Repairable Seismic Moment Frames with Bolted WT Connections: Part II

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ABSTRACT

A moment frame lateral load-resisting system was developed in which inelastic deformations due to seismic loading were intended to be isolated to easily replaceable WT components. Fully bolted connections were utilized to facilitate simple component installation and replacement. In Part I of this series, WT components for the moment frame system were modeled using finite element analysis. Full-scale component testing was performed to verify analytical results. Parameters taken from modeling and testing results were used to develop design provisions. In Part II of this series, an example building was designed using the provisions developed and analyzed under simulated earthquake accelerations to develop appropriate seismic performance factors. WT components designed using recommended geometric parameters resulted in desirable behavior. Recommended design equations correlated well to experimental test results. Experimental results suggested WT components designed using the recommended provisions exhibit adequate low-cycle fatigue performance and deflection capacity to be used with wide flange beams up to a nominal depth of 30 in. (762 mm). Nonlinear time-history analysis suggests seismic performance factors currently published for steel special moment frame systems are appropriate for the proposed WT moment frame system.

Keywords: partially restrained WT connection, seismic moment connection, finite element analysis, nonlinear analysis, performance-based seismic design.

This article is the second in a series describing the evaluation of the ductile WT moment frame (DWTMF) system. The system involves isolating inelastic deformations to relatively easily replaced fully bolted WT components connecting flanges of the moment frame beams to the columns. In Part I of the series, finite element analysis and experimental testing were used to develop geometric parameters that resulted in desirable component behavior and design provisions intended to ensure inelastic deformations were substantially isolated to the flange of the WT components. In this article, the design provisions from Part I are expanded for use in building frame design.

An example building was designed with partially restrained moment connections using the aforementioned design provisions. The methodology outlined in the FEMA P695 report, *Quantification of Building Seismic Performance Factors* (2009), was used to evaluate the adequacy of seismic performance factors for steel special moment frame (SMF) systems as published in ASCE 7-05, *Minimum Design Loads for Buildings and Other Structures* (2005). The objectives of the example building evaluation were to demonstrate the proposed system could be designed to adequately address the design criteria of ASCE 7-05 for a variety of building heights and to verify the seismic performance factors for steel SMF systems given within ASCE 7-05 are appropriate for the proposed system. As stated in Part I, the design criteria of ASCE 7-05 are based on collapse prevention, though limiting damage to replaceable components may inherently achieve higher performance objectives.

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EXAMPLE BUILDING MODELING

In an effort to test the feasibility of using the provisions outlined in Part I of this series in common building design practice, an example building was evaluated using the methodology presented in the FEMA P695 report, *Quantification of Building Seismic Performance Factors* (2009). The building used by AISC to provide design examples for the 2005 *Steel Construction Manual* (AISC, 2005c) was modified to one-, three-, five- and seven-story configurations for this evaluation. The footprint for this building is shown in Figure 1.

The site for the structures was assumed to reside on Site Class D soil, which, along with other seismic characteristics, results in an assumed Seismic Design Category (SDC) D classification. This is the most severe classification addressed within FEMA P695. SDC E is the result of a site in close proximity to a fault (near source resulting in a large spectral response acceleration parameter for a period of 1 s). SDC F applies to essential facility structures located on a site engaging SDC E classification. Neither of these conditions is specifically addressed by the FEMA P695 methodology.

FEMA P695 outlines the development of several structural design groups and associated archetype configurations to fully evaluate a structural system. The evaluation performed herein was limited to a plan configuration consistent with the AISC example, which is comprised of moment frames on the two perimeter column lines in the long direction (along grids A and F only for the full east–west length of the building). The seven-story configuration was the tallest that could be designed to meet the criteria of ASCE 7-05 using the proposed design provisions for the structural moment frame system. Thus, evaluation of designs with a perimeter moment frame configuration from one story through seven stories addresses one design group as described by the FEMA P695 methodology.

The FEMA methodology recommends that several other design groups be developed and evaluated as well, which may include modifying the bay spacing for the moment frames, space frame designs with alternate ratios of seismic weight-to-gravity dead load distributed to each frame, and increased building heights for more robust space frame configurations.

The connection stiffness for the system was based on component modeling and testing in combination with assumptions as to how these results relate to the connection behavior. Subassemblage tests of the beam-column connection could expose alternate factors that influence the connection stiffness and necessitate modifications to the design provisions. While FEMA P695 does provide mechanisms to utilize modeling and experimental data with significant uncertainty, the initial design of the partially restrained connection system is extremely sensitive to variations in elastic connection stiffness. Additionally, FEMA P695 has specific recommendations regarding an oversight committee to assist in determining appropriate archetypes and certainty parameters for the system. Because of the absence of subassemblage testing and the inability to adequately address an oversight committee within the schedule confines of this research, it was determined that limiting the archetypes to those described earlier was appropriate until these concerns could be addressed.

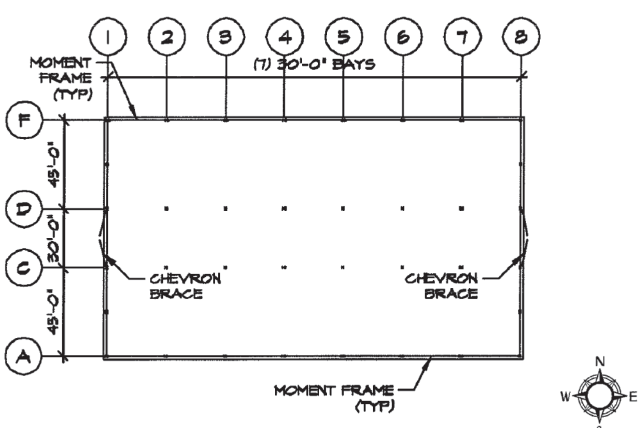

Fig. 1. Example building footprint (AISC, 2005c).

Initial Building Designs

The initial design of each archetype was developed using the Equivalent Lateral Force Procedure per ASCE 7-05 as recommended within the FEMA P695 methodology. Consistent with steel SMF systems, a response modification coefficient, R, system overstrength factor, Ω_0, and deflection amplification factor, C_d, of 8, 3, and 5.5, respectively, were used in the east–west direction. As recommended within the FEMA document, and because of the flexibility of the connections, the period was assumed to be greater than or equal to the upper limit allowed within ASCE 7-05, taken as the product of the upper limit coefficient, C_u, and the approximate fundamental period, T_a. The approximate fundamental period is a lower bound intended to produce a conservative design, while multiplication by the C_u factor is more reflective of the actual period of the system.

The maximum short-period spectral response acceleration parameter, S_S, and the spectral response acceleration parameter for a period of 1 s, S_1, for SDC D are given within the FEMA P695 document as 1.5 and 0.6 g, respectively, where g is the magnitude of the acceleration due to gravity. In addition to the maximum spectral response accelerations, the FEMA P695 methodology involves the design of archetypes using minimum spectral response accelerations, which seldom govern; however, when they do, they suggest alternate criteria may be appropriate in lower seismic design categories. Judgment was made that a seismic design using the minimum spectral response parameters was not necessary because the stiffness needed to satisfy wind loading—or the strength needed to satisfy the most stringent gravity load combination—would likely govern over the minimum seismic design. Thus, design archetypes were not developed for the minimum spectral response parameters. A seismic use group of I is assumed by the methodology resulting in a seismic importance factor, I_E, of 1.0 as defined by ASCE 7-05. A redundancy factor, ρ, of 1.0 is also assumed within the methodology and is appropriate for the example building per ASCE 7-05.

The initial design was performed using a linear-elastic analysis within RISA 3D (version 7). RISA 3D was selected because it is a common tool in consulting engineering. The Direct Analysis Method (DAM) was used as described within the 2005 AISC *Specification for Structural Steel Buildings* (AISC, 2005b). While RISA 3D can perform

Table 1. Ductile WT Moment Connection Design Aid											
Section	ϕP_n, kips	P_{pr}, kips	Equivalent Moment of Inertia for a Section of Length 1 ft, in.^{4}								
			Nominal Beam Depth, in.								
			44	40	36	33	30	27	24	21	18
WT20×99.5	196	371	3618	2999	2438	2055	1705	1388	1103	851	631
WT18×115.5	260	461	5253	4356	3543	2988	2481	2021	1607	1241	923
WT18×67.5	151	355	3444	2854	2319	1955	1621	1319	1048	808	599
WT16.5×110.5	271	468	5719	4743	3858	3254	2702	2201	1751	1353	1006
WT16.5×100.5	227	409	4505	3735	3037	2561	2126	1731	1376	1062	789
WT15×86.5	205	387	4112	3408	2771	2336	1938	1577	1253	967	718
WT13.5×73	185	368	3800	3149	2559	2157	1789	1455	1156	892	661
WT12×52	131	300	2702	2238	1818	1531	1269	1032	819	631	467
WT7×79.5	243	451	4998	4145	3371	2843	2360	1922	1529	1180	877
WT7×72.5	208	403	4080	3382	2750	2318	1924	1566	1245	961	713
WT7×66	196	392	3916	3246	2638	2224	1845	1502	1193	920	683
WT7×60	172	357	3403	2820	2292	1931	1602	1303	1035	798	592
WT7×54.5	147	319	2789	2310	1876	1581	1311	1066	846	652	483
WT7×49.5	129	291	2404	1991	1617	1362	1129	918	728	561	415
WT7×45	109	260	1952	1617	1313	1105	916	744	590	454	336
WT6×39.5	121	275	2345	1942	1577	1328	1101	895	710	547	404
WT6×36	107	260	2085	1726	1402	1180	978	795	630	485	359
WT6×32.5	93	236	1819	1506	1222	1029	853	693	549	422	312

second-order analyses and implement the stiffness reductions that may be required using DAM, it does not have the capability to perform nonlinear analyses.

The first step to the design process was to develop a rotational stiffness associated with the various WT sections that met all required design parameters (these sections were shown in Part I). For a moment frame system using partially restrained connections, the stiffness of the connection must be considered in the analysis and design (AISC, 2005b). For the linear analysis, overall elastic stiffness of the WT assembly was approximated using the equation outlined in Part I. Experimental testing demonstrated deformations from the stem-to-beam connection, which may include bolt slip, bolt bearing, and bolt shear, were relatively small in comparison to the WT flange and were sufficiently represented using the elastic stiffness equation from Part I. Panel zone deformations in the column were assumed negligible relative to the deformation in the WT assemblies. The rotational stiffness of the beam-to-column connection, K_{se}, can then be described as a function of the WT stiffness and the depth of the beam as given by Equation 1. McManus (2010) provides further detail on the development of this equation.

$$K_{se} = \frac{142Ept_{ft}^3}{3g^3}\left(d + t_{st}\right)^2 \tag{1}$$

where

t_{st} = thickness of WT stem, in. (mm)

p = 4.5 in. (114 mm)

g = bolt gage, in. (mm)

d = depth of beam, in. (mm)

Not all analysis programs have the capability of using a rotational spring at the end of the member. Therefore, a modeling mechanism was developed that could likely be used within most analysis programs. The mechanism involves placing a 1-ft (305-mm) element at the end of each beam. By properly proportioning the moment of inertia of the end element, the desired rotational stiffness can be achieved. The rotation, θ, of a section of unit length is equal to M/EI, where M is the moment applied to the section, and E and I are the modulus of elasticity and moment of inertia of the section, respectively. Recognizing the rotational stiffness, K_{se}, is equal to M/θ, K_{se} can then be expressed as equal to EI. Reconfiguration of this relationship gives the moment of inertia, I, as

$$I = \frac{K_{se}}{E} \tag{2}$$

Assuming the moment remains relatively constant along the full length of a section with length, l_s, the equivalent moment of inertia, I_{equiv}, that achieves the proper elastic rotation under a given moment is

$$I_{equiv} = \frac{K_{se} l_s}{E} \tag{3}$$

By assuming the variations in the actual depth of beams from the nominal depth has a minimal affect on the rotational stiffness, the equivalent moment of inertia for a 1-ft (305-mm) section associated with each WT configuration and various nominal beam depths can be tabulated in a design aid as shown in Table 1.

The designs for the three-, five-, and seven-story building designs were governed by the member stiffness and associated connection stiffness required to meet the drift limits prescribed within ASCE 7-05. For this reason, all frames within the design group were assumed fixed to the foundations at the base of the columns. The one-story frames utilized nominal 18-in. (457-mm)-deep beams, which were the shallowest beams deemed reasonable to still achieve an adequate shear connection. The least stiff and lowest strength WT configuration was also used, as was the lightest wide flange column with a nominal 14-in. (356-mm) flange width. This framing system still easily met the load and stiffness criteria for the one-story building with fixed column bases. Compactness criteria were not enforced in an effort to minimize the effect of these criteria on system overstrength. In lieu of traditional strong column/weak beam provisions, primary members were proportioned to develop the expected connection capacity associated with the WT assemblies. This approach is presented in further detail in the following text.

The seven-story design proved to be at the upper limit of achievable stiffness for the perimeter frame configuration using the maximum beam depth of 30 in. (762 mm). The frame elevation, or archetype design, for the seven-story building is shown in Figure 2 as an example with the WT sections used for connections at each level given in Table 2. The size and gage of the A490 flange bolts within in the WT connections were assumed consistent with those shown in Part I.

Figure 3 shows the elastic stiffness of the connection configurations presented in Table 2 in relation to the AISC limits for fully restrained and pinned connections (AISC, 2005b). The elastic stiffness of the connections range from approximately 11 EI/L to 15 EI/L; where E, I, and L are the modulus of elasticity, moment of inertia, and length of the beams, respectively. The comparison verifies the connection configurations used in the example building are classified as partially restrained connections.

Member sizes are relatively consistent with those that would likely be required of a reduced beam section special moment frame (RBS SMF) system. However, the combination of fabrication costs using automated equipment and erection costs using all bolted connections are potentially reduced over a field-welded RBS SMF system. Another

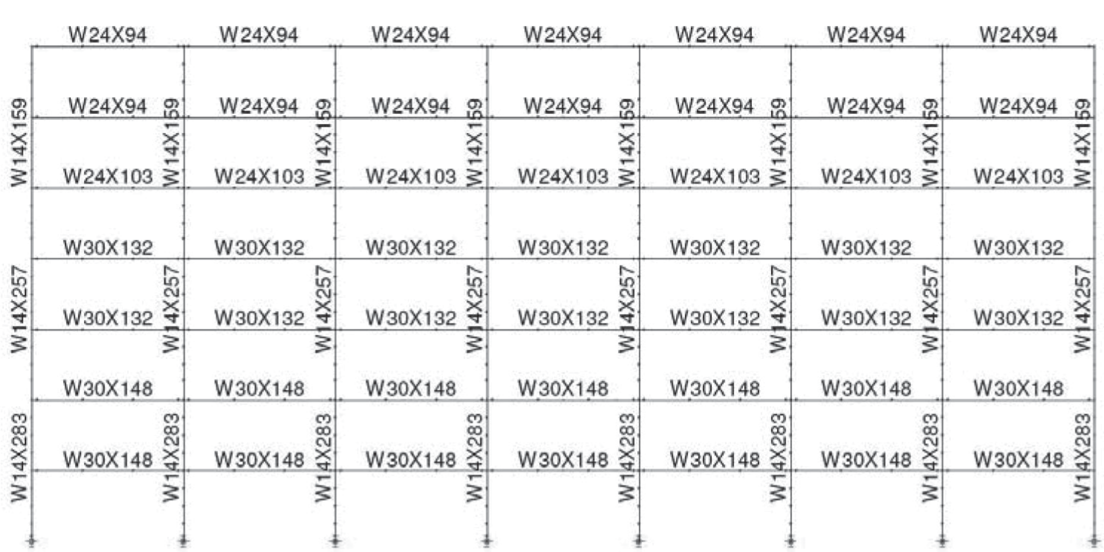

Fig. 2. Seven-story frame design.

Table 2. Example Building WT Connections				
Level	One Story	Three Stories	Five Stories	Seven Stories
2	—	WT18×67.5	WT18×115.5	WT16.5×110.5
3	—	WT18×67.5	WT18×115.5	WT16.5×110.5
4	—	—	WT18×67.5	WT18×115.5
5	—	—	WT18×67.5	WT18×115.5
6	—	—	—	WT13.5×73
7	—	—	—	WT13.5×73
Roof	WT6×32.5	WT18×67.5	WT18×67.5	WT13.5×73

advantage of a field-bolted moment frame system with a substantial percentage of moment frames is that the structure is essentially self-plumbing and self-stabilizing as it is erected.

The provisions used to design the frames follow. The LRFD capacity of the WT moment connection was taken as

$$\phi M_n = \phi P_n \left(d + t_{st}\right) \tag{4}$$

where ϕP_n is the capacity of the WT and flange bolt assembly calculated using the provisions of the 2005 AISC *Specification* including the effects of prying action (AISC, 2005b) with a WT tributary width $p = 4.5$ in. (114 mm). The load in the WT at which initial yield in the WT flange occurs, P_y, is given as

$$P_y = \frac{8pF_{yt}\left(1+\delta\right)t_{ft}^{\,2}}{3b'} \tag{5}$$

where

$p = 4.5$ in. (114 mm)

This is primarily relevant to stiffness assumptions for nonlinear modeling.

The force, P_ω, in the WT at the maximum considered deformation, Δ_{max} = 1.76 in. (45 mm), which corresponds to 0.04 rad of rotation in a nominal 44-in. (1118-mm)-deep

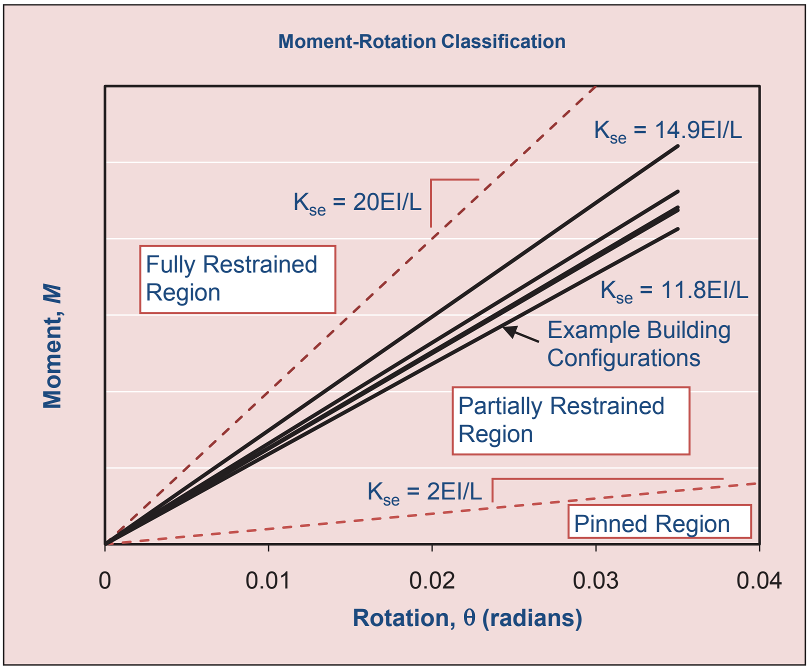

Fig. 3. Example building connection moment-rotation comparison.

beam, to be used for the purposes of determining inelastic stiffness is then given as

$$P_\omega = \omega P_y \tag{6}$$

where

$$\omega = 1.5\sqrt[3]{\frac{3.5}{b^* t_{ft}}}$$

The maximum probable force in a WT for which the limit states of stem gross yield, stem net section rupture, stem block shear, stem compression buckling, stem bolt bearing, stem bolt shear, beam flange bolt bearing, beam flange block shear, column flange local bending, column web local yielding, column web crippling and column web panel zone shear shall be designed (using an LRFD approach) is given as

$$P_{pr} = R_y P_\omega \tag{7a}$$

Because Equation 7a is consistent with the probable force in a WT with a deformation demand, Δ_{max}, based on a 44-in. (1118-mm) nominal depth beam, it is conservative for shallower beams with a lesser associated deformation demand. Alternatively, lower probable WT forces for shallower beams can be taken as

$$P_{pr} = R_y\left[\left(\frac{\Delta_{target}}{\Delta_{max}}\right)\left(P_\omega - P_y\right) + P_y\right] \tag{7b}$$

where

Δ_{target} = $d \sin\theta$, in. (mm)

θ = 0.04 rad = required rotational capacity of connection

Equation 7a was used for the example building designs herein. Local column checks were not performed for the example building designs, recognizing that these limit states can be addressed by the addition of reinforcement such as stiffener and doubler plates should a deficiency exist.

The 2005 AISC *Specification* addresses the capacity of the WT flange and the flange bolts in the same way. As a result, the available strength of the WT flange connection using the AISC provisions will always result in a deficiency because P_{pr} is larger than ϕP_n. Therefore, the capacity of the flange bolts must be checked independently to ensure the available strength of each bolt in tension exceeds the predicted tensile force in the bolt due to the expected strength of the WT flange plus the prying force, q, calculated using a force in the WT equal to P_{pr}. Proper strength of the bolts is ensured by satisfying the criterion

$$8\left(B - q\right) \geq P_{pr} \tag{8}$$

where

B = ϕr_n (LRFD) = available tension per bolt, kips (kN)

An elastic member design was used assuming a load reduction factor, ϕ, of 1.0 to ensure minimal damage to the beams and columns. The required strength of the beams and columns using appropriate load reduction factors as defined by the 2005 AISC *Specification* was also required to exceed the probable moment, M_{pr}, such that

$$M_{pr} = P_{pr}\left(d + t_{st}\right) \leq M_y \leq \phi M_n \tag{9}$$

Axial loads from the applicable load combinations should also be considered in combination with the bending load resulting from Equation 9.

Consistent with the shear requirements for reduced beam section special moment frames per the 2005 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005a), the required strength of the shear connection between the beam and column is given as

$$V_u = \frac{2M_{pr}}{L^*} + V_{gravity} \tag{10}$$

where

L^* = distance between column faces at each end of beam, in. (mm)

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1 + 0.2S$, kips (kN)

f_1 = load factor determined by applicable building code for live loads, but not less than 0.5

D = dead load, kips (kN)

L = live load, kips (kN)

S = snow load, kips (kN)

A potential advantage of the system is the components carrying shear and moment are independent of one another. Thus, if rupture in one moment connection occurs and loads ideally redistribute to other elements and connections, the integrity of the shear connection should not be compromised, having only to undergo the similar rotational and load demands as other gravity connections within the structure. This is contrary to a system intended to isolate damage to hinge locations within the primary beams, where flexural yielding and buckling may result in significant damage to member flanges, which could propagate into member webs and reduce shear capacity.

Nonlinear Analysis of Building Designs

Nonlinear analyses of the example building designs were performed under the guidelines of FEMA P695 using the analysis program SAP2000 Nonlinear (version 12). The approach of FEMA P695 involves nonlinear static pushover analysis to determine the system overstrength factor, Ω_0, and nonlinear time-history analysis under accelerations from actual earthquake records to evaluate the response modification coefficient, R. The deflection amplification factor, C_d, is then a function of the response modification coefficient and the inherent damping of the building.

The system overstrength factor for each archetype is taken as the ratio of the maximum base shear capacity, V_{max}, from the static pushover analysis to the design base shear, V, determined using ASCE 7-05. This relationship is shown schematically in Figure 4. Also of interest is the period-based ductility ratio, μ_T, taken as the ratio of the ultimate roof drift displacement, δ_u, to the effective roof drift displacement, $\delta_{y,eff}$. The period-based ductility influences the acceptable collapse margin ratio, CMR, which is described in greater detail later in this section. Tables for adjusting the CMR are provided within FEMA P695 based on an assumed period-based ductility of $\mu_T \geq 3$. The static pushover analysis can be used to verify this assumption is correct or indicate adjustment factors for the CMR need to be calculated independent of the FEMA tables. Further detail on these parameters is presented in FEMA P695.

The nonlinear dynamic analysis is used to determine the CMR theoretically through the process of incremental dynamic analysis (IDA). The process of IDA as it applies to the FEMA P695 methodology involves a nonlinear time-history analysis of an archetype exposed to an earthquake ground acceleration record, from which the maximum displacement at the roof of the archetype is recorded. The spectral intensity, or factor by which the acceleration record is multiplied, is systematically increased and new time-history analyses run until collapse, or infinite roof displacement, occurs. This process is repeated for two orthogonal acceleration records for each of 22 predefined far-field [≥6.21 miles (10 km) from the fault rupture] acceleration records from the Pacific Earthquake Engineering Research Center's PEER Next-Generation Attenuation (NGA) Database (PEER, 2006). The PEER record set provides a broad representation of acceleration records from several historic large-magnitude seismic events. An example of results from an IDA is shown in Figure 5.

The objective of the nonlinear dynamic analysis is to develop a design for each archetype using a given response modification coefficient, R, such that an acceptable adjusted collapse margin ratio, ACMR, is achieved. This is accomplished by varying the response modification coefficient and associated design until this criterion is achieved. The collapse margin ratio is defined as the ratio of the median spectral intensity, $\hat{S}_{CT}$, to the maximum considered earthquake (MCE) intensity, S_{MT}. The median spectral intensity is defined as the spectral acceleration at which 50% of the time-history analyses for a given archetype and R value result in collapse. The MCE intensity is given as equal to S_{MS} for short-period archetypes and equal to S_{M1}/T for long-period archetypes, where S_{MS} and S_{M1} are functions of short-period and 1-s period spectral accelerations described previously as defined within ASCE 7-05. The threshold between short-period and long-period archetypes is the period T_S, defined as the ratio of the spectral accelerations S_{D1} to S_{DS}, where S_{D1} and S_{DS} are taken as $S_{MS}/1.5$ and $S_{M1}/1.5$, respectively, as described in ASCE 7-05.

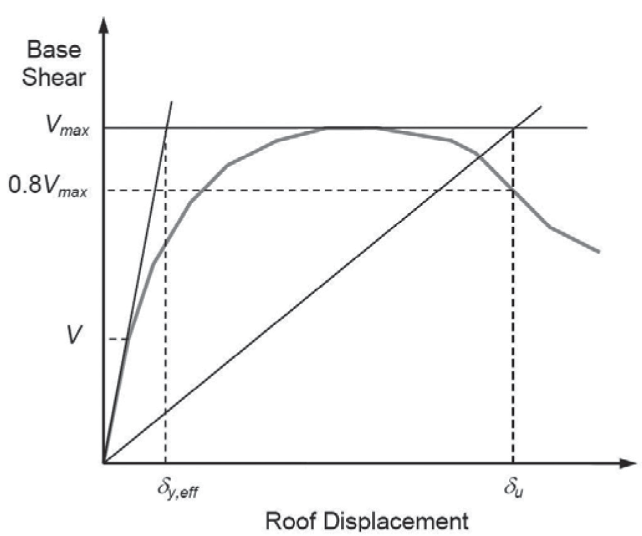

Fig. 4. Idealized nonlinear static pushover curve (FEMA, 2009).

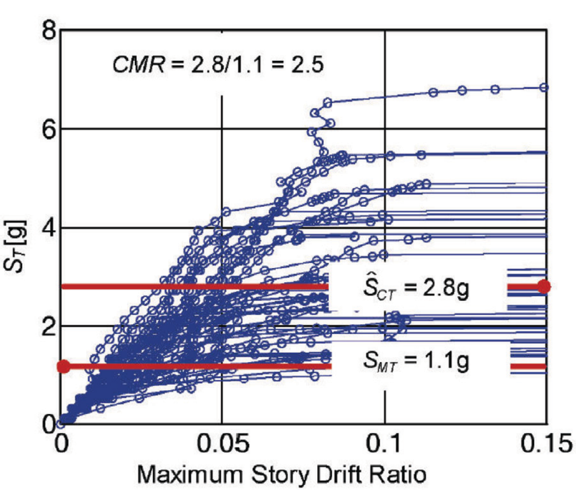

Fig. 5. Example incremental dynamic analysis results plot (FEMA, 2009).

With a target response modification factor in mind, a shortcut can be taken to a full IDA by which the desired ACMR is determined from criteria outlined in FEMA P695, and the required median spectral intensity, $\hat{S}_{CT}$, calculated as the product of ACMR and S_{MT}. With this spectral intensity known, time-history analyses can be performed at this spectral intensity and acceptance criteria for a target R value defined as less than or equal to 50% of the 44 required time-history analyses resulting in collapse. This approach avoids the need to incrementally increase the spectral intensity, thereby significantly decreasing the number of analyses required.

FEMA P695 describes the ACMR as being based on certain values of acceptable collapse probability (10% probability in consideration of the average of all archetypes within a performance group and 20% probability for each individual archetype) and the total system collapse uncertainty, β_{TOT}. The total system uncertainty is a function of the record-to-record collapse uncertainty, β_{RTR}; uncertainty related design requirements, β_{DR}; uncertainty related to the test data on which the system is based, β_{TD}; and uncertainty related to the analytical modeling on which the system is based, β_{MDL}. The individual uncertainties are combined to determine the total system uncertainty using the following equation, which can range in value from 0.275 for minimal uncertainty to 0.950 for high uncertainty.

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \tag{11}$$

FEMA P695 suggests a value of 0.4 for the record-to-record uncertainty, β_{RTR}, is appropriate for systems with a period-based ductility $\mu_T \geq 3$, as was assumed for this system. The remaining uncertainty values are determined through a subjective qualitative ranking process. For example, the uncertainty related to the design requirements, β_{DR}, is determined by ranking the design requirements into the tabulated categories shown in Figure 6. For the system analyzed herein, the completeness and robustness of the design requirements, as well as the confidence in the basis of the design requirements, were assumed to rank as "medium," based primarily on recognition that design requirements are substantially consistent with well-founded AISC standards. The result of these rankings was an uncertainty related to the design requirements of "fair"; thus, $\beta_{DR} = 0.35$. Similar processes were used to determine the rankings associated with the test data and analytical modeling of "poor" and "fair," respectively, and the associated values of uncertainty $\beta_{TD} = 0.50$ and $\beta_{MDL} = 0.35$. Further description of the ranking system and associated categories is presented in FEMA P695, and its application is outlined in McManus (2010).

Subjective rankings for each category were intended to err on the side of conservatism. Combining the individual uncertainties per Equation 11 results in a total system collapse uncertainty, $\beta_{TOT} = 0.81$, which lies in the upper range of values and suggests relatively high uncertainty for the system. The total system collapse uncertainty, β_{TOT}, represents the log-normal standard deviation of the log-normal distribution of spectral intensities with median value $\hat{S}_{CT}$, as assumed within FEMA P695. The FEMA P695 document tabulates the adjusted collapse margin ratio for 10 and 20% probability of collapse, $\text{ACMR}_{10\%}$ and $\text{ACMR}_{20\%}$, respectively, for incremental values of total system collapse

Completeness and Robustness	Confidence in Basis of Design Requirements		
	High	Medium	Low
High. Extensive safeguards against unanticipated failure modes. All important design and quality assurance issues are addressed.	(A) Superior $\beta_{DR} = 0.10$	(B) Good $\beta_{DR} = 0.20$	(C) Fair $\beta_{DR} = 0.35$
Medium. Reasonable safeguards against unanticipated failure modes. Most of the important design and quality assurance issues are addressed.	(B) Good $\beta_{DR} = 0.20$	(C) Fair $\beta_{DR} = 0.35$	(D) Poor $\beta_{DR} = 0.50$
Low. Questionable safeguards against unanticipated failure modes. Many important design and quality assurance issues are not addressed.	(C) Fair $\beta_{DR} = 0.35$	(D) Poor $\beta_{DR} = 0.50$	–

Fig. 6. Quality rating of design requirements (FEMA, 2009).

Table 3. Archetype Period and Required Spectral Intensity			
Archetype	**Modal Period, T_1, s**	**Required Median Spectral Intensity, $\hat{S}_{CT\ 10\%}$**	**Required Median Spectral Intensity, $\hat{S}_{CT\ 20\%}$**
One-story	0.67	4.23	2.91
Three-story	1.00	2.55	1.75
Five-story	1.47	1.73	1.19
Seven-story	2.01	1.27	0.87

uncertainty. The adjusted collapse margin ratios associated with $\beta_{TOT} = 0.81$ were interpolated to be $ACMR_{10\%} = 2.82$ and $ACMR_{20\%} = 1.94$. Therefore, the system is considered adequate using a response modification coefficient $R = 8$ (used for the archetype designs) if less than or equal to 50% of the time-history analyses for all archetypes within the design group result in collapse when each acceleration record applied to the archetypes is multiplied by the median spectral intensity $\hat{S}_{CT\ 10\%} = 2.82S_{MT}$. Additionally, ≤50% of the analyses for each individual archetype must result in collapse when the acceleration records are multiplied by the median spectral intensity $\hat{S}_{CT\ 20\%} = 1.94S_{MT}$. A summary of the period, T_1, resulting from a modal analysis of the initial building designs within RISA 3D, and the associated required median spectral intensities, $\hat{S}_{CT\ 10\%}$ and $\hat{S}_{CT\ 20\%}$, for each of the four archetypes is shown in Table 3.

Nonlinear Modeling Parameters

Archetypes within SAP2000 Nonlinear consisted of two-dimensional, plane-frame models with member sizes, materials and boundary conditions consistent with those determined from the three-dimensional initial building designs. Gravity loads tributary to the moment frame were applied to the frame members directly. A leaning column was included in the plane-frame model to account for the remaining gravity loads to properly account for second-order effects. A load factor of 1.05 was applied to the gravity dead load and 0.2 to the gravity live load, as prescribed within FEMA P695. These loads were applied at constant magnitude during each step of the time-history analyses.

The nonlinear analyses were performed prior to adjusting the tributary width of WT per bolt from 5 in. (127 mm) to 4.5 in. (114 mm) as described in Part I. The result is unconservative approximations of the yield and capping moments by approximately 10%. The effect of the results of the nonlinear analyses is believed to be minimal, but this adjustment should be incorporated in future evaluations of other design groups.

The elastic rotational stiffness of the WT moment connections, K_{se}, as defined by Equation 1, was modeled by assigning the value as a partial fixity rotational spring directly to the ends of the beam elements. The inelastic behavior of the connections was modeled by defining a "hinge" property, which was then assigned to the end of the beam elements. A portion of the SAP2000 hinge property dialog is shown in Figure 7. Point B and B–, as shown in the dialog, represent the moment and associated rotation at which the elastic behavior is abandoned and the hinge behavior is engaged. This point was taken as the yield moment, M_y, and associated rotation at yield, θ_y, defined as follows:

$$M_y = P_y\left(d + t_{st}\right) \tag{12}$$

$$\theta_y \approx \arcsin\left(\frac{P_y}{d}\right) \tag{13}$$

Point C and C– through E and E–, which are all approximately the same value, are taken as the capping moment, M_c, and associated rotation, θ_c, defined as follows.

$$M_c \approx \left[\left(\frac{\Delta_c}{\Delta_{max}}\right)\left(P_\omega - P_y\right) + P_y\right]\left(d + t_{st}\right) \tag{14}$$

where Δ_c is given as the WT component deformation associated with the maximum load obtained from an appropriate WT component cyclic load regimen, which was shown in the experimental testing phase of Part I to be 1.2 in. (30 mm).

$$\theta_c \approx \arcsin\left(\frac{\Delta_c}{d}\right) \tag{15}$$

The values are carried through point E and E– for convenience because SAP2000 automatically reduces the load-carrying capacity of the hinge to zero once the applied moment exceeds the value associated with point E as recommended in Part I. Values for the modeling parameters just described, and as used for the connections within the archetype models, are summarized in Table 4.

Inelastic hinges were placed in the columns at the column bases and directly above and below the beams for the static

Table 4. Nonlinear Beam-End Modeling Parameters						
WT Section	**Nominal Beam Depth, in.**	**Elastic Rotational Stiffness, K_{se}, kip-ft/rad**	**Yield Moment, M_y, kip-ft**	**Yield Rotation, θ_y, rad**	**Capping Moment, M_c, kip-ft**	**Capping Rotation, θ_c, rad**
WT6×32.5	18	61,400	183	0.00224	311	0.0667
WT13.5×73	24	227,400	489	0.00215	687	0.0500
WT16.5×110.5	30	530,000	901	0.00170	1133	0.0400
WT18×67.5	30	320,000	496	0.00155	793	0.0400
WT18×115.5	30	488,100	864	0.00177	1108	0.0400

pushover analyses. The automatic column hinge behavior available with SAP2000 was used at these locations, which is taken from the recommended provisions outlined in Table 5-6 of FEMA 356 (FEMA, 2000).

Nonlinear Static Pushover Analysis Results

The design base shear, maximum base shear and associated overstrength factor for each archetype are summarized in Table 5. The modal period in Table 5 was calculated within SAP2000 Nonlinear and differs from the initial building designs using RISA 3D by no more than 3%. This supports the validity of using a link element to represent the partially restrained connection when the ability to directly assign a rotational stiffness is not present within an analysis program.

The individual values of overstrength factor are consistent with the range discussed in FEMA P695, with the exception of the one-story archetype, which is higher. The higher value for the one-story archetype was expected because, as discussed previously, the members and connections were governed by geometry rather than load and stiffness requirements. The resultant average overstrength factor for this design group is 5.46. FEMA P695 places an upper limit of 3 on the system overstrength factor for practical design considerations, which is consistent with the largest published value for any system within ASCE 7-05. Therefore, the evaluation of these archetypes suggests a system overstrength factor, Ω_0, of 3 is appropriate, consistent with the published values for steel SMF systems within ASCE 7-05. As an example, the plot of the nonlinear static pushover curves for the seven-story archetype is shown in Figure 8. The period-based ductility for each archetype exceeds a value of 3, which validates the assumptions made in determining the adjusted collapse margin ratio, *ACMR*.

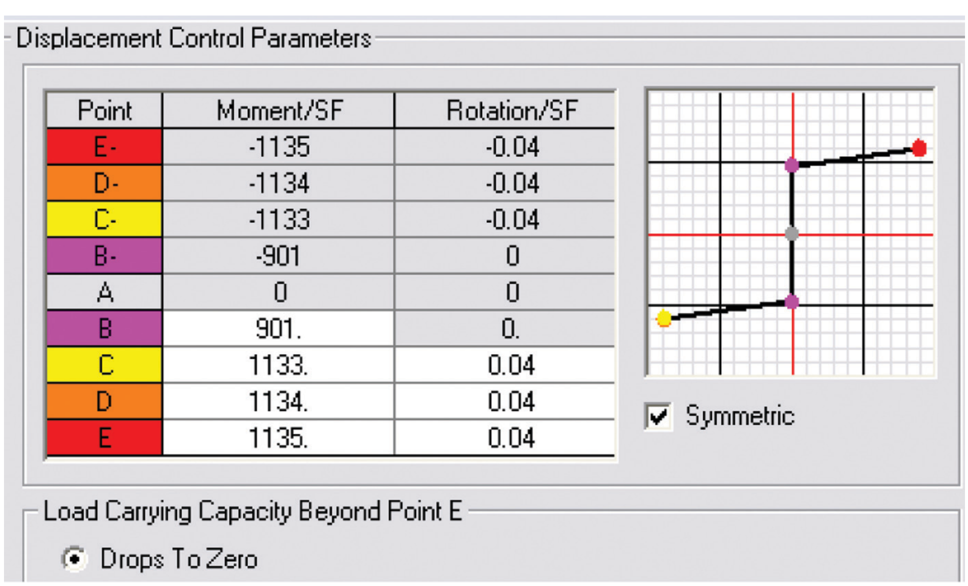

Fig. 7. SAP2000 inelastic hinge property dialogue.

Table 5. Nonlinear Static Pushover Analysis Overstrength Parameters					
Archetype	Fundamental Period, $T = C_u T_a$, s	Modal Period, T_1, s	Design Base Shear, V, kips (kN)	Maximum Strength, V_{max}, kips (kN)	Overstrength Factor, Ω
One-story	0.32	0.40	102 (454)	1110 (4940)	10.88
Three-story	0.78	0.98	561 (2500)	2132 (9487)	3.80
Five-story	1.17	1.42	693 (3080)	2692 (11,979)	3.88
Seven-story	1.54	1.98	774 (3440)	2532 (11,267)	3.27

The first inelastic hinges within each frame occurred at the beam ends as desired. Note that hinges at the beam ends indicate inelastic deformations in the WT components. At large deformations, hinges formed in the column, with the first column hinge occurring at the column base. The column base is the only location of column hinging in the seven-story archetype, whereas hinging did occur further up the column in the three- and five-story archetypes at analysis steps near collapse. Formation of a hinge at the column base occurred relatively early in the pushover analysis of the one-story frame, which suggests a serviceable hinge detail at the column base may warrant development for lower-height structures.

Nonlinear Dynamic Analysis Results

Time-history analyses for the seven-story archetype were conservatively run assuming no inherent damping such that only hysteretic damping from the inelastic behavior of the frame components contributed to the overall damping of the system. Evaluation of the seven-story archetype yielded 38 successful time-history analyses without collapse of the 44 acceleration records considered when using a spectral intensity associated with a 10% probability of collapse. Thus, 14% of the time-history analyses resulted in collapse, which was well under the 50% requirement using a response modification coefficient of 8. Because the collapse criterion was met using a 10% probability of collapse as required for the overall design group, the less stringent individual archetype criterion based on a 20% probability of collapse was inherently met. Therefore, analyses using the lesser magnitude spectral intensity associated with a 20% probability of collapse were foregone. Column hinges were not included in

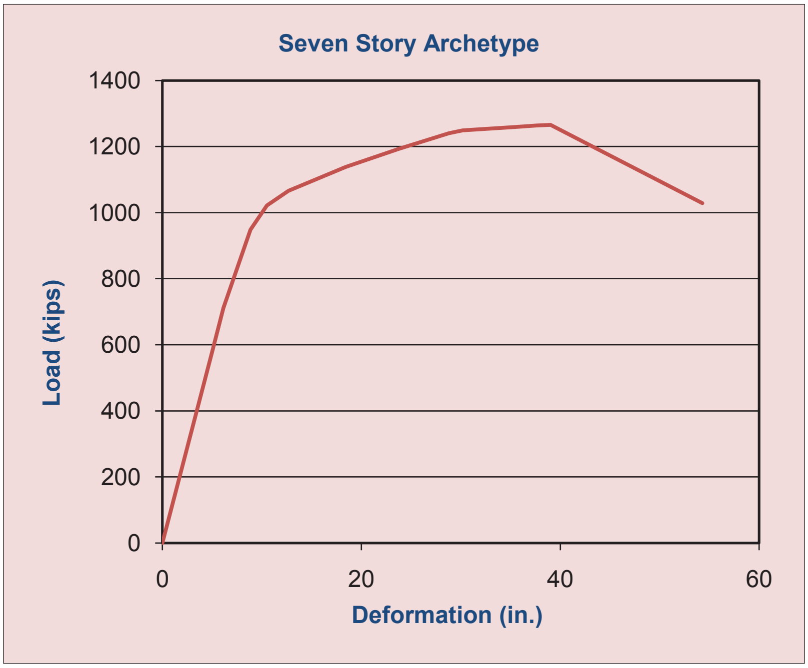

Fig. 8. Seven-story archetype nonlinear static pushover curve.

the time-history analyses because the static pushover analysis suggested the column bases were the only location where column hinging might occur—and that this would only occur at large deformations. Results for the three analyses that produced the largest deformations were examined, and only one record showed moments in the columns near the yield moment. Thus, hinging did appear to be limited to the beam-end locations as desired, which suggests a serviceable system was achieved. An example roof displacement time history record is shown in Figure 9 with units on the vertical axis in inches. These acceleration data are from the 1999 Kocaeli, Turkey, earthquake, which produced the largest roof displacement, 20.0 in. (508 mm), of any of the noncollapsed time-history analyses.

Inherent damping in the five-, three-, and one-story archetypes was assumed at 3% of critical, which is at the lower end of the 2 to 5% range discussed in FEMA P695. Column hinges as discussed in the static pushover section were included in the models for these three archetypes. Time histories were run at the spectral intensity corresponding to a 10% probability of collapse. Analyses were performed until 22 analyses were successfully completed without collapse, thereby meeting the median criteria of less than 50% of the analyses resulting in collapse. The remaining time-history analyses corresponding to a 10% probability of collapse and all analyses corresponding to the less stringent criteria of a 20% probability of collapse were foregone because the acceptance criteria were met. Evaluation of the five-story archetype consisted of analyses using 27 acceleration records, of which 22 were completed without collapse. Evaluation of the three-story archetype consisted of analyses using 40 acceleration records, of which 22 were completed without collapse. Evaluation of the one-story archetype consisted of analyses using all 44 acceleration records, of which 22 were completed without collapse. Collapse was determined by failed convergence during the analysis, and excessive roof displacement was observed in the time-history record.

Because the building designs using a response modification factor of 8 resulted in more than 50% of the time-history analyses for each archetype completed without collapse using spectral intensities associated with a 10% probability of collapse, a response modification factor of 8 is acceptable for the design group.

The FEMA P695 document suggests that for most systems the damping coefficient, B_I, may be taken as unity, which corresponds to an assumed inherent damping, β_I, of 5% of critical. This would result in a deflection amplification factor, C_d, equal to the response modification factor, R, based on the relationship $C_d = R/B_I$ presented in FEMA P695. However, it stands to reason that systems comprised of the same material, designed to carry the same load and meeting the same drift requirements would possess similar inherent damping. Because the response modification factor and system overstrength factor published within ASCE 7-05 for steel SMF systems have been shown herein to be appropriate for the ductile WT moment frame system, it is recommended the deflection amplification factor, C_d, of 5.5 used for steel SMF systems be used for the ductile WT moment

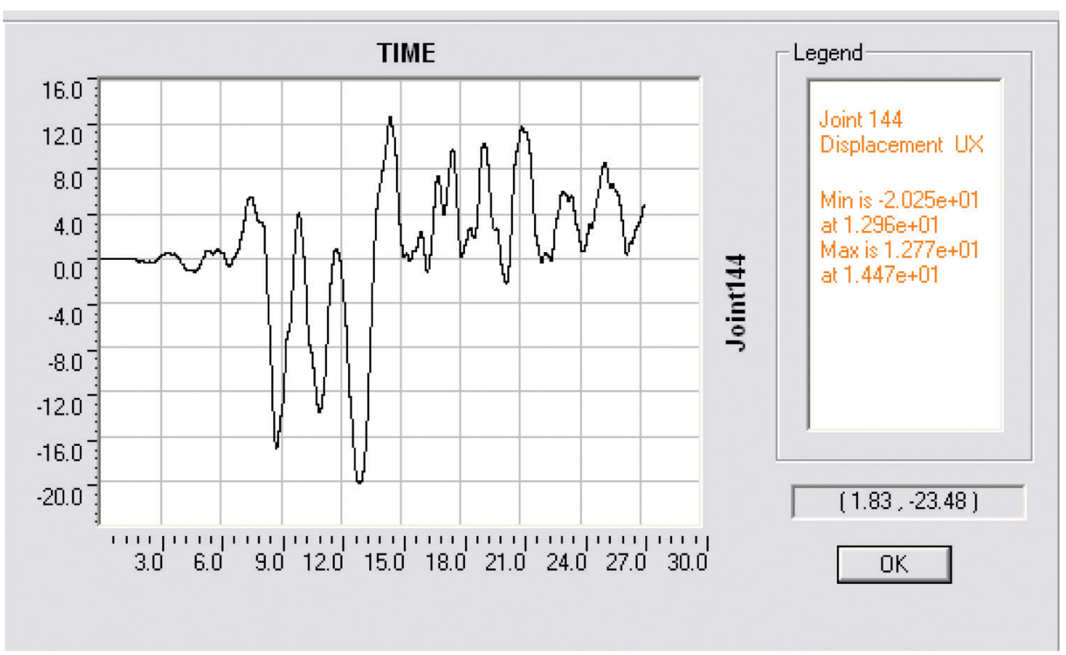

Fig. 9. Seven-story archetype response to 1999 Kocaeli, Turkey, acceleration record.

frame system as well. For simplicity, the ductile WT moment frame system could be classified as an acceptable steel SMF system.

CONCLUSIONS AND RECOMMENDATIONS

The following notable conclusions and recommendations were drawn from the example building modeling and overall evaluation of the system.

1. The ductile WT moment frame system can be designed economically for several building configurations.

2. A ductile WT moment frame system with frames located at the perimeter column lines is applicable to buildings up to 100 ft (30 m) tall, or approximately seven stories.

3. Response modification coefficient, R, system overstrength factor, Ω_0, and deflection amplification factor, C_d, of 8, 3, and 5.5, respectively, are appropriate for the ductile WT moment frame design group considered. The design group consisted of one-, three-, five-, and seven-story designs with moment frames oriented at the perimeter column lines. These seismic performance factors are consistent with steel SMF systems.

4. Further design groups should be analyzed per the FEMA P695 report, *Quantification of Building Seismic Performance Factors* (FEMA, 2009), to verify the recommended seismic performance factors from item 3 are appropriate for other framing configurations and building heights.

5. Experimental testing of beam-column subassemblages using ductile WT connection configurations recommended in Part I should be performed to verify adequate strength and rotational flexibility of bolted-bolted double-angle shear connections and to verify the elastic and inelastic rotational stiffness of the connections. This work should be performed prior to the time-intensive analytical procedures recommended in item 4.

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