INELASTIC BEHAVIOR OF MULTISTORY PARTIALLY RESTRAINED STEEL FRAMES. PART II

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ABSTRACT: The plastic-zone method of analysis described in a companion paper is used in the inelastic analysis of three multistory, multibay fully restrained (FR) and partially restrained (PR) steel frames. Full nonlinear load-deformation response, including the ultimate load and unloading path is computed. The state of connection stiffness and connection rotation at the frame ultimate load condition is discussed, and the spread of yielding within a large-scale steel frame is described in detail. The effect of loading sequence on the ultimate load capacity of FR and PR structural steel frameworks is discussed. Several of the proposed connection classification systems are evaluated within the context of the frames analyzed and designed.

INTRODUCTION

The development of advanced analytical techniques for the design analysis of fully restrained (FR) frames has received significant attention recently (Ziemian 1990; Chen and Lui 1991; Chen and Toma 1994; Liew et al. 1993; White 1992). Many of the advanced analytical techniques used in the ultimate load analysis of FR structural steel frames have implemented concentrated plastic-hinge models (King 1990, 1994; Abdel-Ghaffar 1992). Many researchers, however, have recognized limitations in concentrated plastic-hinge analysis methods and have proposed modifications (King et al. 1990; Liew et al. 1993; Attalla et al. 1995).

Partial connection restraint has long been recognized as an important element in the computation of the ultimate load for steel frames. Ackroyd (1981), Ackroyd and Gerstle (1982), Cook (1983), and Cook and Gerstle (1987) carried out an extensive research program for partially restrained (PR) frames. Deierlein et al. (1990, 1991) provided results for the ultimate load analysis of several two- and three-dimensional PR frames using plastic-hinge-based analytical models. A variety of loading sequences were studied and details related to connection rotation and lateral sway at the ultimate load condition were provided. Foley (1996) presented inelastic analysis results for a series of FR and PR frames. Spread of yielding throughout the member lengths and the state of connection stiffness at the ultimate load were provided. The complete load-deformation response, including limit-point and postcritical response, was computed.

Classification systems for FR, PR and flexible connections are needed for designers to qualify steel frame response prior to obtaining detailed knowledge of the connections. Bjorhovde et al. (1990) outlined a system to classify connections using strength, stiffness and ductility. Kishi et al. (1996) highlighted the shortcomings of the system (Bjorhovde et al. 1990) that result from neglect of the frame response. Goto and Miyashita (1995) studied similar deficiencies in the classification system (European 1990). Goto and Miyashita (1996) evaluated the classification (Bjorhovde et al. 1990) on steel frames with aspect ratios less than 1.0 using top and seat angle connections. Furthermore, they proposed a new classification system for beam-to-column connections that considers overall frame response, connection strength, and connection stiffness. The Bjorhovde et al. (1990) classification system has not been evaluated with respect to large-scale (highly redundant) frameworks likely to be found in practice. Also, the system has not been studied for frameworks containing extended end-plate connections.

The sequence of loading application can have an effect on the inelastic response of structural steel frames. Lehigh (1965) suggests that the case of proportionally applied loads always yields a lower bound estimate for the ultimate load of a structural steel frame when FR connections are used. Darbhamulla (1990) suggests that the load-carrying capacities of nonproportionally loaded beam-columns and PR nonsway frames are substantially less than that of proportionally loaded structures of the same configuration. None of the results reported in the literature quantify the effects of loading sequence on the inelastic ultimate load response for large-scale PR unbraced steel frames.

The main objective of this paper is to contribute to the current knowledge pertaining to the inelastic response of PR and FR steel frames. In addition, a finite element and solution algorithm discussed in the companion paper (Foley and Vinnakota 1999) is used to study connection behavior and evaluate classification systems with respect to connection strength, initial stiffness, and ductility for large PR frameworks. The effect of loading sequence on connection behavior, yielding, and ultimate load capacity of large, steel FR and PR frames is quantified. Plastic-zone analysis allows direct implementation of residual stresses, eccentricity of axial loads with shift in the elastic core of the cross section, yielding along the member length, cross-sectional yielding, and storage of stress/strain data for use in assessing the tendency for local buckling. This technique has been neglected in recent years due to the perceived computational expenses. Today's computers have nearly eliminated the computational expense of the plastic-zone method, and this paper intends to illustrate the computer's use in the ultimate load analysis of large-scale practical frameworks.

PARTIAL CONNECTION RESTRAINT AND CLASSIFICATION

The typical multilinear connection model used in this study is shown in Fig. 1. The connection stiffness beyond the connection rotation, denoted as θ_3, is assumed to be 1.0×10^{-7} kN·m/rad. A negligible hardening behavior is assumed within the connection response. Bjorhovde et al. (1990) define a pa-

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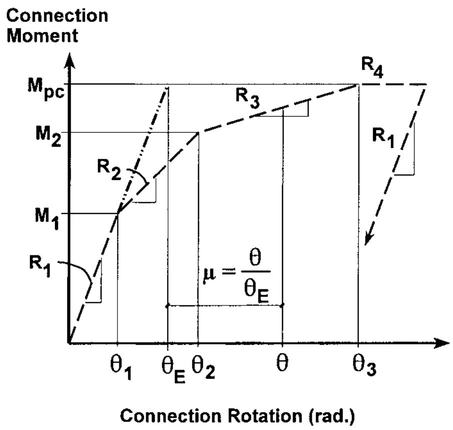

FIG. 1. Multilinear Connection Model with Rotational Demand Parameter μ

rameter used to establish the required ductility of a beam-to-column connection. This parameter is given by

$$\mu = \frac{\theta}{\theta_E} \quad (1)$$

Bjorhovde et al. (1990) also include a constant α that relates the flexural stiffness of a short beam of length αd_b to that of the connection. The flexural stiffness of a beam with equivalent length L_{be} can be written as

$$K_b = \frac{EI_b}{L_{be}} = \frac{EI_b}{\alpha d_b} \quad (2)$$

The parameter α is found by setting the beam flexural stiffness given by (2) equal to the connection initial stiffness, as follows:

$$\alpha = \frac{EI_b}{R_1 d_b} \quad (3)$$

Bjorhovde et al. (1990) describe an FR connection as having an α value of less than 2.0 and a minimum ultimate moment capacity equal to $0.7M_{pb}$.

This method of classifying connections has much potential in advanced analysis applications for PR frames since many connection-specific details are not required for a design analysis. Simple multilinear connection models can be constructed using the parameters in the classification system for the advanced design analysis of a PR framework. A detailed connection design can then proceed with the goal of achieving connection behavioral characteristics similar to those assumed in the design analysis.

Goto and Miyashita (1996) have presented results that indicate connection classification models should include a measure of overall frame response. The connection classification system proposed by Bjorhovde et al. (1990) lacks input from frame analysis. To this end, Goto and Miyashita (1996) propose the following parameter:

$$\eta = \sqrt{\left(\frac{\gamma_{FR} - \gamma_{PR}}{\gamma_{FR}}\right)^2 + \left(\frac{\Delta_{PR} - \Delta_{FR}}{\Delta_{FR}}\right)^2} = \sqrt{\eta_{lf}^2 + \eta_s^2} \le 0.07 \quad (4)$$

The 0.07 limit can be attained assuming that $\eta_{lf} = 0.05$ and $\eta_s = 0.05$. Therefore, the change in ultimate load factor and lateral sway between the FR and PR connection configurations cannot be greater than 5% for the connection to be considered fully restrained. Eq. (4) can be adjusted to account for differences at a prescribed load level through consideration of only η_s with appropriate sway deformations and the 5% limit.

Connection ductility is also important when classifying steel beam-to-column connections as FR, PR, or flexible. With the help of Fig. 1, the required connection rotation at the ultimate load condition can be written as (Bjorhovde et al. 1990)

$$\theta_R = \mu \left[\frac{M_{pc}}{R_1}\right] = \mu\theta_E \quad (5)$$

Bjorhovde et al. (1990) indicate that the connection ductility parameter μ can be set equal to 6. One of the objectives of this paper is to evaluate the use of $\mu = 6$ in defining connection ductility within the realm of this connection classification system.

LIVE LOAD REDUCTION PROCEDURE

Live load reduction is extremely important in the design of multistory, multibay frames. A common problem with the usual methods (ASCE 1995) is that they are based on manual ''column-take-down'' analysis. Therefore, these techniques do not translate well via matrix structural analysis. If advanced analysis techniques are to progress into the realm of practical framework design, methods of incorporating live load reduction are required to ensure that practical member sizes are consistent with current design procedures.

A proposed method (Ziemian 1990; Ziemian and McGuire 1992) allows implementation of live load reduction in matrix structural analysis procedures. The method is based on the fact that the influence area of a girder framing into a column at a given level is different from the influence area for this column. The difference in forces, which results from these influence area differences, is handled via upward compensating forces being directly applied to the beam-to-column joints. These upward forces are considered live loading in the structural analysis and the appropriate load factors are applied; roof live loading (snow) is not reduced.

FRAME STUDY

The three multistory, multibay structural steel frames chosen for this study are shown in Figs. 2–4. It should be noted that

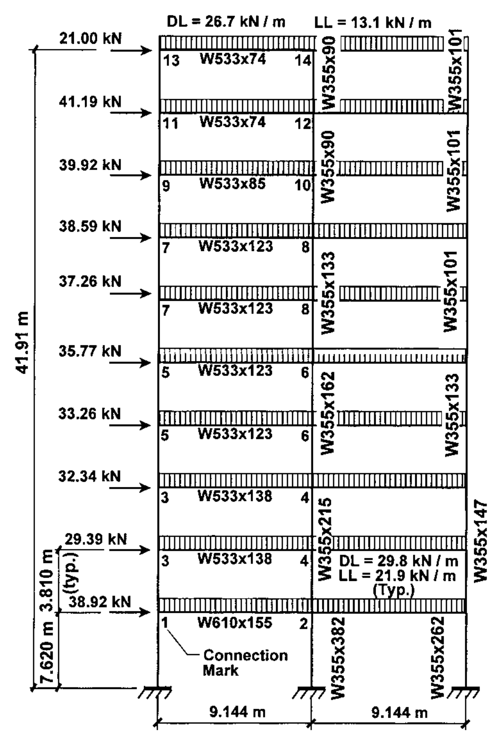

FIG. 2. Configuration of Frame 1 Analyzed in Present Study

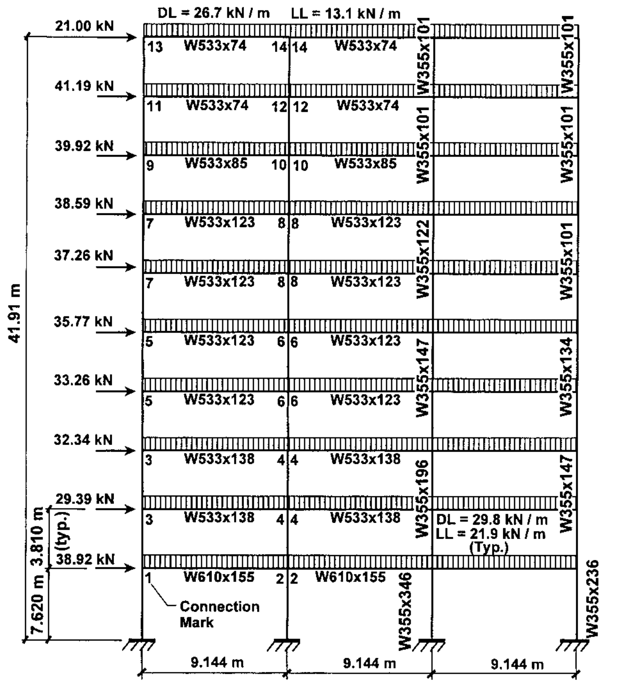

FIG. 3. Configuration for Frame 2 Analyzed in Present Study

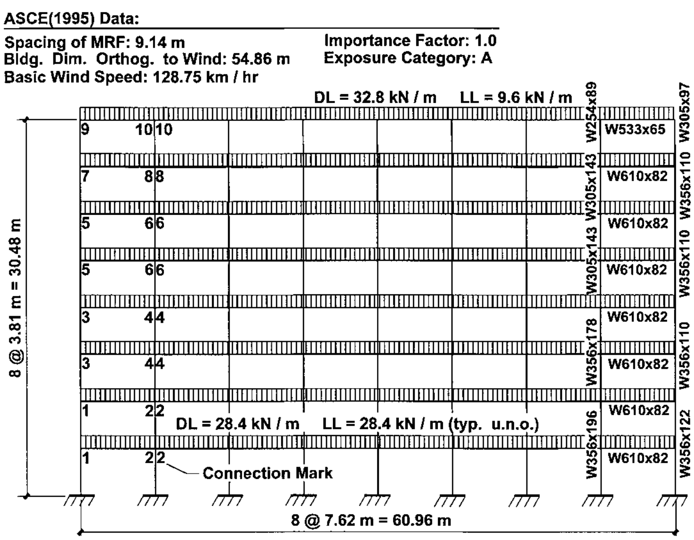

FIG. 4. Configuration of Frame 3 Analyzed in Present Study

roof live loading is considered to be snow loading for all frames. Frames 1 and 2 were designed for the service lateral (wind) and gravity loads indicated in the figures. Live load reduction was not considered for these frames. The lateral loading is taken from a similar frame found in Hoffman et al. (1996). Frame 3 was designed for the service gravity loading shown, and the lateral (wind) loading was based (ASCE 1995) using the data provided in the figure. Leaning columns and cladding loads were not considered. Live load reduction was considered in the design analysis for Frame 3 using the procedure described.

The design analysis for all frames consisted of a first-order elastic analysis assuming FR connections with the member design following (*Load* 1993) appropriate amplification of first-order forces. The following load combinations were considered: (a) 1.2DL + 1.6LL + 0.5SL; (b) 1.2DL + 1.6SL + 0.5LL; and (c) 1.2DL + 0.5LL + 0.5SL + 1.3WL. Lateral (service) deflection was limited to h/400 at both overall and interstory levels, while vertical service live load deflections were limited to L/300. Bending moments obtained from the structural analysis (assuming FR construction) were used to size unstiffened extended end-plate connections using the strength procedure given in Foley and Vinnakota (1995). Reduction factors were not used to establish consistency of member and connection strengths with the inelastic analysis to follow. No consideration was given to the PR nature of the unstiffened end-plate connections in the serviceability checks. Consideration of the extended end-plate connection as being FR is consistent with historical design-office practice. In this study, if the connection strength could not be achieved through modifications of end-plate thickness, bolt diameter, bolt pitch, etc., the column size was adjusted so that no web or flange stiffening in the connected column was required. There was an increase in column size due to connection considerations for the exterior and interior columns at the uppermost stories in Frame 3.

Multilinear PR connection models for the extended end-plate connections were created using strength and initial stiffness computations proposed by Foley and Vinnakota (1995). The nonlinear three-parameter connection model used to create multilinear curves is given as follows

$$M = \frac{R_i\theta}{\left[1 + \left(\frac{R_i\theta}{M_{pc}}\right)^{\beta}\right]^{1/\beta}} \tag{6}$$

where $\beta = 0.95$. Strain hardening within the connection moment-rotation curve was neglected. Nonlinear moment-rotation curves were visually fit with multilinear representations and the resulting connection data for the frames analyzed are given in Tables 1–3. Virtually all the values of α for the present connection models for Frames 1, 2, and 3 are less than 2.0. Furthermore, the ratio of connection to connected beam plastic moment capacity varies significantly for the four frames. Many of the frame connections have moment-capacity ratios around 0.70, but the majority are below this value. Therefore, it can be said that the connections are fully restrained for serviceability considerations based on initial connection stiffness (assuming the connection rotations corresponding to service loading are below θ_1), and they're partially restrained for strength considerations using the classification system proposed by Bjorhovde et al. (1990).

ANALYSIS RESULTS

Frames 1, 2, and 3 were subjected to a plastic-zone analysis using 20 finite elements to model each column and 30 finite elements to model each beam. Sixty-six elemental fibers were used to model each member cross section. No initial imperfections were assumed in the plastic-zone analysis. All computer runs performed in this study were carried out on a Pentium Pro 200 MHz personal computer with 64 MB of RAM. Run times for the frames ranged from 20 min to 2 h (wall clock). Two loading sequences were used to determine the inelastic load-deformation response and ultimate load of the frames:

$$\text{Sequence I: } (1.2DL + 0.5LL + 0.5SL + 1.3WL)\gamma_{I} \tag{7a}$$

$$\text{Sequence II: } 1.2DL + 0.5LL + 0.5SL + (1.3WL)\gamma_{II} \tag{7b}$$

Frame 1 (h/w = 2.29) is the first to be discussed. The load-displacement response curves for this frame are given in Fig. 5. Values of η for roof/second level are 0.36/0.23 and 0.33/0.31, for sequences I and II, respectively. Therefore, this frame is PR according to the analysis-based connection classification proposed by Goto and Miyashita (1996). The proposed system (Bjorhovde et al. 1990) suggests the frame is PR considering connection strength and FR considering connec-

TABLE 1. Connection Parameters for Frame 1

	Stiffness Parameters (×1,000 kN·m/rad)			Rotation Parameters (×0.001 rad)							
Connection (1)	R_1 (2)	R_2 (3)	R_3 (4)	θ_1 (5)	θ_2 (6)	θ_3 (7)	M_{pc}/M_{pb} (8)	α (9)	θ_R (×0.001 rad) (10)	I: θ_{act} (×0.001 rad) (11)	II: θ_{act} (×0.001 rad) (12)
1	4,629.0	199.19	2.2417	1.373	4.789	50.00	0.44	0.98	7.216	22.14	24.10
2	6,237.9	269.76	1.5609	1.250	4.167	50.00	0.53	0.73	6.450	17.30	19.71
3	4,038.3	143.26	1.0843	1.319	5.139	50.00	0.53	0.84	7.705	20.73	23.17
4	5,330.2	125.43	4.0378	1.111	4.167	50.00	0.60	0.63	6.535	18.96	21.81
5	4,308.7	195.35	1.2062	1.111	4.167	50.00	0.59	0.70	7.050	11.18	12.78
6	4,229.5	169.57	1.0810	1.199	5.000	50.00	0.60	0.71	7.304	8.56	10.30
7	4,141.7	149.00	1.0753	1.042	4.028	50.00	0.51	0.73	6.340	3.11	2.02
8	3,137.7	110.39	1.0930	1.319	5.139	50.00	0.52	0.96	8.532	3.15	2.10
9	3,038.5	114.91	1.0755	1.250	4.931	50.00	0.69	0.64	7.695	0.99	0.80
10	3,917.6	102.59	1.0886	0.890	4.041	50.00	0.69	0.50	5.968	0.014	0.41
11	3,850.2	103.76	0.8648	0.959	4.144	50.00	0.80	0.43	6.004	0.32	0.29
12	3,709.9	86.40	0.3401	0.903	4.444	50.00	0.80	0.45	6.231	0.29	0.27
13	3,201.9	101.11	0.8269	0.959	4.041	50.00	0.69	0.52	6.227	0.33	0.23
14	3,345.6	86.75	0.8338	1.041	3.836	50.00	0.80	0.50	6.909	—	0.11

TABLE 2. Connection Parameters for Frame 2

	Stiffness Parameters (×1,000 kN·m/rad)			Rotation Parameters (×0.001 rad)							
Connection (1)	R_1 (2)	R_2 (3)	R_3 (4)	θ_1 (5)	θ_2 (6)	θ_3 (7)	M_{pc}/M_{pb} (8)	α (9)	θ_R (×0.001 rad) (10)	I: θ_{act} (×0.001 rad) (11)	II: θ_{act} (×0.001 rad) (12)
1	2,640.3	123.28	1.7527	1.901	5.845	50.00	0.56	1.72	16.101	21.15	23.27
2	4,782.6	160.70	1.4421	1.288	4.167	50.00	0.68	0.95	10.794	18.24	20.72
3	3,111.8	141.16	1.4507	1.462	5.077	50.00	0.61	1.09	11.380	16.96	20.26
4	3,851.6	169.20	1.327	1.364	4.167	50.00	0.67	0.88	10.098	13.65	18.66
5	2,902.6	163.81	1.698	1.515	5.227	50.00	0.63	1.04	11.174	7.14	8.24
6	2,847.7	110.45	1.374	1.615	5.769	50.00	0.67	1.06	12.113	5.95	6.85
7	2,867.5	120.40	0.9824	1.343	5.149	50.00	0.56	1.05	10.055	3.01	1.34
8	3,221.8	121.83	0.9731	1.250	4.722	50.00	0.54	0.93	8.629	2.52	1.18
9	3,405.2	97.28	0.6759	1.042	4.444	50.00	0.76	0.58	7.563	0.92	0.66
10	3,610.9	115.63	0.8398	0.972	4.167	50.00	0.68	0.54	6.381	0.86	0.61
11	3,110.6	118.57	0.6708	1.111	4.097	50.00	0.84	0.54	7.803	0.80	0.49
12	3,110.6	110.83	0.8410	1.111	4.236	50.00	0.76	0.54	7.060	0.82	0.50
13	2.712.7	99.48	0.5768	1.111	4.167	50.00	0.71	0.61	7.563	0.45	0.26
14	2,666.8	95.34	0.8423	1.319	4.306	50.00	0.78	0.62	8.451	0.51	0.30

TABLE 3. Connection Parameters for Frame 3

	Stiffness Parameters (×1,000 kN·m/rad)			Rotation Parameters (×0.001 rad)							
Connection (1)	R_1 (2)	R_2 (3)	R_3 (4)	θ_1 (5)	θ_2 (6)	θ_3 (7)	M_{pc}/M_{pb} (8)	α (9)	θ_R (×0.001 rad) (10)	I: θ_{act} (×0.001 rad) (11)	II: θ_{act} (×0.001 rad) (12)
1	2,488.2	120.33	13.462	0.950	4.700	20.00	0.52	0.81	7.356	23.21	28.71
2	4,019.9	220.58	15.268	0.900	4.070	20.00	0.78	0.50	6.830	20.79	24.81
3	2,809.7	243.74	9.890	0.970	4.630	20.00	0.60	0.72	7.517	16.15	21.38
4	3,496.3	219.47	13.581	0.950	4.240	20.00	0.73	0.58	7.349	15.83	20.47
5	3,095.2	181.44	10.645	0.870	4.700	20.00	0.61	0.65	6.937	6.40	3.31
6	3,928.0	212.55	13.735	0.900	4.660	20.00	0.77	0.51	6.900	6.61	2.50
7	1,336.2	91.54	7.5378	1.210	5.150	20.00	0.36	1.51	9.483	4.34	—
8	3,725.2	264.31	14.711	0.930	4.140	20.00	0.77	0.54	7.276	3.01	0.45
9	626.5	73.35	5.313	1.210	4.360	20.00	0.26	2.25	14.063	1.71	—
10	1,203.2	122.32	11.435	1.500	5.070	20.00	0.58	1.17	16.334	1.12	0.23

tion initial stiffness. The percentage increase in lateral frame sway (η_s) at the design load level (γ = 1.0) for sequence I is 9.6% and 14.6% for the second and roof levels, respectively. The percentage increases in sway for sequence II are the same. Therefore, the unstiffened extended end-plate connection must be considered as PR in the analysis used for design. The maximum rotation at the ultimate load for each connection is given in Table 1. The state of connection stiffness at ultimate for this frame is given in Fig. 6. Connections in the lower stories are taken into the third stage of stiffness prior to hinge formation. It can be seen that the ductility parameter proposed (Bjorhovde et al. 1990) underestimates the connection rotations at ultimate for the lower stories.

The difference in the ultimate load between the two loading sequences for the FR-connected frame is approximately 11%. In the case of PR connections, the difference between load cases is less than 4%. Therefore, a case of nonproportional loading, with gravity loads constant, results in small differences in the ultimate loads achieved when compared to proportional loading for this slender frame with a limited number of bays. The difference in ultimate loads for the FR versus PR configurations range from 22% to 27% for sequences I and II,

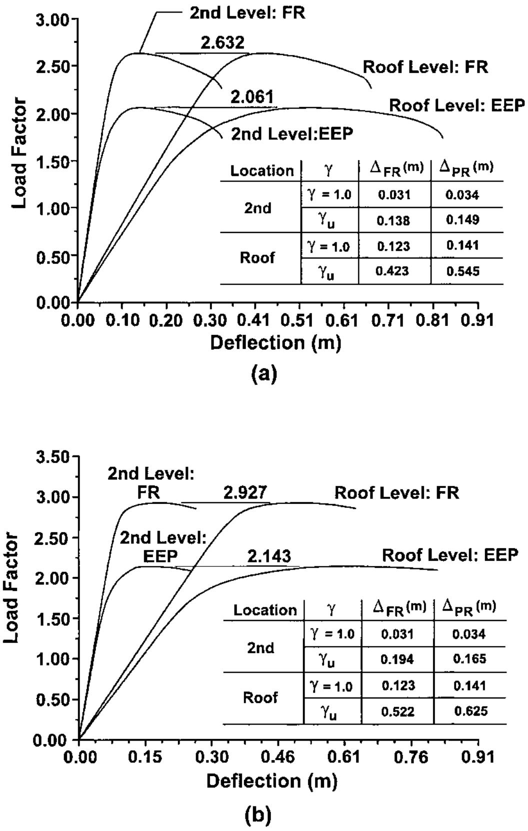

FIG. 5. Load-deformation Response for Frame 1: (a) (1.2DL + 0.5LL + 0.5SL + 1.3WL)γ_I; (b) 1.2DL + 0.5LL + 0.5SL + (1.3WL)γ_{II}

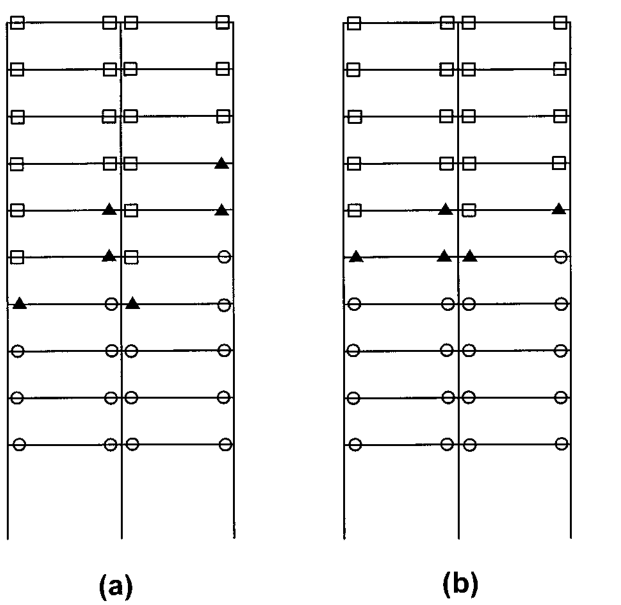

FIG. 6. Connection Stiffness at Ultimate Load Condition for Frame 1; (a) (1.2DL + 0.5LL + 0.5SL + 1.3WL)γ_I; (b) 1.2DL + 0.5LL + 0.5SL + (1.3WL)γ_{II} (● Stage 4, ○ Stage 3, ▲ Stage 2, □ Stage 1)

respectively. This difference is significant and it illustrates the reasons why PR connection behavior is important for assessing frame capacity.

The load-deformation response of Frame 2 (h/w = 1.53) is given in Fig. 7. The shape is virtually the same as that computed for Frame 1, indicating similar overall structural ductility for both frames. Values of η for (roof/second level) are

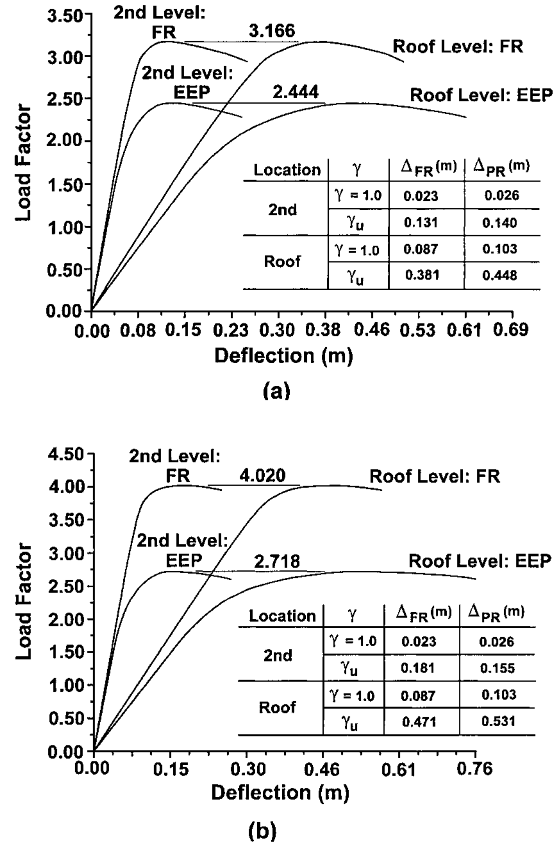

FIG. 7. Load-Deformation Response for Frame 2: (a) (1.2DL + 0.5LL + 0.5SL + 1.3WL)γ_I; (b) 1.2DL + 0.5LL + 0.5SL + (1.3WL)γ_{II}

0.29/0.24 and 0.35/0.35 for sequences I and II, respectively. Values for η_s for sequences I and II at load factors of 1.0 are 13% and 18.4% for the second and roof levels, respectively. Table 2 illustrates the maximum connection rotations obtained via the advanced analysis corresponding to the ultimate load condition. The inadequacy of predicting the connection rotation with μ = 6 is again illustrated. The PR connection configurations decreased the frame ultimate load from 13% to 32% from levels achieved by FR connections for sequences I and II, respectively. Therefore, as with the previous framework, the PR nature of the connections is important and should be considered at the time of design.

The load-deformation response for Frame 3 (h/w = 0.50) is given in Fig. 8. Very high ultimate load factors were attained for both sequences of loading in this framework. The magnitude of these factors will be discussed in greater detail later. Using the connection classification system (Goto and Miyashita 1996), values of η for sequences I and II at the roof level are 0.54 and 0.26, respectively. Therefore, this framework should be considered as PR with respect to the ultimate load. At the design load factor of 1.0, η_s at the roof level is approximately 0.16 for sequences I and II. Thus, as with the previous two frameworks, the extended end-plate connection must be considered as PR at both ultimate load and design levels. The low moment-capacity ratios in the lower stories found in Table 3 for this framework would lead to similar conclusions.

The connection rotations at the ultimate load condition for each sequence are given in Table 3 and the stiffness stage for all connections at ultimate are given in Fig. 9. Load application sequence I results in many of the connections in the lower stories rotating into the fourth stage of connection stiffness (''hinge'' formation). In load sequence II, virtually all of the

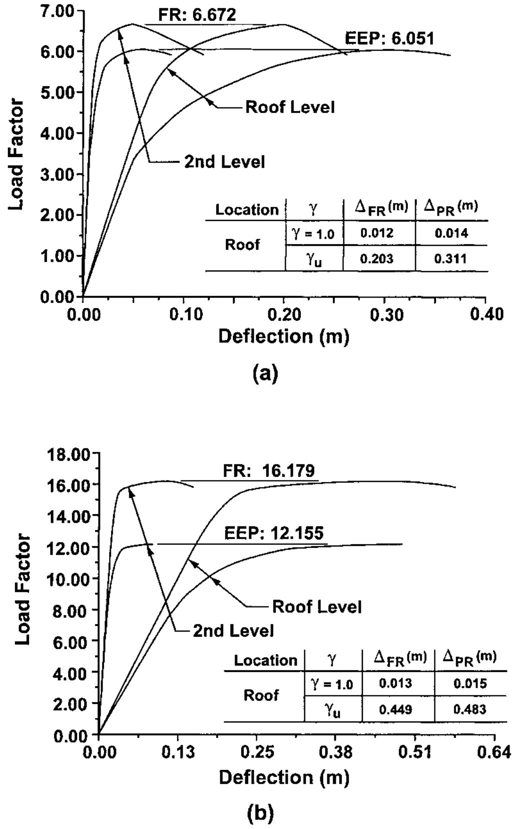

FIG. 8. Load-Deformation Response for Frame 3: (a) (1.2DL + 0.5LL + 0.5SL + 1.3WL)γ_I; (b) 1.2DL + 0.5LL + 0.5SL + (1.3WL)γ_{II}

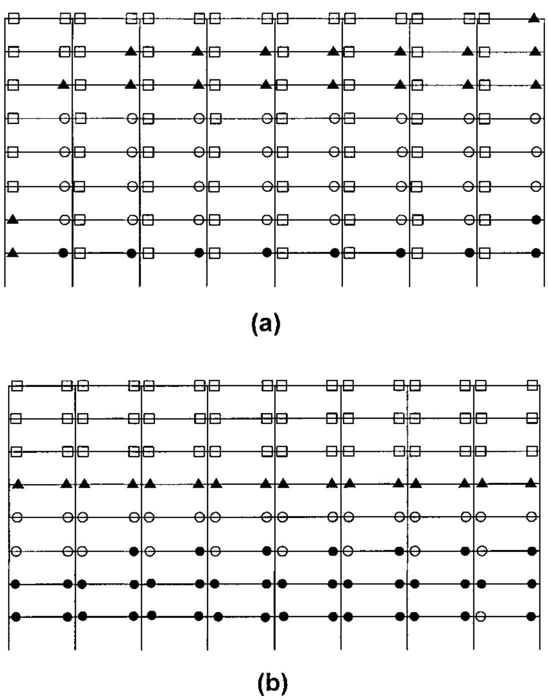

FIG. 9. Connection Stiffness at Ultimate Load Condition for Frame 3; (a) (1.2DL + 0.5LL + 0.5SL + 1.3WL)γ_I; (b) 1.2DL + 0.5LL + 0.5SL + (1.3WL)γ_{II} (● Stage 4, ○ Stage 3, ▲ Stage 2, □ Stage 1)

lower-story connections are taken into the final stage of stiffness. In this case, the extremely large lateral loads required to achieve frame failure cause the significant yielding in the connections at ultimate load. The rotational demand analytically obtained at ultimate is well beyond that predicted by Bjorhovde et al. (1990).

The spread of yielding at ultimate as a percentage of the initial cross-sectional area for Frame 3, and subjected to load sequence I, is given in Fig. 10. Significant yielding within the column members occurs along the entire member length within the lower stories. As indicated, there is little plastic-hinge-type yielding in the lower two stories of this structure for both FR and PR connections. Further, there is moderate yielding near the midspan zone of the lower-story beams in the case of FR connections, and there are many locations where the leeward beam ends exhibit hinge-type yielding. The PR-connected frame results given in Fig. 10(b) again exhibit nonplastic-hinge-type behavior in both the beams and columns. The end yielding within the beams has been relieved significantly as a hinge is now forming within the connections. Also, yielding within the midspan zone of the beams is more significant in the PR frame due to connection rotation. Table 3 shows that the rotational demand in the PR connections for loading sequence II increases significantly more than that for sequence I. However, one must not discount the large load factor and the resulting increased lateral loading contributing to the increase in rotational demand on the connections.

In general, the ultimate load factors obtained for all frameworks were beyond the magnitude considered optimal (γ_u = 1.0). Therefore, the connection demands found through advanced analysis must be tempered with the knowledge that the load levels are beyond the optimal. Furthermore, the loading combination that controlled individual member design played a key role in contributing to the high load factors for the two loading sequences. For the most part, gravity load combinations controlled the sizes of the beams and the corresponding connections for Frames 2, 3, and the upper stories of Frame 1. This can be inferred from the data in Tables 1–3 upon examination of the plastic moment-capacity ratios. Cases where these ratios were above 70% indicate that beam sizes were governed by gravity loading cases and that the connec-

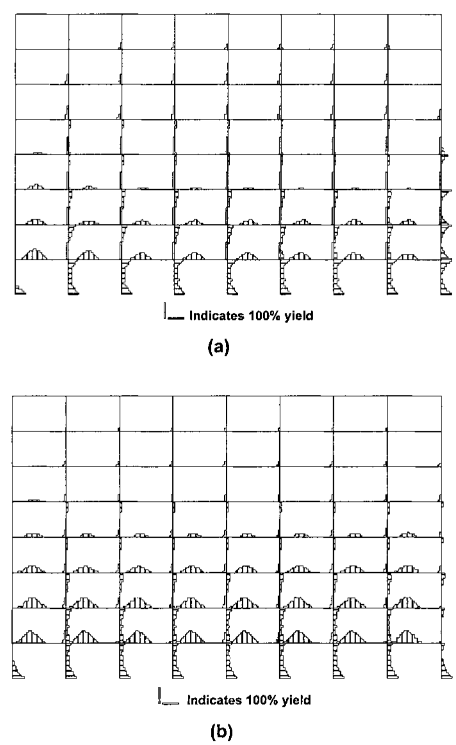

FIG. 10. Spread of Yielding as a Percentage of Initial Cross-Sectional Area for Frame 3 for Proportional Loading, (1.2D + 0.5L + 0.5S + 1.3W)γ_I: (a) Fully Restrained Connections; $\gamma_{I,\,ult}$ = 6.669, (b) Extended End-Plate Connections; $\gamma_{1,\,ult}$ = 6.051

tion moment is closely associated with this beam size. The effect of the controlling load combination for member design is most evident in the results for Frame 3, where the large number of bays and the relatively small lateral loads cause gravity load combinations (1.2DL + 1.6LL + 0.5SL or 1.2DL + 0.5LL + 1.6SL) to control the design of the beam members. Therefore, the load combination chosen to construct the two load sequences has very small live and snow load magnitudes relative to the load combination that controlled the member design. As a result, there is significant reserve strength. The situation is exacerbated for loading sequence II as the gravity load magnitudes are unchanged during application of the lateral loading. A similar rationale can be used for the other frameworks.

Several other factors contributed to the high ultimate load magnitudes. First, member design within the framework was not carried out with full optimization as the goal. Members were sized using a procedure that a practicing designer might employ for large structural frameworks such as those considered in this study. Two design analyses were implemented: The first established preliminary member forces for design; members were then preliminarily sized for these forces. The second design analysis was carried out using the refined member sizes; member capacity checks were then performed with the refined forces. Any member that had an interaction equation value (*Load* 1993) greater than 0.75 and less than 1.0 was deemed adequate. Therefore, in many cases, member designs were conservative. Also, the redundancy of a multistory, multibay framework will generally contribute to higher load factors since member by member design procedures contained in specifications (*Load* 1993) do not consider this redundancy. Finally, the practical consideration of reusing column and beam sections over multiple floors resulted in member sizes that were larger than necessary at alternate floor levels. Combinations of all these factors contributed to the large ultimate loads for the frameworks studied.

CONCLUSIONS

Several conclusions can be reached with respect to the results presented here. The connection classification system proposed by Goto and Miyashita (1996) adequately predicts PR frame behavior. However, the method is more cumbersome from a designer's point of view since a frame analysis including connection effects is needed a priori (future computer software implementing nonlinear connection response will change this aspect). The method proposed by Bjorhovde et al. (1990) seems to adequately predict the PR nature of the extended endplate connections considered with respect to connection moment capacity. However, at design load levels, the initial stiffness of the connection assessed using the α parameter seems to be questionable. The η_s parameter used by Goto and Miyashita (1996) for all frames studied was much greater than 5%, indicating PR behavior at design load levels.

The single-valued connection ductility requirement proposed by Bjorhovde et al. (1990) also seems suspect. For all frames addressed in this study, the required rotations of the connections needed to achieve the ultimate load of the frame were well beyond the predicted values obtained using $\mu = 6$. One should be cautious, however, in interpreting this result. The high load factors attained at collapse (beyond $\gamma_{ult} = 1.0$) for the three frames analyzed contribute to the high demand on the connections. Therefore, the results of this study confirm the difficulty involved in determining a single ductility parameter for all connections within a framework based purely on the connection initial stiffness and plastic moment capacity. A measure of framework behavior should also be considered in ductility requirements, but this may involve computer programs capable of incorporating nonlinear connection behavior in the design office.

The proportional and nonproportional loading used indicates that the overall behavior of PR-connected frames is similar to that of FR frameworks with respect to loading sequences. The case of proportionally applied loading results in the minimum ultimate load factor when compared to nonproportional (gravity constant) loading. The difference in the ultimate load factor for PR frameworks between proportional and nonproportional loading sequences decreases as the number of frame bays is decreased (i.e., increase in aspect ratio; h/w) due to greater instability effects resulting from partial connection restraint.

Finally, the yielding exhibited in Frame 3 indicates that plastic-hinge-based methods may not be applicable in addressing detailed frame response. The simple plastic-zone method of analysis presented here is straightforward to implement; it directly includes residual stresses, penetration of yielding within the cross section, yielding along the member lengths, shift in the elastic neutral axis of the cross section with yielding, and allows detailed fiber strain records to be obtained from the analysis to assess the tendency for local buckling. The method has been implemented on several practically sized frameworks to compute the load-deformation response, including the postcritical (unloading) path. As computing technology advances, implementation of the spread of plasticity analysis in large-scale general frameworks will soon be practically possible.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

DL = dead (gravity) loading;
d = depth of member;
E = modulus of elasticity of steel material;
h = overall height of framework;
I = second moment of area;
K = flexural stiffness parameter for beam member;
L = length of beam member;
LL = live (gravity) loading;
M = bending moment;
R = connection stiffness in context of nonlinear moment-rotation curve;
SL = snow (gravity) loading;
WL = wind (lateral) loading;
w = overall width of framework;
α = parameter used to define equivalent length of beam in terms of depth of connected beam member;
β = parameter defining rate of slope decay in three-parameter power model for nonlinear moment-rotation curve;
γ = load factor;
Δ = lateral sway;
η = connection classification parameter;
θ = connection rotation; and
μ = connection ductility (rotational demand) parameter.

Subscripts

act = actual (computed) quantity;
b = beam member;
be = beam equivalent;
E = pseudoelastic limit value;
FR = FR framework;
I = load sequence I;
II = load sequence II;
i = initial stiffness;
lf = load factor;
m = stage;
PR = PR framework;
pb = beam plastic capacity;
pc = connection plastic capacity;
R = required (predicted) quantity using classification system;
s = lateral sway; and
u = ultimate load level.