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## Collapse analysis of steel moment frames with various seismic connections

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## ABSTRACT

Seismic connections with high ductile capacity are generally considered to be effective for resisting seismic loads. However, additional studies are still needed to evaluate the performance of seismic connections during progressive collapse. In this study the progressive collapse resisting capacity of the Reduced Beam Section (RBS), Welded Cover Plated Flange (WCPF), and Welded Unreinforced Flange-Welded Web (WUF-W) connections, which are seismic connections recommended by the FEMA/SAC project, was investigated. For progressive collapse analysis, two types of steel moment frame buildings were considered; one designed for high-seismic load and the other designed for moderate-seismic load. The vertical displacement at the point of column removal and the plastic hinge rotation at beam ends were checked by using an alternative load path method proposed in the guidelines. The analysis results showed that the performance of the Cover Plate connection turned out to be the most effective in resisting progressive collapse, especially in structures located in moderate-seismic regions.

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## 1. Introduction

Steel moment frames are considered to be highly ductile and thus have been used as a major structural system in high-seismic regions. However, the Northridge earthquake in 1994 and the Kobe earthquake in 1995 revealed that brittle failure might occur at beam–column joints. In this background the SAC Joint Venture supported by the Federal Emergency Management Agency (FEMA) in the US conducted research on seismic performance of steel moment frames, and published a series of reports such as FEMA 350 [1] and FEMA 351 [2], which provide feasible information for seismic design of steel beam–column connections. The seismic performance of the Post-Northridge (PN) connections was verified through full-size experiments [3–5].

Meanwhile, the collapse of the World Trade Center raised special concern for design against abnormal loads. Especially in steel moment frames the ductility and robustness of beam–column connections were investigated to resist the progressive collapse caused by local failure of major load-resisting elements [6–11]. The Side Plate connections [12], originally developed as a seismic connection, are recognized to have excellent progressive collapse resisting capacity and are widely used in federal buildings in the US. However the connection requires more material and additional expense for patent royalty. In this regard it is necessary

to understand the collapse resisting capacity of available seismic connections to insure safety of buildings against progressive collapse.

In this study the progressive collapse-resisting capacities of steel moment frames with three-types of seismic joints, the Welded Unreinforced Flange-Welded Web (WUF-W) [3], Reduced Beam Section (RBS) [4], and the Welded Cover Plated Flange (WCPF) [5], were investigated. Since buildings located in moderate seismic regions will have different member sizes from those in high seismic regions, analysis model structures were designed both for moderate seismicity and high seismicity. Nonlinear static and dynamic analyses were carried out with 3- and 6-story model structures with one of the first story columns suddenly removed. The seismic performances of the model structures were also evaluated and the results were compared with the performance against progressive collapse.

## 2. Seismic joints considered for progressive collapse

After the Northridge earthquake it was observed that the welded steel connections, widely applied at that time, were vulnerable to fracture when subjected to strong earthquakes. The fracture mostly occurred at the weldment of the bottom flange in beams. Sometimes cracks developed at the face between column flange and the weldment and propagated through the entire section of the column. This resulted from inherent characteristics of the connection details used in practice before the Northridge earthquake, such as weld metal, shape of welded joint susceptible to stress concentration, and yield strength of structural steel.

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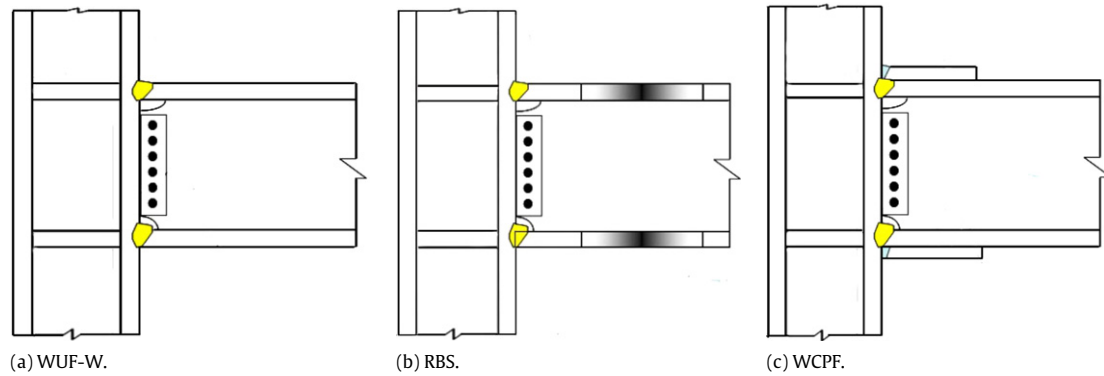


Fig. 1. Details of the FEMA seismic connections.

In the SAC Joint Venture, solutions to those problems were searched, and the Welded Unreinforced Flange-Bolted Web (WUF-B) connection was introduced as a seismic or a Post-Northridge (PN) connection. Compared with the Pre-Northridge WUF-B, there were improvements in many aspects of the PN connection such as weld access hole shape, welding method, weld metal, weld quality control, and so on. However, test results showed that the PN WUF-B still could not guarantee enough plastic rotation capacity [13]; so the AISC seismic provision [14] specified that the PN WUF-B connection should be used only in ordinary moment frames.

In order to improve the performance of the PN WUF-B connection, various variations were proposed as shown in Fig. 1 [3–5]. The WUF-W connection shown in Fig. 1(a) is known to have more plastic rotation capacity than the WUF-B [3]. This connection is similar to the WUF-B in shape, but can experience more plastic rotation in the beam end due to: removal of weld backing; use of tougher filler metal; addition of a reinforcing fillet weld; improved weld-access hole shape; and use of a full-strength welded web joint. Another way of improvement is to shift the possible location of the plastic hinge away from beam–column joints to achieve more ductile plastic rotation capacity. An example is the RBS joint shown in Fig. 1(b), in which plastic hinge locations are assigned in connecting beams by cutting a part of beam flange. In this connection, reinforcement is not added to join the flanges of the beam to the column except for weld metal. Another one is the WCPF (Welded Cover Plated Flange), in which cover plates are added to both the top and bottom flanges of the connecting beams (see Fig. 1(c)). It is shown that the addition of the cover plates significantly reduces the stress concentration in critical region and forces the plastic hinge to develop at the end of the plate, i.e., in the connecting beams [5]. In order to verify the seismic performance of these connection types various experiments were conducted in the SAC Joint Venture. Through the experiments, their seismic performances were verified; however the results are not sufficient enough to directly apply them to prevent progressive collapse.

### 3. Analysis models and methods

#### 3.1. Analysis model

In order to investigate the progressive collapse potential of steel moment frames with various connection types, 3- and 6-story structures were designed both for moderate and high seismic loads. Fig. 2(a) shows the plan of the model structures and Fig. 2(b) depicts the side view of the 3-story moment frame. For structures located in moderate seismic regions, the seismic design load was determined in accordance with the Korean Building Code [15]. The structures were assumed to be located in Seoul, Korea, which belongs to a moderate seismic zone. The structures were designed

Table 1

Member sizes of the 3-story model structures.

Member	Story	Size
(a) Designed for moderate earthquake		
Columns	1–3	H-304 × 301 × 11 × 17
Beams	1–3	H-340 × 250 × 9 × 14
(b) Designed for strong earthquake		
Columns	1–3	H-612 × 325 × 13 × 19
Beams	1–3	H-548 × 213 × 15 × 24

Table 2

Member sizes of the 6-story model structures.

Member	Story	Size
(a) Designed for moderate earthquake		
Columns	1–3	H-400 × 400 × 13 × 21
4–6	H-208 × 202 × 10 × 16	
Beams	1–3	H-450 × 200 × 9 × 14
4–6	H-450 × 200 × 9 × 14	
(b) Designed for strong earthquake		
Columns	1–3	H-648 × 330 × 21 × 38
4–6	H-544 × 312 × 13 × 20	
Beams	1–3	H-693 × 252 × 15 × 24
4–6	H-549 × 214 × 15 × 24	

as ductile steel moment frames with the design seismic coefficients  $S_{DS}$  and  $S_{D1}$  equal to 0.53 g and 0.34 g, respectively, in the IBC 2006 [16] format. For structures located in high seismic regions such as Los Angeles, the design seismic load with  $S_{DS} = 1.33$  g and  $S_{D1} = 0.67$  g was used. The structures were considered as special moment frames and the response modification factor of 8 was used in the computation of design base shear. The columns and girders were designed with SM490 ( $F_y = 310$  MPa) and SM400 ( $F_y = 240$  MPa) steel, respectively. Member sizes of the 3- and the 6-story model structures for each seismic zone are presented in Table 1 and Table 2, respectively.

In the model structures only the perimeter moment frames enclosed in the dotted rectangle in Fig. 2 were analyzed. Three types of post-Northridge connections, WUF-W, RBS, and WCPF, were considered in each model structure. The panel zones were modeled as rigid regardless of the connection types, and the post-yield stiffness was assumed to be 2% of the initial stiffness. Figs. 3–5 show the configuration and analytical modeling of the WUF-W, RBS, and the WCPF connections, respectively, in which nodal points were located where the shape of the members changes. The nonlinear analysis program code OpenSees [17] was used for nonlinear static and dynamic analysis of the model structures.

The beam and column members in the model structures were modeled with ‘nonlinearBeamColumn’ elements provided

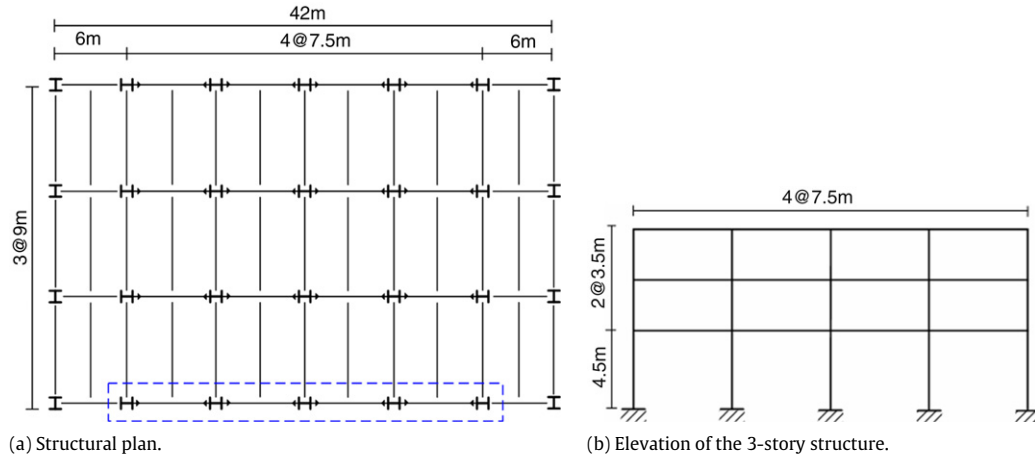


Fig. 2. Structural configuration of analysis model structures.

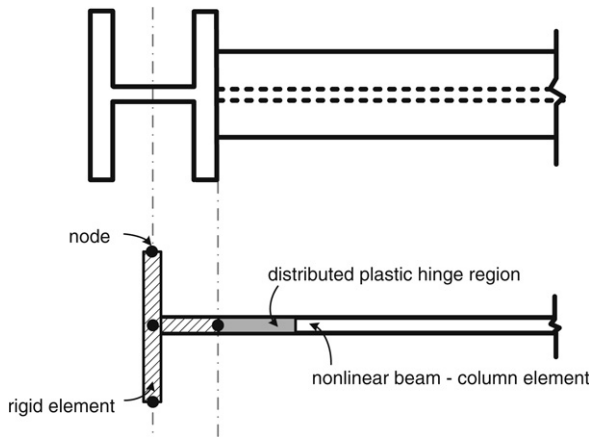


Fig. 3. Plan view and analysis modeling (side view) of WUF-W connections.

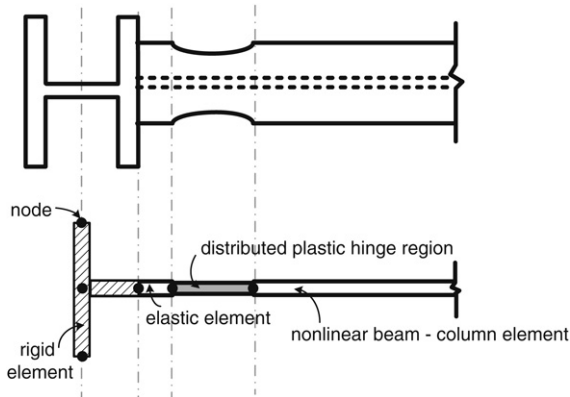


Fig. 4. Plan view and analysis modeling (side view) of RBS connections.

by the OpenSees. The ‘nonlinearBeamColumn’ element considers distributed plasticity along the element and can simulate second-order effect. Using the element, the sections of the beam and column members were modeled by a series of layers (or fibers) using ‘fiberSec’ command in the OpenSees. The P–M interaction in columns is automatically reflected by the layers in each section. The material property of each layer was modeled by the ‘Steel01’ element. The envelope of the ‘Steel01’ element is a bi-linear shape with strain-hardening ratio of 2%.

It is noted that limit state criteria for beam and column members and panel zone were not defined in the analysis. The United

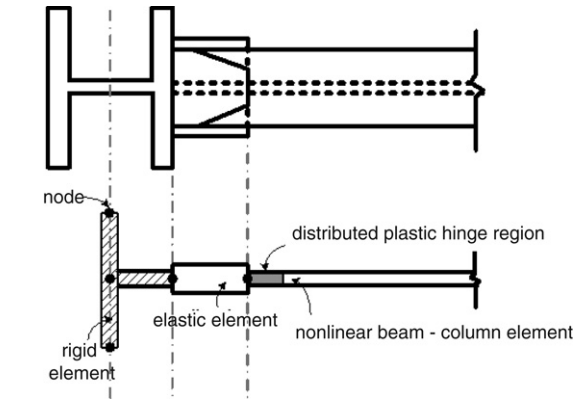


Fig. 5. Plan view and analysis modeling (side view) of WCPF connections.

States General Services Administration (GSA) guidelines [18] recommend criteria for limit state, but most of them are simply adopted from seismic provisions such as FEMA 356 [19] and are not proved by sufficient experimental and/or analytical studies. Therefore, the limit state criteria were excluded in the analysis model to avoid misleading understanding of the analysis results. It is also noted that hysteretic rules such as pinching and cyclic degradation of stiffness and strength were not included in the analysis model. This is because cyclic behavior is not severe in collapse analysis. This feature can be confirmed by analysis results later.

In order to take the change in cross sectional shape into account, the connections were modeled as follows: in the WUF-W connections, nonlinear link elements were located at the end of beams, and in the RBS and the WCPF they were located at the part of the circular radius cuts and at the end of cover plates, respectively. The circular radius cuts in RBS connections were modeled to have equivalent constant width using the equation proposed by Lee [20]. In the analysis models shown in Figs. 3–5 the parts marked with slant lines represent panel zones and the gray parts represent the location where inelastic deformations occur. It can be noticed that inelastic deformation will occur away from the panel zone in the RBS and the WCPF connections. Such models can simulate the formation of plastic hinges caused by bending moment more realistically.

### 3.2. Analysis methods

Nonlinear dynamic seismic analyses were carried out first to investigate the seismic capacity of the model structures designed with the three seismic connection types. Two sets of twenty

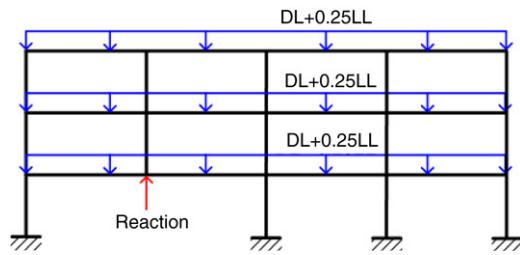


Fig. 6. Application of gravity load and removal of a column for dynamic analysis.

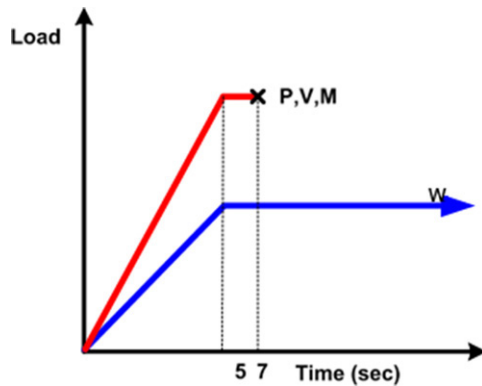


Fig. 7. Time history of imposed load for dynamic collapse analysis.

earthquake ground motions, originally developed for the SAC project [21], were used for seismic analysis. One set of records consists of the LA21 to LA40 records with recurrence period of 2500 years in Los Angeles area and another set is composed of the LA41 to LA60 records with 500 year return period in the same area. For analysis of the model buildings located in moderate seismic region, the ground motions of the LA41 to LA60 were scaled down to fit the design spectrum specified in the KBC 2005. The seismic performance of the model structures was evaluated based on the maximum inter-story drift.

Next, nonlinear static and dynamic analysis methods were applied to investigate the progressive collapse potential of the model structures. For nonlinear static pushdown analysis the load combination  $2(DL + 0.25LL)$  was used as vertical load in the span where a column was removed, and the load combination  $DL + 0.25LL$  was applied in the other spans as specified in the GSA guidelines. For nonlinear dynamic analysis the load  $DL + 0.25LL$  was uniformly applied as vertical load in the entire span as shown in Fig. 6. In order to carry out dynamic analysis the member forces of a column, which is to be removed to initiate progressive collapse, were computed before it is removed. Then the column was replaced by the point loads equivalent of its member forces as shown in Fig. 6. In order to simulate the phenomenon that the column was abruptly removed, the member forces were suddenly removed after a certain time had elapsed while the gravity load remained unchanged as shown in Fig. 7, where the variables  $P$ ,  $V$ , and  $M$  denote the axial force, shear force, and bending moment, respectively, and  $W$  is the vertical load. In this study the member forces were increased linearly for five seconds until they reached their full amount, were kept unchanged for two seconds until the system reached stable condition, and were suddenly removed at seven seconds to initiate progressive collapse.

#### 4. Analysis results of model structures

##### 4.1. Nonlinear time-history seismic analysis

Tables 3 and 4 show the mean values of the maximum inter-story drift ratios obtained from nonlinear time history seismic

Table 3

Mean maximum inter-story drift ratios of the model structures designed for medium seismicity obtained from nonlinear time history analyses (%).

Story	WUF-W	RBS	WCPF
(a) 3-story structure			
1	1.61	1.65	1.59
2	0.88	0.89	0.84
3	0.37	0.40	0.32
(b) 3-story structure			
1	0.67	0.70	0.65
2	0.80	0.83	0.75
3	0.62	0.64	0.56
4	0.43	0.44	0.42
5	0.40	0.42	0.38
6	0.18	0.20	0.19

Table 4

Mean maximum inter-story drift ratios of the model structures designed for high seismicity obtained from nonlinear time history analyses (%).

Story	WUF-W	RBS	WCPF
(a) 3-story structure			
1	2.18	2.19	2.12
2	2.09	2.35	2.02
3	1.12	1.30	1.07
(b) 6-story structure			
1	0.96	1.07	0.82
2	1.21	1.52	1.14
3	1.17	1.35	1.11
4	0.94	1.12	0.86
5	0.90	1.07	0.73
6	0.44	0.58	0.40

analyses of the 3- and the 6-story model structures designed for moderate and high seismicity, respectively. The twenty earthquake records described above were used for dynamic analyses. It can be observed that the inter-story drift ratios of the structure with the WCPF connections are slightly smaller than those of the structure with the WUF-W or the RBS connections. The RBS connections resulted in the largest inter-story drifts, which was expected because some parts of the beam flanges were removed in the RMS connections. However in most cases the differences were not significant. Similar results can be observed in structures designed with strong seismic loads.

##### 4.2. Push-down analysis results

The vertical push-down analysis for progressive collapse was carried out by gradually increasing the vertical displacement at the location of the removed column to investigate the resistance of the structure against such deformation. Figs. 8 and 9 show the pushdown curves of the 3- and the 6-story model structures designed with moderate and strong seismic loads, respectively. In the figures the load factor of 1.0 corresponds to the state that the vertical load reached the gravity load specified in the GSA guideline for nonlinear static progressive collapse analysis. It can be observed that, as expected, the yield strength is highest in the structures with WCPF connections and is lowest in the structures with RBS connections. In structures designed for moderate seismicity the load factors at yield are less than 1.0 in structures with RBS connections. This implies that structures with the RBS connections may have the potential for possible progressive collapse. In structures designed for high seismicity the load factors are higher than 2.0 in most cases, which implies that the progressive collapse potential may be very low. It also can be noticed that the yield strengths of the 6-story structures are generally higher than those of the 3-story structures.

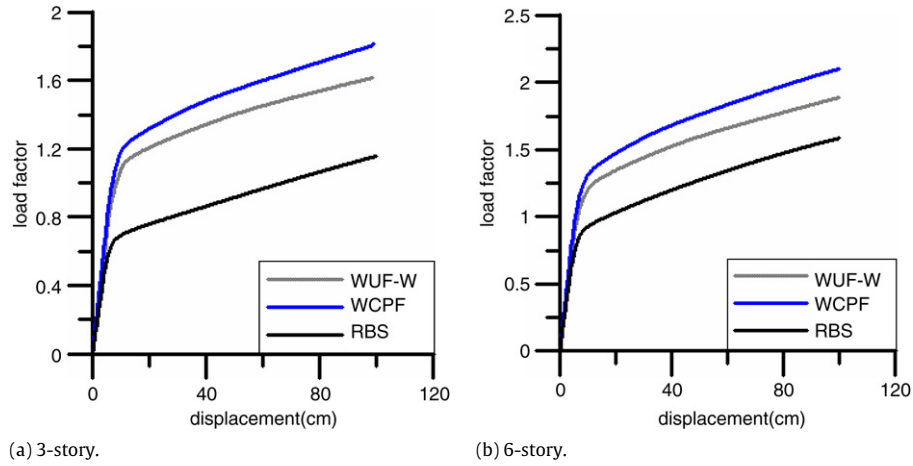


Fig. 8. Pushdown curves of structures located in moderate seismic region.

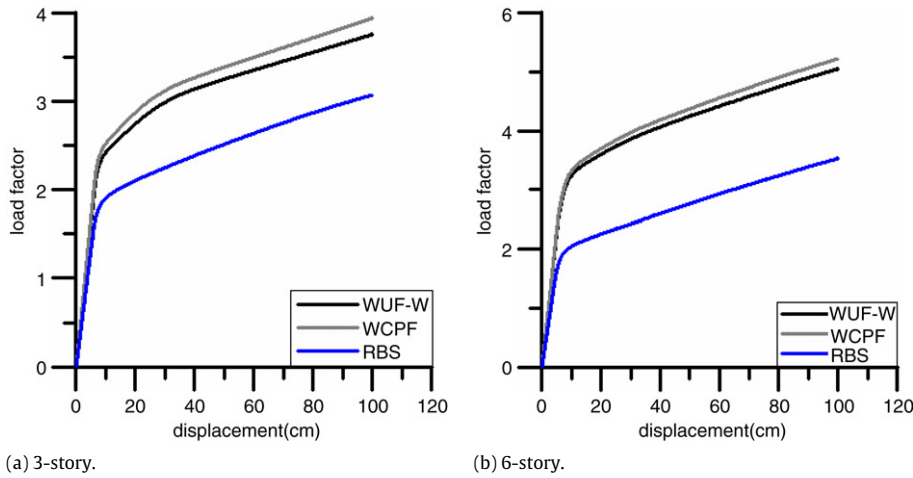


Fig. 9. Pushdown curves of the model structures located in high seismic region.

4.3. Nonlinear dynamic progressive collapse analysis results

Fig. 10 shows the time history of the vertical deflection at the joint where a column was suddenly removed in structures designed with smaller seismic load. It can be observed that the maximum deflections of the structures with RBS connections are significantly larger than those with the WUF-W and the WCPF connections, and that there is little difference between the deflections of the joints with the WUF-W and the WCPF connections. Even though the WCPF connections have larger stiffness and strength than the WUF-W connections, the plastic moments are the same and only the locations of plastic hinges are different. In the RBS connections the plastic hinges generally form at the reduced sections and therefore the maximum strength is the minimum of all connection types. This implies that the flexural strength of beam ends significantly affects the vertical deflection caused by a sudden removal of a column. It also can be observed that the deflections in the 6-story structure designed for moderate seismicity are smaller than those of the 3-story structure. As the number of story increases the relative member size also increases due to increase in lateral load. In addition more load is redistributed due to vierendeel action of moment frames as the number of story increases. Fig. 11 shows the time history of the vertical deflection in the 3-story structures designed with strong seismic load, where the vertical deflections of the structures with all types of connections are within elastic range and the difference depending on connection types is not significant.

Table 5

Ductility demands of model structures designed for medium seismicity obtained from progressive collapse analyses.

Connection types	Yield displacements (cm)	Maximum displacements (cm)	Ductility demands
(a) 3-story structure			
WUF-W	11.6	19.6	1.7
WCPF	10.7	14.2	1.3
RBS	8.09	94.21	11.8
(b) 6-story structure			
WUF-W	11.7	12.7	1.1
WCPF	10.6	9.8	1.1
RBS	8.0	30.0	3.8

Tables 5 and 6 present the ductility demands of the model structures, which are the ratio of the maximum deflections obtained from dynamic analyses and the yield deflections obtained by static pushdown analyses. The GSA guideline recommends the ductility limit of 20 for steel beams regardless of the connection types. It can be observed from the tables that the limit state for ductility demand is not exceeded in all connection types considered in this paper, and that the ductility demands for the 6-story structures are less than those for the 3-story structures. However in the 3-story structure designed for moderate seismic load the ductility demand of the RBS connection exceeds 10, which is much higher than those of the other connection types. In the



