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SEISMIC RESPONSE OF 3-D STEEL BUILDINGS WITH CONNECTION FRACTURES

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SUMMARY

After the 1994 Northridge and 1995 Kobe earthquakes, a large number of brittle beam-to-column connection fractures were discovered in steel frame buildings. Impact of connection fractures on the performance of buildings against future earthquakes has since become a serious concern. Wang and Wen [1998] have developed a connection-fracture hysteresis model for investigation of structural performance. To account for the effects of flexible diaphragms, bi-axial interaction, torsional motion, and partial column failure, a 3-D inelastic structural model based on the member hysteresis model was developed. Numerical studies show that for response evaluation of strong-column, weak-beam steel buildings, conventional shear-beam model assuming rigid diaphragms may serially underestimate the response. It is also found that response increase caused by connection fractures per se is moderate. Connection fractures coupled with biaxial interaction and torsion, however, can lead to large increases in structural responses. Partial column damage causes only moderate increase in structural response. It may explain the fact that although some column damages were found in steel buildings after recent earthquakes, very few collapsed.

INTRODUCTION

After the 1994 Northridge and the 1995 Kobe earthquakes, many connection brittle-fracture failures have been found in welded steel moment frame buildings. The test results of steel connections, e.g., from the US SAC Steel Project, have verified the post-earthquake field survey results and shown a large variability in the load-carrying capacity of these connections. Although very few steel structures collapsed, impact of connection fractures on the performance of buildings against future earthquakes remains a serious concern. There is a need to investigate whether a large number of such structures in high seismicity areas have enough reserve strength to survive future earthquakes.

Earthquake excitations and building responses are 3-dimensional in nature. Torsional motions and bi-axial interactions may become significant, causing considerable amplification in structural response. To examine such effects due to brittle beam-column joint failures, a 3-D building structure model is developed based on the connection-fracture hysteresis model of Wang and Wen [1998]. Floor diaphragms are assumed rigid in plane but flexible out-of-plane, which prohibits in-plane shear distortion and axial elongation but permits out-of-plane bending deformation. Inelastic yielding is assumed to be confined to discrete hinge regions located at the structural member (beam and column) ends. P-Δ effect is considered following Wilson and Habibullah [1987].

Response analyses of a two-story and a three-story steel moment-resisting buildings at Los Angeles, California subjected to ground motions of the SAC Phase 2 project are carried out. Effects of rigid-diaphragm assumption, bi-axial excitation, torsional motion, and partial column damage are investigated.

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MODELING OF ASYMMETRIC MULTI-STORY BUILDINGS

Buildings are 3-dimensional structures. A structure of regular, symmetric configuration and uniform mass distribution in the building plan may be modelled as two-dimensional frame structure. Bi-axial structural interaction and/or torsional oscillation, however, may become significant as a result of bi-axial excitation and/or structural irregularity. Under such circumstances, two-dimensional models are no longer adequate. A commonly used simplifying assumption in 3-D model for asymmetric multi-story buildings is that the floor diaphragms and beams are rigid so that plastic deformation can occur only in the columns [e.g., Yeh and Wen 1989]. Most recent design procedures, however, encourage strong-column, weak-beam (SCWB) systems to prevent inelastic deformations in the columns. Girder flexibility and moment-resisting capacity is also a necessary consideration in realistic modelling of the effects of connection fractures. For building structures under earthquakes, lateral deflections usually dominate, causing large concentrated stresses in the beam-column joint regions. Therefore, in this study the girder flexibility will be considered and inelastic deformation is assumed to concentrate at column and girder ends and modelled by discrete inelastic hinges.

Equations of Motion

Consider an asymmetric multi-story building with mass eccentricity. Each floor diaphragm, of mass M_i and moment of inertia I_i, is assumed rigid in its own plane but flexible out-of-plane. Therefore the motion of the diaphragm at each floor level can be characterized by 3 degrees of freedom (DOF) in its own plane, i.e., translations in the X and Y directions, plus a rotation about the vertical axis. The floor diaphragms are flexible out-of-plane. At the column ends, two rotational DOF's are required. Hence if the axial deformation of members is neglected, the vector of structural (global) response DOF's is given by

$$\mathbf{u} = \left\langle \cdots \quad u_x^i \quad u_y^i \quad u_\theta^i \quad \cdots \quad \Big| \quad \cdots \quad \theta_x^{ij} \quad \theta_y^{ij} \quad \cdots \right\rangle^T = \left\langle \mathbf{u}_a \,\middle|\, \mathbf{u}_c \right\rangle^T \tag{1}$$

in which u_x^i, u_y^i and u_θ^i are the displacements and in-plane rotation of the *i*th floor relative to the ground; θ_x^{ij} and θ_y^{ij} are the column-end rotations of the *j*th column on the *i*th floor. The lateral accelerations of the *i*th-floor mass center are $\ddot{u}_x^i + \ddot{u}_{gx}$ and $\ddot{u}_y^i + \ddot{u}_{gy}$ where $\ddot{u}_{gx}$ and $\ddot{u}_{gy}$ are the two ground acceleration components. If the ground rotation is neglected, the rotational acceleration of the *i*th floor is $\ddot{u}_\theta$. The equations of motion for the *i*th-floor diaphragm can therefore be expressed as

$$\begin{aligned}
&M_i \ddot{u}_x^i + \sum\nolimits_j S_x^{ij} - (1-\delta_{iN}) \sum\nolimits_k S_x^{(i+1)k} = -M_i \ddot{u}_{gx} \\
&M_i \ddot{u}_y^i + \sum\nolimits_j S_y^{ij} - (1-\delta_{iN}) \sum\nolimits_k S_y^{(i+1)k} = -M_i \ddot{u}_{gy} \\
&I_i \ddot{u}_\theta^i + \sum\nolimits_j S_\theta^{ij,U} - (1-\delta_{iN}) \sum\nolimits_k S_\theta^{(i+1)k,B} = 0
\end{aligned} \tag{2}$$

where S_x^{ij} and S_y^{ij} are the shears of the *j*th column on the *i*th floor; $S_\theta^{ij,U}$ denotes the torsional resistance of column *j* on the *i*th floor to its upper floor; $S_\theta^{(i+1)k,B}$ is the torsional resistance of column *k* on the (*i*+1)th floor to its lower floor; δ_{iN} is the Kronecker delta; and *N* is the total number of floors. It is further assumed that inertial resistance to rotation at column-end nodes can be neglected, thereby in dynamic analysis the rotational DOF's at the column-end nodes can be eliminated by static condensation. The equations of motion for the building structure are derived as follows:

$$\mathbf{M}\ddot{\mathbf{u}}_a + \mathbf{T}_s \mathbf{S} = -\mathbf{M}\ddot{\mathbf{u}}_g \tag{3}$$

in which $\mathbf{M}$ is the lumped-mass matrix including the masses and rotational moments of inertia of floor diaphragms; $\ddot{\mathbf{u}}_g$ is the vector of ground accelerations; **S** is the vector of column-end shears; $\mathbf{T}_s$ is the matrix that sums the column-end shear forces acting at a particular floor diaphragm, as well as the torsion induced by those shear forces. Note that damping forces, which have not been included, will be added later.

Discrete Inelastic Hinge Model

The bi-axial interaction effect is generally negligible in beams, whereas it may be significant in columns. Therefore, it is assumed that only one inelastic hinge can form at each beam end and two at each column end. Those deformations constitute a vector of hinge rotations **h**. The rotational deformations of hinge elements at a joint are generally different from one to another. A member-end rotation at a joint is equal to the joint rotation minus the rotation of the connecting plastic hinges. Thus, the vector of member-end forces is expressed as

$$\begin{Bmatrix} \mathbf{S} \\ \mathbf{R} \\ \mathbf{H} \end{Bmatrix} = \begin{bmatrix} \mathbf{K}_{SS} & \mathbf{K}_{SC} \\ \mathbf{K}_{RS} & \mathbf{K}_{RC} \\ \mathbf{K}_{HS} & \mathbf{K}_{HC} \end{bmatrix} \begin{Bmatrix} \mathbf{u}_a \\ \mathbf{u}_c \end{Bmatrix} - \begin{bmatrix} \mathbf{K}_{SH} \\ \mathbf{K}_{RH} \\ \mathbf{K}_{HH} \end{bmatrix} \{\mathbf{h} - \mathbf{z}\} \tag{4}$$

in which **z** is the inelastic component of hinge rotation; **S** = column-end shears; **R** = member-end moments at ends without hinges; and **H** = moments at hinged member-end; the two stiffness matrices in the square brackets relate the member-end forces to the global and hinge displacements. By equilibrium, the member-end moments **H** should be equal to the hinge-element moments, i.e.,

$$\mathbf{H} = \mathbf{K}_e \mathbf{h} + \mathbf{K}_z \mathbf{z} \tag{5}$$

where $\mathbf{K}_e$ and $\mathbf{K}_z$ are the elastic and inelastic stiffness matrices of hinge moments. Since rotational masses at joints are neglected, rotational equilibrium at joints may be expressed in terms of only the member-end moments as follows:

$$\mathbf{T}_R \begin{Bmatrix} \mathbf{R} \\ \mathbf{H} \end{Bmatrix} = \{\mathbf{0}\} \tag{6}$$

where the matrix $\mathbf{T}_R$ sums up the end moments present at a particular joint. Defining the following variables,

$$\mathbf{K}_{ES} = \begin{bmatrix} \mathbf{K}_{RS} \\ \mathbf{K}_{HS} \end{bmatrix}, \quad \mathbf{K}_{EC} = \begin{bmatrix} \mathbf{K}_{RC} \\ \mathbf{K}_{HC} \end{bmatrix}, \quad \mathbf{K}_{EH} = \begin{bmatrix} \mathbf{K}_{RH} \\ \mathbf{K}_{HH} \end{bmatrix} \tag{7}$$

and from Eqs.(3)–(6), one obtains

$$\begin{aligned} &\mathbf{Q}_1 \mathbf{u}_a + \mathbf{Q}_2 \mathbf{h} + (\mathbf{K}_z + \mathbf{K}_e - \mathbf{Q}_2)\mathbf{z} = \mathbf{0} \\ &\mathbf{M}\ddot{\mathbf{u}}_a + \mathbf{Q}_3 \mathbf{u}_a + \mathbf{Q}_4 \mathbf{h} = \mathbf{Q}_4 \mathbf{z} - \mathbf{M}\ddot{\mathbf{u}}_g \end{aligned} \tag{8}$$

in which

$$\begin{aligned} \mathbf{Q}_1 &= \mathbf{K}_{HC} (\mathbf{T}_R \mathbf{K}_{EC})^{-1} \mathbf{T}_R \mathbf{K}_{ES} - \mathbf{K}_{HS} \\ \mathbf{Q}_2 &= \mathbf{K}_e + \mathbf{K}_{HH} - \mathbf{K}_{HC} (\mathbf{T}_R \mathbf{K}_{EC})^{-1} \mathbf{T}_R \mathbf{K}_{EH} \\ \mathbf{Q}_3 &= \mathbf{T}_S \left[\mathbf{K}_{SS} - \mathbf{K}_{SC} (\mathbf{T}_R \mathbf{K}_{EC})^{-1} \mathbf{T}_R \mathbf{K}_{ES} \right] \\ \mathbf{Q}_4 &= \mathbf{T}_S \left[\mathbf{K}_{SC} (\mathbf{T}_R \mathbf{K}_{EC})^{-1} \mathbf{T}_R \mathbf{K}_{EH} - \mathbf{K}_{SH} \right] \end{aligned} \tag{9}$$

If Rayleigh damping is assumed and has the following form

$$\mathbf{C} = a_0 \begin{bmatrix} \mathbf{M} & \mathbf{0} \\ \mathbf{0} & \mathbf{0} \end{bmatrix} + a_1 \begin{bmatrix} \mathbf{Q}_3 & \mathbf{Q}_4 \\ \mathbf{Q}_1 & \mathbf{Q}_2 \end{bmatrix} \tag{10}$$

in which a_0 and a_1 are proportionality constants, one obtains the equations of motion including damping as:

$$\begin{bmatrix} \mathbf{M} & \mathbf{0} \\ \mathbf{0} & \mathbf{0} \end{bmatrix} \begin{Bmatrix} \ddot{\mathbf{u}}_a \\ \ddot{\mathbf{h}} \end{Bmatrix} + \left(a_0 \begin{bmatrix} \mathbf{M} & \mathbf{0} \\ \mathbf{0} & \mathbf{0} \end{bmatrix} + a_1 \begin{bmatrix} \mathbf{Q}_3 & \mathbf{Q}_4 \\ \mathbf{Q}_1 & \mathbf{Q}_2 \end{bmatrix} \right) \begin{Bmatrix} \dot{\mathbf{u}}_a \\ \dot{\mathbf{h}} \end{Bmatrix} + \begin{bmatrix} \mathbf{Q}_3 & \mathbf{Q}_4 \\ \mathbf{Q}_1 & \mathbf{Q}_2 \end{bmatrix} \begin{Bmatrix} \mathbf{u}_a \\ \mathbf{h} \end{Bmatrix} = \begin{Bmatrix} \mathbf{Q}_4 \mathbf{z} - \mathbf{M} \ddot{\mathbf{u}}_g \\ (\mathbf{Q}_2 - \mathbf{K}_e - \mathbf{K}_z)\mathbf{z} \end{Bmatrix} \tag{11}$$

Note that to solve Eq. (11), the inelastic moment-rotation relationship is needed. The smooth hysteretic model by Wang and Wen [1998] is used for this purpose, which has been shown to reproduce well test results of brittle connections. One can then solve numerically for the displacement and hinge rotation (elastic and inelastic) vectors, $\mathbf{u}_a$, $\mathbf{h}$, and $\mathbf{z}$, and their associated derives for given initial conditions to obtain the response time history.

NUMERICAL EXAMPLES

A two-story and a three-story steel building, shown in Fig. 1, are designed in compliance with the practice before Northridge earthquake. According to the code provisions for accidental torsion, a 5% offset of the mass centers from the geometrical centers is assumed. The first three vibration periods of the two-story building with flexible floor diaphragms, are 0.615 (Y), 0.498 (X), and 0.310 seconds (torsion). For the three-story building, the periods are and 0.851 (X), 0.845 (Y), and 0.495 seconds (torsion). The damping ratios of the first two modes are assumed to be 2%. The degradation parameters in the smooth hysteretic model, A, ν, and η are 0, 0.02, and 0.1, respectively [Wang and Wen 1998]. P-Δ effects are taken into consideration.

Material uncertainties are considered by assuming that the yield stress of steel F_y is log-normally distributed and the plastic moduli of member cross-sections Z_x and Z_y are normally distributed random variables [Kennedy and Baker 1984]. For A36 steel (beams) and Grade 50 steel (columns), the mean values of F_y are assumed to be 282 MPa and 345 Mpa, respectively. Both have a coefficient of variation of 0.0848. The bias (ratio of mean to nominal) of plastic moduli has a mean value of 0.99 and a coefficient of variation of 0.0396. The connection capacity against fracture under random loads is modelled by the Park and Ang [1986] damage index, assumed to be uniformly distributed between 0.1 and 2.3 [Wang and Wen 1999] based on limited available test data and statistically independent from one connection to another. Fracture is assumed to occur at beam bottom-flanges only. Uncertainty in reserve strength of fractured columns is accounted for by reducing the cross-sections moments of inertia to 0 to 100% of their original values, uniformly distributed.

Effect of Flexible Diaphragms

Since a large number of pre-Northridge designs were of strong-column, weak-beam type, the assumption of rigid diaphragms becomes questionable. It is of interest to study the impact of diaphragm flexibility. The first three structural periods of the two-story building with rigid diaphragms are 0.396(Y), 0.338(X), and 0.207 seconds (torsion). SAC-2 ground motions of high intensity for Los Angeles, California [LA27 (fault-normal) and LA28 (fault parallel), shown in Fig. 2] are used as excitations. The maximum responses of the building with rigid and flexible diaphragms are listed in Table 1. The maximum column drift ratio (MCDR) is used as the measure of structural performance. It is defined as the maximum value among all column drifts, each is the maximum value of the vector sum of drift ratios in the two orthogonal directions throughout the time history. The response histories of the 1st-floor mass center are also shown in Fig. 2. One can readily see the effects of structural period lengthening and, in this case, amplification in responses and permanent displacements due to the flexibility of the diaphragms. It is seen that rigid diaphragm assumption for strong-column, weak-beam steel buildings may significantly underestimate the structural responses.

Effects of Bi-axial Excitation, Torsional Motion, and Column Damage

To cover a wide range of excitation intensity and at the same time to reduce record-to-record response variation, the median response are calculated under the suites of SAC-2 ground motions for Los Angeles, California [SAC 1997] at three probability levels, 50%, 10%, and 2% probability of exceedance in 50 years. To consider the effects of bi-axial excitation, torsional motion, and column damage on structural response, the following seven cases are investigated:

1. Ductile, 1-Dimensional (D1D): ductile connections (no fracture); only the fault-normal component of ground motion is applied in the weak principal direction of the building.

2. Brittle, 1-Dimensional (B1D): same as Case D1D except connections are brittle.

3. Ductile, No Torsion (DNT): ductile connections; fault-normal and fault-parallel components are applied along the weak and strong directions, respectively of building.; accidental torsion is neglected.

4. Brittle, No Torsion (BNT): same as Case DNT except connections are brittle

5. Ductile, with Torsion (DT): same as Case DNT except a 5% accidental torsion is considered also.

6. Brittle, with Torsion (BT): same as Case DT except connections are brittle

7. Brittle, with Column fracture and Torsion (BCT): same as Case BT; besides, a column connected to any fractured beam connections at its upper end is assumed damaged too.

The structural performance is again examined in terms of MCDR. The median values and coefficients of variation of MCDR's of the two buildings considering the seven cases are tabulated in Tables 2 and 3. The 50-year probability of exceedance of the median MCDR is plotted in Fig. 3.

The results can be summarised as follows:

- Comparing the cases with and without connection fractures, (D1D vs. B1D, DNT vs. BNT, and DT vs. BT), the differences in responses are small. In case BT where both fracture and torsion are considered, only 9% of the connections in the 2-story building, and 19% in the 3-story building fracture in a 2% in 50 years hazard [Wang & Wen 1998]. This indicates that when fractured connections have some residual strength and percentage of fractured connections is not a high, a structure still has adequate resistance against seismic loads.

- Bi-axial excitation by itself does not cause significant response increase in the 2-story building, as can be seen by comparing the MCDR of case D1D with DNT and B1D with BNT. Bi-axial excitation coupled with accidental torsion, however, causes a significant increase in response. Comparing the MCDR between case DNT (3.744) and DT (4.308), BNT (3.871) and BT (4.543) at the 2% probability level, the increases are 15.1% and 17.4%, respectively. It shows that torsional oscillation plays an important role in structural response.

- Bi-axial excitation causes significant increase in response of the 3-story building. At 2% probability level, comparisons of the MCDRs of D1D (3.605) and DNT (5.735), B1D (3.649) and BNT (5.794) , show an increased of about 60%. One contributing factor is that the 3-story building has same vibration periods in the two principal directions, therefore, is susceptible to bi-axial interactions and torsional motions. Such frequency-dependent effects of bi-axial interaction, therefore, need to be considered in structural design.

- Partial column fractures in addition to connection failures cause moderate response increase in the 3-story building, whereas they cause insignificant increase in the 2-story building. It can be attributed to the fact that the three-story building has a higher percentage of columns with damage.

- The moderate response increase caused by partial column fractures (no total failure) may explain the fact that although column damages were found after earthquakes, very few collapsed. Reserve strength of damaged structure members, therefore, plays a pivotal role in collapse prevention.

CONCLUSIONS

Methods of modelling and response analysis are developed for pre-Northridge low-rise steel buildings with connection fractures. Inelastic yielding is assumed to be confined to discrete hinge regions located at the structural member (beam and column) ends. The hysteresis of inelastic hinges is modelled by a smooth connection-fracture hysteresis model. A 3-dimensional model of building structures was developed to account for the effects of bi-axial interaction and torsional motion due to yielding and connection fractures. It successfully models the seismic 3-D response behavior of strong-column weak-beam (SCWB) buildings.

Based on the numerical results of a two-story and a three- story steel building under SAC-2 ground motions, it is found that for SCWB steel buildings, shear-beam models assuming rigid diaphragms may significantly underestimate the responses. Also, torsional oscillation and bi-axial interaction due to bi-axial excitation can cause large increase in the response. Response increases caused by connection fractures per se are only moderate by comparison. Therefore, these 3-D motions deserve careful consideration. Finally, partial column damages due to connection failures cause only moderate increase in structural response. It may explain the fact that although partial column damages were found in many buildings after recent earthquakes, very few collapsed.

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Table 1: Comparison of maximum column drift ratios

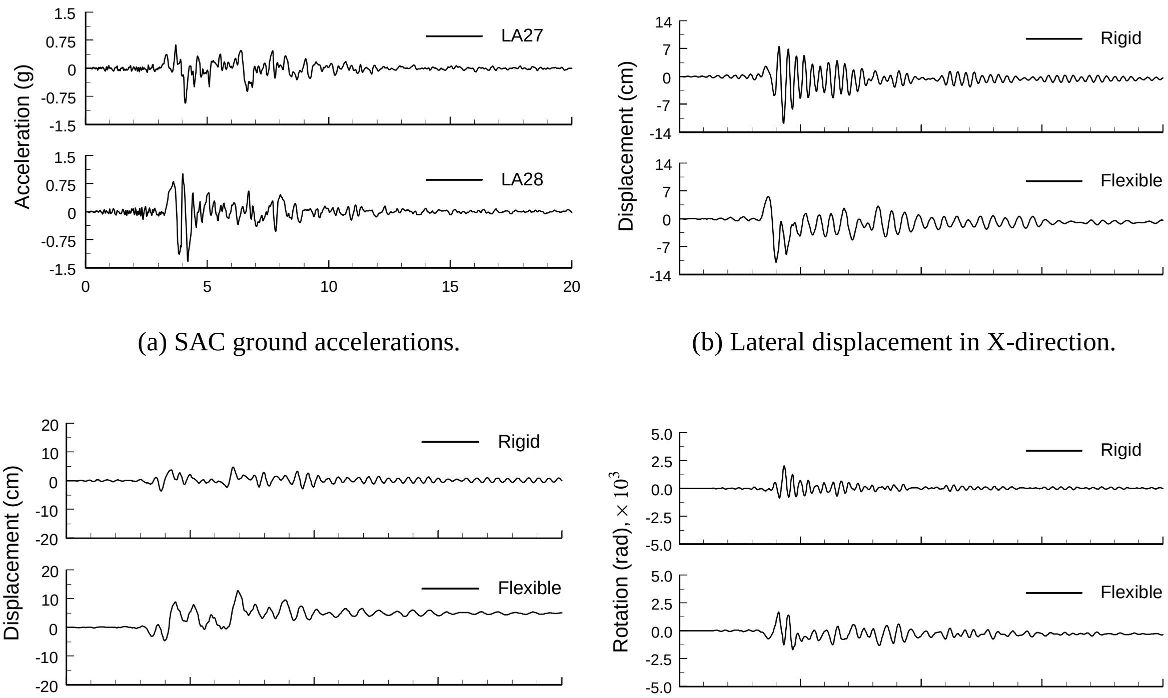
Case	X(%)	Y(%)	MCDR(%)
Rigid	3.012	1.125	3.106
Flexible	4.664	4.023	4.936

Table 2: Medians and coefficients of variation of MCDR (2-story building)

Case	50% in 50 years		10% in 50 years		2% in 50 years	
	Med.(%)	COV	Med.(%)	COV	Med.(%)	COV
D1D	1.059	0.665	1.868	0.335	3.612	0.493
B1D	1.069	0.696	1.900	0.341	3.757	0.505
DNT	1.152	0.621	1.968	0.326	3.744	0.469
BNT	1.160	0.626	2.014	0.339	3.871	0.474
DT	1.229	0.602	2.198	0.349	4.308	0.513
BT	1.240	0.608	2.269	0.383	4.543	0.505
BCT	1.240	0.639	2.215	0.368	4.704	0.596

Table 3: Medians and coefficients of ariation of MCDR (3-story building)

Case	50% in 50 years		10% in 50 years		2% in 50 years	
	Med.(%)	COV	Med.(%)	COV	Med.(%)	COV
D1D	1.085	0.504	1.905	0.293	3.605	0.460
B1D	1.104	0.509	1.922	0.281	3.649	0.497
DNT	1.400	0.393	2.601	0.454	5.735	0.486
BNT	1.398	0.392	2.613	0.437	5.794	0.531
DT	1.485	0.398	2.808	0.398	5.881	0.486
BT	1.490	0.394	2.782	0.384	6.035	0.523
BCT	1.513	0.410	2.832	0.383	6.604	0.547



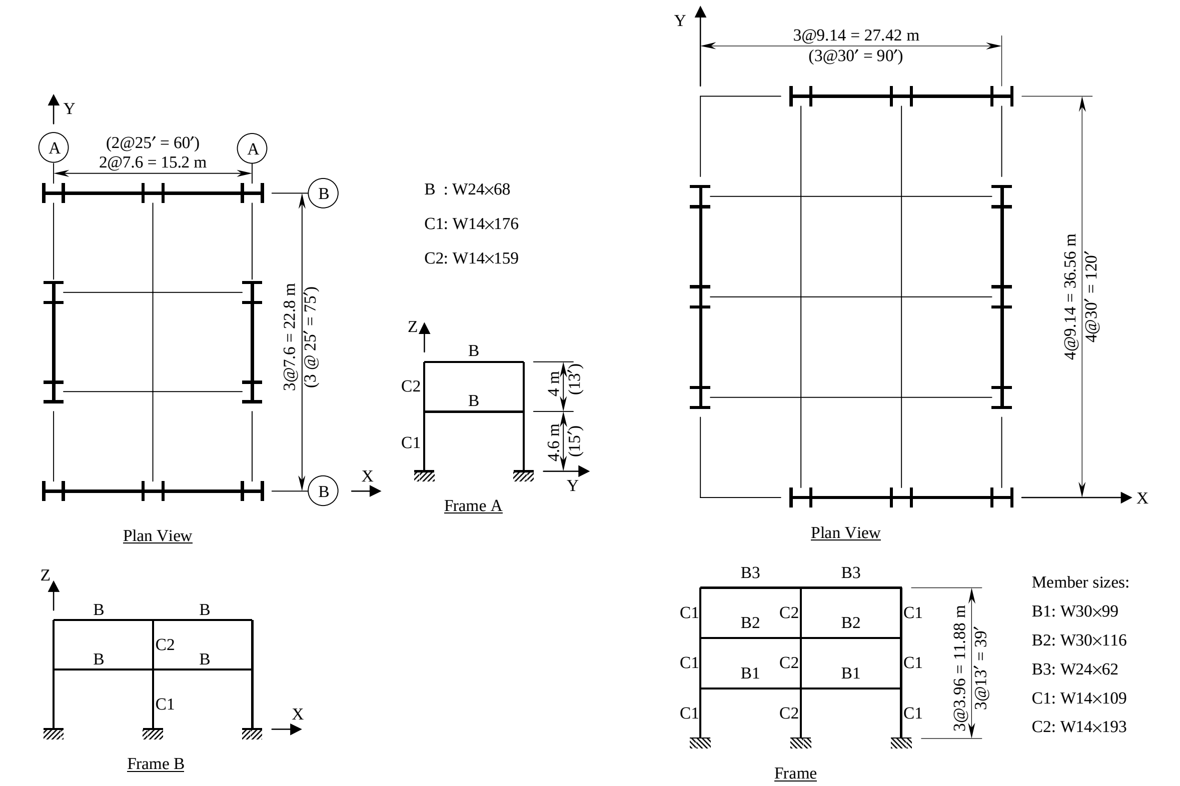
(a) SAC ground accelerations.

(b) Lateral displacement in X-direction.

(c) Lateral displacement in Y-direction.

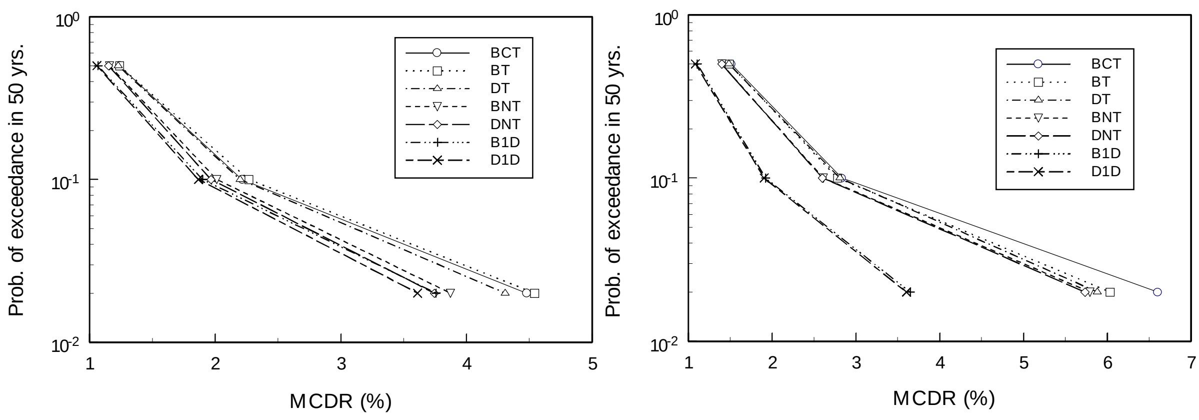
(d) Rotation.

Figure 1: SAC ground motions and displacements of 1st-floor mass center of the two-story building.



(a) Two-story building. (b) Three-story building.

Figure 2: Steel moment frame buildings



(a) Two-story building. (b) Three-story building.

Figure 3: 50-year probability of exceedance of median maximum column drift raito (MCDR) at Los Angeles, California.