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NON-LINEAR SEISMIC RESPONSE OF REINFORCED CONCRETE SLIT SHEAR WALLS

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The reinforced concrete slit shear wall system has been introduced recently as a new breed of earthquake resistant structures. Some theoretical and experimental investigations have been conducted to study the behaviour of isolated connecting beams and slit shear wall models under static load, but little work has been carried out to investigate the dynamic behaviour of the structural system. In this paper, the non-linear seismic response of slit shear walls under earthquake excitation is analyzed. Based on a simplified structural model, which is shown to have sufficient accuracy for slit shear wall structures, the influence of the elasto-plastic behaviour of the connecting beams on the dynamic response of the slit shear wall structure is evaluated. The results reveal that yielding of the connecting beams can significantly reduce the deflection response of the slit shear wall structure and the seismic loading induced on it. Moreover, there appears to be an optimum yield strength value for the connecting beams that would lead to the best overall seismic performance of the slit shear wall system.

1. INTRODUCTION

1.1. THE SLIT SHEAR WALL SYSTEM

The slit shear wall system has been introduced recently as a new breed of earthquake resistant structures [1–5]. A slit shear wall is a shear wall with purposely built-in vertical slits which divide the shear wall into two or more narrower sub-wall units interconnected together by shear connections located along their vertical dividing lines (see Figure 1). It is designed in such a way that under normal wind load conditions, the shear connections would remain elastic so that the slit shear wall behaves like a solid wall as if there are no slits but when overloaded due to earthquake attack, the shear connections would yield thereby decreasing the lateral stiffness and increasing the damping capacity of the structural system. It is hoped that after the shear connections have yielded, the structural system would be de-tuned to have its fundamental frequency falling outside the frequency spectrum of the seismic excitation and the excessive vibration energy could be dissipated to avoid overall collapse. Although the shear connections would be damaged after yielding, their sacrifice would help to protect the wall itself,

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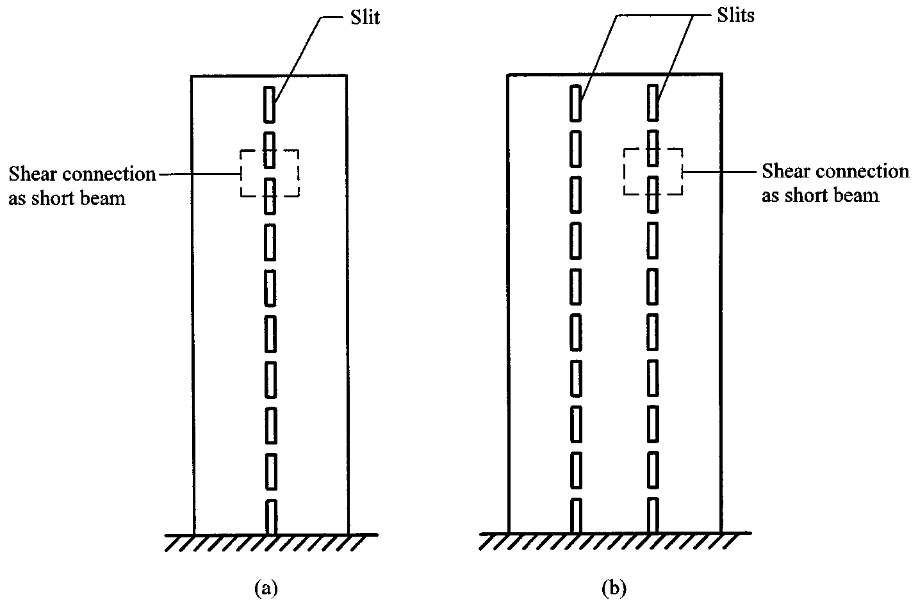

Figure 1. Proposed slit shear wall system. (a) With one band of slits; (b) with two bond of slits.

which is a lot more difficult to repair, from being damaged. Thus, the shear connections formed by the introduction of vertical slits function both as a "structural fuse" and a 'structural damper".

1.2. SLIT SHEAR WALLS AS LIMITING CASE OF COUPLED SHEAR WALLS

It is natural to consider slit shear walls as close relatives of coupled shear wall structures. As a matter of fact, a slit shear wall may be treated as an extreme case of a coupled shear wall structure with very short connecting beams. The elasto-plastic behaviour of coupled shear walls has been studied quite extensively by both theoretical analysis [6–9] and experimental investigations [10–13]. It has been found from these studies that in most coupled wall structures, plastic hinges are formed on the beams before the walls fail and that such plastification can substantially increase the ductility of the structures. Within certain limits, the earlier the beams start to yield, the greater will be the increase in ductility. However, if the beams yield prematurely, the lateral strength of the wall structures might be severely impaired and the ductility of the beams might become exhausted when the walls start yielding. Thus for best overall performance, the beams should yield well before the walls do but not at so early a stage as to cause excessive reduction in lateral strength or breakage of the beams before the walls fail. In other words, both "over-coupling" which causes the beams to remain unyielded even when the walls fail and "under-coupling" which causes the beams to yield prematurely should be avoided [12].

1.3. DESIGN OF SLIT SHEAR WALLS

It is expected that the same trend of behaviour would also apply to slit shear walls. By designing the connecting beams of the slit shear wall to yield before the wall panel fails, the slit shear wall should have a significantly higher ductility than the original solid wall without slits. In fact, the whole idea of the slit shear wall system is to convert the otherwise solid shear wall, which to some extent is an over-coupled wall structure, to a slit shear wall by the introduction of vertical slits so as to reduce the degree of over-coupling and increase the ductility of the wall structure.

There is, however, the question of when the connecting beams should start to yield. From the standpoint of earthquake resistance, the connecting beams should be designed to dissipate as much energy as possible before the structure fails. As shown in Figure 2, the amount of energy dissipated per cycle is equal to the area within the hysteresis loop of the load–deflection curve which is proportional to the product of the yield load and the post-yield deflection. If the connecting beams have relatively high yield strength such that they yield just shortly before the wall fails, the amount of post-yield deflection of the beams will be relatively small and as a result, little energy can be dissipated, Figure 2(a). On the other hand, if the connecting beams yield at an early stage, the yield load will be relatively small, Figure 2(b). Although the post-yield deflection of the beams can be quite large before the wall starts to yield, the energy dissipation through them will still be relatively small. Evidently, there is a certain intermediate value of beam yield strength that would lead to maximum energy dissipation capacity, Figure 2(c).

It should, nevertheless, be borne in mind that whilst yielding of the connecting beams would increase ductility and damping, the lateral stiffness and strength of the structure would at the same time be decreased. Therefore, it is not a straightforward matter to say whether the introduction of slits would improve the seismic performance of the wall structure. Detailed dynamic analysis is needed before any definite conclusion on the benefit of introducing vertical slits can be drawn.

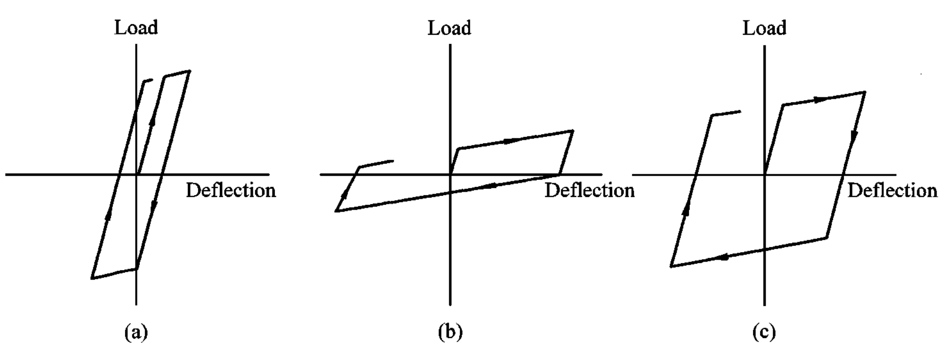

Figure 2. Effects of beam strength on hysteretic behaviour of a slit shear wall structure. (a) Beams have high yield strength; (b) beams have low yield strength; (c) beams have appropriate yield earthquake.

1.4. PREVIOUS STUDIES

Both monotonic and cyclic shear tests of isolated connecting beams of slit shear walls have been carried out [2, 3] and it was found that the short connecting beams would fail only in shear with diagonal compression struts formed inside and all longitudinal reinforcement bars in tension. Some large-scale model tests of reinforced concrete slit shear walls have also been conducted [4]. Two failure modes have been identified. In slit shear walls with weak connecting beams, the connecting beams would yield before the walls fail. However, when the connecting beams are strong, the connecting beams would not yield; the slit shear walls would fail like solid walls without yielding of the connecting beams.

Regarding theoretical studies, the elastic behaviour of multi-storey slit shear walls has been anslyzed by both the continuous connection method and the finite element method [1]. The numerical results indicate that the shear deformation of the short connecting beams and the local deformation at the beam-wall joints have significant effects on the behaviour of the overall structure and thus should be properly allowed for. Using the softened truss model to simulate the inelastic behaviour of the connecting beams and the non-linear finite element method of analysis, the elasto-plastic behaviour of slit shear wall structures has also been studied in detail [5]. The study verified the finding that the ductility of a shear wall structure could be substantially increased by the introduction of vertical slits but this would also cause reduction in lateral strength. By carefully adjusting the depth and reinforcement ratio of the connecting beams, it is possible to reach an optimum design with up to several hundred per cent increase in ductility and less than 30% reduction in strength. All these theoretical studies are on the static behaviour of slit shear walls. So far, little work has been done on the dynamic behaviour of slit shear walls.

1.5. PRESENT STUDY

The present study is a continuation of the above research aiming to develop the slit shear wall system as a new breed of earthquake resistant structures. In this paper, the dynamic behaviour of slit shear walls under seismic excitation is studied using a simplified structural model with the elasto-plastic behaviour of the connecting beams taken into account. Based on a parametric study of the dynamic response of slit shear walls with different connecting beam details, the structural control characteristics of slit shear walls are evaluated and the effectiveness of the structural concept appraised.

2. METHOD OF ANALYSIS

2.1. STRUCTURAL MODELLING

The wide-column frame analogy is used to model the slit shear wall structure. Each sub-wall unit of the slit shear wall is modelled by a column residing at the centroidal axis of the sub-wall unit while the connecting beams are modelled by

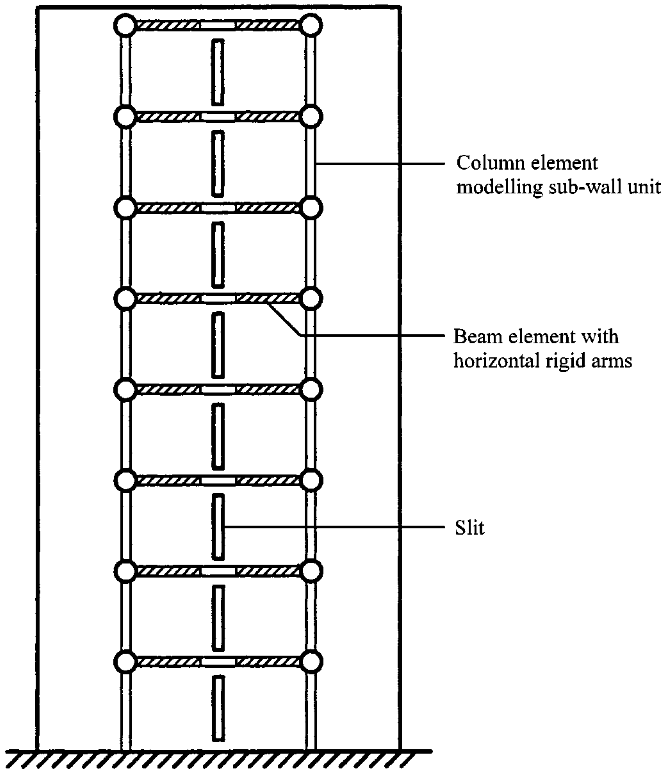

Figure 3. Structural modelling of the slit shear wall structure.

beams with horizontal rigid arms, as illustrated in Figure 3. A standard frame analysis program is modified to analyze this frame model of the slit shear wall structure. To allow for local deformation of the beam-wall joints, the flexible portions of the connecting beams are extended by half the beam depth at each end into the wall. Shear deformation of the connecting beams is taken into account in the derivation of the beam stiffness matrix.

The non-linear inelastic behaviour of the connecting beams is considered in the analysis. However, since the wall units are expected to remain undamaged after the earthquake attack, the column elements which model the wall units are assumed to remain linearly elastic throughout the loading process. In this way, the structural concept of the slit shear wall system is incorporated in the analysis and the effect of yielding of the connecting beams allowed for.

In order to allow for the non-linear inelastic behaviour of the connecting beams, the monotonic load–deflection curves of the connecting beams are first evaluated using the softened truss model theory, as detailed in reference [5]. For monotonic load analysis, the stiffness values of the connecting beams are adjusted in each loading step according to the monotonic load–deflection curves of the beams obtained by the softened truss model. For cyclic load analysis, a degrading, hysteretic and pinched load–deflection curve is constructed for each beam from its

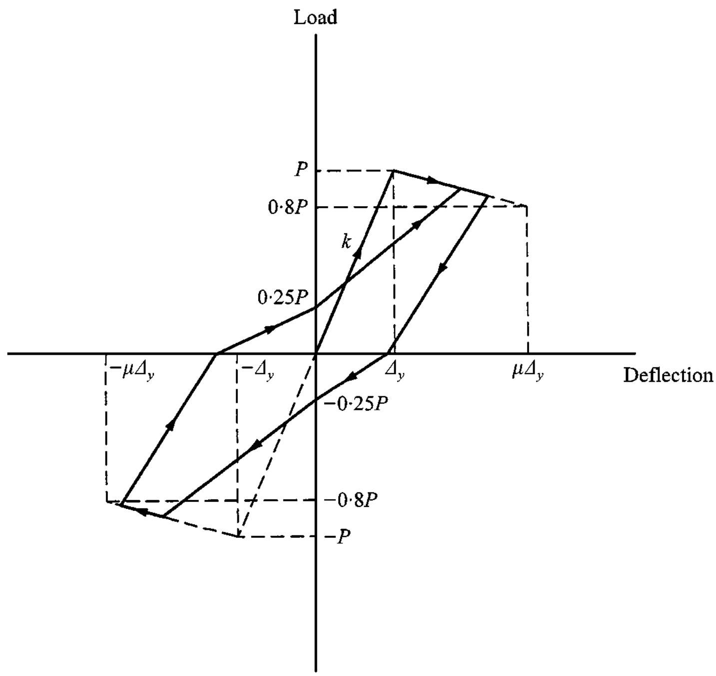

Figure 4. Assumed load–deflection relation of the connecting beams.

monotonic load–deflection curve, as depicted in Figure 4 where k is the initial stiffness, P is the yield strength and λ is the ratio of the deflection at 0·8 of yield strength on the descending branch of the monotonic load–deflection curve to the deflection at yield strength. The shape of the cyclic load–deflection curve follows generally that of the experimentally obtained typical load–deflection curve for a beam with high shear presented by Meyer in reference [14].

2.2. SYSTEM EQUATION OF MOTION

For dynamic analysis , the inertia effect of the building is simulated by lumping the mass of each storey at the corresponding floor level. Vertical inertia is neglected as the seismic excitation and response are mainly horizontal. Since at each floor level there is a floor slab which acts as rigid diaphragm, the whole floor is assumed to move horizontally as a rigid body. The vertical and rotational degrees of freedom are reduced before the dynamic analysis. Let the system stiffness matrix equation of the structure be given in partitioned form as

$$\begin{bmatrix} F_x \\ F_y \end{bmatrix} = \begin{bmatrix} K_{xx} & K_{xy} \\ K_{yx} & K_{yy} \end{bmatrix} \cdot \begin{bmatrix} a_x \\ a_y \end{bmatrix}, \tag{1}$$

in which $\{F_x\}$ and $\{a_x\}$ are the generalized force and displacement vectors of the horizontal degrees of freedom, and $\{F_y\}$ and $\{a_y\}$ are the generalized force and displacement vectors of the vertical and rotational degrees of freedom. Taking $\{F_y\}$ as zero and eliminating $\{a_y\}$, the matrix equation is reduced to

$$\{F_x\} = [K^*] \cdot \{a_x\}, \tag{2}$$

where the equivalent stiffness matrix $[K^*]$ is given by

$$[K^*] = [K_{xx}] - [K_{xy}][K_{yy}]^{-1}[K_{yx}]. \tag{3}$$

With only the horizontal degrees of freedom left as independent variables, the system equation of motion may be expressed as

$$[M]\{\ddot{a}_x\} + [C]\{\dot{a}_x\} + [K^*]\{a_x\} = -[M]\{\ddot{a}_g\}, \tag{4}$$

in which $[M]$ and $[C]$ are the mass and damping matrices, and $\{a_g\}$ is the horizontal movement of the ground due to seismic excitation. A dot at the top denotes differentiation with respect to time once. Two dots at the top denote differentiation with respect to time twice.

Rayleigh damping is assumed. As suggested by Wilson and Penzien [15], the damping matrix is taken to be of the following form:

$$[C] = \alpha_1[M] + \alpha_2[K^*], \tag{5}$$

where α_1 and α_2 are coefficients as calculated below

$$\alpha_1 = \frac{2(\lambda_i\omega_j - \lambda_j\omega_i)}{\omega_j^2 - \omega_i^2}\,\omega_i\omega_j, \tag{6}$$

$$\alpha_2 = \frac{2(\lambda_j\omega_j - \lambda_i\omega_i)}{\omega_j^2 - \omega_i^2}, \tag{7}$$

in which ω_i and ω_j are the natural frequencies of the structure corresponding to modes i and j, and λ_i and λ_j are their respective damping ratios. In the analysis, the 1st and 3rd vibration modes are used, and the damping ratios are taken as 0·05.

2.3. NUMERICAL SOLUTION

The Newmark-β step-by-step time-integration method [16] is employed to obtained the solution of the dynamic equation. The two parameter β and γ of the Newmark integration are taken as $\frac{1}{4}$ and $\frac{1}{2}$ respectively. To achieve a reasonable accuracy of the dynamic response of the structure, the time step is taken as 1 ms which should be sufficiently small compared with the periods of the first few modes of vibration of the structure.

3. VERIFICATION OF STRUCTURAL MODELLING

The wide column frame analogy adopted for modelling the slit shear wall structures has the advantages of being simple, easy to understand and fast to analyze as compared to the more sophisticated finite element method. Its advantages are even more pronounced in dynamic analysis as the structure has to be analyzed again and again many times. Before proceeding to dynamic analysis, the applicability of the structural model is first verified by checking its structural analysis results with those obtained by the more rigorous finite element method.

A typical 20-storey shear wall structure with one central band of vertical slits, as shown in Figure 5, is analyzed. The depth of each beam is taken as 0·5 m. Two types of lateral loads are applied in turn: (a) uniformly distributed load; and (b) triangularly distributed load with maximum load intensity at the top. The total load applied in each case is 600 kN. The analytical results of the wide column frame analogy and those obtained by a standard finite element analysis package SAP90 (details of the finite element analysis have been presented in reference [1]) are compared in Table 1. It is seen from the comparison that the difference between the numerical results obtained by the present method and the finite element method is generally of the order of only 2–3%. Hence, the idealized structural model should be sufficiently accurate for the analysis of slit shear wall structures.

4. DYNAMIC ANALYSIS

The above 20-storey slit shear wall structure (shown in Figure 5) is also used as the numerical example for dynamic analysis. The mass of each storey is taken as

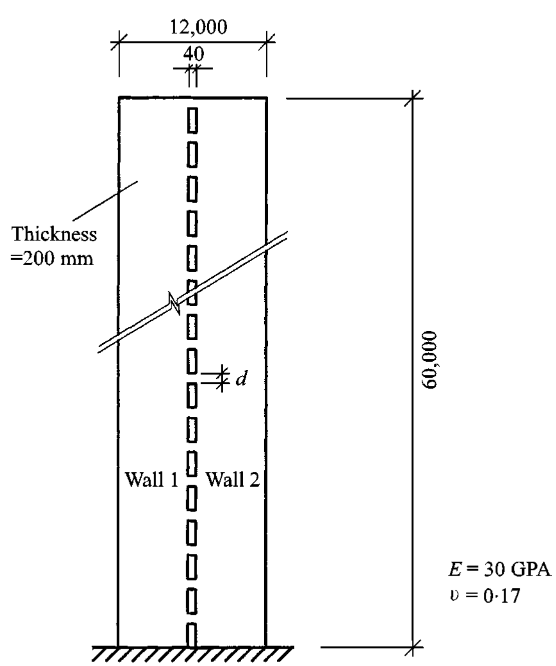

Figure 5. Slit shear wall structure analyzed (all dimensions in mm).

TABLE 1

Comparison with finite element analysis results

Loading type		Uniformly distributed load		Triangularly distributed load	
Method of analysis		Present method	Finite element method	Present method	Finite element method
Top deflection (mm)		19·6	19·9	28·6	29·4
Axial force at base of wall (kN)	Wall 1	2125	2148	2872	2918
	Wal 2	– 2125	– 2148	– 2872	– 2918
Shear at base of wall (kN)	Wall 1	308	312	302	301
	Wall 2	292	288	298	299
Moment at base of wall (kN m)	Wall 1	2656	2680	3364	3242
	Wall 2	2539	2541	3337	3235

80 000 kg. In order to ensure plastification of the connecting beams during the earthquake, the connecting beams are designed to have a smaller depth of 0·35 m. Three levels of longitudinal reinforcement ratio in these beams are considered: 0·005, 0·010 and 0·015. According to the analysis based on the softened truss model theory [5], these reinforcement ratios would give the connecting beams shear strengths of 330, 490 and 600 kN respectively. A solid shear wall model having the same dimensions and mass as the slit shear wall models studied is also analyzed to provide a basis for evaluating the effectiveness of the proposed structural system.

The seismic excitation applied is the El Centro 1940 NS earthquake record with a time duration of 10 s. Each wall model is analyzed twice, first under an earthquake excitation of intensity of 300 gal and then under an earthquake excitation of intensity 450 gal so as to evaluate the seismic performance of the models at different earthquake intensity.

4.1. STRUCTURAL RESPONSE UNDER EARTHQUAKE EXCITATION OF INTENSITY 300 gal

The seismic responses of the slit shear wall models are found to have similar patterns. In all the three slit shear wall models analyzed, the connecting beams start to yield at about 2·0–3·0 s after the onset of earthquake excitation. Before yielding of the connecting beams, the seismic responses of the slit shear wall models are almost identical to that of the solid wall model indicating that the slit shear walls initially behave like a solid shear wall. After plastification of the connecting beams, however, the seismic responses of the slit shear wall models immediately become smaller than that of the solid shear wall model. From Figure 6, where the time-history of the deflection response of the slit shear wall model with beam yield strength $=$ 330 kN is directly compared to that of the solid shear wall model, it can be seen very clearly that yielding of the connecting beams can significantly reduce the deflection response of the structure. Another obvious effect of yielding of the connecting beams is the gradual increase in the period of vibration of the structure. This is due to reduction in lateral stiffness of the structure as the connecting beams yield. The deflection responses of the other slit shear wall models are similar and are thus not plotted in Figure 6 for clarity.

The maximum deflection of the slit shear wall models and that of the solid shear wall model are compared in Figure 7. It can be seen from the deflection values plotted that the maximum top deflections of the slit shear wall models are about 14–25% lower than the corresponding value for the solid shear wall model. Hence, it may be concluded that the introduction of vertical slits can significantly reduce the deflection response of the structure despite reduction in lateral stiffness at the same time.

Likewise, the maximum inter-storey drifts of the various models are compared in Figure 8. About 19–26% reduction in maximum inter-storey drift has been achieved by the introduction of vertical slits. Such reduction in inter-storey drift can help to reduce damage to the other parts of the building structure and the non-structural components during earthquake.

From the results presented in Figures 7 and 8, it appears that in terms of effectiveness in reducing top deflection and inter-storey drift, the two slit shear wall models with beam yield strength equal to 330 and 490 kN, respectively, are similar in performance and they both perform better than the slit shear wall model with beam yield strength equal to 600 kN. It is, therefore, evident that the performance of a slit shear wall system depends on the yield strength of the connecting beams.

The analytical results also reveal that a lower beam yield strength would in general lead to larger post-yield shear deflections of the connecting beams. Figure 9 plots the maximum shear deflections of the connecting beams in the three slit shear

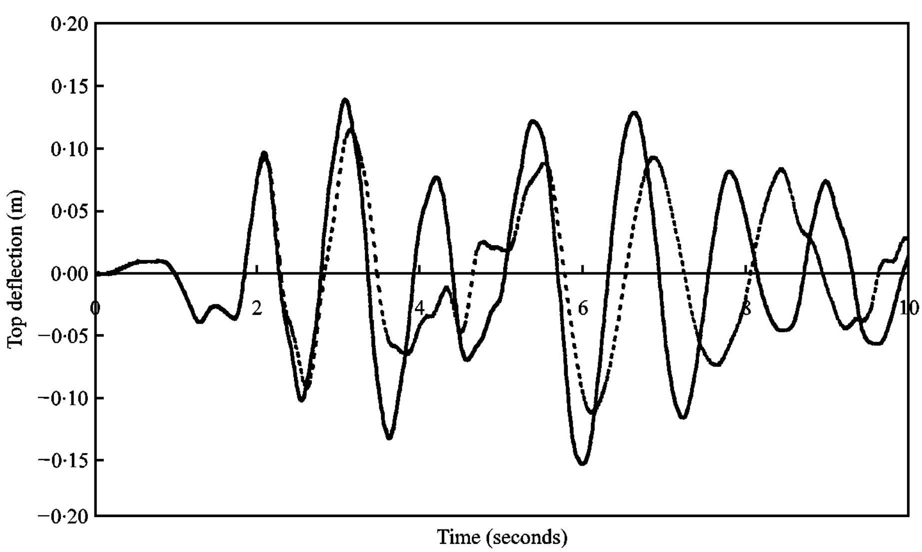

Figure 6. Time-histories of top deflection (seismic wave intensity = 300 gal).—— Solid wall; ····· slit wall (P = 330 kN)

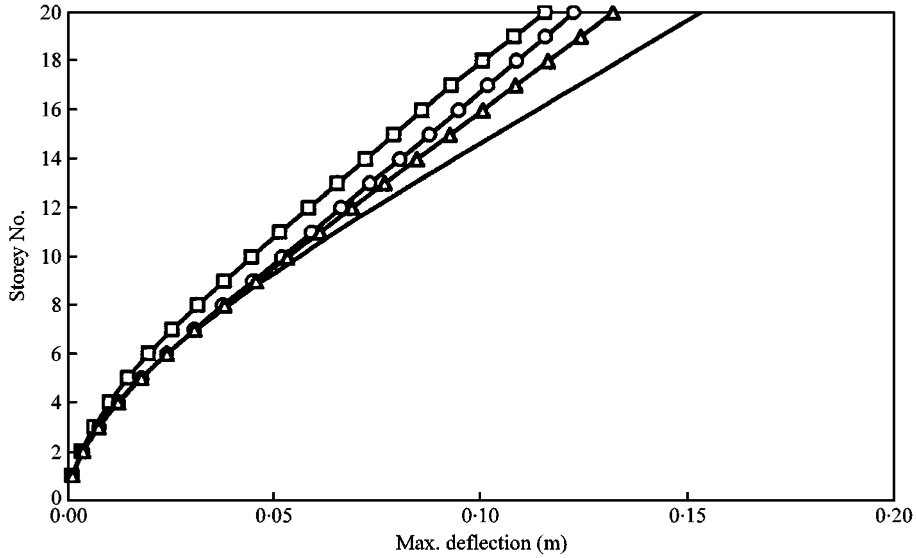

Figure 7. Envelops of floor deflections (seismic wave intensity = 300 gal). —— Solid wall; —□— slit wall (P = 300 kN); —○— slit wall (P = 490 kN); —△— slit wall (P = 600 kN).

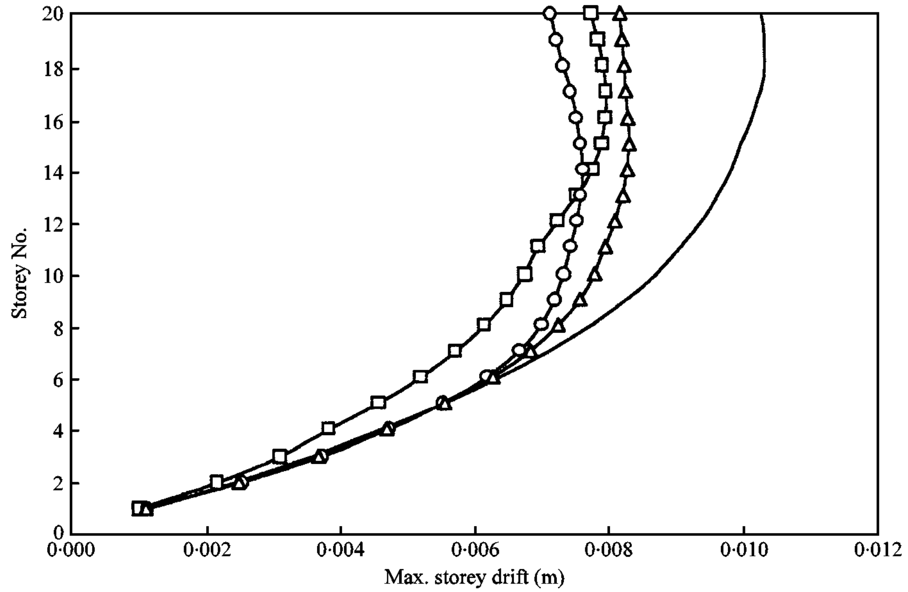

Figure 8. Envelops of inter-storey drift (seismic wave intensity = 300 gal). —— Solid wall; —□— slit wall (P = 330 kN); —○— slit wall (P = 490 kN); —△— slit wall (P = 600 kN).

wall models against the height of the structure. It can be seen from this graph that the shear deflections of the connecting beams in the model with the lowest beam yield strength are the largest. The large shear deflections would impose a high ductility demand on the connecting beams which may be difficult to deal with.

Hence, in the design of the connecting beams, both yield strength and ductility demand need to be considered. Since a lower yield strength would lead to a higher ductility demand, the connecting beams should not be designed to have such low yield strength as to cause their ductility to be exhausted (i.e. breakage of the beams) before the wall panel fails.

Table 2 summarizes the above results and presents the maximum shear and moment at the base level of the wall models. The wall shear and moment results reveal that with the introduction of vertical slits, the base shear induced onto the wall structure can be reduced by 25–33% whilst the base moment in the wall can be reduced by 21–25%. Among the beam yield strengths of 330, 490, and 600 kN considered, the beam yield strength that would cause the greatest reductions in wall shear and the moment is 330 kN. This is because a lower beam yield strength would

TABLE 2

Seismic response under excitation of El Centro wave (intensity = 300 gal)

	Solid wall	Slit wall (P = 330 kN)	Slit wall (P = 490 kN)	Slit wall (P = 600 kN)
Max. top deflection (mm)	153	115	122	132
Max. inter-storey drift (mm)	10·3	7·9	7·6	8·3
Max. shear deflection of beam (mm)	—	6·1	5·3	3·9
Max. base shear (kN)	625	420	434	469
Max. base overturning moment (kN m)	2112	1583	1643	1674

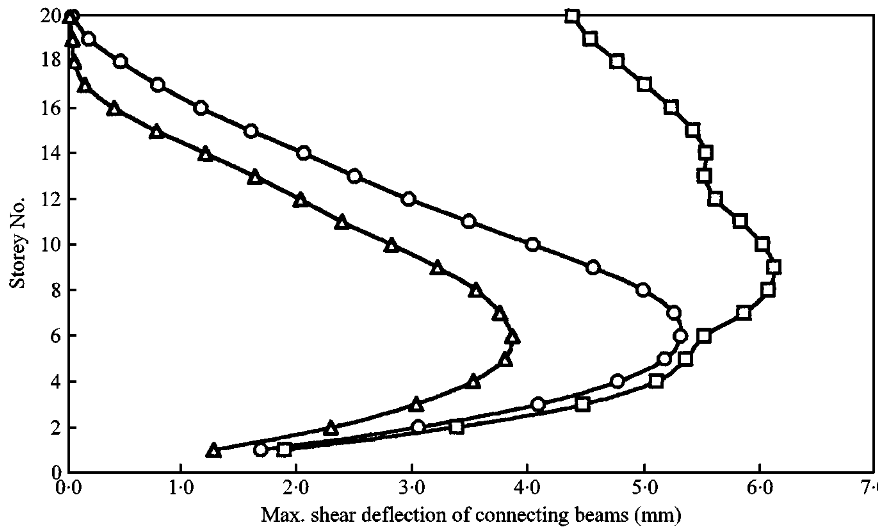

Figure 9. Envelops of shear deflection (seismic wave intensity = 300 gal). —□— slit wall (P = 330 kN); —○— slit wall (P = 490 kN); —△— slit wall (P = 600 kN).

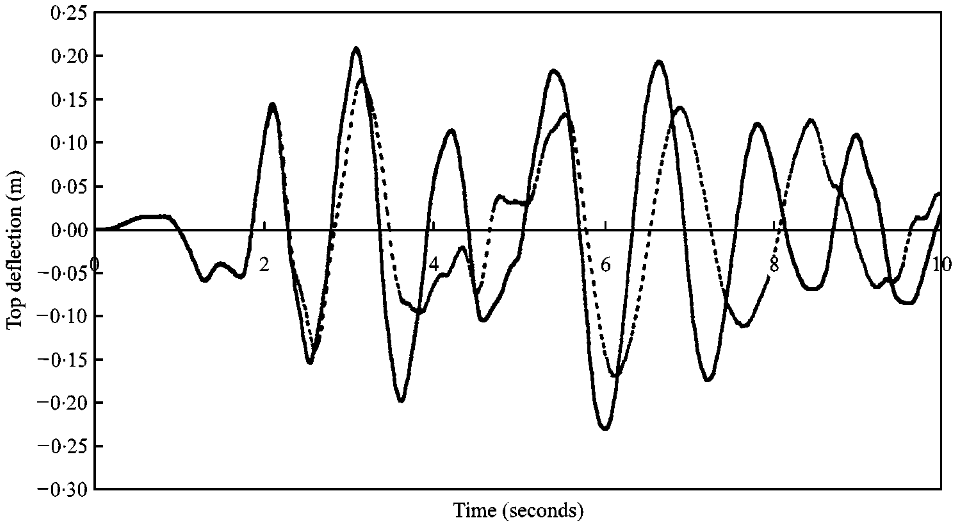

Figure 10. Time-histories of top deflection (seismic wave intensity = 450 gal). ——Solid wall; - - - - slit wall (P = 490 kN).

also lead to a smaller lateral stiffness and therefore a a smaller reaction from the wall structure for a given lateral deflection. Nevertheless, balancing the seismic performance of the slit shear wall structure in terms of seismic load reduction and the ductility demand on the connecting beams which may not be easy to meet, it seems that for this particular case, a beam yield strength of around 490 kN would give the best overall performance. At this beam yield strength level, the reduction in top deflection, inter-storey drift, base shear and base moment are 20, 26, 31 and 22% respectively.

4.2. STRUCTURAL RESPONSE UNDER EARTHQUAKE EXCITATION OF INTENSITY 450 gal

The structural responses of the various models under a higher earthquake wave intensity of 450 gal are essentially similar to those in the previous case. Because of the higher earthquake intensity, the connecting beams of the slit shear wall models generally yield at earlier time than in the previous case. As before, yielding of the connecting beams lead to significant reduction in the deflection response of the wall structure. This is illustrated in Figure 10 where the time-history of the deflection response of the slit shear wall model with beam yield strength = 490 kN is compared to that of the solid shear wall model.

The maximum deflections and inter-storey drifts of the various models are compared in Figures 11 and 12 respectively. This time, it is seen that the introduction of vertical slits can reduce the top deflection by 16–24% and the inter-storey drift by 19–31%.

The maximum shear deflections of the connecting beams are plotted in Figure 13. Again, it is seen that a lower beam yield strength would lead to a larger maximum shear deflection of the connecting beams. Since the earthquake

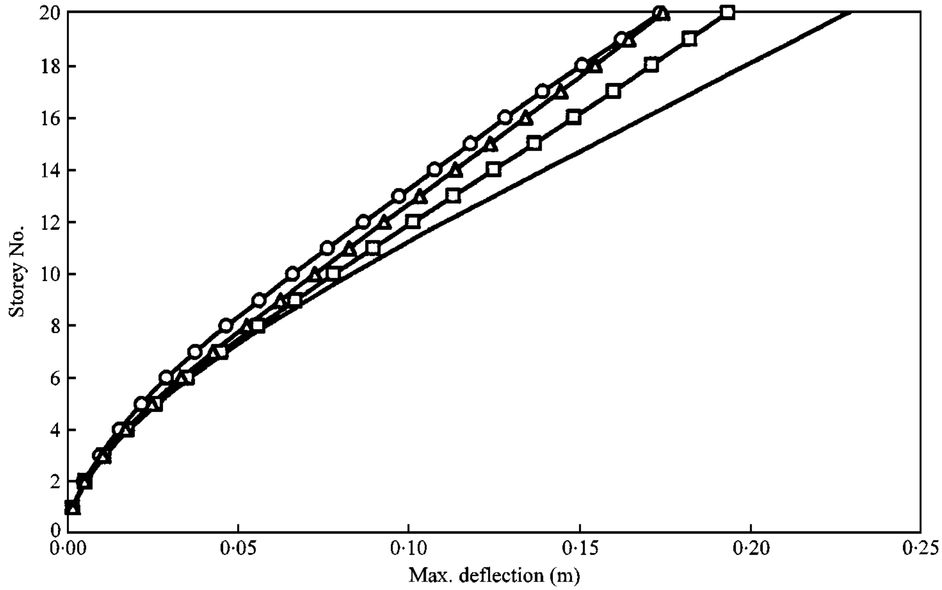

Figure 11. Envelops of floor deflections (seismic wave intensity = 450 gal). —— Solid wall; —□— slit wall (P = 330 kN); —○— slit wall (P = 490 kN); —△— slit wall (P = 600 kN).

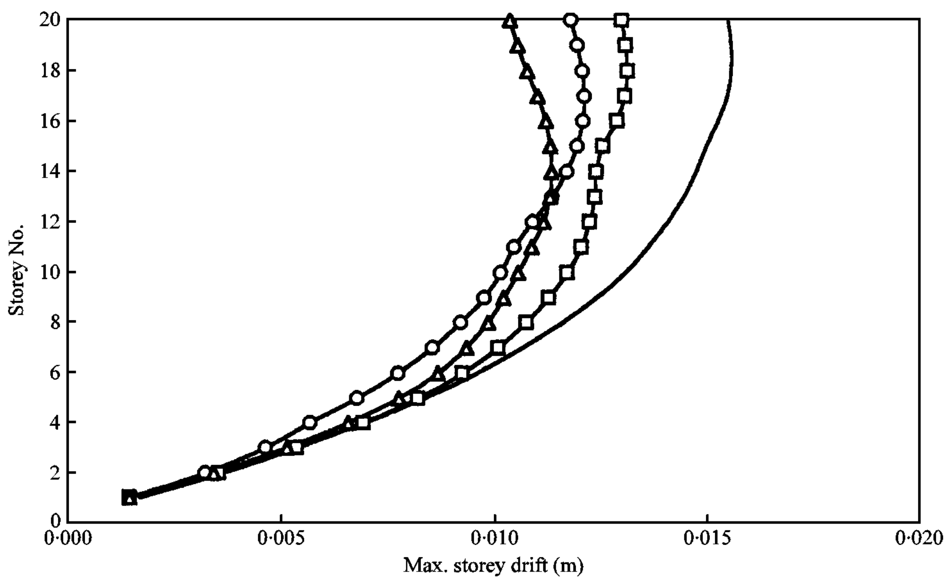

Figure 12. Envelops of inter-storey drift (seismic wave intensity = 450 gal). —— Solid wall; —□— slit wall (P = 330 kN); —○— slit wall (P = 490 kN); —△— slit wall (P = 600 kN).

excitation applied this time is of much higher intensity, the maximum shear deflections of the connecting beams are higher than those in the previous case. Thus, at a higher earthquake intensity, it is necessary to use a higher beam yield strength so as to reduce the ductility demand to a more tractable level.

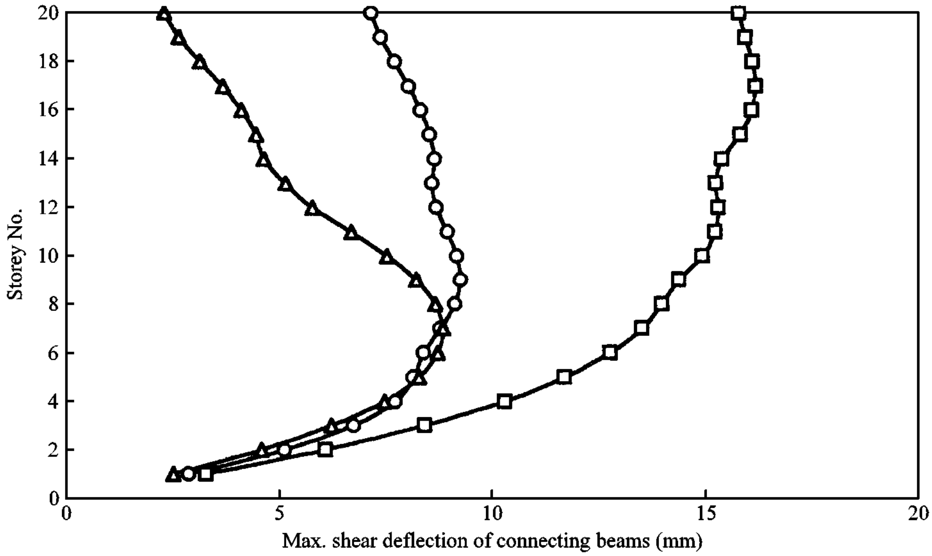

Figure 13. Envelops of shear deflection of beam (seismic wave intensity = 450 gal). —□— Slit wall (P = 330 kN); —○— slit wall (P = 490 kN); —△— slit wall (P = 600 kN).

TABLE 3

Seismic response under excitation of El Centro wave (intensity = 450 gal)

	Solid wall	Slit wall (P = 330 kN)	Slit wall (P = 490 kN)	Slilt wall (P = 600 kN)
Max. top deflection (mm)	230	194	174	175
Max. inter-storey drift (mm)	16	13	12	11
Max. shear deflection of beam (mm)	—	16·1	9·3	8·9
Max. base shear (kN)	937	745	750	754
Max. base overturning moment (kN m)	3168	2329	2375	2404

The maximum shear and moment at the base level of the wall models are tabulated in Table 3 where the above results are also put together as a summary. Reductions in the base shear and base moment of 20 and 26%, respectively, have been achieved by the introduction of vertical slits. From this table, it can also be seen that the optimum beam yield strength for maximum reduction in deflection is 490 kN, the optimum beam yield strength for maximum reduction in inter-storey drift is 600 kN, whilst the beam yield strength that would cause the greatest reduction in wall shear and moment is 330 kN. Balancing the seismic performance of the slit shear wall structure and the ductility demand on the connecting beams, it is suggested that for this particular case, the beam yield strength should be set at 600 kN. At this beam yield strength level, the reduction in top deflection, inter-storey drift, base shear and base moment are 24, 31, 20 and 24% respectively.

5. DISCUSSION

The above analysis reveals that yielding of the connecting beams formed between consecutive slits can significantly reduce the seismic response of the wall structure. There is a certain optimum beam yield strength for best overall performance of the slit shear wall system which depends on the intensity of earthquake excitation that the structure needs to withstand. With near optimum design of the connecting beams, the deflection response of the structure and the seismic loads acting on the structure can both be reduced by about 20–25%. There is no straightforward design rule for finding the optimum beam yield strength. From the results obtained so far, it appears that the optimum beam yield strength is generally within 50–80% of the maximum load that the beam will be subjected to during the earthquake if they remain elastic. A good starting point is to take the beam yield strength as 65% of this maximum load. It is believed that with the yield strength of the beams set at such level, the slit shear wall system should perform reasonably well. An iterative trial-and-error procedure may then be started to successively improve the performance of the system by changing the yield strength of the beams.

Since the connecting beams have to yield before the wall panel does, they will be subjected to cyclic inelastic deformation in the course of action. For best performance, the connecting beams should maintain their load carrying and energy dissipation capacities until the whole structure fails. This would impose a certain ductility demand on the connecting beams. Hence, in the design of the connecting beams, both the strength requirement and ductility demand need to be considered. The strength requirement and ductility demand are, however, interrelated. As revealed in the above dynamic analysis, a lower yield strength would lead to a higher ductility demand and a higher yield strength would lead to a lower ductility demand. The strength requirement is seldom a concern as the depth of the beams can always be increased to achieve the required strength. It is the ductility demand that is the major problem as it is not always easy to meet. Hence, if the ductility demand is found intractable, the yield strength of the beams should be increased so that the ductility demand may be lowered and become easier to deal with.

For the case of reinforced concrete connecting beams, the ductility is higher if the steel reinforcement yields before the concrete is crushed. Therefore, it is important to design the reinforced concrete connecting beam in such a way that the steel reinforcement is not excessive so as to ensure steel yielding before failure. Previous research has, however, shown that it is not easy to design a short concrete beam to achieve high ductility [3]. Alternative designs using diagonal reinforcement in the connect beams or even changing to use steel for the shear connections between adjacent sub-wall units should be considered. Further research is recommended.

6. CONCLUSIONS

The seismic response of slit shear walls has been analyzed with the elasto-plastic behaviour of the connecting beams taken into account. By analyzing slit shear wall models with different connecting beam details, a parametric study of the effect of the beam yield strength on the seismic performance of the slit shear wall system has been carried out. From the theoretical study, the following conclusions may be drawn:

(1) By introducing vertical slits into a wall structure and designing the connecting beams formed between consecutive slits to yield before the wall panel fails, the deflection response of the structure and the seismic loading induced onto the structure can both be reduced by 20–25%.
(2) There is a certain optimum beam yield strength for best overall performance of the slit shear wall system. The optimum beam yield strength depends on many factors including the intensity of the earthquake that the structure needs to withstand but is generally within 50–80% of the maximum load that the beams will be subject to during the earthquake if they remain elastic.
(3) In the design of the connecting beams, both the strength requirement and the ductility demand, which are interrelated, need to be considered. A lower yield strength would lead to a higher ductility demand and *vice versa*.
(4) It is anticipated that the major difficulty with the design of a slit shear wall system is the high ductility demand on the short connecting beams. Alternative designs using for example diagonal reinforcement for the concrete beams or even structural steel which may have higher ductility for the shear connections should be considered.

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