Ultimate shear capacity of perforated steel plate shear walls with multiple circular holes

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This paper presents an analytical study to investigate the effect of providing perforations in the infill plates of Steel Plate Shear Walls (SPSW) on their behavior in shear. The infill plates were considered to be perforated with multiple circular holes that were distributed evenly across the area of the plate. Perforations in the infill plates are used to reduce their shear capacity by a certain degree to allow them to yield in the elastic range of the surrounding boundary elements and hence guarantee a ductile behavior of the wall. That is without the need to use thinner plates that might be practically impossible. A finite element model of a SPSW panel was built and a parametric study was conducted to investigate the influence of each of the parameters governing the shear behavior of the wall. The study results showed that the shear capacity of SPSW can be reduced by the required degree if the infill plates of the wall are perforated with circular holes. In most cases, the reduction in shear strength is not accompanied by a major change in the overall deformation and ductility behavior of the wall.

يقدم هذا البحث دراسة تحليلية لدراسة تأثير وجود ثقوب بالألواح بحوائط القص اللوحية على سلوك هذه الحوائط في القص. حيث تم إعتبار وجود ثقوب دائرية عديدة باللوح موزعة بإنتظام على كامل مساحتة. حيث يتم استخدام مثل هذه الألواح المثقبة لتقليل مقاومة القص بالشكل الذى يضمن حدوث تلدن بها قبل انهيار العناصر المحيطة بها وبالتالي ضمان سلوك مطيل للحائط. هذا دون الحاجة الي استخدام الواح قليلة السمك والتى قد تكون مستحيلة من الناحية العملية. ولقد تم تقديم نموذج تحليلي بطريقة العناصر المحددة وذلك لدراسة تأثير العوامل المختلفة والتى تحكم تصرف هذا النوع من حوائط القص. ولقد بينت الدراسة أن وجود الثقوب بألواح الحائط يؤدى بشكل عام إلي تقليل مقاومة القص القصوي للحائط بالدرجة المطلوبة بدون أن يؤثر ذلك على السلوك الكلى له من حيث التشكل أو المطوليه.

Keywords: Shear walls, Perforated, Buckling, Shear strength, Infill plates

1. Introduction

During the past three decades, Steel Plate Shear Walls (SPSW) have gained a lot of interest as a lateral load resisting system for mid and high rise buildings. A steel plate shear panel is simply a steel infill plate that is surrounded by a column-beam system. This panel is repeated in every story and therefore serves (more or less) like a vertical cantilever plate girder that resists the lateral forces. In this system, the columns work as the top and bottom flanges of the girder, while the story beams function like its transverse stiffeners, and the steel plate works like its web. Of course, there are many differences between SPSW and plate girders [1]. For example, SPSW have very stiff boundary elements (the columns and the beams) while the plate girders flanges are typically plates with a little in-plane bending stiffness. Many studies [2-4] have shown that a carefully designed SPSW has many benefits over traditional lateral load resisting systems such as braced frames. SPSW are higher in strength and stiffness and have better energy absorption capability. Furthermore, SPSW have more stable hysterics characteristics and they are better in lateral force distribution compared with other lateral force systems [4-5]. In comparative studies [2, 3], it was demonstrated that the overall cost of a building can be reduced significantly if reinforced concrete walls are replaced with SPSW. SPSWs are much lighter, which ultimately reduces the loads on columns and foundations and reduces the seismic load. They are also significantly faster to construct and hence the construction duration is reduced. Furthermore, increased versatility and space-savings can be gained

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with the use of SPSW as a result of its smaller cross-sections compared to reinforced concrete shear walls. In cold regions, steel construction with SPSWs is more practical and efficient than concrete construction that is usually subjected to freeze-thaw cycles that can result in durability problems. Another benefit of the SPSW system is that it can be used to retrofit or upgrade existing buildings, which was reported by many researchers [6, 7].

Several experimental and analytical investigations are reported on the behavior SPSW. Thorburn et al. [4] developed an analytical method to describe the shear resistance of a thin unstiffened SPSW. A parametric study was conducted to investigate the effect of panel dimensions, plate thickness, and boundary element stiffness on the behavior of SPSW. Timler and Kulak [8] reported an experimental investigation to verify the analytical method proposed by Thorburn et al. [4]. Based on the results of their investigations a modification to the model proposed by Thorburn et al. [4] was presented. Tromposch and Kulak [9] reported an experimental investigation on a one–storey, two–panel specimen similar to that tested by Timler and Kulak [8] except that the beam-to-column connections were bolted and stiffer columns and thinner infill plates were used. The hysteretic behavior of the specimen was investigated and compared with the analytical model proposed by Thorburn et al. [4]. The benefits of the post-buckling strength and the relatively stable hysteresis characteristics of unstiffened thin steel panels were clearly demonstrated. Roberts and Sabouri-Ghomi [10] conducted cyclic loading tests on small-size slender unstiffened shear plate panels to explore their load-displacement characteristics. Some of these samples were perforated (with a single hole at the center of the infill plate). It was concluded that the wall strength and stiffness decreases linearly as the diameter of the hole increases. Elgaaly et al. [11] developed a “truss” model based on the model developed by Thorburn et al. [4] to describe the shear resistance of unstiffened SPSWs. Another truss model was also employed to study the hysteretic behavior of the SPSW specimens. Driver et al. [12, 13] performed tests on a large-scale four-storey single bay steel plate shear wall specimen. Their test results showed that a properly designed steel plate shear wall system can work as an excellent lateral load-resisting system for seismic loading. Behbahanifard et al. [14] conducted an experimental and numerical investigation on steel plate shear walls and presented a parametric study to identify the non-dimensional parameters governing the behavior of a single panel steel plate shear wall. Using plastic analysis theory and the assumption of discrete strips to represent the infill plate, Berman and Bruneau [15] derived equations to calculate the ultimate strength of single and multi-storey steel plate shear walls with either simple or rigid beam-to-column connections. Kharrazi et al. [16, 17] proposed an analytical model called the Modified Plate-Frame Interaction (M-PFI) model to analyze the shear and bending of ductile steel plate walls.

In early design methods, the limit state of SPSW was governed by the out-of-plane buckling of the infill plates [15]. Therefore, designers had to use heavy stiffeners or very thick plates, which was uneconomical. However, earlier study [18] showed that buckling does not necessarily represent the limit of useful behavior of plates and there is a considerable post-buckling strength in an unstiffened shear panel. The post-buckling tension field action of steel plate shear walls can provide very considerable strength (several times its elastic buckling strength), stiffness, and ductility. In many cases, failure of systems that are designed to prevent plate buckling most likely occurs in the building columns long before the plate develops a fraction of its actual strength. In order to avoid that, thinner plates must be used to develop lower ultimate load capacity and therefore fail before buckling occurs in the boundary columns of the SPSW. However, design equations [19] showed that the thicknesses of the plate that are able to achieve failure in the plate before failure of the boundary columns are in many cases unavailable or impractical, which may lead to use thicker plates than needed for a given design situation. That in turn will increase the sizes of the boundary members, as well as, the foundation demands [17].

In order to solve this problem, there are two ways to achieve failure in relatively thick infill plates before failure of their boundary columns:

1- The use of light-gauge cold-rolled and Low Yield Strength (LYS) steel for the infill panels, which is reported in a number of investigations [6, 20].

2- The placement of a pattern of perforations in the infill plates to reduce the strength and stiffness of the panel by the desired amount [20]. Placing holes or perforations in the SPSW has another advantage of allowing utility access through the wall.

The available data on the behavior of perforated SPSW are very limited and not enough to fully understand their behavior. Therefore, the objective of this paper was to investigate the effect of perforations in SPSW on their ultimate shear capacity where the infill plates were considered to be provided with multiple circular holes. An analytical model was built and a parametric study was conducted to investigate the influence of each of the parameters governing the shear behavior of the wall.

2. Analytical model

2.1. Description of the model

The SPSW system for the current study is shown in fig. 1. The model represents an interior panel of a multi storey SPSW. The panel is bounded by two I-section columns and two rigid elements that represent the story beams. The reason of choosing the rigid elements in lieu of actual beam sections is that the tension fields generated at each two adjacent story beams tend to counteract the double curvatures expected in a beam in a drifting frame. This assumption goes well with the test results reported by Driver et al. [13], Rezai [21], and Lopez et al. [22]. They showed that the strains developed in the top and bottom flanges of the storey beams are relatively small, indicating that the contribution of flexural and axial stiffness of the floor beams to the overall behavior of the shear wall is relatively small and that the shear wall system behaves more as a cantilever wall than a frame.

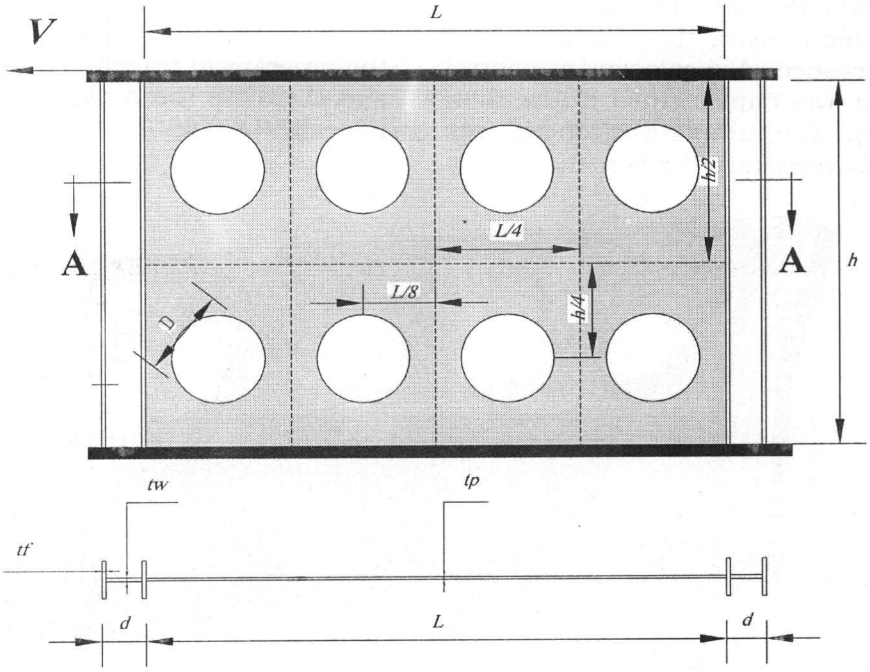

Fig. 1. Sketch of the perforated SPSW panel under investigation.

Assuming a cantilever behavior, the rotational flexibility of the lower floor beam of an isolated panel in a multi-storey building can be neglected with the top floor beam allowed to rotate as a rigid body relative to the lower floor beam. This allows each panel of a multi-story shear wall to be analyzed separately (panel-by-panel analysis) if the effect of over-turning moments from the top stories is considered in the analysis. Again, assuming a cantilever behavior and considering the SPSW system behaves like a vertical plate girder, any overturning moment is carried out by the flanges, i.e., the columns. In this study, no overturning moment was inspected and only pure shear was considered.

The ABAQUS [23] finite element program was used to build the model. Both the infill plates and the boundary members, for all models, were modeled using shell elements. The element used was a general-purpose 4-node doubly curved shell element with reduced integration. This element accounts for finite (large) membrane strains and arbitrary large rotations. The standard steel (52) was used for the boundary elements while the standard steel (37) was used for the infill plates. The material was assumed to be elastic perfectly plastic. The selected models were subjected to loading until the ultimate shear capacity was reached. A displacement control loading scheme was implemented rather than loading control. The ultimate strength was considered to have occurred when the drift of the panel exceeds 2% of the story height [11, 24]. Fig. 2 shows the finite element model built for the case with eight holes.

2.2. Method of analysis

The objective of the current analysis was to investigate the behavior of SPSW under quasi-static pushover loading conditions. Therefore, the model was analyzed at first using the static implicit method (ABAQUS/Standard) [23]. Several nonlinear solution techniques were tried in the analysis that included the Newton-Raphson method, the modified Riks method, and the dissipating energy technique. Although very small loading step values were considered (some were less than 1E-8), the analysis failed to continue. That was attributed to the fact that formulation of the tension fields is always accompanied by a dynamic displacement snapping effect which occurs directly after buckling of infill plate and happens as a result of the load redistribution in the plate [5]. At a certain load, the shear load is switched from being resisted by both tension and compression stresses in the plate to be resisted by only tension stresses which known as the point of "tension field formulation". Therefore, serious convergence problems occur at this loading point that do not allow the program to trace the solution up to failure and therefore the ABAQUS/Explicit was used instead.

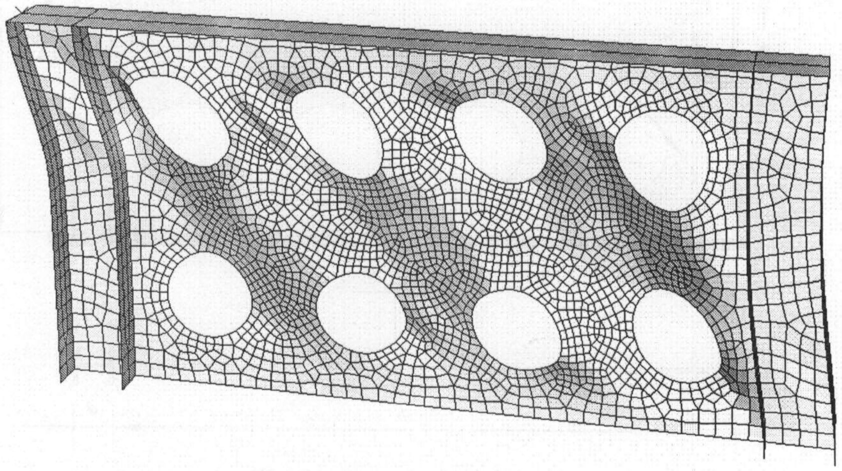

Fig. 2. The finite element model.

The ABAQUS/Explicit [23] is a true dynamic procedure originally developed to model high-speed impact events in which inertia plays a dominant role in the solution. The quasi-static technique can be conducted herein but the loading rate must be controlled so that the effect of the inertial forces can be neglected and therefore, the quasi-static loading condition can be achieved. That was done by first using ABAQUS/Standard module to calculate the natural frequency of the system and the corresponding period of the lowest mode. Accordingly, the loading rate required to obtain the proper static response was defined. A loading duration that is corresponding to ten times the period of the lowest mode was considered to guarantee a quasi-static condition when using the ABAQUS/Explicit module. Another important precaution when using the explicit module is that the loading curve should be smooth enough to prevent any sudden movements that can cause stress waves that might result in noisy or inaccurate solutions. ABAQUS has a simple, built-in smooth step amplitude curve that automatically creates smooth loading curve. The program automatically connects each of data pairs with curves whose first and second derivatives are smooth and whose values are zero at each of data points. Since both of these derivatives are smooth, a displacement loading can be applied with a smooth step amplitude curve using only the initial and final data points, and the intervening motion will be smooth.

The stable time increment chosen by the program during its running is defined by the following equation

$$\Delta t = Le/Cd. \tag{1}$$

Where (Le) is the minimum element length in the model and Cd is the dilatational wave speed of the material. The dilatational wave speed for a linear elastic material with Poisson's ratio equal to zero is given by the equation:

$$Cd = \sqrt{E/\rho}. \tag{2}$$

Where ρ is the material density and E is its modulus of elasticity.

From the above equation, it is clear that the minimum element size is controlling the processing time and therefore the element size was maintained as large as possible yet without sacrificing the accuracy of the solution.

In order to guarantee quasi-static response, the various model energies were monitored in the analysis. According to ABAQUS/explicit module, the energy equilibrium equation is as follows:

$$E_I + E_V + E_{KE} + E_{FD} - E_W = E_{total} = \text{constant}, \tag{3}$$

where E_I is the internal energy (both elastic and plastic strain energy), E_V is the energy absorbed by viscous dissipation, E_{KE} is the kinetic energy, E_{FD} is the energy absorbed by frictional dissipation, E_W is the work of external forces, and E_{total} is the total energy in the system. If a simulation is quasi-static, the work applied by the external forces is nearly equal to the internal energy of the system. The viscously dissipated energy is generally small. Furthermore, the inertial forces are very negligible in a quasi-static analysis and therefore the kinetic energy of the system becomes very small. As a general rule, the kinetic energy of the deforming material should not exceed a small fraction (typically 5% to 10%) of its internal energy throughout most of the process [23]. Therefore, the ratio of kinetic energy to internal energy of the system was monitored during analysis to guarantee the caliber of quasi-static analysis. Moreover, each of the energies was monitored independently to guarantee that there are no oscillations values during the loading. That is because smooth loading should result in smooth energy results [23] otherwise the assumption that the loading is quasi-static might not be accurate.

Displacement controlled loading strategy was implemented in the current analysis. The shear load was obtained from the reactions of the model base. It was preferred herein over a load control strategy because the stiffness of the system usually reduces significantly when the load becomes near its ultimate load. That means that a very small increment of load results in a large displacement. In addition, applying a load in a load control scheme that is larger than the capacity of the shear wall will result in an unstable dynamic solution.

2.3. Geometric and initial imperfections

An initial imperfection in the infill plate was considered in the current analysis. The imperfections were considered to be proportional to the fundamental buckling mode of the SPSW. That was done by first conducting an eigenvalue analysis in which the elastic buckling loads and their corresponding buckling modes were identified. The buckling mode corresponding to the least buckling load, i.e. the fundamental mode, was used to modify the coordinates of all nodes by an amplitude value. In the study done by Behbahanifard et al. [14], it was reported that the initial imperfection magnitude does not have a major effect on the ultimate capacity of the SPSW however it slightly affects the stiffness of the system. It was reported that as long as the imperfection magnitude is less than $0.01\sqrt{L.h}$, the effect is very small and can be neglected, where L and h are the length and the height of the shear wall panel, respectively. Therefore, the maximum amplitude of imperfection was set to a value that was small enough to be less than $0.01\sqrt{L.h}$ yet large enough to initiate buckling.

2.4. Automation of analysis

In order to handle this large amount of analysis runs, each finite element model was first constructed and then a PYTHON [23] script was written to automate the parameter changes and analysis runs.

In each run, the following steps were performed:

1- Modifying the model parameters (dimensions, and perforations).
2- Running an eigenvalue analysis in order to identify the fundamental buckling mode of the model and saving the deformation results to a temporary file.
3- Identifying the suitable loading rate from the natural frequency analysis.
4- Applying the initial imperfection to the model and the loading rate
5- Running the analysis
6- Extracting the results.
7- Checking the kinetic energy value and shape and if it is suitable for quasi-static solution. If not, the analysis is rerun but with lower loading rate in order to maintain a quasi-static behavior. Fig. 3 represents the flowchart of the analysis procedures. In order to handle this repetitive scheme, another PYTHON script was implemented.

2.5. Verification of the model

The model results were compared with the results of Vian and Bruneau [20] for a perforated SPSW specimen. Their tested specimen had a width of four meters and a height of two meters. The infill plate was formed of low yield steel that had a thickness of 2.6 mm. The specimen had a beam-to-column connection detail that included reduced beam sections at each end. The ABAQUS program was used to estimate the shear capacity and the stiffness of the specimen. The previously described quasi-static loading technique was implemented and a displacement-controlled loading strategy was used. Fig. 4 shows the predicted storey shear versus storey drift obtained from the finite element model results and those obtained from the experimental test results reported. It is clear from figure that the analytical results are very comparable with the experimental ones.

3. Parametric study

3.1. Governing parameters

The main parameters governing the behavior and capacity of a SPSW system can be summarized into the following dimensionless parameters. These parameters were proposed by Behbahanifard et al. [14] and they have been used confidently in many following investigations.

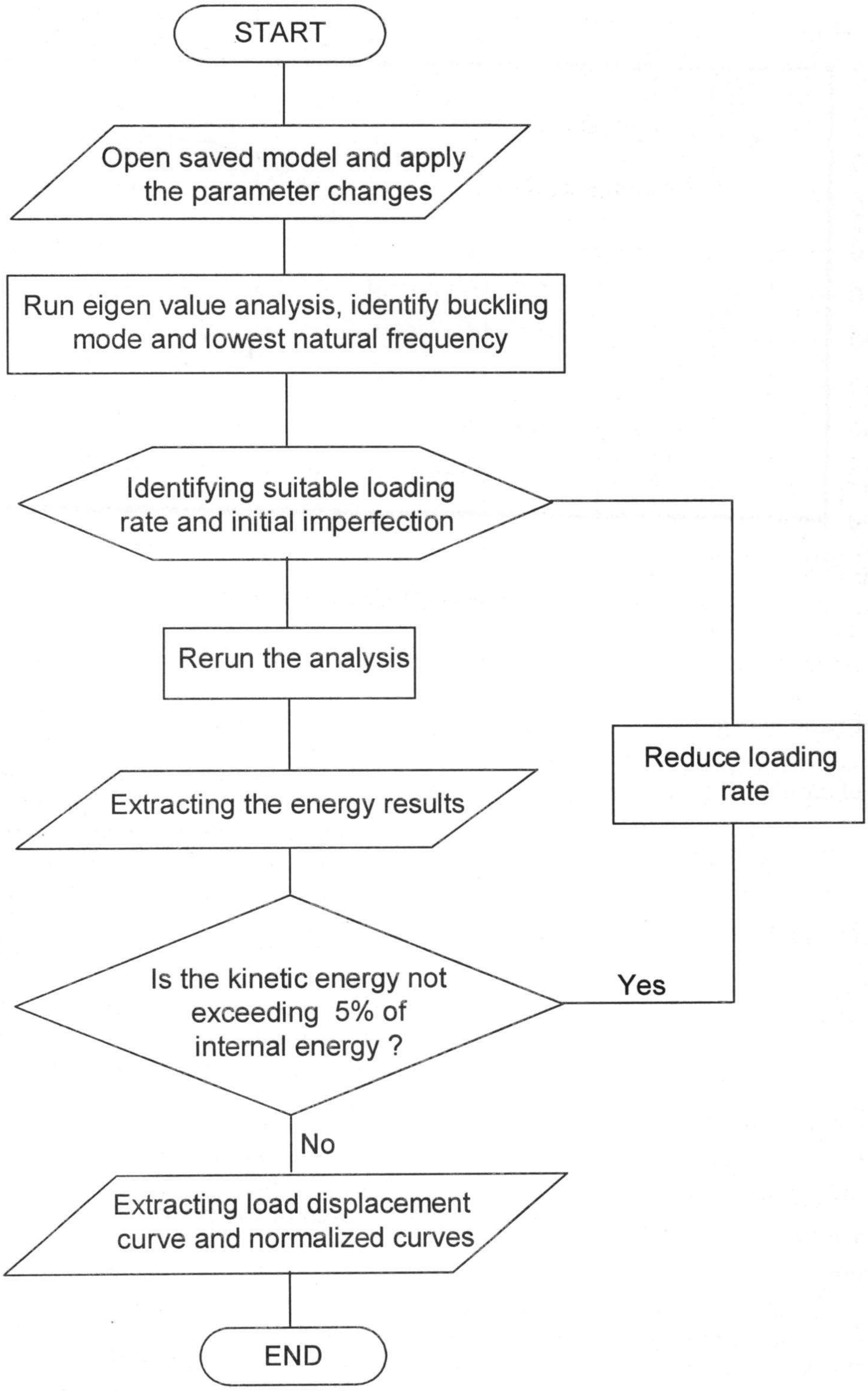

Fig. 3. Flow chart of the procedures of analysis.

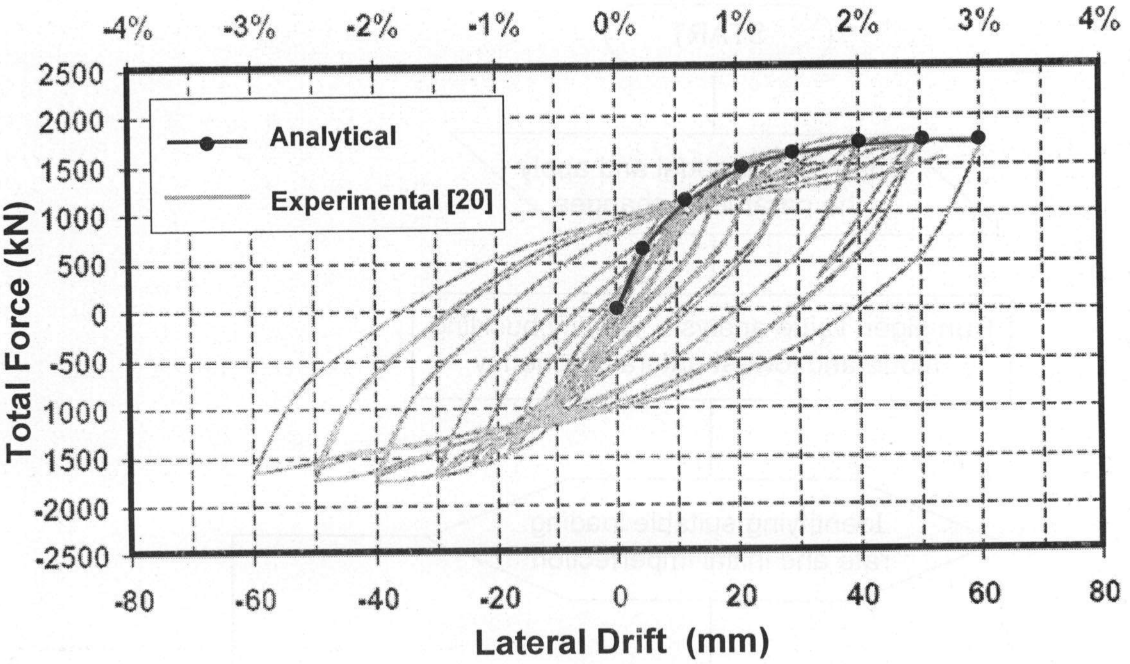

Fig. 4. Verification of the finite element model.

$\beta_1 =$ Aspect ratio $= L/h$. (4)

$\beta_2 =$ Ratio of axial stiffness of infill plate to that of columns $= \dfrac{t_p L}{2A_c}$. (5)

$\beta_3 =$ Column flexibility parameter

$$= 0.7\sqrt[4]{\frac{h^4 t_p}{2LI_c}}\ . \quad (6)$$

where:
L is the panel length
H is the panel height
t_p is the infill plate thickness
A_c is the cross section area of the column
I_c is the moment of inertia of the column

The shear load of the perforated wall (Vperf) was expressed throughout the resulted of this investigation as the ratio of the shear capacity of the same wall but with solid infill plates (Vsolid).

3.2. Details of the parametric study

Perforations in the SPSW were done by considering circular holes in the infill plate. Circular holes have the advantage of minimizing the potential for stress concentrations or fracture failures of the infill plates, especially that the infill plates considered in the study were unstiffened light gage ones. In order to capture the effect of perforations on the infill plate contribution, a frame-only panel were studied and compared with the same models with the presence of infill plates.

The following parameters were introduced to describe the perforations in the infill plate:
$\beta 11$ = perforation ratio = total area of holes /area of the plate
N = Number of holes

The study was conducted for SPSW with different number of holes (N) and for multiple values of the perforation ratio ($\beta 11$). The holes were distributed uniformly over the area of the plate. The infill plate was divided into 1, 4, 6 or 8 areas and each hole was positioned at the centre of each area. The study was conducted for several cases of the parameters of the problem.

4. Model results and discussion

4.1. Perforation ratio and hole numbers

Fig. 5 shows the load-drift relations for different values of the perforation ratio ($\beta 11$). The figure demonstrates that the overall deformation behaviors of the perforated shear walls are similar for the different cases of the ($\beta 11$) but with different initial stiffness and capacity. It is evident from fig. 6 that the shear capacity of the wall generally decreases with the increase of the perforation ratio. However, it is obvious from fig. 7 that for the same value of ($\beta 11$), the number of holes has no considerable effect on the shear capacity of the wall. Nevertheless, the case with the single hole may exhibit slightly different shear capacities than those with multiple ones, especially when the perforation ratio ($\beta 11$) is relatively high. Comparing the shear capacity values in fig. 6 with that of the frame only, i.e., the beams and columns only (shown as a line in figure), it can be concluded that even after cutting 40% of the plate area, the plate contribution to the ultimate shear capacity of the system is still significant. The conclusion herein is that perforated shear walls have similar overall behaviors to that with solid plates but with lower values of shear capacity and stiffness. The number of holes (excluding the single hole case) has no major effect on the shear capacity of the wall.

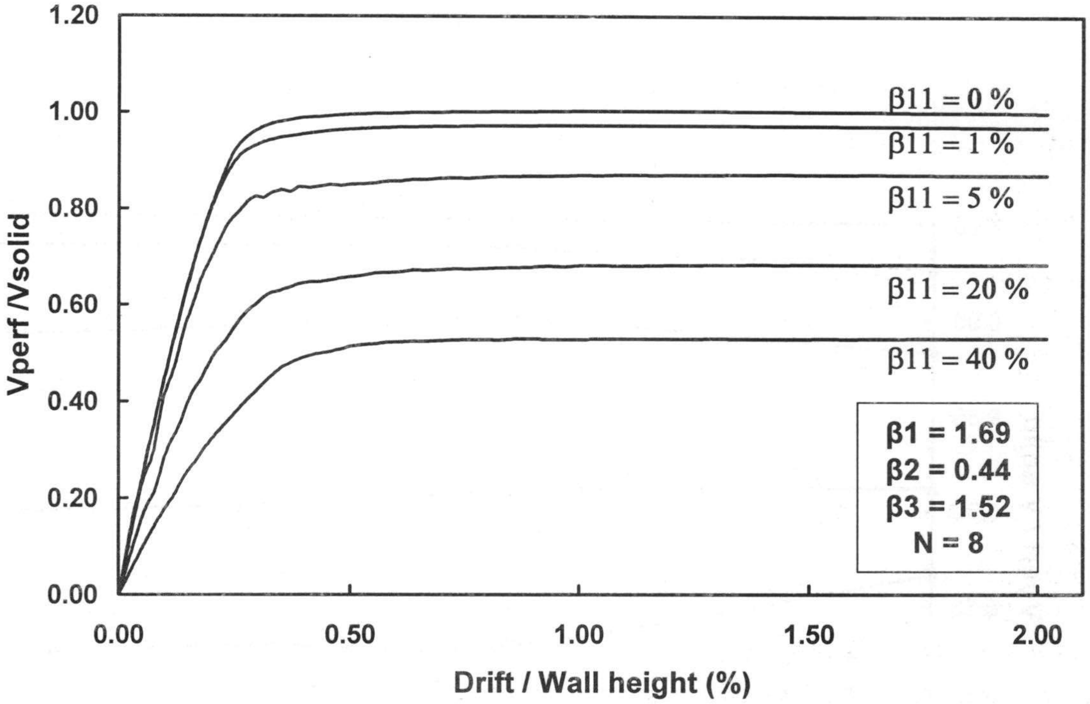

Fig. 5. Load-drift relations for different values of perforation ratio $\beta 11$.

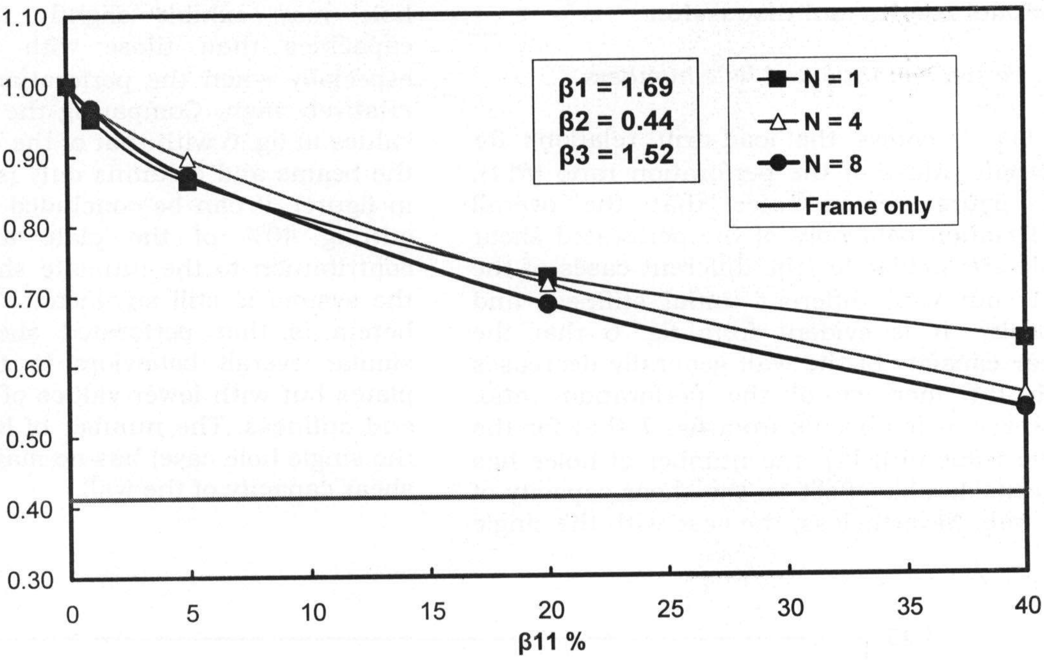

Fig. 6. Effect of perforation ratio $\beta 11$ on the shear capacity of wall.

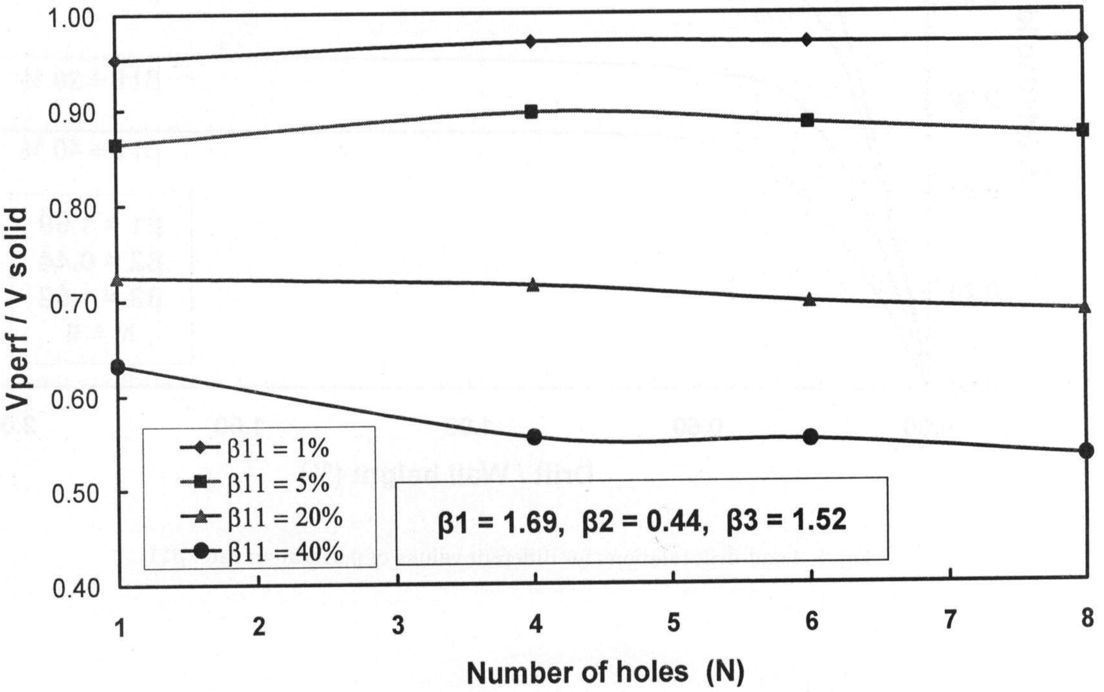

Fig. 7. Effect of hole numbers on the shear capacity of the wall for different cases of $\beta 11$.

4.2. Effect of aspect ratio

In order to investigate the effect of the aspect ratio ($\beta1$), different models having different aspect ratios and different hole numbers (N) were analyzed. Fig. 8 shows the load-drift relations for different cases of the aspect ratio ($\beta1$). It is clear from figure that the walls with higher aspect ratios tend to have considerably lower shear stiffness and capacity. That is also clear in fig. 9 where the relationship between the aspect ratio $\beta1$ and the shear capacity of the wall indicates that the shear capacity of the wall generally decreases considerably with the increase of the aspect ratio. Fig. 10 demonstrates that the effect of the number of holes on the shear capacity is not considerable and it generally depends on the values of the aspect ratio.

4.3. Axial stiffness ratio β2

The effect of the axial stiffness ratio ($\beta2$) on the shear capacity of perforated walls is discussed through the results shown in fig. 11 through fig. 13. The load-drift curves of the three cases of $\beta2$ shown in fig. 11 indicate that the effect of $\beta2$ is not significant for perforated steel plate shear wall. A slight decrease in the shear capacity of the wall is experienced with the increase of $\beta2$. Plates with higher number of holes generally provided lower shear capacities. It is clear from fig. 14 that the effect of $\beta2$ becomes more significant with the plates with the higher perforation ratios.

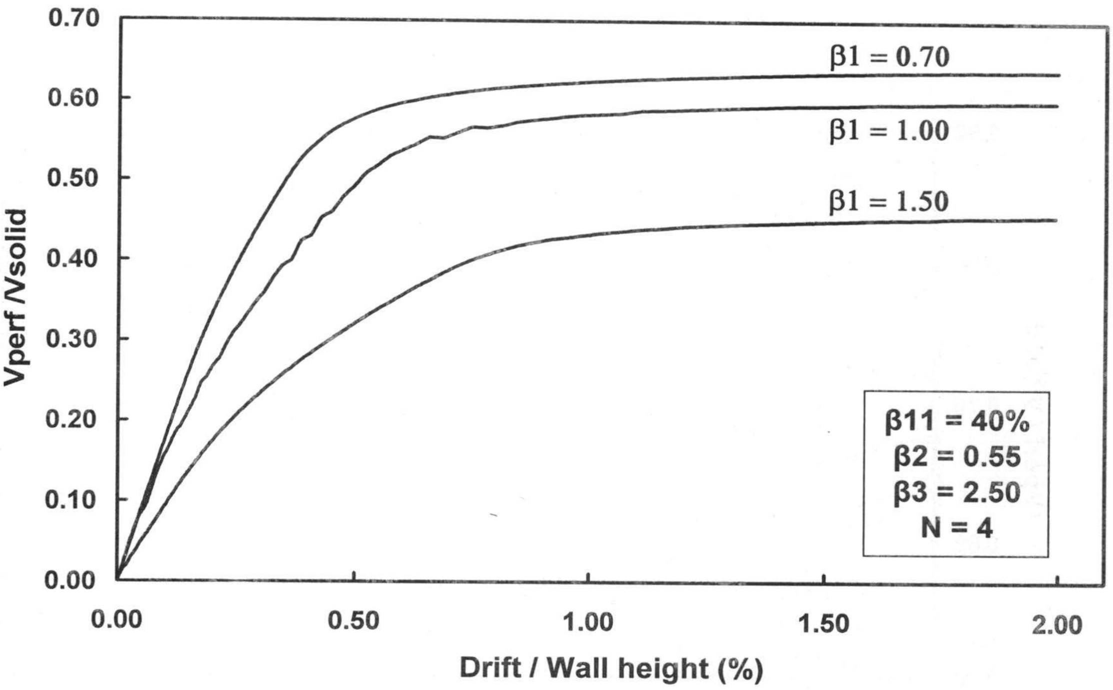

Fig. 8. Load-drift relations for different values of $\beta1$.

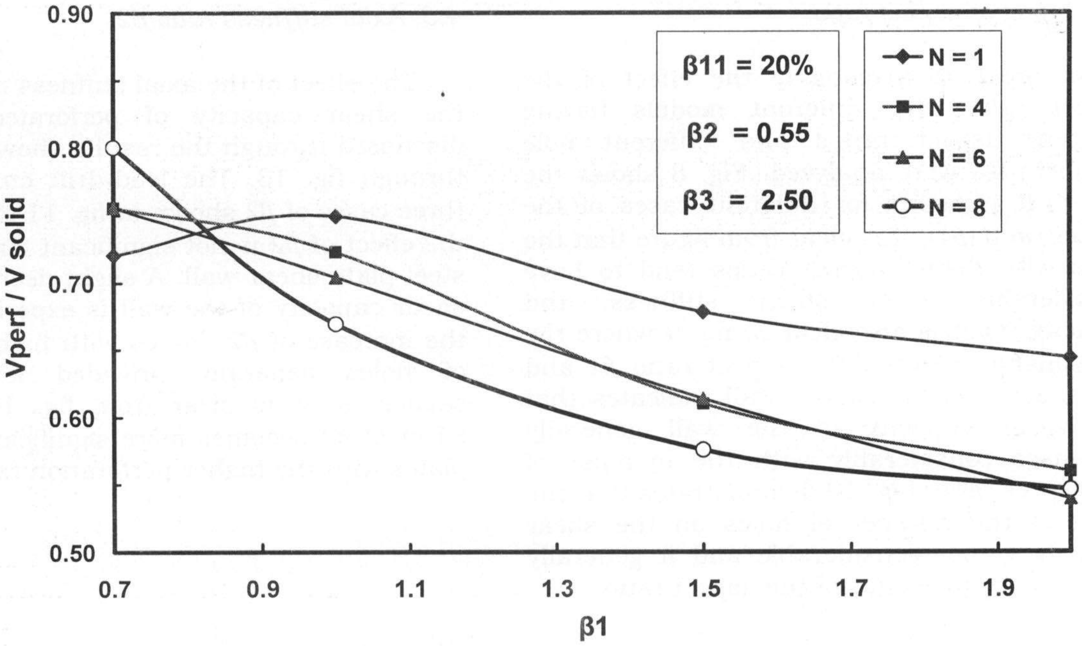

Fig. 9. Effect of aspect ratio on the shear capacity of the wall.

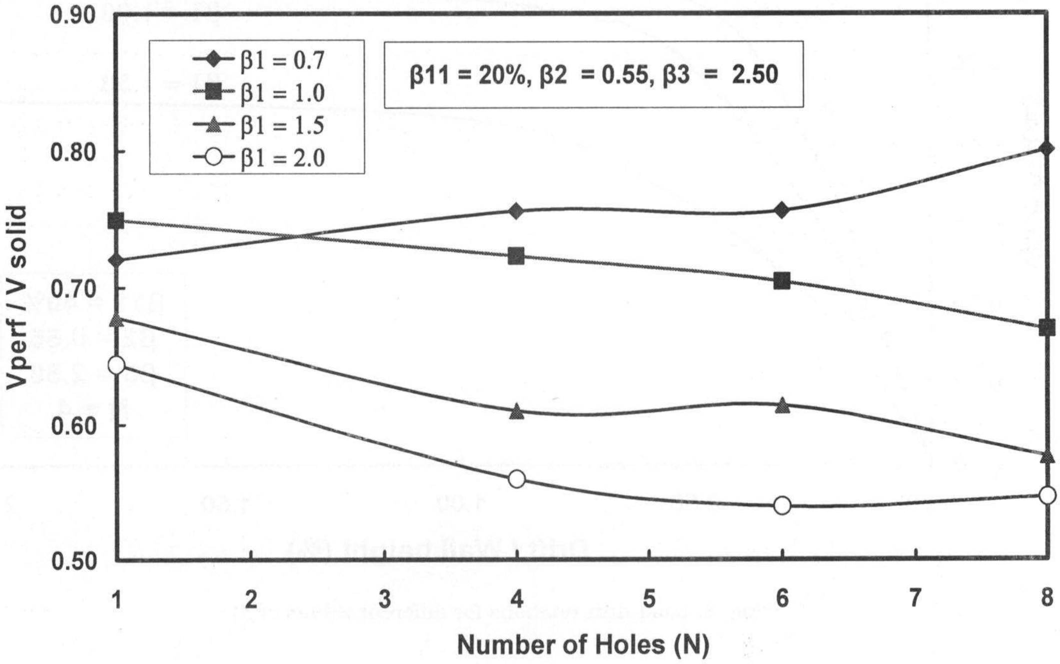

Fig. 10. Effect of hole numbers on the shear capacity of the wall for different cases of $\beta 1$.

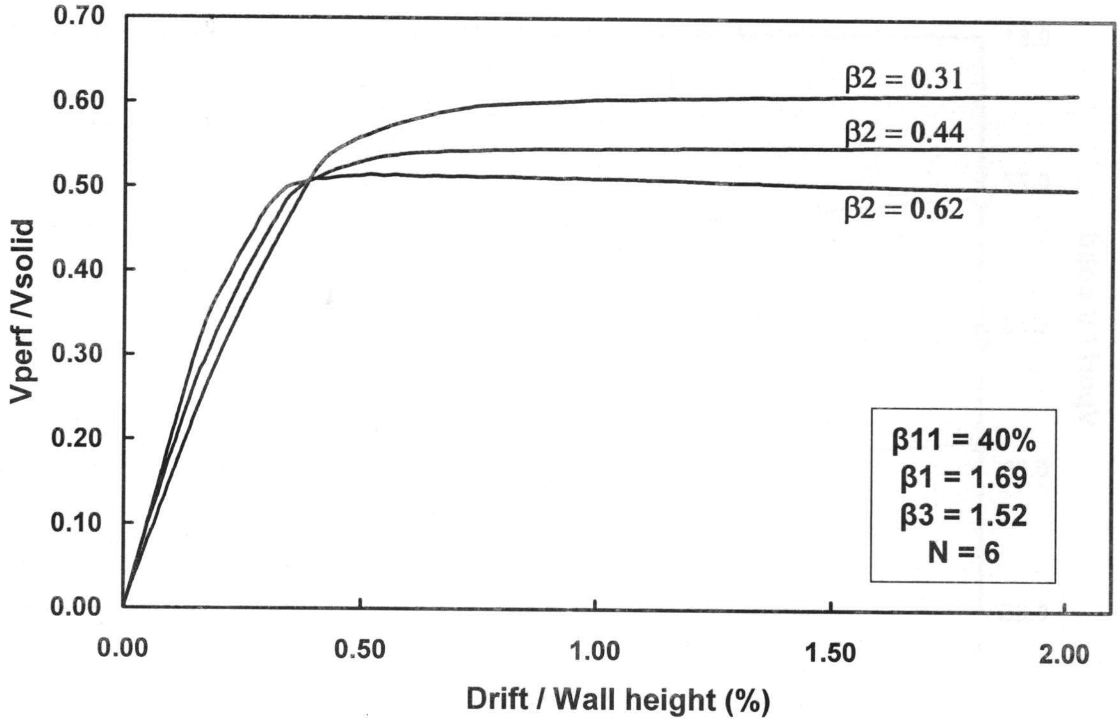

Fig. 11. Load-drift relations for different values of $\beta 2$.

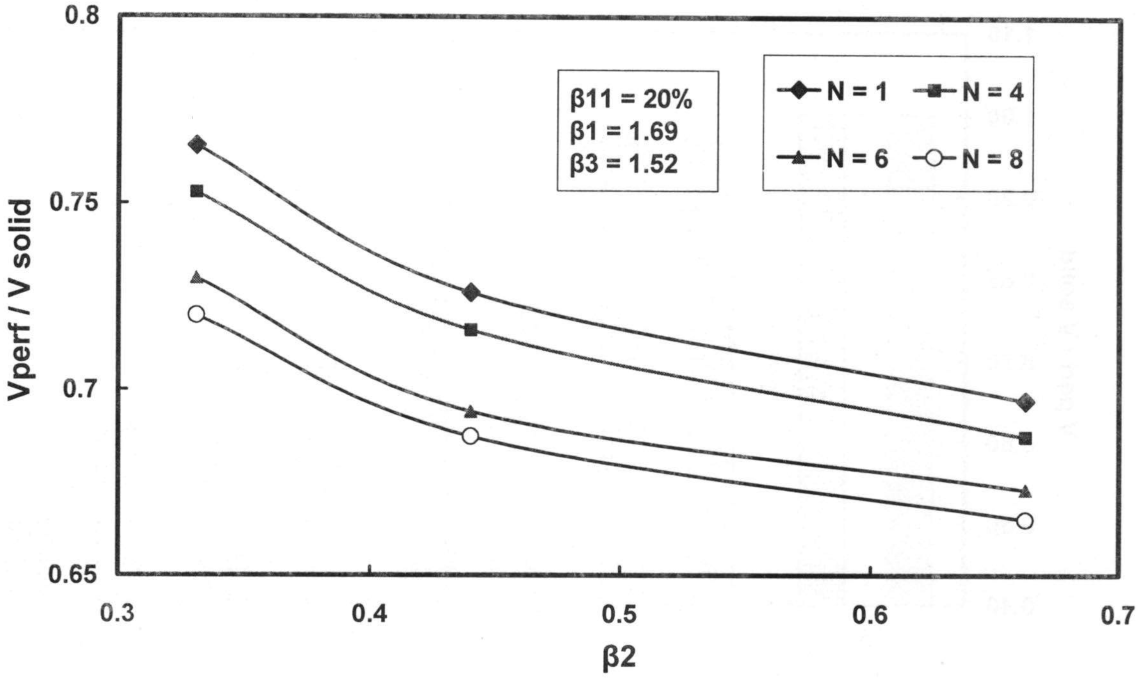

Fig. 12. Effect of axial stiffness $\beta 2$ on the shear capacity of the wall.

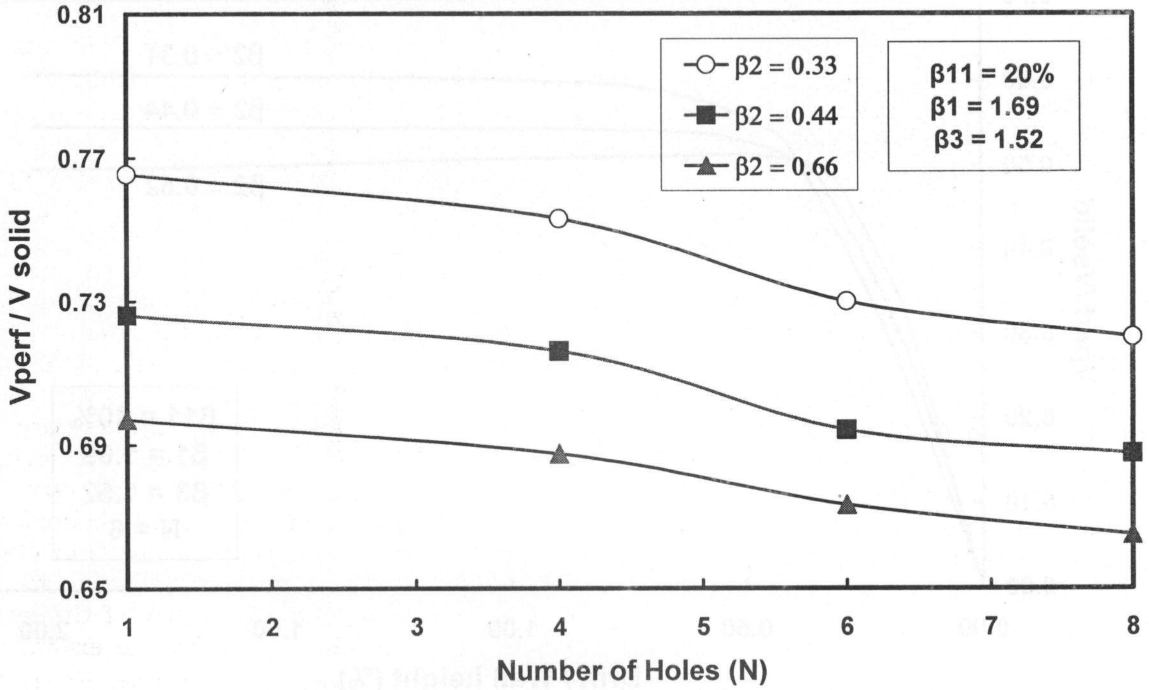

Fig. 13. Effect of hole numbers on the shear capacity of the wall for different cases of $\beta 2$.

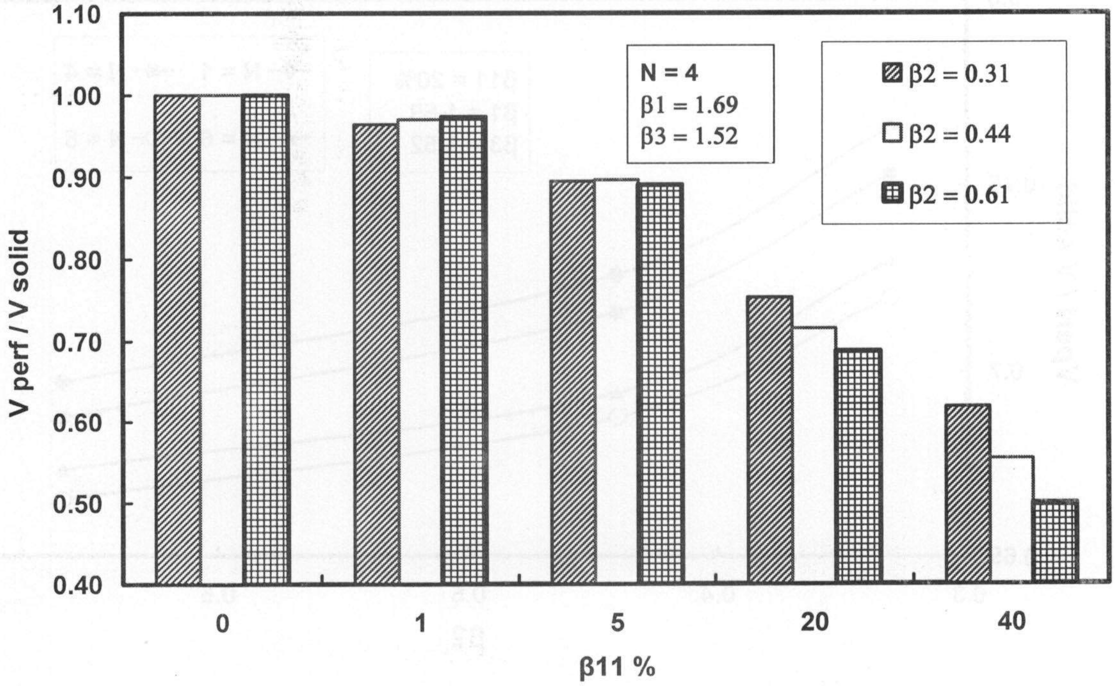

Fig. 14. Effect of $\beta 11$ and $\beta 2$ on the shear capacity of the wall.

4.4. Column flexibility parameter $\beta3$

The importance of this parameter arises from the fact that systems with boundary elements with enough rigidity experience a relatively uniform tension field formation. On the other hand, flexible boundary elements may fail to generate those fields and hence reduces the benefits of the web plate (infill plate). For this reason, some specifications limit column flexibility parameter to 2.5 [25]. Fig. 15 through fig. 18 show the effect of the column flexibility parameter ($\beta3$). It is clear from fig. 15 that $\beta3$ has a significant effect on the behavior of the perforated wall. That is in terms of the shear capacity and deformation behavior. The figure indicated that walls with higher $\beta3$ tend to exhibit significantly lower initial shear stiffness. Furthermore, the greater the value of $\beta3$ the lower the value of the ultimate shear capacity of the wall becomes. Fig. 16 demonstrates that the shear capacity of the wall generally decreases with the increase of $\beta3$. The number of holes has no significant effect on the shear capacity of the wall for any value of $\beta3$, which is clear in fig. 17. It can be concluded from fig. 18 that effect of $\beta3$ becomes more significant in cases with higher values of perforations ratio.

4.5. Overall reduction in shear capacity of the plate

From the results of the current study, it is clear that providing perforations in the shear wall results in a significant reduction in the shear capacity of the wall. Therefore, providing these perforations may be a way to reduce their shear capacity by the required degree that allows the infill plates to yield before failure of the surrounding boundary elements and hence allow for ductile behavior of the wall without the need to use thin plates that might be practically impossible. In most cases, the overall ductile behavior of the shear wall panel was not affected by the existence of the perforations. Walls with higher $\beta3$ are however prone to exhibit loss in stiffness when the shear wall plate is provided with perforations.

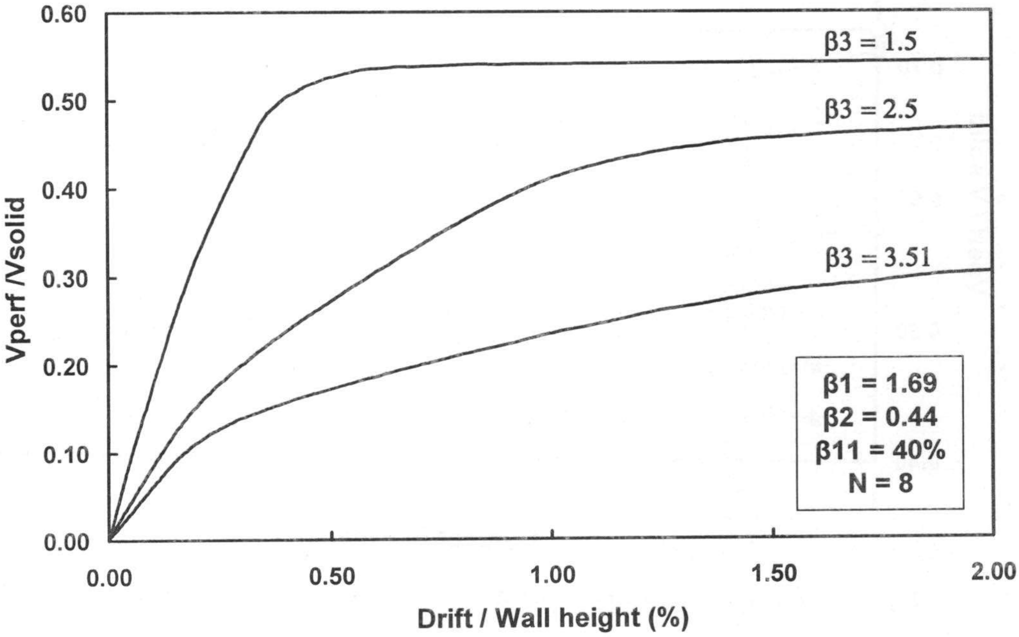

Fig. 15. Load-drift relations for different values of $\beta3$.

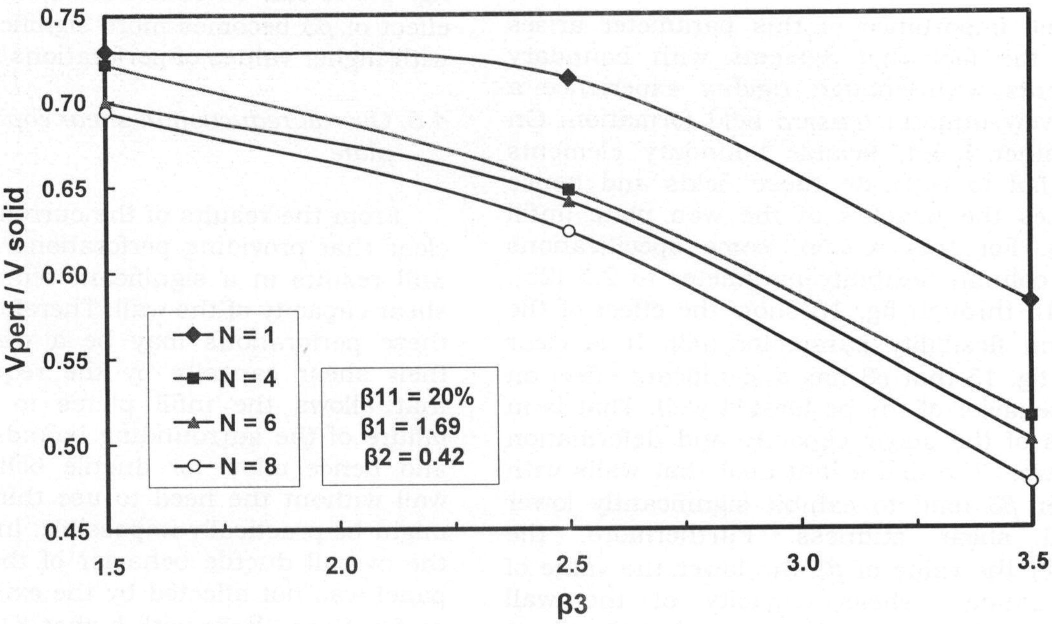

Fig. 16. Effect of $\beta3$ on the shear capacity of the wall.

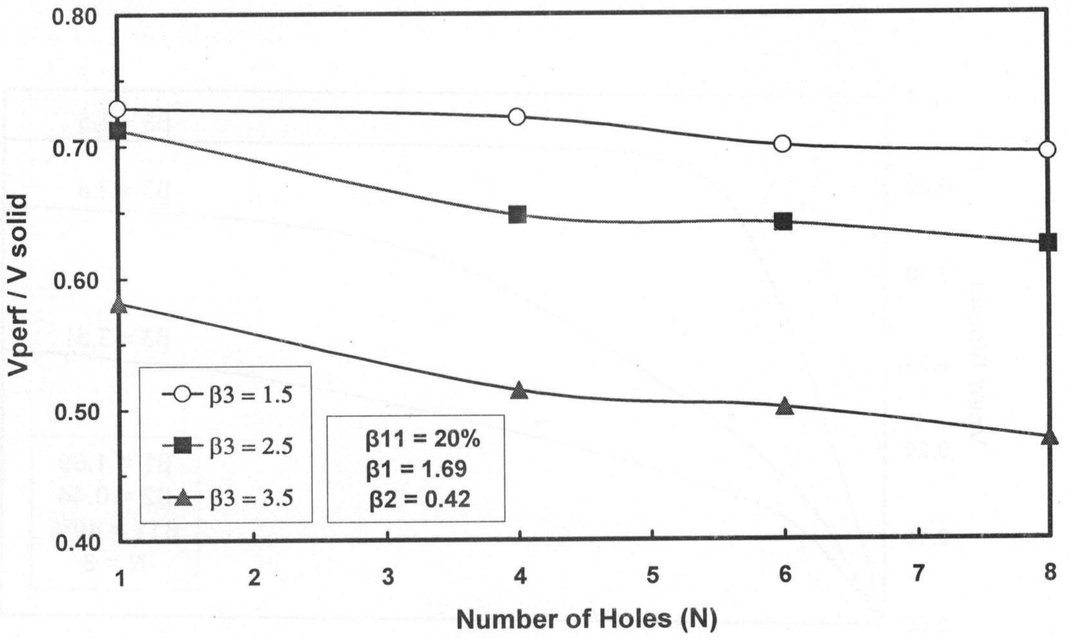

Fig. 17. Effect of hole numbers on the shear capacity of the wall for different cases of $\beta3$.

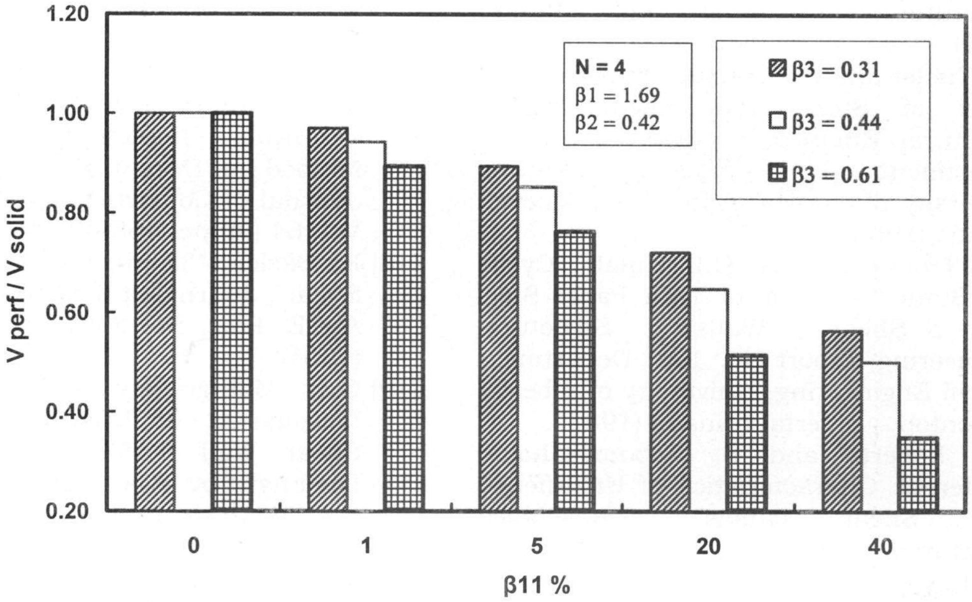

Fig. 18. Effect of $\beta 11$ and $\beta 3$ on the shear capacity of the wall.

5. Conclusions

The following can be concluded from the results of the current study:

1- The required reduction in the shear capacity of SPSW can be gained if the infill plates are perforated with circular holes.

2- In most cases, the reduction in shear strength is not accompanied by a major change in the overall deformation behavior of the wall.

3- The reduction in shear strength of the wall increases with the increase of aspect ratio ($\beta 1$), axial stiffness ratio ($\beta 2$), and column flexibility parameters ($\beta 3$).

4- For the same percentage of perforations, the number of holes has no major effect on the shear capacity of the wall. However, walls with a single hole exhibit slightly higher shear capacities than those with multiple ones.

5- Walls with higher column flexibility parameters ($\beta 3$) may exhibit different overall deformation behavior with reduced stiffness.

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