

Analysis and Design of Steel Slit Panel Frames (SSPFs) for Seismic Areas

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ABSTRACT

The steel slit panel frame (SSPF) system is a new seismic force-resisting system (SFRS) intended for areas of high seismicity. The main components in an SSPF are steel beams and columns and steel slit panels (SSPs). SSPs are steel plates with rows of vertical slits cut at equidistant spaces. Links (small columns) are formed between the slits. When the panel is subjected to shear load, these links behave as beams in double curvature, reaching their plastic moment capacity at both ends. Beams are simply connected to the columns in the SSPF. The SSPs provide all the stiffness, strength and energy dissipation in the system. This paper presents the design of the SSP and the SSPF system. This paper also presents two prototype buildings that were designed for a site in Los Angeles, California. Finally, a simplified model developed for analyzing SSPs is presented.

Keywords: steel slit panels, seismic force-resisting system.

The steel slit panel frame (SSPF) system is a seismic force-resisting system (SFRS) intended for areas of high seismicity. The frame is composed of beams and columns, simply connected, and steel slit panels (SSPs) bolted between beams. An SSP is a steel plate with rows of slits cut from the plate so that small vertical flexural members, termed *links*, are created within the steel plate. When an SSP is subjected to shear deformations, the links behave as beams in double curvature, developing their plastic moment capacity at both ends and dissipating energy. Two edge stiffeners are welded to the main plate. These provide out-of-plane stability to the panel. Figure 1 shows a diagram of an SSP, highlighting its main attributes (e.g., the edge stiffeners, band zones, links and slits). The main parameters used for describing a panel are also shown in the figure. The configuration of the SSP is such that the system offers architectural flexibility (i.e., it can be placed within walls and around doors, windows and corridors) and offers retrofit possibilities.

This paper provides guidelines for the analysis and design of the SSPF system and includes prototype building design examples and a simplified model that may be used for analyzing the SSPs. These guidelines were created based on finite element analyses (FEAs) and experiments performed on

the SSPs and on the SSPF (Cortes, 2009), as well as strength and stiffness equations developed for steel slit walls (Hitaka and Matsui, 2003) and adapted for the SSP (McCloskey, 2006; Cortes, 2009). In addition to the extensive parametric studies performed analytically, ten $\frac{1}{3}$ -scale tests were performed. These tests were divided in two series: the “panel-alone” series and the “panel frame” series. The first series of testing looked into the strength, the stiffness, the strain distributions and the failure modes of the panels. The second series of tests, the panel frame series, was intended to study the interaction of the panels with the frame. The equations for strength and stiffness were verified against the experimental data. Finite element analysis results suggested that the results from the $\frac{1}{3}$ -scale tests could be extrapolated to the full-sized panels (Cortes, 2009).

Ductile behavior is observed in the panels, with yielding in the links dominating the response to large drift levels. Figure 2 shows the load versus interstory drift plot for one of the panels tested, up to 4% drift (left) and until failure, at 7% drift (right). Global lateral-torsional buckling (LTB) occurred at the last cycle of 4% drift. LTB did not reduce the load carrying capacity of the panel. The strength degradation observed in Figure 2 was caused by cracks formed at the ends of the slits. At 3% drift, the upper and lower rows of links developed small cracks at the slit ends. At 7% drift the cracks had propagated enough to cause fracture of the links at the bottom in the lowest row. Tests of SSPFs (i.e., frames with SSPs) showed similar, ductile response; all panels were capable of reaching interstory drifts of at least 5% before failure.

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DESIGN PROCESS

The design of an SSPF system requires two principal steps: (1) design of the steel slit panels (SSPs) to be used and (2) design of the steel slit panel frame (SSPF). The main equations and parameters for designing an SSP are provided in the next section. It is recommended that the engineer design a few different panels with different strength and stiffness capacities. This will provide different alternatives when designing the frame. In an SSPF, all beams and columns should remain elastic; inelastic deformation and any resulting damage should only occur at the panels (SSPs), which are intended to be easily replaceable. Beams are connected to the columns using simple shear connections (e.g., shear tabs, angles, etc.). Figure 3 shows a three-story SSPF with two panels at the exterior bays of each story.

Once the number of panels required to resist the design story shears in the frame is known, the frame should be analyzed and interstory drift checked. Interstory drifts that satisfy code requirements may be the final check, or the designer may further conduct a performance evaluation of the building according to ASCE 41 (2006); note that this study was conducted with the then-available pre-standard, FEMA 356 (2000). The last step is to design the bolted connections used to connect the panels to the beams and to the foundation. Design of an SSP, design of an SSPF, connection design, prototype building design and modeling using a simplified model are presented in the following sections.

STEEL SLIT PANEL DESIGN

The design of a steel slit panel includes choosing a configuration (i.e., link and edge stiffener dimensions) for which the links should develop their plastic moment capacity before any instability. Stiffness and strength can then be determined using equations based on mechanics principles. The first step is to determine the panel's overall geometry (e.g., its height, width and thickness). Because the panels are placed between beams, the height is obtained from the clear distance between beams. Part 9 of the AISC *Steel Construction Manual* (AISC, 2005d) recommends a clear distance of $\frac{1}{2}$ in. between connecting members; thus, the total height of the panel, h , is the clear space between beams at the story where the panel will be placed, minus 1 in. ($\frac{1}{2}$ in. at each connection). The width of the panel, B , should be chosen to be near one-half of the panel's height to satisfy the 1:2 width-to-height ratio that characterizes the SSP system. As an initial estimate, $\frac{1}{2}$ -in. panels might be considered for buildings with three or fewer stories, or $\frac{3}{4}$ in. for taller buildings, up to ten stories. The thickness suggestion is based on FEA performed on buildings of one, three, six and ten stories located at a site in Los Angeles. Note that the SSPF system has only been studied for low- to mid-rise buildings with ten stories or less.

Once the global geometry of the panel has been defined, the next step is to define the geometry of the links and their pattern. Two or three rows of links should be used. The

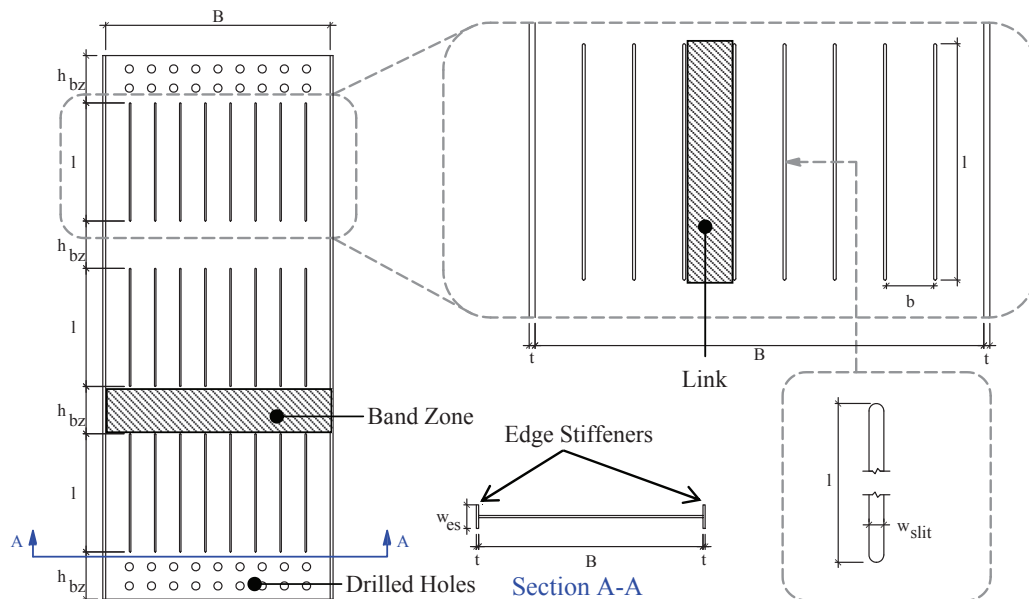


Fig. 1. Steel slit panel.

number of links, n , used in each row should be at least seven (McCloskey, 2006). The width of the slits will depend on the method used to make the cuts (e.g., plasma torch, water cutting, saw, etc.). If no information is known at the initial design, the designer may assume that for a $\frac{3}{8}$ -in.-thick panel, the width of the slit, w_{slit} , will be $\frac{1}{8}$ in., for a $\frac{1}{2}$ -in.-thick plate the width will be $\frac{3}{16}$ in., and for a $\frac{3}{4}$ -in.-thick plate, the width will be $\frac{1}{4}$ in. These dimensions were observed in samples of steel where the slits were made using a plasma torch (McCloskey, 2006). The width of a link, b , may be determined from Equation 1.

$$b = \frac{B - (n - 1)(w_{slit})}{n} \quad (1)$$

where

B = width of the panel (not including the edge stiffeners)
 n = number of links
 w_{slit} = width of the slits

There are three main parameters that need to be considered when designing an SSP: α , β and b/t . Typical values for

these parameters were based on experimental observations on slit walls studied in Japan (Hitaka and Matsui, 2003). The slit walls are similar to the SSP but have an aspect ratio of 1:1 instead of 1:2 and are relied upon to resist only 25 to 30% of the lateral load in the system. Note that by 2003, three buildings, the tallest at 19 stories, had been built in Japan using a steel slit wall–moment frame dual system. Here, α is the link's aspect ratio given by l/b , where l is the length of the link. In addition, α ensures that a link is long enough so that flexure controls, rather than shear; α should be between 2.5 and 5.0. Defined as the fraction of the total height of the panel that is composed of links, β can be calculated as ml/h , where m is the number of rows. The recommended range for β is 0.65 to 0.85. This parameter ensures that the behavior of the panel is controlled by the links. The third parameter that needs to be checked is the width to thickness ratio of the link (b/t). This parameter controls premature out-of-plane deformations of the links. The recommended range for b/t is 10 to 15.

One pair of edge stiffeners is welded to the panel, one on each vertical edge. The two main purposes of the edge

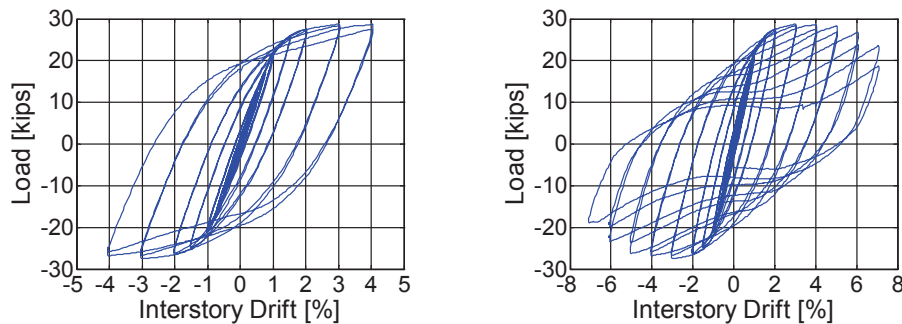


Fig. 2. Load vs. interstory drift plots; up to 4% (left); until failure at 7% (right) (Cortes, 2009).

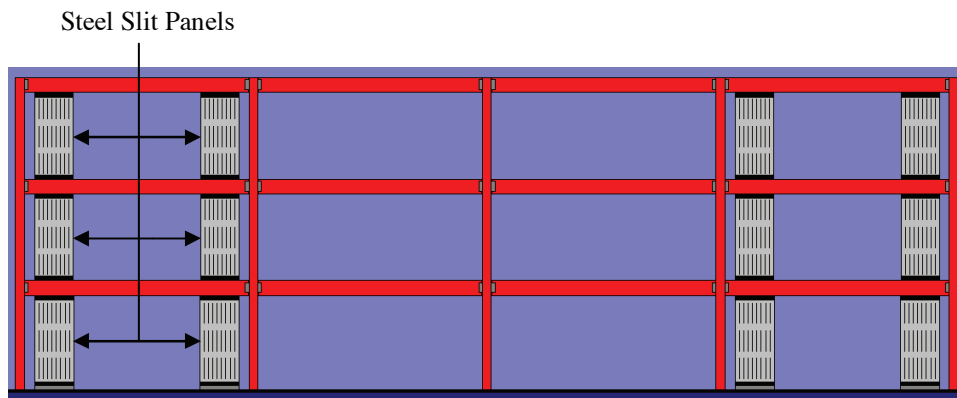


Fig. 3. Steel slit panel frame elevation.

stiffeners are (1) resisting the overturning moments and (2) providing resistance to out-of-plane deformations and buckling in the SSP. In addition to these main purposes, the edge stiffeners increase the lateral stiffness and the ultimate strength of the steel slit panel. The edge stiffeners are designed to take all of the overturning moment in the panel. The overturning moment is divided into two coupling forces, or boundary forces, which are determined from force diagrams based on the mechanics of the panel (see Figure 4). The boundary force, P_{bound} , is given by Equation 2.

$$P_{bound} = \frac{Q_{ult} \left(\frac{h}{2} \right) - nM_p}{B} \quad (2)$$

where

Q_{ult} = shear capacity of the panel, assuming full plastic moment capacity of the links (Equation 6 or 7)

M_p = plastic moment capacity of the link

The width of the edge stiffeners, w_{es} , is calculated from the assumption that the edge stiffeners undergo axial yielding when subjected to P_{bound} . It is practical to use edge stiffeners with a thickness equal to that used for the panel. If necessary (e.g., to increase the moment of inertia of the panel or the stiffness), the plate used for the edge stiffeners may be thicker; however, the equations given in this paper assume that the main plate and the edge stiffeners are made from the same plate thickness. Experimental tests have shown that the edge stiffeners undergo yielding at top and bottom. This yielding was caused by the high local bending demand

caused by the lateral deformations of the panel rather than by the axial load demand. However, the same experiments demonstrated that sizing the edge stiffeners assuming compressive yielding resulted in an appropriate design. The edge stiffeners were capable of resisting the boundary element forces and prevented the panel from undergoing out-of-plane deformations until larger drift levels. The width of the edge stiffeners is calculated from Equation 3.

$$w_{es} = \frac{P_{bound}}{F_y t} \quad (3)$$

where

F_y = minimum specified yield strength of the panel

Once the edge stiffeners are sized, local buckling shall be checked (AISC, 2005a). Also, the width of the edge stiffeners should not interfere with partition walls or other non-structural components of the building. The edge stiffeners are connected to the main plate using welds. Because the panel acts as a flexural member in double curvature, the design load at the edge stiffener (P_{bound}) acts on half the length (only half of the length is in compression at each stiffener). In other words, the length used for design of the weld should be $h/2$. The welding procedure should be carefully planned such that initial imperfections in the panels due to the heating-cooling process are minimized. Initial imperfections are not desired because these reduce the out-of-plane resistance of the panel. This was observed in most of the $1/3$ -scale panels tested. It is expected that for full-scale panels, the effect will not be as important as it was for the smaller $1/3$ -scale

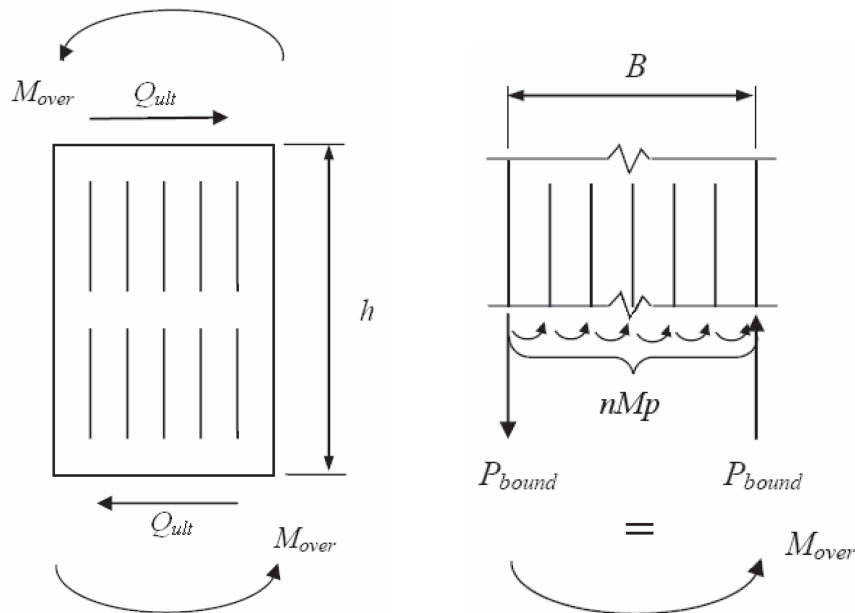


Fig. 4. SSP force equilibrium diagram (adapted from McCloskey, 2006).

specimens, because the panel thickness is greater. Nevertheless, caution is advised.

The stiffness of an SSP is determined from the contributions of the band zones (regions above, below, and between the links) and the links. Three terms are considered: (1) the shear stiffness of the band zones, (2) the shear stiffness of the links and (3) the flexural stiffness of the links. The equation, developed by Hitaka and Matsui (2003), is shown here:

$$K_{panel} = \frac{1}{\frac{k(h - ml)}{GBt} + \left(\frac{m}{n}\right)\left(\frac{kl}{Gbt}\right) + k(\alpha)\left(\frac{m}{n}\right)\left(\frac{l^3}{Etb^3}\right)} \quad (4)$$

where

- k = shear deformation shape factor (1.2 for rectangular sections)
- G = shear modulus
- E = Young's modulus
- $k(\alpha)$ = factor that considers the level of restraint of the links (given by Equation 5)

and other parameters are as previously defined.

$$k(\alpha) = (1 + \alpha^{-1})^3 \quad (5)$$

Here, $k(\alpha)$ was determined from experimental results (Hitaka and Matsui, 2003). Equation 4 assumes that the stiffness provided by one link is $12EI/l^3$, where I is the moment of inertia of the link; however, the stress concentrations around the slits reduce the level of restraint provided to the links. The $k(\alpha)$ factor reduces the stiffness provided by the links from $12EI/l^3$ to a value ranging from $4.4EI/l^3$ to $6.9EI/l^3$, depending on α ; $4.4EI/l^3$ corresponds to an α of 2.5 (lower limit of α); $6.9EI/l^3$ corresponds to an α of 5.0 (upper limit of α).

The ultimate strength of an SSP is obtained from the case when all the links in a row reach their plastic moment capacity. The equation is determined by adding the shear forces developed in the interior links and the shear forces developed in the two exterior links in a row. The two exterior links are treated differently than the interior links because these work together with the edge stiffeners to form a T-shaped link. As such, the equation for strength will depend on the location of the plastic neutral axis of these links. Two cases are possible: (1) when the plastic neutral axis occurs in the edge stiffener and (2) when the plastic neutral axis occurs in the link. Assuming that the thickness of the edge stiffeners and the panel is the same, one may determine the applicable case by comparing the width of the edge stiffener (w_{es}) to the width of the link (b). If w_{es} is greater than b , the plastic neutral axis will be inside the edge stiffener; otherwise, the plastic neutral axis will be in the link. Equation 6 is used for the first case ($w_{es} > b$); Equation 7 is used for the second case ($w_{es} \leq b$). If $w_{es} = b$, both equations will provide

the same result.

$$Q_{ult} = \frac{F_y t}{2l} \left[(b^2 n) + 2b^2 (t - 1) + 2w_{es} (2b + t) \right] \quad (\text{for } w_{es} > b) \quad (6)$$

$$Q_{ult} = \frac{F_y t}{2l} \left[(b^2 n) + 4bw_{es} + 2w_{es}^2 \right] \quad (\text{for } w_{es} \leq b) \quad (7)$$

Ideally, the panel should be designed to reach its ultimate strength (e.g., plastic moment capacity of all links in a row reached); however, other failure modes may control the design of the SSP. Because the SSP has an aspect ratio of 1:2, the parameters developed by Hitaka and Matsui (2003) for the 1:1 slit walls are not sufficient for preventing instabilities, particularly global instabilities. Three strength limit states should be checked: (1) global lateral-torsional buckling, (2) lateral-torsional buckling of all links in a row and (3) shear buckling of the panel. Global lateral-torsional buckling of the panel causes the panel to twist and, therefore, to behave in a different way than intended. At low drift levels, the edge stiffeners help the panel stay in plane; however, at higher drift levels, out-of-plane deformations will certainly occur. Global lateral-torsional buckling should be delayed to inter-story drifts of at least 2 to 2.5%, corresponding to the drift limit for a design-basis earthquake (ASCE, 2005). Figure 5 shows an SSP undergoing global lateral-torsional buckling.



Fig. 5. Global lateral-torsional buckling of an SSP.

Equation 8 provides a prediction of the capacity associated with global lateral-torsional buckling. This equation was developed by Timoshenko and Gere (1961) for determining the lateral-torsional buckling capacity of a cantilevered beam with a narrow cross section loaded at the free end. Half of the height of the panel (more precisely, the distance from the centroid of the group of bolts to mid-height of the panel) is used for the length of the cantilever beam ($h_{LTB}/2$). This equation provided predictions within 6%, on average, compared to experimental results from SSPs tested that underwent LTB (Cortes, 2009).

$$Q_{LTB} = \left[\frac{4.013}{\left(\frac{h_{LTB}}{2}\right)^2} \right] \sqrt{EI_{panel} C} \quad (8)$$

where

- I_{panel} = moment of inertia of the panel about its weak axis, neglecting the slits (assuming solid panel), including the edge stiffeners (i.e., $B + t$)
- C = torsional constant (given by Equation 9)
- h_{LTB} = height used for this calculation, shown in Figure 6.

$$C = \left(\frac{Gt^3}{3} \right) (2w_{es} + B - 2t) \quad (9)$$

In addition to global lateral-torsional buckling, lateral-torsional buckling of the links in a row should be checked. This limit state did not control any of the panels studied;

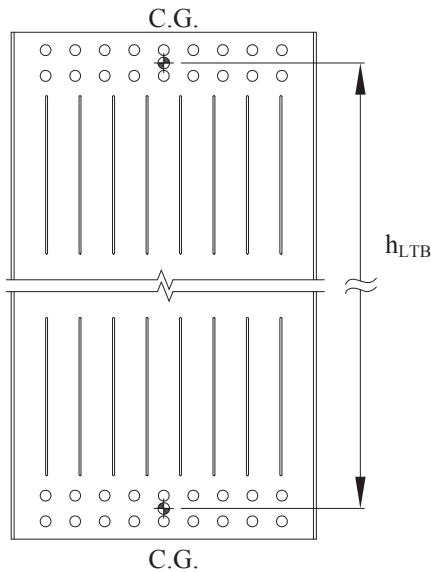


Fig. 6. Height used for calculating the global lateral-torsional buckling capacity.

however, it was observed in studies of the steel wall with slits performed in Japan (Hitaka and Matsui, 2003). To determine the capacity of the panel associated with lateral-torsional buckling of the links, the capacity of one link is determined based on Timoshenko and Gere (1961) and then multiplied by the number of links in a row, n . The final equation for capacity associated with lateral-torsional buckling of the links is given by Equation 10.

$$Q_{LTB \text{ links}} = n \left[\frac{4.013}{\left(\frac{l}{2}\right)^2} \right] \sqrt{EI_{link} c_{link}} \quad (10)$$

where

I_{link} = moment of inertia of the link about its weak axis ($bt^3/12$)

c_{link} = torsional constant given by Equation 11

$$c_{link} = GJ = \left(\frac{Gbt^3}{3} \right) \left[1 - 0.63 \left(\frac{t}{b} \right) \right] \quad (11)$$

The last limit state that needs to be checked is shear buckling of the panel. The slits in the panel create a rather complex geometry; however, for the purpose of determining capacity, the panel is treated as a solid panel. Equation 12 is used for determining the shear buckling capacity of the panel.

$$Q_{scr} = k_v \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t^3}{B} \right) \quad (12)$$

where

ν = Poisson's ratio (taken as 0.3 for steel)

k_v = parameter that depends on the edge support conditions, the type of stress and the aspect ratio of the plate

Equation 12 is an equation developed by Timoshenko and Gere (1961) to obtain the elastic buckling unit stress in a plate, multiplied by the cross-sectional area of the panel (Bt) to obtain the result in strength units. For simplicity, the area of the edge stiffeners is neglected for this calculation.

Timoshenko and Gere (1961) provided the k_v values for different boundary conditions and aspect ratios of the plate. It is recommended to assume simply supported boundary conditions (McCloskey, 2006) for which k_v is approximated by Equation 13.

$$k_v = 5.34 + 4.0 \left(\frac{B}{h} \right)^2 \quad (13)$$

The capacity of an SSP is given by Equation 6 or 7, which should be less than the minimum of the three other possible limit states discussed earlier. This is shown in Equation 14.

$$Q_{panel} = Q_{ult} (< Q_{GLTB}, Q_{LTB \text{ links}}, Q_{scr}) \quad (14)$$

Equation 14 is used to enforce ductile and stable behavior of an SSP; Q_{panel} should be equal to Q_{ult} . This ensures that the SSP is able to reach its ultimate strength (Q_{ult}) without any other limit state occurring, which is when all the links are able to develop their plastic moment capacity at both ends. For all panels studied experimentally (Cortes, 2009), with the exception of the $\frac{3}{16}$ -in. panels, the Q_{ult} was the controlling limit state. For the $\frac{3}{16}$ -in. panels, the equation for global lateral-torsional buckling (Equation 8) controlled over the Q_{ult} equation, but Q_{GLTB} was about 91% of Q_{ult} . Even

though the LTB limit state controlled for these panels, they still exhibited stable loops and no strength degradation until at least 4% interstory drift.

The preferred material for plates is the ASTM A36 (AISC, 2005b), with a minimum specified yield stress of 36 ksi. Although A572 steel may also be used for plates, it might be better to use A36, because the links should develop their plastic moment capacity before other instabilities occur. Steel with a higher yield capacity will increase the ultimate strength of the panel (Q_{ult}), but the capacity of the panel may now be controlled by the other limit states, which do not depend on the yield strength. Figure 7 shows a

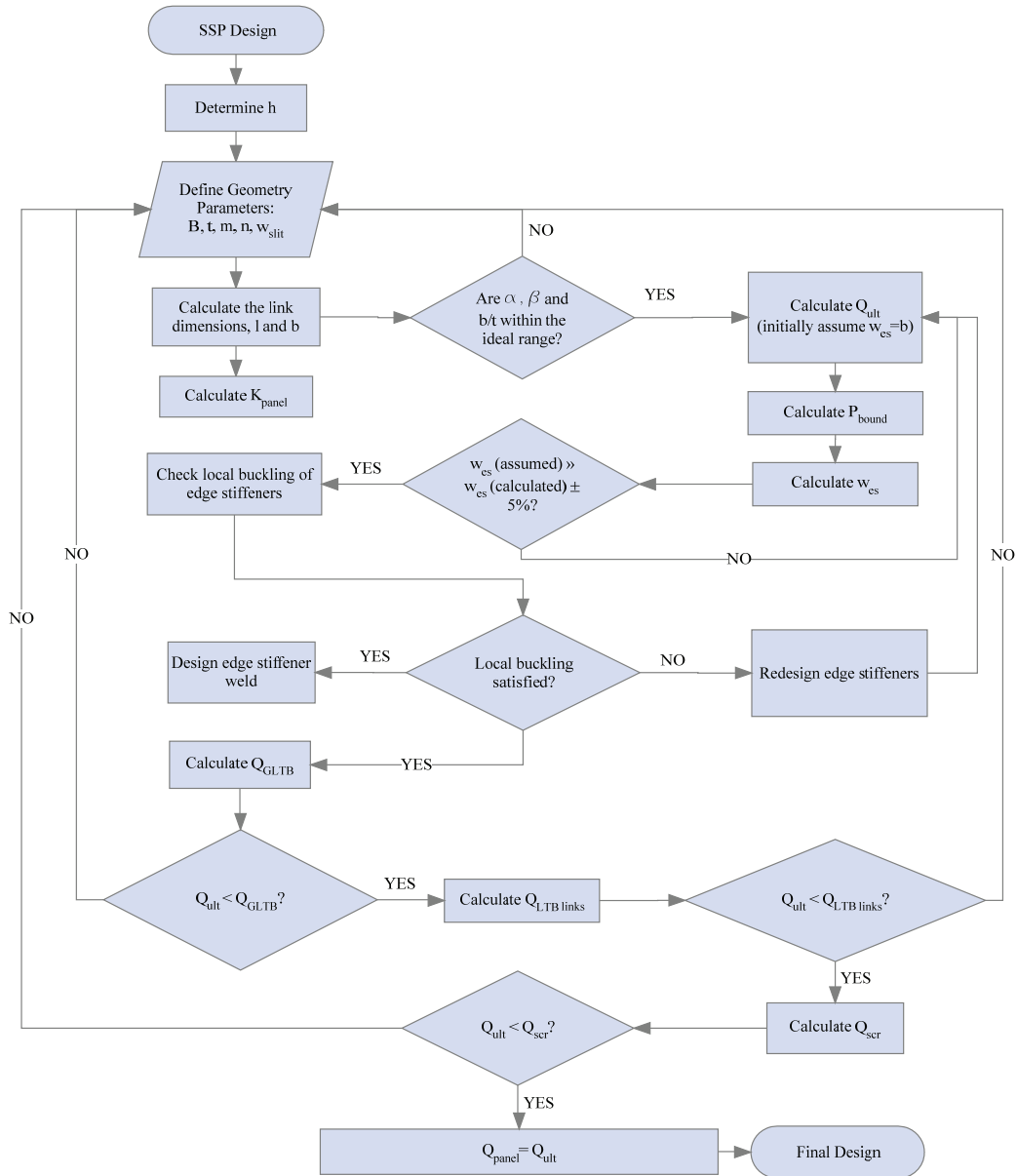


Fig. 7. SSP design flowchart.

flowchart that summarizes the steps required for the design of a steel slit panel.

STEEL SLIT PANEL FRAME

A steel slit panel frame (SSPF) is a frame that uses SSPs to resist the base shear and to provide the required stiffness. The SSPs are connected to the beams, using moment resisting connections (see Figure 14). In an SSPF, beams are pin connected to the columns and columns are pin connected to the foundation. An example of an SSPF is given in Figure 3. Design of an SSPF includes sizing of the frame elements (beams and columns) and determining the number of panels required for story shear demands and check of interstory drift limits, with consideration for the effects of the panel-frame interaction on the panel stiffness. Note that atypical load patterns develop in the beams when the frame is subjected to lateral loads. These loads, caused by the SSPs, need to be fully considered for the design of the beams.

One may determine the number of panels required by dividing the story shear by the capacity of the panel to be used (Equation 15).

$$\text{Number of panels}_{v_i} = \frac{V_i}{Q_{\text{panel}}} \quad (15)$$

where

V_i = story shear demand

Q_{panel} = capacity of the panel, determined from Equation 14

For determining the base shear demand, a response modification factor, R , of 5.5 is recommended. Once the SSPF has been designed for strength, the next step would be to verify that interstory drifts are within limits. When verifying

drift limits using the linear elastic method, a C_d value of 4.5 should be considered. The R and C_d values were preliminarily established from engineering judgment, based on similar seismic systems, by the project oversight committee. This committee included practicing engineers, fabricators and researchers. While both the R and C_d correspond well to nonlinear analyses conducted as part of this study (Cortes, 2009), both should be verified following the FEMA P695 methodology (FEMA, 2009).

Because the SSPF system is relatively flexible, interstory drifts may dictate the number of panels required at each story. Thus, it may be more practical to initially estimate the number of panels required to satisfy interstory drift limits. The number of panels required to satisfy interstory drift limits may be obtained from Equation 16.

$$\text{Number of panels}_{\kappa_i} = \frac{K_{ei}}{K_{eff}} \quad (16)$$

where

K_{ei} = elastic stiffness demand in story i (given by Equation 17)

K_{eff} = effective stiffness provided by the panels within the frame (from Equation 18)

$$K_{ei} = \left(\frac{C_d}{I} \right) \left(\frac{V_i}{\Delta_a} \right) \quad (17)$$

where

I = importance factor according to ASCE 7 (ASCE, 2005)

Δ_a = allowable displacement according to ASCE 7 (ASCE, 2005)

While the SSPs provide the stiffness in the frame, the

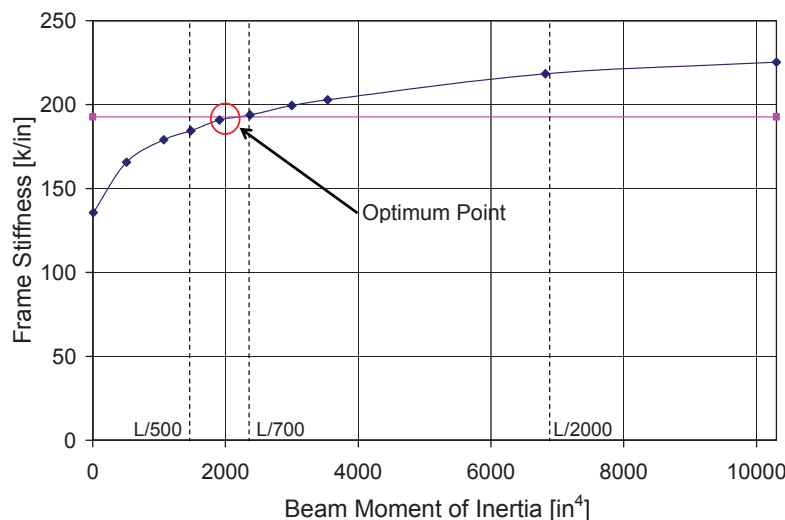


Fig. 8. Frame stiffness as a function of the beam's moment of inertia.

panel-frame interaction plays a major role in the stiffness provided by the SSPF. Here, K_{eff} is the panel stiffness as modified by the frame, and K_{panel} (Equation 4) estimates the stiffness of a panel subjected to relatively stiff boundary conditions. However, the actual boundary conditions provided by the beams depend on their flexural stiffness. Also, at stories above the first story, the rotation of the frame further reduces the stiffness provided by the panels. In addition, the number of bays in the SSPF plays a significant role in the effective stiffness provided by a panel. The effective stiffness provided by an SSP may be determined from Equation 18.

$$K_{eff} = [R_{bm} R_{Story}] R_{\# \text{ Bays}} K_{panel} \quad (18)$$

Equation 18 considers the reduction in stiffness caused by the flexibility of the beam, R_{bm} , the story where the panel is placed, R_{Story} , and the number of bays in the SSPF with panels, $R_{\# \text{ bays}}$. These factors are multiplied by the stiffness of the panel, K_{panel} (Equation 4), to obtain the effective stiffness provided by the panel. The following paragraphs explain the factors used in Equation 18 (i.e., R_{bm} , R_{Story} and $R_{\# \text{ Bays}}$).

The factor R_{bm} accounts for the flexibility of the beam in the frame. A beam with a low flexural stiffness will lower the effective stiffness provided by the panels it bounds. Meanwhile, a beam with a higher flexural stiffness will provide stiffer boundary conditions to the panels, and therefore, higher SSPF stiffness. Figure 8 shows a plot of frame stiffness for a one-story, one-bay frame, with column spacing equal to 30 ft, a story height of 13 ft, and a 1/2-in.-thick panel placed at the center of the bay and bounded by beams with different moments of inertia. Note that as the moment of inertia of the beam bounding the panel increases, the frame stiffness also increases. The horizontal line represents the stiffness predicted by Equation 4. The optimum point, also shown in the figure, represents the intersection of the stiffness predicted by Equation 4 and the actual stiffness obtained from the plot. For this frame, the optimum point was reached for beams that satisfy a deflection limit of nearly $L/700$ [deflection limit for a uniformly loaded simply supported beam, assuming no panel, subjected to service gravity loads ($D + L$)], where L is the beam span. Note that a beam should satisfy a deflection limit of $L/500$ to limit initial stresses due to gravity loading (McCloskey, 2006).

Finite element analysis (FEA) revealed that in frames with more than one story, the stiffness decreases as a function of the story in the building, decreasing from the bottom floor to the upper floor. This was confirmed by experimental results (Cortes, 2009). For example, Figure 9 shows the deflected shape of a three-story SSPF with a 30-ft bay and 13-ft stories. The story shear at the first story causes a rotation, θ_1 . This initial rotation in the first story is passed to the second story. Thus, the total rotation at the second story is the rotation from the first story, θ_1 , plus the rotation due to

the shear demand on the second story, θ_2 . This effect can be observed in Figure 10, which shows the story stiffness (normalized by K_{panel}) as a function of the beam's moment of inertia. The stiffness obtained by the first story is higher than that in the second story; the stiffness is further reduced at the third story. Note also that only the first story reached the optimum point; however, it requires a higher beam moment of inertia than the case with only one story. Figure 10 considers both the effect of the beam flexibility and the frame rotation.

In Equation 18, the reduction factor for the beam's flexural rigidity and the story where the panel is placed are inside of a bracket to indicate that these two factors may be obtained from one plot such as Figure 10 or Figure 11. Figure 11 shows a plot of the floor number versus story stiffness (panel stiffness, normalized by K_{panel}) for one-, two-, three-, four- and five-story frames. All frames studied had 30-ft bays, story heights of 13 ft, W24x84 beam sections, and 1/2-in. panels. In this plot, the abscissa represents the story stiffness and the ordinate represents the story number. The stiffness at the top floor of a frame having n number of stories is related to the stiffness at the top floor for a frame having $n - 1$ number of stories, $n - 2$, and so on. In other words, the stiffness of the upper story was compared for all frames. Then, the stiffness of the story below the upper story was compared for all frames, and so on. A similar pattern was observed for the stiffness in each of the floors, and a regression can be used to fit the values of all stories. The dashed line is a power regression used to fit all data points. The only floor where the average line has a large variation is at the first floor.

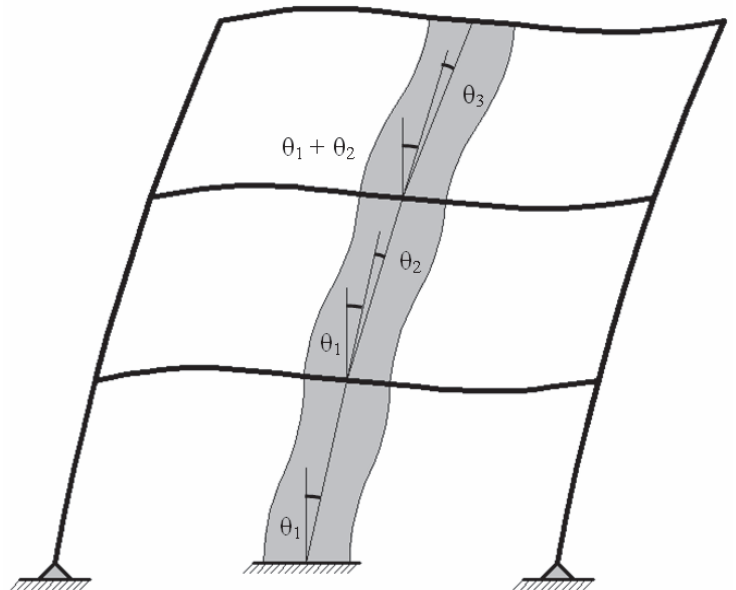


Fig. 9. Displaced shape of a three-story SSPF.

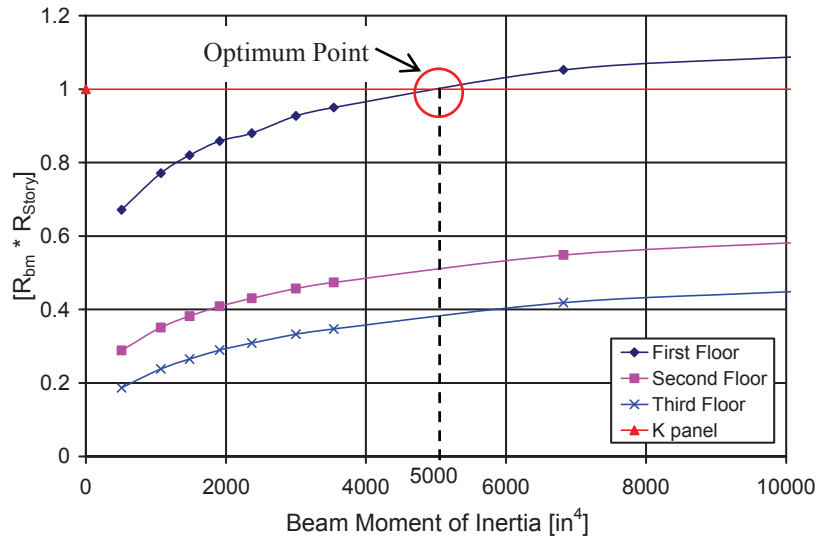


Fig. 10. Reduction caused by frame rotation for a three-story SSPF.

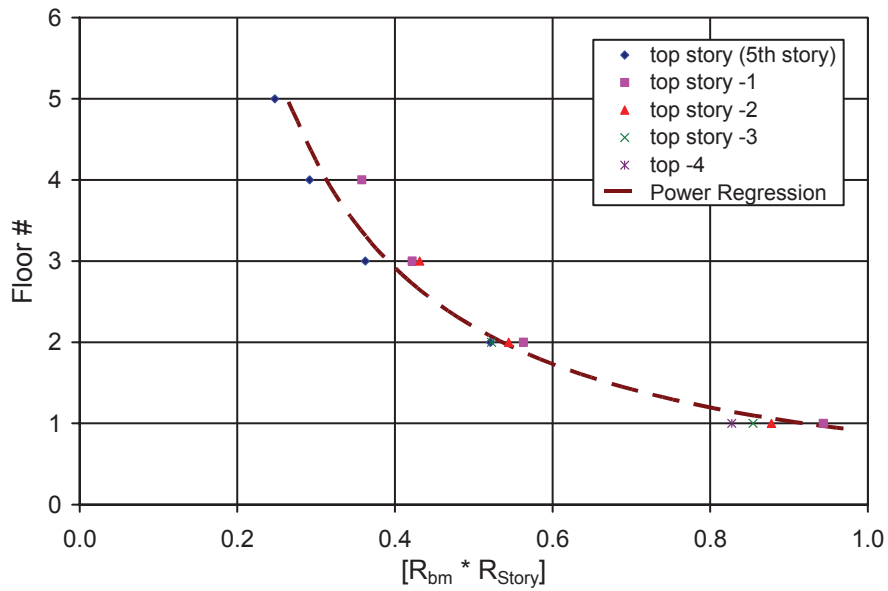


Fig. 11. Stiffness as a function of the floor number.

Nevertheless, this plot results in a very good approximation for estimating the reductions in stiffness caused by the beam flexibility and by the frame rotation.

The other main factor that affects the stiffness in a frame is the number of bays with panels in the SSPF. Parametric studies of the frame revealed that as more bays in an SSPF are filled with panels, the effective stiffness provided by the panels increases. For example, the effective stiffness provided by the panels in a frame with five bays with panels is higher than the effective stiffness of the panels in a frame with two bays with panels. Five 10-story frames with 30-ft-wide bays and two $\frac{3}{4}$ -in. panels at each bay were studied. All beam sections were W24 $\times$ 117; columns were W14 $\times$ 90 at all stories. For these frames, all the properties were the same except for the number of bays. The number of bays with panels was varied from two to five. The results of this study are summarized in Figure 12. The abscissa in this plot represents the stiffness provided by each panel at each story, normalized by the panel stiffness (obtained from Equation 4). The floor number is given in the ordinate.

Note that as more bays are part of the SSPF, the effective stiffness for each panel increases. The stiffness at each panel in the tenth floor is about 20% higher for the five-bay frame compared with the two-bay frame. This increase in stiffness is caused by the additional columns in the seismic frame. The beams are pin connected to the columns; however, because the columns are continuous from the foundation to the top (assuming that the column splices provide continuity), those adjacent to bays with panels appear to provide additional shear stiffness and flexural stiffness.

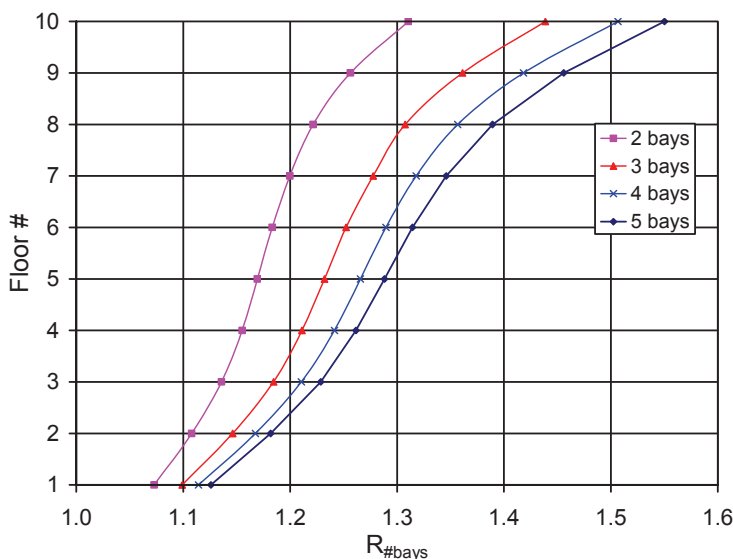


Fig. 12. Stiffness increase for more bays.

Determining the effective stiffness provided by a panel (K_{eff}) might be cumbersome initially because the analysis of many models is needed initially. However, Figures 10 and 11 may be used to approximate the number of panels required to satisfy stiffness. Figure 11 combines factors R_{bm} and R_{Story} into one. Figure 12 provides the $R_{\#bays}$ factor for certain

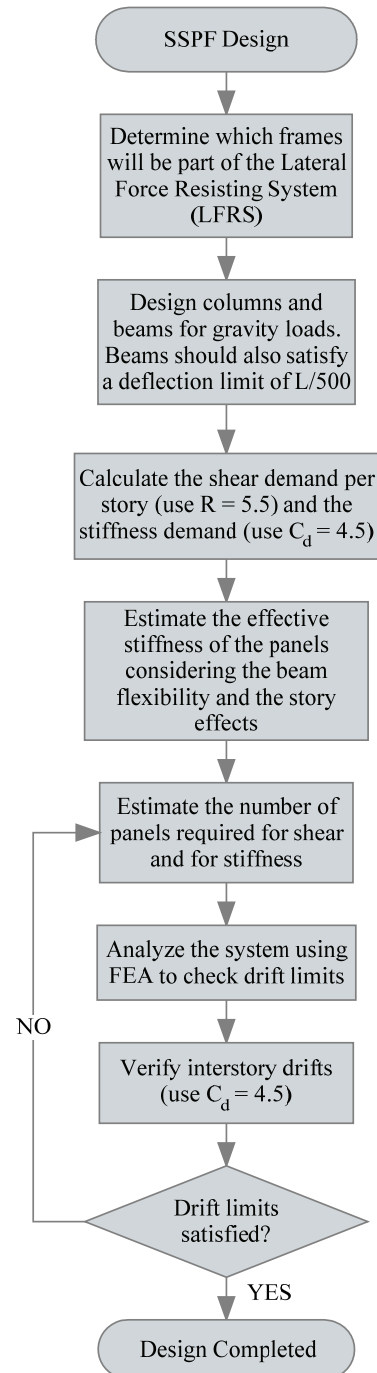


Fig. 13. SSPF design flowchart.

configurations. In an example shown in a later section, Figure 11 alone was used to estimate the required number of panels to satisfy drift limitations. Although these two plots were developed for one specific frame and panel geometry, they may be used to obtain an initial estimate.

Figure 13 shows a flowchart that summarizes the main steps for designing an SSPF.

STEEL SLIT PANEL-BEAM CONNECTION

SSPs are connected to the beams using fin plates. The fin plates are shop welded to the beams and bolted to the SSPs. These connections must be able to transfer the shear force and the moment generated by the panel. Slip of the connection is undesirable at service loads (dead, live and wind loads). Thus, high-strength structural bolts pretensioned on faying surfaces as specified by AISC 341 (AISC, 2005c) should be used. The design load for the connection (i.e., fin

plate to beam and fin plate to SSP) is the ultimate capacity of the panel multiplied by 1.3. This factor accounts for the possibility of the yield strength of the panel being higher than the minimum specified yield strength and is the recommended R_y value for A36 steel used for plates (AISC, 2005c).

Bolt slip, shear and bearing capacity must be considered for both shear and overturning moment ($1.3Q_{ult} h/2$) demands. Moment capacity and shear capacity of the fin plate (i.e., shear yield, shear rupture, block shear rupture) must also be considered. The design load and moment should also be used to design the weld that connects the beam to the fin plate.

The connection must provide a uniform level of restraint to all the links. Special attention must be paid to the two exterior links. The bolts must extend to cover the exterior links. Figure 14 shows two connection details; the left detail is not recommended because experimental results have

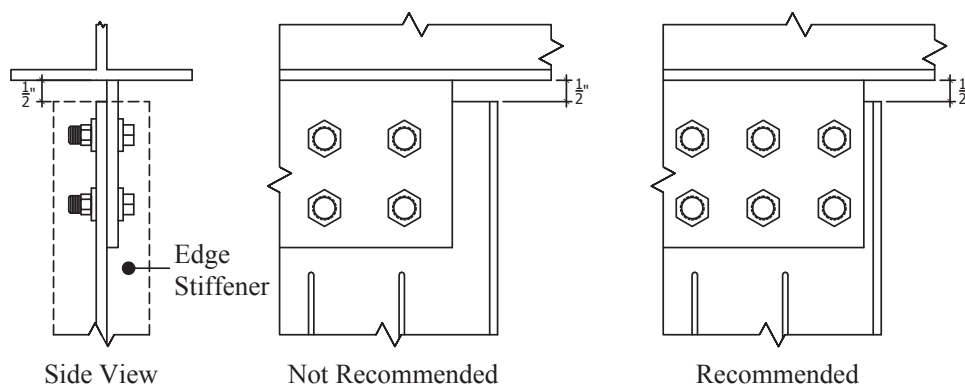


Fig. 14. Connection detail.

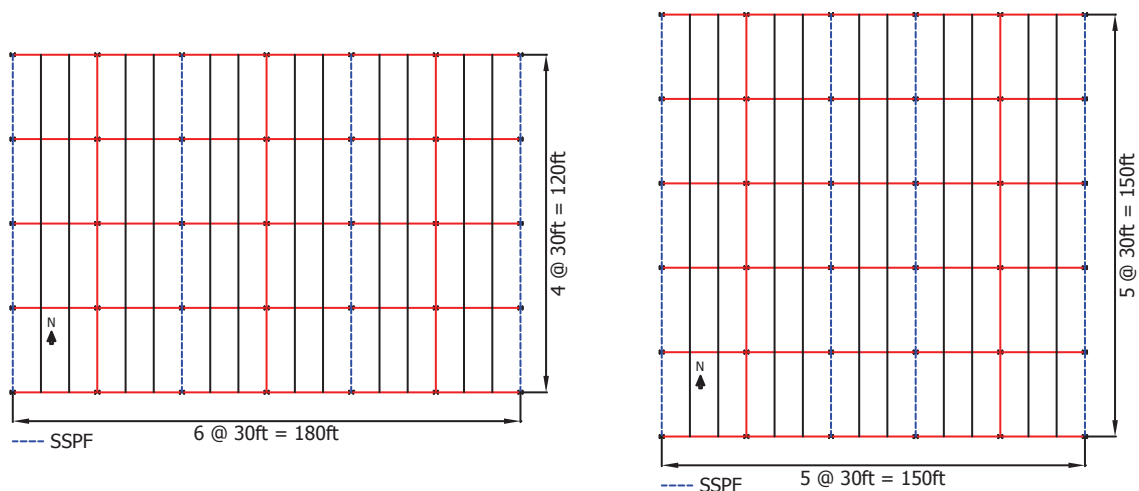


Fig. 15. Plan view of the three-story prototype building (left) and ten-story prototype building (right).

Table 1. T1 Panel Properties									
m	n	B (in.)	H (in.)	T (in.)	I (in.)	b (in.)	α	β	b/t
3	9	59.375	130	0.5 or 0.75	32	6.375	4.9	0.72	12.8

Table 2. Required Strength and Stiffness		
Story	V_i (kips)	K_{ei} (kip/in.)
1	509	587
2	428	494
3	266	307

Table 3. Required Number Panels for Strength					
Floor	Force (kips)	Panel Thickness (in.)			
	V_i	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$
1	509	4.9	3.7	2.9	2.4
2	428	4.1	3.1	2.5	2.0
3	266	2.6	1.9	1.5	1.3

shown that it will not provide enough restraint to the exterior links. The detail shown on the right is expected to provide a more uniform restraint to all the links. The exterior links will not be able to reach their ultimate strength if they are not well restrained. The stiffness of the panel may also be affected.

Other types of connections may be used (e.g., single-angle connections). However, all tests to date were performed for panels connected using fin plates. Thus, testing other connections prior to their use is recommended.

PROTOTYPE BUILDING DESIGN

This section presents an overview of the design of two prototype buildings using SSPFs as their SFRS. The two prototype buildings were designed according to ASCE 7 (ASCE, 2005) for a site in Los Angeles, California. Both buildings were designed based on seismic weights of 125 psf at the roof and 114 psf at intermediate levels (Richards and Uang, 2003). These weights include dead load and 25% of the live load. The first prototype building is a three-story building with a rectangular plan. It has four bays in one direction and six in the orthogonal direction. All bays are 30 ft wide. The second prototype building is a ten-story building with a square plan and five 30-ft bays in each direction. For both

buildings, the floor to-floor height of all stories is 13 ft. Figure 15 shows the plan views of both prototype buildings. The dashed lines indicate the bays where panels are placed. For the three-story building, $\frac{1}{2}$ -in. Type 1 (T1) panels (Cortes, 2009) were used. For the ten-story building, $\frac{3}{4}$ -in. T1 panels were used. The main dimensions and parameters for the $\frac{1}{2}$ -in. and $\frac{3}{4}$ -in. T1 panels are presented in Table 1.

The following paragraphs present the design process for the three-story building; for the ten-story building, only the results are presented. Following the design flowchart shown in Figure 13, the SSPFs were identified; these are indicated in Figure 15 with the dashed lines. Beams and columns were designed for gravity loads; beams also satisfied a deflection limit of $L/500$. The deflection limit was checked for a simply supported beam subjected to service dead and live loads ($D + L$), assuming a 50-psf live load for office buildings (ASCE, 2005). The moment of inertia required to satisfy the $L/500$ limit is 1912 in.⁴ (assuming a tributary width of 15 ft). W24×84 beams (2,370 in.⁴) were provided. W14×90 sections were used for the columns. Lateral demands were also calculated following ASCE 7 (ASCE, 2005). Table 2 shows the required shear strength and lateral stiffness (Equation 17) per story. The importance factor (I) was taken as 1. Table 3 shows a summary of the required number of T1 panels with thicknesses of $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$ and $\frac{3}{4}$ in. to resist story shears. The capacities of these panels were calculated with Equation 14. The next step would be to estimate the number of panels required to satisfy drift limits. Given the complexity of the panel-frame interaction and its effect on stiffness, one should then verify drift levels with a finite element model. All attributes of the panel (i.e., links, slits, edge stiffeners) should be included in the finite element model. Shell elements shall be used to model the panel. As an alternative to using shell elements, a simplified model of the SSP is presented in the next section. Table 4 shows the stiffness demand, K_{ei} , the reduction factor, $[R_{bm} \times R_{Story}]$, the effective

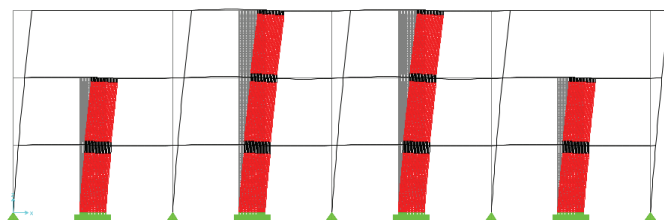


Fig. 16. Three-story building, second configuration.

Table 4. Required Number of Panels for Stiffness (½-in. panels)				
Floor	K_{ei} (kip/in.)	$[R_{bm} \times R_{Story}]$	$K_{eff} = K_{panel}[R_{bm} \times R_{Story}]$ (kip/in.)	Number of Panels
1	587	0.92	175	3.4
2	494	0.54	103	4.8
3	307	0.4	76	4.0

Table 5. Interstory Drifts for Configuration 1				
Floor	δ_{xe} (in.)	Δ (in.)	Δ_a (in.)	$\Delta < \Delta_a$
1	0.6	2.7	3.9	True
2	1.3	3.3	3.9	True
3	2.0	2.8	3.9	True

Table 6. Interstory Drifts for Configuration 2				
Floor	Deflection	Δ (in.)	Δ_a (in.)	$\Delta < \Delta_a$
1	0.65	2.92	3.90	True
2	1.47	3.69	3.90	True
3	2.44	4.37	3.90	False

stiffness of the panel, K_{eff} , and the number of ½-in. panels required to satisfy drift limits. The reduction factor was obtained from Figure 11, which only considers the flexibility of the beams and the story effects. In other words, the $R_{\#bays}$ was taken as 1. This is a conservative assumption because this value is always higher than 1. This should yield a reasonable approximation; furthermore, interstory drift values will be checked using finite element analysis. Observe that drift requirements control the required number of panels for the second and third story; the number of panels required in the first story is controlled by the shear demand. A few steel slit panel frame configurations with panels located in different locations were studied; two are discussed next.

The first configuration studied is shown in Figure 15. Four panels were used at all three stories. The finite element software SAP2000 was used to analyze the frame. Frame elements were used to model the beams and columns, and shell elements were used to model the panels (Cortes, 2009). The models of the panels included the edge stiffeners and the slits. The panels were connected to the beams using rigid links. The interstory drifts obtained for this configuration are shown in Table 5. This table shows the interstory drifts obtained from the elastic analysis (δ_{xe}); the drift values (Δ), which are amplified by C_d to account for the nonlinear effects; and the allowable interstory drift (Δ_a). The allowable drift is 3.9 inches, which corresponds to a 2.5% interstory drift. All stories satisfied interstory drift limits for this configuration.

The second configuration has four panels in the first and second stories and two panels in the upper story. All panels were located at the center of the bay. Figure 16 shows a deformed shape of the second configuration. Table 6 shows a summary of the drift values obtained for the three stories. Note that the third story did not satisfy drift limits. One could add more panels in this story or use thicker panels or use beams with a higher moment of inertia. For this example,

the simplest solution would be to use beam sections with a higher moment of inertia. If W24×103 beams are used, all three stories satisfy interstory drift limits.

A similar procedure was performed for the ten-story prototype building. Two ¾-in. T1 panels were used at each bay. The SSPF used for the ten-story building is shown in Figure 17. Figure 17 also shows the interstory drift values obtained from the elastic analysis and amplified by C_d . Note that the interstory drift limit of 2% (ASCE, 2005) is satisfied for all stories.

SSP SIMPLIFIED MODEL

An SSP can be modeled in FEA software using shell elements. Its geometry needs to include the slits—and thus the links—and the edge stiffeners. Figure 18 shows a panel

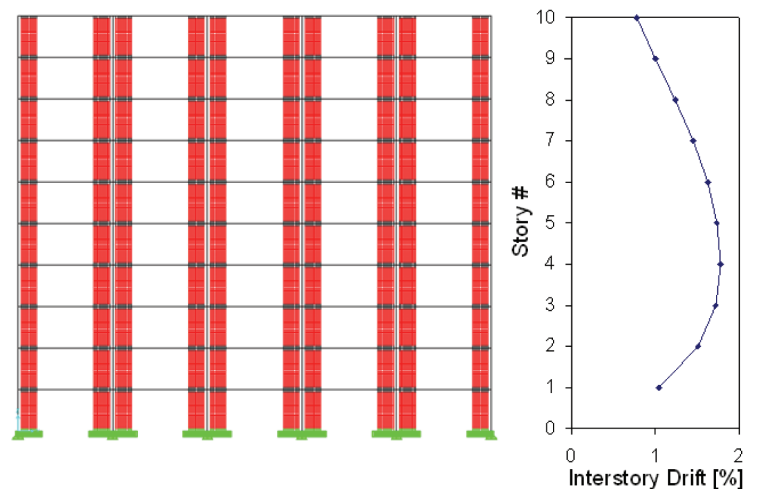


Fig. 17. Ten-story frame used in prototype building.

modeled in the FEA software SAP2000. While the most proper way of modeling the panels is using shell elements, there are reasons a simplified model may be needed. The first reason for a simplified model is that some FEA software may not have the capacity to model shell elements. Even if the software has the capacity to model the panels using shell elements, these may be limited to the elastic range. While most analyses for design are performed in the elastic range, more advanced analyses require the consideration of the material plasticity. This is especially true for the SSPs, which undergo much yielding at the link ends. The simplified model presented next is capable of capturing yielding at the links. The second reason for a simplified model is that it is simpler to model and thus is less computationally expensive than a model using shell elements.

The simplified model uses frame elements to model the SSP. The frame elements have cross sections equivalent to the actual dimensions of the links and the band zones. For

the external links, the edge stiffener is modeled together, creating a T-shaped section. For each of the interior links, the centroid coincides with the geometrical center, but for the exterior T-shaped links, calculations are needed because the frame element should coincide with the centroid of the T-section. Rigid elements are used to connect the links to the center of the band zones. If it is desired to consider the material nonlinearity, each link may be modeled with plastic hinges at both ends to capture the plasticity at these regions. This is a good approximation because yielding of the panels only occurs at these regions. Figure 19 shows the simplified model. In this figure, the black circles represent the plastic hinge locations in the panel. The plastic hinge properties are based on the moment-rotation curves of the links.

The moment-rotation curve used to define the plastic hinges is very important because it dictates the global behavior of the panel. Figure 20 shows the moment-rotation curve recommended by ASCE 41 (ASCE, 2007). Point B

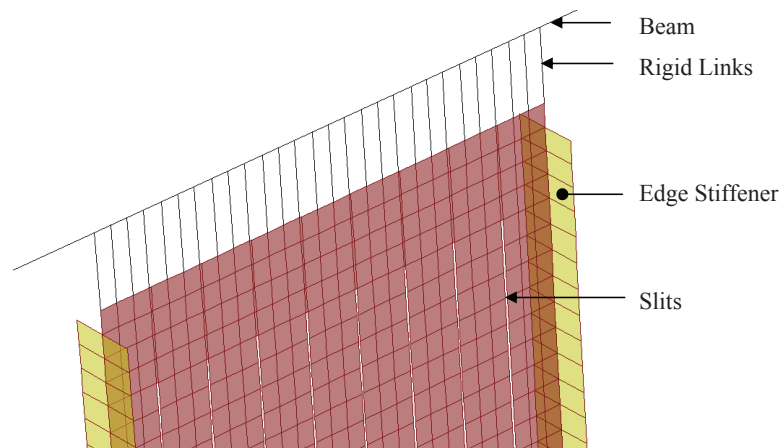


Fig. 18. Finite element model of the panel using shell elements.

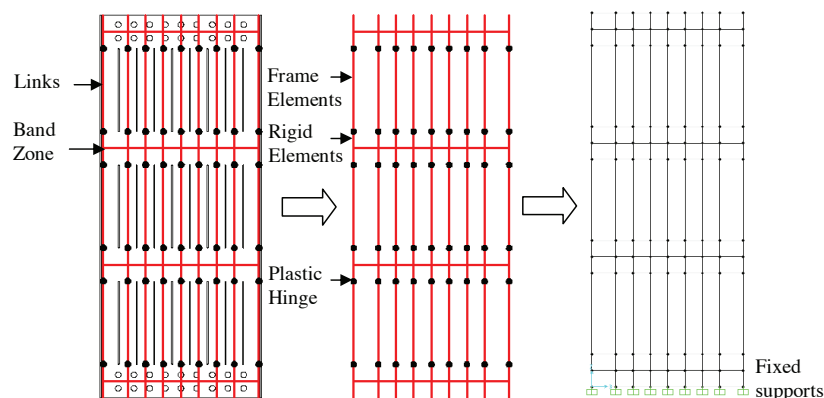


Fig. 19. Simplified panel model.

represents the point of first yield at the link. The yield moment, M_y , is given by Equation 19. In this equation, S_x is the section modulus.

$$M_y = S_x F_y \quad (19)$$

The yield rotation, θ_y , may be obtained from Equation 20 (ASCE, 2007). This equation assumes that the point of contraflexure occurs at mid-height of the link. In this equation, Z_x is the plastic modulus of the link and l is the length of the link; other parameters were previously defined.

$$\theta_y = \frac{Z_x F_y l}{6EI_{link}} \quad (20)$$

Point C of Figure 20 is the moment that causes the link to fail. Conservatively, this point was calibrated to occur when the panel reaches a displacement of 4% of its height. The moment corresponding to point C is the plastic moment of the section. For square sections, as is the case of the inner links, the plastic capacity of the section is 1.5 times the yield moment capacity. However, a value of 1.1 of the yield moment capacity is suggested for two main reasons: (1) the full plastic moment capacity may not necessarily be developed (e.g., if lateral-torsional buckling or shear buckling occurs) and (2) experiments correlated better with the suggested value of 1.1. In addition, the FEMA 356 Commentary (FEMA, 2000) mentions that the slope from B to C is usually between 0 to 10%. Point D was taken as the rotation corresponding to 4.5% interstory drift, and it was assumed that the residual moment capacity is 0.3 times the yield moment capacity. The 4.5% drift value was used to avoid a sharp transition from point C to D; otherwise, computational difficulties may occur. The residual strength was conservatively

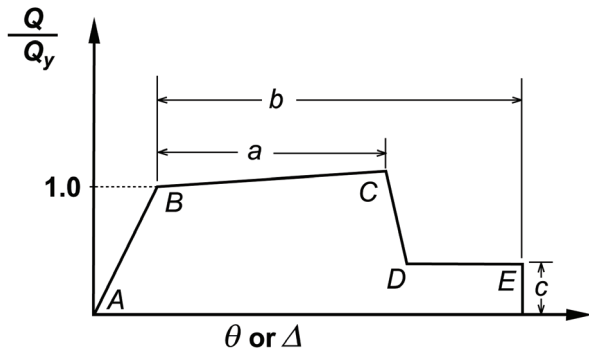


Fig. 20. Moment-rotation curve used for hinges (ASCE, 2007).

assumed as $0.3M_y$ to account for the capacity of the panel to resist loads after drift values beyond 4.5%. Point E was defined at 6% interstory drift based on experiments. However, it was observed that even if no residual strength had been assumed, similar results would be achieved.

The rotation at the links may be calculated from the rotation of the story, which is equivalent to the interstory drift. Because only the links rotate (the band zones do not rotate), the rotation at the links is calculated by multiplying the rotation of the story (interstory drift) by the ratio of the story height to the height of all links. This is expressed in the following equation:

$$\theta_{link} = \left(\frac{H_{story}}{ml} \right) \theta_{story} \quad (21)$$

In this equation, H_{story} represents the story height, and ml is the number of rows of links, m , multiplied by the height of one link, l . Note that this equation is used to convert the global behavior of the panel, observed in experimental tests, to local behavior in the links. Thus, the interstory drifts at which points B, C, D and E of Figure 20 occur are converted to rotation at the links by means of this equation.

To validate the simplified model (considering material plasticity), the results from the two experimental tests were compared to simplified models of the specimens. Figure 21 shows the moment-rotation curves used for modeling the nonlinear hinges in the interior and exterior links. Figure 22 shows the load-displacement curve for one test and the results from a nonlinear static analysis of the simplified model. The results from the simplified model compare fairly well with the experiment.

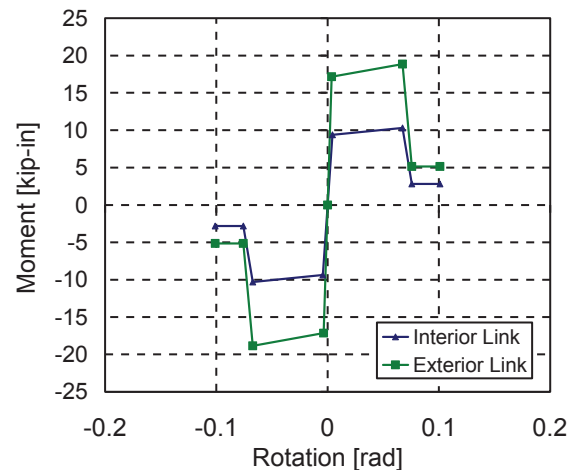


Fig. 21. Moment-rotation curve used for PF-3 panel hinges.

CONCLUSIONS

Steel slit panels (SSPs) exhibit ductile behavior and can be used in steel slit panel frames (SSPFs) for lateral resistance. The SSPF is designed to resist 100% of the base shear. The procedures for design of an SSP and SSPF have been provided. Recognizing that drift is likely to control the design, a method for designing the frame to satisfy drift limits was explained. Two prototype building designs were presented: one building has three stories; the other has ten stories. Two configurations were studied for the three-story frame. This study indicates that SSPFs are a viable SFRS for low- to medium-rise buildings.

Future research should be carried out to assess the performance of the system according to ASCE 41 (ASCE, 2007). In addition, the FEMA P695 methodology (FEMA, 2009) should be followed to determine seismic design parameters such as R and C_d . Finally, further experimental studies (e.g., quasi-static cyclic tests, shaking table tests) of full-scale frames are highly recommended. These would allow further verification of the design formulas and design approach explained in this paper.

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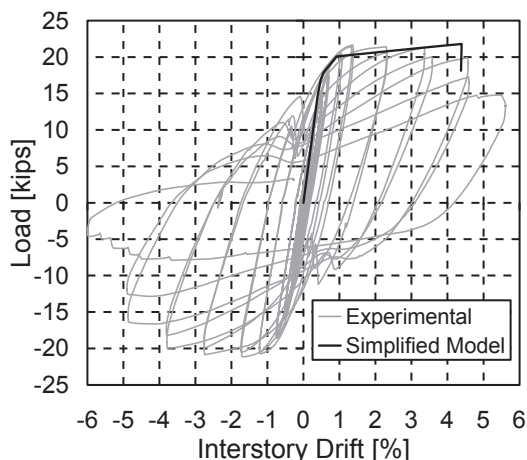


Fig. 22. PF-3 load-displacement curve from test and from simplified model.

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