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Seismic performance evaluation of steel moment resisting frames through incremental dynamic analysis

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ABSTRACT

Earthquake hazards effect significant damage to structures and cause widespread failure throughout buildings. Moment resisting frames are widely used as lateral resisting systems when sufficient ductility is to be met. Generally three types of moment resisting frames are designed in practice namely Special, Intermediate and Ordinary Moment Frames, each of which has certain level of ductility. Comparative studies on the seismic performance of these three different types of structure are performed in this study. Analytical models of connections are employed including panel zone and beam to column joint model. Incremental dynamic analysis is then utilized to assess the structural dynamic behavior of the frames and to generate required data for performance based evaluations. Maximum annual probability of exceeding different limit states may reveal the superiority of a ductile structure in which a greater behavior factor is employed. Special moment resisting frames are expected to perform better once a certain level of ductility is to be met but the amount of superiority may be the subject of investigation especially from a performance based design standpoint.

1. Introduction

Earthquake induced forces may impose extensive damage on structural elements resulting collapse and loss of lives particularly in seismically active areas. As a consequence, several design provisions and building codes directly address the significance of ductile design of structures located on sites where ground shaking hazards are considerable. As of 1994 Northridge California earthquake, designers would assume that typical connections function properly when subjected to reversal loading but the poor performance of welded moment frame connections led to the introduction of innovative moment resisting frames of different kinds. Steel moment resisting frames comprise columns and beams that are typically joined by welding or high-strength bolting or a combination of both. Shearing and flexural actions in the members contribute the most in lateral resistance of these types of structures. The AISC Seismic Provisions [1] define three types of steel moment frames namely Ordinary Moment Frames, Intermediate Moment Frames and Special Moment Frames (OMF, IMF and SMF hereafter). SMFs are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. Inelastic deformations of IMFs are more limited in comparison with those of SMFs. OMFs are less ductile than IMFs, and are expected to sustain only limited inelastic deformations within their components and connections.

Several researchers have investigated the performance and responses of moment resisting frames. Luco and Cornell [2] conducted research on the behavior of moment frames in which fracturing connection models were utilized to realistically simulate the response of the structures. Jalayer and Cornell (1998, 2000) and Cornell et al. [3] studied the seismic reliability of steel frames and presented an introduction to the probabilistic basis for a new set of seismic designs. Yun et al. [4] carried out seismic performance evaluation for steel frames based on nonlinear dynamics and reliability theory and established a framework in which a simple method for estimating a confidence level for satisfying the performance level given a hazard level is provided.

Different types of moment frames exhibit different responses and consequently meet certain performance objectives differently. As for the structural behavior, it is feasible to evaluate the extent of damage and vulnerability of SMF comparing to IMF and OMF through the use of nonlinear static and dynamic analyses as indicated in FEMA-350 [5]. Furthermore, seismic reliability analyses and performance based studies may reveal differences between moment frames of varying types. The concept incorporates three primary elements namely ground motion intensity, drift demand as well as drift capacity [3]. Comparative performance studies based on nonlinear dynamic procedure for three above-mentioned moment frames have not been carried out extensively

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List of symbols

A_{eff}	Effective shear area
b_{cf}	Column flange width
d_b	Beam depth
d_c	Column depth
G	Shear modulus
G_1, G_2	Gravity loads on beams
H	Horizontal load
H_c	Column height
K_{el}	Elastic stiffness
K_1	Post-elastic stiffness
L	Beam length
M_y^{pa}	Yield moment
M_{sh}^{pa}	Second yield moment
M_p	Beam bending strength at plastic hinge location
M_{max}	Maximum beam moment
P	Axial force
P_y	Axial yield force
R	Response modification factors
t_{bf}	Thickness of beam flange
t_{cf}	Thickness of column flange
t_{cw}	Column web thickness plus doubler plates thickness
V_y	Yield shear force
γ_y	Yield panel zone deformation
γ_p	Panel zone deformation at the beginning of strain hardening
$\bar{\tau}_y$	Von Mises yield shear stress
σ_y	Yield stress
$S_a(T_1, 5\%)$	5%-damped first-mode spectral acceleration
θ_{max}	Maximum peak interstorey drift ratio
$H(s_a^{\hat{C}})$	Approximated hazard curve
β_{D/S_a}	Dispersion of drift demands
β_C	Dispersion of drift capacity
P_{PL}	Annual probability of performance level
C	Capacity of structure
D	Demand of structure
γ	Demand variability factor
γ_a	Analysis uncertainty factor
φ	Resistance factor
λ	Confidence index parameter

in the literature. Moreover, as the structure height increases, the differences tend to become more significant and the level of confidence may become more dependent upon parameters defined in nonlinear dynamic analyses, thereby intensifying the inherent uncertainties of the results. In this paper, to evaluate and compare the performance of moment resisting frames of varying types, three 5-storey buildings (SMF, IMF and OMF) and two 10-storey buildings (SMF and IMF) are designed according to the Iranian Seismic Code 2800 [6]. It should be noted that provisions given in the Iranian national code are very similar to those of AISC [1] and FEMA-350 [5] and relevant design considerations are going to be explained in detail. The capacities are estimated for all structures through Incremental Dynamic Analysis (IDA), Vamvatsikos and Cornell [7], a computer-intensive procedure that offers demand and capacity prediction capability by using a series of nonlinear dynamic analyses under suitably multiplied scaled ground motion records. Analytical models of buildings are developed using nonlinear finite element program OpenSees [8] which is capable of performing nonlinear static and dynamic analyses. A robust and detailed analytical model is given in which connections, panel zones, beams and columns are all modeled to simulate the behavior of real moment resisting frames as accurately as possible. Moreover, some parametric studies are carried out through the use of modal and nonlinear static analyses to reveal the different behavior and responses of the structures. Based on the reliability analysis done by Cornell et al. [3], as well as performance evaluations and confidence level predictions presented in FEMA-350 and Yun et al. [4], performance evaluation studies are carried out for all buildings both locally and globally which characterizes the differences between the systems.

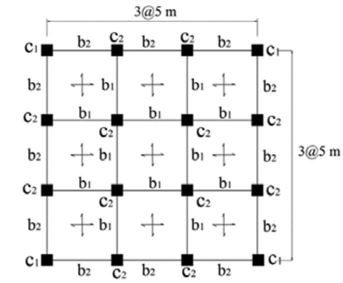

Fig. 1. Plan view of buildings.

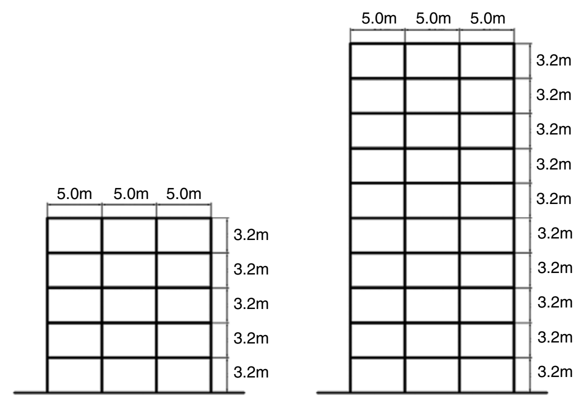

Fig. 2. Elevations and spans of buildings.

Table 1
Different types of seismic steel moment-resisting frames [6,10].

Structures	R	Height limitation (m)	Inelastic rotation capacity for rigid connections (rad)
OMF	5	15 (in low and moderate seismic zones)	0.01
IMF	7	50	0.02
SMF	10	150	0.03

2. Design of structures

To evaluate the performance of SMF, IMF and OMF buildings, three 5-storey buildings (SMF, IMF, OMF) and two 10-storey buildings (SMF, IMF) are designed for a site[1] which represents a high seismic zone. These buildings assumed to be located on soil type B (Average shear wave velocity to a depth of 30 m would be 360–750 m/s). The buildings are square in plan and consist of three bays of 5.0 m in each direction and the storey heights of buildings are 3.2 m. (Figs. 1 and 2). A rigid diaphragm can be assumed according to the roof system existing in usual structures. Gravity loads are supposed to be similar to common residential buildings in Iran [9]. The values of response modification factors (i.e. R) which are utilized by Iranian Seismic Code 2800 [6] are shown in Table 1. Drift criteria are considered and result in beam and column

[1] Tehran, Iran.