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NON-LINEAR BEHAVIOR OF STEEL PLATE SHEAR WALL WITH LARGE RECTANGULAR OPENING

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SUMMARY

Steel plate shear walls have been used more and more in the steel structures to resist earthquake and wind forces. This system offers several advantages as compared to the other usual lateral load resisting systems. Steel saving, speed of erection, reduced foundation cost, and increased usable space in buildings are some apparent advantages of the steel plate shear walls. Steel plate shear walls also provide major stiffness against building drift for the hi-rise buildings. The hysteretic Characteristic, ductility and the energy absorption capacity make this system suitable to be used as seismic resistant element in the steel structures. It is frequently demanded to introduce relatively big opening in the steel plate shear walls. These openings reduce the shear and energy absorption capacity of the plate.

The steel plate shear walls were analyzed by the non-linear finite element program NISA II. More than 50 different models were evaluated by this program. We have studied the effect of some important geometrical parameters such as plate thickness, opening aspect ratio and opening percentage. We have determined the optimum aspect ratio for opening and we have shown that the optimum aspect ratio for opening depends mostly on the plate thickness rather than the percentage of the opening. We have shown also that for a determined opening percentage the thinner plates attain their maximum shear capacity with smaller opening aspect ratio.

INTRODUCTION

In seismic regions, moment resisting frames, reinforced concrete shear walls or steel bracings are usually used as the lateral load resisting systems. However, their strength, ductility, energy absorption capacity and hysteretic characteristic during the moderate and severe seismic events impose different types of problems. In recent years, the steel plate shear walls (SPSW) have been also used in buildings to resist seismic forces. This system is lighter and more ductile than most other systems. They present suitable hysteretic characteristic in plastic zone and good energy absorption capacity. The steel plate shear walls offer other advantages such as increase in the speed of erection and usable space, which make this innovative system competitive to the usual aseismic resistant systems. They also exhibit construction simplicity and fabrication repetition.

The steel plate shear walls consist of thin vertical steel plates welded or bolted to their surrounding columns and beams. These panels can be installed in one or more bays in all the stories of a steel structure. The surrounding frame may be either simple or moment-resisting.

In order to provide an economical design, the thickness of the steel plate is usually reduced. Thin steel plates, having high slenderness ratio and unavoidable out-of-plane imperfections evidently have very low buckling strength. To improve the buckling limit, it is generally suggested that longitudinal and transverse stiffeners stiffen the steel plate seriously. Although using closely spaced stiffeners has significant influence upon the plate behavior and its shear capacity, but it highly increases the weight of the plate.

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Most of the structures that have already been built up with the steel plate shear walls, use relatively thick plates or plates that are heavily stiffened to prevent plate buckling. Recent investigations show that the use of thick steel plate shear walls is neither necessary nor economic for seismic resisting structures. The steel plates are capable of sustaining much higher load because of their post buckling strength. Unlike column buckling, buckling of the plate is not synonymous with failure. Enormous post-buckling strength can be achieved in thin plate which is restrained suitably at its edges and subjected to in-plane shear load.

When an increasing shear load is applied to the steel plate shear wall, equal tensile and compressive stress will be developed within the plate until its buckling limit. Then the plate loses its capacity to carry any additional load. At this stage of post-buckling if the plate is adequately connected to its surrounding frame, a new load carrying mechanism will be developed. An inclined tension field will carry any increase in applied shear load. To maximize the efficiency and the strength / weight ratio of the plate, close attention should be paid to the post-buckling capacity of the plate.

It is sometimes necessary to introduce openings in the steel plate shear walls. This happens frequently when the steel plate shear walls are used as the facade panels. In these cases, the openings should normally be large enough to be used as windows. The introduction of such a large opening could change drastically the stress distribution within the plate and will in most cases influences the collapse mode and behavior of the loaded plate. To compensate the losses and prevent the buckling at very low levels, the plate should be stiffened. The required stiffeners should be placed on all sides of the opening and eventually continued for height and width of the panel.

THEORETICAL BACKGROUND

The valuable investigations have been already carried out to evaluate the behavior of the steel plate shear walls and slender web of plate girders. A good collection of literatures concerning these subjects are available [Elgaaly and Liu, 1997], [Deylami, 1998], [Driver, Kulack, Kennedy and Elwi, 1996] and [Sabouri-Ghomi and Roberts, 1992]. However, these studies are not much valid for steel plate shear walls with large openings. Very little information is available on the behavior and shear capacity of these types of plates [Deylami and Daftari, 1998], and [Roberts and Sabouri-Ghomi, 1992].

The technical and economical advantage of the steel plate shear walls show that the precise information specially concerning the effects of opening features on the elastic and the post-buckling shear capacity of the plate, are of vital importance to designers of these types of structures.

Narayanan and Rockey (1981) had studied the openings in the web of plate girders. They had proposed an approximate method of analysis. A more accurate theoretical method for computing the ultimate shear capacity of plate girders with opening in webs were proposed by Narayanan (1983) The analytical models formerly suggested by Porter, Rockey and Evans (1975), for estimating the ultimate capacity of plate girders were expanded by Narayanan to the web plates with openings.

In our investigation we have also expanded the application of analytical method presented by Porter et al (1975) and Narayanan (1983) to our steel plate shear walls with very large opening. This method is based on the assumption of the equilibrium of the frame in collapse state. The collapse load is considered to be the sum of the following contributions:

- elastic buckling load
- post-buckling (tension field) load
- the load carried by columns
- the load carried by stiffeners (if any)

We have developed a more accurate method for computing the ultimate shear capacity of steel plate shear wall containing rectangular large opening. The ultimate shear capacity of steel plate shear walls were computed as function of different parametric factors, such as plate thickness (t) and aspect ratios (H/B),opening percentages and opening aspect ratios (H0/B0) . The analytical results were compared with the results obtained by the numerical nonlinear finite element method (not presented in this paper). The numerical analyses were carried out by nonlinear finite element program *NISAII* [EMRC, 1992] Good agreement were found between the finite element and theoretical analyses.

MODELING

The details of the steel plate shear walls considered in this study are shown in Figure 1. The computer models consist of a single bay, single-story moment-resisting steel frame with thin steel plate shear wall. For all cases the height and width of the steel plates (or façade panels) were considered about 272 cm., which coincided with the typical effective story height in normal buildings. In order to evaluate the effect of plate thickness on the behavior of the system, a wide range of the plate thickness were considered. These were 3.0 mm, 4.5 mm, 6.0 mm, 7.5 mm, 9.0 mm and 10.5 mm.

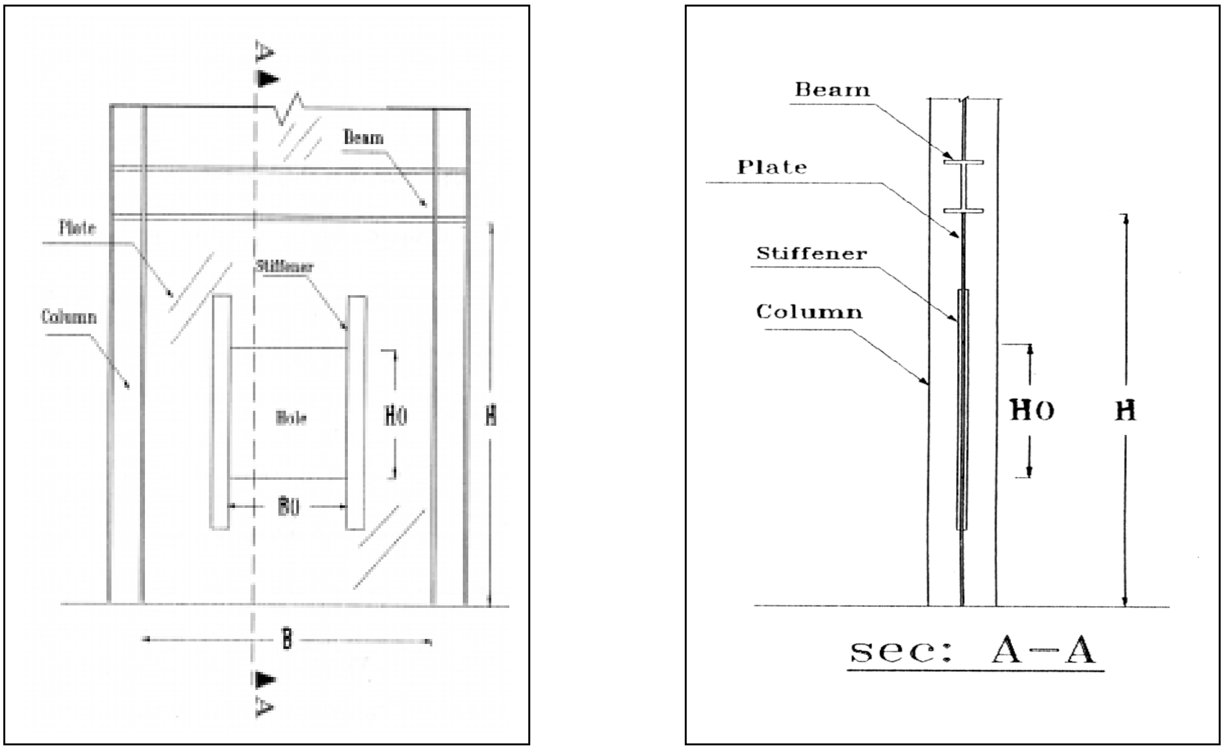

Figure 1: Details of the computed steel plate shear wall

The steel plate was considered to be welded (clamped) to the surrounding beams and columns. IN order to anchor the tension field and getting more general results, the beams were assumed to be very stiff

Nonlinear analyses of steel plate shear wall were carried out by using the general-purpose nonlinear computer program NISA*II (*EMRC, 1992). The steel plates were modeled by three-dimensional general shell elements with 8 nodes (element NKTP=20). Each node of this element has 6 degrees of freedoms (3 degrees of displacement and 3 degrees of rotation) . In general coordinates, the rotation around the vertical axis to the plane of the plate was restricted. In order to obtain more accurate results the beams and columns were also modeled by 3D general shell elements instead of normal space beam elements. This program offers the possibilities of static, dynamic, buckling and nonlinear analysis.

As the goal of this investigation was to determine the shear capacity of the steel plate shear walls, the post-buckling behavior of the panels has been seriously considered. Hence, both geometrical and material non-linearities of the system were taken into account. The materials were modeled as complete elastic-plastic. Von Mises yield surface model was used as the yield criterion. In modeling the geometrical non-linearity the large deflection theory was used

In order to consider the actual edges support conditions of the steel plate shear walls, the deflection of the nodal points of beams in the vertical direction to the panels were restricted. In addition, the connection of the plate elements along the base plate or the lower beam was taken as hinge for out of plane rotation. Regarding the large dimensions of the plate the above restriction had negligible effect.

For simplicity the applied horizontal loads were assumed to be equally distributed to the nodal points of the upper beam elements (Figure 2).

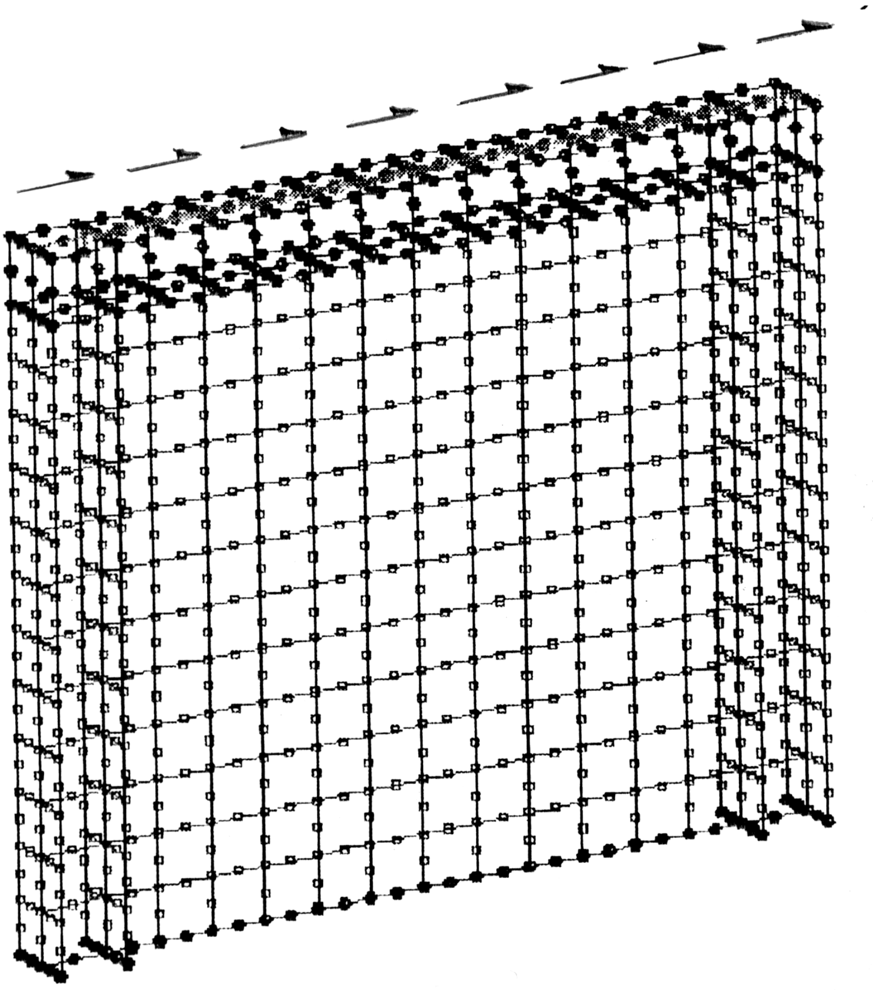

Figure 2: Finite element model and loading

RESULTS AND CONCLUSIONS

The Diagrams of the variation of the shear capacity of the steel panels versus the opening percentages for different opening aspect ratios H0/B0 (H0/B0=0.50, 0.60, 0.75. 0.80 and 1.0) are shown in Figure 3.

It can be noted from these diagrams that, for smaller opening percentage, the reduction of shear capacity depends much more on the plate thickness than the cases of large opening percentage.

The introduction of the opening, even at relatively small percentage, causes important decrease of shear capacity. However, by increasing the opening percentage, the rate of decreasing of shear capacity reduces seriously.

To determine the optimum opening aspect ratio H0/B0, the diagrams of shear capacity of the panels versus opening aspect ratios are shown in Figure 4. It can be depicted from these diagrams that:

The thinner steel plate shear walls attain their maximum shear capacity at lower opening aspect ratio (H0/B0).

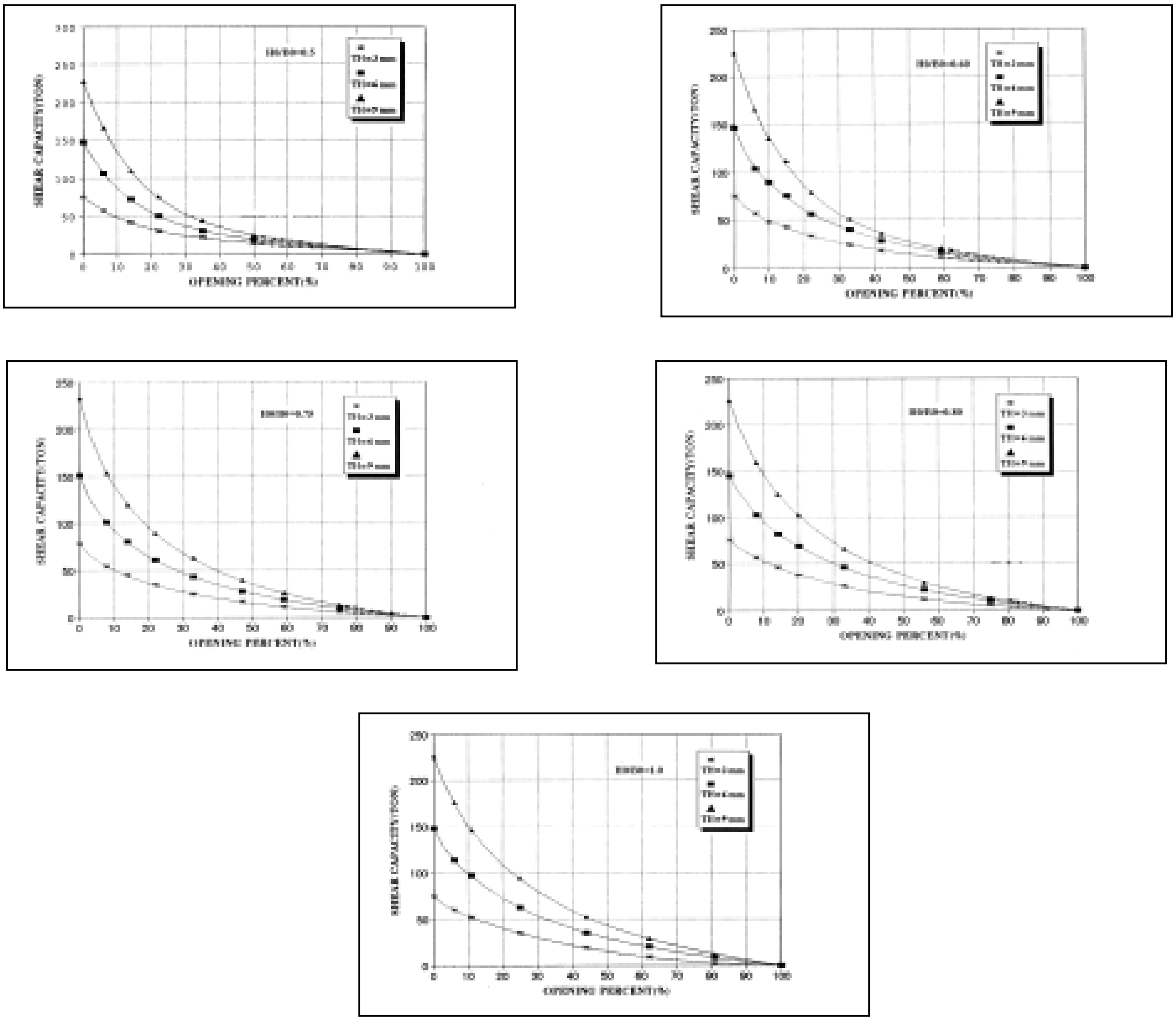

Figure 3: Effect of the opening aspect ratios

The rate of reduction of shear capacity after reaching the maximum level slows down more smoothly for the thicker plates as compared to the thinner ones.

The diagram of the optimum opening aspect ratios versus plate thickness is presented in Figure 5. It shows that the optimum opening aspect ratio is a function of the plate thickness and does not depend on the opening percentage.

For determined plate thickness and opening percentage, the opening aspect ratio has a significant influence on the shear capacity. However, in all cases, to attain the optimum shear capacity, the opening aspect ratio should be greater than 1. In other word, to consider the economy of the structure, it is recommended that the openings (windows) in the case of steel plate shear walls should have the rectangular shape with height significantly longer than the width.

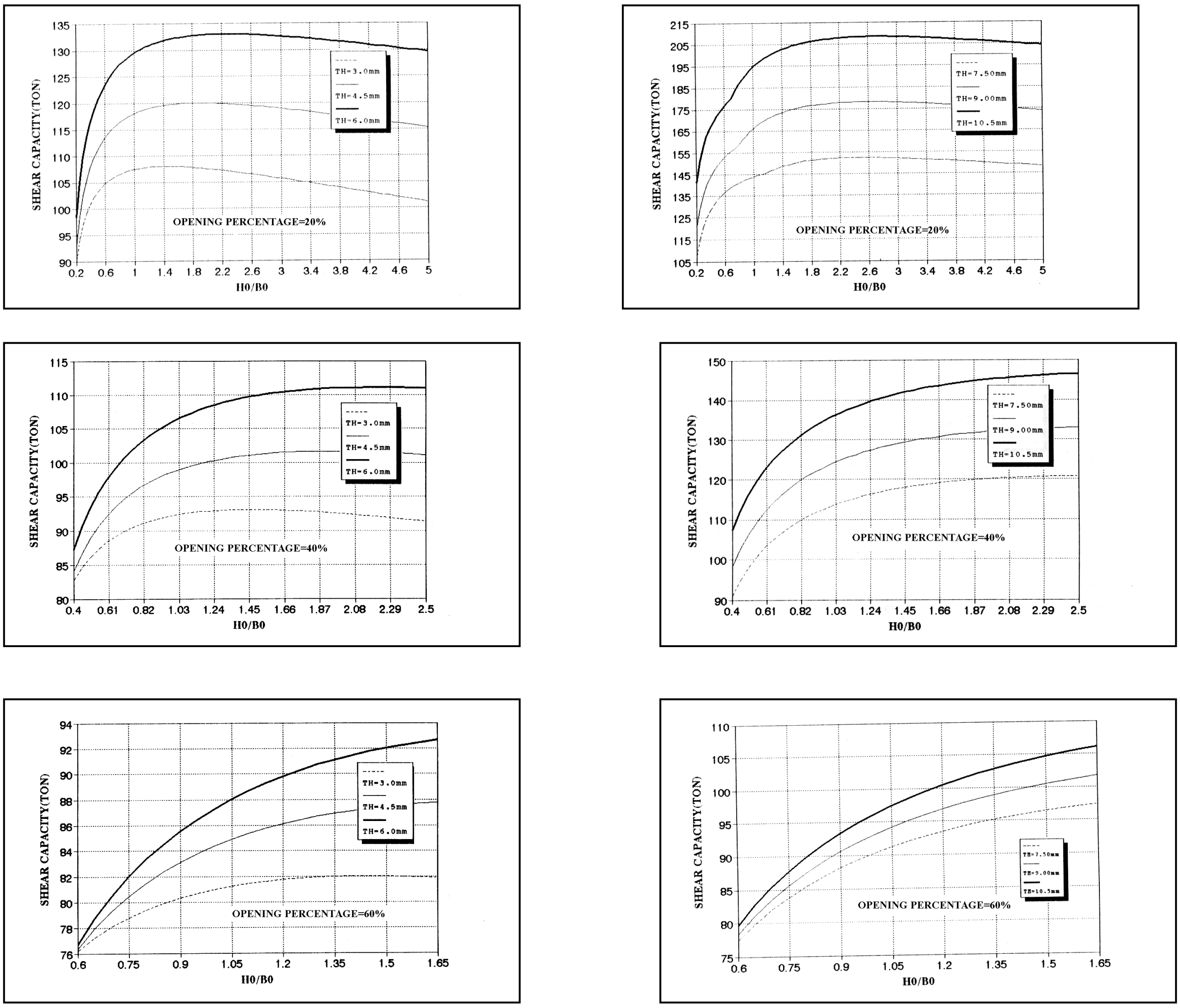

Figure 4: Effect of the opening percentages

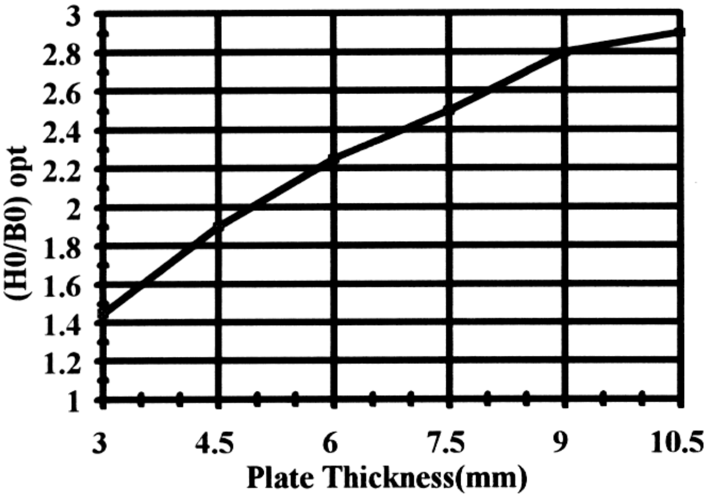

Figure 5: Optimum opening aspect ratio

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