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A new experimental set-up for high shear loading of reinforced concrete walls

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ABSTRACT: A new experimental set-up for cyclic testing of R/C walls with low shear-ratio (M/VL, where M is the moment, V is the applied shear and L is the cross section length) is described and shown to simulate in a more realistic manner the behaviour of critical regions of walls of multistorey buildings with low shear ratios.A total of 3 tests are described and analised, with a view to investigate boundary condition effects. Some behavioral observations from previous tests are shown to be boundary conditions dependent effects. This is due to the fact that in those tests the proximity of the point of load application to the base of the wall generates a direct transfer of load to the foundations which is not possible in multistorey buildings as the walls are higher and more prone to beam behaviour.

1. INTRODUCTION

Structural walls are today widely used in multistorey R/C buildings, as they provide stiffness, strength, ductility and energy dissipation capacity. Code provisions for R/C walls, in particular shear design, are based on results of tests in beams and walls. However, the conditions in which those tests are performed do not always simulate adequately the members being studied, what may lead to misleading conclusions.

The purpose of this paper is to describe a new experimental set-up for cyclic testing of R/C walls under high shear forces, developed at Imperial College. The specimens are supposed to model the behaviour of the critical region (near the base section, where the nonlinear behaviour is supposed to take place) of walls of multistorey buildings. It is considered that the new set-up provides more realistic boundary conditions than the set-up commonly used in this kind of tests for walls with shear dominated behaviour, i.e., low shear-ratios M/VL.

2. LOADING SYSTEM FOR R/C WALLS TESTING

Tests on R/C walls for earthquake resistant design are usually performed by applying horizontal, and sometimes vertical loads, on top of the walls. The loads are usually applied on the top of the wall by a stiff beam which distributes the applied loads along the top section of the wall, as shown in figure 1.

3. DISADVANTAGES OF PREVIOUS LOADING SYSTEM

The loading system shown in figure 1 is simple and suitable for most cases, specially flexural walls. However, to test walls with shear dominated behaviour it is necessary to impose low shear ratios M/VL.

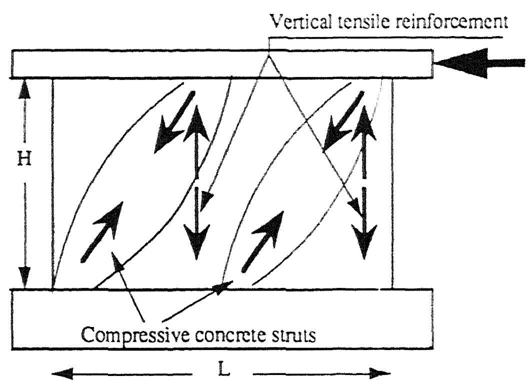


Figure 1 . Direct transfer of load in low rise wall

For low values of M/VL, close to unity for instance, the use of the loading system shown in figure 1 leads to specimens with height similar to the length of the cross section. The stiff beam on top is therefore placed close enough to the bottom region of the wall to influence the overall behaviour. In these conditions part of the load may be transferred directly to the foundation, as shown in figure 1. This figure indicates that this loading system may not be suitable to represent the loading conditions in bottom storeys of multistorey buildings. In this case the wall is much higher than its critical region and the whole member is more prone to beam behaviour. As the load is applied to the wall by the slabs along the whole height of the structure, the direct transfer of load to the foundations is not possible. Therefore, it can be concluded that tests in low-rise walls (with H similar to L, as in figure 1) may not represent adequately the load transferring mechanisms in bottom regions of walls of multistorey buildings.

4. PROPOSED SOLUTION

The proposed set-up comprises a wall with an aspect-ratio considerably higher than the required shear-ratio. The aspect-ratio H/L must be close to 2 or more for the member to be prone to beam behaviour and to avoid direct transfer of load to the foundation. In order that the applied shear-ratio is less than the wall aspect-ratio the set-up shown in figure 2 was devised. The load is applied not directly to the wall but to steel frames that encase the top beam of the wall. The load is applied to the steel frames at the level required to induce the desired shear-ratio at the base of the wall.

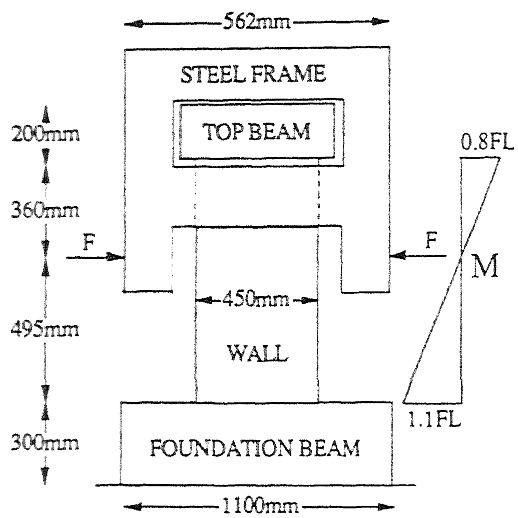


Figure 2. Proposed loading system

5. PHILOSOPHY OF THE TESTS

The purpose of the tests was to reproduce the conditions more likely to lead to shear failure in critical regions of R/C walls of multistorey building. The applied shear-ratio was 1.1, as this was considered a reasonable lower limit of the values that can be found in medium and high-rise buildings. A static analysis considering only the contribution of the walls and a triangular distribution of horizontal loads, as usually prescribed by the codes, would generally lead to much higher values of the shear-ratio. However, several factors may reduce the shear-ratio considerably:

- Interaction with the frames.
- Influence of the higher modes of vibration.
- The overall nonlinear behaviour of the structure.
- The concentration of the nonlinear behaviour of the wall on its bottom region
- Redistribution of shear from a wall in tension to a wall in compression.

Previous studies (Wang et al, 1975) have shown that the shear-ratio of a wall of a ten storey R/C building may be as low as 1.5 .

The aspect-ratio of the specimens is 1.9 in order that the behaviour of the bottom region of the wall, where the inelastic behaviour is concentrated, is not affected by the mode of load introduction on top. The bending moment diagram in the wall is as shown in figure 2. It is intended that the upper part of the wall (above the point of contraflexure) remains elastic, as its sole purpose is to transfer the load to the bottom region. In order to ensure that, the top section was designed with extra flexural reinforcement to increase its flexural capacity and the point of load application is set closer to the top beam than to the foundation beam, as shown in figure 2. The ratio between the moments on the top and bottom sections is 0.73.

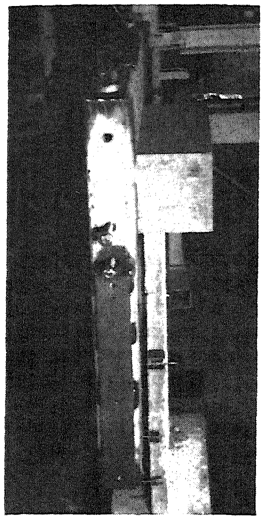
6. DESCRIPTION OF THE TEST SET-UP AND SPECIMENS

Details of the connection between the specimens and the steel frames are shown in figure 3. Each steel frame encases the top beam on either side of the wall. The width of the top and foundation beams is 300mm and the width of the steel frames is 100mm. This allows each frame to be placed on the side of the wall, which width was 4.5mm, leaving a small gap. This can be clearly seen in figure 3.a. The two frames are connected by means of a top plate and two side plates as shown in figure 3.b. The whole system is adjusted by means of a set of steel plates and studs. The photograph of the front view of the system in figure 3.c shows the top beam encased by the frames. To avoid concentration of loads, a set of steel plates is placed between the top beam and the steel frames on all sides. The system is finally adjusted by a set of vertical and horizontal steel studs in such a way that the top beam is rigidly connected to the frames. This connecting system is preferred to alternative ones, such as grouting the gap between the frames and the top beam, due to its versatility. It can be assembled or dismantled in about 15 minutes, and misalignments can be easily corrected by releasing the studs and adjusting the position of the frames.

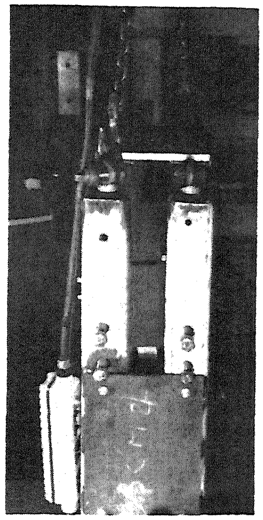
The horizontal load is transferred from the jack to the steel frames by means of two cylindrical rollers which are connected to an horizontal frame that can only move in the horizontal direction in the plane of the wall. A schematic representation of the test-rig is shown in figure 4.

Figure 2 shows the dimensions of the specimens tested and figure 5 shows the design of the reinforcement of wall SW13. The two other models, SW16 and SW17, were designed with the same amount and distribution of vertical reinforcement in the edge members. These models had no vertical web reinforcement and the amount of horizontal web reinforcement below the level where M=0 was 4 and 5 stirrups of 2 branches, respectively. Table 1 lists the properties of the steel. The specified concrete cube strength was 45 MPa.

The load history was the same for all specimens. The tests were displacement controlled and three cycles were performed at each displacement amplitude. This is based on observations from another study at Imperial College (Elnashai et al,1990) where it was noted that little or no degradation occurs beyond the second or third cycle at each displacement amplitude. The displacement amplitude of each group of three cycles was incremented approximately 0.8mm.



a) Wall with one frame - side view



b) Wall with two frames and connecting plates - side view

c) Front view

Figure 3. Photographs of the conection wall-frames

Table 1 - Steel properties

ϕ (MPa)	Yield stress (MPa)	Ultimate stress (MPa)
4	413	471
6	608	667
8	436	623

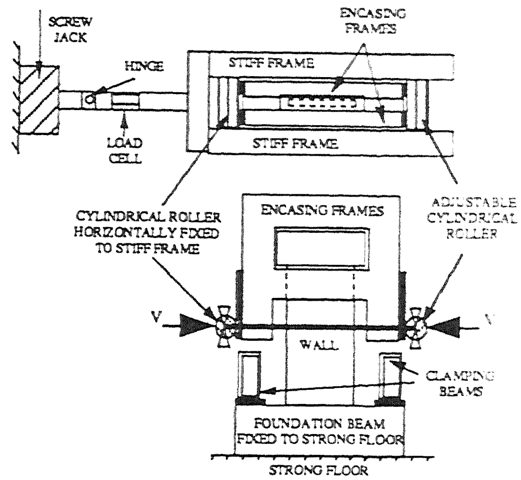


Figure 4. Schematic representation of the test-rig

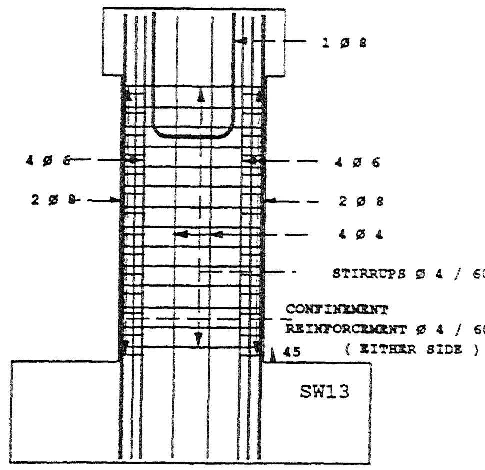


Figure 5. Reinforcement details of wall SW13

The instrumentation comprised a load cell and a maximum of 17 displacement transducers and 32 strain gauges.

7. TESTS AND RESULTS

A full description of the tests and results can be found elsewhere (Lopes, 1991). The load and horizontal displacement (registered at the level where M=0) at ultimate load and failure for the three walls are presented in table 2.

8. DISCUSSION

The shear strength provided by the steel (horizontal web reinforcement) was 72.8kN for wall SW13, 41.6kN for SW16 and 52kN for SW17. The ultimate

Table 2. Force and displacement at ultimate load and failure

Wall	Ultimate load		Failure	
	Force (kN)	Disp (mm)	Force (kN)	Disp (mm)
SW13	108.3	7.5	80.6	9.3
SW16	80.3	4.1	65.0	8.1
SW17	83.6	4.8	66.8	8.1

loads achieved by all walls, presented in table 2, were substantially above these values.

A detailed analysis of the main results, presented by Lopes (1991), has led to the conclusion that, at the ultimate limit state, the shear strength (with respect to the diagonal tension mode of failure) of the plastic hinge zone of R/C walls is essentially the sum of two components: the tensile force resisted by the horizontal web reinforcement that crosses the potential failure surface in the zone of the wall under axial tension and the shear strength of the compressive zone near the base of the wall. The second component can be calculated as the product of the cohesion of the concrete and the area of the compressive zone. Results from a series of tests in walls of shear ratio 2 (Pilakoutas, 1990) has led to the same conclusion. The shear resistance is therefore independent of the shear ratio. This conclusion contradicts code provisions (EC8 1988, ACI 1983, New Zealand Standard 3101, 1982) according to which the contribution of concrete increases for walls with low shear ratios and the efficiency of the horizontal web reinforcement decreases while vertical web reinforcement becomes effective in resisting shear for those walls. Those code provisions result from observations from tests of low rise walls. The authors consider that the reason for those observations is the fact that part of the load was transferred directly to the foundations, as shown in figure 1. The resistance associated with this load transferring mechanism, in which the horizontal web reinforcement does not participate, was attributed to the concrete. It should be noted that strain gauge readings in the horizontal web reinforcement of the different walls described in this paper give no indication of the decrease in efficiency of that kind of reinforcement. The observed efficiency of the vertical web reinforcement in previous tests is due to its participation in the new load transferring mechanism.

The most relevant codes establish a minimum length of the plastic hinge zone equal to the length of the cross section. However, strain gauge readings in the vertical reinforcement indicate that plasticity in the flexural reinforcement spread much less than that. Again, this is attributed to the different boundary conditions. Code provisions may be a consequence of observations from previous tests of low rise walls where the boundary conditions allowed part of the load to be transferred directly to the foundations. The arch mechanism associated with this load transferring mechanism imposes a constant force on the tensile reinforcement, what does not happen if the element is more prone to beam behaviour.

9. CONCLUSIONS

A new experimental set-up for cyclic testing of R/C walls with low shear ratio was described. It differs from traditional rigs because it imposes different boundary conditions. The specimens are designed with an aspect ratio H/L higher than the desired shear ratio M/VL in such a way that the element is more prone to beam behaviour and direct transfer of load to the foundations is not possible.

It was concluded that some observations from tests of low rise walls which gave rise to code provisions for walls with low shear ratios are boundary conditions dependent effects. These effects are the higher contribution of concrete to shear resistance, the increased efficiency of vertical web reinforcement and the decreased efficiency of horizontal web reinforcement in resisting shear and the length of the plastic hinge zone. Therefore, these observations reflect the behaviour of low rise walls but they should not be extrapolated as representing also the behaviour of higher walls with low shear ratios.

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