DUCTILITY BEHAVIOR OF A STEEL PLATE SHEAR WALL BY EXPLICIT DYNAMIC ANALYZING

P. Memarzadeh

Faculty of Civil Engineering, Islamic Azad University, Najafabad Branch, Iran
Email: p-memar@iaun.ac.ir, parham_memarzadeh@yahoo.com

ABSTRACT :

AISC-341 provides requirements for designing of SPSWs by means of a capacity-design method in which boundary elements are designed for forces corresponding to the fully yield strength of the web plate. In present research, an explicit dynamic analysis is conducted to a typical multi-storey SPSW designed according to AISC requirements. The SPSW model is subjected to a base earthquake acceleration record to simulate the time history response of the structure. The analysis utilizes a finite element method involving both material and geometric nonlinearities. The energy dissipation and ductility behavior of the structure are investigated through time history response of the system.

KEYWORDS: Steel Plate, Shear Wall, Transient, Ductility, Energy Dissipation

1. Introduction

Before the 1960s the ductility notion was used only for characterizing the material behavior. After the Housner's studies of earthquake problems, this concept has been extended to a structure level (Gioncu and Mazzolani, 2002). In the design of buildings in seismic areas, the ductility provides an evaluation of the performance of structure by indicating the quantity of energy absorbed through plastic deformations.

A Steel Plate Shear Wall (SPSW) is a lateral-load-resisting system consisting of steel web plates connected to the surrounding beams and columns and installed along the height of the structure. Web plates in SPSW provide a large area of steel to dissipate the earthquake-induced energy by yielding. They are not required for the resistance of gravity loads, and so, damage in the web plates is preferable to damage in the boundary elements which supports the building weight. Therefore, AISC 341-05 (2005) provides requirements to confine ductility demands to the web plates (Sabelli and Bruneau, 2007).

The distribution of ductility demand between floors requires that web plates be designed with similar overstrength. Non-similarity of the overstrengths may causes the web plates at some levels do not participate in providing system ductility, and the seismic drift demands on the other levels will be greater (Sabelli and Bruneau, 2007). Thus, it is recommended to proportion the web plates to storey shear a closely as possible and not to provide unnecessary overstrength (Berman and Bruneau, 2003).

The objective of this study is the evaluation of the ductility imposed on the storey plates of a typical SPSW when subjected to El Centro earthquake record. In addition, other seismic performances of the building including storey displacements and drifts, as well as, interstorey shear and internal forces over the height of VBE are evaluated. The subject of the investigation is a typical nine-storey SPSW building designed according to AISC (2005) requirements (Sabelli and Bruneau, 2007).

2- Formulations for Explicit Dynamic Analysis

The present dynamic analysis implements an explicit integration rule together with the use of diagonal or "lumped" element mass matrices resulted in the computational efficiency of the explicit procedure. Using the simple inversion of the lumped mass matrix, the accelerations at the beginning of the increment are computed from the equations of motion (Eqn. 2.1) as below (Bathe 1996):

$$\ddot{\mathbf{U}}^{(i)} = \mathbf{M}^{-1}(\mathbf{F}^{(i)} - \mathbf{I}^{(i)}) \tag{2.1}$$

Then, the equations of motion (Eqn. 2.1) for the body are integrated using the explicit central difference integration rule:

$$\begin{aligned} \dot{\mathbf{U}}^{(i+\frac{1}{2})} &= \dot{\mathbf{U}}^{(i-\frac{1}{2})} + \frac{\Delta t^{(i+1)} + \Delta t^{(i)}}{2}\ddot{\mathbf{U}}^{(i)} \\ \mathbf{U}^{(i+1)} &= \mathbf{U}^{(i)} + \Delta t^{(i+1)}\,\dot{\mathbf{U}}^{(i+\frac{1}{2})} \end{aligned} \tag{2.2}$$

where $\dot{\mathbf{U}}$ and $\ddot{\mathbf{U}}$ are velocity and acceleration vectors evaluated at nodes. $(i-\frac{1}{2})$ and $(i+\frac{1}{2})$ refer to midincrement values. The central difference integration operator is explicit in that the kinematic state can be advanced using known values of $\dot{\mathbf{U}}^{(i-\frac{1}{2})}$ and $\ddot{\mathbf{U}}^{(i)}$ from the previous increment.

Special treatment of the mean velocities $\dot{\mathbf{U}}^{(i+\frac{1}{2})}, \dot{\mathbf{U}}^{(i-\frac{1}{2})}$ etc. is required for initial conditions, certain constraints and presentation of results. For presentation of results, the state velocities are stored as a linear interpolation of the mean velocities:

$$\dot{\mathbf{U}}^{(i+1)} = \dot{\mathbf{U}}^{(i+\frac{1}{2})} + \frac{1}{2}\Delta t^{(i+1)}\,\ddot{\mathbf{U}}^{(i+1)} \tag{2.3}$$

The value of the mean velocity $\dot{\mathbf{U}}^{(-\frac{1}{2})}$ needs to be defined to start the central difference operator. The initial values (at time $t = 0$) of velocity and acceleration are set to zero. We assert the following condition:

$$\dot{\mathbf{U}}^{(+\frac{1}{2})} = \dot{\mathbf{U}}^{(0)} + \frac{\Delta t^{(1)}}{2}\ddot{\mathbf{U}}^{(0)} \tag{2.4}$$

Substituting this expression into the update expression for $\dot{\mathbf{U}}^{(I+\frac{1}{2})}$ yields the following definition of $\dot{\mathbf{U}}^{(-\frac{1}{2})}$:

$$\dot{\mathbf{U}}^{(-\frac{1}{2})} = \dot{\mathbf{U}}^{(0)} - \frac{\Delta t^{(0)}}{2}\ddot{\mathbf{U}}^{(0)} \tag{2.5}$$

3- SPSW Structure

The geometry and section properties of the SPSW structure considered in this investigation are presented in figure 1. This structure is a 9-storey 3-bay frame with infill plates in the second bay's panels. The x-translation inertia of the frame is 5,440 kips distributed equally between the first to ninth floors. The structure has been designed by Sabelli and Bruneau (2007) according to AISC 341-05 (2005) for the lateral earthquake forces specified using the equivalent lateral force procedure.

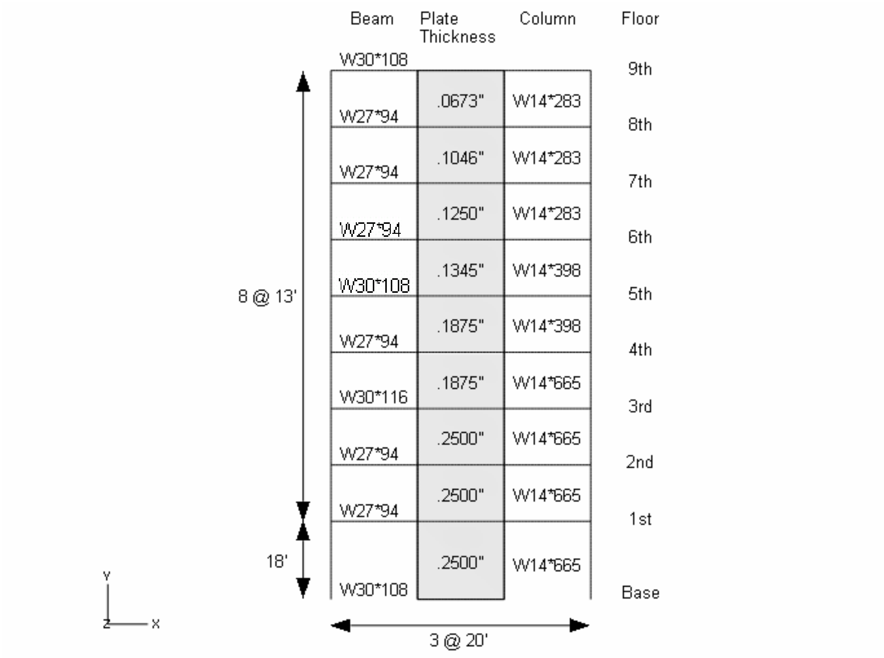


Figure 1 Dimensions and properties of SPSW building (Sabelli and Bruneau, 2007)

4- Description of Finite Element Model

4.1. Finite Elements and Meshing

The boundary members are modeled by use of the 2-node finite beam element with linear interpolation formulations in three dimensional space. Totally, 25 section points are specified to use for integration; 9 points in web, 9 in each flange. Also, the 4-node doubly-curved shell element is used to model the web plates. This element uses linear interpolation and thick shell theory for finite membrane strains and arbitrary large rotations.

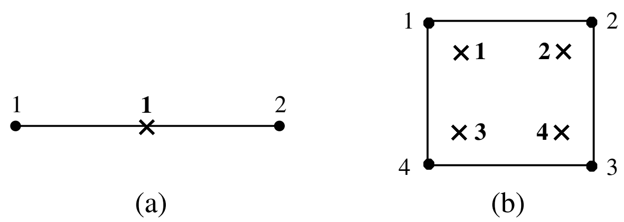


Figure 2 Integration point in (a) 2-node beam element and (b) 4-node shell element

A mesh of 20 by 13 elements (20 elements over the width of the shear wall) is used to model the infill plates except for the first storey infill plate whose mesh is 20 by 18 elements. The length of the beam elements is selected to match the mesh size in the infill plates. Seven Gaussian integration points through the thickness of the shell elements are used. Figure 2 illustrates integration points in the beam and shell elements.

4.2. Initial Imperfections, Boundary Conditions and Material Properties

The infill plates are taken to have an initial imperfection pattern corresponding to the deformation of a clamped edges similar plate loaded by normal pressure on surface of the plate. The peak amplitude for the first storey panel's out-of-plane deformation is set at 0.0183 inch whereas that value for the other plates is 0.006 inch. These values are around 2 to 9 percent of the corresponding plate thicknesses.

All the nodes at the base of the SPSW model are fixed to simulate the rigid boundary condition at the base of the shear wall. In addition, in order to prevent the local distortion and the out-of-plane deformation at the floor levels, all the nodes at these levels are restrained at u_z and θ_y degrees of freedom.

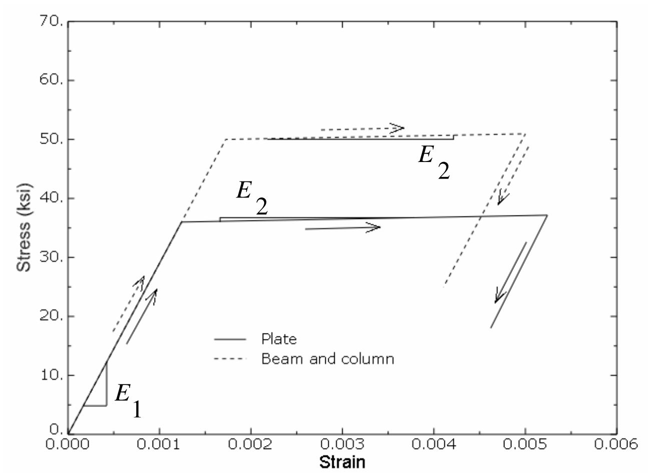


Figure 3 The assumed stress-strain relationship of the material

A kinematic hardening model is used for dynamic analysis. Material properties of the boundary members and the plates are assumed with the following specifications:

$$
\begin{aligned}
& E_1 = 29{,}000\,(ksi),\ E_2 = 290\,(ksi) \\
& \nu = 0.3,\ \rho = 489\,(lb/ft^3) \\
& F_{yp} = 36\,(ksi),\ F_{yb} = 50\,(ksi)
\end{aligned}
\tag{4.1}
$$

in which E, ν and ρ are Young's modulus, Poison's ratio and density of steel material, and F_{yp}, F_{yb} are the plate and boundary member yield stresses, respectively. All beam and column material properties are the same as the boundary member properties. Figure 3 shows the assumed stress-strain relationship of the plate, beam and column materials.

5- Validation of the Finite Element Model

In order to evaluate the validity of the model, the finite element model of the SPSW structure was subjected to a sinusoidal force at the roof level with a frequency of $0.9043\,Hz$ (equal to frequency of the first vibration mode) for a time interval of 5 seconds and then, it was allowed to freely vibrate. The lateral displacements of some floors are shown in figure 4. This figure shows a resonance in the first 5 seconds interval and also decreases of the free vibration amplitudes due to function of the damping. Also, some analyses were performed to checking the efficiency of the degree of the meshing and the results confirmed that for the various energy quantities, shear forces as well as the displacements and accelerations.

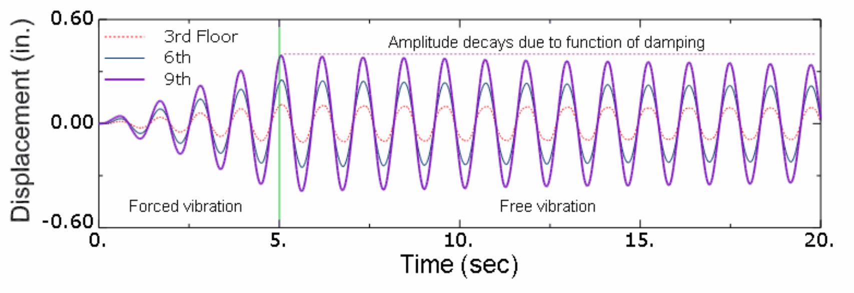


Figure 4 Lateral displacements of floors

6- Numerical Results of the Explicit Dynamic Analysis

The finite element model of the SPSW structure is subjected to a base acceleration recorded in the El Centro earthquake of May 18, 1940 (N-S component). The nonlinear dynamic response to this excitation in computed by digital computer, using the explicit dynamic procedure. The complete response history is determined during the 30 seconds of the earthquake (Figure 5).

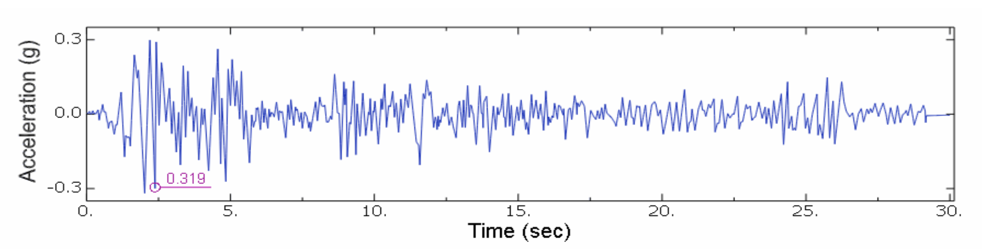


Figure 5 El Centro earthquake, May 18, 1940, N-S component.

6.1. Energy Balance

Energy output is particularly important in checking the accuracy of the solution in an explicit dynamic analysis. An energy balance for the entire model can be written as:

$$E_{Total} = E_I + E_K + E_V - E_W \tag{6.1}$$

where E_{Total} is the total energy, E_I, E_K, E_V and E_W are the total strain energy, kinetic energy, energy dissipated by viscous effects and the external work, respectively. Total strain energy (E_I) is defined as below:

$$E_I = E_s + E_p + E_a \tag{6.2}$$

in which E_s, E_p and E_a are the recoverable strain energy, plastic dissipated energy and artificial strain energy, respectively. The artificial strain energy associated with constraints used to remove singular modes (such as hourglass control) should be negligible compared to real energies such as kinetic energy (E_K) and recoverable strain energy (E_s).

Figure 6 illustrates the energy time histories for the entire SPSW model. As shown, the values of the total energy and the artificial strain energy of the model are negligible comparing to the input energy allover the time history; these confirm an energy balance in the system and imply that the solution enjoys an acceptable accuracy. The results of figure 6 also shows that eventually the SPSW structure dissipates by viscous damping and yielding all the energy supplied to it.

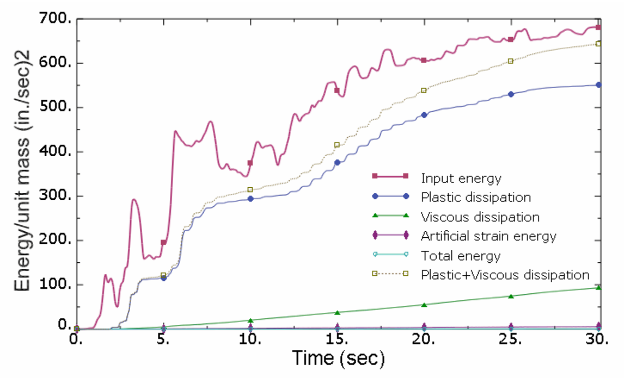
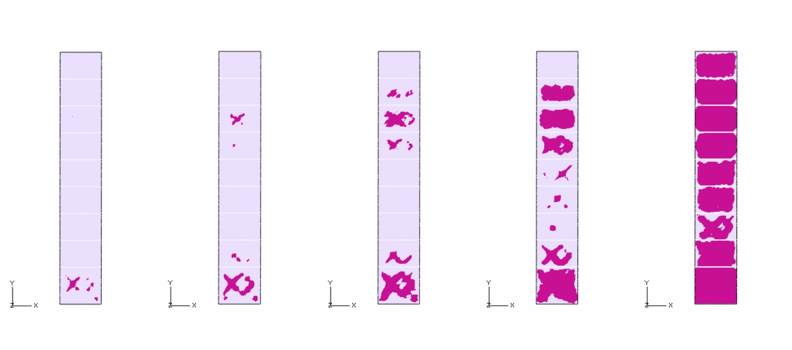


Figure 6 Energy time histories of the SPSW model



(a) (b) (c) (d) (e)

Figure 7 Areas of plates where permanent equivalent plastic strains at end of the analysis exceed the values of (a) 0.04, (b) 0.03, (c) 0.02, (d) 0.01 and (e) 0.001.

Figure 7 illustrates areas of the SPSW storey plates where permanent equivalent plastic strains at the end of the analysis exceed the values of 0.04, 0.03, 0.02, 0.01 and 0.001. This figure reveals that, as expected, a large area of the each plate or whole experiences significant plastic strains.

6.2. Tension Field

When the buckling occurs in the infill plate of a SPSW, a diagonal tension field is formed through the plate (Memarzadeh, et al., 2007). Fig. 8 illustrates three steps of principal stresses behavior, as the tension field reverses. Fig. 18 also shows the procedure of reversing the inclination of the shear waves. As seen, the inclination angle of both the tension field and shear waves are about 45°. Fig. 9 shows the Procedure of reversing inclination of the shear waves.

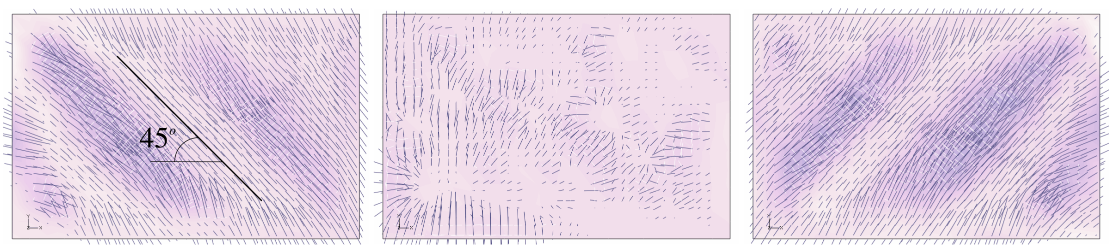


Figure 8 Variations of principal stress orientations while reversing the tension field.

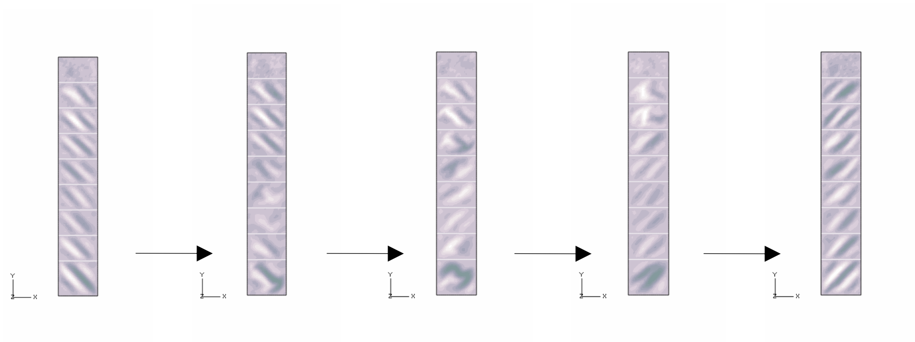


Figure 9 Procedure of reversing inclination of the shear waves.

6.3. Energy Dissipation and Ductility Demands

Hysteretic energy includes cumulative effects of repeated cycles of inelastic response and, therefore, the effects of earthquake duration are included in this quantity (Bozorgnia and Bertero, 2002). This paper proposes a definition for the energy ductility, μ_e, which is convenient to use for the case of dynamic response of SPSWs as follow:

$$\mu_e = \frac{Max(E_p + E_s)}{Max(E_s)} \tag{6.3}$$

where E_p and E_s are the hysteretic and recoverable strain energy, respectively, as defined previously.

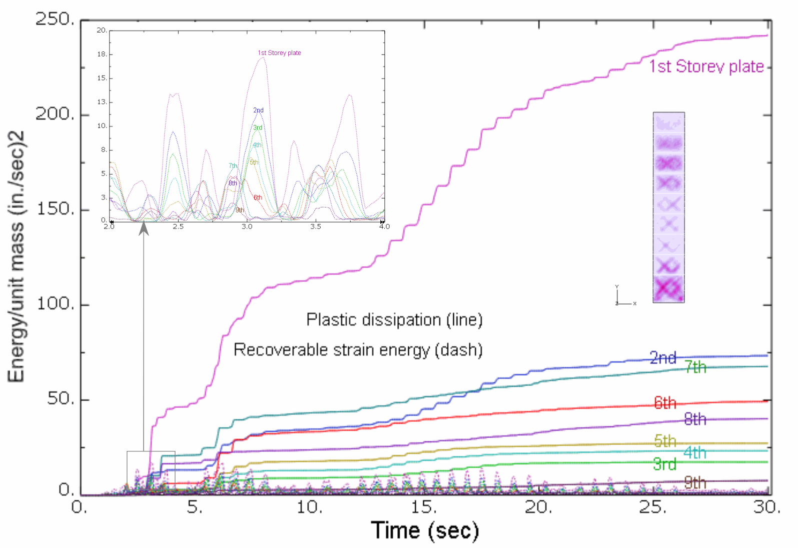


Figure 10 Histories of Plastic dissipation and recoverable strain energy for storey plates of the SPSW structure.

This proposed energy ductility factor, μ_e, is numerically equal to the displacement ductility of a monotonically loaded SDOF system that dissipates the same energy (Mahin and Bertero, 1981). Figure 10 gives a comparison between the plastic dissipation (hysteretic energy) and the recoverable strain energy for the storey plates of the SPSW structure. According to Eqn. 6.3, the energy ductility demands for the storey plates of the SPSW structure are shown in Figure 11. As shown, the energy ductility demands of the storey plates are in a range between 2.25 and 14.25. The relative wide range of the ductility factors implies a non-uniform distribution of the ductility over the height of the structure, as the contributions of the 3rd, 4th, 5th, and 9th storey plates to ductility of the structure are less than those for the other plates (Figure 11). This result is confirmed by Figures 10.

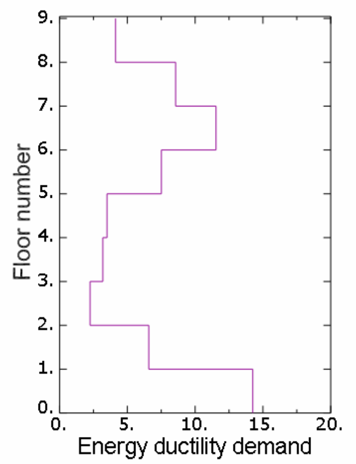


Figure 11 Energy ductility demands of storey plates and distribution of them over height of the structure.

6. Summary

This paper investigates energy dissipation requirements of a typical SPSW structure designed according to AISC 341 (Sabelli and Bruneau 2007). A finite element model of the structure is subjected to the El Centro earthquake and is analyzed by the explicit dynamic procedure to evaluate the energy dissipation of the components of the system. This paper also proposes a new definition of the energy ductility which is convenient to use for the case of dynamic response of SPSWs. According to this definition, the energy ductility factors of the web plates are values between 2.25 to 14.25. The relative wide range of the ductility factors implies a non-uniform distribution of the ductility over the height of the structure. This may be due to non-similarity of the plate overstrengths.

Acknowledgment
The author wishes to thank Professor M.M. Saadatpour at Isfahan University of Technology for helpful supervising this research.

REFERENCES

AISC (2005). ANSI/AISC 341-05, Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction Inc., Chicago, IL.
Bathe, K.J. (1996). Finite Element Procedures, Prentice-Hall, Englewood Cliffs, NJ.
Berman, J.W., Bruneau, M. (2003). Plastic analysis and design of steel plate shear walls. *ASCE Journal of Structural Engineering* **129:11**, 1448-1456.
Bozorgnia, Y., Bertero, V.V. (2002). Improved damage parameters for post-earthquake application. *Proceedings of SMIP02 Seminar on Utilization of Strong-Motion Data*. May 2, Los Angeles, California.
Gioncu, V., Mazzolani, F.M. (2002). Ductility of Seismic Resistant Engineering Structures, Spon Press.
Mahin, S.A, and Bertero, V.V. (1981). An evaluation of inelastic seismic design spectra. *ASCE Journal of Structural Division* **107:ST9**, 1777- 1795.
Memarzadeh, P., Azhari, M., and Saadatpour, M.M. (2007). Principle stress behaviour of a steel plate shear wall concerning buckling modes, *6th Int Conf Earthquake Resistant Engineering Structures*, June 11-13, Bologna, Italy.
Sabelli, R., Bruneau, M. (2007). Steel Plate Shear Walls, ASCE Steel Design Guide 20, USA.