

DEVELOPMENT OF SEISMIC STEEL REINFORCEMENT PRODUCTS AND SYSTEMS

SP-184



Development of Seismic Steel Reinforcement Products and Systems



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PREFACE

Over the past decade there have been many changes to the traditional range of reinforcing systems available for engineers and contractors. These changes have included development of new types and styles of welded wire reinforcement. With new steel-making technology and the latest practices of cold-working rod to wire, the industry is producing higher strength and higher ductility wire and welded wire for more structural uses. New headed reinforcing bar criteria is discussed that promises mechanical anchorage of reinforcing to eliminate or reduce development lengths and ease construction over conventional hooks. There is new guidance criteria to qualify mechanical reinforcement splices for designs requiring high-strain energy capacity.

At the ACI 1997 Spring convention in Seattle, Wash., ACI Committee 439, Steel Reinforcement, sponsored a full day technical session comprised of two parts. Both parts were a mix of various reinforcement products and systems. The presentations provided state-of-the-art coverage of important developments in reinforcing systems that have occurred in recent years. Seven papers were submitted for this symposium volume that cover welded wire reinforcement applications and design approaches, headed reinforcing bar applications and mechanical reinforcement splice system design, and performance standards.

These papers will provide engineers and contractors with up-to-date information on new technologies that are available now to improve the performance of reinforced concrete structures, especially in zones of high seismicity and to make design and construction more cost effective.

The manuscripts in this publication were assembled by R. H. Reiterman, who also ensured that each was reviewed according to Institute publication policies.

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Interrelationships between Reinforcing Bar Physical Properties and Seismic Demands

by J. McDermott

This paper (Title no. S-17) was published in the March-April 1998 ACI *Structural Journal*, p. 175-182. Therefor, the following is a summary of the paper, plus a postscript included in the convention presentation.

Reinforcing bar physical properties are main determinants for reinforcing-bar seismic demands. Consequently, seismic codes set appropriate single upper or lower limits on reinforcing bar yield strength and tensile/yield ratio, but they do not consider the variable-parameter effects of the shape of the reinforcing-bar stress-strain curve on what tensile/yield ratios and ductilities should realistically be required of reinforcing bars in seismic-resistant structures. Therefor, a theoretical study was performed to evaluate the effect of range of allowable steel yield strength, shape of steel stress-strain curve (strain and tangent modulus of elasticity at onset of strain hardening), and beam slenderness (S/d , where S is the clear span length and d is the effective depth to the reinforcing bar centroid, Figure 1) on the minimum values of steel tensile/yield ratio and useful ductility that are necessary to accommodate 2 % seismic drift by plastic hinging at the ends of beams, Figure 1, of concrete rigid frames reinforced with Grade 60 steel reinforcing bars.

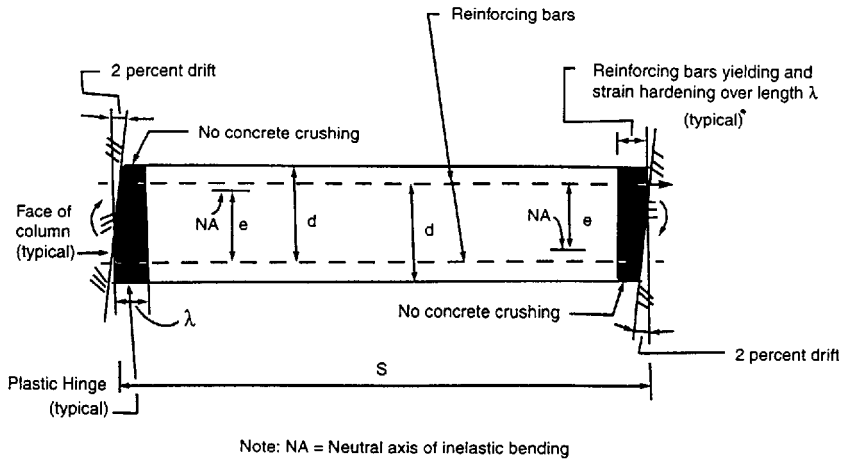
Keywords: ductility; modulus of elasticity; reinforcing bar

The analysis strategy included (1) relating S/d to e/L , where L is the beam elastic length and e is the depth from the reinforcing bar centroid to the neutral axis, (2) stating a reasonable definition for tensile stress along the beam, in terms of the unknown length of plastic hinging, bottom of Figure 1, (3) evaluating the half-beam elastic rotation as $(\sigma_y L)/(4Ee)$, where σ_y is the steel yield strength and E is the steel modulus of elasticity, (4) for given values of the parameters, evaluating the total plastic-hinge rotation as 2 % minus the half-beam elastic rotation, (5) expressing the strain definitions of the steel stress-strain curve in terms of a plastic proportion parameter (plastic-hinge length/beam elastic length), top of Figure 2, (6) equating the already-calculated total plastic hinge rotation to the product of the unknown plastic-hinge length times the average curvature within the plastic hinge (reinforcing bar average strain/ e) to obtain a quadratic equation relating the plastic-proportion parameter, yield strength, tangent modulus of elasticity, strain ratio (s , Figure 2), e/L , and total plastic hinge rotation, bottom of Figure 2, giving a numerical evaluation of the plastic-proportion parameter, and (7) evaluating the terminal point (required-useful-ductility abscissa and required-tensile-strength ordinate) of the stress-strain curve, top of Figure 2.

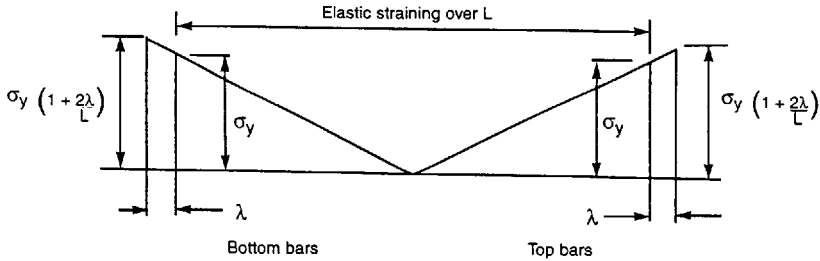
The calculation results, summarized in Figure 3, indicate that the generally accepted requirement of a minimum tensile/yield ratio of 1.25 for reinforcing steel in seismic design is both prudent and probably satisfactory. However, considering the high steel strains, particularly about 0.1 in./in. for squat beams, the study strongly suggests that it would be prudent, for seismic-

resistant reinforced concrete structures, to either specify A706 steel or specify A615 steel to have at least a 10 % minimum elongation in 8 inches (203 mm). For reinforcing in very squat beams (with close to the minimum length-to-depth ratio permitted by ACI 318-95) the calculations indicate that the stress-strain curve for the reinforcing steel should not exhibit too low a tangent modulus of elasticity at onset of strain hardening (possibly causing ductility capacity to be exceeded) or too high a tangent modulus (possibly causing tensile strength capacity to be exceeded).

Postscript. The desired useful ductility refers to a 10 % strain at only one station at the end of the plastic hinge, so that the corresponding average bar strain within the plastic hinge would be closer to 5 %. That, considering comparisons of plastic-hinge lengths (calculated in the present study to range from 10 % to 68 % times the beam effective depth) with lengths of reinforcing bar mechanical splices (e.g., ranging from about 6 to 36 inches for #18 reinforcing bars), provides reasonable justification for requiring 4 % average tensile straining to determine the loading for the cyclic testing of seismic-rated mechanical splices of reinforcing bars.

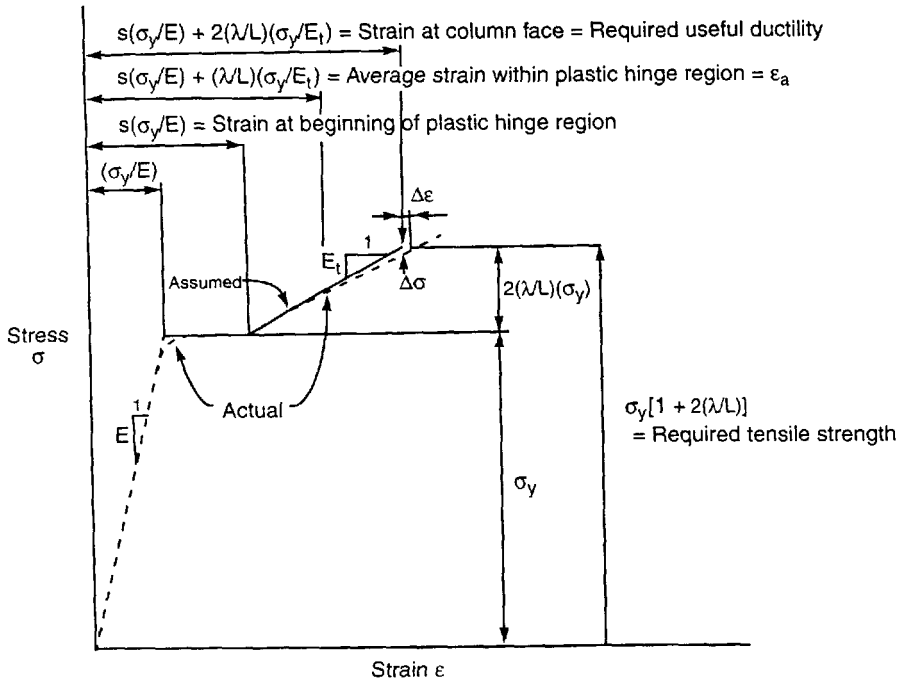


(a) Elevation view of rigid-frame beam subjected to maximum allowable column drift



(b) Assumptions for tensile stresses in reinforcing bars

Fig. 1—Simplifying assumptions for analyses.



Plastic hinge rotation, θ , assumed = $\lambda\epsilon_a/e$, from which the
portion of span length within one plastic hinge = $\lambda/L = \frac{\{-s(\sigma_y/E) + \sqrt{[s(\sigma_y/E)]^2 + 4\theta(e/L)(\sigma_y/E_t)}\}}{2\sigma_y/E_t}$

Fig. 2—Reinforcing bar straining within region of plastic hinging.

- Straining up to beginning of strain hardening
- Strain hardening to maximum for slender beam ($e/L = 1/24$)
- Strain hardening to maximum for squat beam ($e/L = 1/5$)

CONVERSIONS

1 ksi = 6.895 MPa
60 ksi = 414 MPa
78 ksi = 538 MPa
100 ksi = 689 MPa
500 ksi = 3450 MPa
1 in./in. = 1 mm/mm

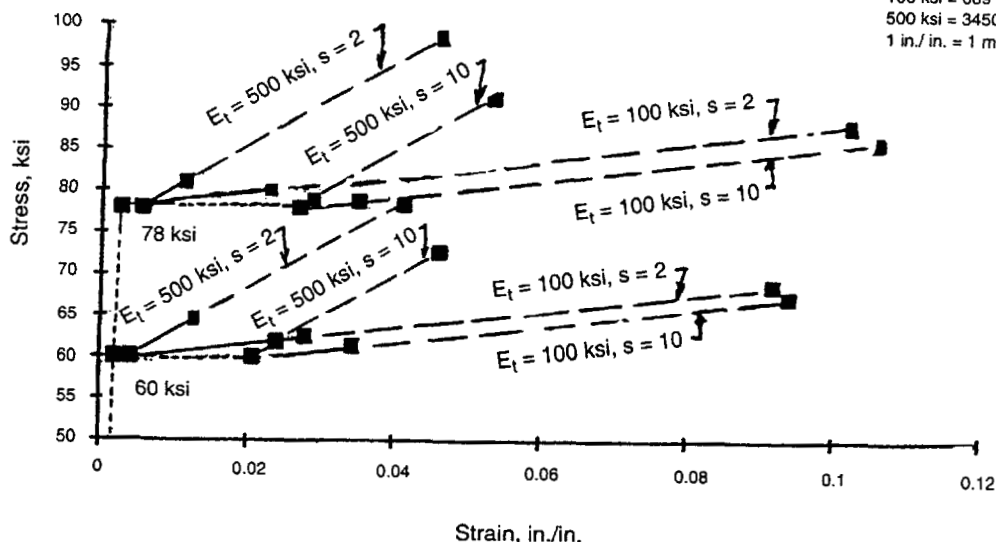


Fig. 3—Effect of beam slenderness and shape of steel stress-strain curve on seismic demands for reinforcing bar ductility and tensile strength, based on 2 percent drift.

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Design and Performance of Bridge Cap Beam/Column Joints Using Headed Reinforcement and Mechanical Couplers

by S. Sritharan, J. Ingham, M. Priestley, and F. Seible

ABSTRACT

The application of headed reinforcement as a replacement for conventional reinforcement was investigated in two projects relating to the seismic design of bridges. In the first project, a test unit composed of a column, cap beam, footing and knee joint was designed entirely with headed reinforcement, and in the second project a test unit representative of a multi-column bridge bent was investigated, having a cap beam design utilizing both headed reinforcement and a mechanical coupler system. In both investigations the use of recently-developed reinforcement products facilitated simplified detailing, particularly in the cap beam/column joint region, resulting in reduced reinforcement congestion in the joint zone and improved constructability. The design and performance of the test units under simulated seismic loading are presented.

Keywords: cap beam; column; footing; reinforcement

INTRODUCTION

The 1989 Loma Prieta earthquake caused significant damage to bridge stock in the San Francisco Bay area [4]. This damage, combined with post-earthquake analysis, identified several design shortcomings in existing bridge structures [8], emphasizing the need for a critical review of California bridge seismic design procedures. Consequently, comprehensive research programs were initiated at several institutions in California investigating possible retrofit techniques for existing structural deficiencies and establishing seismic design guidelines for modern bridges.

One of the design deficiencies identified in existing bridges was inadequate detailing of cap beam/column connections, whose performance is critical at the survival limit state. Collapse of, or damage to a number of bridges in the Loma Prieta earthquake, including the double-deck Cypress viaduct, was attributed to poor detailing of the beam/column joints [4]. As outlined in the following section, when joints are detailed in accordance with the conventional design philosophy based directly on shear forces, considerable reinforcement congestion is likely. In this paper, testing conducted at the University of California at San Diego (UCSD) is used to demonstrate that simplified reinforcement details can be obtained for structural members when utilizing new reinforcement products such as headed rebars and mechanical couplers in conjunction with joint force transfer models. This significantly reduces congestion problems, particularly in cap beam/column connections, while providing satisfactory overall seismic performance for the structure.

SEISMIC DESIGN PROCEDURE

The capacity design philosophy, which now forms the basis for bridge design in most seismically active countries of the world, emphasizes ductile structural performance under severe seismic loading. In concrete bridges, ductile response is typically developed by forming plastic hinges at the top and/or bottom of bridge columns. The reinforcement located in these hinge regions is carefully detailed to accommodate large inelastic reinforcement strains and local member rotations, allowing seismic energy to be dissipated in the form of hysteretic damping. The remaining elements of the structure are protected from significant inelastic action by providing a strength hierarchy sufficient to cope with potential strain hardening and uncertainties in material strengths.

The elastic design of non-critical structural members (typically the bridge cap beam) is generally well established. However, the design of joints which the bridge members frame into is in comparison poorly understood and is not specifically addressed in bridge design codes such as AASHTO [1] and Caltrans specifications [3]. Two alternative methods may be considered for detailing bridge joints to ensure satisfactory performance complying with capacity design criteria.

1. Building Code Approach

In building codes that require specific design of joints, such as NZS 3101:1995 [2] and ACI 318-95 [12], the design of beam/column connections is based upon the maximum joint shear force which is expected at the ultimate limit state. If a similar approach is considered for the design of bridge joints, robust joint performance is ensured. However, this design procedure, when applied to bridge joints, has been found to require an unnecessarily conservative amount of reinforcement, resulting in major congestion within the joint [6,9,11].

2. Rational Force-Transfer Method

In research studies at UCSD the design of bridge joints has been investigated [5 – 11] using force transfer mechanisms which ensure a satisfactory pathway for forces through the joint. It has been shown experimentally that good seismic bridge joint response can be obtained using significantly less than the code-recommended quantity of joint reinforcement when the design is based on force transfer mechanisms.

In a well-designed bridge joint, the flexural capacity of the column, which frames into the joint, dictates the shear demand within the joint. Consequently, when a concrete bridge bent is designed with high longitudinal reinforcement content in the columns ($\rho_l \geq 2.5\%$), the required reinforcement in the joint region based on force transfer models can also create congestion problems. In such circumstances, joint reinforcement congestion can be alleviated using alternative reinforcement products as demonstrated in the two large-scale tests presented in this paper.

¹ Headed Reinforcement Corp., 11200 Condor Ave., Fountain Valley, CA 92708.

RECENTLY-DEVELOPED REINFORCEMENT PRODUCTS

In recent years a large number of products have become available in the United States to simplify the anchorage and lap splicing of conventional reinforcement. Two such products, namely headed reinforcement and mechanical couplers (or bar extenders) were used in large-scale experiments on bridge structural systems at UCSD. These reinforcement products were designed and manufactured by Headed Reinforcement Corporation¹.

When new reinforcement products are used in seismic design, it is *not* always sufficient to assess these products based upon monotonic stress-strain response. Depending upon the design, it may be necessary to ensure that the product can withstand cyclic inelastic strains to produce a satisfactory structural response when subjected to earthquake loading. In all cases, it is required that the ultimate capacity of the reinforcement product be not less than the ultimate capacity of the parent reinforcing bar. Description and relevant laboratory tests performed on the headed reinforcement and bar extender products are as follows.

1. *Headed Reinforcement*

Headed reinforcement provided in the knee joint unit was manufactured by friction welding forged circular heads to conventional ASTM A706/A706M-90 grade 60 (414 MPa) weldable reinforcement (see Fig. 1a). In the process of quality-assurance tests performed by the manufacturer, the headed reinforcement exhibited cyclic behavior identical to that of the parent reinforcing bar with ultimate failure consistently occurring in the reinforcement, not at the friction weld. To verify that the full capacity would be developed in the parent reinforcing bar, a total of eight headed bars were randomly selected during construction of the knee joint unit at the UCSD facilities, and tested in uniaxial tension. In all cases, fracture of these randomly selected samples occurred in the reinforcing bar, away from the friction weld.

2. *Mechanical Coupler (or Bar Extender) System*

The mechanical coupler system used in the second test unit incorporated two fixtures which coupled two headed reinforcing bars using standard threads (see Fig. 1b and 1c). The reinforcement heads were formed using a technique

suitable for field implementation by heating the bar ends and forging them with a hand-held hydraulic device. Quality-assurance testing conducted by the manufacturer consisted of uniaxial tension tests on the mechanical coupler system, ensuring that full reinforcement capacity could be developed when using the coupler. Several couplers were also tested at UCSD with failure occurring in a similar fashion. This reinforcement product was used exclusively in the cap beam design of the test unit. Consistent with the capacity design philosophy, the beam reinforcement connected by the coupler was not expected to undergo any significant inelastic strains.

EXPERIMENTAL INVESTIGATION

1. *Bridge Knee Joint*

A sequence of four 1/3rd scale units having a geometry (see Fig. 2) based upon the pin-based two-column I-980 Bent #38 in Oakland, California, were tested under simulated seismic conditions [5]. These units represented the as-built joint and its' prototype repair, together with a retrofit and redesign of the as-built joint based upon rational force transfer models. The reinforcement details of the redesigned joint are shown in Fig. 3, clearly indicating that there was major congestion of reinforcement in the joint region of this unit despite the use of rational design models. Strain data confirmed that an appropriate quantity of joint reinforcement had been provided in the redesigned unit, enabling a column plastic hinge to form at the joint interface for both directions of loading without major damage to the cap beam or joint region.

A further unit was then designed using headed reinforcement throughout, eliminating the need for hooks with horizontal tails to the embedded longitudinal column reinforcement and vertical tails for the embedded longitudinal cap beam reinforcement [7]. Combining the use of rational procedures for assessing realistic joint force levels with careful attention to detailing of the headed reinforcement in the joint region ensured that flexural forces acting at the joint boundaries could be effectively transmitted through the joint via a single diagonal strut for each direction of loading, as shown in Fig. 4. The resulting design required no conventional joint reinforcement other than joint cross-links, which were provided to prevent excessive out-of-plane joint dilation and assist in confinement of the joint core. Details of the joint reinforcement for this unit are shown in Fig. 5, and footing reinforcement details for the test unit are depicted in Fig 6. Clearly the elimination of joint hoops or stirrups greatly reduced congestion of reinforcement in the joint zone, as did elimination of 90° bends to the embedded longitudinal reinforcement.

This reduced difficulties associated with construction of the test unit when compared to difficulties encountered when constructing the previous joint (see Fig. 3). Also relevant was the headed vertical ties between the top and bottom reinforcement mat of the footing, being considerably less difficult to place than the “J-hooks” used in conventional California Department of Transportation (Caltrans) designs [3].

When subjected to simulated seismic loading, performance of the test unit with headed reinforcement was very similar to the redesigned unit with conventional reinforcement, as indicated by the force-displacement records shown in Fig. 7 for the two units. The similarity between the response of two units is attributable to the fact that in both cases a column plastic hinge developed at the joint interface, and response was not dictated by joint performance until final stages of the test. Also important was that, as for the test unit with conventional reinforcement, the joint with headed reinforcement supported a maximum joint shear stress of $16.8\sqrt{f'_c}$ (psi units), significantly in excess of the ACI code limit of $12\sqrt{f'_c}$ (psi units) [12], despite having no conventional joint shear reinforcement.

At final stages of testing the unit with headed reinforcement exhibited significant spalling of joint cover concrete, followed by deterioration of joint struts with repeated cycling. Spalling of joint cover concrete also developed for the unit with conventional reinforcement, but in that case the large quantity of transverse reinforcement prevented deterioration of the joint core. Consequently, it was concluded that while headed reinforcement provided excellent anchorage to the longitudinal reinforcement within the joint, and aided in delineation of force paths through the joint, headed joint cross-links were less effective than conventional spirals in ensuring confinement of the joint core in situations where joint stress levels were expected to be of a magnitude sufficient to cause significant spalling of joint cover concrete, and where cyclic loading was expected to accelerate deterioration of the joint core.

2. Multi-Column Concrete Bridge Bent

Following a number of large-scale tests on individual bridge knee and tee joints incorporating as-built, retrofitted and redesigned reinforcement details [6,9], two multi-column bridge bents were designed and tested at UCSD [10]. These multi-column units incorporated joint detailing which had been developed using force transfer models and proven to be effective in the earlier series of tests. By adopting maximum feasible longitudinal reinforcement

levels in the columns, the joints of the multi-column units were subjected to maximum practicable shear demand in order to further establish design suitability. Also examined in this investigation was the overall seismic behavior of multi-column bents under seismic type loading. The test unit which incorporated headed reinforcement and bar extenders is discussed in this section.

A three-column bent from the Santa Monica Viaduct in Los Angeles, California, was selected as the prototype structure, with the test unit having appropriate modifications consistent with recent design recommendations [8]. Because of the test set-up adopted, it was established that in order to effectively represent the prototype structure it was only necessary to model two columns as shown in Fig. 8, one representing an interior and the other modeling an exterior column [10]. In addition, the columns were provided with typical Caltrans pinned-base details, which provide a reduced, but non-zero moment capacity. In Fig. 9 the test set-up and overall dimensions of the multi-column unit are presented. The seismic and gravity forces acting on the prototype structure were simulated at the 50% test unit scale using four hydraulics actuators, two mounted side by side horizontally (i.e., $\pm E$ in Fig. 9) and two vertically (i.e., $W3 \pm P$ and $3 \times W2$), and a vertical tie down (i.e., $W1$) in the long stub region supporting the bridge overhang.

The multi-column test unit was designed in accordance with capacity design philosophy, with plastic hinge locations at the top of columns, and the cap beam and joints designed to support actions corresponding to the column overstrength moment capacities. When a column hinging mechanism is developed for a multi-column bent, the shear demand upon each joint is dictated by the longitudinal reinforcement ratio of the column framing into the joint. Consequently, the columns of the test unit were designed with high reinforcement ratios to ensure that the maximum practicable joint shear was developed. A longitudinal reinforcement content of 4% has been considered the maximum feasible value in practical situations [8]. Hence, this ratio was adopted for the interior column, while a longitudinal reinforcement content of 3% was adopted for the exterior column in order to avoid excessive cap beam seismic moments. These column reinforcement ratios were significantly higher than commonly adopted values, which are in the 1.0% – 2.5% range.

The cap beam of the test unit was detailed to ensure that the longitudinal reinforcement did not yield. The headed longitudinal cap beam reinforcement had mechanical couplers located in two regions of the bent cap (see Fig. 11). As discussed below, continuous top and bottom longitudinal beam

reinforcement was preferred through the knee joint. Hence, couplers were located just outside the exterior joint to simplify construction. Mechanical couplers were also provided in the low bending moment region to reduce the bar diameter over zero lap length (see Fig. 1c and Fig. 11).

As noted previously, detailing of the joint regions was performed using force-transfer mechanisms in order to minimize the joint reinforcement and hence improve constructability. The tee joint of the test unit was designed considering that 50% of the total column tension force would be supported partly by external compression struts developing outside the joint in the cap beam as shown by D_2 in Fig. 10a. The reinforcement requirements for supporting such a joint mechanism, which include external ties to resist the vertical component T_v of D_2 and additional cap beam bottom reinforcement to carry the horizontal component ΔT_h , are discussed in detail in references [8] and [9]. When the tee joint was detailed accordingly, it resulted in significantly reduced amount of reinforcement within the joint when compared to an equivalent joint designed in accordance to the building code approach based upon the maximum joint shear forces. A similar design method was also employed in detailing the knee joint under *opening* moments.

For the response of knee joint under *closing* moments, the joint reinforcement was reduced by supporting a large proportion of the joint diagonal strut by the continuous beam longitudinal reinforcement just outside the joint, thus providing adequate clamping to the column tension bars (see Fig. 10b). This eliminated the need for any special joint reinforcement and only nominal reinforcement was required for closing actions. The continuous reinforcement in the cap beam was provided using short U-shaped headed rebars through the knee joint and then connecting to the headed longitudinal beam reinforcement close to the column face by means of bar extenders (see Fig. 11). Welding of the reinforcement in the joint region was not considered as a desirable alternative solution. The remainder of the joint diagonal strut was supported by the reinforcement provided in the long stub region. More complete details of the design may be found elsewhere [10].

Key reinforcement details of the test unit are shown in Fig. 11 while the force-displacement behavior when subjected to simulated cyclic loading is presented in Fig. 12. A satisfactory hysteretic response was obtained prior to the first cycle to displacement ductility 6. The horizontal displacement of the test unit at this ductility level corresponded to a column drift of 8.7 percent, which is significantly greater than that expected during design level earthquake loading. Gradual deterioration in seismic force resistance, seen in Fig. 12 during cycling at displacement ductility 6, was due to some damage occurring to the interior

(tee) joint, with satisfactory response obtained for the knee joint of the test unit. During the fourth push direction semi-cycle at $\mu_{\Delta} = 6$, the longitudinal reinforcement in the exterior column buckled in the plastic hinge region adjacent to the joint. When the load was reversed, one of the longitudinal bars fractured under tension due to low cycle fatigue, confirming that the flexural moment capacity of the exterior column was fully developed adjacent to the knee joint interface. This resulted in further degradation to lateral force resistance of the system. Given that damage to the tee joint occurred at a column drift of 8.7 percent, it was concluded that the reinforcement details of both joints were effective [10]. Detailed analysis of the joints using the experimental data has been recently completed [11]. This study found that the force transfer across the joints was in general consistent with the design mechanisms shown in Fig. 10 and that modifications to the design mechanisms are necessary to avoid joint damage at the ultimate limit state. Modified design mechanisms and the corresponding joint detailing procedure are incorporated in this document.

CONCLUSIONS

The use of headed reinforcement and a new mechanical coupler system in the seismic design of bridges is discussed in this paper, with particular emphasis on efficient detailing of cap beam/column joints. Following good seismic performance of a bridge knee joint having highly congested conventional reinforcement, a similar test unit was designed entirely with headed reinforcement and without any conventional joint shear reinforcement. The design of the latter unit was simplified due to readily identified force paths through the joint. The detailing of the joint and other structural members of the latter unit also ensured simplified construction while resulting in similar response under simulated seismic loading.

In the second test, a multi-column bridge bent was designed with a mix of conventional and headed reinforcement in conjunction with bar extenders (or mechanical couplers). The use of recently developed reinforcement products was aimed at reducing joint congestion and providing continuity in the longitudinal beam reinforcement over zero lap length. The response of the test unit when subjected to extreme seismic loading conditions confirmed that the alternative reinforcement details considered in the test unit were effective.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by Headed Reinforcement Corp. (HRC) and the California Department of Transportation (Caltrans) for the bridge knee joint test and the multi-column test respectively. The headed reinforcement and mechanical couplers used in both tests were provided by HRC. The efforts of the technical staff of the Charles Lee Powell Structural Research Laboratories towards the completion of this project are gratefully appreciated. Conclusions and opinions expressed in this paper are those of the authors' alone, and should not be construed as being endorsed by HRC or Caltrans.

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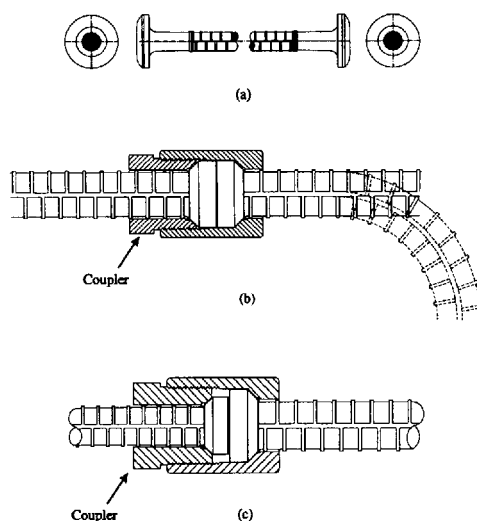


Fig. 1—Recently developed reinforcement products used in this study: (a) headed reinforcement; (b) bar extender (or mechanical coupler) system to provide continuity in reinforcement; and (c) bar extender for reducing diameter of reinforcement.

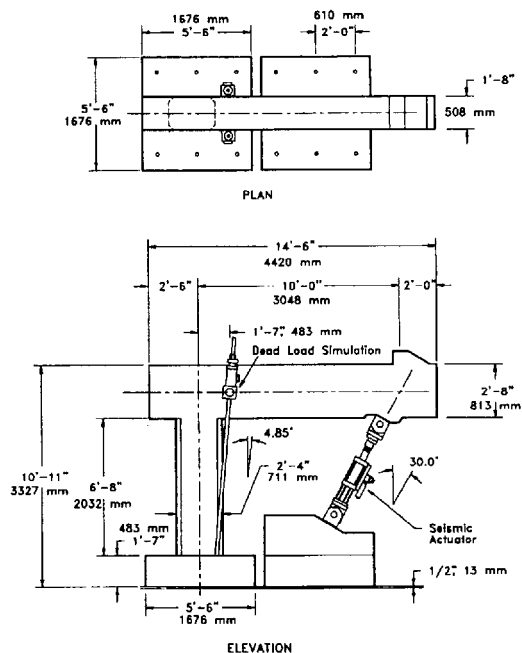


Fig. 2—Configuration of bridge knee joint test setup.

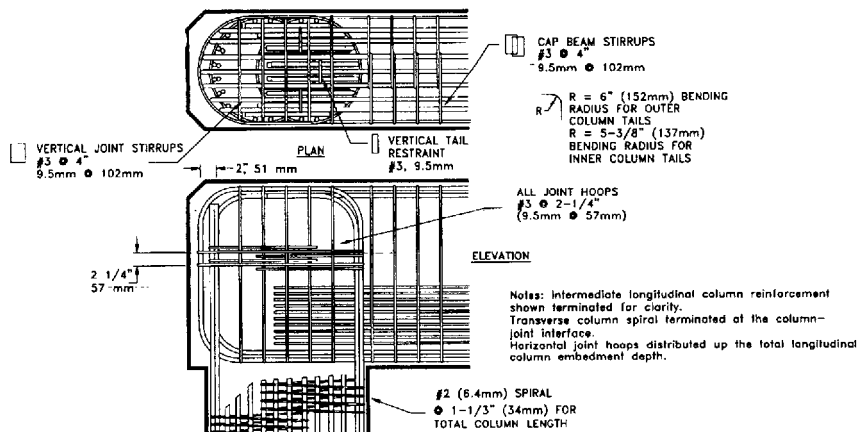


Fig. 3—Reinforcement details for knee joint with conventional reinforcement.

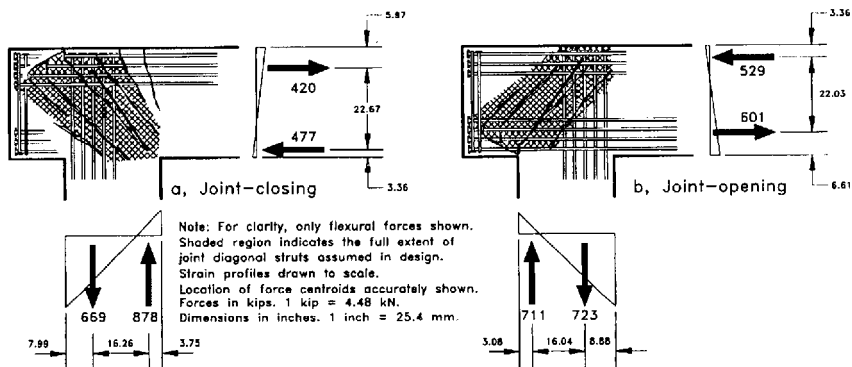


Fig. 4—Design force transfer mechanisms of bridge knee joint with headed reinforcement.

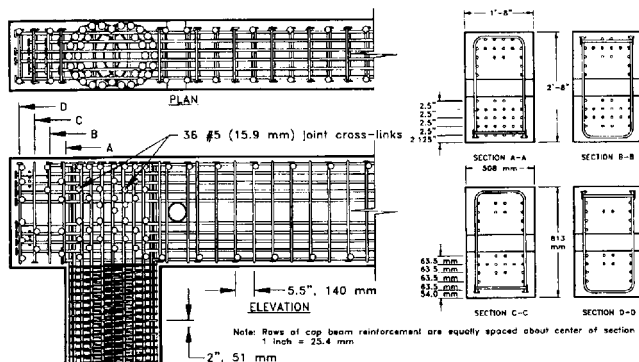


Fig. 5—Reinforcement details for knee joint with headed reinforcement.

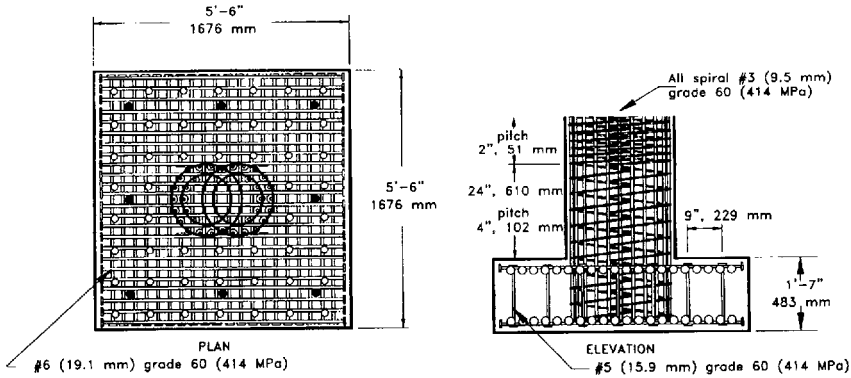


Fig. 6—Footing details of knee joint test model with headed reinforcement.

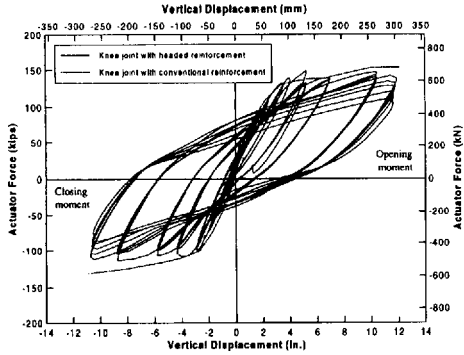


Fig. 7—Force-displacement history of headed reinforcement unit and response envelope of equivalent unit with conventional reinforcement.

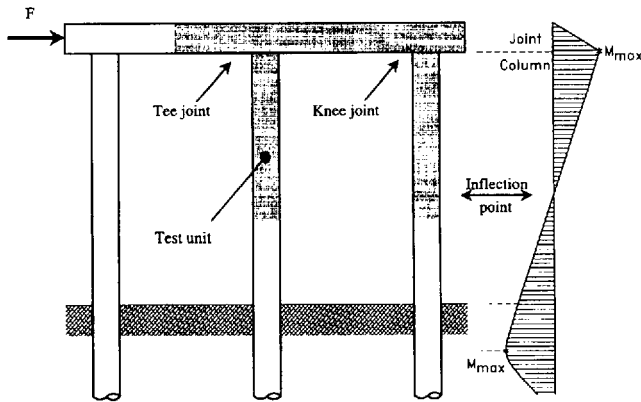


Fig. 8—Prototype multicolumn bent and test unit representative of original structure.

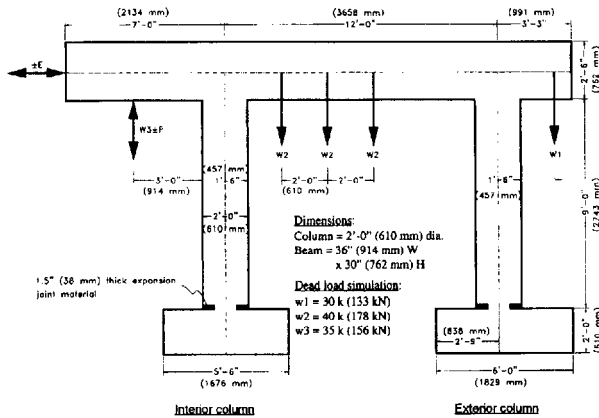


Fig. 9—Overall dimensions and test setup of multicolumn bridge bent.

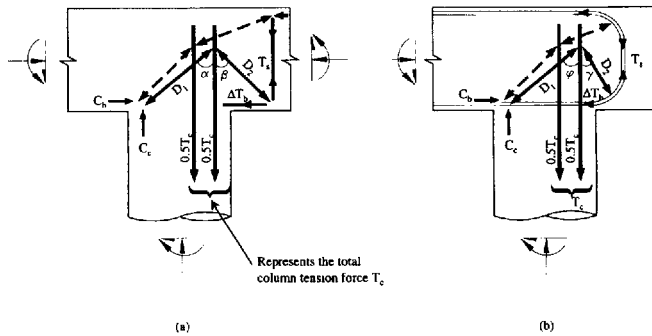


Fig. 10—Design force transfer mechanisms used for multicolumn test unit to ensure adequate anchorage of column reinforcement in cap beam [8,9]: (a) tee joint; and (b) knee joint under closing moments.

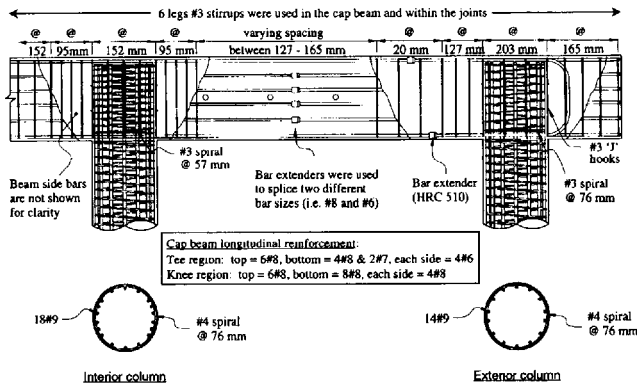


Fig. 11—Key reinforcement details of multicolumn test unit.

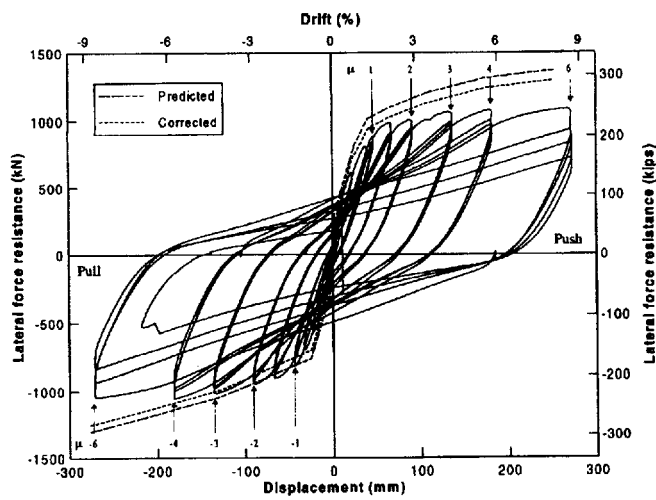


Fig. 12—Force-displacement response of multicolumn test unit [10].

Design and Detailing with Headed Reinforcement for Seismically Resistant Concrete Bridge Structures

by D. Berner, T. Dahlgren, and K. Dahl

Abstract

Headed reinforcement uses one or more anchorages, called heads, attached to the end(s) of steel reinforcing bars. Such heads serve to develop a bar in a relatively short distance, and can also better confine the interior concrete. For over a decade, headed reinforcement has had extensive field use in major structures subjected to cyclic fatigue and dynamic loading, as well as thorough laboratory testing on both bare steel bars as well as on concrete members with headed reinforcement. Such tests have also demonstrated the superior performance of headed reinforcement under seismic loading conditions, even in high moment zones, and joint regions.

This paper addresses both: (i) aspects of design and detailing with headed reinforcement for seismic resistance, and (ii) aspects of the concrete material performance as it is modified by headed reinforcement. Specific advanced design tools are discussed including empirical equations, strut-tie modeling procedures, a new membrane stress theory, and a new cyclic reinforcing bar bond-slip theory, together with design examples for bridge structures.

Currently, ACI 349, CSA 474, and several overseas codes provide design rules for headed reinforcement. Where necessary these rules may be supplemented by experience, engineering judgment, empirical guidelines, and test results. New standards, regarding the use of headed reinforcement in concrete, are pending with both ASTM and ACI 318; which when incorporated should further facilitate the design process.

Keywords: anchorages; design; dynamic loading; reinforcing bars

Potential advantages of the use of headed reinforcement include:

- (i) Shorter development lengths.
- (ii) Reduced steel congestion.
- (iii) Reduced bond-slip.
- (iv) Enhanced confined concrete strength.
- (v) Better control of stress paths through the member.
- (vi) Reduced member size.
- (vii) Enhanced member ductility.
- (viii) Efficiency of reinforcing steel placement during construction.
- (ix) Facilitates the use of high-strength lightweight aggregate concrete.
- (x) Facilitates detailing of non-linear regions, such as: joints, corbels, deep beams.
- (xi) Facilitates the use of larger bars to replace more numerous smaller bars.
- (xii) Extends compressive fields in the concrete to the ends of the head reinforcing bars, thus increasing the capacity of these compressive fields as compared to conventional bars.
- (xiii) Minimizes the risk of crushing the concrete under the head(s), as can occur beneath bent and hooked anchorages.

1.0 Introduction:

Headed reinforcement, as illustrated in Figure 1, uses one, or more anchorages, called heads, friction welded on the end(s) of steel reinforcing bars. These heads typically are designed to immediately, and fully, develop the tensile capacity of the reinforcement at the location where a head bears against the concrete (and/or underlying reinforcing bars) beneath the head. (It is also possible to design a head so that some bond length is required beyond the head in order to fully develop the tensile capacity of the bar, see ACI 349-90, Section 12.63) Furthermore, headed reinforcing generally uses steel that meets the requirements for ASTM 706; however, friction welding can be used to join heads and bars with a variety of compositions including stainless steel, and both low and high carbons steels.

Headed bars can be used as longitudinal, transverse (see Figure 2 for an example of the use of such headed reinforcing bars from the seismic retrofit of the foundations for the Richmond-San Rafael Bridge, in San Francisco Bay, California), or out-of-plane reinforcement, and are most commonly designed to function at, or below, the ultimate limit state of reinforced concrete members, in applications such as in offshore concrete platforms, which are currently the largest market for the use of headed reinforcement. However, headed reinforcement can

also be designed to function well in post-ultimate applications, such as in seismic resistant concrete bridge structures (see Ref. 1), which is the topic of this article.

While reading this paper many engineers will ask themselves several questions including:

- (i) "Why should I use headed reinforcement for seismic design?"; (ii) "How do headed reinforcing bars work?"; (iii) "How will headed reinforcement affect the concrete?"; and (iv) "How should I use headed reinforcement for seismic design?" Hopefully, after reading this paper these engineers will have at least some answers to these questions.

2.0 Benefits from the Use of Headed Reinforcement for Seismic Design

There are many reasons why engineers should design concrete structures using headed reinforcement. Some of these are addressed in greater detail in subsequent sections, however, the following provides an overview:

1. Constructability: Headed reinforcement can facilitate constructability not only because the heads eliminate the need for cumbersome hooks and tails which frequently must be tucked into congested areas, but also because a head can fully develop a larger bar, thus allowing for the replacement of more numerous smaller bars, with fewer larger bars. Also, as the heads are more compact than hooks and tails, the over-all dimensions of members can frequently be made smaller, thus reducing mass and materials. Occasionally, where restricted member dimensions make it impossible to install hooks and tails, the use of headed reinforcement can make it possible to fully develop bars that would otherwise be subjected to bond failures due to inadequate development length.
2. Performance in non-linear regions: Headed reinforcement is perhaps most valuable when used in non-linear regions. Non-linear regions are those portions of a member where plane sections remain plane after loading. In non-linear regions such as joints, corbels and deep beams strain distributions are inherently non-linear even in the elastic range; during a seismic event many linear regions can be transformed into non-linear regions due to material plasticity. In non-linear regions, headed reinforcement can enhance structural performance. Also, advanced analytical techniques such as the strut-tie and the Membrane Arch State, MAS, techniques discussed in Section 4, are required in non-linear regions.

Finally, the use of headed reinforcement helps to simplify such analyses by allowing the designer to have better control of the stress paths through the member and by reducing the designer's reliance on the bond of conventional reinforcing steel, which can degrade during an earthquake.

3. Enhanced concrete material properties: Headed reinforcement can enhance the performance of concrete in many ways, including: (a) aiding in the transformation of high-strength lightweight aggregate concrete from a brittle to a ductile material (see Ref. 2), thereby allowing seismic designers to take advantage of the reduced mass offered by lightweight concrete; (b) moderately increasing the ultimate strength of normal weight concrete by enhancing aggregate interlock due to the superior restraint offered by headed reinforcement against crack opening, which can lead to premature concrete failure; and (c) more efficient utilization of high strength steels and concrete mixes by minimizing bond-slip and ductility problems (see Ref. 3).

4. Superior Designs Compared to Conventional Reinforcing: Conventional reinforcement has sustained seismic failures that headed reinforcement could have prevented had they been used. Destruction from recent earthquakes such as the Northridge in California, and the Hyogo-Ken Nanbu near Kobe, have illustrated the need for better technology for the design of seismic resistant reinforced concrete structures. Several types of failures associated with traditional reinforced concrete occurred during these earthquakes including:

- (i) Opening of spiral hoops,
- (ii) Opening of tails on shear ties and stirrups,
- (iii) Buckling of longitudinal steel bars
- (iv) Brittle failure of the concrete in compression.

The use of headed reinforcement offers solutions to such problems occurring with the use of traditional reinforcement (see Ref. 4).

In order to properly design and detail with headed reinforcement for seismically resistant bridge structures, a designer must first understand several different issues including: