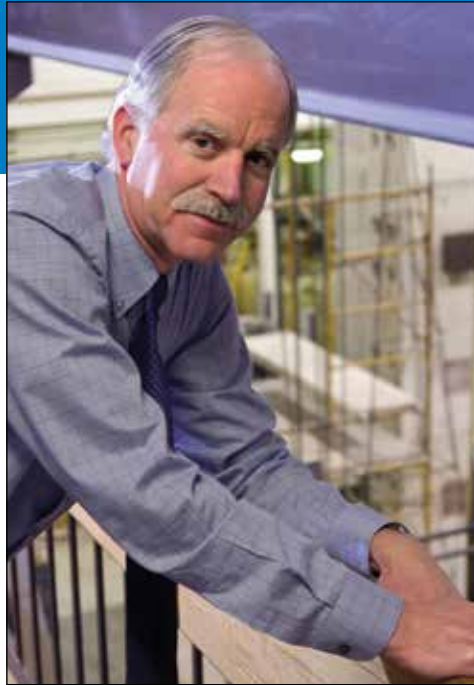


An ACI Technical Publication

SYMPOSIUM VOLUME



James K. Wight: A Tribute from his
Students and Colleagues

SP-311

Editors:

Gustavo J. Parra-Montesinos and Mary Beth D. Hueste



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Preface

Professor James (Jim) K. Wight has been one of the most remarkable researchers and educators in the field of reinforced concrete structures in the past several decades. Jim's engineering career started at Michigan State University, where he obtained his BS and MS in 1969 and 1970, respectively. After completing his MS studies, he went on to the University of Illinois at Urbana-Champaign to pursue doctoral studies under the supervision of Professor Mete A. Sozen, obtaining his PhD in 1973.

It was while a student at the University of Illinois that Jim Wight made his first major contributions to the field of behavior and design of reinforced concrete structures, particularly under earthquake excitations. He was likely the first to study the phenomenon of shear strength decay in reinforced concrete columns during large shear reversals. He also identified and explained the "disappearance" of the yield plateau in longitudinal reinforcing bars of flexural members subjected to moment gradient. Referring to this, Mete Sozen later said that had Jim been in the field of Physics, he would have won the Nobel Prize.

In 1973, Jim Wight joined the faculty at the University of Michigan. In a career that has spanned over 40 years as a Professor of Structural Engineering, Jim has exemplified excellence in teaching, research, and professional service. Jim has made enormous contributions to the field of behavior and design of reinforced concrete members, including beam-column and slab-column connections, structural walls, and deep beams. Much of his research has led to key advances in the safety and performance of reinforced concrete building structures during seismic events. Further, he has advised over 30 PhD students, several of whom are currently faculty members at major research universities. Jim has also contributed to the education of thousands of structural engineers as co-author (with Professor James MacGregor) of the widely used textbook *Reinforced Concrete – Mechanics & Design*. He has made significant contributions to the development of design guidelines and codes for reinforced concrete structures as Chair of ACI-ASCE Committee 352 in the early 1980s and of ACI Committee 318 during the 2002-2008 Code cycle. His dedication and involvement in the American Concrete Institute includes the distinction of serving as President in 2012-2013.

It was therefore with great joy that a group of researchers and practicing engineers who, over the years, had the opportunity to interact closely with Jim, decided to honor his illustrious career with a series of technical sessions and this Special Publication. Fifteen presentations, distributed in three sessions named "James K. Wight: A Tribute from his Students and Colleagues," were given at the 2014 ACI Fall Convention in Washington, DC. All speakers consisted of students of Jim's; colleagues in ACI technical committees; and his doctoral advisor, Professor Mete A. Sozen. The sessions were well attended by former students, academicians, researchers, and practitioners. A room-packed reception and a dinner were also offered in honor of Jim Wight. This Special Publication contains 12 papers related to the presentations made during the three technical sessions in Washington, DC. Also, Professor James O. Jirsa contributed with his personal perspective of Jim Wight's contributions to the design of beam-column joints.

This Special Publication is but one small token of the appreciation and gratitude that all those involved have for Jim Wight. He has been a mentor, role model, and a source of inspiration to many, as well as an example of honesty, integrity, dedication, and unselfishness. Professor James K. Wight is, without a doubt, a true educator in the broadest sense of the word. We all feel very grateful to have had the opportunity to honor such an outstanding individual.

Gustavo J. Parra-Montesinos
Mary Beth D. Hueste

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Jim Wight and Joints

A Brief Overview of the Development of Design Guidelines for Joints in Reinforced Concrete Structures and Dr. James K. Wight's Contributions to ACI 352

By

James O. Jirsa

Department of Civil, Architectural, and Environmental Engineering
The University of Texas at Austin

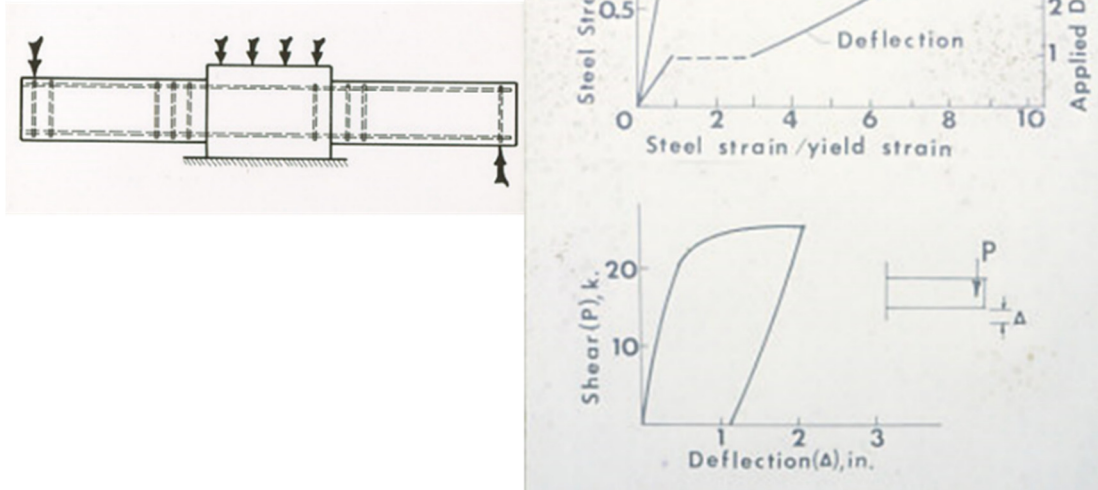
I am pleased to have been asked to participate in this Symposium honoring Jim Wight and recognizing his many contributions to the field of reinforced concrete structures through his research, teaching, and service to ACI and the profession. He has served the Institute in so many ways—as a member of a variety of technical committees, as Chair of a number of committees, notably ACI 318, *Building Code Requirements for Structural Concrete*, and as President. His textbook *Reinforced Concrete: Mechanics and Design* is widely used in both graduate and undergraduate reinforced concrete classes. Above all, he has been a wonderful friend and colleague. However, my comments here will be directed to his contributions related to the design of joints and connections in reinforced concrete structures.

To recall some history of the Joint ACI-ASCE Committee on Joints and Connections in Monolithic Structures (ACI 352). The committee was organized in 1966 and the First meeting was held in Oct. 1967 at an ACI Convention in Des Moines, Iowa. The committee was formed to address a gap in the design and construction of monolithic concrete structures—the joints between elements. There was no guidance for designers in any codes in use at that time. The mission of the committee was to “*Study the detailed design of joints and connections in monolithic concrete members and develop practical aids for the designer and the constructor.*”

A key question was posed by Raymond C. Reese (a long time member of ACI who served on ACI 318 and as President) “*Is it not short-sighted, and possibly dangerous, to spend so much time in making involved frame analyses and pursuing sophisticated methods which are directed almost entirely to the main members, and leave the joints and connections to detailers?*” The committee issued its first report in 1976. The document was based on the concept that failure should not occur in the connections. It was also understood that joint design requirements should meet the demands of loads and deformations produced by ordinary loads as well as those imposed where energy absorption is needed as in seismic zones.

At the time the committee was developing the report, the PhD research of Jim Wight at the University of Illinois figured prominently in the discussions. His work on strains in hinging regions provided important data on which the report was based. The figure below illustrates the manner in which strains and deflections are related in a hinging zone where a beam frames into a column as he reported in a paper in 1973.

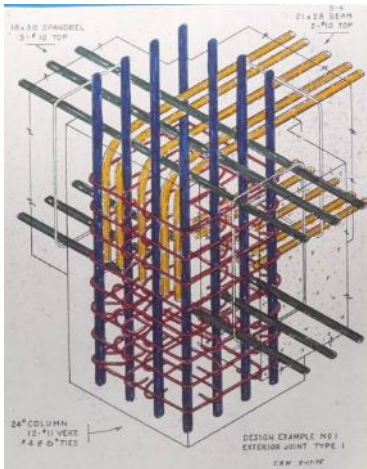
Measured strain and
applied deflection of a
cantilever beam
Wight and Sozen, 1973



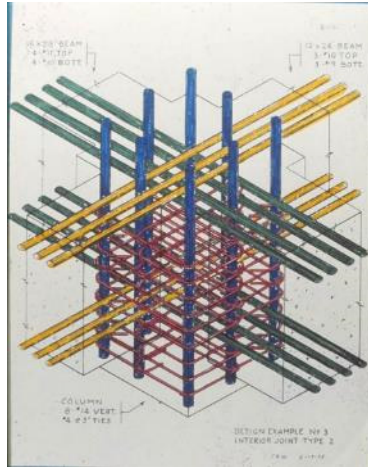
Key elements of the ACI 352 report:

- Joints must develop specified levels of steel strains (given as a multiple of yield strain) for each type. *Only two types were included—ordinary and seismic.*
- Stresses in the bars had to be consistent with the strains--strain hardening had to be considered. *A 1.25 stress multiplier was specified for joints in structures subjected to large deformations. Values from 1.2-2.0 were considered.*
- Required ductility was defined. *For cyclic loads, the joint was expected to withstand five times first yield for five cycles and maintain a strength at least 75% of the first cycle strength.*
- Geometry. *The joint included only the column within the depth of the beams framing into the column. Column width had to be equal to or greater than the beam width.*
- Column capacity had to be greater than the capacity of the beams framing into the joint
- Calculation of the shear strength and transverse reinforcement within the joint was specified. The manner in which this was to be calculated was contentious. *Many engineers were concerned that the addition of joint transverse reinforcement for joint integrity would raise costs and affect the competitiveness of concrete construction.*
- Anchorage of bars in the joint. *No large bars in small columns/joints.*

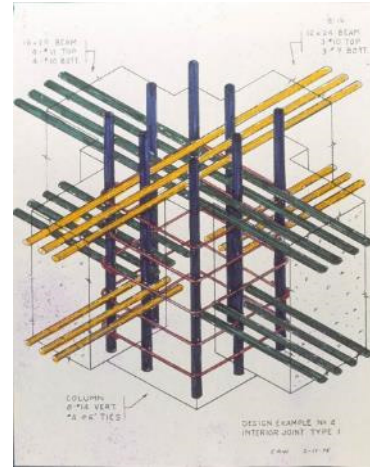
The report included some sketches of typical joints as shown below to help the user of the ACI 352 document visualize the reinforcement and the joint geometry.



Exterior Seismic Joint



Interior Seismic Joint



Interior Ordinary Joint

Perhaps the most important part of the ACI 352 report was a section identifying research needs. As a result, a number of research projects were conducted in the US, Japan, New Zealand, Canada, and China. Workshops were held to exchange information, coordinate test programs, and discuss design requirements. Different approaches for the shear strength of joints and transverse steel requirements were discussed at great length but there was never general agreement on the best approach. The effect of bond deterioration in the joint when subjected to cyclic loads was also extensively discussed and researched. During this period, Jim Wight served as a member and as chair of ACI 352. A list of his publications provides an indication of his significant influence on joint design and detailing.

- 1985 Wight and Ehsani “*Effect of Transverse Beams and Slab on Behavior of RC Beam-to-Column Connections*”
- 1985 Wight and Durrani “*Behavior of Interior Beam-Column Connections under Earthquake-Type Loading*”
- 1985 Wight and Ehsani “*Exterior RC Beam-to-Column Connections subjected to Earthquake-Type Loading*”
- 1987 Durrani and Wight “*Earthquake Resistance of RC Interior Connections Including a Floor Slab*”
- 1995 Raffaele and Wight “*Reinforced Concrete Eccentric Beam-Column Connections Subjected to Earthquake-Type Loading*”
- 1999 LaFave and Wight “*RC Exterior Wide Beam-Column-Slab Connections Subjected to Lateral Eq. Loading*”
- 2001 Quintero-Febres and Wight “*Experimental Study of RC Interior Wide Beam-Column Connections Subjected to Lateral Loading*”

Jim has been a vital contributor to the development of design guidelines for joints and connections during his career and during the life of ACI 352 since its formation in 1967. His imprints on the guidelines that ACI 352 has developed are indelible and profound and we can only hope that his involvement continues for many more years. Jim, congratulations on your outstanding achievements and my best wishes as you continue to serve our profession.

Innovative Approaches for Predicting RC Beam-Column Connection Joint Shear Resistance

James M. LaFave and Jaehong Kim

Synopsis: In this paper, an existing analytical approach for estimating joint shear behavior of reinforced concrete (RC) beam-column connections is critically examined and then modified in light of its limitations. The approach considers that joint shear strength is provided by an equivalent diagonal compression strut activated by force transfer to the joint through direct bearing from beam and column compression zones, and via bond between beam or column reinforcement and surrounding concrete. The model had first been developed for estimating joint shear behavior of composite RC column-to-steel beam connections, and then adjusted for application to certain RC beam-column connections (based on limited test results). This paper describes some shortcomings of that approach in light of a much broader experimental database of RC beam-column connection subassembly test data for developing joint shear resistance models. The research then uses probabilistic methods to derive a general relationship between joint shear deformation and the so-called principal strain ratio (including proper consideration of concrete compression softening and reasonable upper limits on joint shear deformation). This updated relationship can be used as a modification in the model for increased accuracy and broader applicability. The resulting modified model compares favorably to other approaches available in the literature for characterizing the joint shear behavior of RC beam-column connections.

Keywords: analytical models, Bayesian parameter estimation, beam-column connections, database, joint shear, reinforced concrete

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Jaehong Kim is Deputy General Manager, Bridges and Structures, for the Samsung C&T Corporation. He received his PhD in CEE from UIUC in 2007. He is also a licensed Professional Engineer in the state of California. His interests are in analytical modeling of beam-column connections and performance-based design for bridge structural systems.

INTRODUCTION

Proper management of the joint shear behavior in reinforced concrete (RC) beam-column connections, through appropriate design and construction, is required for maintaining reasonably ductile overall structural response when RC moment resisting frames are subjected to significant lateral earthquake loading. Numerous experimental and analytical studies have been conducted over the years to improve understanding of RC joint shear behavior for structural design and assessment, with several different approaches having been proposed to predict RC joint shear response (Jeon et al. 2014).

In this paper, some recent developments related to various innovative RC beam-column connection joint shear resistance modeling approaches are presented, based in part on using an extensive experimental database reflecting much of the previous testing work. After the database itself is briefly described, a full-range joint shear behavior prediction model is presented, which was developed employing a probabilistic methodology (Bayesian parameter estimation). An alternative approach to RC joint shear behavior modeling – previously proposed by others and based on basic joint shear resistance mechanisms – is then also discussed and critically evaluated. Finally, this second approach is modified in order to improve its reliability and enhance its application for modeling diverse types of RC beam-column connections.

So, two approaches for modeling joint shear behavior in RC beam-column connections are explained and evaluated in this paper, one by using a probabilistic methodology and another informed by basic mechanics considerations. Results of this research can be beneficial to improving overall understanding about the relative merits of these diverse approaches for predicting joint shear resistance, and for better characterizing RC joint shear behavior in general for purposes of structural analysis, design, or evaluation / assessment.

EXPERIMENTAL DATABASE

A consistent set of inclusion criteria were employed to construct an extensive experimental database for RC beam-column connection subassemblies (laboratory test specimens) from the literature. Those database inclusion criteria can be summarized as follows: (i) all cases were subjected to quasi-static reverse cyclic lateral loading, (ii) all cases experienced joint shear failure, either in conjunction with or without yielding of longitudinal reinforcement, (iii) all specimens were larger than one-third scale, (iv) only deformed bars were used for all longitudinal beam and column reinforcement, (v) all specimens had only conventional anchorage types (no headed bars or anchorage plates), and (vi) all cases had proper seismic hooks. Then, qualified experimental data were further classified according to in-plane geometry, out-of-plane geometry, and failure mode sequence. In the *total database* (Kim et al. 2009), 341 RC beam-column connection experimental cases were included, with no limit on the degree of joint confinement (i.e., some specimens even have no joint transverse reinforcement at all). This built upon a previously presented *modern database* (subset of 182 specimens) that only comprised cases maintaining proper joint confinement (Kim and LaFave 2008), which was in turn an expanded version of a *basic database* (subset of 136 specimens) that also excluded any cases having eccentricity between the beam(s) and column(s) or with out-of-plane members (Kim and LaFave 2007). Details about all of the specimens in the total database, including the specific inclusion criteria for each of the two subsets, can be found elsewhere (Kim and LaFave 2009).

In this research, key points displaying the most distinctive stiffness changes for overall (subassembly story shear vs. overall drift) and local (joint shear stress vs. joint shear strain) cyclic response have been identified for each test specimen in the database, and then envelope curves were constructed by connecting those key points. Figure 1 is an example of this for the overall and local behavior of an interior connection exhibiting joint shear failure (along with beam reinforcement yielding). The first key point (A) is related to initiation of diagonal cracking within the joint panel, the second key point (B) corresponds to yielding of longitudinal beam reinforcement or joint transverse reinforcement, and the third key point (C) is at peak response. A descending branch has also been considered, in order to describe the full range of RC joint shear stress vs. joint shear strain behavior. The descending branch key point (D) has its vertical coordinate simply assigned as being 90% of the maximum joint shear stress (this ordinate value is typically similar to the level of point B stress).

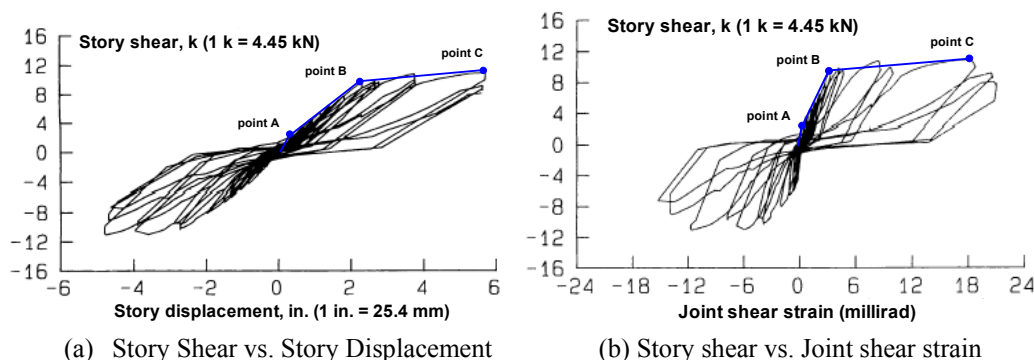


Figure 1 – Comparison of key points displaying distinct stiffness changes (test data per Leon 1990).

RC JOINT SHEAR BEHAVIOR MODEL USING PROBABILISTIC METHODOLOGY

Background on Bayesian parameter estimation methods

Probabilistic methods have recently been applied to reduce prediction error (and scatter) for models of the behavior of RC members, especially for shear capacity. Gardoni et al. (2002) suggested a probabilistic procedure to construct RC column shear capacity models, relying on an existing deterministic model as a starting point. Then, this approach was updated to even develop capacity models without relying on an existing model (Song et al. 2010). That is:

$$\ln[C(\mathbf{x}, \Theta)] = \sum_{i=1}^p \theta_i h_i(\mathbf{x}) + \sigma \varepsilon \quad (1)$$

where C is the experimental shear capacity, $\mathbf{x}$ is the vector of input parameters measured during testing, $\Theta = (\theta, \sigma)$ denotes the set of unknown model parameters that are introduced to fit the model to the test results, θ is the uncertain model parameter, ε is the normal random variable (with zero mean and unit variance), and σ is the unknown model parameter representing the magnitude of model error that remains after bias-correction. A Bayesian parameter estimation method can be employed to find the distribution of uncertain parameters that makes the models in Eq. (1) best fit the test results. Song et al. (2010) introduced a constant bias-correction term, as shown in Eq. (2).

$$\ln[C(\mathbf{x}, \Theta)] = \ln[c_d(\mathbf{x})] + \theta + \sigma \varepsilon \quad (2)$$

This equation can evaluate the overall bias and scatter of any particular deterministic model (c_d); a deterministic model is less biased when the posterior mean of θ is closer to zero, and it has less scatter when the posterior mean of σ is smaller.

Modeling of RC beam-column connection joint shear strength

A rational procedure to develop RC joint shear resistance models (at point C) using such a Bayesian parameter estimation method, in conjunction with the constructed experimental database, has been employed in this research as follows: (a) possible influence parameters were introduced to describe diverse conditions within joint panels of RC beam-column connections; (b) the Bayesian parameter estimation method was employed to find an unbiased joint shear strength model based on the experimental database; (c) at each stage, the least informative parameter was identified; (d) an unbiased model was again constructed after removing the least informative parameter; and (e) steps (c) and (d) were repeated until only the parameters that are most important to determining RC joint shear capacity remained.

A series of possible influence parameters have been introduced, after careful consideration from a review of the relevant literature and also by qualitative assessment (Kim and LaFave 2009). The following parameters were included: (1) concrete compressive strength (f'_c); (2) in-plane geometry ($JP = 1.0$ for interior, 0.75 for exterior, and 0.5 for knee joints); (3) beam-to-column width ratio (b_b/b_c) in the plane of loading; (4) beam height to column depth ratio (h_b/h_c) in the plane of loading; (5) beam reinforcement index (BI , defined as $(\rho_b \times f_{yb})/f'_c$, in which ρ_b is a beam's total reinforcement ratio and f_{yb} is the yield stress of beam reinforcement); (6) joint transverse reinforcement index (JI , defined as $(\rho_j \times f_{yj})/f'_c$, in which ρ_j is the joint transverse reinforcement ratio in the direction of loading and f_{yj} is the yield stress of joint transverse reinforcement); (7) A_{sh} ratio (provided-to-recommended amount of joint transverse reinforcement per *ACI 352R-02* (ACI 2002) design recommendations); (8) spacing ratio (provided-to-recommended spacing of joint transverse reinforcement per *ACI 352R-02*); (9) out-of-plane geometry ($TB = 1.0$ for zero or one transverse beam, and 1.2 for two transverse beams); and (10) joint eccentricity ($1 - e/b_c$, where e is the eccentricity between the centerlines of the beam and column). Normalized column axial stress and provided-to-required length of beam reinforcement (as a function of bar diameter) in the joint were not included in developing the prediction models because the Bayesian parameter estimation method indicated that these parameters are not particularly informative, and including them can reduce the efficiency of using the constructed database (due to certain complexities involved in consistently defining these parameters for use across the entire database).

Among the possible influence parameters, those representing out-of-plane geometry, joint eccentricity, joint confinement by transverse reinforcement, joint "confinement" provided by longitudinal beam reinforcement, in-plane geometry, and concrete compressive strength have been found to be more informative than the others. It is interesting to note that the beam-to-column width ratio, which is a factor in the current *ACI 352R-02* and *ACI 318-11* (ACI 2011) approaches for determining RC joint shear strength, was not found here to be a particularly informative parameter.

Eq. (3) is the developed joint shear resistance model for stress (v_j) that only includes the most informative parameters, and Eq. (4) is a simple and unified RC joint shear strength model based on Eq. (3), where 1 MPa = 0.145 ksi. Concrete compressive strength is the most informative parameter, with its optimized contribution at around 0.75 for the power term, while the parameters become somewhat less informative as one moves from right to left in Eq. (3).

$$v_j(\text{MPa}) = 1.21(TB)^{0.981} \left(1 - \frac{e}{b_c}\right)^{0.679} (JI)^{0.136} (BI)^{0.301} (JP)^{1.33} (f'_c)^{0.764} \quad (3)$$

$$v_j(\text{MPa}) = \alpha_i \beta_i \eta_i \lambda_i (JI)^{0.15} (BI)^{0.30} (f'_c)^{0.75} \quad (4)$$

In Eq. (4), α_i is a parameter for in-plane geometry (1.0 for interior connections, 0.7 for exterior connections, and 0.4 for knee connections); β_i is a parameter for out-of-plane geometry (1.0 for subassemblies with 0 or 1 transverse beams, and 1.18 for subassemblies with 2 transverse beams); $\eta_i = (1 - e/b_c)^{0.67}$ describes joint eccentricity; and $\lambda_i = 1.31$, which simply makes the average ratio of Eq. (4) to Eq. (3) equal 1.0. The numerical values of the in-plane geometry parameter for exterior and knee connections tend to indicate that their current treatment in *ACI 352R-02* and *ACI 318-11* with respect to RC joint shear strength may be a bit too optimistic, at least in comparison to interior connections. And, in a similar vein, the expression given above for the joint eccentricity parameter implies that the current treatment of eccentric connections by *ACI 318-11* is fairly conservative. A more detailed assessment of various ACI design approaches for RC joint shear strength, using similar probabilistic approaches in conjunction with the modern database, may be found elsewhere (Kim and LaFave 2008).

Figure 2 is a plot of experimental joint shear resistance (stress) vs. the simple and unified RC joint shear strength model (Eq. (4)). The experimental joint shear stress is defined over an effective joint shear area taken as the product of the effective joint width (average of beam and column widths) and column depth. The total database (except specimens with no joint transverse reinforcement) was used for the comparison in Fig. 2. Within the total database, 18 cases had no joint transverse reinforcement – for these cases, an experimental joint shear stress to Eq. (4) ratio can be computed by using a trial value of Jl . The average of experimental joint shear stress to Eq. (4) is 1.0 when the trial Jl is equal to 0.0128, which means that using a low, but non-zero, “virtual” Jl of 0.0128 enables Eq. (4) to predict joint shear strength even for cases with no joint transverse reinforcement. When Eq. (4) is used as a deterministic model in Eq. (2), the means of θ and σ are -0.011 and 0.153 , respectively; the modest scatter is much less than for other commonly used models (Kim and LaFave 2009). The simple and unified model (Eq. (4)) is an unbiased model, which means that roughly half of the experimental cases are below their respective Eq. (4) values. For safe application to joint shear strength design, Eq. (4) could for instance be multiplied by 0.80-0.85 in order to only have about 10% or fewer of the cases exhibiting lower experimental joint shear stress values than such an adjusted joint shear strength model.

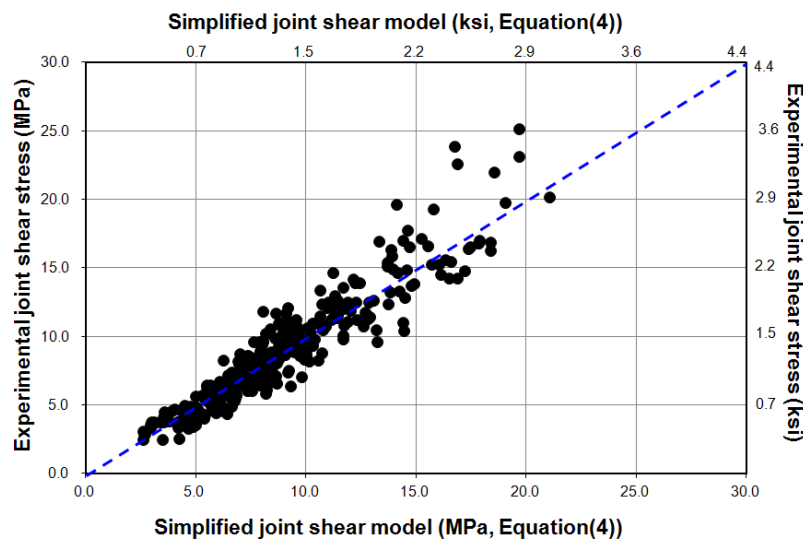


Figure 2 – Experimental joint shear stress vs. simplified joint shear strength model (Eq. (4)).

Modeling of joint shear deformation at Point C

The same established probabilistic procedure has also been employed to construct RC joint shear deformation capacity models (using the roughly 40% of cases in the database that provided joint shear strain information). As before, possible influence parameters were first carefully determined (Kim and LaFave 2009). To improve this model’s accuracy, the ratio of the joint shear strength model (Eqs. (3) or (4)) to concrete compressive strength was even included as a possible influence parameter. Kim and LaFave (2007) determined that minimum proper joint confinement is maintained when the A_{sh} ratio is equal to or above 0.70. For the data group maintaining minimum proper joint confinement, certain values for a parameter describing in-plane geometry (JPR) were determined to result in the strongest linear relation between normalized joint shear strain and normalized joint shear stress divided by JPR . Those JPR values are 1.0, 0.59, and 0.32 for interior, exterior, and knee joints, respectively. For the total database, then, a new in-plane geometry parameter ($JPRU$) was determined by dividing JPR by 1.2 to reflect insufficient joint confinement – when the A_{sh} ratio is equal or above 0.7, $JPRU$ is simply 1.0, 0.59, and 0.32, whereas when the A_{sh} ratio is below 0.7, $JPRU$ is 1.0/1.2, 0.59/1.2, and 0.32/1.2 for interior, exterior, and knee joints, respectively. Except for these two parameters (joint shear strength model over concrete compressive strength and $JPRU$), the other included parameters are the same as for development of the RC joint shear strength model.

RC joint shear capacity (deformation, γ , as well as stress) at the peak point (C) is mainly dependent on out-of-plane geometry, in-plane geometry, joint eccentricity, confinement by joint transverse reinforcement, confinement by longitudinal beam reinforcement, and concrete compressive strength. Eq. (5) is the developed joint shear deformation

model only including the most informative parameters, and Eq. (6) is a simple and unified RC joint shear deformation model based on Eq. (5).

$$\gamma(\text{Rad}) = 0.00565 BI \left(1 - \frac{e}{b_c}\right)^{-0.628} (JI)^{0.0982} (TB)^{1.85} \left(\frac{v_j(\text{Eq.}(3))}{f'_c}\right)^{-1.73} (JPRU)^{2.11} \quad (5)$$

$$\gamma(\text{Rad}) = \alpha_{\gamma t} \beta_{\gamma t} \eta_{\gamma t} \lambda_{\gamma t} BI (JI)^{0.10} \left(\frac{v_j(\text{Eq.}(4))}{f'_c}\right)^{-1.75} \quad (6)$$

In Eq. (6), v_j (Eq.(4)) is the developed simple and unified RC joint shear strength model; $\alpha_{\gamma t}$ ($= (JPRU)^{2.10}$) is a parameter for in-plane geometry; $\beta_{\gamma t}$ is a parameter for out-of-plane geometry (1.0 for subassemblies with zero or one transverse beam, and 1.4 for subassemblies with two transverse beams); $\eta_{\gamma t}$ ($= (1 - e/b_c)^{-0.60}$) describes joint eccentricity (1.0 for no eccentricity); and $\lambda_{\gamma t}$ ($= 0.00549$) is a factor introduced to make the average ratio of the predictions by the models in Eqs. (6) and (5) equal to 1.0. Figure 3 is a plot of experimental joint shear strain vs. the simple and unified RC joint shear deformation model (Eq. (6)). When Eq. (6) is used as a deterministic model in Eq. (2), the means of θ and σ are -0.117 and 0.410 , respectively. Model uncertainty for joint shear deformation is distinctively greater compared to that for joint shear strength. This increased scatter is in part due to the fact that experimental joint shear deformation data have not always been collected in exactly the same way across different testing programs, and is also the result of having just one equation to address all cases of joint shear failure, including those with some beam longitudinal reinforcement yielding that can indirectly contribute to measured shear deformations at the joint. In any event, even this level of scatter in joint shear strain is less than what would be expected from other commonly used models (Kim and LaFave 2009).

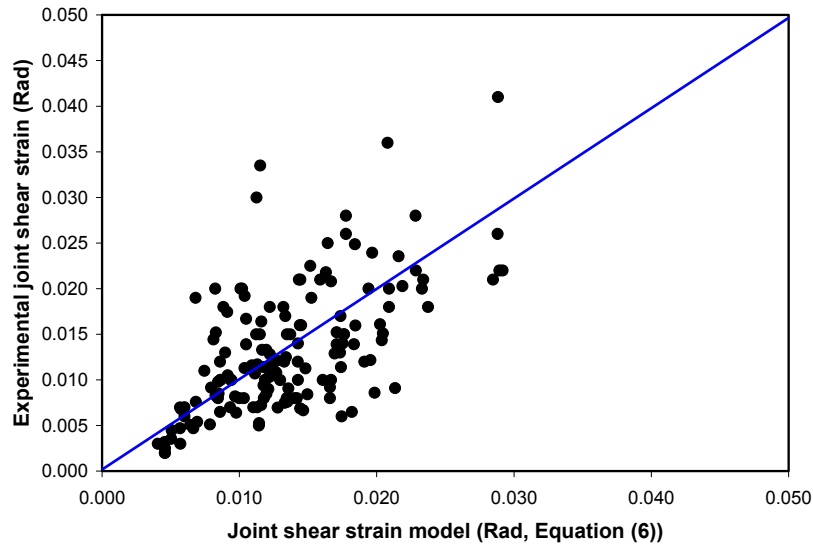


Figure 3 – Experimental joint shear strain vs. simplified joint shear strain model (Eq. (6)).

Overall modeling of RC beam-column connection joint shear resistance

Kim and LaFave (2009) have provided a detailed explanation about further RC joint shear stress and strain models that employ the Bayesian parameter estimation method at the other key points (A, B, and D). Those developed models indicate that essentially the same key influence parameters (most informative explanatory terms) on joint shear stress and strain are maintained across all the key points. Therefore, for these other key points, joint shear stress and strain models can alternatively be suggested as simply the product of constant factors times the simple and unified resistance models (Eqs. (4) or (6)). For joint shear stress, these constant reduction factors are 0.44, 0.89, and 0.90 for points A, B, and D, respectively, and for joint shear strain the factors are 0.02, 0.36, and 2.02 for those same three points (Kim and LaFave 2012).

Such an approach leads to an overall joint shear stress vs. joint shear strain resistance model that can be applicable across the full range of explanatory terms found in the total database. Kim and LaFave (2009) have compared experimental joint shear behavior vs. the full range of this relatively simple joint shear behavior model. The proposed model reasonably matches with experimental joint shear behavior, with little increase in model uncertainty compared to using separately developed expressions for each point, although some modest local biases do exist (Kim and LaFave 2012). For example, Fig. 4 compares experimental and suggested joint shear stress vs. strain for the extreme cases of JI , BI , and f'_c (see Kim and LaFave (2009) for more details). Even for these cases, the suggested model shows quite good agreement for the envelope curve of joint shear stress vs. strain, when compared to the experimental results.

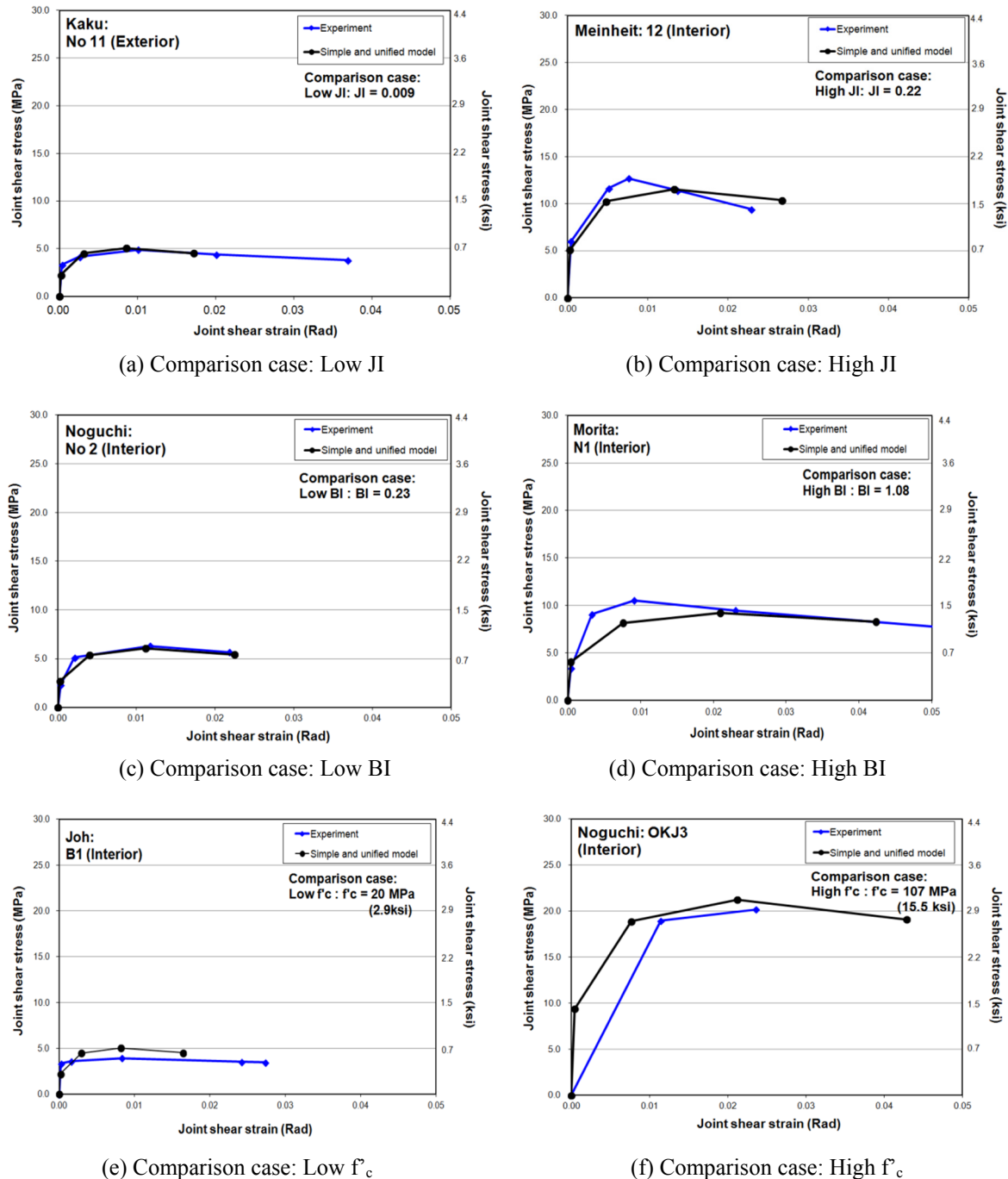


Figure 4 – Comparison of full range of experimental and suggested (model) joint shear stress vs. strain.

The *ASCE/SEI 41* Supplement 1 joint shear behavior model (Elwood et al. 2007) has also been examined, using the constructed database in conjunction with probabilistic methods similar to those described above. It is notable that there are still no experimental cases for certain of the *ASCE/SEI 41* Supplement 1 RC joint subcategories, so additional focused validation testing may even be needed. Selected adjustments to some of those joint shear strength factors have been developed anyway, which can result in more consistent conservatism across all joint categories, and a simple plastic joint shear strain model has also been suggested (Kim and LaFave 2012).

JOINT SHEAR BEHAVIOR MODEL USING BASIC JOINT SHEAR RESISTANCE MECHANISMS

Description of the Parra-Montesinos and Wight model

Parra-Montesinos (2000) suggested another innovative analytical approach, for composite RC column-to-steel beam connections, in order to estimate their joint shear behavior. In that approach, principal joint shear strain was defined in a joint panel by relating principal tensile and compressive strains. Parra-Montesinos and Wight (2002) then extended the research by adjusting this analytical approach for estimating joint shear behavior of RC beam-column connections. The Parra-Montesinos and Wight model is presented and critically analyzed here in this section, in conjunction with the experimental database. The model is then refined in a subsequent section (in part using probabilistic approaches) for broader applicability and improved reliability.

Concrete struts and/or trusses are the generally accepted RC joint shear resistance mechanisms – a strut mechanism is formed from resisting the transferred force by concrete compression zones of adjacent beam(s) and column(s), whereas a truss mechanism is formed from resisting the transferred force via bond between reinforcement and surrounding concrete in the joint panel. Parra-Montesinos and Wight (2002) considered that RC joint shear strength is provided by an equivalent diagonal compression strut activated by force transfer to the joint by direct bearing from the beam and column compression zones and by bond between the beam and column reinforcement and the surrounding concrete. Figure 5 shows the equivalent concrete strut that they suggested.

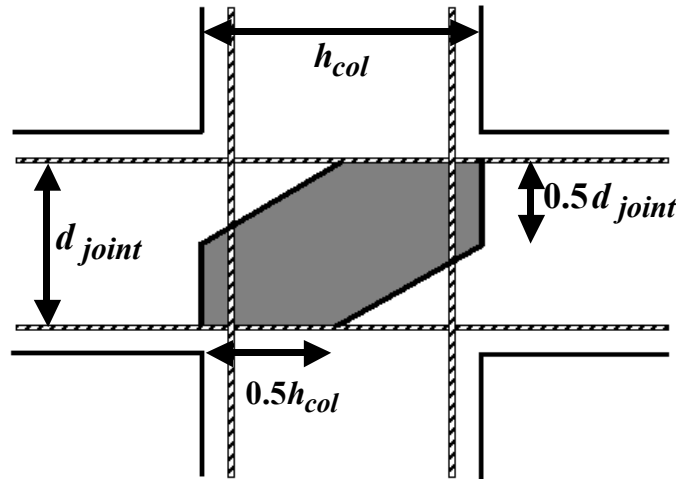


Figure 5 – Equivalent strut per Parra-Montesinos and Wight (2002).

The shape of this equivalent strut can be determined by the geometry and reinforcement array of the beams and columns. The width of the equivalent strut (b_{strut}) in the out-of-plane direction is simply the average of the widths of the beams (b_b) and columns (b_c) connecting at the joint. The depth of the equivalent strut (d_{strut}) in the in-plane direction is taken as:

$$d_{strut} = \frac{d_{joint} h_c}{\sqrt{d_{joint}^2 + h_c^2}} \quad (\text{in-plane direction}) \quad (7)$$

where d_{joint} is the center-to-center distance ($d - d'$) between the top and bottom layers of beam reinforcement, and h_c is identical to h_{col} in Fig. 5 (per Parra-Montesinos and Wight). In the Parra-Montesinos and Wight model, the principal compressive direction is assumed to be aligned with the direction of the diagonal equivalent strut (θ_{strut}), given as:

$$\theta_{strut} = \tan^{-1} \left(\frac{d_{joint}}{h_c} \right) \quad (8)$$

Parra-Montesinos and Wight (2002) further suggested a possible relationship between RC joint shear strain and principal strain ratio, based on their nine collected experimental test results; that is:

$$K_{tc} = 2 + k_s \gamma \quad (9)$$

where K_{tc} is the ratio of principal tensile strain to principal compressive strain ($-\epsilon_t/\epsilon_c$); γ is the joint shear strain; and k_s is a suggested slope for the K_{tc} vs. γ relationship ($= 500 + 2000 e/b_c$). Parra-Montesinos and Wight commented that Eq. (9) is valid only up to about 1% joint shear deformation (for subassemblies maintaining proper confinement within the joint panel) because they conservatively derived this relation for γ up to 0.01 based only on those nine test results. They also recommended that more data may need to be evaluated in order to derive generally applicable relationship(s) between the principal strain ratio and joint shear deformation.

Plane strain within a joint panel can be expressed with the following three equations, using strain coordinate transformation; that is:

$$\epsilon_c = \frac{\epsilon_x + \epsilon_y}{2} + \frac{\epsilon_x - \epsilon_y}{2} \cos(2\theta_p) + \frac{\gamma}{2} \sin(2\theta_p) \quad (10)$$

$$\epsilon_t = -K_{tc} \epsilon_c = \frac{\epsilon_x + \epsilon_y}{2} + \frac{\epsilon_x - \epsilon_y}{2} \cos[2(\theta_p + \frac{\pi}{2})] + \frac{\gamma}{2} \sin[2(\theta_p + \frac{\pi}{2})] \quad (11)$$

$$\gamma = \tan(2\theta_p)(\epsilon_x - \epsilon_y) \quad (12)$$

where ϵ_x , ϵ_y , ϵ_c , and ϵ_t are the strain for the X (horizontal), Y (vertical), principal compressive, and principal tensile directions, respectively. When joint shear strain (γ) is given and the principal strain direction (θ_p) is known (i.e., $\theta_p = \theta_{strut}$), four plane strains (ϵ_x , ϵ_y , ϵ_c , and ϵ_t) can be determined by employing Eqs. (9), (10), (11), and (12).

Parra-Montesinos and Wight (2002) then employed the concrete stress vs. strain model of Sheikh and Uzumeri (1982) in order to find the principal compressive stress at the calculated principal compressive strain. This concrete stress-strain model accounts for an increase in compressive strength of confined concrete, and it has four segments – ascending (parabolic), peak plateau (flat), descending (linear), and lower plateau (flat). The expressions for those segments are a function of the volumetric ratio, spacing, and yield stress of transverse tie and column longitudinal reinforcement in the joint region. In calculating the principal compressive stress of the equivalent strut, Parra-Montesinos and Wight applied the compression softening parameter (β), originally suggested by Vecchio and Collins (1993), as follows:

$$\beta = \frac{1}{0.85 + 0.27 K_{tc}} \quad (13)$$

However, they set the minimum value of β to be 0.6, which is different from the suggestion of Vecchio and Collins. The horizontal component of the diagonal compressive force could then finally be taken (by simply multiplying by $\cos(\theta_p)$) as the horizontal joint shear force for any given joint shear strain.

Assessment of the Parra-Montesinos and Wight model

Kim and LaFave (2009) have provided analytical joint shear stress vs. joint shear strain values, computed by using the proposed model of Parra-Montesinos and Wight (2002). The values were determined – up to 1% joint shear deformation – for interior and exterior confined RC connections from the experimental database having no out-of-plane members (due to limitations of the method). Figure 6 is a plot of experimental joint shear stress vs. joint shear stress defined by the Parra-Montesinos and Wight model, which is also compared to the experimental values vs. those suggested by the simple and unified model (Eq. (4)).

Joint shear strength (as defined by Parra-Montesinos and Wight) appears to generally be determined too conservatively ($\theta = 0.190$ and $\sigma = 0.359$), in part due to Eq. (7) and the upper limit (1%) on joint shear deformation (which had been conservatively set in order to control damage and drift). On the other hand, the specific connections used by Parra-Montesinos and Wight to calibrate and validate the model had their joint shear strengths predicted remarkably well by that model. By perhaps redefining Eq. (7), using the total database presented above in order to consider even more diverse types of RC beam-column connections, some of the current limitations of the Parra-Montesinos and Wight model can be reduced – such a modification to the Parra-Montesinos and Wight model is discussed in the next section.

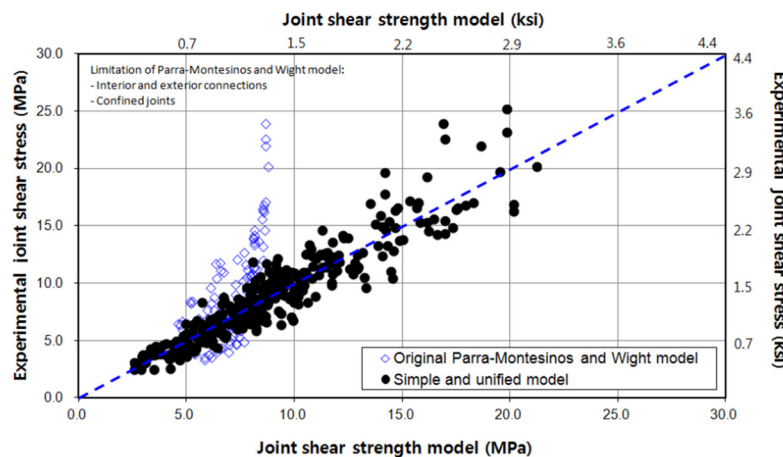


Figure 6 – Comparison of simplified and Parra-Montesinos and Wight (2002) RC joint shear stress models.

Figure 7 represents a plot of experimental joint shear strain vs. joint shear strain by the Parra-Montesinos and Wight (2002) model, which is also compared to the suggested simple and unified model (Eq. (6)). The overall means of θ and σ are computed as 0.442 and 0.410, respectively, for the Parra-Montesinos and Wight model. The conservative constant upper limit (1%) for joint shear deformation, which had been set in order to control damage and drift, is therefore not reasonable as a general approach to use when trying to determine joint shear deformation at maximum response.

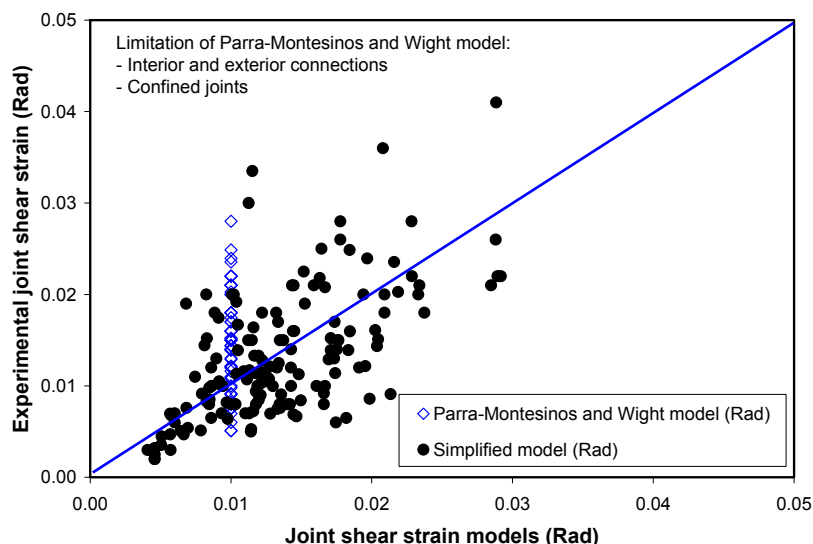


Figure 7 – Comparison of simplified and Parra-Montesinos and Wight (2002) RC joint shear strain models.

MODIFIED JOINT SHEAR MODEL USING BASIC JOINT SHEAR RESISTANCE MECHANISMS

Description and development of a *modified* Parra-Montesinos and Wight model

The Parra-Montesinos and Wight (2002) model for RC joint shear resistance has been modified in this current research, in conjunction with the constructed experimental database of RC beam-column connections. Their basic assumptions have first been reviewed and critically analyzed. Then, certain identified deficiencies of the model are modified using the database, in conjunction with probabilistic methods. Finally, the performance of this modified Parra-Montesinos and Wight model is evaluated.

Due to the fact that there is no clear method for quantifying bond capacity after several cycles of loading in this sort of macro-modeling (one element as a joint panel), Parra-Montesinos and Wight (2002) assumed a fixed equivalent strut direction and shape. In the current research, the fixed direction and shape of the equivalent strut is also employed. Even so, after fixing the direction and shape of the equivalent strut, the effect of bond condition may still be indirectly considered in the derived relation between the principal strain ratio and the joint shear strain.

Parra-Montesinos and Wight (2002) applied the concrete stress vs. strain model proposed by Sheikh and Uzumeri (1982) for RC columns. A joint panel is considered as part of an RC column in the current model analysis and design philosophy, so using this concrete constitutive model seems reasonable. Differing from the suggestion of Vecchio and Collins (1993), however, Parra-Montesinos and Wight suggested a lower-bound value for the concrete compression softening factor ($= 0.6$). On the other hand, the original method proposed by Vecchio and Collins is applied in this current research.

Parra-Montesinos and Wight (2002) used only nine RC beam-column connection subassemblies to inform their model. Therefore, their simple derived relation between the principal strain ratio and joint shear deformation may not correctly describe the full effects of diverse conditions on RC beam-column connection behavior. In the current research, this relationship is modified, in conjunction with the constructed database, to better predict joint shear behavior for diverse types of RC beam-column connections. From the constructed database, the principal strain ratio can in all cases actually be computed for each test specimen, at any given experimental joint shear strain, by employing the following ten-step procedure.

Step 1: Determine the “equivalent strut shape” and the principal compression direction (θ_p).

Step 2: Select a joint shear strain (γ) from the laboratory test (corresponding to an experimental joint shear).

Step 3: Choose a trial principal strain ratio ($K_{tc} = -\varepsilon_t/\varepsilon_c$)

Step 4: Compute the plane strain (ε_x , ε_y , ε_c , and ε_t) corresponding to this θ_p , γ , and K_{tc} , per Eqs. (10), (11), and (12).

Step 5: Compute the compression softening factor (β), using Eq. (13).

Step 6: Compute the concrete principal compressive stress (f_c) at the calculated ε_c . The principal compressive stress is computed employing the concrete stress vs. strain model proposed by Sheikh and Uzumeri (1982), in conjunction with the compression softening factor determined in Step 5.

Step 7: Compute the total diagonal compression of the equivalent strut ($=f_c*b_{strut}*d_{strut}$).

Step 8: Compute the horizontal component of the total diagonal compression (computed joint shear).

Step 9: Compare the experimental and the computed joint shear values. Return to Step 3, and choose a new K_{tc} , until the computed joint shear is equal to the experimental joint shear.

Step 10: Go to Step 2 for a new joint shear strain, in order to repeat this process across a range of γ values.

The actual principal strain ratio (K_{tc}) has specifically been computed for each test specimen at the key points of joint shear behavior (A, B, C, and D). In general, the computed principal strain ratio is proportional to joint shear deformation, which is shown for example in Fig. 8 for an interior connection with no-out-of plane members and no joint eccentricity (Noguchi and Kashiwazaki 1992).

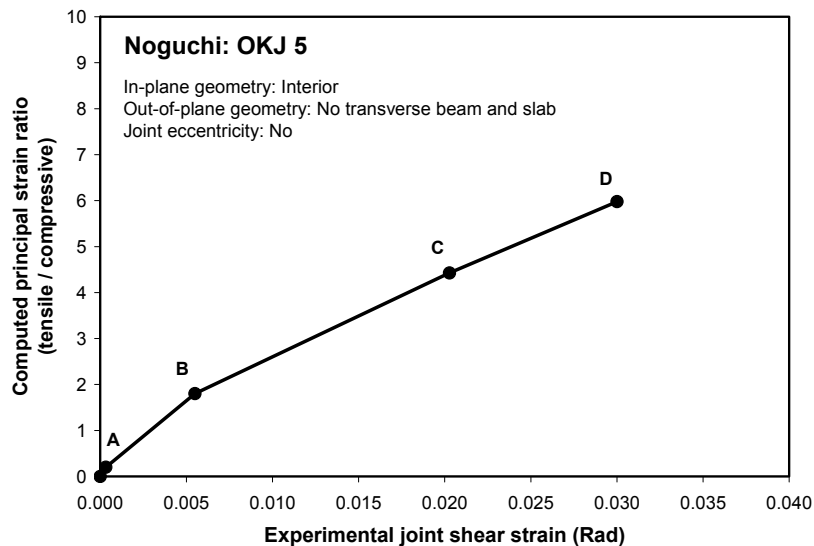


Figure 8 – Computed principal strain ratios at given joint shear strains for an example test specimen.

This plot of computed principal strain ratio vs. experimental joint shear strain can be simplified into the form of Fig. 9 because the region between points B and C generally comprises the vast majority (about 64%, on average) of the joint shear deformation up to maximum resistance (point C), as indicated previously. The slope between points B and C is then simply extended back to zero joint shear strain. In Fig. 9, the principal strain ratio at zero joint shear strain is referred to as the initial principal strain ratio ($K_{tc,i}$), while the slope is referred to as the principal strain ratio slope ($K_{tc,s}$).

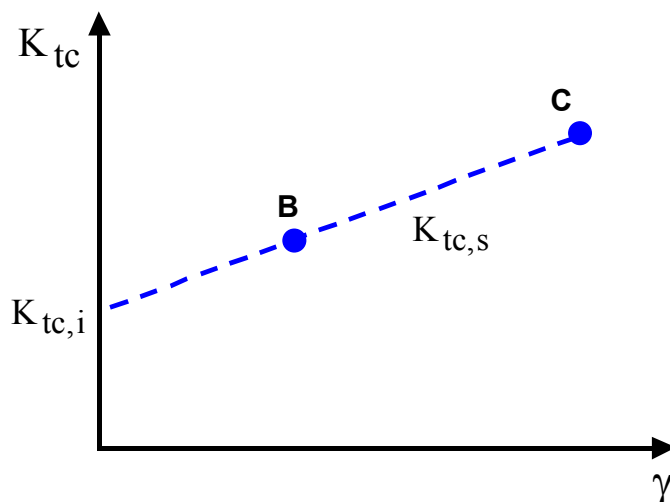


Figure 9 – Simplified representation of principal strain ratio vs. joint shear strain.

Using this simplification, both the initial principal strain ratio and the principal strain ratio slope have been found for every experimental subassembly in the database that provides information about joint shear deformation behavior. The key relationship between the principal strain ratio and joint shear strain can then be expressed as:

$$K_{tc} = K_{tc,i} + K_{tc,s} \times \gamma \quad (14)$$

Equations for the initial principal strain ratio ($K_{tc,i}$) and the principal strain ratio slope ($K_{tc,s}$) have been developed using the experimental database in conjunction with the Bayesian parameter estimation method. In constructing unbiased expressions for $K_{tc,i}$ and $K_{tc,s}$, nine explanatory terms (A_{sh} ratio, spacing ratio, beam-to-column width ratio, beam height to column depth ratio, joint transverse reinforcement index, beam reinforcement index, joint eccentricity, out-of-plane geometry, and in-plane geometry) were considered. After removing four insignificant influence parameters, $K_{tc,i}$ and $K_{tc,s}$ can then be expressed as Eqs. (15) and (16), respectively. The means of σ are 0.700 for Eq. (15) and 0.507 for Eq. (16).

$$K_{tc,i} = 2.96 (JI)^{0.062} (BI)^{0.362} (TB)^{-4.14} \left(1 - \frac{e}{b_c}\right)^{-2.38} (JP)^{-0.711} \quad (15)$$

$$K_{tc,s} = 41.42 \left(1 - \frac{e}{b_c}\right)^{0.0846} (TB)^{-0.408} (JI)^{-0.235} (BI)^{-0.678} (JP)^{-1.88} \quad (16)$$

The initial principal strain ratio can be further simplified into Eq. (17); that is:

$$K_{tc,i} = \alpha_i \beta_i \eta_i \lambda_i (JI)^{0.062} (BI)^{0.362} \quad (17)$$

In Eq. (17), α_i is a parameter describing in-plane geometry (1.0 for interior, 1.23 for exterior, and 1.64 for knee joints); β_i is a parameter describing out-of-plane geometry (1.0 for 0 or 1 transverse beam, and 0.47 for 2 transverse beams); $\eta_i (= 1 - e/b_c)^{-2.38}$ is a parameter describing joint eccentricity; and λ_i is 2.958 in order to have 1.0 as the average of Eqs. (17) and (15). When employing Eq. (17), the overall means of θ and σ are -0.355 and 0.699 , respectively.

The principal strain ratio slope can also be further simplified, as follows:

$$K_{tc,s} = \alpha_s \beta_s \eta_s \lambda_s (JI)^{-0.235} (BI)^{-0.678} \quad (18)$$

In Eq. (18), α_s is a parameter describing in-plane geometry (1.0 for interior, 1.72 for exterior, and 3.68 for knee joints); β_s is a parameter describing out-of-plane geometry (1.0 for 0 or 1 transverse beam, and 0.93 for 2 transverse beams); η_s ($= (1 - e/b_c)^{0.0846}$) is a parameter describing joint eccentricity; and λ_s is 42.4 in order to have 1.0 as the average of Eqs. (18) and (16). The overall means of θ and σ for Eq. (18) are -0.120 and 0.701 , respectively.

RC beam-column connection joint shear stress vs. joint shear strain can now be “predicted” by employing this modified Parra-Montesinos and Wight model, utilizing the same series of steps as listed before – with one notable exception that makes the process no longer iterative. In Step 3, rather than just choosing a trial principal strain ratio, an appropriate principal strain ratio at a given joint shear strain is instead simply determined using Eq. 14, with the initial principal strain ratio ($K_{tc,i}$) calculated per Eq. (17) and the principal strain ratio slope ($K_{tc,s}$) computed per Eq. (18).

Assessment of the modified Parra-Montesinos and Wight model

For each subassembly in the database that has information about joint shear load and deformation behavior, the joint shear stress vs. joint shear strain behavior can be plotted following this modified Parra-Montesinos and Wight model. For example, Fig. 10 represents a typical RC interior beam-column connection’s joint shear behavior (from an actual experiment and per various models; see Kim and LaFave 2009 for further details). Based on visual comparison (of this particular case, and in general), the modified Parra-Montesinos and Wight model greatly improves accuracy in the prediction of RC joint shear resistance behavior compared to the original Parra-Montesinos and Wight model.

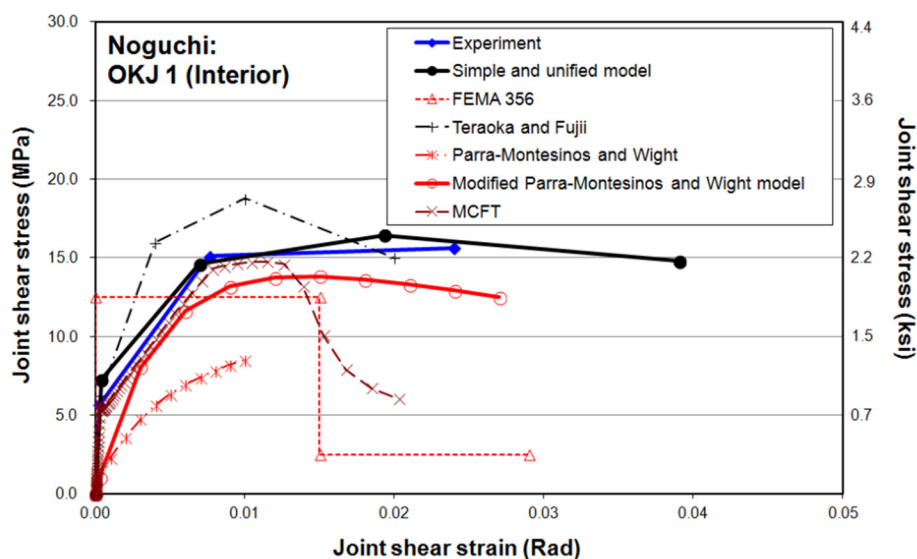


Figure 10 – Comparison of experimental RC beam-column connection joint shear behavior to various models.

For maximum joint shear stress, the modified Parra-Montesinos and Wight model provides overall means of θ and σ that are -0.031 and 0.215 , respectively. As can be seen in Fig. 11, reliability of this updated model is still less than that of the simple and unified model (Eq. (4)) reported earlier. However, reliability of this modified Parra-Montesinos and Wight model is distinctively better than that of the original Parra-Montesinos and Wight (2002) model (which had $\theta = 0.190$ and $\sigma = 0.359$).

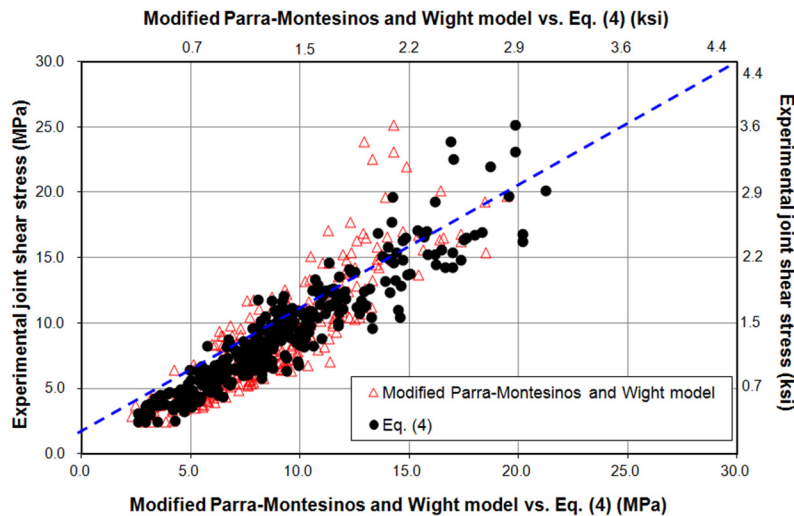


Figure 11 – Comparison of modified Parra-Montesinos and Wight model vs. Eq. (4) for point C stress.

For joint shear strain at the maximum stress, the modified Parra-Montesinos and Wight model results in overall means of θ and σ that are 0.311 and 0.467, respectively. In the case of the original Parra-Montesinos and Wight model, the overall means of θ and σ are 0.422 and 0.410, respectively, for joint shear strain at maximum stress. The modified model is therefore less biased and more reliable than the original model, although the improvement is not nearly as significant as was the case for maximum joint shear stress.

SUMMARY AND CONCLUSIONS

An extensive database was constructed of RC beam-column connection tests, subjected to cyclic lateral loading, where all cases within the database experienced joint shear failure. Using the experimental database, envelope curves of joint shear stress vs. joint shear strain behavior have been created by connecting key points such as cracking, yielding, and peak loading. Various prediction approaches for RC beam-column connection joint shear resistance have then been presented, discussed, and even modified, as informed by the constructed experimental database.

RC beam-column connection joint shear strength and deformation models have first been presented based on using the database in conjunction with a Bayesian parameter estimation method, and then a complete model applicable to the full range of RC joint shear behavior has been suggested. This simple and unified unbiased model was developed to also minimize scatter in its predictions across the diverse set of subassemblies comprising the entire experimental database. Some of the model's parametric values imply that adjustments could be needed in order to make current ACI and ASCE/SEI design and assessment approaches more consistently rational.

An existing RC beam-column connection joint shear prediction model (by Parra-Montesinos and Wight), which was founded upon using basic joint shear resistance mechanisms, has also been described and critically assessed. Limitations of this model, in part due to it having been originally established in order to conservatively limit damage and drift, have been addressed by again using the experimental database, in conjunction with probabilistic methods, to refine certain aspects of it pertaining to the relationship between the principal strain ratio and joint shear deformation of a joint's equivalent compression strut. These refinements have enabled a *modified* Parra-Montesinos and Wight model to become a similarly reliable method as the simple and unified joint shear behavior model described above, for accurately characterizing RC beam-column connection joint shear resistance across a wide range of diverse connection types.

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Hybrid Testing of Beam-to-Column Connection Regions of a Code Compliant RC Moment Frame Building

Burcu Burak Bakir

Synopsis: The beam-to-column connection regions of reinforced concrete frames are generally considered to be rigid and the inelastic behavior of the joint is not considered both in analysis and design applications. However, joint shear distortions that are observed under earthquake loading have a major contribution to the story drift of a structure. Even if the story drifts are not significant, a collapse mechanism may be formed due to extensive deformations in the beams, columns or connection regions. Therefore, the seismic performance of each member should be investigated independently from that of the frame. In this experimental research, the seismic performance of beam-to-column connection regions of a 1/2-scale three-story three-bay RC frame is studied. Hybrid (pseudo-dynamic, PsD) testing method is utilized and earthquake excitations with increasing intensities are applied sequentially to the frame. The reinforced concrete moment resisting frame building is designed following the code requirements, however a joint mechanism is formed under high intensity seismic loads. The joint shear distortions are observed to be significant even for the basic safety earthquake level, when soft soil conditions are present. In this paper, the seismic performance of the frame and individual member responses are discussed for earthquake loadings with different probabilities of exceedance. The interaction between the beam end rotation and joint shear distortion and the effect of connection performance on the seismic behavior of the frame are explained in detail.

Keywords: beam-to-column connections, hybrid testing, joint mechanism, joint shear distortion, performance assessment

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INTRODUCTION

In the design of reinforced concrete moment resisting frame (MRF) structures, the beam-to-column joints are expected to be in the elastic range and generally considered as rigid zones. However, experimental results obtained from numerous subassembly tests indicated that they undergo large inelastic shear deformations and the inelastic behavior of connections subjected to cyclic loading significantly affects the overall behavior of the specimen. There is also limited research on the interaction of members and how the individual member behavior, especially that of the connection regions, affects the overall seismic performance of a frame or a building. During the assessment of the seismic performance of a structure, it is not adequate to assess merely the base shear versus roof drift response (Unal and Burak, 2012, Kim and LaFave, 2012, Jeon et. al., 2014). Even if the members exceed the acceptance limits, due to the robustness of the global response, it may not be significantly affected from modifications in earthquake load characteristics. Therefore, individual member behavior needs to be assessed in addition to the global load-drift response of the frame.

In current rehabilitation codes such as ASCE SEI 41-06, Seismic Rehabilitation of Existing Buildings (2006), performance levels are described as Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) for which the limit states are determined based on the observed damage and displacement levels. For a structure to satisfy IO, there should be very limited structural damage, where the transient drift in the vertical members are less than 1% and the permanent drift is negligible. For LS damage state, the members experience extensive cracking and hinge formation with transient and permanent drifts limited to 2% and 1%, respectively. In CP damage state, the structure is on the edge of collapse where the drifts are less than 4%.

Based on the code requirements, a multi-level performance assessment of each individual member can be carried out under increasing intensity earthquake ground motion records. Several dynamic testing techniques are utilized for the seismic assessment of buildings. The most commonly used ones are shaking table and hybrid (PsD) testing methods. In shaking table tests, structures are subjected to actual earthquake acceleration records and in hybrid testing method, displacement time-history is computed in real time by solving dynamic equations of motion for an earthquake excitation input. There are some experimental studies on the seismic performance assessment of RC frame structures utilizing aforementioned methods, some with high joint distortions (Nagae et.al., 2015), however none with the formation of a joint mechanism that indicates the effect of the inelastic behavior of the connection region to the increased story drifts and the exceedance of the acceptance limits for the members.

In this paper, seismic performance assessment of beam-to-column joints and adjoining beams of a well-detailed 1/2-scale three-story three-bay RC frame is discussed. During hybrid testing, four site specific ground motion records with increasing damage levels and return periods ranging from 72 to 2475 years are applied to the frame. To investigate the site effect, the soil type is modified as firm or soft soil for the 475 year return period earthquake record. At the end of the test, a joint mechanism is formed that leads to the failure of the specimen. Therefore, in addition to the inelastic behavior of the frame and individual members, the influence of connection performance on the overall performance is also discussed.

DESCRIPTION OF SPECIMENS AND TEST SETUP

The test frame is a three-story three-bay 1/2-scale model of a reinforced concrete moment resisting frame building (**Figure 1**). The reinforcement detailing of the members satisfy ACI Committee 318, Building Code Requirements for Structural Concrete (2011) and the joint reinforcement also complies with the ACI-ASCE Committee 352, Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures (2002). For the prototype building, natural period is 0.35 sec., maximum base shear is 142.9 kN (32.1 k) and the seismic load reduction factor is taken as 8. Site specific design spectrum obtained based on the Turkish Earthquake Code (TEC 2007) is used for design. The reinforcement detailing of beams, columns and joints are presented in **Figure 2**. As it can be observed from this figure, 4 mm (0.16 in.) diameter stirrups are placed with a spacing of 40 mm (1.58 in.) in both exterior and interior joints. Slabs are present at each floor level with an effective flange width (b_e) of approximately eight times the slab thickness to consider the slab contribution and provide the

Hybrid Testing of Beam-to-Column Connection Regions of a Code Compliant RC Moment Frame Building

confinement of the connection region. The gravity loading provided by the steel block weights are also placed on the slabs. Mean compressive strength of all members is 20 MPa (2.9 ksi). The mean yield strength and the ultimate strength of the longitudinal and transverse reinforcement are 434 MPa (62.9 ksi), 630 MPa (91.4 ksi), 250 MPa (36.3 ksi) and 370 MPa (53.7 ksi), respectively.



Figure 1 – Test frame.

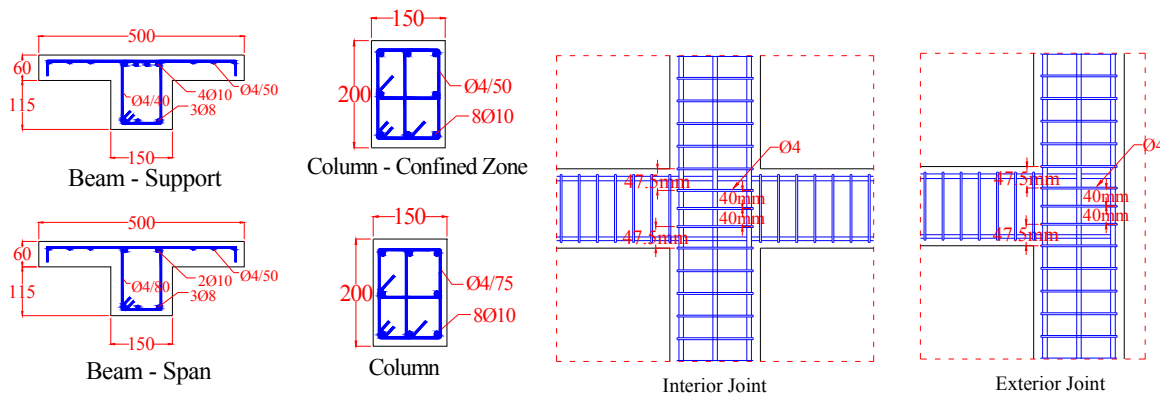


Figure 2 – Reinforcement detailing (all units are in mm, 1 mm = 0.039 in.).

The displacement demands are imposed to the specimen by the use of three displacement-controlled hydraulic actuators, one at each floor level. A synthetic acceleration-time history record that is composed of four sequential earthquake data representing different hazard levels is generated to be used for PsD testing (**Figure 3**). D1 is a record having a probability of exceedance of 50% in 50 years. D2 and D3 have probabilities of exceedance of 10% in 50 years. All ground motion records except D3 are produced for firm soil corresponding to Class B in ASCE SEI 41-06 (Seismic Rehabilitation of Existing Buildings) (2006), whereas a soft soil of Class C is selected for D3 ground motion. The final record, D4, has the probability of exceedance of 2% in 50 years.

The member deformations and the story displacements are measured by the use of Linear Variable Displacement Transducers (LVDTs). Two LVDTs each are placed diagonally, vertically and horizontally to one interior and one exterior first story joint to obtain the joint shear response. Moreover, two LVDTs are attached to the top and bottom on both ends of the beams in each story in order to obtain the beam rotations. Two strain gage based transducers are attached at the bottom of the first story exterior columns above the foundation level (Canbay et al.

2004) to measure shear, axial forces and moment at the base of the exterior columns of the indeterminate system (Figure 4).

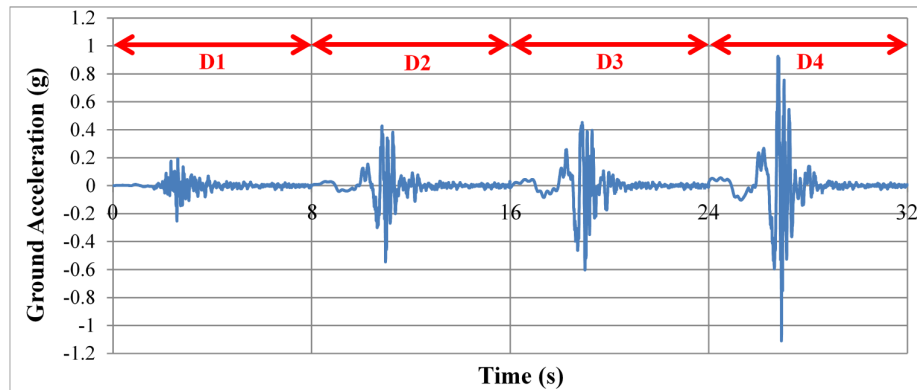


Figure 3 – Applied ground motion.

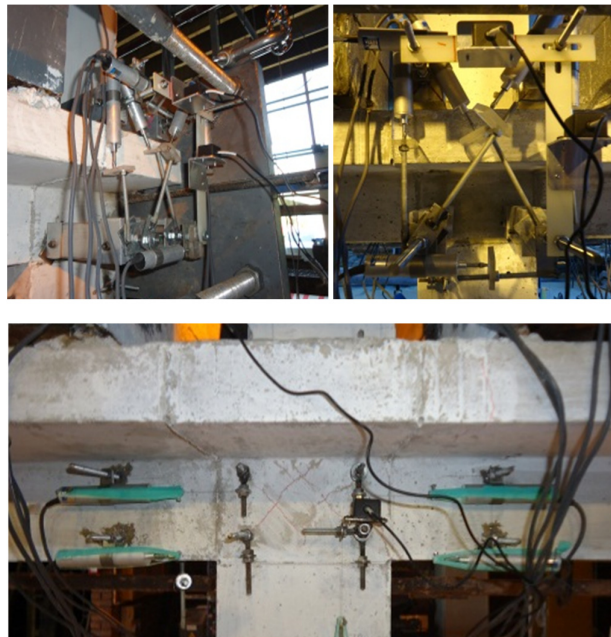


Figure 4 – Instrumentation.

EXPERIMENTAL RESULTS

Observed damage

During the application of D1 ground motion, no cracks were observed except for the minor cracking of the first story interior joints (Figure 5), because the observed interstory drifts were significantly low between -0.3 and 0.2% (Figure 6). With D2 ground motion, number of cracks increased in the first story. Hairline cracks opened at the beam-column interface of interior connections and the first story exterior joints (Figure 7). All interstory drifts remained below 2%. However, for D3 ground motion, which was scaled for a soft soil site, number and width of cracks in the first story interior joint and the beam-column interface increased drastically, correlating well with increased interstory drifts of 2-3%. The first story exterior joints had relatively narrower cracks and hairline diagonal cracks were observed in the second story interior joints. During the application of D4 ground motion, number and width of cracks continued to increase in the first story interior joints and considerable concrete spalling was observed both in the joint and at the beam-column interface followed by the formation of a joint

mechanism. Exterior joints had significantly narrower cracks and there was no cracking at the beam-column interface. First story columns exhibited flexural hinging, but cracks were not as severe as the ones observed in interior connection regions. Plastic deformations were mostly concentrated on the first story members, the second story members had fewer and thinner cracks and no cracks were observed in the third story members.

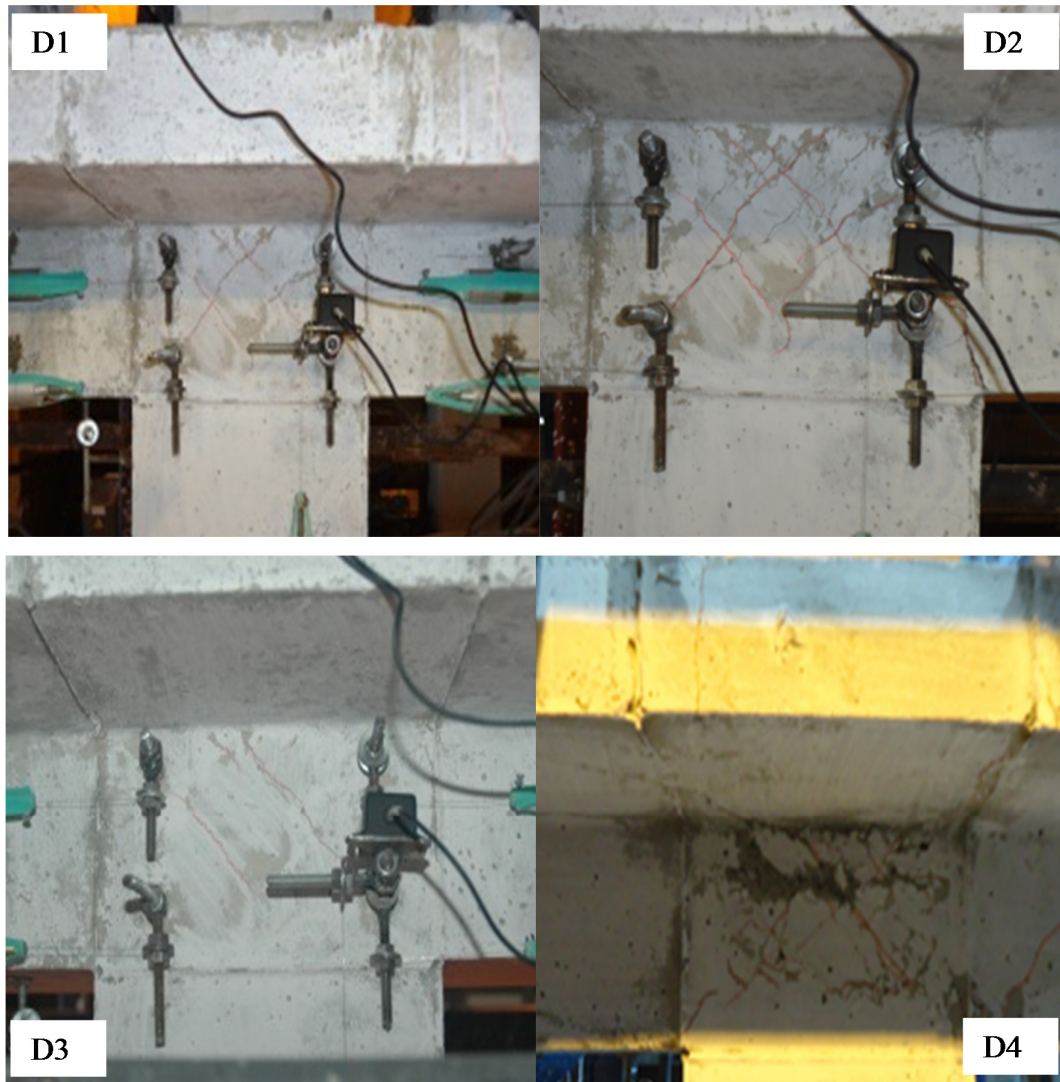


Figure 5 – Interior joint damage.

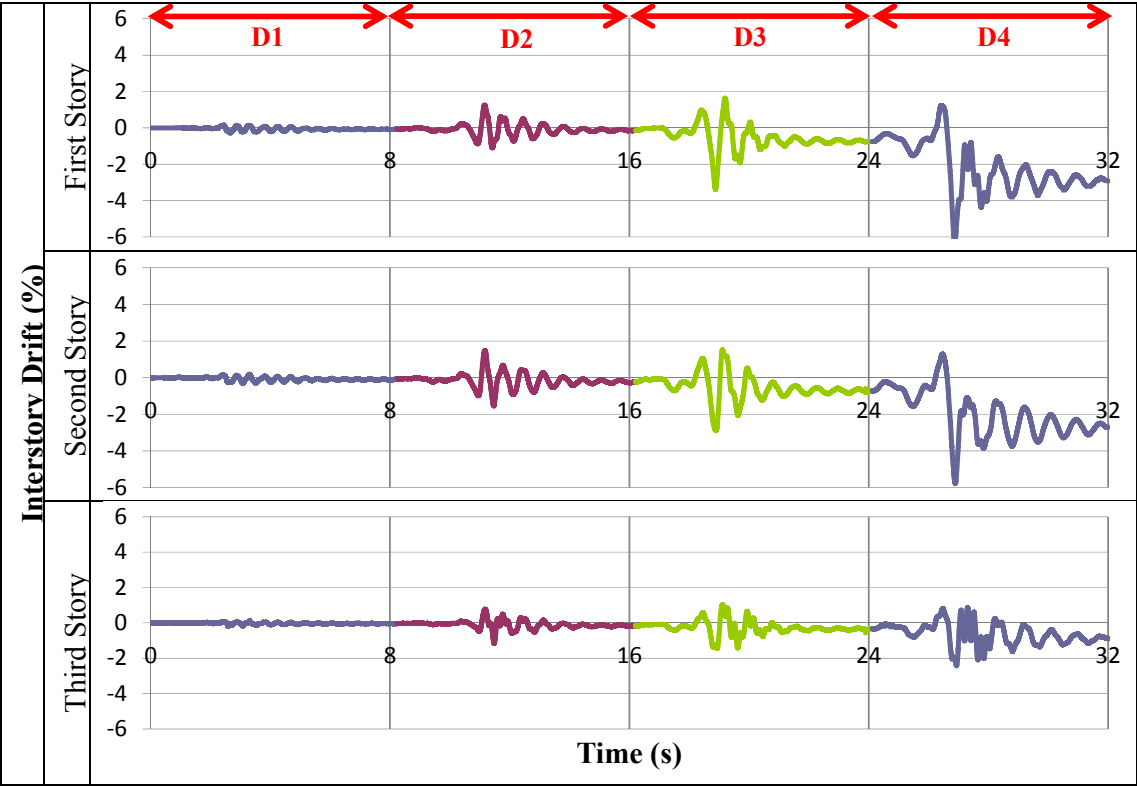


Figure 6 – Interstory drifts.

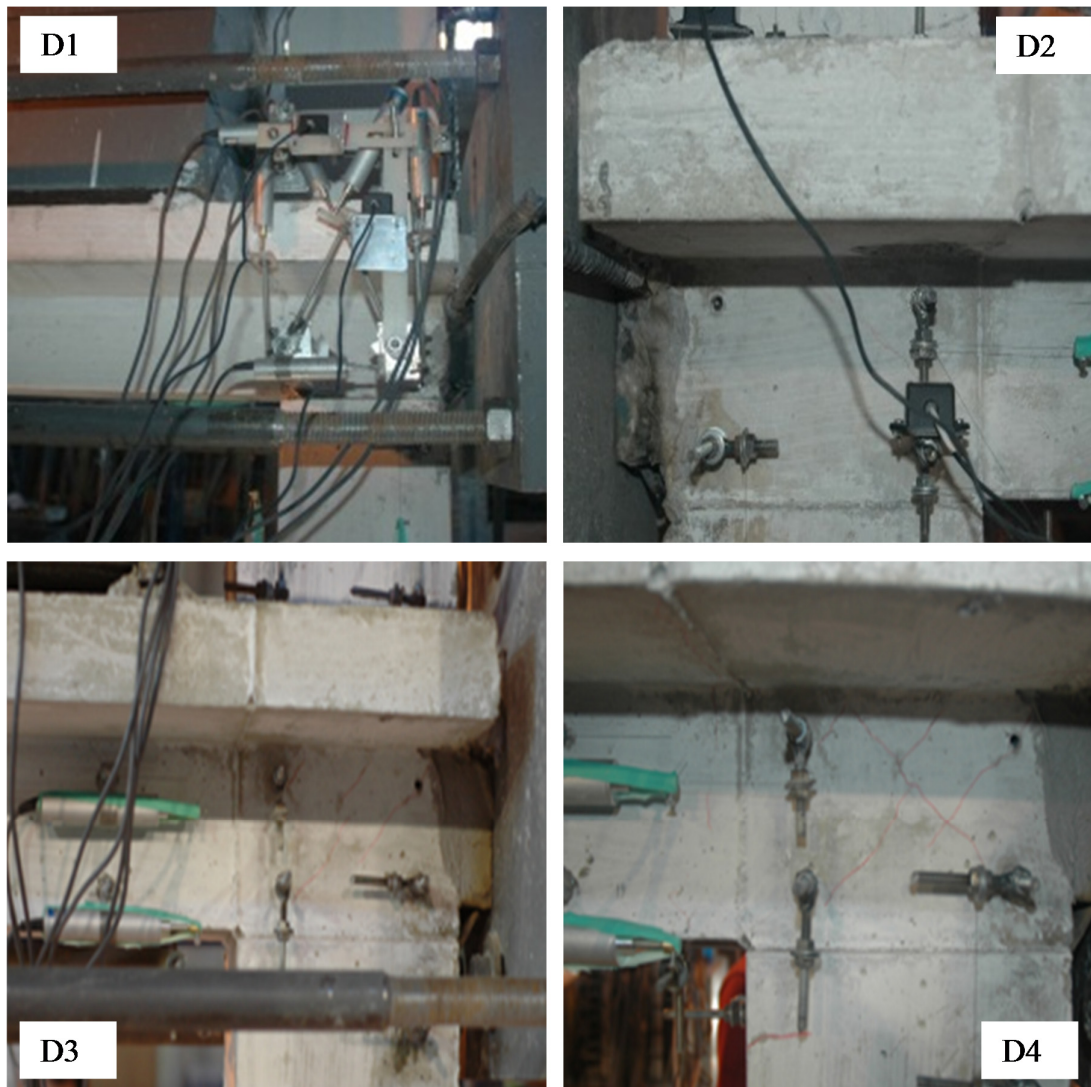


Figure 7 – Exterior joint damage.

Performance assessment

Performance of the connection regions are assessed based on the acceptance criteria for nonlinear assessment procedures in ASCE SEI 41-06 (Seismic Rehabilitation of Existing Buildings) (2006) and Update to ASCE/SEI 41 Concrete Provisions (Elwood, et.al, 2007). Performance levels considered in this assessment are the Immediate Occupancy Level, Life Safety Level and Collapse Prevention Level. The nonlinear acceptance criteria for the exterior and interior joints and the adjoining beams examined in this study are given in **Table 1**. In this table P is the design axial load on the column above the joint and A_g is the gross cross-sectional area of the joint. Transverse reinforcement is defined as conforming (C) if stirrups are placed at a smaller or equal spacing than one third of the column depth for a joint and one third of the beam depth for the plastic hinge region of a beam. The design shear force (V) for the beams and joints are taken as the maximum shear forces observed during the loading history. Shear strength of the joint (V_n) is computed following the requirements of ACI 352-02 guidelines. **Table 2** presents parameters related to the shear strength of the joint. In this table, moment strength ratio, M_r , is the ratio of sum of the column moments to that of the beams framing into the joint. $A_{sh-provided}$ is the total cross-sectional area of the transverse reinforcement, whereas $A_{sh-required}$ is the minimum required value recommended by ACI 352-02.

Table 1 – Acceptance criteria for the first story joints and the adjoining beams based on ASCE 41-06.

				Acceptance Criteria				
Joints	$\frac{P}{A_g \cdot f_c}$	Transverse Reinforcement	$\frac{V}{V_n}$	Shear Distortion (radians)				
				IO	Primary		Secondary	
					LS	CP	LS	CP
Exterior	0.094	C	1.135	0	0	0	0.015	0.020
Interior	0.154	C	1.211	0	0	0	0.019	0.029
Beams	$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse Reinforcement	$\frac{V}{b_w \cdot d \cdot \sqrt{f_c}}$	Plastic Rotation (radians)				
				IO	Primary		Secondary	
					LS	CP	LS	CP
EB-EJ	0.283	C	2.157	0.007	0.014	0.022	0.020	0.038
IB-IJ	0.283	C	1.509	0.007	0.014	0.022	0.020	0.038
EB-IJ	0.283	C	2.157	0.007	0.014	0.022	0.020	0.038

EB-EJ: Exterior Beam Framing into the Exterior Joint

IB-IJ: Interior Beam Framing into the Interior Joint

EB-IJ: Exterior Beam Framing into the Interior Joint

 f_c in psi**Table 2 – Joint properties.**

	Moment Strength Ratio (M_r)	Joint Shear Strength (V_n) (kN) (k)	Maximum Shear Load (V) (kN) (k)	$A_{sh,required}$ (mm ²) (in ²)	$A_{sh,provided}$ (mm ²) (in ²)
Exterior Joint	1.95	133.6 (30.0)	151.6 (34.1)	28.0 (0.043)	37.7 (0.058)
Interior Joint	1.28	167.0 (37.5)	202.3 (45.5)	28.0 (0.043)	37.7 (0.058)

The experimental behavior of joints and beams are compared with the acceptance criteria presented in **Table 1** for the assessment of seismic performance (**Figures 8-11**). The shear response of the connection regions are obtained by using the joint shear strains that are computed from the horizontally, vertically and diagonally placed LVDTs following the same methodology utilized by Burak and Wight (2008). Beam rotations are computed by dividing the difference of deformation measurements of top and bottom potentiometers to the vertical distance in between these LVDTs. The joint shear force and beam moment values are computed based on the data obtained from two strain gage based transducers attached at the bottom of the first story exterior columns. The rest of the base shear force carried by interior columns are considered to be equal.

During the application of D1 ground motion record, which has a return period of 72 years, only hairline cracks were observed on all members. Joint shear strains were below 0.1% for exterior and 0.2% for interior joint and the beam rotations were less than 0.2%. Since no shear strain is allowed in the joints based on ASCE 41-06 requirements, the frame exceeds the Immediate Occupancy Limit State at the end of this test, which is considered to be highly conservative. No significant damage was observed in any member after the application of D2 ground motion that has a 475 year return period. The damage can be considered to satisfy the Life Safety Performance Level. Joint shear strains, which were below 1.0% for both exterior and interior joints, satisfy the acceptance criteria for secondary joints in ASCE 41-06, although, these joints are part of the primary lateral load resisting system. Adjoining beams had a maximum of 0.5% rotation, all the interstory drifts and the roof drift remained below 2% (**Figure 12**), which conform with the LS performance level. Although D3 ground motion has the same return period with D2, since it is produced for a soft soil site, the number and width of cracks increased significantly and spalling off of cover concrete initiated during the application of this ground motion. Shear strains exceeded 1% for the exterior joint and reached 2% for the interior joint. Beam rotations exceeded 1.4% due to widening of the interface crack observed on the face of the interior beam, which indicate the Collapse Prevention Performance. The first and second story drifts and the roof drift exceeded 2%, however, the frame did not lose any of its load carrying capacity and it can be observed that the deterioration of member behavior is more pronounced than that of the base shear vs. drift response. At the end of the test, when D4 ground motion that has a return period of 2475 years was applied, formation of a joint mechanism was observed, however the frame continued to carry the applied dead load without collapse. The total beam end rotations increased significantly partly due to the widening of the beam-column interface cracks at the interior connection region. New crack

formation was not observed in the exterior joint which had not reached collapse prevention level. Furthermore, the exterior joint shear distortions decreased when compared to those observed for the D3 ground motion, which is believed to be due to the increase in the exterior beam rotations. Therefore, the distribution of plastic deformations among the members of a connection region should be considered in the performance assessment of members, because, it is clearly observed that the joint distortions decrease when the beam rotations increase and vice versa. The moment strength ratios (M_r) of the exterior joints given in Table 2 are relatively high, therefore only hairline cracks were observed. However, since this ratio is close to the limit of 1.2 for interior joints, significant shear distortions were observed leading to the formation of a joint mechanism.

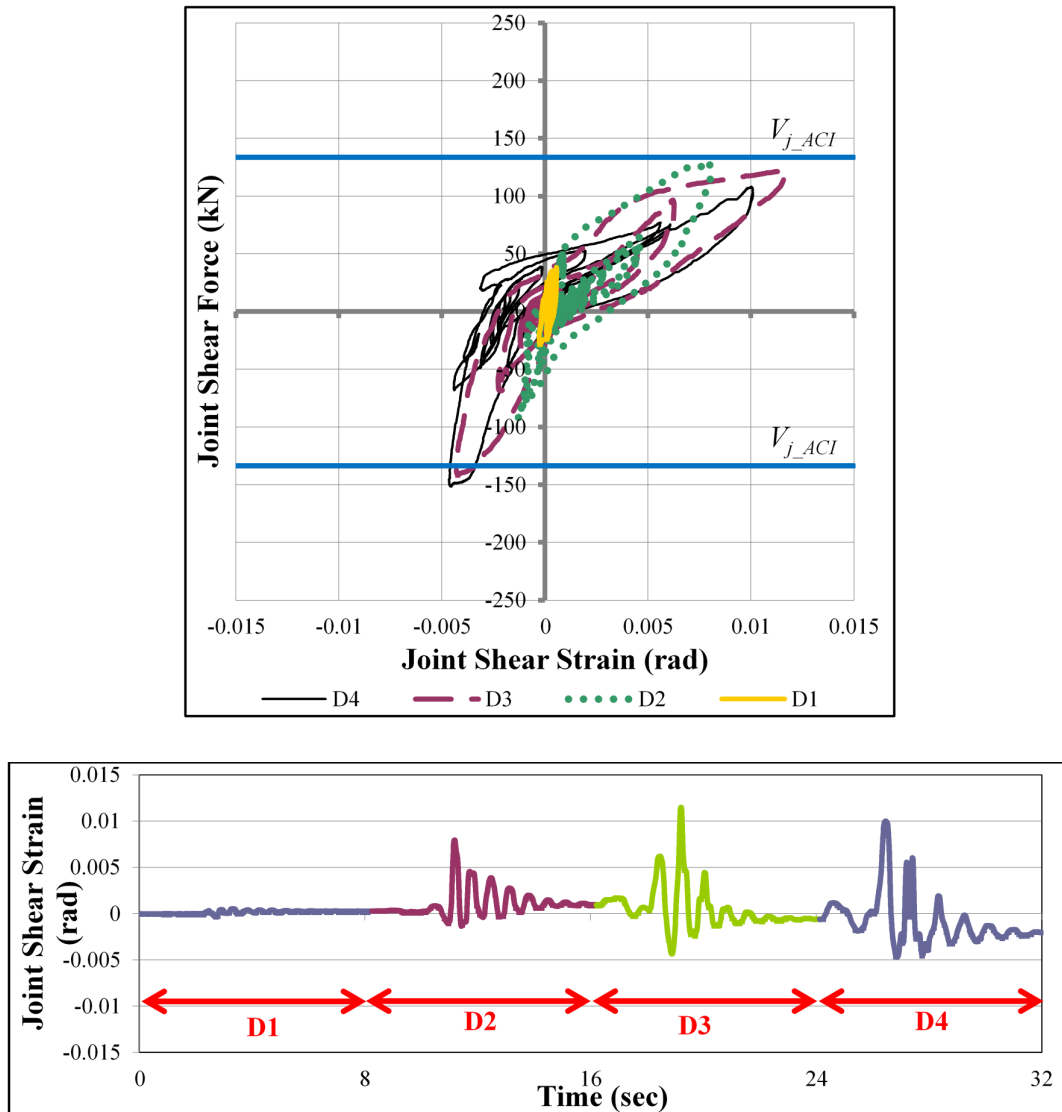


Figure 8 – Joint shear response of the exterior joint (1 kN = 0.225 kips).

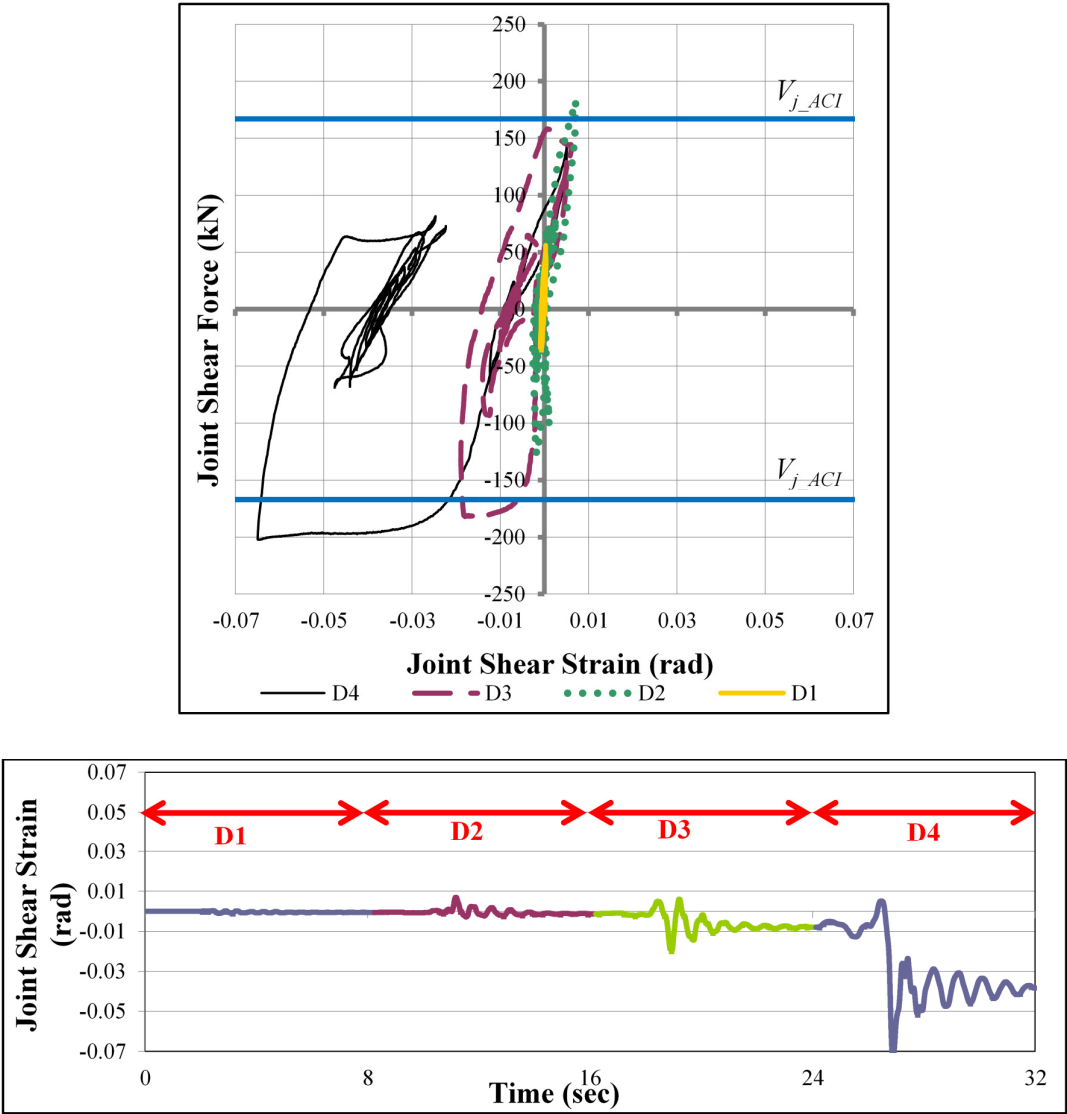


Figure 9 – Joint shear response of the interior joint (1 kN = 0.225 kips).

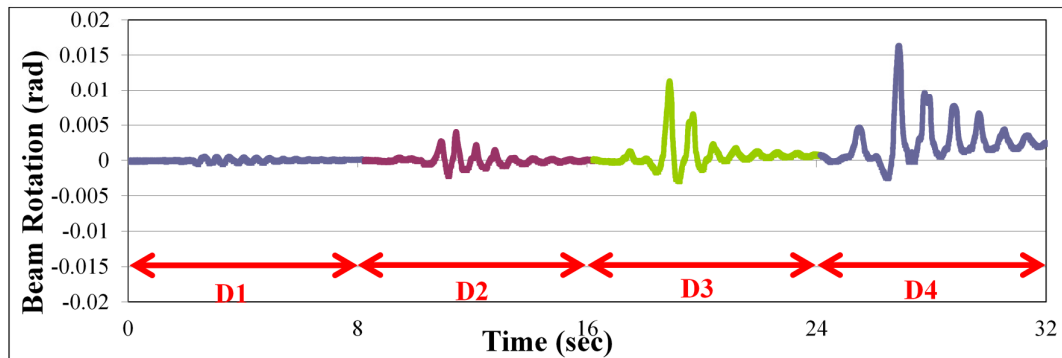
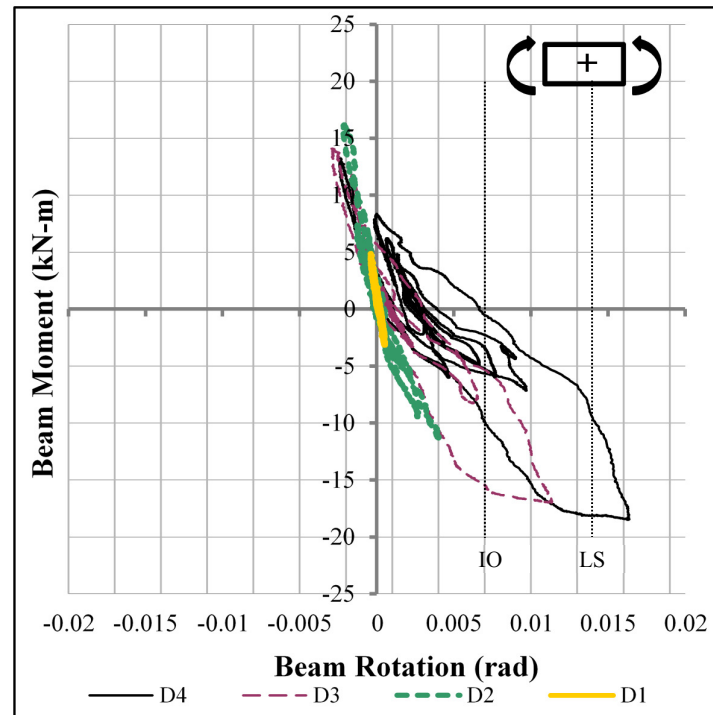


Figure 10 – Moment-rotation response of the beam adjoining the exterior joint (1 kN-m = 0.74 kip-ft).

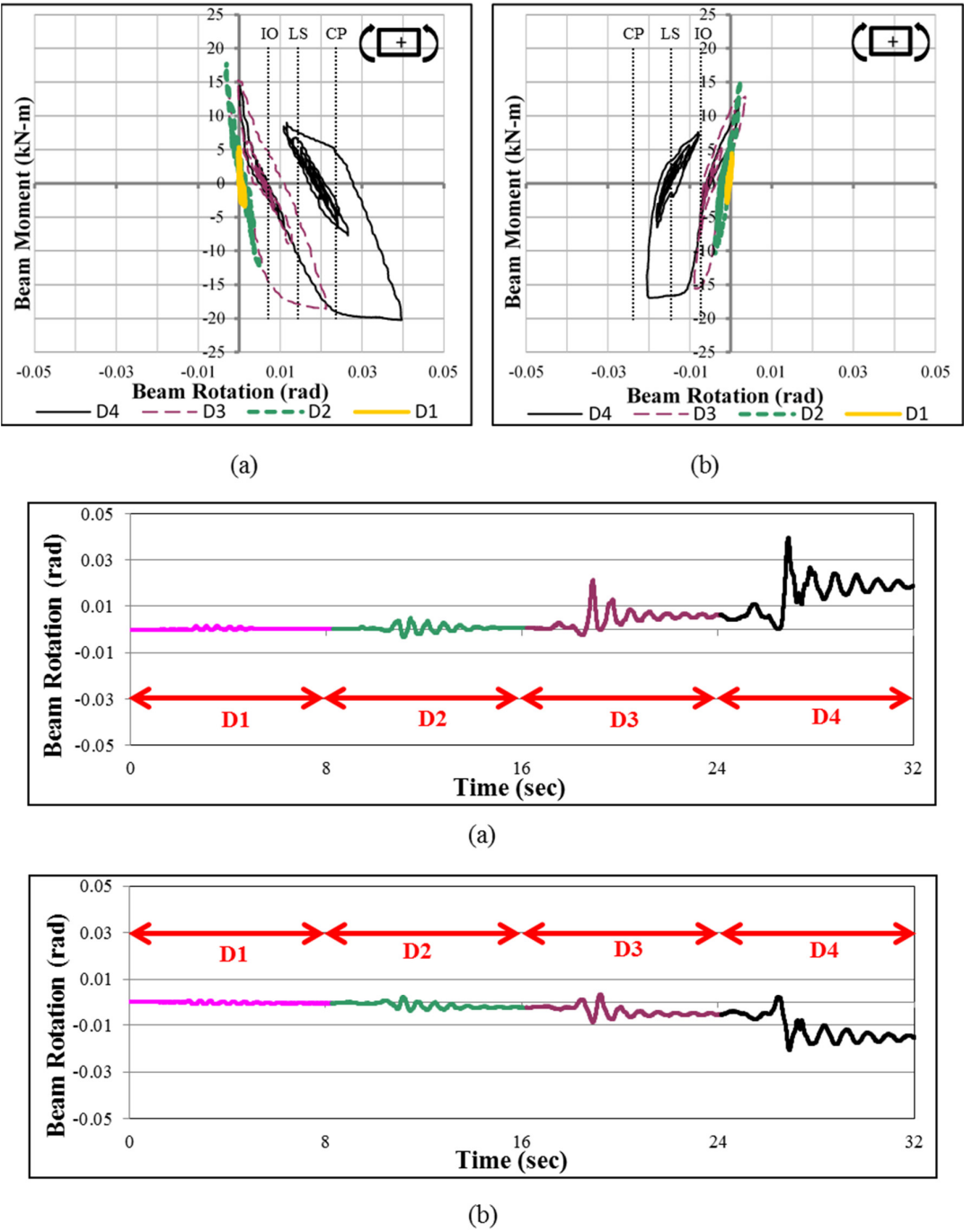


Figure 11 – Moment-rotation responses of the beams adjoining the interior joint
(a) interior beam (b) exterior beam (1 kN-m = 0.74 kip-ft).

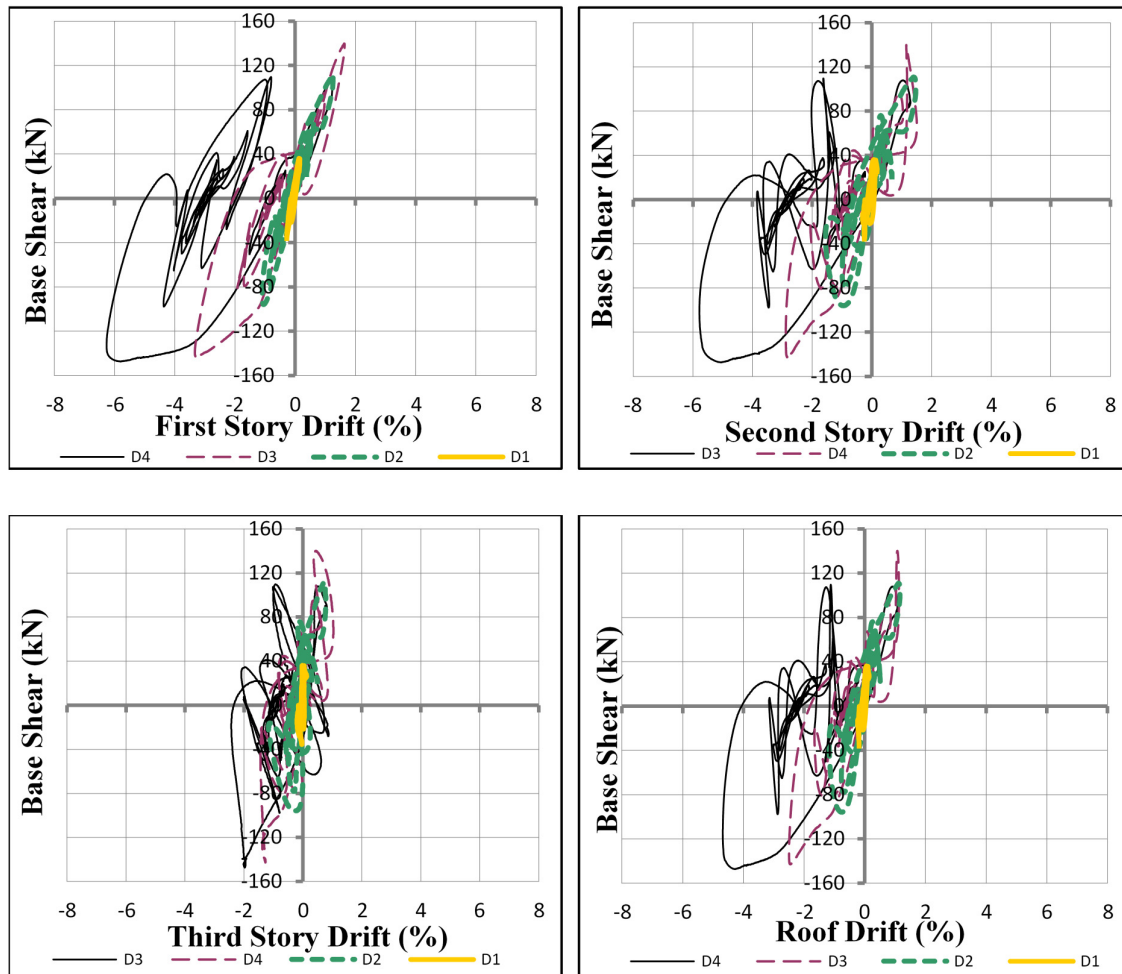


Figure 12 – Base shear vs. drift responses (1 kN = 0.225 kips).

CONCLUSIONS

A 1/2-scale three-story three-bay RC frame is tested under hybrid (PsD) loading for ground motion records with return periods of 72, 475 (on firm and soft soil) and 2475 years. The performance assessment of beam-to-column connection regions is carried out considering the joint shear distortions, beam end rotations and the observed damage and the following conclusions are drawn:

- The test results indicated that even well-detailed reinforced concrete frame buildings may experience failure due to the formation of a joint mechanism under high intensity earthquake loading.
- Moreover, the joint shear distortions could be significant even for the basic safety earthquake level, if soft soil conditions are present.
- The experimental results revealed the importance of assessing individual member performance. Even when the plastic deformations in the members exceeded the acceptance limits, the roof and interstory drifts satisfied the code requirements.
- When the joints had increased shear distortions, the beams had reduced rotations. Therefore, in the assessment of member performance, the analytical models should include the influence of joint distortions on the beam rotations to accurately label the beams as deficient without overestimating the rotations.
- ASCE 41-06 acceptance criteria for the joints are observed to be conservative estimates and should be reevaluated.
- The effect of the moment strength ratio on the performance of the connection regions should also be reexamined to determine whether the limit of 1.2 required by ACI 352-02 guidelines is adequate or it should be increased back to 1.4.

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REINFORCED CONCRETE IN MOTION

A Retrospective of Developments on Knowledge Related to Flexural Response of Reinforced Concrete

By

Mete A. Sozen

Synopsis: Among his many achievements, Professor Wight's observation and explanation of the irrelevance of the linear-plastic model for the moment-curvature relationship of a structural element subjected to a moment gradient (rather than constant moment) along its span stands out as one of his best contributions and one that will stand the test of time.

Keywords: deflection limit, ductility, flexural strength, load-deflection relationship, moment-curvature

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INTRODUCTION

An admittedly narrow perspective of early research on reinforced concrete reveals that until mid-20th century, the main target of developmental work in Urbana, IL, on reinforced concrete was strength. Talbot tested and reported tirelessly beams with and without longitudinal reinforcement in the 1900's focusing on strength^{1,2}. V. P. Jensen produced an elegant set of expressions defining flexural strength³. E. Hognestad broadened Jensen's concepts to include axial loading⁴. However, displacement did not receive more attention than that contained in definitions of flexural stiffness for cracked and uncracked sections. It is not unfair to attribute to N. M. Newmark the sensitivity to the coordinates of the yield moment as well as to the limiting unit curvature to define a linear-plastic shape for the relationship of moment to unit curvature for reinforced concrete.

As long as the demand referred to energy absorption or limiting strength, the linear-plastic concept was sufficient for intelligible design procedures related to static and impulsive loads. With N. M. Newmark's genius for making the difficult simple and C. P. Siess's rare talent for not missing detail in observation as well as in expression, experimental work by J. R. Gaston⁵ and H. McCollister⁶ focused on the entire moment-curvature relationship to produce convincing confirmation of, in an environment of no or little moment gradient, the quantitative definition of the linear-plastic moment-curvature relationship for moment increasing monotonically to failure. When, sometime after WWII, the issue became earthquake demand and it had to be recognized that reinforced concrete had to be evaluated in motion, the tests by N. Burns, R. Yamashiro, and C. P. Siess^{7,8} indicated that definitions, based on monotonically increasing load, of the coordinates for yield moment and limiting unit curvature would not hold. But the linear-plastic preconception continued to rule the day.

It was J. K. Wight¹⁰ who tested reinforced concrete elements simulating conditions of frame members subjected to cycled lateral deformation and showed that the effect of the "flat top" stress-strain curve of reinforcing steel would not be reflected in the moment-rotation relationship if a moment gradient existed along the member. He also reported that the magnitudes of both strength and deflection at the limit would be affected by displacement cycles. Wight's work destroyed the comfort of the linear-plastic response and revealed that the shear demand on a girder could be higher than that based on the yield moment by as much as the ratio of the limiting to yield stress of the reinforcement. In practice, that ratio would have to be amplified by the ratio of the actual to the nominal strength.

The following text, written in celebration of Professor James K. Wight's iconic career, traces the movement of symbolic representations of the response of reinforced concrete in flexure from A. N. Talbot to J. K. Wight in Urbana, IL, in order to focus on only one of Wight's achievements in experimental research.

THE LINEAR-PLASTIC MODEL

As it can be found in any text book on design of reinforced concrete, the relationship between bending moment and unit curvature can be classified as linear-plastic if the section is reinforced with steel that, after yield stress is reached in the tensile reinforcement, stretches with negligible increase in stress. The linear-plastic representation of the moment response of a reinforced concrete section is appropriate and adequate for dimensioning reinforced concrete sections to resist static loads. For reinforced concrete elements subjected to impact or earthquake, the linear-plastic model can also be used but it is important for the engineer to be sensitive to the actual response of reinforced concrete.

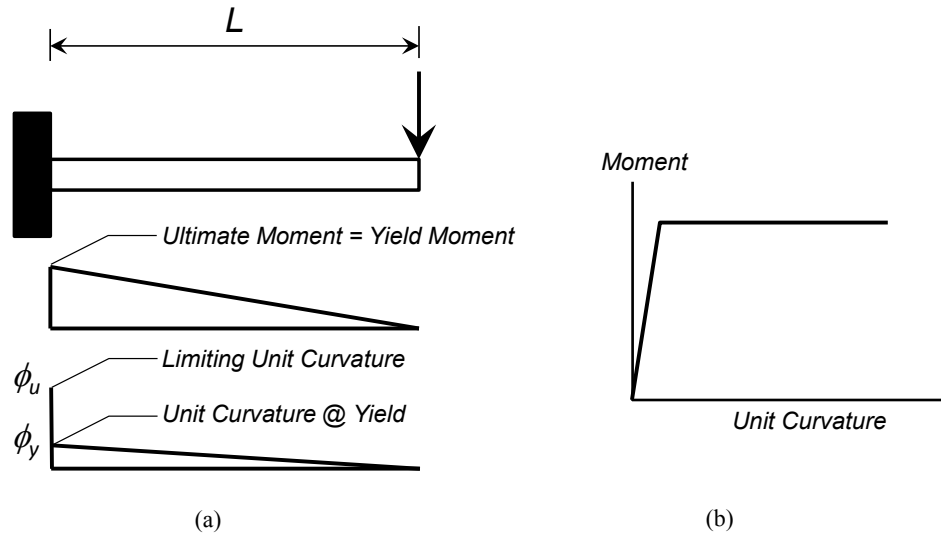


Figure 1 – Distribution of unit curvature along the span of a cantilever with a concentrated load.

Consider a cantilever weightless beam loaded at its free end (Fig. 1). To determine the response of the beam, the moment-curvature relationship for its critical section is assumed to be linear-plastic. With moderate and higher amounts of tensile reinforcement, the effect of the tensile strength of the concrete may be neglected. If the load is increased to reach the yield moment at the support, the yield deflection Δ_y may be estimated simply by taking the first moment of the unit curvature diagram varying from the unit yield curvature, ϕ_y , at the support to zero at the loaded end with respect to the free end.

$$\Delta_y = \frac{1}{3} \phi_y * L^2 \quad (1)$$

Δ_y : Yield deflection at free end of beam.

ϕ_y : Unit curvature at yield.

L : Beam span.

Limiting condition for deflection corresponds to the reaching of the limiting unit curvature, ϕ_u , at the support. Because the limiting moment is assumed to be equal to the yield moment (linear-plastic response), the increase from ϕ_y to ϕ_u occurs over no distance at the support as illustrated in Fig. 1. Considering that the unit curvature has the dimension 1/length, to make a difference in the deflection it has to be integrated over a finite length. In this case the length is zero. The conclusion that defies observation is that the limiting deflection does not exceed the yield deflection. The idealized relationship between moment and unit curvature is clearly in error. What is the root cause of the error? The following sections summarize the conceptual models used to define the flexural response of reinforced concrete by A. N. Talbot^{1,2}, V.P. Jensen³, N.M. Newmark, C.P. Siess, Gaston⁵, H. McCollister⁶, and J. K. Wight¹⁰.

A.N. TALBOT

It is not an irresponsible claim to suggest that Talbot learned almost all that needs to be learned about the response of reinforced concrete structural elements through methodical testing starting at the beginning of the 20th century. He reported his preliminary findings on response of reinforced concrete columns and girders in 1904¹. A strict follower of the principle “observation without preconception” he tended to refrain from expressing his observations in symbolic expressions. Instead of using a formula, his concept of flexural strength was stated as “So far as the strength of the

beam is concerned, the load when the steel is stressed to its elastic limit seems the proper basis for the factor of safety.” His remarks showed his sensitivity to the strain distribution over the depth of the beam not being linear at cracked sections (Fig. 3). In his 1904¹ bulletin, it appeared that his focus was on reinforced concrete behavior up to yielding of tensile reinforcement (Fig. 2).

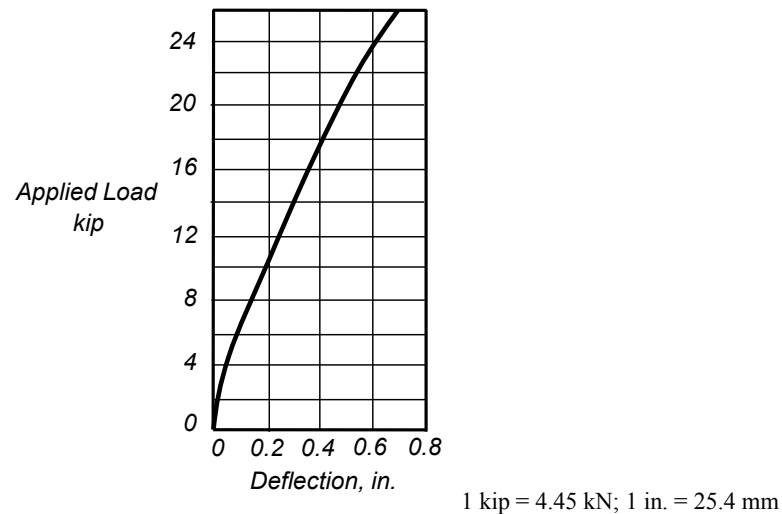


Figure 2 – Result of a test reported by Talbot.

Talbot’s test beams were loaded at more than one point along the span. Measured deflection response was dominated by the distribution of curvature in segments of the test beams with low or no moment gradient along the span.

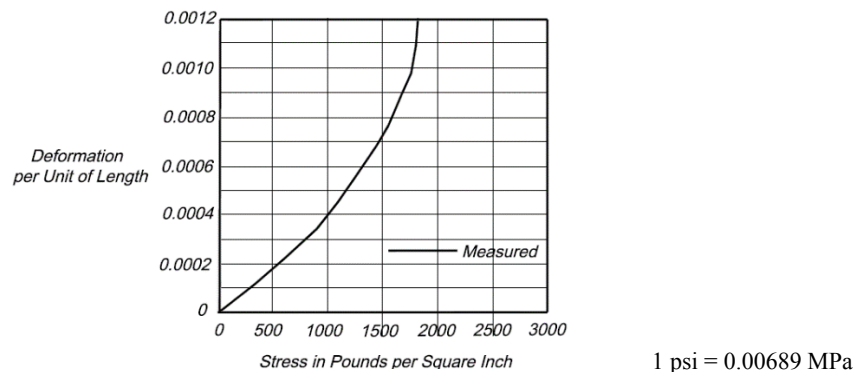


Figure 3 – Talbot's concept of the stress-strain relationship for compressed concrete.

In 1907² he reported tests of beams with a single load at mid-span. He observed that such a loading condition led to a maximum load that exceeded those observed in test beams, having the same cross-sectional properties, with multiple loading points. His inference was to ignore the added strength above that given by the “ordinary beam formula” on the strength of his observation that beams loaded at more than one point as well as uniformly loaded beams of similar cross-sectional dimensions yielded similar results if they had the same amounts of reinforcement of similar strength. His definition of an under-reinforced beam was “beams having shallow depth as compared with their length and having a moderate amount of reinforcement may, when tested with usual way of loading, be expected not to fail before steel has been stretched to its yield point.” With that condition, the flexural strength of the beam could be estimated closely by the product of the yield force in the steel and a distance close to but smaller than the effective depth of the section. He recognized the nonlinear response of the compressed concrete and concluded that the higher the concrete

strain at the extreme fiber in compression, the lower would be the internal lever arm. Most importantly, he emphasized that as long as the reinforcement yielded in tension, concrete properties would not dominate the strength of the section.

V. P. JENSEN

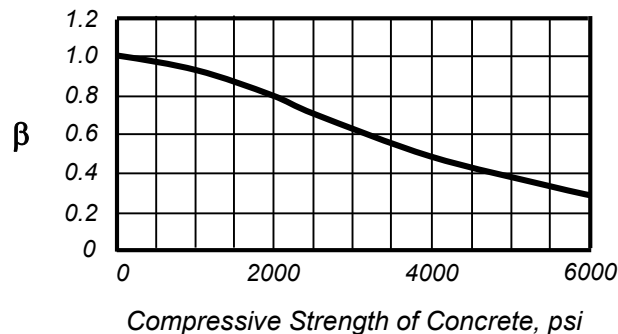
In contrast to Talbot's insistence on "observation without preconception" and on drawing conclusions from observation in language expressed softly, Jensen, who may be considered to be a member of the second generation of researchers on reinforced concrete at Urbana, developed an elegant set of symbolic expressions for flexural response of reinforced concrete in the linear and nonlinear ranges. He recognized strain hardening of the reinforcement and observed that a value of elastic modulus of 30,000 ksi for the steel reinforcement was "sufficiently accurate." He thought the constant stress portion or segment following yielding of the reinforcement could extend up to a strain from 0.005 to 0.02. He suggested that the slope of the stress-strain curve in the strain hardening range would depend on the material type and should be measured for specific applications if accuracy was essential.

Jensen's representation of the stress-strain relationship of concrete was, as he stated, "entirely empirical." In an earlier publication of the University of Illinois Engineering Experiment Station⁹, Richart and Jensen had defined Young's modulus for normalweight-aggregate concrete as

$$E_c = \frac{30 \times 10^3}{5 + \frac{10}{f'_c}} \quad (2)$$

E_c : Young's modulus for normalweight-aggregate concrete in ksi

f'_c : Compressive strength of normalweight-aggregate concrete, ksi (6x12-in cylinder)



1 psi = 0.00689 MPa

Figure 4 – Jensen's coefficient defining the constant-stress portion of the stress-strain relationship of compressed concrete.

It was an expression that was very convenient for evaluation at a glance. Because the numerator was chosen as the assumed Young's modulus of steel, the denominator was the modular ratio.

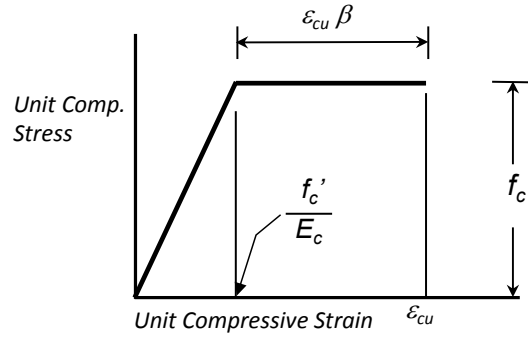


Figure 5 – Jensen's idealization of the stress-strain relationship for compressed concrete.

Jensen idealized the stress-strain relationship for concrete to be linear-plastic (Fig. 5). The plastic segment of the idealized curve he defined using a coefficient β (Fig. 4).

$$\beta = \frac{1}{1 + \left(\frac{f'_c}{4000}\right)^2} \quad (3)$$

f'_c : compressive strength of the concrete, psi

leading to a definition of the limiting strain in compression defined as

$$\epsilon_{cu} = \frac{f'_c}{(1-\beta)E_c} \quad (4)$$

ϵ_{cu} : limiting unit strain of concrete in compression

E_c : Young's modulus for concrete

Combining the two expressions, Jensen's choice for the limiting unit compressive strain, ϵ_{cu} , can be written as

$$\epsilon_{cu} = \frac{f'_c}{\left(1 - \frac{1}{1 + \left(\frac{f'_c}{4}\right)^2}\right) \left(\frac{30,000}{5 + \frac{10}{f'_c}}\right)} \quad (5)$$

with the resulting relationship between limiting strain and compressive strength being as shown in Fig. 6 with the limiting strain varying from approximately 0.003 to 0.002 for the range of compressive strengths he considered from 2000 to 6000 psi. He took a conservative view vis-à-vis the value adopted later by ACI 318.

His crowning achievement is in his choice to explain the process of determining flexural strength in graphic format as shown in Fig. 7. The figure contains two curves. One is simply the assumed stress-strain curve for the tensile reinforcement. The other curve is based on the material properties of a rectangular section and, given assumed or known properties of the concrete and the steel, is derived from the assumed geometry (linear strain distribution over depth of section) and statics. Jensen expressed the resulting relationship between steel stress and strain as:

$$f_s = \frac{\frac{(1+\beta)f'_c}{2\rho}}{1 + \left(\frac{(1-\beta)E_c}{f'_c}\right)\epsilon_s} \quad (6)$$

f_s : unit stress in tensile reinforcement, ksi