STRUT-AND-TIE MODEL CONCEPTS FOR SEISMIC DESIGN AND ASSESSMENT OF CONCRETE BRIDGE JOINTS

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SUMMARY

A comprehensive experimental and analytical study on bridge cap beam-to-column concrete joints has been conducted at the University of California at San Diego (UCSD) over the past decade. In this study, the improvement of both detailing and the in-plane seismic performance of bridge joints has been examined by representing the force transfer across the joint using simple strut-and-tie models, which evolved into a rational force transfer method for designing and assessing bridge joints subjected to seismic actions. Starting with joint force conditions and failure modes, this methodology is described in this paper. Some specific strut-and-tie details relevant to seismic behaviour of bridge joints are then discussed, followed by several key joint mechanisms and examples of design models to facilitate application of the proposed method.

INTRODUCTION

Application of strut-and-tie modeling to structural design problems has been widely used under static loading. Researchers at UCSD have examined utilising the same basic concepts to detail structural systems subjected to seismic loading. This paper specifically considers studies on the in-plane behaviour of knee (exterior) and tee (interior) joints of multiple circular-column bridge bents [Priestley, 1993; Ingham, 1995; Sritharan, 1998].

Motivated by (a) use of inadequate joint details in practice, and (b) unacceptably congested details of bridge joints when using the building joint design method based directly on shear forces, investigation of the in-plane seismic behavior of joints sought to establish sufficient and less conservative joint reinforcement details. This investigation, which consisted of large-scale seismic tests on twenty bridge joints and parallel analytical studies, evolved into a rational force transfer method for seismic design and assessment of concrete bridge joints. In the force transfer method, required reinforcement details are established based on simple strut-and-tie models representing not just the joint, but the joint disturbed region (D-region).

JOINT FORCES AND STRESSES

Seismic design of concrete bridge structures is currently based on the capacity design philosophy [Priestley *et. al*., 1996], in which plastic hinges are preselected at the top and bottom of the columns and inelastic actions developing outside these hinges is prohibited by using strength hierarchy in the design. As a consequence of developing column plastic hinges adjacent to the cap beam/column interface, the joint shear in the horizontal direction, $\mathbf{V_{jh}}$, can be approximated based upon the assumption that the column overstrength moment uniformly diminishes over the full depth of the cap beam as illustrated in Fig. 1 [Priestley *et. al*., 1996].

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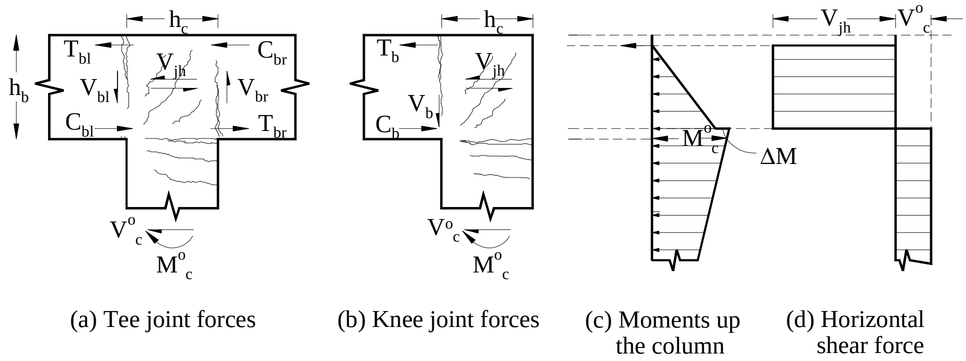


Fig. 1 Horizontal joint shear forces in bridge joints.

$$\mathbf{V_{jh}} \approx \frac{\mathbf{M_c^o}}{\mathbf{h_b}} \tag{1}$$

where $\mathbf{M_c^o}$ is the overstrength moment capacity of the column at the joint interface and is obtained from a section analysis with due consideration to column axial force resulting from gravity and seismic actions, and $\mathbf{h_b}$ is the beam depth. Using the estimate for the horizontal shear forces, joint shear stress can be obtained from Eq. (2)

$$\mathbf{v_j} = \frac{\mathbf{V_{jh}}}{\mathbf{b_j D}} \tag{2}$$

where $\mathbf{b_j}$ is the joint effective width and is taken as greater of $\sqrt{2}\mathbf{D}$ or $\mathbf{b_w}$ (Fig. 3b), and $\mathbf{D}$ is the column diameter. With appropriate estimates for the joint vertical and horizontal normal stresses [Priestley *et. al.*, 1996; Sritharan *et. al.*, 1999], the joint principal compression and tensile stresses are:

$$\mathbf{p_c, p_t} = \frac{\mathbf{f_v + f_h}}{\mathbf{2}} \pm \sqrt{\left(\frac{\mathbf{f_v - f_h}}{\mathbf{2}}\right)^2 + \mathbf{v_j^2}} \tag{3}$$

where $\mathbf{f_v}$ is the vertical normal stress due to column axial compression and $\mathbf{f_h}$ is the horizontal normal stress due to cap beam axial force. Since the principal stresses have better correlation to joint damage state than do other parameters such as joint shear force, $\mathbf{p_t}$ and $\mathbf{p_c}$ are used as initial design parameters in the force transfer method.

JOINT FAILURE MODES

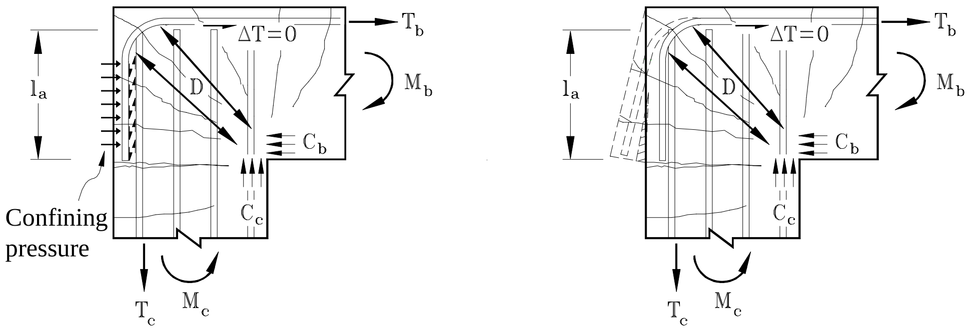
Failure of bridge joints may occur in four different modes [Sritharan, 1998]. Each of these failure modes is described below.

Compression Failure: A compression failure occurs in bridge joints in a brittle manner as a result of crushing of concrete in the joint core. This failure mode is typical in prestressed joints, and in reinforced concrete joints detailed with sufficient shear reinforcement such that they remain elastic during seismic response. Compression failure of joints will substantially reduce the lateral force resistance of the structure, most likely leading to total structural collapse.

Tension Failure: A tension failure is typically developed in reinforced concrete joints when shear reinforcement responsible for mobilising the joint compression field is subjected to large inelastic strains. Since inelastic strains are irreversible, a growth of the joint panel occurs under seismic loading. Consequently, the effective concrete strength of the joint core is significantly reduced, which often results in crushing of the joint strut at large displacement ductilities. Although significant lateral strength loss, which may lead to structural collapse, is associated with such a joint failure, strength degradation will occur in a gradual manner. Tension failure is also expected in older bridge joints detailed with little or no shear reinforcement as column longitudinal reinforcement provides some tensile resistance to the joint at small shear strains.

Anchorage Failure: For satisfactory seismic performance of a bridge structure, it is essential that the column and cap beam longitudinal reinforcement be sufficiently anchored into the joint. Inadequate anchorage detail will result in bond slip of the reinforcement, introducing additional member end rotation at the joint interface and thus reducing the lateral strength resistance of the structure. The bond slip rotation resulting from anchorage failure can contribute in excess of 40% to the total lateral displacement [Sritharan *et. al.*, 1998]. Given that the bond slip mechanism does not provide adequate force resistance, nor a profound energy dissipation system, the structure will exhibit a poor force-displacement hysteresis response, with gradual strength deterioration and loop pinching effect as displacement ductility and/or number of load reversals is increased. The column longitudinal reinforcement, which is typically anchored with straight bar ends [Priestley *et. al.*, 1996], is more susceptible to bond slip due to the development of high inelastic strains at the column-joint interface.

Lap Splice Failure: Lap splice failure typically occurs in bridge knee joints subjected to closing moments. As shown in Fig. 2a, the column tension force may be transferred to the top beam reinforcement by bond if adequate confinement is provided for the lap splice. If the confining pressure is not sufficient to prevent splitting of concrete between the reinforcement and straightening the hook of the beam bars, a failure may ensue as illustrated in Fig. 2b. A lap splice failure can also occur if the lap length between the reinforcement is not sufficient to transfer the column tension force to the beam bar.



(a) Force transfer by bond (b) Failure mode

Fig. 2 Lap splice failure in a bridge knee joint.

FORCE TRANSFER METHOD

To obtain satisfactory joint performance while ensuring constructable joint details with no unnecessary congestion of reinforcing steel, the force transfer method has been developed for the seismic design of joints. In this methodology, the design is performed for the ultimate limit state using the corresponding joint principal stresses as the initial design parameters. At the serviceable limit state, the joint principal tensile stress is kept below $\mathbf{0.25\sqrt{f'_c}}$ ($\mathbf{f'_c}$ in MPa) with no special detailing requirement. For a typical bridge column longitudinal reinforcement content in the range of 1.0% – 4.0% and comparable dimensions for the column diameter and beam depth as is used in practice, accomplishing the serviceability design criterion is not difficult.

The joint reinforcement in the force transfer method is quantified using a force transfer model with clearly identified mechanism(s). Therefore, application of this design method is only required when sufficient joint cracking is expected to develop and mobilise the joint reinforcement typically beyond the serviceable limit state. Different force transfer models will provide different reinforcement quantities in the joint region. However, in all cases, the force transfer method is expected to provide less conservative joint details than the more traditional approach based directly on joint shear forces. It is the authors' opinion that the most efficient force transfer model for seismic joint design is that which requires the least amount of reinforcement within the joint while ensuring satisfactory joint behaviour. More guidelines for the force transfer method are given below.

Design and Assessment Guidelines

In accordance with the force transfer method of design, the following guidelines are suggested for the design and assessment of joints in bridge structures:

1. Using the principal tensile stress obtained at the ultimate limit state, joint is designed using a force transfer method when $\mathbf{p_t > 0.42\sqrt{f_c^{'}}}$, where $\mathbf{f'_c}$ is the unconfined compressive strength of concrete in MPa. For $\mathbf{p_t \le 0.25\sqrt{f_c^{'}}}$, limited joint shear cracking is expected and thus nominal reinforcement as described in Refs. [6] and [10] is sufficient within the joint. If the principal tensile stress is within these two limits, then a linear interpolation of the two reinforcement requirements is recommended.

   In the assessment procedure, nominal joint reinforcement may be permitted up to $\mathbf{p_t = 0.29\sqrt{f_c^{'}}}$. For $\mathbf{p_t > 0.42\sqrt{f_c^{'}}}$, adequacy of joint reinforcement must be ensured using an efficient force transfer model. A linear interpolation of the required reinforcement may be considered when $\mathbf{p_t}$ is between the two limits.

2. When $\mathbf{p_t}$ is greater than the lower limit, nominal joint reinforcement can be permitted if it is shown that the column bars can be satisfactorily anchored into the joint main strut [Sritharan, 1998; Sritharan *et. al.*, 1999].

3. The joint principal compression stress should always be maintained below $\mathbf{0.3f_c^{'}}$ unless shown otherwise that a larger stress can be tolerated in critical joint struts.

4. The column bars should be anchored into the cap beam with straight bar ends. Use of headed bars for the column longitudinal reinforcement is favoured in the force transfer method. However, employing column bars with hooks or tails at the top is expected to cause unnecessary congestion.

   If the column bars are prematurely terminated in existing bridge joints, increasing their embedment length into the joint may be necessary, for example by haunching of the joint [Ingham *et. al.*, 1998]. This modification should be included in the force transfer model.

5. A minimum anchorage length for the beam and column longitudinal reinforcement into the joint should be provided assuming a uniform bond stress of $\mathbf{1.17\sqrt{f_c^{'}}}$ along the embedded portion of the bar.

6. Column bars should be extended as close to the top beam reinforcement as possible to provide adequate embedment of the extreme column tension bars into the joint diagonal strut.

7. A realistic contribution of tension carrying capacity of the joint cracked concrete can be included in the force transfer model (see details below).

Influence of Repeated Loading

Influence of seismic or cyclic type loading is indirectly taken into account in the force transfer method. As implied in Eq. 1, joint forces required for calculating the joint principal stresses are obtained including the column reinforcement strain hardening effect. Hence, the maximum possible forces that the joint can be subjected to during repeated loading are satisfactorily incorporated in the design method.

Strength deterioration resulting from repeated loading is addressed by appropriately selecting permissible material strengths. Since no significant hardening is expected in joint reinforcement, the steel stress-strain relation obtained from uniaxial tension testing can be satisfactorily applied to cyclic loading. Therefore, for an estimated maximum joint reinforcement strain, the corresponding stress can be readily established. Allowable compression and tension stresses suitable for the joint concrete under repeated loading are discussed below.

SPECIFIC STRUT-AND-TIE DETAILS

Basic strut-and-tie concepts are readily available in the literature (e.g., [Schlaich *et. al.*, 1987]) and hence, only specific details relevant to seismic behaviour of bridge joints are discussed in this section.

Joint Compression Force Flow: Determining a suitable force path for compression flow across the joint is the most critical feature of the force transfer method as this essentially determines the node locations and orientations of the compression struts. Observed crack patterns, experimental data and nonlinear finite element analyses are used for establishing different force transfer mechanisms and design models.

Struts: Critical struts in beam-to-column joint regions have bottle-shaped stress field [Schlaich *et. al.*, 1987; Ingham, 1995; Sritharan, 1998]. When the joint force is transferred between nodes through a bottle-shaped stress field under in-plane loading, both in-plane and out-of-plane tensile stresses are developed perpendicular to the force transfer direction, reducing the capacity of the strut. This reduction should be accounted for when establishing allowable compression stresses for the struts. Based on experimental observations, different stress limits are recommended in Table 1 as a function of the maximum tensile strain in the joint shear reinforcement. If this parameter is not known, conservative stress limits must be used.

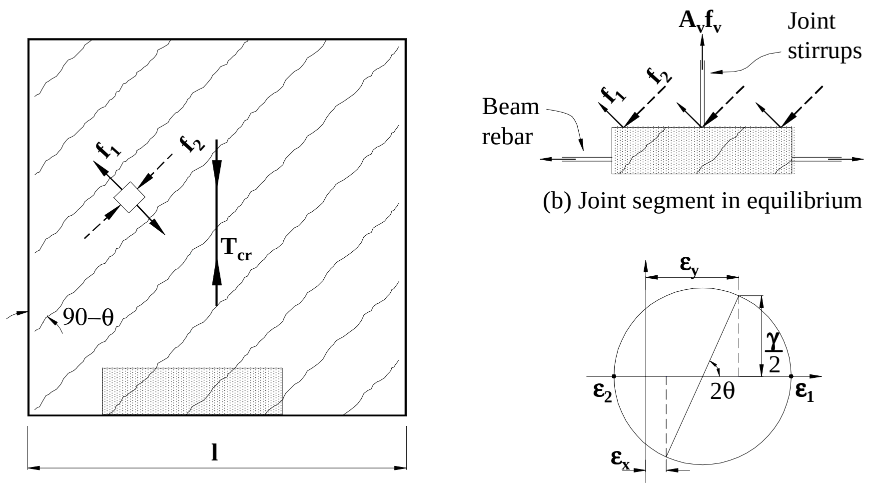
Table 1: Permissible stresses suggested for critical bridge joint struts [Sritharan, 1998].

Permissible stress	Strut Description
$\mathbf{0.68f_c'}$	For joint struts with only minor cracking such as that expected in prestressed joints.
$\mathbf{0.51f_c'}$	Struts in reinforced concrete joints with reinforcement not subjected to significant strain hardening ($\varepsilon_s \le 0.01$).
$\mathbf{0.34f_c'}$	Struts in unreinforced joints or in joints with potential for initiation of tension failure following development of high inelastic strains in the joint reinforcement ($\varepsilon_s \ge 0.02$).

*For $0.01<\varepsilon_s<0.02$, consider linear interpolation to obtain appropriate permissible stresses.

Contribution of Ties: It is straightforward to take the reinforcement contribution into account if the effective steel area in the direction of the tie is estimated. It has been recently reported that cracked concrete also provides substantial tensile resistance in the joint region [Sritharan, 1998]. An estimate for this component may be found using a blanketed approach as illustrated in Fig. 3. From equilibrium of the joint segment in Fig. 3b, the total tensile resistance in the vertical direction is

$$\mathbf{T_{cr} = f_1(\cos^2\theta)wl} \tag{4}$$



(a) Idealized joint panel stresses

(b) Joint segment in equilibrium

(c) Mohr's circle for average strains

Fig. 3 Estimating tension carrying capacity of joint cracked concrete.

where $\mathbf{T_{cr}}$ is the tension force carried by the cracked concrete, $\mathbf{f_1}$ is the tensile resistance in the principal stress direction, **w** is the joint width and **l** is the length of the joint panel. For estimating $\mathbf{f_1}$ in a design model, the following empirical relationship obtained from Collins and Mitchell [1997] can be used.

$$\mathbf{f_1 = \frac{0.23\sqrt{f_c'}}{1+\sqrt{500\varepsilon_1}}} \tag{5}$$

where $\mathbf{f_c'}$ is the unconfined concrete strength and $\boldsymbol{\varepsilon_1}$ is the average joint principal tensile strain expected at the ultimate limit state. A less conservative estimate for $\mathbf{f_1}$ given by Eq. (6) [Vecchio and Collins, 1986] together with **1.5l** instead of **l** in Eq. (4) is suggested for assessment purposes.

$$\mathbf{f_1 = \frac{0.23\sqrt{f_c^{'}}}{1+\sqrt{200\varepsilon_1}}} \tag{6}$$

Having estimated the tension carrying capacity of the joint cracked concrete at the ultimate limit state, this contribution can be represented with one or several discrete ties in a force transfer model.

Nodal Failure: The most common nodal failure expected in bridge joints is within nodes where longitudinal column bars are anchored with straight bar ends [Sritharan, 1998]. Such a nodal failure can be avoided in design and predicted in assessment by ensuring/examining the column bar embedment length into the joint. As suggested under *Guidelines*, the required embedment length for column bars can be obtained assuming a uniform bond stress of $\mathbf{1.17\sqrt{f_c^{'}}}$, which would result in a minimum anchorage length

$$\mathbf{l_a = 0.30\, d_{bl}\, f_y \Big/ \sqrt{f_c^{'}}} \tag{7}$$

where $\mathbf{d_{bl}}$ and $\mathbf{f_y}$ are respectively the diameter and yield strength of the column bar. In addition, it must also be ensured that the column bars are extended into the joint as close to the top beam bars as possible [Priestley *et. al*., 1996; Sritharan *et. al*., 1998]. If this is not satisfied, adequate clamping of the column bars into the joint strut will not occur and nodal failure can develop despite satisfying the minimum anchorage length requirement. For assessing bridge joints with inadequately embedded column bars into the joint, the maximum force that can be developed in the column bars may be estimated using a uniform bond stress of $\mathbf{0.76\sqrt{f_c^{'}}}$ along the embedded portion of the reinforcement [Sritharan *et. al*., 1998].

If the column bar anchorage is addressed as detailed above, it is then suggested that no further check on nodal failure is required.

KEY JOINT MECHANISMS

Using the general strut-and-tie concepts and specific details presented above, several different joint mechanisms can be formulated for bridge joints. Some of the most efficient mechanisms are presented in this section.

Clamping Mechanism

In this mechanism, the column tension force, $\mathbf{T_c}$, is directly anchored into the joint using a joint diagonal strut and an inclined strut in the beam (i.e., $\mathbf{C_2}$) adjacent to the column tension side. As noted in Fig. 4, this mechanism can be used to transmit up to 50% of $\mathbf{T_c}$ in reinforced concrete joints and 100% of $\mathbf{T_c}$ in fully prestressed joints [Sritharan, 1998; Sritharan *et. al*., 1999].

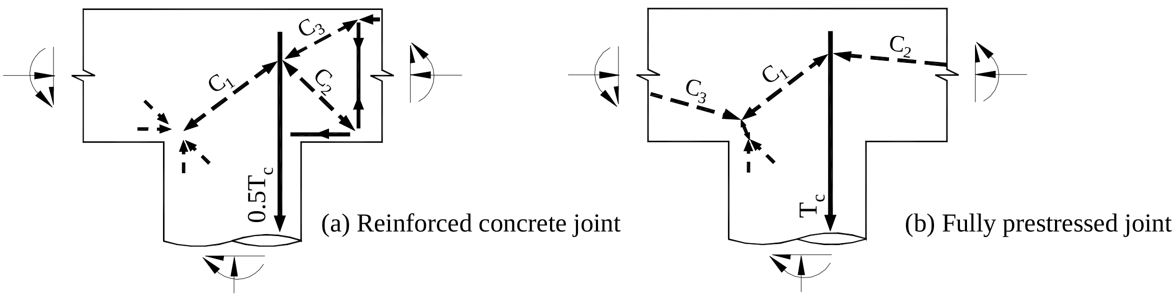
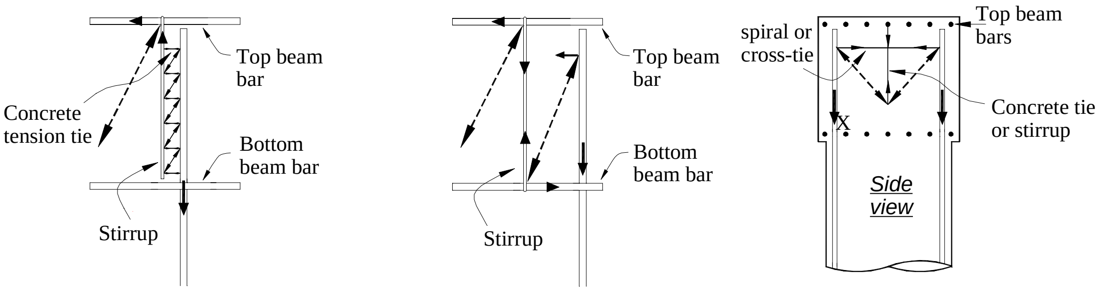


Fig. 4 Clamping mechanism for a bridge tee joint

Splice Transfer Mechanism

This mechanism relies upon transferring the column tension force to the top beam bars and then anchoring it with a joint diagonal strut. The column force transfer is assisted by concrete ties and/or joint vertical stirrups as shown in Fig. 5. Although, this figure shows only the in-plane force transfer, the column force can be transferred using struts and ties in three dimensions [Sritharan, 1998]. This mechanism, which can potentially anchor 50% of $\mathbf{T_c}$, will diminish as cap beam prestressing increases.

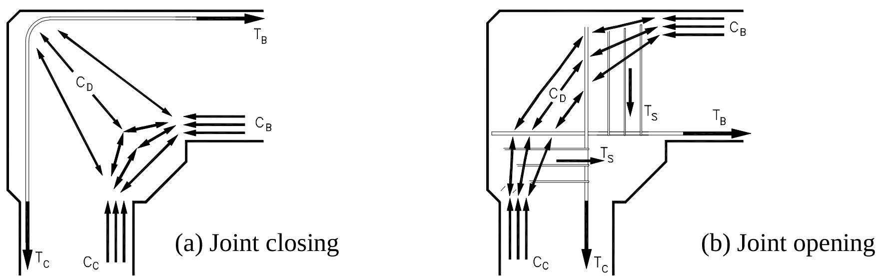


(a) Conventional mechanism (b) Single strut mechanism (c) Out-of-plane mechanism

Fig. 5 Splice transfer mechanism.

Haunched-Joint Mechanism

Special mechanisms can be relied upon for haunched joints such as that illustrated in Fig. 6 for a bridge knee joint [Ingham *et. al.*, 1998]. The haunched-joint mechanism will alleviate possible compression failure under closing moments and improve anchorage of column reinforcement under opening moments. Haunching of joints, which is primarily used when retrofitting existing joints with poor column reinforcement anchorage and/or insufficient joint shear reinforcement, increases the joint size. Hence, additional joint reinforcement can also be added if needed without causing steel congestion in the joint.

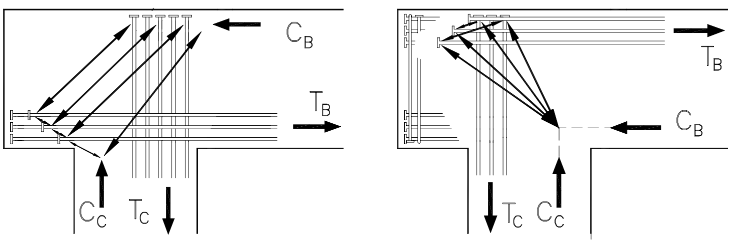


(a) Joint closing (b) Joint opening

Fig. 6 Haunched-joint mechanism for a bridge knee joint.

Distributed Strut Mechanism

By strategically placing headed reinforcement in the cap beam of a knee joint with a short stub, the distributed strut mechanism can be developed, which will alleviate joint compression and anchorage failure. As shown in Fig. 7, the beam bars should be vertically distributed with the heads staggered in the stub [Ingham *et. al.*, 1996].



(a) Joint opening (b) Joint closing

Fig. 7 Distributed strut mechanism for a bridge knee joint.

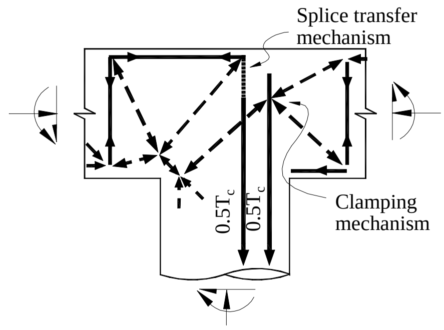


Fig. 8 Modified external strut force transfer model for a bridge tee joint.

FORCE TRANSFER MODELS

Using a single or combination of several joint design mechanisms, joint force transfer models suitable for design or assessment can be formulated. Some examples are presented below.

Modified External Strut Force Transfer Model: Combining the clamping and splice transfer mechanisms, a force transfer model for a bridge tee joint is shown in Fig. 8. If a fully prestressed cap beam is used, the joint can be detailed using the model shown in Fig. 4b, ignoring the contribution of the splice transfer mechanism.

Haunched-Joint Force Transfer Model: As explained above, this model will combine the haunched-joint mechanism with the splice transfer mechanism unless the cap beam is fully prestressed.

Distributed Strut Force Transfer Model: Based solely on the distributed strut mechanism, the joint forces can be transferred satisfactorily. The necessary joint reinforcement can be quantified using the strut-and-tie models shown in Fig. 7.

CONCLUDING REMARKS

A rational force transfer method for seismic design and assessment of concrete bridge joints is presented in this paper. This method determines the satisfactory amount of joint reinforcement using simple analytical models based on strut-and-tie concepts under repeated loading. In order to facilitate this approach, several guidelines, the most efficient joint mechanisms and design/assessment models are also presented. Unlike the conventional joint design approach, in which the joint shear is treated as an independent force, the force transfer method addresses joint shear as part of the complete force transfer across the joint. As a result, the suggested approach provides reduced and less conservative reinforcement and improves constructability of joints.

REFERENCES

1. Collins, M. P. and Mitchell, D., *Prestressed Concrete Structures*, Response Publications, Canada, 1997.

2. Ingham, J. M., *Seismic Performance of Bridge Knee Joints*, Doctoral Dissertation, Division of Structural Engineering, University of California at San Diego, California, 1995, 511 pp.

3. Ingham, J. M., Priestley M. J. N. and Seible, F., *Seismic Performance of a Bridge Knee Joint Reinforced with Headed Reinforcement*, Structural Systems Research Project, Report No. SSRP 96/06, University of California at San Diego, California, September 1996, 104 pp.

4. Ingham, J. M., Priestley M. J. N. and Seible, F., "Cyclic Response of Bridge Knee Joints with Circular Columns", *Journal of Earthquake Engineering*, Vol. 2, No. 3, 1998, pp. 357-390.

5. Priestley, M. J. N., *Assessment and Design of Joints for Single-Level Bridges with Circular Columns*, Structural Systems Research, Report No. SSRP 93/02, University of California at San Diego, California, February 1993, 62 pp.

6. Priestley, M. J. N., Seible, F. and Calvi, M., *Seismic Design and Retrofit of Bridges*, John Wiley & Sons, New York, 1996, 686 pp.

7. Schlaich, J., Schafer, K. and Jennewein, M., "Toward a Consistent Design of Structural Concrete", *PCI Journal*, Vol. 32, No. 3, May-June 1987, pp. 75-149.

8. Sritharan, S., Ingham, J. M., Priestley, M. J. N. and Seible, F., "Bond Slip of Bridge Column Reinforcement Anchored in Cap Beams", *ACI Special Volume on Bond and Development of Reinforcement – A Tribute to Dr. Peter Gergely*, ACI SP-180, 1998, pp. 319-345.

9. Sritharan S., *Analysis of Concrete Bridge Joints Subjected to Seismic Actions*, Doctoral Dissertation, Division of Structural Engineering, University of California at San Diego, California, 1998, 407 pp.

10. Sritharan, S., Priestley M. J. N. and Seible, F., "Enhancing Seismic Performance of Bridge Cap Beam-to-Column Joints Using Cap Beam Prestressing", *PCI Journal*, Vol. 44, No. 4, July-August 1999, pp 74 – 91.

11. Vecchio, F. J. and Collins, M. P., "The Modified Compression Field Theory for Reinforced Concrete Elements Subjected to Shear," *ACI Structural Journal*, Vol. 83, No. 2, March-April 1986, pp. 219-231.