Paper No. TS-3-1

NUMERICAL STUDIES OF SEISMIC PERFORMANCE OF SPECIAL PLATE SHEAR WALLS UNDER LONG DURATION EARTHQUAKES

Carlos MATOS[1] and Karina CARBAJAL [2]

SYMMARY

Special plate shear wall (SPSW) is a novel Earthquake-resistant alternative, with excellent ductility and energy dissipation; easy to be replaced if damaged after a major seismic event. Long duration earthquakes as the last Japan's earthquake of 2011 present new challenges. Some of them, relevant to buildings with SPSW are addressed here. This study tries to quantify SPSW inelastic behavior under long duration earthquakes. In a previously designed building, concrete shear walls were replaced by steel plates. The new system was analyzed and designed using response-spectrum obtained from actual ground motion records from Tohoku Earthquake. Finally, we perform push-over analyses to investigate behavior under extreme loads. Steel is modeled as elastoplastic and isotropic hardening has been considered. SPSW (Beams, columns and walls) are discretized with shells elements, this allows seeing the distribution of plasticity on beams, which cannot using traditional concentrated plasticity models. Simo's Radial Return algorithm for numerical integration of plasticity constitutive equations was used. We compare numerically obtained capacities against capacity provided by AISC 341. We compute triaxiality indexes to evaluate probability of fracture, since this is a key parameter to determine fracture during earthquakes. Results show that steel plates yielded extensively without signs of brittle fracture.

INTRODUCTION

Shear walls in buildings have been the main lateral resistant system used in our country and in most part of the world. It has been tested during major earthquakes. However, there are several shortcomings of the system. Especially there is a concern to repair damaged shear walls. The Special plate shear wall (SPSW) is a feasible novel alternative, that can be easily repaired or replaced if damaged in a major event, also allows expediting the construction of buildings since no form-work is required. There is currently an extensive research of SPSWs in USA and Canada. The concept of the SPSW is based on plate girders used for long span bridges developing post-buckling, developed during early sixties by Basler [6]. SPSW is highly resistant to the type of damage that would reduce structural integrity. SPSW is designed to develop large inelastic deformation and keep elastically boundary elements. Experimental tests of SPSW show that the system is very ductile and dissipates energy; developing large inelastic deformation before failure [8]

[1] Senior Structural Engineer, JEG, USA.

[2] Structural Engineer, Carbajal y Asociados, Perú.

Long duration earthquakes have been very destructive. Not only for the large input of energy, but for the larger peak accelerations. A particular characteristic of the response spectrum of long duration strong motions has been reported [8], this consist of a second peak located in the range of 2 to 3 seconds. That can be appreciated on the spectrum of Japan Earthquake depicted in Figure 1. For normal earthquakes typically design response spectra is composed of a ramp, a plateau and finally a decreasing curve. Thus for tall buildings and long span bridges the response falls on the decaying part and the lateral system is controlled by wind forces, which would not be the case of structures subjected to long duration earthquakes. The large duration earthquake also produces a large additional number of load cycles, which in many cases take the structure many more times into the plastic range. This known as low-cycle-fatigue, and it can onset and develop fatigue cracks, which is not addressed here.

The effort of this paper is two folded. First, in this study we try to present de design advantages of SPSW system over the traditional Reinforced Concrete shear wall. Second, this study tries to quantify the inelastic effects of long duration earthquakes on SPSW. Our main objectives are to understand the elastoplastic distribution of stresses, investigate the possibility of fracture, and validate AISC design equations. For the elastoplastic analysis, we use the J_2 flow plastic theory. To investigate the possibility of brittle fracture we use triaxiality index [1].

This paper is organized as follows. Section 2 exposes the SPSW and the design requirements of AISC 341. We also present a comparative design of seven story building designed using Reinforced concrete shear walls against the SPSW. In both cases we design for a long duration earthquake spectrum. We conclude the section presenting some key comparative results.

In section 3 we describe the procedure of inelastic finite element analysis used, a few details of the radial return algorithm used to integrate the incremental constitutive differential equations. We also describe the pushover analysis of the SPSW presented in section 2.

Finally in section 4 we present the conclusions and recommendations of the present study.

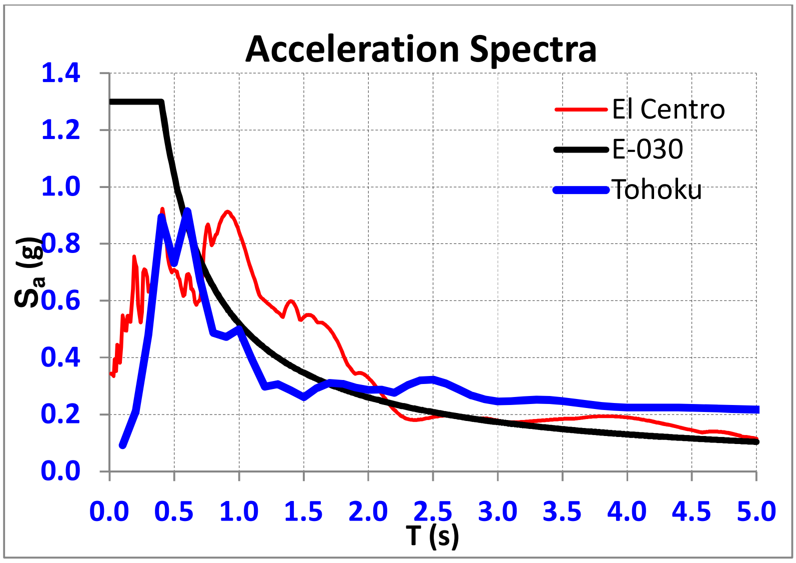

Figure 1. Strong ground motion Tohoku and corresponding Spectrum

SPECIAL PLATE SHEAR WALLS

Description of SPSW

The special steel plate shear wall (SPSW) is a lateral-resisting system consisting of vertical steel plate infill connected to surrounding beams and columns and installed in one or more bays along the full height of the structure to form a cantilevered wall presented in Figure 2(a). When subjected to cyclic inelastic deformations exhibits high initial stiffness, behave in a very ductile manner, and dissipate significant amounts of energy. These characteristics make them suitable to resist seismic loading. SPSW can be used not only for the design of new buildings but also for the retrofit of existing construction. Lateral loads are transferred through the plate by the principal tension stresses. This post buckling behavior is typically referred to as “tension-field action” This is illustrated in Figure 2(c). Lateral loads are resisted through diagonal tension in the web plate or steel plate, rather than in pure shear. Boundary elements are designed to permit the web plates to develop significant diagonal tension; for high seismic design, this is to permit the web plates to reach their expected yield stress across the entire panel. Beam-to-column connections in SPSW may be either of the simple type or moment-resisting type. Steel plate shear walls have been used in a large number of buildings, including in the United States, Canada, Mexico, and Japan. Building types have ranged from single-family residential to high-rise construction.

Design requirements for SPSW

During the last two decades considerable research on SPSW was carried out in the U.S. Design provisions were included in the 2005 AISC 341 [4], [9]. Main requirements by AISC 341 are the limits on aspect ratio of $0.8<L/h\leq 2.5$ (this allows for fairly wide walls, but not tall and slender walls) and the nominal strength of the web plate

$$V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha. \quad (1)$$

The AISC 341 also requires in order to achieve the high level of ductility expected of SPSW, the boundary members have to be designed to remain elastic as the web plate yields. As a result, size of beams and columns are designed based on the plate thickness. HBE’s and VBE’s have to satisfy the weak beam/strong column requirements for moment frames without consideration of the web plate. The moments in VBE’s caused by the web plate tension field are fairly severe. Similarly, the axial loads caused by the overturning moments from web plates yielding are quite high. The required strength of VBE’s shall be based upon the forces corresponding to the expected yield strength, in tension, of the web calculated at an angle, α. To reach this level of force the beams are required to develop plastic hinges [9]. Plate design, however, is based on angle of inclination of the tension field, which is dependent on the boundary members. The design process, therefore, becomes iterative. Another factor to consider when using thin web plate is the shear that causes the plate to buckle. Plate buckling is expected to occur as the tension field develops, but it is not desirable for service loads

AISC 341 does not limit plate slenderness; instead, the commentary states that story drift limitations will indirectly limit the plate slenderness. Acceptable plate material includes A36 or A1011 Grade 55.

Design Example of a 7-story building

To expose the design methodology, we re-design a Reinforced Concrete building. Shear walls have been replaced by SPSW. We are going to compare both lateral resisting systems. Design of structure with SPSW was based on the AISC 341, AISC 360 and ASCE 7 and RNC. A seven story education Building was selected that has 8-24' bays in one direction and 3-27' bays in the other direction. The seven floor heights are each 9.85'. SAP software was used to analyze the structure. Tohoku Earthquake (long duration) Spectrum was used. Figure 2(b) shows an isometric view of building. An elevation of the SPSW design is shown in Figure 2(a). Steel used is A36.

In the example building shown in Figure 2(b), the 9.85' story heights would require a wall 24' long. SPSW web plate was 24' long with 1/2" thickness. On upper floors could reduce the web plate, but we have worked with the same web plate on all floors, this could be optimized, plate can have more capacity than needed.

Four SPSW panels were required in large direction and seven in the building direction. The VBE design considered the moments and axial forces due to all web plates fully yielding and HBE's developing plastic hinges.

The columns are W18x97 and the beams are W14x109. Shear walls were determined to be the best structural system to accommodate the constraints of high design seismic forces and high stiffness (assumed adequate at the time to protect educational building systems from damage so they could stay in service after a severe earthquake). The shear capacity for the web plate using equation 1 was 931Kips.

We summarize some key findings of this comparative study:

The structure with SPSW system has only 83% of the traditional Reinforced Concrete shear wall system's weight.

Consequently, the building's base shear with SPSW system decreases in 17% and this reduction makes that the structure's demand also decreases in all structural system (walls, beam, and columns).

The shear force in the analyzed wall with SPSW system has only 81% of the Reinforced Concrete shear wall systems' shear force which is suitable in buildings located in high seismicity regions.

The fundamental period of the building with SPSW system is greater that the fundamental period of the same building with the Reinforced Concrete shear wall system. Increasing the fundamental period of the bulging SPSW, it also increases the lateral displacement, but they are still within the limits of the Peruvian seismic code.

On the analyzed shear wall was found that the ratio of lateral stiffness of the Reinforced concrete shear wall and SPSW building was 2.32, it clearly explains why the Reinforced Concrete shear wall building is stiffer.

For the fundamental period (T=0.38s) of the SPSW building, the spectral acceleration of "El Centro" ground motion has been scaled by 1.57, and for "Tohoku" ground motion has been scaled by 1.60. Both records were scaled in order to obtain similar demand levels than the Peruvian seismic code.

For the fundamental period (T=0.46s) of the Reinforced Concrete shear wall building, the spectral acceleration of “El Centro” ground motion has been scaled by 1.55, and for “Tohoku” ground motion has been scaled by 1.42. Both records were scaled in order to obtain similar demand levels than the Peruvian seismic code.

The spectral acceleration of “Tohoku” ground motion has not high demands on short-period buildings, but it has higher demands on long-period buildings, it means that this record should be useful in the analysis and design of tall buildings in our country.

Advantages of the system

SPSW offers significant advantages in terms of cost, performance, and ease of design compared to Reinforce concrete shear walls, the reduced thickness (and thus plan area devoted to them) represents a substantial benefit. The reduced mass can also be significant in the design of the foundation. Thickness of walls is important to the architects too [9] and [11]. Most importantly, however, steel plate shear walls can be erected in significantly less time than concrete shear wall structures, no curing time.

The ductility of steel web plates in SPSW results in good performance under severe seismic loading. Because SPSW can provide significant strength and stiffness, shorter bays can be used. This results in greater flexibility for use of the space.

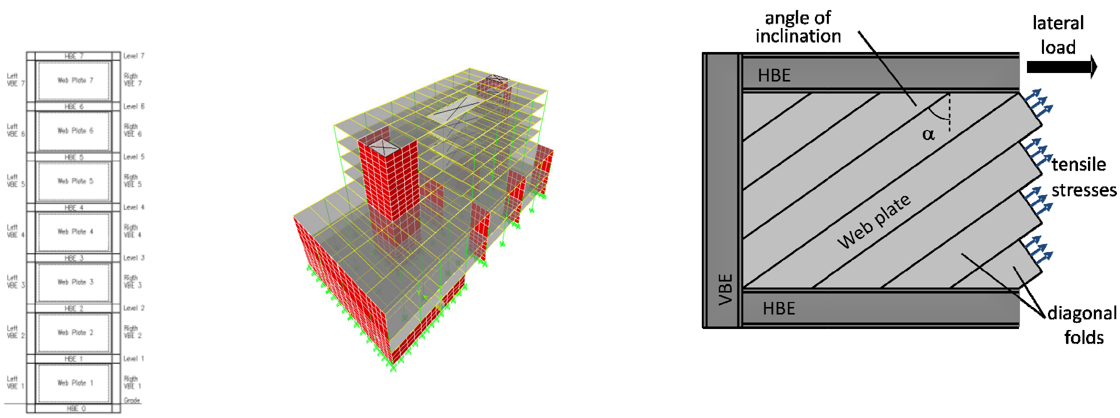

Figure 2. (a) An elevation view of typical Steel plate shear (b) Completed building with special moment frames the traditional Reinforced Concrete shear wall, and (c) SPSW Tension Field

INELASTIC FEM ANALYSIS OF SPSW

To investigate the inelastic behavior of SPSW system we perform nonlinear finite elements analyses. Shell elements are the most appropriated element to model the web plate. Alternatively solid elements could be used if resulting stresses are large and influence of the third dimension is important, although computationally prohibited since several elements through the thickness are required in this bending dominated problem. Shell element used is from the family of nonlinear geometrically exact shells with six degrees of freedom and full integration [2]. Elastoplastic constitutive model is introduced by a J_2 flow plastic rate-independent analysis. Isotropic hardening of steel has been considered.

In a typical load increment, for a global Newton iteration, we solve:

$$K_T \Delta u_{n+1} = P - I_n$$

Where:

- K_T is the tangent stiffness; mathematically the Gateaux derivative of the internal forces or the Hessian of the potential energy [3,6]. $K_T = \int B_n^T D_{ep} B_n$
- D_{ep} is the algorithmic-consistent constitutive model, which is lightly different from the continuum-consistent constitutive model
- P is the external applied load, and $I_n = \int B_n^T \, F d\Omega$ is the internal forces vector. Components of the internal forces vector $N_{\alpha\beta} = \int \sigma_{\alpha\beta} dx_3$, $M_{\alpha\beta} = \int \sigma_{\alpha\beta} x_3 dx_3$, $\alpha, \beta = 1{,}2$ are computed through numerical integration using Gauss-Lobatto with five points of integration. Gauss-Lobatto allows computing the stress on the surface without the necessity of extrapolating results. A minimum of 5 points is required to model elastoplastic loading/unloading.

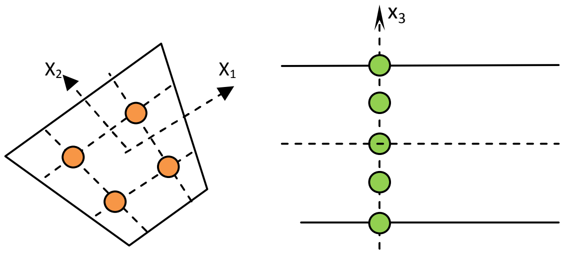

Figure 3. Shell integration points: 1-2 axes (2x2 Gauss points), Axel 3 (5 Gauss-Lobatto points)

Displacements are updated directly by $u_{n+1} = \Delta u_{n+1} + u_n$ and the increment of strains by $\Delta\epsilon_{n+1} = B_{n+1}\Delta u_{n+1}$. Stresses cannot be updated in the same fashion; instead we use the radial return algorithm to integrate plasticity equations i.e. flow rule, hardening law, Kuhn-Tucker loading/unloading conditions (yielding criteria), etc. To compute the current stresses $\sigma_{n+1} = \Re\left(\sigma_n, \Delta\epsilon_{n+1}, \overline{\epsilon_p}, D_{ep}(\Delta\epsilon_{n+1})\right)$. For details see [2], [3], and [5]

Elastoplasticity is a strain driven problem. For 3D or 2D plane-strain, a close-form solution for elastic-perfectly-plastic and for bi-linear plasticity can be derivated. However, this is not the case for 2D-plane stress, where the constrain $\sigma_{33}=0$ must be satisfied. Simo [3] introduced the projection of stresses concept, which make easier and efficient the algorithm. Local Newton iterations are required to satisfied plane-stress constrain. Factors θ_{n+1} *and* $\bar{\theta}_{n+1}$ are introduced in equation (2) to assure the quadratic rate of convergence of global Newton iterations.

$$D_{ep} = \kappa \mathbf{1} \otimes \mathbf{1} + 2\lambda\theta_{n+1}\left[\mathbf{I} - \tfrac{1}{3} 1 \otimes 1\right] - 2\bar{\theta}_{n+1}\boldsymbol{n}_{n+1} \otimes n_{n+1} \quad (2)$$

κ and λ are elastic constants, $n = \frac{dev(\sigma)}{\|dev(\sigma)\|}$ a normal vector, and *dev* is the operator defined by $dev(\circ) = (\circ) - \frac{1}{3} tr(\circ)$

Web plates are typically made of A36 or A572 steel. These are ferritic steels, which are fracture prone if certain adverse conditions are present, e.g. high triaxiality, low temperature, or high loading rates. To quantify the possibility of fracture, the triaxiality index has been used [1]. Once known the stress fields, the triaxiality index can be easily computed.

$$I_T = \frac{\sigma_H}{\sigma_{VM}} \quad (3)$$

Where:

$\sigma_H = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$ is the hydrostatic stress

$\sigma_{VM} = \sqrt{\frac{1}{2}(\sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2)}$ is the Von Mises stress.

Stresses σ_H and σ_{VM} are computed at each Gaussian point from an incremental iterative process. A triaxiality Index I_T of less than one is required to preclude fracture.

APPLICATION TO A 7-STORY SPSSW BUILDING

SPSW designed in previous section has been studied. A push-over analysis has been performed. A monotonically increasing loading has been applied each story level. Distribution of loads comes from resulting forces from spectral analysis. Loads have been applied up to reach the collapse of the SPSW. Numerically, this occurs when the tangent stiffness matrix become singular. Analysis program applied the load increment in an adaptive fashion.

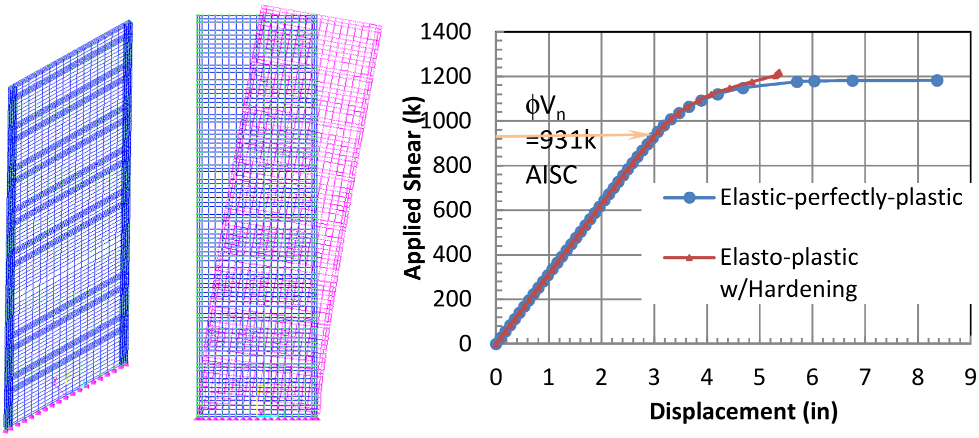

Figure 4. (a) Finite Element Mesh, (b) Deformed at failure load, and c) Applied Shear vs Lateral Displacement

Figure 4(c) describes the global force-displacement at the seventh story level. It can be seen the elastoplastic global response. The inclusion of hardening adds a bit of capacity. We compare the capacity load obtained by AISC 341, this value falls near the elastic limit.

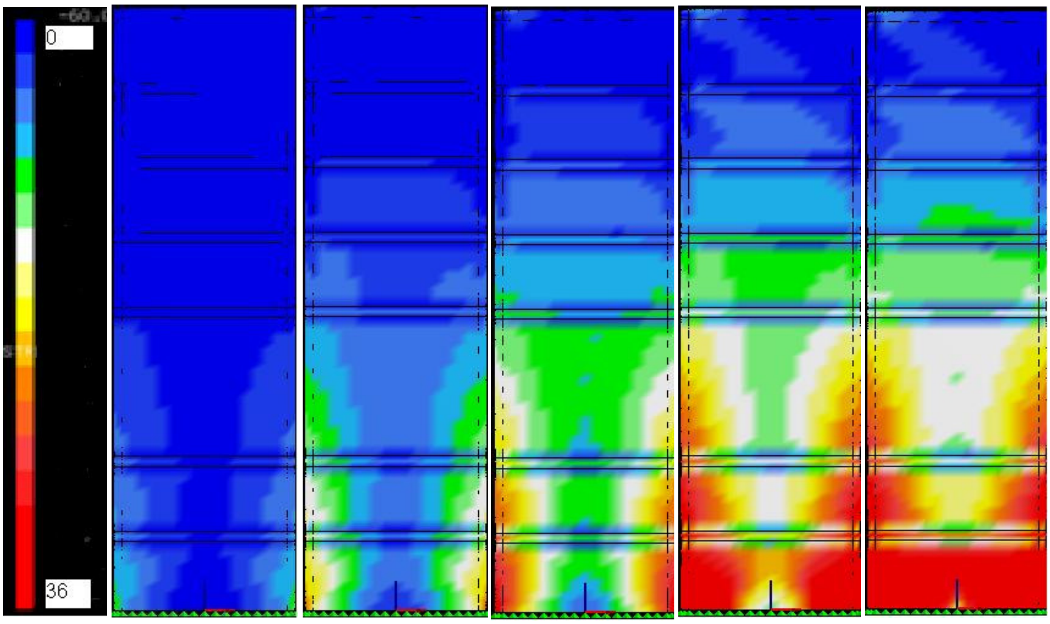

Figure 5. Evolution of Von Mises stresses at 24%, 47%, 71%, 95%, and 100% of failure load

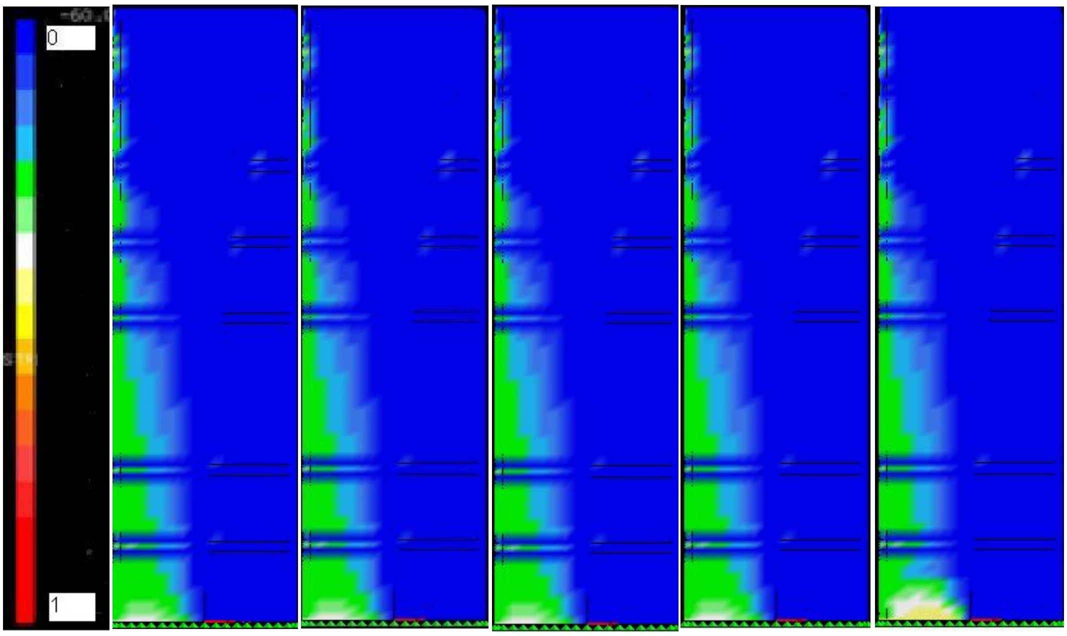

Figure 6. Evolution of Triaxiality Index at at 24%, 47%, 71%, 95%, and 100% of failure load

Figure 5 depicts the evolution of the Von Mises stress of the SPSW. At 95% can be seen that plasticity has spread out extensively. At limit load the first floor SPSW has completely yielded and it become a plastic-hinge. Figure 6 shows the distribution of the triaxiality index for different levels of loading. Maximum value of this index is less than 0.6, this occurs at the welding of the SPSW with the foundation. This maximum value of the index of triaxiality tells us that we are far from possibility of brittle fracture for the level of stresses applied.

CONCLUSIONS AND RECOMMENDATIONS

The important conclusions and recommendations of these numerical studies of seismic performance of special plate shear walls under long duration earthquakes are summarized as follows:

- SPSW is a viable alternative to replace Reinforced concrete shear walls. For new buildings and to retrofit earthquake damaged buildings.
- The Peruvian seismic code spectrum cannot provide adequate safety for structures of periods between 2.5 y 3.0 seconds when they are subjected to long duration earthquakes; Future versions should take into account this.
- Due to second peak, more modes could necessary to satisfy dynamic mass participation.
- Capacity predicted by AISC 341 is adequate; results show that SPSW yield extensively under extreme lateral loads, without signs of fracture in web plates.

REFERENCES

1. Tamura H, Sasaki E, Yamada H, Katsuchi H, and Chanpheng T. "Involvements of Stress Triaxiality in the Brittle Fracture during Earthquakes in Steel Bridge Bents". International Journal of Steel Structures, September 2009, Vol 9, No 3, 241-252.
2. Simo JC, Fox DD, Rifai MS," On a stress resultant geometrically exact shell model, Part III: Computational aspects of the nonlinear theory", Comput. Methods Appl. Mech. Engrg. 79 (1990) 21-70.
3. Simo JC, Hughes TJR, "Computational Inelasticity", Springer, 1998.
4. American Institute of Steel Construction AISC, "Seismic Design Manual", 2006.
5. Souza-Neto EA, Peric D, Owen DRJ, "Computational Methods for plasticity", Wiley, 2008.
6. Belytschko T, Liu WK, Moran B, "Nonlinear Finite Elements for Continua and Structures", Wiley 2000.
7. Basler K, "Plates Girder under combined bending and Shear", Fritz Engineering Laboratory Report No. 252-21, Lehigh University, 1961.
8. Lignos DG, Chung Y, Nagae T, Nakashima M, "Numerical and experimental evaluation of seismic capacity of high-rise steel buildings subjected to long duration earthquakes", Computers and Structures 89 (2011) 959-967.
9. Sabelli R, Bruneau M, "Steel Design Guide, steel Plate Shear Wall", 2007.
10. Reglamento Nacional de Construcciones RNC. Ministry of Housing Construction and Sanitation, Perú, 2006.
11. Eatherton M., "Design and Construction of Steel Plate Shear Walls", 2006.