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Experimental evaluation of steel slit panel–frames for seismic resistance

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ABSTRACT

The steel slit panel–frame is a new system for seismic resistance of buildings. The steel slit panels are bolted to beams which are simply connected to the columns. The steel slit panels are steel plates with rows of vertical slits forming a series of flexural members within the plate. The steel slit panels are designed to resist the entire lateral load and provide all the stiffness and energy dissipation in the system. The steel slit panel–frame system was studied via an experimental program, divided into two series of testing. The first series studied the fundamental characteristics of the steel slit panels, while the second series studied the behavior of the steel slit panels within the frame (i.e., behavior of the steel slit panel–frames). A concise introduction to the system is presented first, followed by the findings from the experimental study.

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1. Introduction

In the 1994 Northridge earthquake, a number of steel moment-frame buildings experienced brittle fractures of beam-to-column connections [1]. This event spurred a number of studies on topics such as methods to improve the behavior of moment resisting frames. Among the results is the specification of "Pre-qualified Connections" [2] for new moment resisting frames. Another result of the Northridge event is that numerous alternatives to the moment resisting frame have been studied; one such alternative is the Steel Slit Panel–Frame (SSPF).

The SSPF system is a Lateral Force Resisting System (LFRS) developed for buildings located in seismic regions. Its main components are columns and beams, simply connected, and Steel Slit Panels (SSPs). An SSP is a steel plate with vertical slits cut to create a number of rows of flexural members, called links. The SSP is bolted at top and bottom to the beams. When the panel (SSP) is subjected to lateral deformations, the links behave as beams in double curvature, reaching their plastic moment capacity at both ends and dissipating energy. In addition, the panel has a pair of vertical edge stiffeners mainly intended to provide out-of-plane stability to the panel, but these stiffeners also increase the panel's strength and stiffness. SSPs may be placed at selected bays to provide the entire design shear resistance and the stiffness for the system. Fig. 1 shows an SSPF with two panels per bay and an amplified view of one of the panels. The main parameters describing an SSP are the panel's width, B, the panel's height, h, the panel's thickness, t, the number of rows of links, m, the number of links in a row, n, the height of the links, l, the width of the links, b, and the height of the band zones, h_{bz}. These parameters are also illustrated in Fig. 1.

An SSPF has numerous advantages: (1) the system is highly ductile; (2) the panels allow for architectural flexibility; (3) the fabrication process for the SSPs is relatively simple; (4) no field welds are required; (5) the panels are easily field bolted to the beams; (6) damaged panels can be easily replaced; (7) the system may be used to retrofit or strengthen existing structures.

This paper presents the experimental work conducted on the SSPs and the SSPF system. A brief summary of the most relevant work conducted previously is presented first. Then, fundamental information about the SSPs and the SSPFs is provided. A discussion of the three panel configurations studied follows. A brief explanation of the finite element models (FEMs) used for studying the system analytically is presented. The last sections explain the experimental program, including the main parameters studied in each test. The results of the experiments are presented and compared against predictions from equations and from finite element models.

2. Previous research

Slitted reinforced concrete (RC) walls were introduced as energy absorbers in 1968 [3,4]. This system has been referred to as the first passive structural control system [5]. The slitted wall is a reinforced concrete wall with equally spaced slits located at mid-height of the wall. The slits were made by completely cutting the concrete and the steel reinforcement. The slitted RC wall provided more ductility than solid reinforced concrete shear walls, but had a lower strength. At low levels of drift, the slitted wall worked as a shear wall, limiting deflections. At large drift levels, the slitted wall worked as a series of flexural members, dissipating

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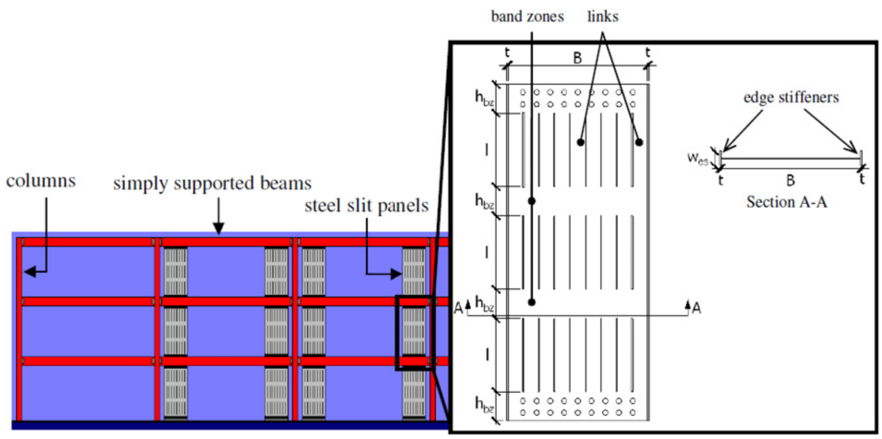

Fig. 1. Steel slit panel–frame.

energy. In contrast with typical RC walls that are continuous all the way from their foundations to the tops, the slitted walls were placed between beams in a story. This system was implemented in the first high-rise building in Japan, the 36-story Kasumigaseki building in Tokyo. After the Kasumigaseki building, over a dozen other buildings were built using slitted RC walls. While the slitted walls added ductility, they also added substantial weight to the structure and consequently increased the inertial forces generated by the earthquake motion. Moreover, reinforced concrete suffers rapid deterioration under cyclic plastic deformation. The added weight and rapid deterioration are believed to be the reasons that ended the use of this system [6].

Hitaka and Matsui [7] conducted a thorough experimental and analytical study of the steel wall with slits. Their system is similar to the RC slitted wall, but instead of using reinforced concrete walls, they used thin steel walls. They studied the effect of the slit patterns, by varying three main parameters: b/t (width to thickness ratio of the links), α (link length to thickness ratio), and m (number of rows of links). They also studied the effects of the stiffeners on the panels. Most of the specimens tested underwent a story drift ratio of more than 3% without initiation of cracks or abrupt strength degradation. Hitaka and Matsui observed that transverse (out-of-plane) buckling of the plate was the main cause of strength degradation.

In a separate research study, Hitaka and Matsui [8] studied the interaction of the slit walls with the moment frame. In this research Hitaka and Matsui tested 6 specimens using scale models that were 1/3–1/4 of the full scale model. Each of these tests was conducted under cyclic incremental loading. Two different slit walls were used; the overall size was the same for both specimens but the number of links changed. From this investigation, the authors found that the stiffness of the beam over the slit wall plays a major role. The stiffness of the wall was reduced in comparison with previous tests in which the walls were attached directly to a much stiffer loading beam.

The primary goal for the slit walls as used by Hitaka and Matsui was to provide an energy dissipating system in combination with the typical moment resisting frame utilized in Japan. Three buildings were designed in Japan using moment frames in combination with slit walls (dual system). The buildings built ranged from 7 to 19 stories in height. The steel walls with slits were designed to take 10%–25% of the seismic base shear. The remaining base shear was resisted by the moment frames.

The steel wall with slit system is similar to the SSPF. The two main differences are: (1) SSPFs are designed to resist the entire base shear while steel walls with slits are designed to take only 10%–25% of the total base shear, thus requiring moment frames in a dual system, and (2) the height to width aspect ratio of the SSPs is nearly 2:1, while for the steel walls with slits, the ratio is 1:1 or larger, usually blocking more of the bay.

McCloskey [9] proposed the use of SSPs with a 1:2 width to height aspect ratio. He considered the different parameters that control the behavior of the panel ($b/t, \alpha, m$) and verified the applicability of the strength and stiffness formulas proposed by Hitaka and Matsui [7] for the SSPs. McCloskey also evaluated equations for the different failure mechanisms for the panels, such as shear buckling of the panel and lateral–torsional buckling of the links. In addition, he studied the slit geometry, the slit end condition, and different aspects of the edge stiffeners. He found that the slit width, the plate thickness and the slit geometry do not significantly affect the stress concentrations at the slit ends. He recommended using edge stiffeners that extend over the entire height of the panel. McCloskey also studied methods for reducing the gravity load demands in the panels. He recommended designing the beams of the SSPFs to satisfy a deflection limit of beam span/500, assuming simply supported beams (neglecting any support contribution from the panels) subjected to a uniformly distributed load.

3. Steel slit panel strength and stiffness

This section provides basic information on strength and stiffness of an SSP. Only the most relevant information needed to understand the results of the experimental work has been included here. A more comprehensive discussion of the design of the SSPs and SSPFs may be found in [10,11].

Ductile behavior of SSPs depends on the geometry of the panels, specifically the geometry of the links. Hitaka and Matsui [7] used three parameters to control the geometry of the steel walls with slits and ensure ductile behavior. These parameters and their recommended ranges were based on results from experimental tests for steel walls with slits, and the applicability of these ranges for SSPs was confirmed numerically by McCloskey [9]. The parameters are α, β and b/t. α is the link aspect ratio (l/b). It ensures that the link is long enough that flexure controls the behavior of the links. The recommended range for α is between 2.5 and 5. The second parameter, β, is defined as the fraction of the panel height that is composed of links (ml/h). This parameter ensures that a good portion of the panel is composed of links. The recommended range for β is between 0.65 and 0.85. The third parameter is the aspect ratio of the link's cross section (b/t). This parameter controls premature out-of-plane deformations of the links. The recommended ratio for b/t is between 10 and 15.

The shear strength of an SSP is obtained from the sum of the shear strengths of the links and the edge stiffeners. It is given by

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