PROGRESS: CAST MODULAR NODES FOR SEISMIC RESISTANT STEEL FRAMES

This report contains the progress made on NSF **CMS01-** for the time period ending September 30, 2000. The report is organized according to connection concept: (I) Modular Connector; (II)Modular Node; and, (III) Post-tensioned Connecting System. A separate PDF file contains the results from this research.

Abstract

Modular connectors are being developed for use in seismic-resistant steel moment frames. The connectors are engineered specifically to meet performance requirements corresponding to optimal seismic response. The versatility in design required to accomplish this task is not readily available with traditional rolled shapes. Thus, the designs rely on advancements in materials and casting technology to create connectors specifically configured for seismic performance.

To date, three modular connection configurations have been developed: (1) a semi-rigid modular bolted connector for partially-restrained frames; (2) a cast modular node for moment-resisting frames; and, (3) a superelastic post-tensioned connecting system. Trial designs have been developed for each of the three configurations and an analytical program has been initiated. The research about cast modular node will be described in the report.

A. MODULAR SEMI-RIGID CONNECTOR FOR PRF'S

Cast modular connectors (MCs) are being developed for partially-restrained moment frames (PRF's) in high seismic zones. The connector is not restricted to the rolled shapes currently exclusively used as detail material, and thus the configuration of the MC can be engineered to produce improved behavior. These connectors will provide significant, but not full fixity to the beam-to-column joint and thus are semi-rigid connections. Analytical and experimental research is being performed leading to prototype development.

A.1. Introduction

A.1.a Concept of Modular Connector

The MC concept is intended to provide improved and reliable seismic performance in comparison to traditional bolted semi-rigid connections. The MCs will be specifically engineered to deliver superior hysteretic behavior through improved cyclic ductility and good energy dissipation. To do so, the MC cannot be limited to any disadvantages inherent in the configuration or fastening procedures associated with traditional rolled shapes. It is envisioned that a cast form of the connector will provide the versatility in geometry required.

The primary objective for improved performance in the MC is the reduction in bolt prying forces and the elimination of concentrated plastic strain regions. These results are obtained utilizing the optimal geometric configuration made available with casting technology. The MC also eliminates the potential brittle behavior associated with welding construction, achieves comparable strength and stiffness to other semi-rigid connectors at reduced strain demand, and is easily erected.

Thus, the modular semi-rigid connector preserves the beneficial characteristics of the bolted construction. However, since the connector is not restricted to the rolled shapes currently exclusively used as detail material, the configuration of the detail piece can be modified to produce improved behavior.

A.1.b. Features of the Modular Semi-Rigid Connector

The connector configuration contains three modifications which distinguish it from a traditional rolled WT shape (see Figure 1): (1) The principal flexural spans (arms) of the connector are transitioned into a variable cross-section piece; (2) The end regions of the connector are joined by a secondary element (base) parallel to the arms, and separated by a short distance (gap); and, (3) a portion of the arm protrudes to the back of the connector (compression pad).

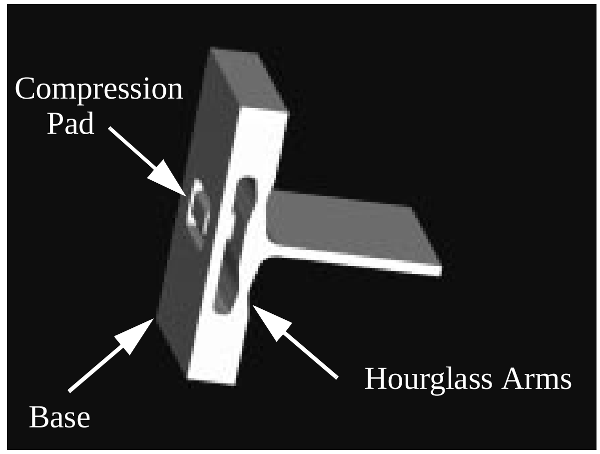

Figure 1. Modular Connector: Alpha Prototype.

Hourglass Arms

In traditional bolted tee-stubs, concentrated plastic hinges develop at the bolt line and stem. One feature of the MC is the reduction of these strain concentrations through the use of variable cross-section elements, or arms. These arms will incur plastic zones which will spread along the length of the arm. The cross-section follows the general shape of a high reverse-curvature moment gradient. However, non-negligible shear due to the small span-to-depth ratio and axial force due to catenary effects are also present in the arm.

An interaction equation [ASCE # 41] derived for a rectangular beam under moment, shear and axial force is used to estimate the capacity of each section. The optimal contour of the MC arm is determined by adjusting the cross-section dimensions to produce a constant value for the interaction equation at each section. The variable cross-section distributes plastic deformations over a larger portion of the arm. The arm thickness follows the general shape of the high reverse-curvature moment gradient. Non-negligible shear and axial force are also present in the arm. Thus, the thickness contour of the arm is determined by enforcing a constant value at each section to the following interaction equation:

$$\frac{M}{M_p}+\left(\frac{P}{P_y}\right)^2+\frac{\left(\frac{V}{V_p}\right)^4}{\left[1-\left(\frac{P}{P_y}\right)^2\right]}=C$$

where P is the internal axial force, *V* is the internal shear force, and *M* is the internal bending moment at each section of the arm as determined through finite element analysis[1]; P_y is the section axial yield capacity, V_p the section plastic shear capacity, and M_p the cross-section plastic moment. The calculations result in the hourglass shaped cross-section shown in Figure 1. Sections near mid-length are increased to prevent fracture. P, V, and M are determined at each section by the finite element results, thus the procedure is iterative. The procedure results in the hourglass shaped cross-section shown in Figure 1.

Base and Gap

A base connects the end regions of the MC to reduce prying forces developed in the bolts. By connecting each end region, the outward rotation which causes prying at one end tends to counteract the rotation at the other end. Compression develops in the base to equilibrate the catenary tension in the arm. Thus, the gap distance plays a role in the bolt force reduction. This self-

1. A process greatly facilitated by ANSYS parametric design language.

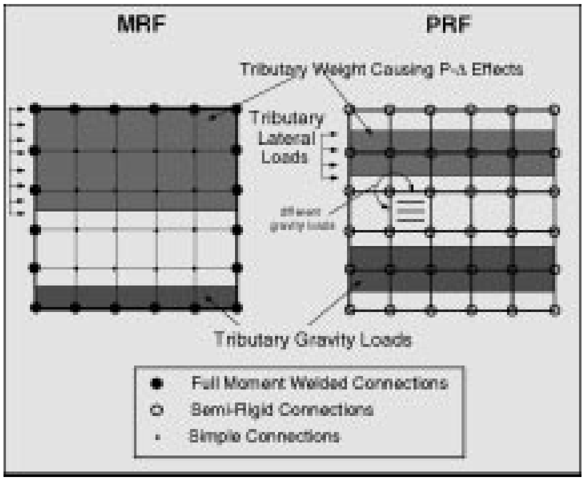

Figure 2. Plan Schematic of PR Frame.

Figure 3. Bolted Tee-Stub Detail.

equilibrating action reduces the demand on the bolt. This behavior is desirable from a capacity design approach as the brittle element (bolt) remains virtually elastic as the ductile elements (MC arms) sustain load while deforming plastically, thus dissipating energy introduced by an earthquake.

Compression Pad

During seismic response, the MC will be cycled between tension and compression loading as the beam end-moment reverses. The compression pad is provided to transmit the beam flange compressive loads directly to the column. This feature is necessary to prevent pinching of the hysteresis curves and greatly reduce the strain demand in the connection.

A.1.c. Background

The modular connector concept being developed is a modified version of a bolted semi-rigid connection, the tee-stub. Readers unfamiliar with the mechanics of the tee-stub connection as it relates to seismic detailing may find the following brief review instructive.

Partially-restrained frames (PRFs) possessing bolted semi-rigid connections are being investigated as an alternative to perimeter welded moment frames [Leon, 1999]. In this approach, energy dissipation in the PRF is anticipated to occur within the semi-rigid connections. Figure 2 shows the PR frame in plan; Figure 3a shows the tee connection schematic, 3b the tee. The advantages of this approach include the avoidance of weld failure, and cost-effectiveness. The disadvantages include a tradeoff between increased strength/stiffness and concentrated plastic strain demand, and the potential for bolt failure due to large prying forces.

To develop hysteretic energy dissipation in the bolted tee-stub connection, concentrated plastic hinges form in the tee-section (WT) adjacent to the bolt head and the outstanding leg (See Fig. 4). Assuming the outstanding leg has been designed with an overstrength against tension and block-shear limit states [Leon], the useful life of the tee-stub connection is governed by

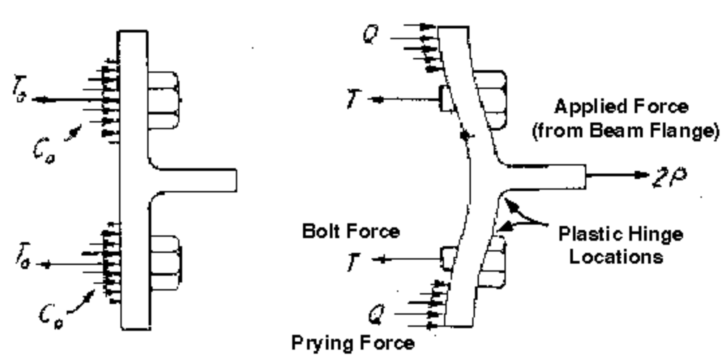

Figure 4. Prying Forces on Bolted Tee-Stub.

one of the following: (1) low-cycle fatigue created by cyclic plastic strain at one of the plastic hinge regions in the tee; (2) exceedance of the plastic strain capacity due to successive axial load increments in the bolt threads; or, (3) cyclic plastic strain demand due to flexure of the bolt shank.

The amplified axial force in the bolt threads and the flexure of the bolt shank is caused by additive forces which develop beyond the bolt, termed prying forces (See Fig. 4). Even in the absence of a bolt-controlled limit state, these actions can lead to less efficient energy dissipation from slippage due to successive losses in pretension.

A.2. Analytical Modeling

Nonlinear finite element analyses were performed to compare the response of the modular connector to the traditional tee-stub connection. A two-dimensional plane-strain finite element formulation is used to model the MC. The two-dimensional model was verified using a three-dimensional solid brick element model. The two-dimensional tee-stub model was also compared to experiments [Leon, 1999]. Both comparisons showed excellent agreement. The MC concept is new, thus no data exists to guide possible configurations. Therefore, initial development utilizes basic principles of mechanics and trial and error procedures to identify potential configurations.

A.2.a. Description of Analytical Models

A non-linear finite element model was developed to evaluate designs. In order to rapidly evaluate many configurations, a 2-D model was used to represent the cross-section of the MC. The 2D model is a plane-strain representation of a unit width of the MC tributary to a column bolt with a scaled (unit) representation of the bolt. Plane-strain quadrilateral solid elements model the angle; plane-stress quadrilateral solid elements model the bolt; point-to-point psuedo-elements capture contact between the MC, bolt and column flange. The load is applied in displacement control at the first bolt row of the MC stem.

The validity of this approach, including boundary condition effects and appropriate mesh refinement was established in related research [Sims]. The variation in configuration required in the modeling were facilitated by the parametric design language and adaptive meshing features of the commercial software ANSYS. Figure 5 shows the 2D FE model in half-symmetry representation about the stem.

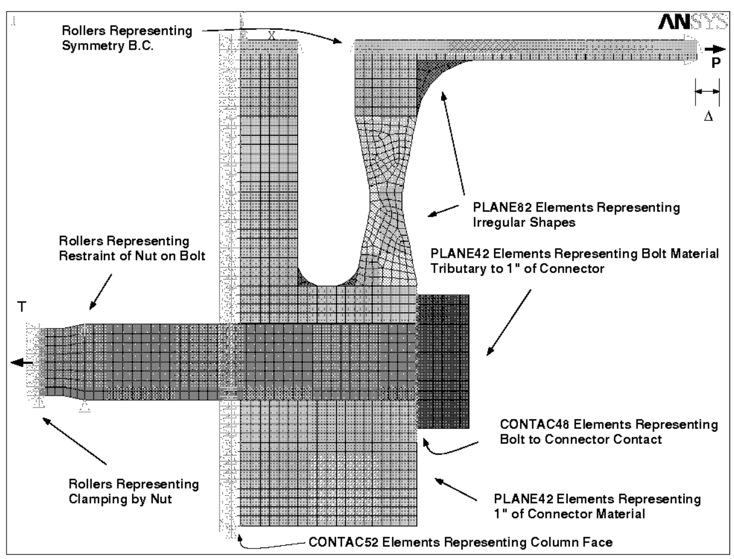

Figure 5. Two-dimensional FE model of MC.

The elements modeling the connector utilize a plane-strain formulation: linear quadratic elements are typical; higher order elements are used for accuracy in regions of irregular geometry. The model includes material and geometric nonlinearity, the latter due to changing contact surface and catenary action at large deflection. Thus, the element constitutive relationships utilize material plasticity and large deformation capabilities.

The distributed plasticity formulation inherent in the solid model is instrumental in capturing shear and shear/flexure modes present in the MC. The material model employs multi-linear kinematic hardening principles, the Von Mises yield function, and the Prandtl-Reuss Flow Equation (ANSYS Theory). Mild steel stress-strain curves are reproduced from uniaxial tension tests for the cast steel [Dynamo Steel] and converted into true stress/logarithmic strain for large deformation analysis. The stress-strain curve is represented by a detailed piecewise-linear relationship in order to accurately capture the high strain gradients within the plastic zones.

The elements modeling the bolt use a plane-stress formulation equivalent to the area of the bolt tributary to a unit width of MC. The aspects of bolt behavior captured in the model include preload, slip, bearing, axial/rotational flexibility, non-uniform pressure distributions, and fastener inelasticity. The nominal geometry of the shank region is preserved in the elements representing the bolt; the geometry of the thread region and material model is calibrated to match yield load, yield deflection and secondary stiffness from uniaxial test data of individual fasteners. Pinned supports model the restraint of the nut, thus providing a reasonable representation of the flexural rigidity provided by the bolt. The coefficient of friction is empirically selected as 0.55. The bolt pretension is developed by applying an initial interference to the angle-bolt head interface using an equilibrium step. An initial gap between the bolt shank elements and the adjacent angle elements corresponding to one-half the standard bolt hole tolerance (1/32”) allows realistic bearing response following bolt slip.

The 2D model is at an appropriate computational level for a large study and hence it was used for the parametric study. A three-dimensional (3D) models for the flange component was also created. The computationally intensive 3D model was used for verification of the internal stress and strain distributions within the 2D model. A companion experimental program of direct axial load tests of the flange component was used to verify global behavior and calibrate the boundary condition treatment of the analytical models.

A.2.b. Performance Criteria

The following results were measured and used as performance criteria for comparison between new Connectors and bolted tee-stub connectors (see Figure 6):

- overall load-displacement of the stem (P-Δ)
- maximum equivalent plastic strain in bolt shank, bolt threads, and connector (ε_{plsh},ε_{plth},ε_{plcon}) vs. energy dissipated
- bolt shank moment (M_b) and rotation (θ) at bolt head
- total bolt tension (T) and prying force (Q)

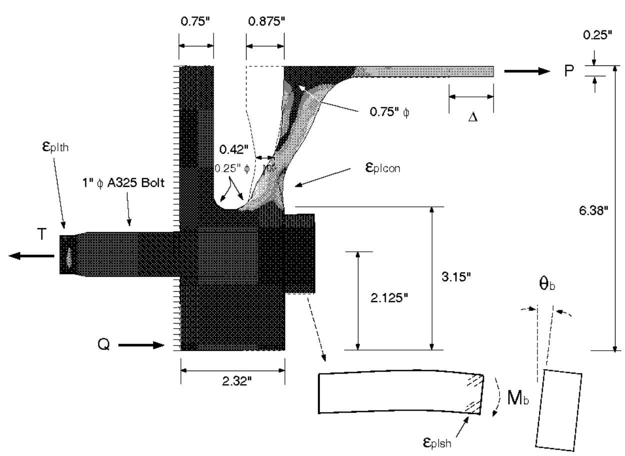

Figure 6. Schematic of MC measurements/dimensic

Bolt prying force is the resultant of the interfacial stresses integrated over the contact area. The maximum equivalent plastic strain is defined as:

$$\varepsilon_{pl} = \frac{1}{1+\nu}\left(\frac{1}{2}[(\varepsilon_1-\varepsilon_2)^2+(\varepsilon_2-\varepsilon_3)^2+(\varepsilon_1-\varepsilon_3)^2]\right)^{1/2}$$

ε_1, e_2, e_3 are the principal strains and υ is the effective plastic poisson’s ratio (0.5).

A.2.c. Model Validation

In order to validate the 2-D model, a quarter symmetry 3-D solid brick element model was created (See Fig. 7). Due to computational time and disk space, the 3-D model was only loaded into the initial stages of the post-yield branch.

This global comparison of the 2-D and 3-D model is shown in Figure 8a and deemed sufficient to validate the 2-D model. Figure 8b shows the prying forces incurred in the bolt by plotting bolt forces vs. applied load.

Figure 8b demonstrates that the internal force magnitudes in the bolt are comparable between the 2-D and 3-D models. Variation in the local stress and strain did occur across the 3-D model, however these variations were not critical enough to require further 3-D modeling.

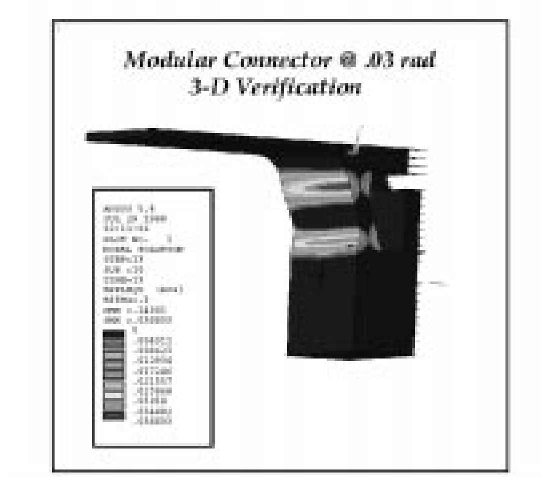

Figure 7. Equivalent Plastic Strain

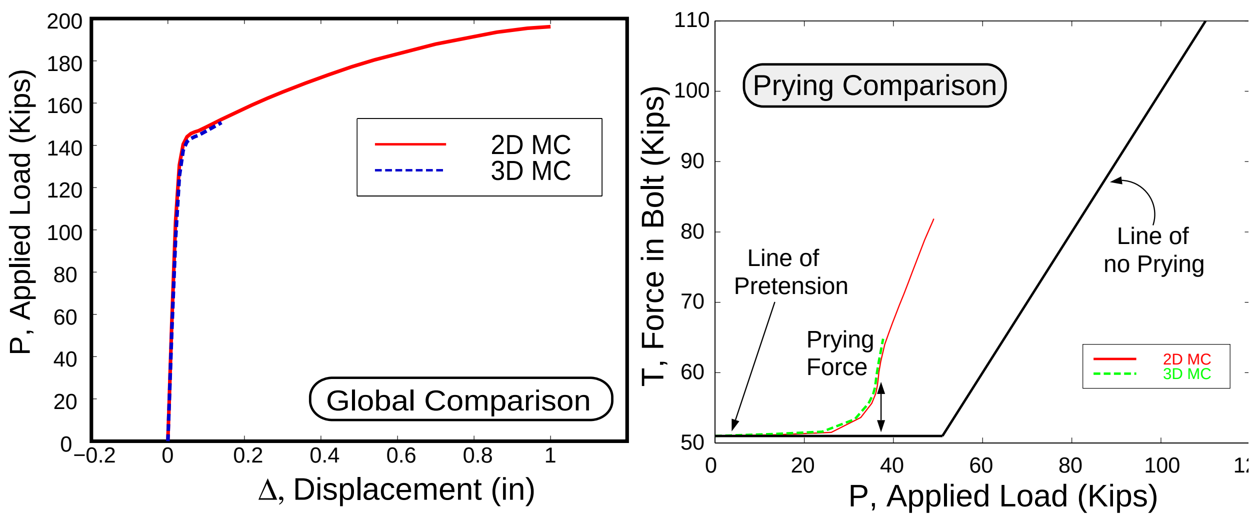

Figure 8: 2D vs. 3D MC comparison: (a) global comparison; (b) bolt response.

A.3 Prototype Development and Experimental Program

The development of the prototype connector design involved collaboration with representatives of the Steel Founder's Society of America (SFSA). Compatibility of software permitted solid models to be electronically exchanged between the research team and the SFSA members. Successive modifications were made to optimize both the structural performance and casting integrity (See Figs. 9a,b).

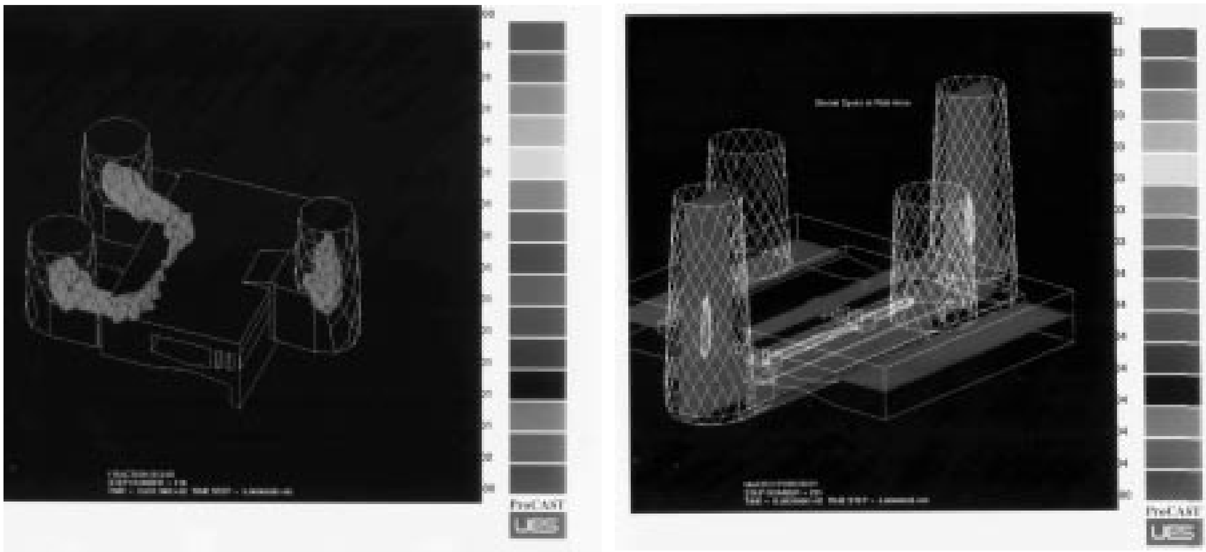

Figure 9. Casting Simulations of MC prototype: (a) solidification, (b) macro-porosity.

Once the dimensions of the alpha prototype were determined, the modular connector was created by industry partner Eagle Alloy Inc. of Muskegon MI. Shown in Figure 10 is: (a) the wood tooling used to form the mold for the outer sand shell; and, (b) the sand shell inner cores used to create the hollow section (gap) within the casting.

Figure 10. Casting Forms for MC: (a) Wood Pattern; (b) Sand Core.

The alpha prototype was cast by Eagle Alloy Inc. in November 1999. Figure 11 shows some photos of the casting process. In Figure 19a, Eagle Alloy employees fill the casting with molten steel through gates in the top of the shell. Various stages of the cooling process are shown in Figure 19b

Figure 11. Casting of alpha prototype: (a) pouring of molds; (b) cooling of molds.

The alpha prototype Modular Connector is shown in Figure 12. The outside edges are left squared rather than rounded to save time in the creaton of the original prototype,

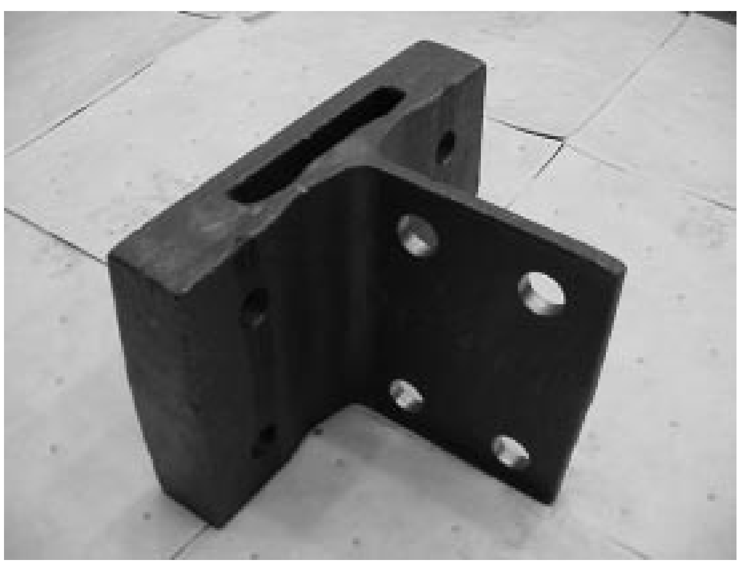

Figure 12. Alpha prototype.

The alpha prototype MC was tested using a cyclic axial-load testing fixture. This fixture approximates the loading of the flange component of a beam-to-column connection. These experiments occured at the Large Scale Structures Laboratory at the University of Notre Dame (see Sims, 2000 for a complete description of the test set-up).

Figure 13 shows the alpha prototype MC #1 prior to loading. Before Loading. Instrumentation includes LVDTS on the actuator, the heel of the connector, and the inner and outer edge of the bolt head. Two of the bolts have strain gauges on the outer and inner edge of the shank adjacent to the bolt head.

Figure 13. Alpha Prototype in testing fixture: (a) side view; (b) top view.

B. MODULAR NODES FOR SEISMIC RESISTANT STEEL MOMENT FRAMES

B.1. Introduction

Modular nodes are being developed for use in seismic-resistant steel moment frames. The connectors are engineered specifically to meet performance requirements corresponding to optimal seismic response. The versatility in design required to accomplish this task is not readily available with traditional rolled shapes. Thus, the designs rely on advancements in materials and casting technology to create connectors specifically configured for seismic performance.

B.1.a. Statement of Problem

The impetus for developing the modular nodes is the recently discovered susceptibility of steel special moment-resisting frames (SMFs) to fracture during earthquakes. These structures rely on the strength, stiffness and ductility of welded moment connections at the beam-to-column joints to create an efficient lateral-load resisting system [Popov et al, 1989]. However, more than 100 SMFs suffered fracture at these welded joints during the Northridge [Malley, 1998] and Kobe earthquakes [Watanabe et al, 1998]. The poor performance of the welded moment connections has raised questions regarding the reliable ductility of these systems. This paper describes an attempt to find solutions in an innovative way by combining aspects of modular construction, alternative manufacturing processes, and new materials.

B.1.b. Concept of Research

The underlying concept in the development of the new connections is the introduction of new configurations for the beam-to-column joint. Beam-to-column connection concepts are being developed by considering the seismic performance requirements first, and configuring the joint directly to meet these requirements. The versatility in design available through a casting process is envisioned as necessary to meet this goal. Premium seismic response is then obtained through fine tuning capabilities in the connection, without affecting the surrounding members. A key feature of these connection configurations is the removal of field welds or high strength fasteners from critical sections, thereby minimizing the prospect of brittle failure.

B.2. Background

B.2.a. Northridge Welded Connection Failures

More than 100 steel special moment resisting frames (SMFs) suffered extensive brittle fracture during the Northridge California earthquake of January 17, 1994 [Malley, 1998]. These fractures occurred in the moment connections of the beam-column joints. The fracture patterns originated from the tension zone of the full penetration welds at the beam flange and/or the surrounding heat affected zone [Housner, 1995].

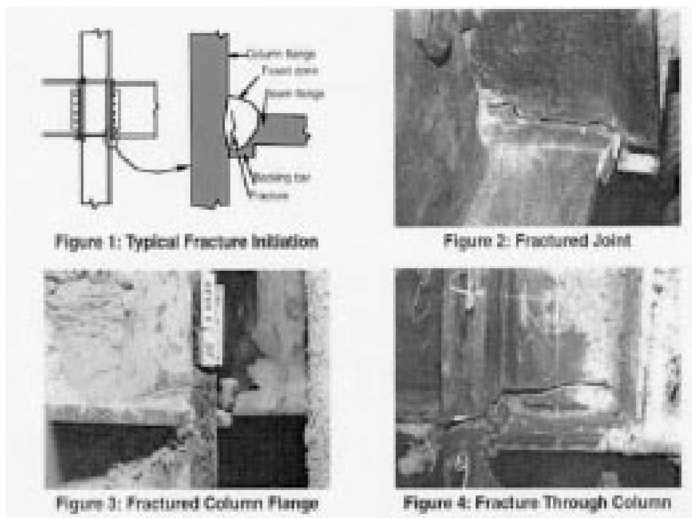

Fig.14 Fractured Welded Connection in Northridge Earthquak

The structures in question were modern and had been designed to meet the stringent seismic detailing of the building codes [Malley, 1998]. The ineffectiveness of the specifications in this particular case were attributed in part to factors relating to the low notch-toughness welds and improper welding technique [Sabol and Engelhardt, 1996]. However, investigators concluded that the elimination of welding-related construction issues was not enough in itself to guarantee prevention of these failures [Krawinkler, 1996].

B.2.b. Detrimental Features Inherent in Current Welded Connections

The full-penetration weld area is inherently less ductile than the parent material due to the possibility of incomplete fusion, porosity, slag inclusions, or develop initial cracks due to high residual stress. Additionally, the heat-affected zone (HAZ) loses much of the beneficial toughness characteristics created by structural steel's careful manufacturing process [Dexter, 1995]. High residual stresses develop due to the fit-up sequence of highly restrained joints. Furthermore, construction procedure dictates that the critical joints be connected by field welding, a process that has been shown to be significantly less reliable than shop welding due to such factors as tolerance control, weld precision, accessibility and ambient conditions.

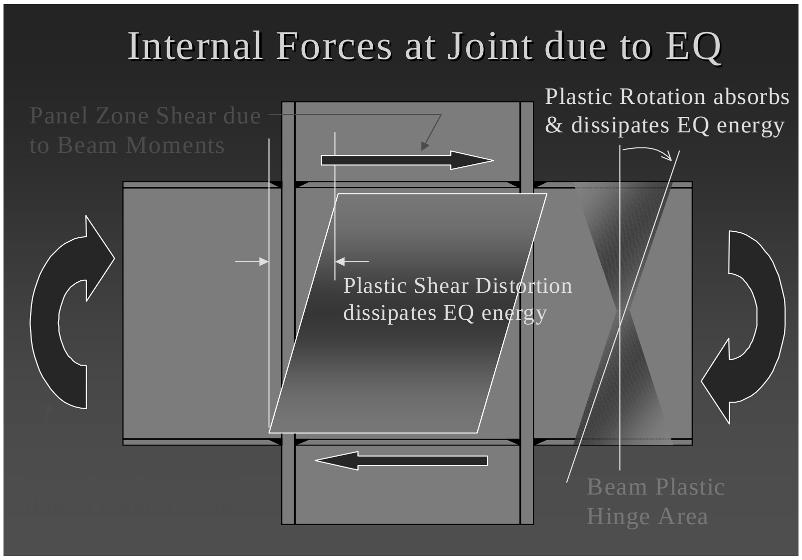

Fig.15 Internal Forces at Joint due to EQ

High triaxial restraint can exist in the connection at the beam-column interface, particularly near web elements. This condition can suppress ductile yielding in the material, leading to a brittle cleavage fracture. An expression for triaxiality can be defined as the ratio of maximum principal stress to Von Mises equivalent stress [Schafer et al, 1999]. The calculated value can be compared to fracture criteria.

B.2.c. Comment on Traditional Welded Moment Connection Configuration

The susceptibility of the full-penetration weld region to brittle failure compromises the intent of the seismic design of the SMFs. To assure a ductile plastic hinge formation at the beam end region, a reliable design would involve placing the weld at a non-critical section. This desired feature has long been recognized by designers and code-writers. These welds remain at the critical section, therefore, not due to the failure of designers to recognize capacity design dictates, or, for that matter, any structural requirement, but because of the manner in which structures are assembled.

Steel frames have traditionally been configured by member-based requirements. Rolled shapes are an economical manufacturing technique for steel members; wide-flange sections are structurally efficient in flexure-dominated frames. However, the joint is then required to meet the awkward fitting of wide-flange members. The result is a basic joint configuration that is not conducive to ductile behavior [SEAOC, 1996].

B.3. Modular Node for SMFs

B.3.a. Connection Concept

The underlying concept in the development of the modular node is the consideration of seismic performance requirements first, and the configuration of the joint directly to meet these requirements. This approach represents a significant departure from current construction. The nodes will be created from high-strength high-value steel using a casting process, thus providing a versatility in design and isotropy of material behavior not available under current procedures. The casting approach also renders the modular node viable from a manufacturing and constructability standpoint.

B.3.b. The Research Progress

B.3.b.1. The verification of 2D model

A three-dimensional finite-element model of the subassemblage utilizing solid-to-plate-to-beam element transitions is used to evaluate the accuracy of 2D model. We found that 2D model can give very good approximation of results with 3D model. So in the future research work, we decided to use 2D models to do parametric study and 3D models to study triaxiality.

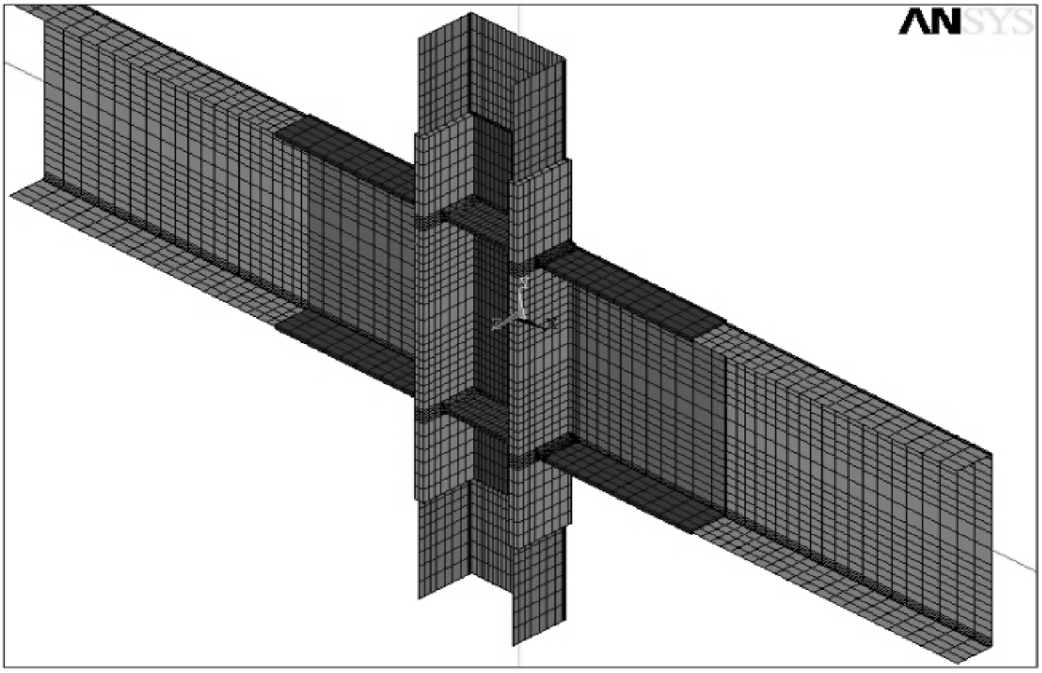

Fig. 16 Full-size 3D finite-element model of precast connection

B.3.b.2.Setting standard for new connection development

A full-size 3D model of traditional bolted model was developed to provide the standard behaviors to evaluate the new connections. To check the validation of this 3D model, we compare the results such as the distribution of stress and strain with the datas achieved by other researchers (B.W. Schafer et al). We found both results have very good similarity.

From this standard model, we studied the following fetures to be used to evaluate the new connections:

1) Panel Zone Energy dissapation;
2) Plastic Hinge Energy dissapation;
3) Column flange deformation curvature;
4) Beam shear distribution;
5) PZ shear development;
6) Maximum plastic strain in local regions;
7) Triaxiality (evaluating fracture).

C.POST-TENSIONED CONNECTION SYSTEMS FOR SEISMIC-RESISTANT STEEL FRAMES

C.1. Introduction

The failure of welded connections in steel moment frames during recent earthquakes has led to a renewed interest in bolted connections for seismic zones. While bolted connections eliminated the difficulties associated with welding, and can provide large plastic deformation capacities within detail material C.such as angles, several major deficiencies exist in such structural systems. These deficiencies include low service level stiffness, pinched behavior due to permanent deformation of detail material, and fasteners without any significant deformation capacity. The latter point has significance in that large prying forces can develop in connections, leading to overstressing of the fasteners. Either a non-ductile fastener failure or simply loss of energy-dissipating efficiency can occur due to loss of preload. One solution is to replace individual fasteners on each face of the column with a post-tensioning element across the face of the column. This paper describes such a post-tensioned connection system under development and the results of analytical and experimental studies.

C.1.a. Post-Tensioned Connection System Concept

The intent of the post-tensioned connector system is to provide the high initial rotational stiffness of welded full-moment connections, the reliable ductility of semi-rigid connections, and the self-centering capabilities of prestressed construction. The system utilizes steel connectors and special post-tensioning elements. The post-tensioning gage length across the connection is relatively short thus a special post-tensioning material is required. Several high-strength low-modulus materials have been evaluated for the post-tensioning element including aluminum, titanium and fiber-reinforced composites. Thus far, the superelastic properties of the shape memory alloy, Nitinol, have provided the optimal solution.

The post-tensioned connection system is comprised of mild-steel beam flange connectors post-tensioned across the column face by post-tensioning elements (See Figure 17).

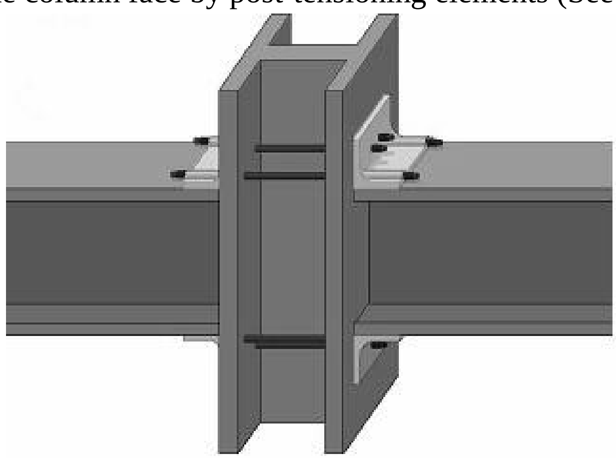

Figure 17. Schematic of Post-Tensioned Connecting System.

The connectors serve two purposes: (1) They serve as a reaction block for the post-tensioning; and (2) They provide supplemental stiffness, strength and energy dissipation to the connection system. The post-tensioning strands/bars extend across the depth of the column and twice the length of the connectors (L_{PT}). This span is several times the typical bolt grip, but an order of magnitude lower than the post-tensioning spans common in civil structures. For this reason, steel, even in its high strength forms, may not be the most efficient material for the post-tensioning strands. For this reason, a number of forms of superelastic material were evaluated, including with and without significant hysteretic capabilties. Thus, the connecting element can be designed solely as a post-tensioning element, or primarily as a dissipator, or a combination of both.

Figure 18 shows the stress-strain curves for the two types of superelastic materials evaluated in the study. Both are from the family of nickel-titanium shape-memory alloys known commonly as Nitinol. In its shape memory application, crossing of a transformation temperature causes the reverting of the material from an austensitic phase to a martensitic phase, and in this act the reversing of permanent shape deformation termed twinning. In its superelastic application, this detwinning is accomplished through stress-inducing a stable form martensite [Duerig et al, 1990]. This behavior is obtained provided the operating temperature range is above the superelastic transition temperature. Fortunately, available transition temperatures from the various manufacturing Nitinol processes cover a wide range.

The transition temperatures for transforming between the austensitic phase and martensitic phase due not coincide with the reverse process. Thus, a hysteresis is realized in the process of stress-inducing superelastic behavior (See Fig. 19a). Typical values of the key material properties identified in the figure are listed in Table 1. In an alternate cold-worked form.

Nitinol exhibits nearly hysteresis-free behavior (See Fig. 19b). Nitinol can exhibit this linear superelastic behavior to strains up to and 4%. Table 2 gives typical properties for linear superelastic Nitinol (LSNiTi) as indicated in Figure 2b.

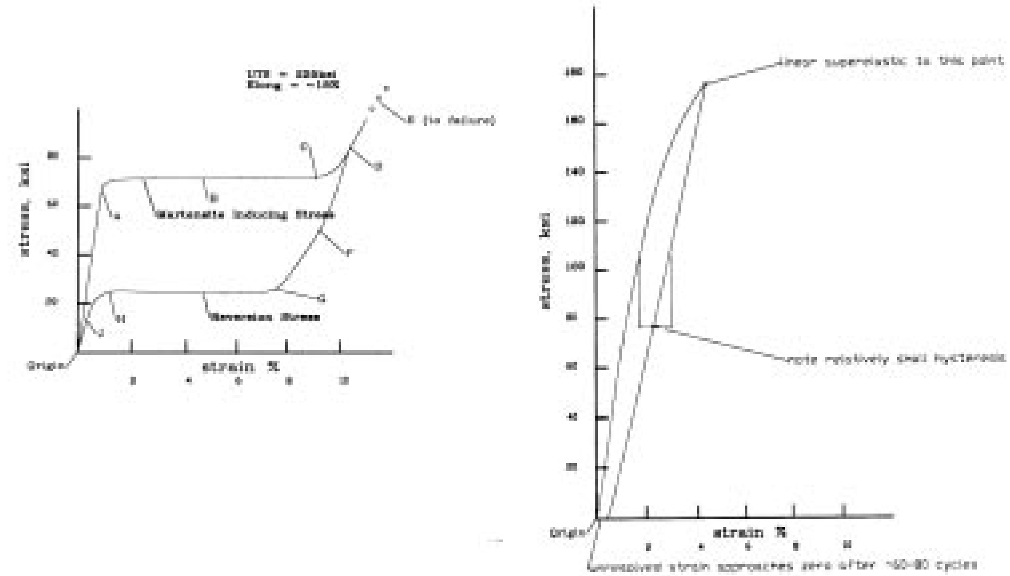

Figure 18 Nitnol Stress-Strain Characteristics: (a) Superelastic (b) Linear Superelastic.

C.1.b. Design Parameters Details

The connecting system is being developed within a performance-based engineering framework to meet requirements related to structural damage and drift. For service wind loading or low-level seismic events, the connectors are detailed to possess high inherent elastic stiffness in comparison to traditional semi-rigid connections, rendering the structural drifts within acceptable limits.

At design earthquake levels, the connectors provide stable hysteretic behavior while incurring only modest damage due to the elastic nature of the post-tensioning. For survival level events, the post-tensioning enters its superelastic ranges reducing the permanent drift in the frame.

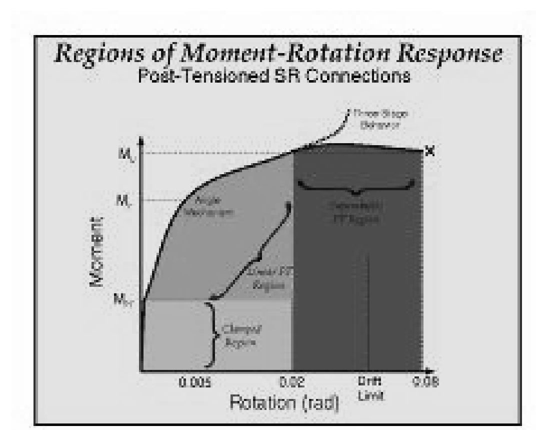

Figure 19. Schematic of Behavior Regions for PTC.

The primary parameters controlling the behavior of the post-tensioned connecting system are: (1) the ratio of mild steel connector strength to overall strength; (2) the level of pre-tension in the tendons, (3) the length of the post-tension tendons. The position of the tendons The development of the connecting system is driven by a performance-based design approach. Figure 19 shows a schematic indicating the regions of behavior provided by the connection.

C.c.2. Analytical Program

Finite element analyses are being performed to examine the hysteretic behavior of the post-tensioned system Parameters include location (vertical) of the post-tensioning element, pre-tension level, and the relative strength of the post-tensioning to the connector. A prototype design has been developed.

Currently, nonlinear dynamic analyses of structures employing the prototype design are being performed. The objective of these analyses is to provide system designs that meet accepted performance criteria for multiple levels of seismic intensity. The system has shown promising response reduction and self-centering effects.

C.2.a. Analytical Models:

Connection Model: A two-dimensional plane-strain finite element model was created to evaluate the hysteretic characteristics of the post-tensioned connector. Figure 20 shows a schematic of the model. The model employs one-dimensional line-type beam elements to model the beam and column. The mild connectors, in this case flange angles, are modeled using plane-strain solid elements. The degree-of-freedoms (DOFs) at the ends of the angles are constrained to the beam elements to obtain the appropriate kinematic relationships required by compatibility. Contact psuedo-elements are provided between the angle elements and slaved boundaries representing the contact surface on the column flange to provide the compression zone created by the post-tensioning. Figure 21 shows hysteresis curves for the connection.

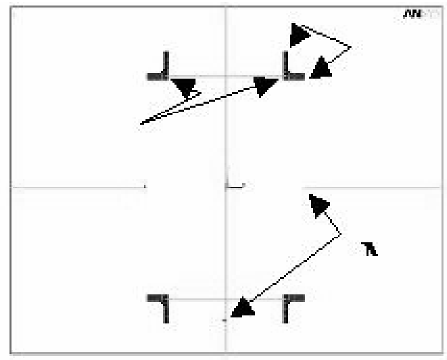

Figure 20. Schematic of 2D FE Model of PTC.

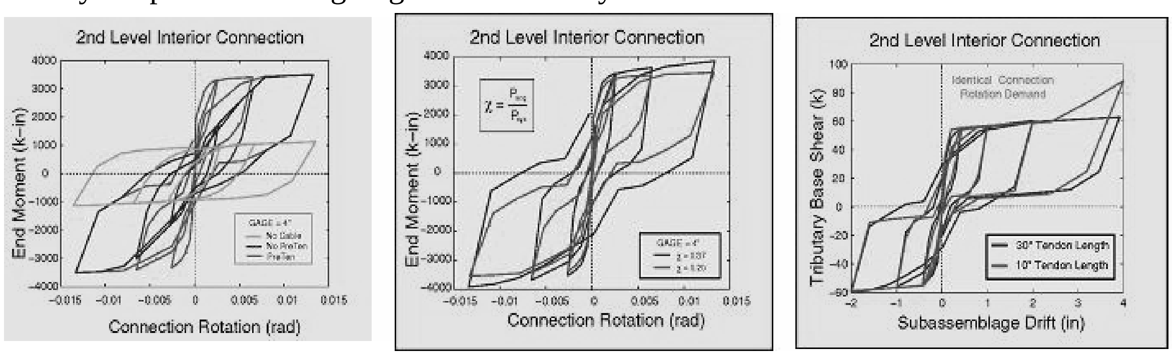

Figure21. Moment-Rotation Characterstics of PT System: (a) Level of PT; (b) PT/Mild ratio; (c) PT Length.

Structural Model: Based on the hysteretic characteristics provided in the finite element analyses, a simpler representation was created in DRAIN-2DX to examine the connection system's performance under seismic demands. The connection is modeled as translational springs slaved to the beam-column elements similar to the FE model. A pair of springs, one ideal elasto-plastic and one slip-type behavjoir were used to reproduce the pinched hystereis of bolted semi-rigid connection. The flag-shaped hysteresis of the superelastic tendons were obtained by two elements: an elastoplastic translational spring in parallel with a linearly superelastic truss element.

C.2.b. Analytical Program

A three-story LA structure developed for the SAC program was used as the prototype frame. A second, six-story frame of the same plan and floor loading was designed to meet the 1997 UBC. The contribution of the floor slab to the bare frame was ignored in the representation of the structure. The first set of records were performed on a full frame. These results were compared to the results of a single column line (a multi-story sub-assemblage). The results were reasonably close, indicating that overturning effects in the structure and axial force effects in the connecting system were not significant. Therefore, the large parametric studies were performed on the multi-story assemblage. The SAC ground motions were used in the evaluations.

C.3. Prototype Development and Experimental Program

C.3.a. Alpha Prototype Experiment

Meetings were held with industry partners to develop the prototype. Subsequently, a Nitinol strand configuration was devised that meets the specifications of the prototype design. At this time, the proper end-anchorage detail is being investigated. An experimental setup has been designed and built to test the post-tensioned connection system. The experiments occured following the anchorage design is complete.

Figure22.

Figure23.

Figure24.

Figure25.