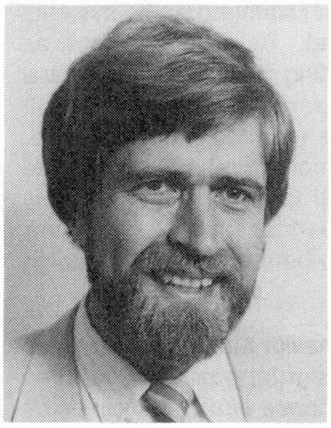
# Performance assessment of cap-column joints under seismic loading

# Performance Assessment of Cap-Column Joints Under Seismic Loading

## Evaluation de la résistance de joints pile-pont soumis aux séismes

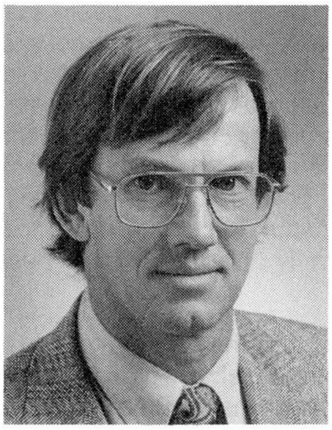
## Begutachtung von Rahmenecken unter Erdbebenbelastung

**Frieder SEIBLE**
Prof. of Struct. Eng.
Univ. of California, San Diego
La Jolla, CA, USA



Frieder Seible has been a member of the UCSD faculty for 7 years. He has Civil Engineering degrees from Germany, Canada and the USA and his research combines large scale experimental testing and nonlinear analystical modeling of structural concrete systems.

**M. J. Nigel PRIESTLEY**
Prof. of Struct. Eng.
Univ. of California, San Diego
La Jolla, CA, USA



M.J. Nigel Priestley has received numerous international awards (ACI, PCI) for his earthquake engineering and structural concrete research contributions. He was a member of the faculty of the University of Canterbury, Christchurch, New Zealand for eleven years prior to joining UCSD.

**SUMMARY**

An overview of joint damage in beam-column joints of multi-level bridge bents during the 1989 Loma Prieta earthquake, is presented. Assessment criteria for evaluating seismic performance characteristics and retrofitting schemes for existing concrete bridge structures, are developed. The application of analytical concrete models of various complexities in support of structural concrete design is demonstrated.

**RÉSUMÉ**

On présente ici une vue d'ensemble des dommages causés aux angles des cadres soutenant des ponts multi-niveaux, advenus lors du tremblement de terre de Loma Prieta en 1989. Les critéres d'évaluation de base concernant l'estimation des caractéristiques de résistance aux séismes et les possibilités de renforcement des ponts existants sont développés; à cet effet, l'application de modèles analytiques de divers niveaux de difficulté servant de support au dimensionnement des structures en béton armé est démontrée.

**ZUSAMMENFASSUNG**

Über Schäden in Knotenpunkten von Balken-Stützenverbindungen in mehrstöckigen Brückenrahmen während des Loma Prieta Erdbebens von 1989 wird ein Überblick gegeben. Beurteilungskriterien für die Begutachtung und Einschätzung des Erdbebenverhaltens und der Verstärkung von bestehenden Betonbrücken werden aufgezeigt. Die Anwendung von analytischen Modellen im konstruktiven Betonbau von unterschiedlicher Komplexität wird demonstriert.

## 1. INTRODUCTION

The Loma Prieta (San Francisco) earthquake of October 17, 1989 reemphasized the vulnerability of structural concrete systems to cyclic displacements resulting from seismic attack. The dramatic collapse of a one-mile section of the Cypress Viaduct in Oakland can be traced to inadequate performance of the cap-column joint region in the supporting bents of the elevated bridge structure. Investigation of the joint reinforcement showed inadequate structural detailing of the joint region for the encountered seismic force levels. While the Cypress collapse was well publicized in the press and technical literature, only limited information can be found on similar structural joint damage to other elevated roadways in the San Francisco–Oakland Bay Area, see Fig. 1, which led to the temporary or permanent closure of several major freeway arteries including the Embarcadero Viaduct (I-480), the China Basin/Southern Freeway Viaduct (I-280) and the Central Viaduct (Highway 101) in San Francisco, as well as the Southbound Connector (I-980) in Oakland [1].

While most of these bridge sections were designed and built in the 1950's and 1960's, some of them were completed as late as 1985. This raises questions concerning not only past but current detailing practice for structural concrete joints. Design rules for beam and column members of structural concrete frame systems seem to be widely accepted and standardized in similar form around the world. However, as soon as aspect ratios of structural members approach unity, design guidelines and supporting design models show a wide range of different approaches. Limited detailed design models for these regions exist when full three-dimensional force transfer of axial, flexural and torsional structural action is required simultaneously. Also, most design models focus on single monotonic structural loading and do not address fully reversed cyclic loading patterns. Thus, the question arises whether a unified approach for design and analysis models in support of structural concrete detailing exists or if the state-of-the-art in structural concrete detailing still relies primarily on experience to design and detail complex members for realistic loading conditions.

Both analytical models to study the in-depth mechanism of structural concrete behavior through various limit states and design models developed to unify the structural detailing and design approach have seen comprehensive recent developments. In a direct extension of early structural concrete design principles by Ritter (1899) and Mörsch (1909), Schlaich et al. have developed a comprehensive design approach toward structural concrete detailing which ensures internal force transfer through discrete compression and tension (strut and tie) members, satisfying equilibrium by simple truss mechanisms [2]. This approach has become a powerful design tool since it allows a variety of detailing solutions as long as basic anchorage and stress limit states are observed, but most importantly it allows and forces the design engineer to develop a consistent design model resulting in an engineered solution rather than in a design resulting from a recipe application. Problems and limitations arise when design solutions based on inappropriate truss mechanisms are attempted and when the discrete member forces are of magnitudes which cause stress limit and anchorage problems and thus require a distributed or smeared approach. Parallel to the consistent strut and tie model development, Collins et al. developed the juxtaposed position of a smeared or distributed behavior model [3], which is based on homogeneous behavior of structural concrete even in its cracked state and resulting orthogonal principal compression and/or tension fields of the internal forces. Based on mechanical principles of an orthotropic homogeneous material, the orientation of the resulting stress fields is derived from compatibility and equilibrium conditions. The resulting stress fields are subsequently discretized in concrete and reinforcement action which forms the basis for a rational structural concrete design approach. Similar to the discrete strut and tie model, additional considerations for anchorage and local concentrated force transfer are required and limitations exist where either reinforcement is heavily concentrated rather than distributed, and where structural action results in a few large cracks rather than in the ideal distributed (smeared) crack pattern. Thus, while both design models are different in the approach, they are rather complementary in the overall design process, especially when in addition to the force transfer in the joint or member, deformation limit states also need to be considered.

Both of the above models provide comprehensive design approaches to structural concrete detailing but are fully applicable only when simple monotonic loading conditions exist up to design levels with sufficient margin to the ultimate limit state. Where deteriorating bond phenomena along the reinforcement, opening and closing of cracks under reversed cyclic loading, deterioration of concrete contribution in developing local failure mechanisms, and the development of ductile hinges (which incorporate all of the above aspects) are present, the above models may not be adequate and additional considerations to both design approaches are needed as outlined by Paulay et al. for structural concrete joints under seismic action [4]. It will be shown in the following that, with these additional considerations, the above design models can also be directly applied toward the design of retrofit measures of existing critical structural concrete regions as found in the San Francisco double-deck freeways. Further, it will be shown that complete failure sequences and limit states of these structural systems can be traced using advanced nonlinear analytical structural concrete models.

The critical role of structural concrete joints in beam-column systems and their behavior under seismic loading is evaluated in this paper on the example of joint performance in elevated bridge structures during the Loma Prieta earthquake and the applicability of various design and analysis models to the structural concrete joint problem is demonstrated, both for the assessment of joint performance during the earthquake and subsequent repair and retrofit strategies.

## 2. SEISMIC PERFORMANCE ASSESSMENT

### 2.1 General Assessment Approach

To assess the expected seismic performance of structural concrete beam column joints, it is important that the joint under consideration is evaluated in direct relationship to the actual adjacent member capacities. This requires a state or capacity determination of adjacent beams and columns with consideration of (1) actual material properties at the time of evaluation, i.e., probable concrete strength, not the design strength $\sqrt{f'_c}$, actual stress strain behavior for the reinforcement not nominal specified design yield levels, (2) proper consideration of axial load effects, (3) proper consideration of possible confinement effects from transverse reinforcement, (4) reduced concrete shear contribution in areas of large fully reversed cyclic deformation, and (5) realistic bond and anchorage estimates particularly for large diameter reinforcing bars.

A preliminary performance assessment of joints comprises the following steps: *Step I:* Realistic member capacities based on the above considerations are derived for both flexure and shear, and the critical failure mechanism is determined by direct comparison of the shear capacity with the plastic flexural limit state shear $V_p$ derived from the appropriate flexural plastic hinge failure model of the member. If $V_p$ is larger than the calculated shear capacity, a potentially brittle shear failure can be expected without the formation of ductile flexural plastic hinge mechanisms. *Step II:* Based on the possible member failure mechanisms, the expected global collapse mechanism for the complete structural system is derived by comparing combined dead load and lateral seismic force action with the derived capacities. *Step III:* From the identified systems collapse mechanism, critical joint forces can now be determined at the collapse state and a direct comparison with most probable joint capacities will indicate if joint distress degrades the capacity of the collapse mechanism or if the joint behaves as ideally assumed within or close to the elastic range. *Step IV:* Finally, equivalent seismic base shear forces are estimated corresponding to the lateral force level which causes collapse based on the above failure mechanism. Excessive joint distress can lead to a reduction of this base shear coefficient, particularly when a large number of cyclic load reversals and the associated joint degradation is considered.

Application of the above preliminary seismic assessment procedure to the San Francisco double-deck bridge bents has shown that the joints did not meet design criteria for earthquake resistant ductile structures summarized by Paulay et al. [4] as: (1) joint strength should exceed the maximum strength of the weakest connecting member, (2) structure capacity should not be jeopardized by strength degradation in the joint, and (3) joint response should be elastic during moderate seismic disturbances.

The preliminary joint behavior assessment outlined above can be supplemented and refined by more detailed analysis and design models as demonstrated in the following for one specific case study performed following the 1989 Loma Prieta earthquake.

### 2.2 The Oakland Southbound Connector, I-980, Bent 38

A single-deck outrigger bent (bent #38) with only a 0.92 m (3 ft) outrigger cap beam extension past the superstructure on I-980 featured heavy joint damage as shown in Fig. 1c,d,e. Built in 1985, the column was well confined with an interlocking spiral, see Fig. 3, however, this spiral did not continue into the joint region where it was replaced by a 5 gauge wire spiral with D = 5 mm at 10 cm ($\phi$ 0.2 in. @ 4 in.). Also, the cap beam reinforcement, aside from the top and bottom bars, see Fig. 3, did not extend into the joint region.

A capacity check on the cap beam and column capacities showed that the cap beam capacity is critical for positive moment due to the insufficient anchorage length of 1.8 m (72 in.) for the D = 57 mm (#18) bars which, based on ACI 318-89, require a basic development length of 3.0 m (117 in.) which is likely to be on the conservative side. In the other loading direction (negative moment in the joint), the column capacity is critical. Joint shear force levels derived from simple stress models, Fig. 4, show joint shear stress levels of $0.37\sqrt{f'_c}$ MPa ($4.3\sqrt{f'_c}$ psi) and $0.5\sqrt{f'_c}$ MPa ($6.0\sqrt{f'_c}$ psi), under positive and negative moment loading, respectively, which are both above an assumed level of $0.33\sqrt{f'_c}$ MPa ($4.0\sqrt{f'_c}$ psi), where diagonal tension cracking in the joint can be expected. Since the shear capacity of the 5 gauge wire spirals does not add significant joint shear capacity, the formation of any flexural hinge mechanism in adjacent members was inhibited.. This explains the encountered diagonal joint crack patterns during the Loma Prieta earthquake, see Fig. 1c,d,e.

In addition to the diagonal crack patterns, large areas of cover concrete spalling along the outer cap corner as well as a ruptured D = 57 mm (#18) reinforcement bar which was bent on a 45 cm (1'–6") radius were observed, see Fig. 1d,e.

The first phenomenon of cover concrete spalling can be explained with the fully reversed cyclic loading. Under negative moment, flexural cracks open on the cap surface as shown in Fig. 3 and under subsequent positive moment loading, the entire compression force has to be transferred through the negative moment reinforcement until the cracks can close. These high compression forces in the negative moment reinforcement transferred to the concrete by bond, have a tendency to spall off the concrete cover between flexural cracks developed in the previous tensile excursions. The second phenomenon, the ruptured reinforcing bar, see Fig. 1d, points to a potentially critical problem which needs further investigation. Common ultimate strain levels in D = 57 mm (#18) $f_y$ = 414 MPa (Grade 60) bars are in a range from 7 to 15%. Introducing a R = 45 cm (18 in.) radius bent into a D = 57 mm (#18 or $\phi$ = 2.25 in.) bar causes strain levels of $D/(2R) = 225/(2 \times 18) = 6.25\%$, which is close to the ultimate strain range. The very low strain reserves and possible strain aging effects which raise the notch ductile temperature at which steel will fail in a brittle mode can cause sudden failure in these bent bars at very low additional strain levels.

In addition to the simple stress models of the knee bent joint, detailed nonlinear finite element simulations, see Fig. 2, based on extended compression field principles were performed to determine analytically failure modes and joint deformation contributions to the overall bent deformations. Reinforcement development of straight bars was modeled by assuming a reduced yield level in the anchor zone decreasing linearly from the full yield at the ACI 318-89 basic development location to zero at the bar end. Subsequently derived yield patterns in bar anchorage regions therefore are indicative of bond failure or slip. Superimposed to the dead load case, the half bent, Fig. 2, was subjected to lateral force, and force-deformation envelopes with major event indicators, depicted for positive moment loading in Fig. 5, were obtained. Associated crack, slip/yield and first crushing patterns, see Fig. 5, indicate the failure mechanisms in the joint region. The heavy yield in the joint center, both horizontally and vertically, indicates the deficiency of both vertical and horizontal joint shear reinforcement, particularly under negative moment on the joint. Also, the crushing of the concrete in the outer joint region under the bent negative moment reinforcement under loading to the left or negative moment indicates the high compressive stress state and associated transverse prying forces in this region and the need for sufficient transverse confinement reinforcement, as outlined by Schlaich et al. [2], for strut and tie models for negative moment knee joints.

Since joints should be detailed based on capacity considerations such that the major inelastic action occurs in the cap or column, and since they should remain effectively elastic for small seismic disturbances (Paulay et al. [4]), the nonlinear finite element analysis was repeated with a linear elastic joint for a direct comparison of deformation limit states. As can be seen from Fig. 5, while joint deformations did not contribute significantly to the initial overall structural deformations, the failure mode in the positive moment direction shows improved ductile behavior when the failure mechanism is shifted from joint distress to flexural cap beam hinging. It should be noted that the cap beam capacity still may be artificially low due to the reduced yield strength in the bottom D = 57 mm (#18) reinforcing bars based on a reduced development length according to ACI 318-89. Negative moment loading behavior is also improved by forcing the yield mechanism clearly into the column (not shown). Since the bent joint failure is the critical link in the overall behavior, repair/retrofit of these joints is a logical next step.

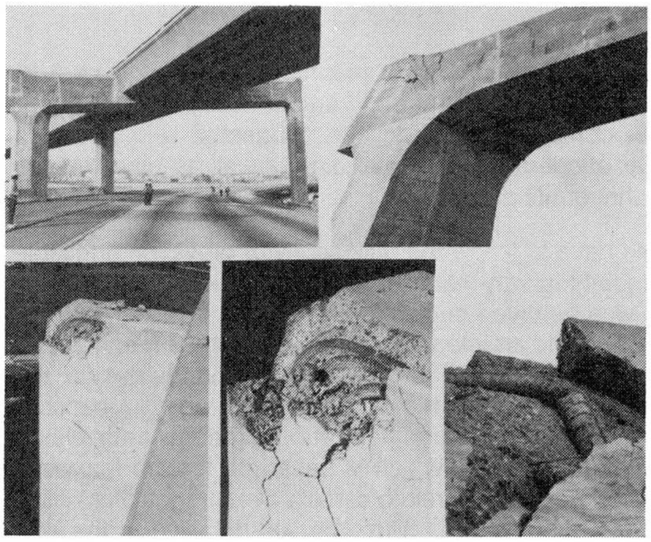


Fig. 1 Outrigger bent joint damage

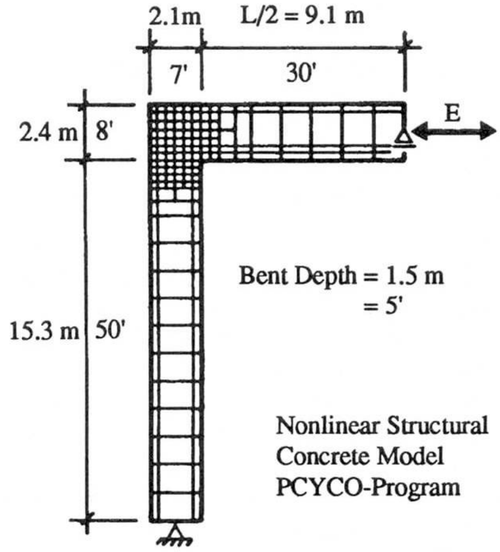



Fig. 2 Analytical bent model

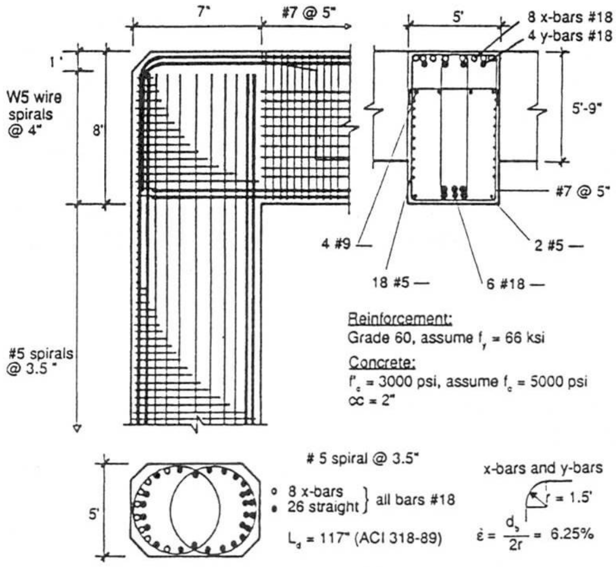



Fig. 3 I-980, #38, Reinforcement details

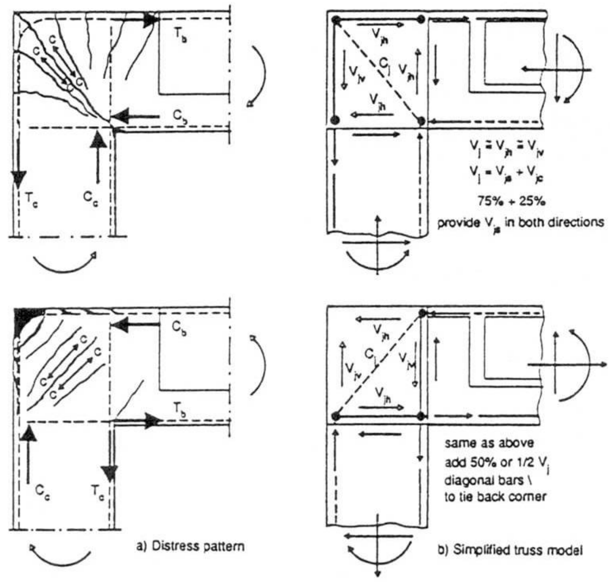



Fig. 4 Joint behavior models

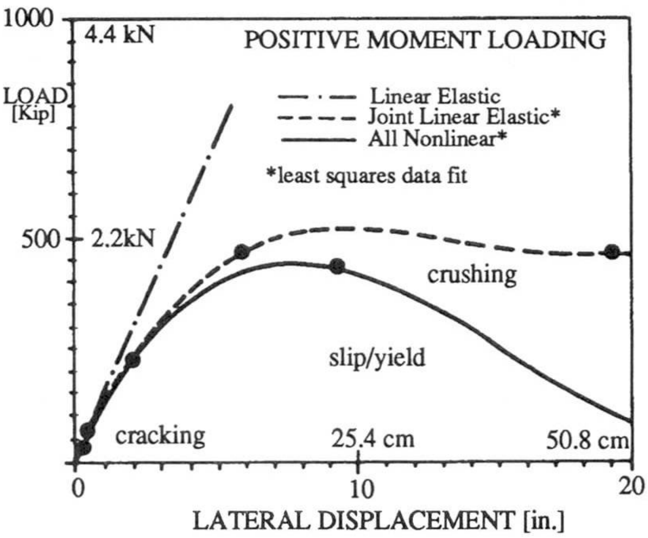



Fig. 5 Positive moment limit states

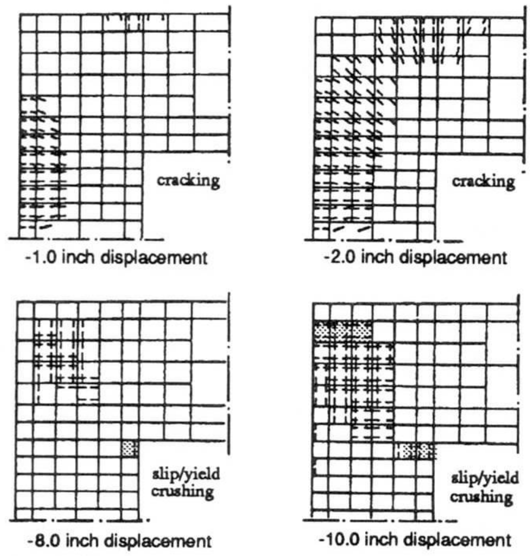



Fig. 6 Negative moment distress patterns

## 3. REPAIR AND RETROFIT

Cap/column joints in elevated roadways are probably the most difficult members in a bridge bent to successfully repair and/or retrofit for seismic loading. Both horizontal and vertical heavy reinforcement patterns from cap beams and columns provide for complex geometric and congested reinforcement patterns in the joint. Also, the critical aspects of bar anchorage within the joint as well as plastic hinge development directly adjacent to the joint complicate the retrofit design.

Criteria for seismic retrofit design of cap/column joints have to follow the philosophy of providing a reliable ductile structural system. Since ductility within the joints is very hard to achieve, the joint retrofitting is geared toward the formation of clearly defined and well behaved ductile plastic hinges in either the cap beam or the column. In many bridge decks, the cap beam is an integral component of the superstructure which would make post-earthquake repair in this member difficult. Thus, frequently in bridge design, the ductile framing system is provided by reliable column hinges. For the retrofit design, again a capacity design approach should be employed which ensures in the case of the joint retrofit predominantly elastic behavior of the joint region. This can be achieved by joint design which is based on factored nominal column design moments, e.g., $1.5 \times M_n$, where the factor accounts for reinforcement overstrength, including strain hardening, confinement effects and concrete strength increase with time. In detailing of the joint repair retrofit, the congested reinforcement layout within the joint as well as high nominal joint shear stress levels typically require an increase in size of the joint region.

Detailing for repair or retrofit of the damaged knee joint on I-980, bent #38, can be derived using either strut and tie models as outlined in Fig. 3 with additional considerations as outlined in [2] for transverse splitting under negative moment and tie back for fully reversed cyclic loading, or directly from the force states derived from the compression field analysis, see Fig. 5. A possible repair measure consists of providing an additional 23 cm (9 in.) concrete jacket around the existing joint with horizontal and vertical distributed joint reinforcement and a diagonal corner tie back based on force and reinforcement quantities derived in Fig. 4. The damaged joint concrete can either be removed and the joint rebuilt completely or the damaged joint can be epoxy injected subsequent to removal of loose concrete, roughening of the interface, and adequate doweling bonding of the added structural concrete jacket.

## 4. CONCLUSIONS

The assessment of beam-column joint performance during the 1989 Loma Prieta earthquake has shown that compression field based analysis and design models are invaluable tools to investigate existing capacities, failure modes, and deformation limit states. Strut and tie models are particularly suited for design of new joints and retrofit and repair measures. However, in capacity assessment, where dependence on distributed concrete tensile stresses is essential and where bond forces cause distributed shear input, the basic strut and tie model is limited. The detailing of structural concrete joints subjected to fully reversed cyclic loading requires additional design detailing considerations which account for opening and closing of large flexural cracks and the associated compression force transfer through the reinforcement. Due to the large column height, the joint deterioration was shown to contribute insignificantly to the initial structural deformation, while non-ductile joint failure resulted in substantially reduced ultimate deformation capacities.

## REFERENCES

[1] PRIESTLEY M.J.N. and SEIBLE F., Assessment of Bridge Damage During the Loma Prieta Earthquake. Structural Systems Research Project, Report No. SSRP-90/01, March 1990.

[2] SCHLAICH J. and SCHÄFER K., Konstruieren im Stahlbetonbau. Betonkalender 1989, Verlag Ernst & Sohn, pp. 563-715.

[3] COLLINS M.P., Towards a Rational Theory for RC Members in Shear. ASCE Journal of the Structural Division, Vol 104, April 1978, pp. 649-666.

[4] PAULAY T., PARK R. and PRIESTLEY M.J.N., Reinforced Concrete Beam-Column Joints under Seismic Actions. ACI Journal, Vol. 75, No. 11, November 1978, pp. 585-593.