4). The observed damage and experimental data confirmed that the detailing adopted for the columns, cap beams and column/footing pin connections was sufficient to ensure dependable response of bridge bents consistent with the design philosophy. A good seismic performance was also obtained for the joints in MCB1 with a prestressed cap beam, which supported the use of nominal joint reinforcement recognizing that the external strut required for the clamping mechanism would be substituted by prestressing in the cap beam. In MCB2, the reinforced concrete tee joint designed based on the external strut force transfer model performed satisfactorily up to 5.8% column drift and it gradually failed at larger displacement drifts. However, the knee joint of this test unit, whose opening action was designed using the external strut force transfer model, exhibited satisfactory response throughout the seismic test. The performance of the knee joint in MCB2 was also satisfactory under closing moments and the corresponding joint reinforcement was reduced within the joint by using continuous longitudinal beam reinforcement to support the joint main strut. Complete details of the design and seismic performance of the test units are given in reference [6].

In a recent study [8], several bridge joints including those from the multi-column bent test units were examined in detail using 2D nonlinear finite element methodology and strut-and-tie modeling of joint regions complying with experimental data. Deficiencies of the design model were found to be responsible for the damage that occurred to the tee joint of MCB2. Further, in the joints designed with the external force transfer model, it was identified that the force transfer across the joint was through two distinct mechanisms, namely splice mechanism (at node D) and clamping mechanism (at node C) as shown in Fig. 5a. Both mechanisms supported almost equal amount of column tension force with an external strut assisting the claming mechanism by transferring $\mathbf{0.15T_c}$ to the beam stirrups. The inclination of this external strut was estimated to be $\mathbf{45^{\circ}}$ to the vertical axis. Also identified in this study was the critical contribution of tensile resistance of cracked concrete in the joint region to the force transfer. In the original joint design model (Fig. 3), the external strut was assumed with a reduced inclination with its magnitude in the vertical direction being $\mathbf{0.25T_c}$. The splice mechanism and participation of tensile resistance of cracked concrete in the joint force transfer were not identified in the design model. Considering the two mechanisms contributing to the joint force transfer, a modified external strut force transfer model has been proposed [8] and is illustrated for a tee joint in Fig. 5. Detailing of joints based on the modified joint force transfer model is included in the next section.

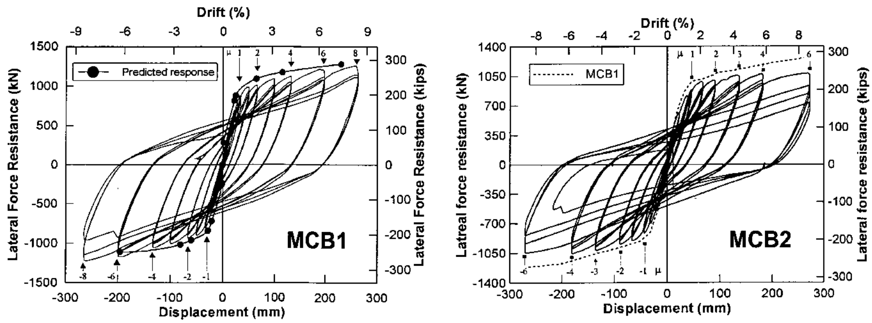
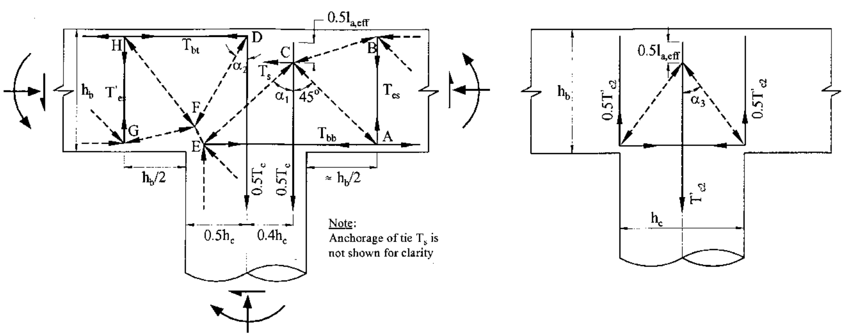


Fig. 4 Performance of the multi-column bent test units under simulated seismic loading.



(a) Force transfer model (b) Simplified splice mechanism

Fig. 5 Modified external strut force transfer model for seismic design of bridge tee joints [8].

5. DESIGN RECOMMENDATIONS

Based on the design and performance of the multi-column bent test units and analysis of the test data, the following recommendations are made for seismic design of single level bridge bents having multiple circular columns. It is assumed that multi-column bents are designed using the capacity design philosophy with hinges forming at column ends.

Column Design

- To ensure sufficient ductility capacity of multi-column bents, the plastic hinge region of the columns should be designed with a confinement requirement given by Eq. 1 [4,1].

$$\rho_s = 0.16\frac{f'_{ce}}{f_{yc}}\left[0.5 + \frac{1.25P}{f'_{ce}A_g}\right] + 0.13(\rho_l - 0.01) \qquad (1)$$

where ρ_s is the volumetric ratio of the transverse reinforcement content, f'_{ce} is the expected concrete compressive strength, P is the column axial load, A_g is the gross area of the section and ρ_l is the longitudinal reinforcement ratio.

- Undesirable shear failure in the columns can be avoided by designing the shear reinforcement using the UCSD three component model [4], which recognizes the concrete, axial load and steel contributions independently. The required amount of spiral reinforcement should be obtained assuming inclined flexure shear cracking at $\theta = 35^\circ$ to the column axis.

Column/Footing Pin Connection

- Axial and shear force transfer should be adequately provided at the pin connection. Allowable concrete shear stress across the interface should be limited to 5.5 MPa [2].

- Pin longitudinal reinforcement should be bundled as close to the center of column as possible to avoid any excessive residual strains developing in the pin reinforcement at the column/footing interface during seismic loading. If the pin longitudinal reinforcement is distributed along the perimeter of the concrete key, significant residual strains can develop in the reinforcement at the interface, creating a weak sliding plane during load reversals.

- To avoid damage occurring to base of the column due to force transfer through the expansion joint filler material, an appropriate pad thickness should be designed considering the maximum displacement capacity of the system. Calculations should be carried out to show that the rotation in the plastic hinge would not cause larger compression than 60% of the initial pad thickness.

- Shear resistance of the transverse reinforcement is effectively reduced in the pin region of a column as a less number of spirals is activated at the column end when compared to a region away from the pin connection [6]. This should be accounted for in column shear design.

Cap Beam Design

- Design of cap beam should be performed using the typical strength reduction factor $\phi_f = 0.9$.

- When joints are designed using force transfer models, the cap beam longitudinal reinforcement should be designed such that it does not develop stresses significantly beyond yield strength at the ultimate limit state.

- When a multi-column bent is designed with a wider cap beam, it would be appropriate to distribute a portion of the longitudinal reinforcement down the sides while ensuring 75% of the bottom reinforcement passes through the column reinforcement cages.

Cap beam/Column Joint Design

The cap beam/column joints of multi-column bents should be designed using the *average* joint principal tensile stress, $\mathbf{p_t}$, as the initial design parameter. In order to reduce the joint reinforcement and improve constructability, the modified external strut force transfer model is used for joint design. When quantifying the necessary reinforcement in the joint region, equal dimensions were assumed for column diameter and cap beam depth [8]. If the two dimensions are significantly different, it is suggested that the tension demand in the joint zone should be evaluated and the reinforcement should be quantified appropriately.

Reinforced Concrete Joints

- When $\mathbf{p_t > 0.42\sqrt{f'_c}}$ (in S.I. units), joint reinforcement is provided, recognizing that about $\mathbf{0.5T_c}$ is anchored by the clamping mechanism with the assistance of an external joint strut while the remaining column tension force is supported by the splice mechanism.

- Accordingly, *a bridge tee joint* is detailed with the following reinforcement:
  (a) Additional external vertical stirrups in the cap beam with a total area equal to

$$\mathbf{A_{es} = 0.125\lambda_o A_{sc}\frac{f_{yc}}{f_{yv}}} \tag{2}$$

  This reinforcement should be provided in the cap beam over a distance $\mathbf{h_b}$ from the joint interface as an addition to the beam shear requirement on both sides of the joint.
  (b) Vertical joint stirrup reinforcement amounting to

$$\mathbf{A_{js} = 0.095\lambda_o A_{sc}\frac{f_{yc}}{f_{yv}}} \tag{3}$$

  (c) Volumetric ratio of the joint horizontal reinforcement equivalent to greater of

$$\rho_s = \mathbf{\frac{0.3A_{sc}\lambda_o f_{yc}}{f_{yh} l_a^2}} \quad \text{(4a)} \qquad or \qquad \rho_s = \mathbf{\frac{0.29\sqrt{f'_c}}{f_{yh}}} \quad \text{(4b)}$$

  (d) Area of additional top beam longitudinal reinforcement amounting to

$$\mathbf{A_{bt} = 0.17\lambda_o A_{sc}\frac{f_{yc}}{f_{yb}}} \tag{5}$$

  (e) Area of additional bottom beam longitudinal reinforcement equal to

$$\mathbf{A_{bb} = 0.15\lambda_o A_{sc}\frac{f_{yc}}{f_{yb}}} \tag{6}$$

  In the above equations, the material properties may be approximated to:
  (a) $\lambda_o = \mathbf{1.4}$ with an assumption of $\mathbf{f_{yc} = f_{yv} = f_{yh} = f_{yb}}$, or
  (b) $\lambda_o = \mathbf{1.3}$ with measured values of $\mathbf{f_{yc}}$, $\mathbf{f_{yv}}$, $\mathbf{f_{yh}}$ and $\mathbf{f_{yb}}$

- *A bridge knee joint subjected opening moments* can be detailed using the procedure described for the tee joint except that no additional beam top reinforcement given by Eq. 5 is required. Further, beam longitudinal reinforcement should be provided as continuous reinforcement through the joint or an equivalent detail should be used to ensure anchorage of the joint strut resulting from the splice mechanism.

- No special reinforcement is required for *a bridge knee joint subjected closing moments* when it is designed for opening moments as recommended above.

- When $\mathbf{p_t \leq 0.29\sqrt{f_c'}}$, joint reinforcement is reduced to nominal requirements with no additional steel in the cap beam. Nominal joint reinforcement should satisfy the following details [4]:
  (a) Vertical joint reinforcement amounting to

$$\mathbf{A_{js} = 0.0625\lambda_o A_{sc} \frac{f_{yc}}{f_{yv}}} \tag{7}$$

  (b) Volumetric ratio of the joint horizontal reinforcement as required by Eq. 4b.

- When $\mathbf{0.29\sqrt{f_c'} \leq p_t \leq 0.42\sqrt{f_c'}}$, a linear interpolation of the reinforcement required for the two principal tensile stress limits can be considered.

Prestressed Concrete Joints

When the bent cap of a multi-column bent is designed with partially or fully prestressed details, the joint shear reinforcement is reduced to nominal requirements in most cases. This would result in less joint reinforcement than that required in an equivalent reinforced concrete joint designed according to the procedure outlined above.

- A fully prestressed joint shall be designed with nominal reinforcement within the joint and no additional reinforcement outside the joint in the cap beam if:
  (a) the most extreme column tension bar can be anchored into the joint main diagonal strut by bond, *or*
  (b) the average joint principal tensile stress is less than $\mathbf{0.29\sqrt{f_c'}}$.

- To ensure sufficient anchorage of the extreme longitudinal column tension bar, the depth of joint strut at the location of the reinforcement may be approximated to the neutral axis depth of the beam section adjacent to the column tension side and with a conservative bond stress value of $\mathbf{2.5\sqrt{f_c'}}$.

- The joint nominal reinforcement should be obtained as follows:
  (a) Area of the vertical joint stirrup reinforcement in accordance with Eq. 7.
  (b) Volumetric ratio of the joint horizontal reinforcement as given by Eq. 4b.
  Material properties in the above equations may be taken as recommended for design of reinforced concrete joints.

- A partially prestressed joint shall be designed with nominal joint shear reinforcement as detailed above for a fully prestressed joint if the prestressing is designed to sustain at least

50% of the cap beam design (negative) moment. In addition, top and bottom beam reinforcement amounting to those given by Eqs. 8 and 9 should be provided across the joint for force transfer.

(a) Area of additional top beam longitudinal reinforcement

$$\mathbf{A_{bt} = 0.085\lambda_o A_{sc} \frac{f_{yc}}{f_{yb}}} \tag{8}$$

(b) Area of additional bottom beam longitudinal reinforcement

$$\mathbf{A_{bb} = 0.075\lambda_o A_{sc} \frac{f_{yc}}{f_{yb}}} \tag{9}$$

If either of the criteria established for designing a prestressed joint with nominal reinforcement is not satisfied, an appropriate joint detail consisting of additional reinforcement in the cap beam should be obtained using the procedure outlined for reinforced concrete joints.

Other Requirements for All Joints

In addition to the above reinforcement requirements, the following details should be satisfied in all cap beam/column joints to ensure satisfactory performance.

- All column bars should be extended as close to the top beam longitudinal reinforcement as possible with a minimum embedment length into the joint as given by Eq. 10, where $\mathbf{d_{bl}}$ is the diameter of the column longitudinal reinforcement.

$$\mathbf{l_a \geq 0.30\, d_{bl}\, f_{yc} / \sqrt{f'_c}} \tag{10}$$

- All the vertical stirrups in the joint region should be provided as closed ties with appropriate number of crossties in accordance with the current design practice.

- When the column framing into a joint is detailed with longitudinal reinforcement ratio $\rho_l >$ 3%, a minimum joint concrete strength of 35 MPa should be ensured [6].

- The average joint principal compression stress should be limited to $\mathbf{0.3f'_c}$. A larger principal compression stress may be permitted in a prestressed joint when it is shown using a rational procedure that an undesirable joint failure would not occur by comparing the capacity and demand upon the joint strut [8].

6. CONCLUDING REMARKS

When bridge multiple column bents are designed with the recommendations given above, it is expected that they would exhibit dependable seismic performance in accordance with the capacity design philosophy. Hysteretic actions would be concentrated within the plastic hinge regions of the columns and confinement detailing of the hinges would ensure 50% reserve displacement capacity for the structure when compared to the displacement demand expected under design level earthquakes.

ACKNOWLEDGMENT

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Research update by the authors:

A full-scale concrete bridge bent having three steel jacketed columns and a reinforced concrete cap beam was designed in accordance with the above recommendations and tested recently under simulated seismic loading at UCSD. This test, which was funded by the Alaska Department of Transportation, validated the recommended design procedures. A dependable seismic response was obtained for the test unit until cycled to system ductility $\mu_\Delta = 10$. At this ductility, column overstrength moment capacities were developed in preselected plastic hinges with the corresponding damage in the joints being limited to well-distributed fine cracking. The* *strain data confirmed that the joint spiral and stirrup reinforcement was not subjected to tension demand significantly exceeding the yield strength.*

Note that the recommendations given above for prestressed bent caps were verified in earlier laboratory tests [5,6]. The test units having fully prestressed cap beams were constructed from precast modules and it was demonstrated that precast construction of multiple column bents provides equally efficient systems as cast-in-situ bridge bents for resisting seismic forces.

[*]Silva, P. D., Sritharan, S., Priestley M. J. N. and Seible, F., *Full-Scale Proof Test of a Bridge Bent Having Three Steel Jacketed Columns - Preliminary Report to the Alaska Department of Transportation*, Structural Systems Research, University of California at San Diego, California, September 1998.