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The design and seismic performance of low-rise long-span frames with semi-rigid connections

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ABSTRACT

Moment-resisting steel frames are used frequently in low-rise and mid-rise buildings located in high seismic areas due to their high ductility and economic solutions. In these type of structures, strong-column weak-beam design requirements result in larger column sections and overdesign in low-rise long-span buildings. To mitigate this problem, moment-resisting steel frames with energy-dissipative semi-rigid/partial strength connections can be used as an alternative to perimeter frames. By using energy-dissipative semi-rigid connections, the strong-column weak-beam requirement is eliminated and more economical column sections are used. In this study, a three-span three-bay frame with 7 and 9 m span lengths is designed with semi-rigid connections having four different capacities in high seismic zones. Their seismic performance is evaluated analytically under three different earthquake levels by modeling connections with two different moment-hardening ratios and two different hysteretic behavior models. The design with reduced connection capacity resulted in an increase in the beam weights, a decrease in the column weights and an overall decrease in the structural weight. The seismic performances of 26 sample frames are evaluated with pushover and dynamic analyses under 25 real strong ground motion records. All of the sample frames satisfied the acceptance criteria and showed reliable performance under earthquake loading. The overdesign problem in low-rise long span-buildings is eliminated to some extent without using the perimeter frame approach. Furthermore, under some specific ground motion records, the top displacements in semi-rigid frames become lower than those of their rigid counterparts.

1. Introduction

Moment-resisting steel frames are used frequently in low-rise and mid-rise buildings located in high seismic areas due to their high ductility and economic solution options. These frames, especially their fully welded connections, were heavily and unexpectedly damaged during the 1994 Northridge and the 1995 Kobe earthquakes.

In these structures, the strong-column weak-beam design approach is used to allow for plastic hinges to develop in the beams prior to the columns and increase the ductility of structure to prevent collapse. One of the major shortcomings of strong-column weak-beam provisions is that they result in larger column sections and overdesign in low-rise long-span buildings. In order to overcome these shortcomings, a typical practice in the US is to utilize lateral load-resistant frames only in the perimeter frames. However, low redundancy and lack of redistribution capacity are the main disadvantages of using such an approach. Furthermore, minor local damages in the perimeter frames could increase the eccentricity of the structure and even result in total collapse of the building. The most famous example of this is the California State University Parking Structure [1].

By designing low-rise long-span structures with energy-dissipative zones in semi-rigid/partial strength connections, most of the aforementioned shortcomings can be eliminated. Field-bolted connections shorten the field erection process, require less skilled labor and provide more reliable construction quality. In addition, the need for strong-column weak-beam provision can be eliminated by only designing the columns to be stronger than the connections. As a result, smaller column sections can be used and an alternative to perimeter frames can be obtained.

Some of the first experimental works investigating the seismic behavior of frames with semi-rigid connections were reported in [2–5]. As a result of these studies, it was concluded that properly designed semi-rigid connections and frames can exhibit ductile and stable hysteretic behavior. In addition, when the connection stiffness increased the base shear consequently increased, but the lateral drift did not decrease in a similar manner. Finally, it was pointed out that the optimum system should be searched for in

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Nomenclature

b_f	Width of flange
d	Bolt diameter
f_{ub}	Ultimate tensile strength of a bolt
h	Distance from inside of compression flange to inside of tension flange
h_s	Story height
l	Span of the beam
t	Thickness of either end plate or column flange
t_f	Thickness of flange
t_w	Thickness of web
C^*	Effective stiffness of the beam and its connections
C_d	Deflection amplification factor
E	Elasticity modulus of the material
F_u	Specified minimum ultimate strength
F_y	Specified minimum yield stress
F_{ye}	Expected yield strength
I_c	Moment of inertia of the column
I_g	Moment of inertia of the girder
K_i	Initial stiffness of the idealized capacity curve
L_c	Unsupported length of the column
L_g	Unsupported length of the girder
M_{pb}	Plastic moment capacity of the beam
M_{pc}	Plastic moment capacity of the connection
P	Axial force in a member
P_{CL}	Lower bound compression strength of a column
P_i	Portion of the total weight of the structure including dead, permanent live and 25% of transient live loads acting on all of the columns within story *i*
R	Seismic response modification factor
SI_H	Spectrum intensity
T	Period of vibration
V_d	Design lateral strength
V_u	Ultimate lateral strength
V_y	Yield lateral strength
V_{yi}	Total plastic lateral shear restoring capacity at story *i*
α	Post-yield slope of the idealized capacity curve
μ	Ductility ratio
θ_y	Yield rotation
ξ	Damping coefficient
Δ_i	Calculated lateral drift of story *i* under design seismic loads
Δ_u	Ultimate displacement of the system
Δ_y	Yield displacement of the system
Ω_d	Overstrength factor
Ψ_i	Nondimensional stability coefficient of story *i*

order to develop the least possible base shear with acceptable lateral deformations.

Early analytical work was also carried out to assess the seismic behavior of semi-rigid frames [6–9]. The results showed that there is a significant potential to design semi-rigid frames whose seismic performance is comparable to that of rigid frames in moderately and highly seismic regions. However, most of these studies investigated frames that were not initially designed with semi-rigid connections or overlooked several connection properties such as connection capacities and moment hardening.

Studies investigating the cyclic behavior of field-bolted connections increased after the late 1980's. One of the primary distinctions between these studies is the source of inelastic behavior. Relatively few researchers have investigated the inelastic response of connection members (end plate, angle, T-stub, panel zone, column flange, etc.) and the others investigated the inelastic response of the connecting beam. The cyclic behavior of partial strength extended end plate connections were experimentally investigated in [10,11] among others. In both of the studies, it was seen that extended end plate connections have adequate rotation capacity and stable hysteresis behavior if brittle failure modes are prevented. Furthermore, in the latter study, three failure mode requirements and the corresponding resistances have been recommended to assure that the end plate connection can provide enough rotation capacity and energy dissipation capacity under earthquake loading and its ultimate failure mode is ductile.

In this study, three-story three-bay buildings with energy dissipation zones in semi-rigid connections are designed in high seismic areas and their seismic performances are evaluated analytically with different connection capacities, frame geometries and earthquake levels. By using energy-dissipative semi-rigid connections, the strong-column weak-beam requirement is eliminated and alternative economic systems to perimeter frames are investigated. Brittle failure such as occurred in fully welded connections during the 1994 Northridge and the 1995 Kobe earthquakes is also eliminated by using field-bolted connections. Buildings with 7.0 and 9.0 m span lengths are designed by using rigid and $0.7M_{pb}$, $0.6M_{pb}$ and $0.5M_{pb}$ capacity semi-rigid connections. Semi-rigid connections are taken into account with two different moment-hardening ratios, 1.1 and 1.4 (ratio of moment capacity at 0.035 rad to plastic moment capacity), and two different hysteretic behavior models. The seismic performance of these buildings is evaluated through eigenvalue analyses, nonlinear pushover analyses and nonlinear time history analyses. In the time history analyses, 25 strong ground motion records are used with three earthquake levels.

All of the sample frames satisfied the acceptance criteria and showed a reliable performance under the earthquake effects. The overdesign problem in low-rise long-span buildings was eliminated to some extent without using the perimeter frame approach. Furthermore, under some specific ground motion records, the top displacements in semi-rigid frames become lower than those of their rigid counterparts.

2. Description and design of the buildings

In this study, a three-story three-bay symmetric office building is considered with two different span lengths (7.0 and 9.0 m). The story heights are chosen as 4.2 m for the first floor and 3.6 m for the remaining floors. A typical plan and elevation drawing of the sample building with 7.0 m span is given in Fig. 1. The lateral resistance of the buildings is provided by special moment-resisting frames in the EW direction, whereas the NS direction is assumed to have a braced system. The behavior of the buildings in the NS direction is not in the scope of this study, so only the moment-resisting frames (EW direction) are designed and investigated. All the inner and outer frames are designed as being lateral load resistant so only a typical interior frame is considered. A total of eight frames having 7.0 and 9.0 m span lengths and four different connection capacities (rigid, 70%, 60% and 50% capacity semi-rigid connections) are designed.

The structural steels used in the buildings are A992 (F_y = 345 MPa) for beams and columns and A36 Gr.36 for end plates ($F_y = 250$ MPa). The bolts in the semi-rigid connections are ASTM A490.

Gravity loads, seismic loads and load combinations are determined according to ASCE 7-05 [12] and IBC 2006 [13] codes. For gravity loads, a dead load of 3.20 kN/m^2 including the self-weight of structural steel, mechanical and electrical equipment is applied to both the floors and the roof. The live load is taken as 1.00 kN/m^2 for the roof and 3.80 kN/m^2 for the floors, and this includes 0.50 kN/m^2 for partition walls.