STEEL SLIT PANEL CONFIGURATIONS

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ABSTRACT :

Steel Slit Panel Frames (SSPFs) are a lateral force resisting system (LFRS) created to resist seismic loads in buildings. The main benefits of the proposed system are: high energy dissipation capabilities, potential for architectural flexibility, and seismic retrofitting possibilities. The energy in the system is dissipated by the Steel Slit Panels (SSPs). This paper first introduces the system; it then discusses three different SSPs that may be used for the system.

KEYWORDS: Steel Slit Panels, Lateral Force Resisting System, Seismic Prone Regions, Earthquake Resisting System, Steel Buildings

1. INTRODUCTION

Steel Slit Panels (SSPs) are made of steel plates, with rows of vertical slits at equidistant spaces forming series of links in between the slits. Figure 1.1 shows a schematic drawing of an SSP along with the geometric parameters that describe the panel. When the panel is subjected to a horizontal displacement, the links behave as beams in double curvature, dissipating energy in flexure. Shear walls with slits were first tested using reinforced concrete (Muto, 1973). It is believed that these "slitted walls" were the first passive energy dissipation system for buildings (Martínez-Rueda, 2002)(Nakashima and Chusilp, 2003). More recently, researchers Hitaka and Matsui studied shear walls with slits made from steel (2000, 2003). The main difference between the shear walls studied by Hitaka and Matsui and the SSPs is the width:height ratio of the panel; SSPs have a 1:2 ratio instead of a 1:1 ratio. While the general behavior of a panel is similar for both systems, the global behavior of a frame containing SSPs is rather different. What makes the SSP different is not only the aspect ratio, but also the design objective for SSPs to take all the base shear force in the frame rather than a fraction, as is the case of the shear walls with slits studied in Japan.

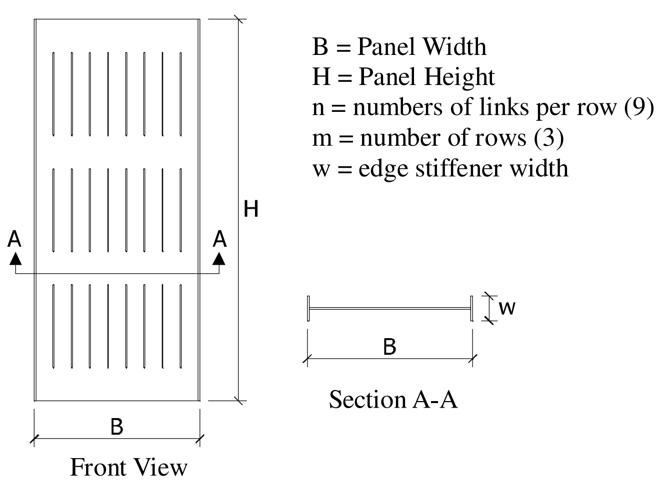

Figure 1.1. T1 panel schematic drawing.

A Steel Slit Panel Frame (SSPF) is a lateral force resisting frame which uses SSPs to provide the full lateral resistance. The SSPF is being studied via a combined analytical and experimental investigation. The main benefits are its energy dissipation capacity, its architectural flexibility, and its relatively simple installation. All beam-column connections are pinned; the steel slit panels are bolted to fin plates which are shop welded to the beams. Figure 1.2 shows a schematic drawing of a 3 story SSPF. For this frame only two bays have been occupied with panels. Two panels occupy each bay. The clear space between the panels is near 6 m (20ft). This provides space for windows, doors, or other openings between partitions.

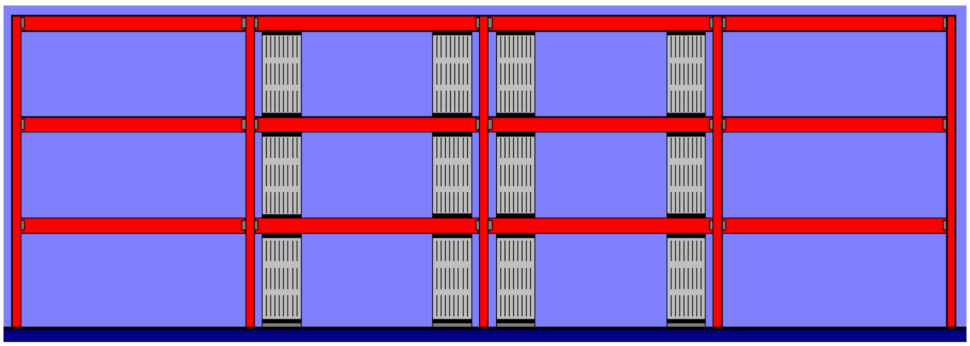

Figure 1.2. Three story steel slit panel frame.

A typical building using SSPs as its Lateral Force Resisting System (LFRS) will require enough panels to satisfy interstory drift levels and strength demands. If too many panels are needed to satisfy either of these, the system will not be architecturally attractive or economical. The capacity of the panels is a function of the geometry of the links and the strength of the steel. The stiffness depends on the overall geometry of the panel. Analytical work has been done to optimize the strength and stiffness of the panels. The parameter which influences the capacity and the stiffness the most is the width of the links. The wider the link, the greater capacity and stiffness. However, if the links are too wide in comparison to their thickness, buckling could occur. The number of rows of links is also an important parameter for stiffness.

Section 2 of this paper introduces the behavior of the SSPF. Specifically, the interaction of the panels with the frame. Section 3 discusses three types of panels. These are the T1 panel, the T2 panel, and the buckling restrained panel (BR panel). The T1 panel is similar to that studied in Japan except for the aspect ratio. While the T1 panel performs satisfactorily according to analytical studies, a panel with higher stiffness might be needed to satisfy drift demands in some buildings. With that in mind, the T2 panel and the BR panel were created. The T2 panel uses fewer rows of links, while the BR panel uses fewer links in a row, but links that are braced to prevent buckling.

2. STEEL SLIT PANEL FRAME BEHAVIOR (OVERVIEW)

2.1. Gravity Loading

The panels will inevitably, although unintentionally, act as compression members. A portion of the gravity loads is transferred from one story to the next one through the panels. High demands of gravity loading could have an adverse effect on the behavior of steel slit panels. When panels are placed in a single column configuration (i.e. at same position in a bay for all stories), the gravity loads add up from stories above and, depending on the number of stories and the level of load, the load demand could be higher than the critical load the panel can resist. This loading condition has been studied analytically and experimentally; for the cases studied it did not represent a problem. However, the effect of gravity loading should be studied for each specific design.

2.2. Panel Strength and Frame Stiffness

SSPFs must have: (1) enough strength to resist the lateral loads, and (2) sufficient stiffness to avoid excessive sway of the building. The panel strength can be obtained from Eqn. 2.1 (Hitaka and Matsui, 2003). It is also important to check global lateral torsional buckling and shear buckling of the panel. For most prototype buildings studied, stiffness requirements controlled the number of panels needed in the frame. Since the beams bounding the panels are not perfectly rigid, they rotate, and this rotation reduces the stiffness provided by the panels. The more flexible the beam is, the more it will rotate, resulting in less stiffness for the SSPF. This is shown in Figure 2.1; the stiffness of the SSPF increases as the flexural stiffness (EI) of the beams increases. Note also in this plot a horizontal line with a magnitude of 33.6 KN/mm (192 kip/in). This line represents the optimum stiffness of the panel. The optimum stiffness is defined as the theoretical stiffness of a panel; it considers the shear stiffness of the zones above, below, and between links, the shear stiffness of the links, and the flexural stiffness of the links. The optimum stiffness can be calculated using Eqn. 2.2 (Hitaka and Matsui, 2003).

$$Q_{ult} = \frac{nF_y tb^2}{2l} \tag{2.1}$$

$$K_{initial} = \frac{1}{\frac{k(h-ml)}{GBt} + \left(\frac{m}{n}\right)\left(\frac{kl}{Gbt}\right) + \left(\frac{m}{n}\right)\left(\frac{l^3}{Etb^3}\right)} \tag{2.2}$$

Where F_y = yield strength; E = Young's Modulus; G = shear modulus; k = shear deformation shape factor (1.2 for rectangular sections); B = panel width; h = panel height; t = panel thickness; m = number of rows of links; n = number of links in a row; l = link length; and b = link width.

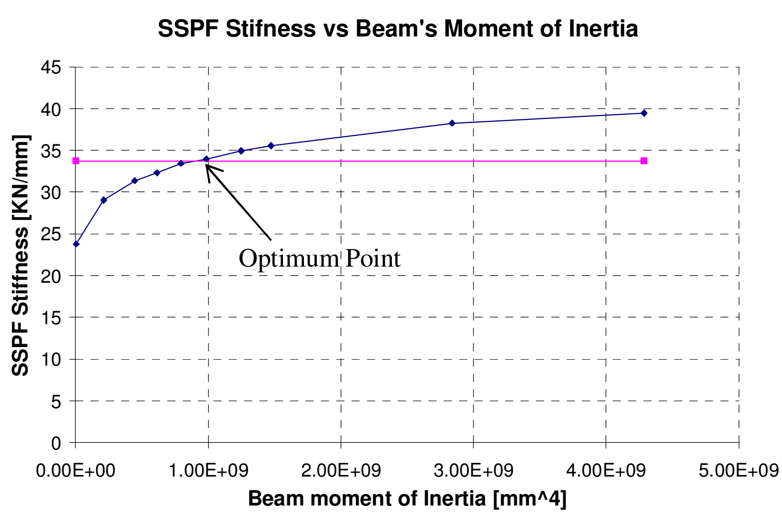

Figure 2.1. Frame stiffness vs. beam flexural stiffness.

In Figure 2.1, the optimum point represents the point at which the lateral stiffness of the panel in the frame reaches the stiffness given by Eqn. 2.2. The optimum stiffness is not the maximum stiffness; finite element analyses have shown that the optimum stiffness is about 80% of the maximum stiffness.

The stiffness provided by a panel also depends on the story in which it is located. At stories above the first story, the reduction in stiffness becomes greater (for single column configuration). This phenomenon can be appreciated in Figure 2.2; for the first level, the total rotation is θ_1. However, for the second level, the total rotation is $\theta_1 + \theta_2$.

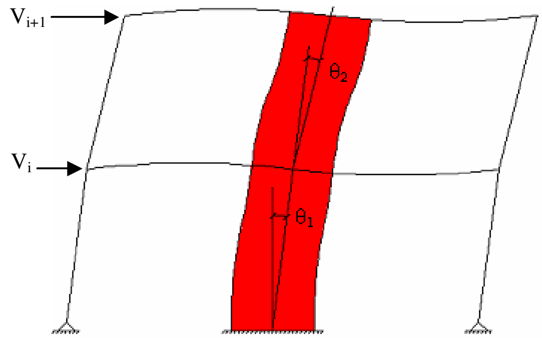

Figure 2.2. Two story frame subjected to lateral loading.

A three story-one bay SSP frame with one panel placed at the center of each bay was analyzed. The stiffness obtained from each of the stories is plotted in Figure 2.3. Note that only the first story reached the optimum stiffness. The second story reached 70% of the optimum stiffness, while the third story was further reduced to about 57% of the optimum stiffness. This plot clearly reflects the reduction of stiffness occurring at higher levels.

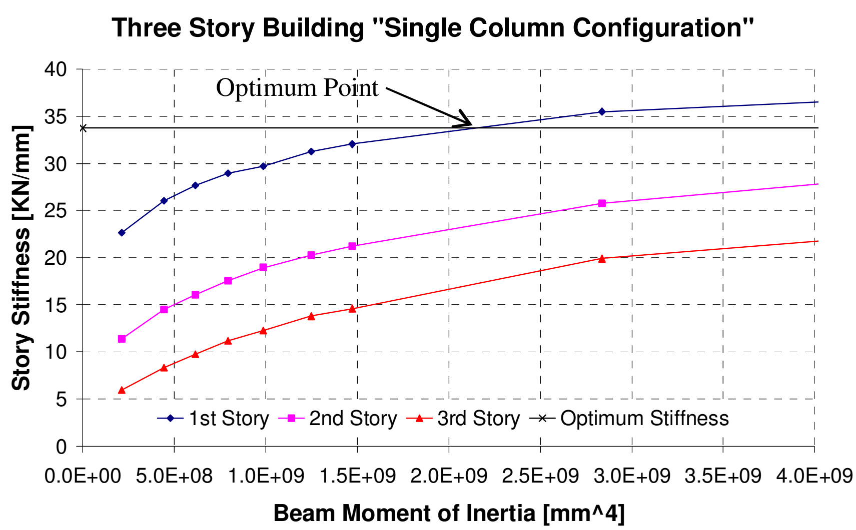

Figure 2.3. Frame stiffness vs. beam flexural stiffness.

The location of the panels in a bay, the number of panels, and the thickness of the panels are other factors that affect the SSPF stiffness. The stiffness is significantly reduced by some of these factors; thus, ignoring these reductions could lead to excessive building drift and difficulty satisfying design criteria. Design guidelines which consider all of these factors are being developed as part of this research.

3. STEEL SLIT PANEL TYPES

3.1. T1 Panel

The T1 panel has been shown previously in Figure 1.1. This panel has 3 rows of 9 links. It is the simplest panel to fabricate. If properly designed, it provides ductile behavior and dissipates a great amount of energy. Two thicknesses have been studied, 12.7 mm (½”) and 19.05 mm (¾”). Thicker panels may be used but such panels could become too expensive to construct and difficult to handle.

3.2. T2 Panel

Many panels are needed for the design of medium-rise buildings because of the reduction in stiffness at higher floors. With that in mind, the T2 panel was created. The T2 panel was designed to have a higher stiffness than the T1 panel. This panel is about 20% stiffer than the T1 panel. It has two rows of 8 links, and uses two rigid elements, one at top and one at bottom of the panel, between the panel and the main beams. For the two rigid elements, called 'stub beams', W shapes could be used. Figure 3.1 shows a schematic of the T2 panel. Although the T2 panel appears to be more involved than the T-1 panel, it could also be fully prepared at the shop, and bolted to the beams at the site.

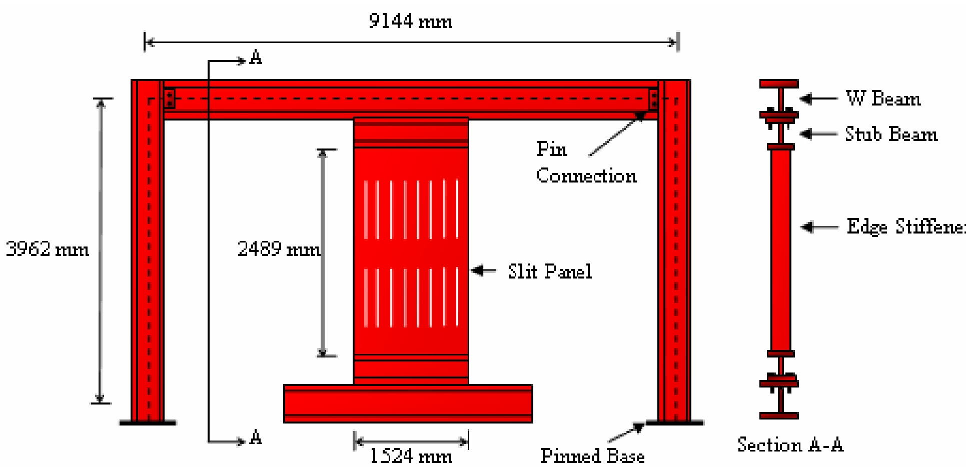

Figure 3.1. T2 panel schematic drawing.

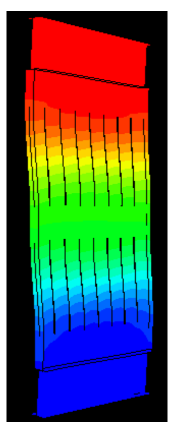

Figure 3.2. FEM showing deflected shape of T2 panel.

The lateral stiffness of the T-2 panel obtained from the FEM is 58.14 KN/mm (332 k/in) (Figure 3.2). A W16X40 stub beam was utilized to calculate the panel stiffness with the FEMs. However, the panel stiffness is sensitive to the stiffness of the stub beam. Therefore, more specifically, for a panel with a stub beam web thickness of 7.75 mm (0.305 in.), the stiffness is 58.14 KN/mm (332 k/in). A stiffness of 56.6 KN/mm (323 k/in) was obtained from the analytical formulas (Eqn. 2.2). The role played by the stub beam is explained in more detail below.

The stub beam plays a major role in the response of the T2 panel. The idea behind the stub beam is to have very rigid elements at the end of the panels so that its deformation is as small as possible. This will allow the panel to have a smaller height and therefore, a higher stiffness. It is very important that the stub beam satisfies a minimum stiffness. This minimum stiffness has been related to the stiffness of the band zone (i.e. regions above, below, and between rows of links) in the panels. Eqn. 3.1 provides the web thickness required to satisfy the minimum stiffness.

$$t_w = \frac{FK \cdot t_{panel} \cdot d_{beam}}{\left(\frac{h-ml}{m+1}\right)} \tag{3.1}$$

Where FK = ratio of stub beam stiffness to band zone stiffness (FK≈1.6); h = panel height; t_{panel} = panel thickness; m = number of rows of links; and l = link length.

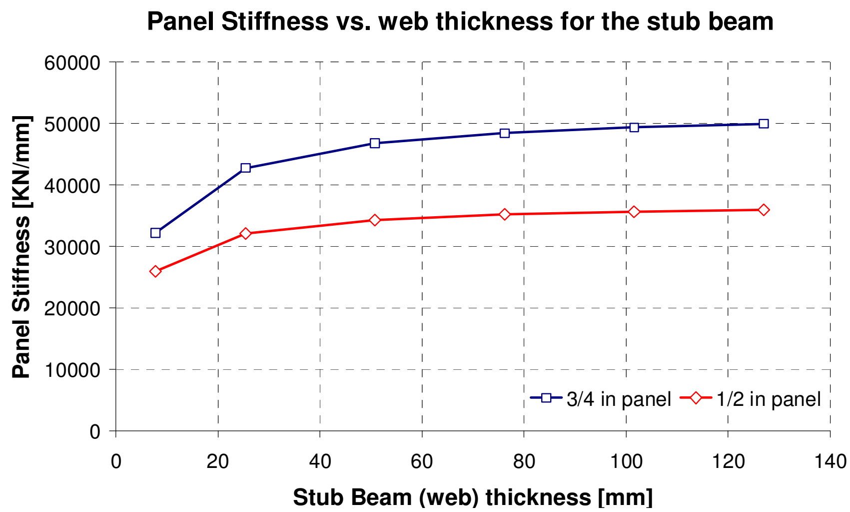

Figure 3.3. Panel stiffness as a function of the stub beam thickness.

Since the recommended stiffness for the web (according to Eqn. 3.1) of the stub beams is very thick (25.4 mm), selecting a satisfactory W shape is almost impossible. Therefore, a doubler plate could be used. FEA were performed to simulate the W shape with a doubler plate. From the analysis, it was observed that the doubler plate does not have the same effect as a single plate with the required thickness. This observation suggests that plug welds might be needed to ensure that the beam web and the doubler plate work as a unit. Another option may be to use a welded, built-up I-beam.

Prying action needs to be considered when designing the stub beam. The flange thickness required to prevent prying action is typically very large. Therefore, it is convenient to allow prying action to occur, and to design for it. For example, for the 12.7 mm (½”) panel, W16X40 beams satisfy the prying action limits. However, for a 19.05 mm (¾”) thick T2 panel, a thicker flange is required. A W16X45 beam satisfies the requirements.

3.3. BR Panel

This panel has the same overall dimensions as a T1 panel. The main difference is the number of links and their width. The strength of a panel is proportional to the square of the width of each link (see Eqn. 2.1); thus, increasing the width of the links (b) will increase the panel strength. The same is true with the stiffness; increasing b increases the shear stiffness and the flexural stiffness of the links, and therefore, increases the total stiffness of the panel. This is evident by looking at Eqn. 2.2. Using a higher width, b, in the links, means that the b/t ratio will increase. As reported by McCloskey (2006), the b/t ratio should be in the range of 10 to 14.

This recommendation was based on observations of tests carried by Hitaka and Matsui (2003) in which panels with a b/t ratio near 10 performed well at higher interstory drifts. When higher values of b/t were used, the panels tended to buckle and the hysteretic loops underwent significant strength degradation and stiffness reduction at relatively low interstory drift values. In order to avoid buckling, one brace will be provided to restrain the out-of-plane deformation. Figure 3.4 shows a schematic drawing of the BR panel. One channel section is used at each row of links for each side of the panel. This results in 6 channel sections.

A similar concept was used for steel panels with slits used as shear fuses in studies conducted by Deierlein et al. (2007) at Stanford University. When braces preventing out-of-plane deformation were used, the steel panels with slits underwent higher amounts of interstory drift without suffering from strength degradation. From testing by Deierlein et al., it was also observed that the stiffness reduction was significantly smaller when braces were used.

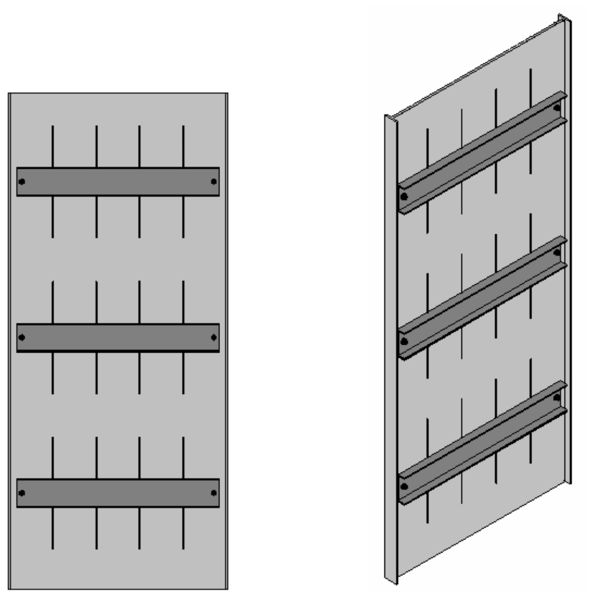

Figure 3.4. Front view of the T1 panel (left) and isometric view (right).

The theoretical strength of the panel is 876.3 KN (197 kips); that is approximately 33% higher than that of the T1 panel. The stiffness of the BR panel is 88.96 KN/mm (508 kip/in). The BR panel is approximately twice as stiff as a T1 panel. This type of panel is very attractive since the production cost would be similar to that of a T1 panel but its capacity is significantly higher. More stiffness means fewer panels needed to control interstory drift; this would result in a more economical building. Figure 3.5 shows the force-deformation curves for the T1, T2 and BR panels.

The BR Panel is very promising; however there is the possibility of adverse effects from stress concentrations at the holes in the exterior links. Furthermore, panels with wider links could fail by a global buckling mode rather than by lateral torsional buckling of individual links. If this were the case, the braces might not be very helpful in preventing buckling. These and other behavioral aspects are being studied in further detail.

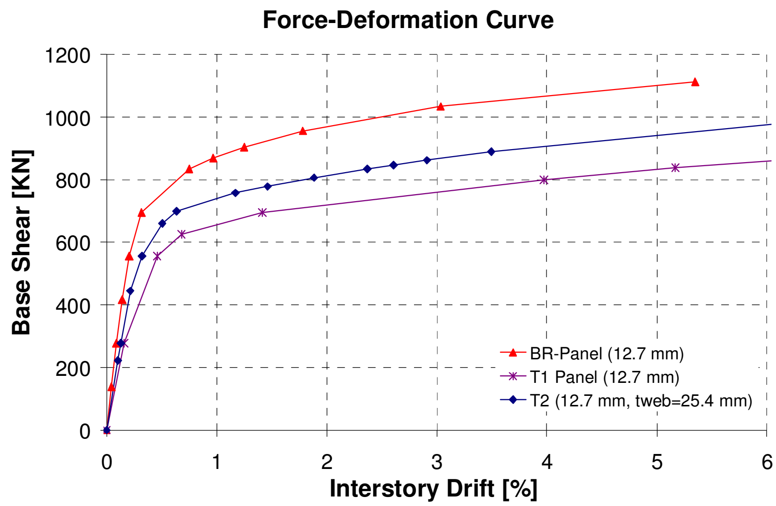

Figure 3.5. Force-deformation plot for the three panel types.

4. CONCLUDING REMARKS AND FUTURE WORK

Steel Slit Panel Frames are a new LFRS being investigated by a combined analytical and experimental study. This paper introduced the system and discussed 3 types of panels proposed for the system. These are: the T1 panel, the T2 panel and the Buckling Restrained panel (BR panel). Future work includes experimental tests which are intended to study the 3 types of panels and to study the panel-frame interaction.

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