PERFORMANCE OF MOMENT-RESISTING CONNECTIONS

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SUMMARY

During the January 17, 1994, Northridge Earthquake, steel frame buildings were damaged. The SAC Steel Project was funded by the US Federal Emergency Management Agency (FEMA) to address this problem. Phase 2 of this project included extensive connection analysis and testing. An overview of the experimental and analytical research work on steel frame connections is provided. Eleven experimental tasks and four analytical tasks with funding of approximately 2.3 million dollars were included in the Connection Performance portion of the SAC Phase 2 program. These studies considered a wide range of bolted and welded steel moment frame connection types. Individual test and analytical programs are briefly summarized, and the some goals and results from each study are noted.

The ultimate goal of this work is the development of practical, simple models for predicting the resistance, stiffness, ductility and performance of connections which are suitable for seismic design. The methods used for developing these models are summarized and illustrated. The detailed results of this work will be provided in a comprehensive state of the art report, which is discussed. This work aids structural engineers in the design of new buildings and evaluation, repair, and retrofit of existing steel frame buildings.

INTRODUCTION

Steel moment frames sustained cracking and fracture of their connections during the January 17, 1994, Northridge Earthquake. The damage occurred in the pre-Northridge welded flange-bolted web connection illustrated in Figure 1. The damage was not expected, and structural engineers were uncertain as to the appropriate course of action with respect to this commonly used structural system. The SAC Steel Project was initiated to provide practical guidance to the structural engineers designing, evaluating, and repairing these connections. The SAC Phase 1 Program provided immediate, but preliminary guidance regarding these connections, while the Phase 2 program provides an in-depth examination of the issues.

Because of the wide range of questions raised by the earthquake, the Phase 2 effort was divided into 5 research teams with a team leader and Technical Advisory Panel (TAP). The author is the team leader for the Connection Performance TAP. The Connection Performance TAP coordinated a wide range of experimental and analytical research. The goals of this research were to identify the causes of past connection damage, to develop an improved understanding of connection behavior, to provide a range of alternative connections to engineers, and to develop the necessary knowledge and tools for the engineer to use these connections. The experimental and analytical tasks were closely coordinated to assure that all critical issues are addressed without excessive repetition or critical gaps. The coordination also assured that the results were useful to the engineering profession. All experimental studies included a component of analysis to assure maximum benefits from the limited funds for experiments. The analyses consisted of finite element analyses, parameter studies, which extended the range of applicability of the test results, and evaluation of past experiment results, which were combined with test results from the SAC Program. The Connection Performance TAP planned and coordinated this work. Progress meetings between the research teams, the Connection Performance TAP and members of the

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Project Management Committee were held at regular intervals to facilitate the transfer of information between the different groups and tasks.

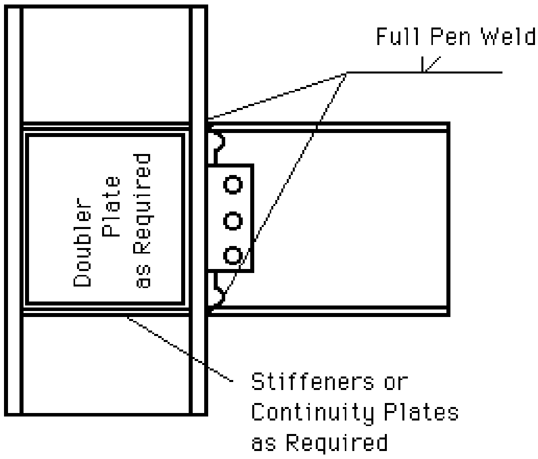

Figure 1. Sketch of Typical Welded Flange-Bolted Web Connection

The Northridge Earthquake showed that connections have many different failure modes. Some of these failures may occur after large plastic deformations, but an unacceptably large number of the failure modes observed during the Northridge Earthquake occurred with little or no plastic deformation. Design rules [ICBO (1991)] in use at the time of the earthquake were simple, but they were believed to be adequate to assure ductile performance of steel moment frames. The earthquake clearly illustrated that it is dangerous and unrealistic to assume that ductile seismic behavior can be assured by prescribing a specific connection type with overly simplistic design rules. Changes in engineering practice occur with time, and these changes effectively negate simple rules which were originally proposed to assure good performance. Some of these changes include -

- changing from shielded metal arc welding to the self shielded flux core E70T-4 process.

- reduced redundancy in steel frames buildings caused by reducing the moment resisting frames to a few isolated bays of the building as opposed to widely distributed moment resisting connections used in early moment frame construction.

- increased reliance upon panel zone yielding.

- use of heavier and deeper beams and columns because of the reduced redundancy noted earlier.

- long term changes in the properties of materials and construction methods.

These changes produce different failure modes, which have a dramatic effect on the seismic behavior. The Connection Performance team efforts were aimed at understanding all failure modes and yield mechanisms that could occur in steel frame connections. The work also included all types of connections that could be used for seismic design, because there are a wide range of seismic demands in different parts of the United States, and engineers need to have different options for these variable conditions.

Yield mechanisms and failure modes are related but are inherently different. Failure modes cause fracture, loss of deformational capacity, or significant loss of resistance. They include weld fracture, panel zone tearing, beam or column fracture, tearing or loss of load capacity due to local or lateral torsional buckling, shear or tensile fracture of bolts, net section fracture, and fracture of local connection elements such as plates or angles. Yield mechanisms induce inelastic deformation and result in dissipation of energy and changes in stiffness without inducing fracture or excessive loss of resistance. Yield mechanisms include flexural yielding of the beams, shear yielding of panel zones, local yielding of plates angles or other connection elements, and other modes of nonlinear behavior. Some yield mechanism and failure mode combinations produce ductile behavior, others result in brittle failure, and still others produce intermediate results. These factors must be understood if each connection type is to be rationally used in seismic design. As a result during Phase 2 research, every yield mechanism and failure mode combination that can be achieved with each connection type was examined, and the strength, stiffness, energy dissipation, and deformation capacity associated with each combination were evaluated.

OVERVIEW OF CONNECTION PREFORMANE RESEARCH

Task 5.3 included the analytical studies on connection performance. Prof. Deierlein at Cornell University and later at Stanford University examined crack growth based on elastic and inelastic crack propagation theory. The primary goal of this work was to establish the degree analytical tools can be used for predicting crack growth and unstable fracture in moment resisting connections. Initial work [Chi et.al. (1997)] focused on pre-Northridge connections and attempted to match the different failure modes and crack directions noted after the Northridge Earthquake and during Phase I testing. The analyses showed good correlation between the measured and calculated force deflection behavior, and it indicated that the weld was the critical region of the pre-Northridge connection because of the flaws in this region and the low toughness associated with welds with E70T-4 electrodes. The work showed that early yielding of beam steel shields the critical regions of weld until redistribution of stress and strain hardening occur. Analysis suggests that overmatched weld metal may facilitate shielding and it also has potential impact in the selection of the materials for the weld, beam and column. The work also indicated that continuous supplemental fillet weld on the underside of the backing bar benefits the weld of existing pre-Northridge connections, since it translates an external initial crack into a less critical internal flaw. The analysis shows that if fracture of the welds is negated through elimination of flaws in the welds and the use of tougher electrodes, other locations such as the weld cope region may be susceptible to cracking. This study will provide a basis for estimating the maximum flaw size requirements and the material properties needed to control unstable fracture in connections.

A companion study performed by Professors Sherif El Tawil and Sashi Kunnath [El-Tawil et.al. (1998)] at the University of Central Florida used the ABAQUS Computer Program to predict inelastic deformation of the connection, while examining local stress and deformation for cracking potential. The study evaluated a wide range of parameters which may affect yielding and crack potential. These include shear yielding of the panel zone, beam and column flange thickness, beam depth, web and continuity plate thickness, size and geometry of weld cope hole, relative yield and ultimate tensile stress, shear tab thickness, and span to depth ratio. The results suggest that panel zone yielding increases the potential for cracking. The ductility of the connection is insensitive to yield/tensile ratios less than 0.8, but ductility was reduced by greater ratios.

Task 5.3.2 was done by Prof. Roeder of University of Washington, and it was a coordination study of the entire connection performance research program. This project develops simple design type analytical models for all connections used in seismic design. This work evaluated more than a thousand past connection tests in addition to the tests conducted during the SAC Project. The work established relatively simple but accurate models for establishing the strength, stiffness and ductility of moment frame connections. These models provide the basis for the yield mechanism and failure mode evaluation described in a later section.

The experimental research was staged so that some studies could pass information and knowledge to other later research. Task 7.01 was completed first and developed a detailed test plan and test and instrumentation program. Most of the connection tests used the T-shaped subassemblage illustrated in Fig. 2 with a cyclic deformation pattern similar to that defined in ATC-24 [ATC-24 (1992)]. However, the test program was modified to more closely simulate the inelastic deformation demands expected during real earthquakes. Tasks 7.02 through 7.12 constitute the experimental portion of the Connection Performance Team effort. Tasks 7.02, 7.03 and 7.04 were started first. In Task 7.02, Professors Goel and Stojadinovic at the University of Michigan evaluated unreinforced welded flange bolted web connections with and without improved welding procedures. The goals were to fully understand the behavior of pre-Northridge connections, to establish the extent that improved welding will improve connection performance, and to examine the effect of panel zone yield, beam depth and material properties on the connection performance. This work clearly shows that even with notch tough electrodes, the welded flange bolted web connection is unable to achieve the required seismic ductility with the member sizes commonly used in today's steel moment frames. A modified free flange connection proposal, which shows considerable promise for seismic design, was examined as part of this study.

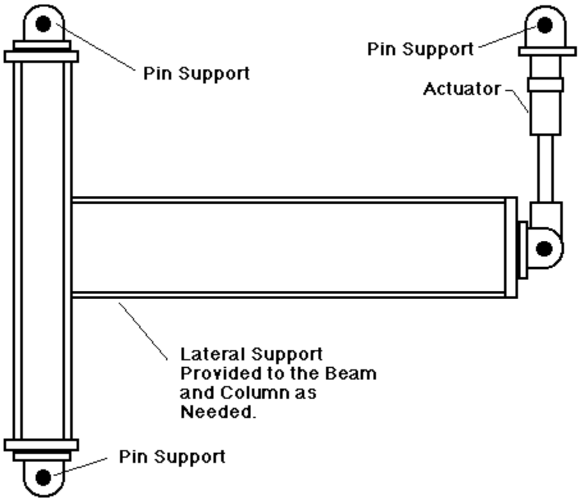

Figure 2. T-Shaped Test Subassemblage

Task 7.03 was done by Professor Leon at Georgia Institute of Technology and it evaluated the seismic performance of the bolted T-Stub connection. The work addressed the complex behavior of these connections including prying forces, connection stiffness, and the large number of failure modes and variations in bolted connections. Many of these tests are relatively simple push-pull tests as illustrated in Fig. 3, since they were economical and help to understand the complexities of bolted connection behavior. Other full connection assemblies were then tested and compared to simple test behavior. Task 7.04 focused on simple connections including the effects of the composite slab, and this work was done by Prof. Astaneh-Asl at the University of California at Berkeley. Moment frames use simple bolted web shear tab connections at most gridline intersections. These connections provide some resistance and stiffness, which are not considered in the seismic design of new buildings. This study evaluated this stiffness and resistance so that they can be used as an aid in resisting seismic loading and reducing the cost of retrofit. Specimens with composite slabs such as those used in Task 7.04 required the cruciform test subassemblage as illustrated in Fig. 4.

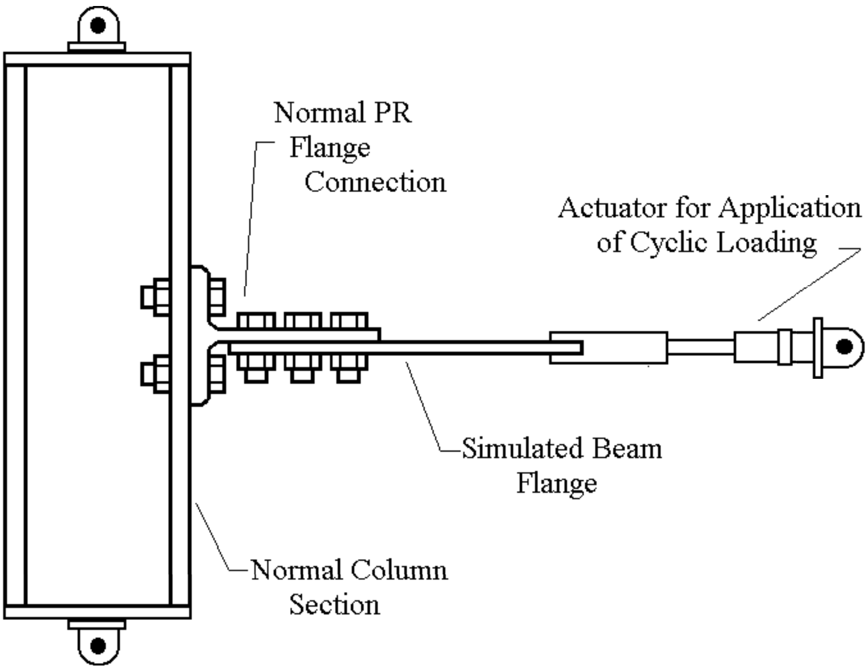

Figure 3. Push-Pull Test Configuration

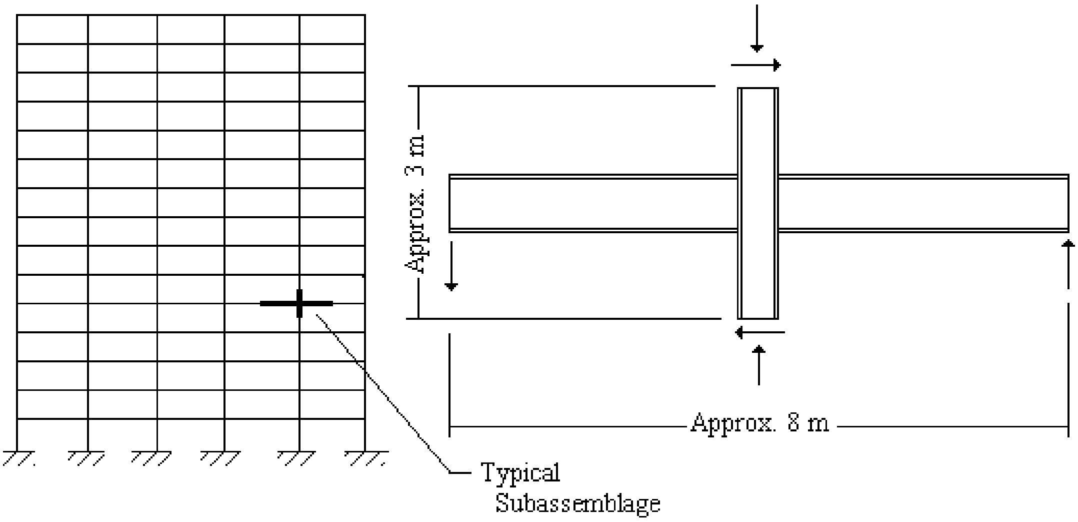

Figure 4. Cruciform Test Subassemblage

Task 7.05 was done by done by Professors Ricles and Lu at Lehigh University and this work followed Task 7.02. It examined further effects of improved welding on connection performance including the effects of cope details, continuity plates, and web welds on connection performance. This work has shown that connections with thoroughly welded webs provide increased ductility over their bolted web alternatives, but this work and most later studies were incomplete as of the date of preparation of this paper. Task 7.06 has the ultimate goal of developing a reliable design model for the reduced beam section (RBS) connection. The work is done by Professors Fry and Engelhardt at the University of Texas at Austin and Texas A & M. This study examines the significant body of test data on the RBS connection accumulated since 1994, and the test program addresses failure modes not considered in past testing. This work focus on the effects of the composite slab, lateral-torsional buckling and panel zone yielding on the RBS system behavior. Task 7.07 is being done by Prof. Anderson at University of Southern California and examined economical methods such as weld overlays for improving modest sized existing connections. Task 7.08 focuses on the welded flange plate connection and the coverplated connection. The work is done by Professor Bertero and Dr. Whittaker at University of California, Berkeley, Earthquake Engineering Research Center. It will develop a design procedure for welded flange plate connections, and it will address several failure modes which have recently come to light with the coverplated connection.

Task 7.09 was performed by Professor Schneider at the University of Illinois and it addresses bolted flange plate connections. This study examined failure modes of bolted flange connections, and will improve design models for prediction of connection behavior. This study also examined the possibility of supplementing bolted connections with friction dampers, but the size of the connections required for friction damping appeared to render this option impractical for modern steel frame construction. Task 7.10 focuses on the bolted extended end plate connections and is performed by Professor Murray at Virginia Tech University. Extended End Plates are one of the more promising bolted connection options for seismic design, but failure modes are complex and methods for predicting the behavior required improvement. A large body of past test data on this connection was available, and this test program filled gaps in this data and further developed design models for the connection.

Task 7.11 consists of confirmation testing and is being done by Professor Uang at the University of California, San Diego. All other tasks test components of frame and address specific goals from the research program, but this task is intended confirm the findings from these individual studies for global structural behavior. The tests focused on the reduced beam section and other connections which have a high probability of producing ductile connection behavior. It considered practical issues such as connections to box columns, lateral support requirements, and weak axis column bending. Task 7.12 was performed by Dr. Johnson of the Edison Welding Institute. This work provided supplemental weld material property tests in support of the overall connection test program. It will provide detailed information regarding the tensile and notch toughness properties of the weld metal and heat affected zone for various connections.

TRANSLATION OF RESEARCH INTO PRACTICE

The ultimate goal of the SAC Connection Performance research effort is the development of simple models which can be used by structural engineers to accurately predict the seismic performance of the connection. This prediction must include the strength (or resistance), stiffness and ductility of the connection. The connection stiffness is important, because it must be used by the structural engineering in the analytical models used to establish the seismic demands for the building. Some connections are so stiff that they may be treated as rigid, but many are not. The connection strength or resistance is important, because the connection resistance is combined with member resistance to establish the global structural resistance. This global structural resistance also combines with the structural analysis to establish inelastic ductility demands. The inelastic demands must be compared to the connection ductility to determine whether the connection is adequate for given seismic loading and performance requirements.

This brief discussion shows considerable interaction between the connection resistance, stiffness and ductility and the seismic demand and performance evaluations of the system. At the same time, it must be recognized that there is not a direct closed form solution for the strength, stiffness and ductility for any given connection type. These vary widely depending upon the connection type and the yield mechanisms and failure modes which control the connection behavior. Many yield mechanisms and failure modes may occur depending upon the type of connection. The extended plate connection is used as an illustration of this concept. Figure 5 illustrates failure modes which have been noted in past experiments on extended end plate connections. Plastic bending of the beams has been a fairly common failure mode, where ultimate failure may consist of ductile tearing of the beam steel, deterioration of load capacity due to excess strain, or excessive deformation due to lateral torsional or local buckling. Fracture of the bolts, fracture of the weld between the end plate and the beam, and fracture or tearing of the end plate itself are also failure modes which have been noted with the extended end plate connection. In a few cases, extended end plate connections have failed because of damage to column.

The failure mode strongly influences the ductility of the connection. This is illustrated for the extended end plate connection in Fig. 6. The figure shows the plastic rotation as a function of the beam depth for a large number of extended end plate experiments which were subjection to cyclic inelastic loading. Different failure modes are identified by different symbols on the plot. First, it should be noted that the plastic rotation achieved for connections with shallower beams is consistently much larger than that achieved with deeper connections. This phenomenon is noted for all steel moment frame connection types. This behavior is a concern with seismic design of steel buildings, because US practice has evolved toward the use of steel frames with lateral resistance concentrated into a few bays of framing with the resulting deep beam connections. The figure clearly shows that regardless of the beam depth much greater ductility is achieved when flexural yielding of the beam controls connection failure rather than fracture of the bolts or welds. Plastic deformation of the end plate may also produce significant plastic deformation as shown in the figure. Examination of the yield mechanisms for the extended end plate connection show that a number of different factors may contribute to plastic deformation including panel zone yield of the column, plastic flexure of the beam, flexural yielding of the plate, local deformation of the column flanges and webs, and elongation of the bolts. However, only flexural yielding of the beam, shear yield of the panel zone and inelastic deformation of the plate appear to have the capability of providing good seismic performance under a range of different conditions.

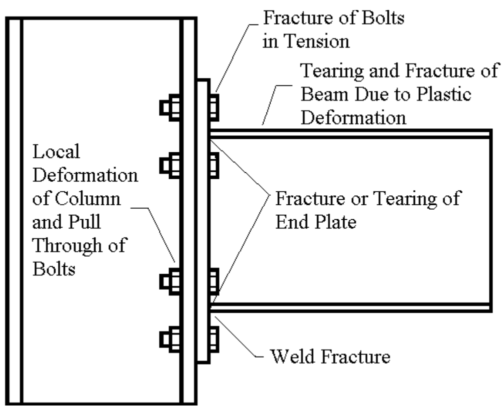

Figure 5. Failure Modes for Extended End Plate Connection

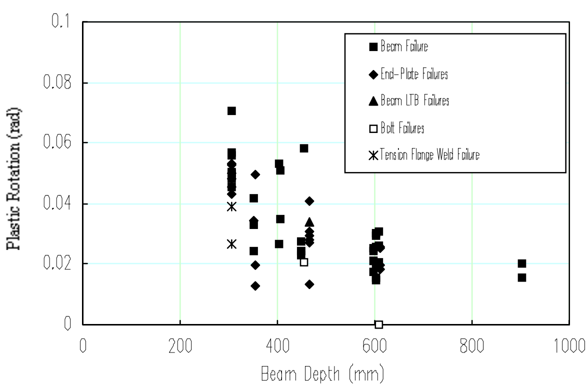

Figure 6. Plastic Rotation as Function of Beam Depth and Failure Mode for Extended End Plate Connection

Therefore, it is important that the structural engineer have reliable but simple design models to predict the connection full range of connection behavior for the full range of design possibilities with the extended end plate or any other connection. These design models must provide -

- accurate prediction of the yield mechanism and the mode of failure for each connection type,
- reliable estimation of the resistance associated with each mechanism and mode of failure, and
- and the ductility and deformational capacity expected for the connection.

These diverse goals are achieved by first predicting the resistance associated with each yield mechanism and mode of failure for the given connection. A yield mechanism, which is capable of sustaining significant inelastic deformation, provides a ductile mechanism to the connection if the resistance associated with this yield mechanism is significantly smaller than all failure modes which may produce brittle or sudden failures. The failure mode which is lower than all other failure modes is considered the critical failure mode for the connection. The ductility and deformational capacity that can be achieved is a function of the yield mechanism and failure mode for the connection, as well as the proximity of the resistances for these two conditions. In comparing failure modes and yield mechanisms, it is important to recognize that yielding or failure may occur at different locations, and thus the comparisons must be normalized to a standard location on the connection. Many different yield mechanisms and failure modes are possible for each connection type, and so a number of simple yet thorough equations are needed for each connection. These equations are obtained by analysis of the connection and comparison of predicted behavior with experimental results.

While the concept of balancing the yield mechanisms and failure modes is conceptually quite easy, the reality of accomplishing this balance is often relatively difficult. The difficulties arise because of the scatter and statistical variability noted in the past observed behavior, because of substantial coupling that occurs between some yield mechanisms and failure modes, because of limitations in our understanding and predicting connection behavior, and because of variations in the properties of materials and connectors. This can be illustrated for the extended end plate connection by noting that flexural yielding of the beam is a desirable yield mechanism, since it is capable of achieving large ductility and plastic rotational capacity. However, plastic bending of the bending also represents of significant failure mode, and the yield mechanism and the failure mode combination are coupled, since the development of the beam flexure yield mechanism will often lead to the beam flexure failure mode. While the yield mechanism and failure mode are coupled, they are still very different and are associated with very different resistance levels. Many hundreds of connection experiments have been examined, and better estimates of resistance have been developed based upon the examination of these results. It is clear that flexural yielding of the beam will normally occur when the maximum moment in beam (normally the moment at the connection of the beam to the end plate) reaches

$$My = S\ Fy \tag{1}$$

where S is the elastic section modulus of the beam and Fy is the yield stress of the steel. Actual yielding will commonly occur at even lower moments due to residual stresses in the beam and the connection. While My provides a reasonable estimate of initial yielding, it is not a good estimate of the yield mechanism moment, since the inelastic structural deformation associated with this initial yielding are negligible. The plastic moment capacity, Mp, is a bit large for the yield mechanism estimate, since significant plastic deformation has occurred before this moment is developed. This plastic capacity is estimated by

$$Mp = Z\ Fy \tag{2}$$

where Z is the plastic section modulus of the beam. Thus, examination of the experimental evidence provides a balance to how these two equations may be used together. The issue is further complicated by the fact the yield stress is precisely known in experimental research results, but yield stress is only approximately estimated by a lower bound estimate in design. To assure ductile behavior, it is necessary to assure that the moments associated with the yield mechanism are smaller than the lowest of all the failure modes regardless of the properties actually provided at the construction site. As a result, Equation 2 used with the nominal yield stress is a more conservative and realistic estimate of yield mechanism for real buildings.

As noted earlier, yielding of the beam is a desirable yield mechanism for the extended end plate connection, but it is often coupled with a failure mode associated with plastic bending of the beam. Again the hundreds of experiments were examined [Coons (1999)]. This flexural failure invariably occurs after large plastic strains in the beam coupled with local and sometimes lateral torsional buckling. It was shown that Equation 2 severely underestimates the moment at which this failure occurs, and that a much better estimate may be

$$\text{Mpfailure} = Z\,\frac{Fy + Ft}{2} \tag{2}$$

where Ft is the tensile strength of the steel. The issue is further complicated by the many other yield mechanisms and failure modes which may occur for each connection type. Failure of the end plate and the end plate yield mechanism both must consider plastic deformation of the plate, and complex yield line analyses have been proposed by various researchers to address these behaviors. These have also been analyzed, and the models which provide the best estimates of behavior have been selected. Nevertheless, there is great variation in these resistance estimates, and none of the models examined to date provide the accuracy achieved with many other behaviors. Shear yield of the panel zone and other issues also complicate the evaluation. It is not possible to provide a detailed description of the models proposed for this connection or any other connection in this short paper. However, it is hoped that this brief description provides a basic understanding of the issues addressed in the connection performance evaluation.

CONTINUING WORK

Work on the SAC Phase 2 program is nearing completion. However, considerable addition evaluation will be needed before the final results can be presented. This paper has attempted to illustrate the concerns, focus and scope of this work. However, it clearly cannot present a comprehensive presentation of the research results. Upon completion of Phase 2 of the SAC Steel Project a comprehensive Connection Performance State of the Art Report will be prepared. This report will include detailed descriptions of a wide range of steel moment frame connections. Summaries of the past research on each connection type will be made, and the discussion of the yield mechanisms and failure modes for each connection type will be provided. The ductility that has been noted in past research studies for these various yield mechanisms and failure modes will be estimated in the report, and simplified models which may be used to estimate the resistance associated with these modes and mechanisms will be discussed. Early drafts of this document are now in progress, but it cannot be completed until the research projects have been finished.

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