SHEAR CONTROLLED ULTIMATE BEHAVIOR OF NON-DUCTILE REINFORCED CONCRETE COLUMNS

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ABSTRACT:

Reinforced concrete columns supporting buildings with a soft-storey in regions of low and moderate seismicity are typically non-ductile and susceptible to shear failure. Half-scaled reinforced concrete column specimens representative of current construction practices were tested to study their cyclic force-deformation behaviour. Importantly, the gravitational load carrying capacity of the column at ultimate conditions was of particular interests in the investigation. The innovative Vision Methology System (VMS) was used to measure the deformation of the column in 3D. Using the VMS technique, deformation in the column has been resolved into flexural, shear and yield penetration components. Experimental results were presented in this paper and compared with behaviour predicted using various existing models. The discrepancies between the experimental results and the model predictions are highlighted. The development of an accurate deformation models forms part of the displacement-based methodology for the seismic performance assessment of buildings with a soft-storey.

1. INTRODUCTION

Buildings supported by soft-storey columns are well known to be vulnerable to collapse and severe damage under earthquake excitations. The vulnerability of such buildings is aggravated by non-ductile behaviour of the columns and this behaviour is often expected in regions of low to moderate seismicity where detailing is generally poor. These columns, particularly those with a low shear span-depth ratio, are susceptible to brittle failure associated with shear. A recent survey of soft-storey columns around the Melbourne Metropolitan Area (MMA) revealed very low shear span-depth ratios. These columns are susceptible to shear failure during an earthquake. To identify the vulnerability of these soft-storey buildings, the load-deformation behaviour of such columns under cyclic loading was modelled accurately as part of the displacement-based seismic evaluation methodology.

Column deformation is made up of flexural, shear and yield penetration components. Methods that are used to predict deformation due to flexural and yield penetration such as those developed by Watson et al. [1994] and Alsiwat and Saatcioglu [1992], have been shown to be capable of providing realistic estimations. However, the truss analogy method [Park and Paulay, 1975] has been investigated as a part of this project and does not give a good prediction of shear deformation particularly when the behaviour of the column is controlled by shear. The more recently developed model based on Modified Compression Field Theory (MCFT) [Vecchio and Collin, 1986] has been found to be more reliable in providing realistic predictions. Estimates made using this theory have been compared with experimentally determined values in this paper.

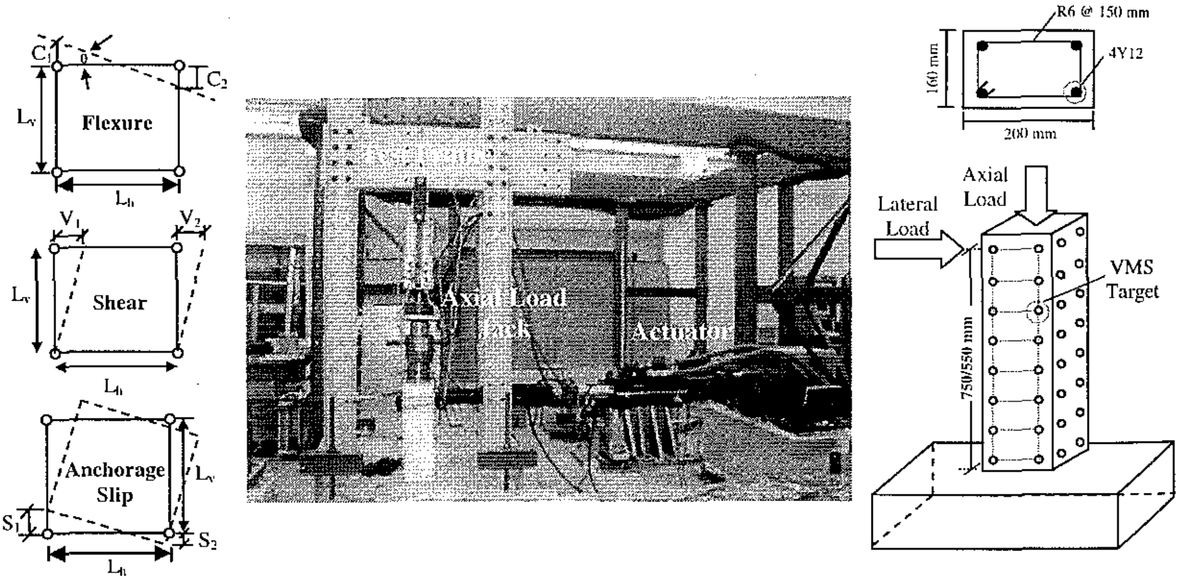
An experimental program was undertaken to investigate the cyclic force-deformation behaviour of half-scaled cantilever column specimens that are representative of real conditions in the MMA. The effects of shear on the deformation behaviour and failure mechanisms were of particular interest. The measured deformation has been resolved into the flexural, shear and yield penetration components using *Digital Close-range Photogrammetry Technique* (which is also known as the *Vision Methology System*, *VMS*). Movement of column segments, which cannot be measured reliably in such detail by conventional methods, was monitored using this technique. The measured deformation components have been compared with theoretical predictions.

With traditional experimentation, load testing of the column is normally terminated when the lateral load resisting capacity of the column has deteriorated by 20%. This approach seems justified in high seismic regions which are characterized by high energy demand associated with strong ground shaking which would require substantial residual strength capacity of the column to laterally support the building. Under conditions of displacement controlled behaviour which is low in energy demand, the building could survive the earthquake regardless of its residual lateral strength. The ability of a damaged column to carry axial load becomes the criterion to define the limit of ultimate performance. Whilst many column tests have been done in the past, only few tests emphasize the axial load carrying capacity of the column in the damaged stage [Moehle et al., 2002]. The experimentation described in this study uses this criterion to identify the deformation capacity of non-ductile columns.

2. EXPERIMENTAL PROGRAM

Surveys of soft-storey columns in the MMA revealed potential non-ductile behaviour due to lack of confinement. These columns were designed in accordance with the Australian Standard but without seismic provisions. The quantity of the transverse reinforcement was usually controlled by code-specified minimum requirements. An axial load ratio of 0.2 is typical for ground floor columns when calculated according to the suggestion in ATC-40 [1996]. Shear span-depth ratios vary between 2.0 and 4.6. The reinforcement ratio is found to be in order of 1.5%. For these columns, the influence of shear actions on the column behaviour becomes pronounced when the shear span-depth ratio is low (i.e. less than 3.5) in which case brittle failure may result.

To study the influence of shear on the behaviour of non-ductile columns, half-scaled reinforced concrete column specimens with varying shear span-to-depth ratios were tested. The specimens consisted of cantilever columns supported by a 300mm concrete block which was secured to the strong floor of the laboratory. Column sectional dimensions were 200mm in the direction of the applied load and 160mm in the orthogonal direction. An axial load ratio of 0.2 was selected to represent typical ground floor columns and shear span-to-depth ratios of 3.75 and 2.75 were used. The lengths of column specimens were 750mm and 550mm for first specimen (S1) and second specimen (S2) respectively. The reinforcement ratio for both columns was 1.4%. Detailing of column specimen is shown in Figure 1c.



(a) (b) (c)

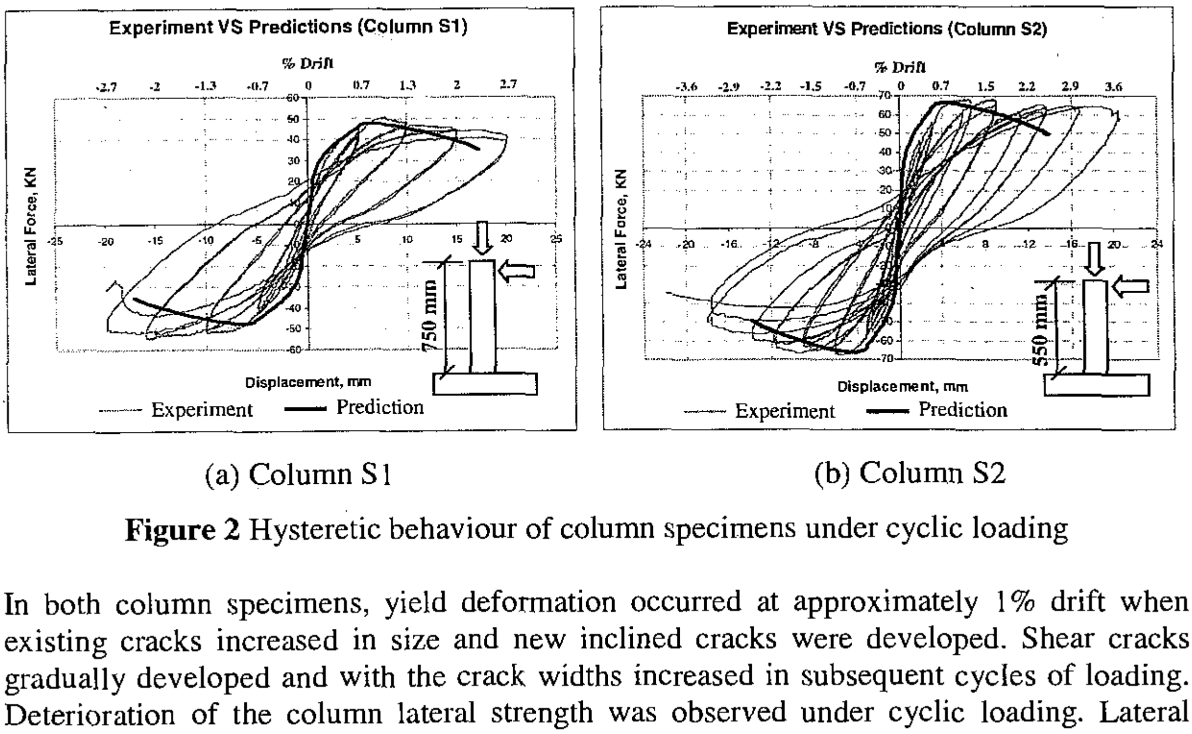
Figure 1 Experimental set up and interpretation of deformation from VMS data

The innovative VMS technique was used for measuring section deformations and surface strains of the columns in 3D (using closely spaced "targets" which were attached to the surface of columns as shown in Figure 1c). Using this technique, sectional deformation of the columns can be resolved into flexural, shear and yield penetration components as illustrated in Figure 1a.

The experimental set up as shown in Figure 1b was used to simulate gravity and lateral earthquake loads. The constant axial load of 280kN (axial load ratio = 0.2) was applied to a specimen using a 500kN-jack. The axial load was monitored and kept constant throughout the test. The lateral load was applied using a 250kN-hydraulic actuator. Specimens were subjected to quasi-static loading history as suggested by Priestley and Park [1987]. The loading history was modified to suit the non-ductile behaviour of the column. A column specimen was initially subjected to one cycle of lateral loading ±0.75 times the ideal yield strength at the critical section (F_y). The yield displacement was then found by extrapolating a straight line from the origin through the force-displacement point at $0.75F_y$ to the theoretical flexural strength F_y. The average of the two values calculated for the two cyclic reactions was adopted. Subsequent loading consisted of displacement-controlled testing to ductility ratios (μ) of ±1, ±2, ±3, ±4, ±5, and ±6. Two cycles of loading were used with each ductility ratio to ensure that the hysteretic behaviour could be maintained. Digital photos of the columns were taken when the peak displacement had been reached in each load cycle. The test was terminated only when the column had lost its axial load carrying capacity.

3. EXPERIMENTAL RESULTS

The force-deformation behaviour of the column specimens (S1 and S2) subjected to cyclic loading is shown in Figure 2. With the flexure-dominated column (specimen S1), the gravitational load carrying was lost following the buckling of the compression reinforcement. With column specimen S2, high shear forces have resulted in the formation of shear cracks distributed along the length of specimen. The specimen was capable of carrying full axial load whilst its lateral strength has deteriorated.



(a) Column S1

(b) Column S2

Figure 2 Hysteretic behaviour of column specimens under cyclic loading

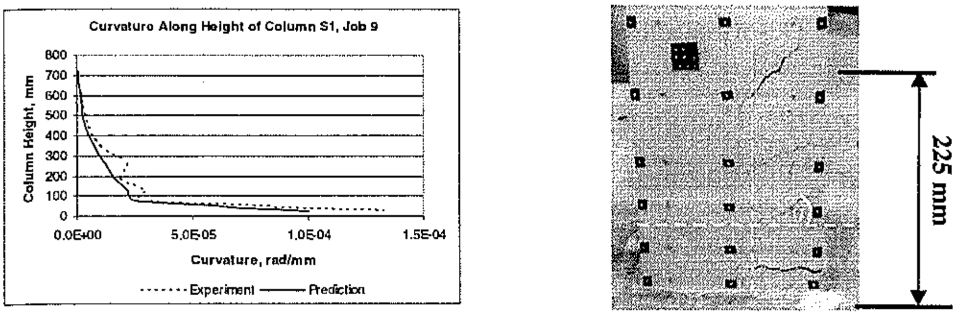
In both column specimens, yield deformation occurred at approximately 1% drift when existing cracks increased in size and new inclined cracks were developed. Shear cracks gradually developed and with the crack widths increased in subsequent cycles of loading. Deterioration of the column lateral strength was observed under cyclic loading. Lateral strength of column specimens dropped by 20% at a drift limit of approximately 2.7%. Bar buckling in column specimen S1 (due to spalling of concrete cover at the critical section) resulted in the sudden loss of the column gravitational load carrying capacity. It is concluded that column S1 failed in flexural compression given that column failure was initiated by the buckling of the compression reinforcement. In contrast, column S2 was able to sustain axial load following a 20% reduction in its lateral load resisting capacity. Such desirable behaviour is associated with the uniform distribution of shear cracks (as opposed to the localized spalling of concrete) in the column as shown in Figure 3b. Due to the influence of shear, the column is able to undergo more displacement before the compression strains reach a value sufficient to cause spalling of the cover concrete. Finally, at a drift of 3.6%, shear failure associated with the opening of diagonal cracks and buckling of longitudinal bars could be observed just before the column lost its axial load carrying capacity. Column S2 eventually failed in compression flexural shear. Figure 4 contains photos of the test columns when their axial load carrying capacities have been lost.

Curvature distribution along the column height is shown in Figure 3 along with the observed crack patterns. In specimens 1 and 2, combined shear and flexure stresses resulted in diagonal tension cracking. Crack angle (θ) tends to be steeper when the column is subjected to higher shear forces. At a certain drift limit, pronounced diagonal cracking in column S2 was observed over a wider zone compared to cracks formed in column S1. The comparison between curvature observed from the VMS technique and curvature calculated from bending moment at a section showed good agreement for column S1 but disagreement for column S2. This is because diagonal tension cracks in plastic hinge zones increase the available plastic rotation by spreading the zone of yielding along member [Park and Paulay, 1975]

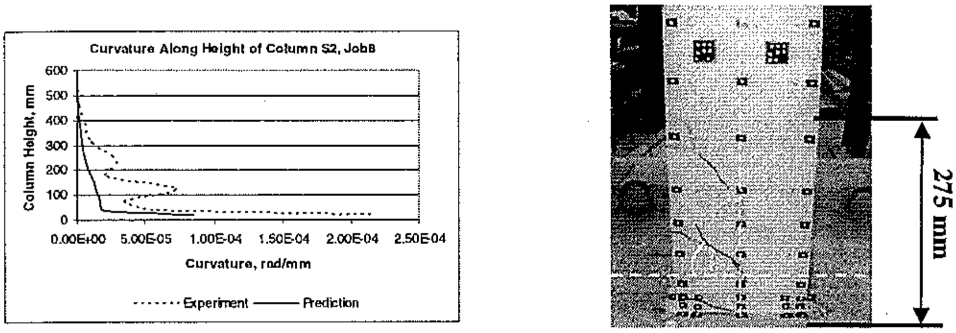
The VMS technique was employed to resolve deformation into the flexural, yield penetration and shear components. The deformation patterns in Figure 1a were used to interpret data obtained using VMS techniques (refer to Table1). From this table, it is clear that flexural deformation and yield penetration make a significant contribution to the total deformation of the column. Shear deformations calculated from the prescribed pattern were found to be approximately 5% and 10% of the total deformation for column S1 and S2 respectively. However, as explained previously, diagonal cracks in column S2 increased the available plastic rotation above the level predicted by flexural theory. There were difficulties in accurately accounting for additional deformation due to the effect of shear cracking.

4. THEORETICAL PREDICTIONS

The envelope curves of hysteretic force-deformation behaviour of column specimens were constructed by plotting lateral shear force against the summation of deformation components comprising flexural deformation, yield penetration and shear deformation. Flexural deformation can be estimated by integrating curvatures that have been calculated in accordance with representative stress-strain relationships of concrete and steel [Watson et al., 1994] assuming plane sections remain plane. The effects of yield penetration in the column longitudinal reinforcement at the anchorage to the foundation were estimated in accordance with the recommendations by Alsiwat and Saatcioglu [1992].



(a) Column S1 at 1.3 % drifts.



(b) Column S2 at 1.2 % drifts.

Figure 3 Curvature distributions along column height and their crack patterns

(a) Column S1

(b) Column S2

Figure 4 Column specimens after loss of axial load carrying capacity

Shear deformation was calculated using the truss analogy method which is consistent with behaviour of a cracked section [Park and Paulay, 1975]. Linear elastic behaviour of the concrete "strut" and the steel (stirrups) "ties" is assumed in the method. This led to an over-prediction of shear deformation in non-ductile columns. An alternative method is to use modified compression field theory (MCFT) [Vecchio and Collin, 1986] in which each concrete crack is treated as an element in its own right and with its own characteristics (i.e. equilibrium and constitutive relationships formulated in terms of the average stresses and strains). In this study, the MCFT was used to predict shear deformation of the column specimens. It was found that shear deformations predicted by the MCFT were in broad agreement with experimental results.

Table 1 Deformation components of column specimens

		Deformation Components Obtained From VMS								**Predictions**	
	Half Cycle	**Total Def.**		**Flexural Def.**		**Yield Penetration**		**Shear Def.**		***Total Def.***	***Shear Def.(MCFT)***
		(mm)	**%Drift**	**(mm)**	**(%)**	**(mm)**	**(%)**	**(mm)**	**(%)**	***(mm)***	***(mm)***
Specimen S1	3	**5.06**	*0.67*	**3.9**	*77.19*	**0.98**	*19.35*	**0.17**	*3.46*	***4.47***	***0.17***
	4	**4.86**	*0.65*	**3.71**	*76.35*	**0.94**	*19.25*	**0.21**	*4.40*	***4.47***	***0.17***
	7	**9.90**	*1.32*	**8.05**	*81.36*	**1.43**	*14.40*	**0.42**	*4.24*	***9.30***	***0.30***
	8	**9.89**	*1.31*	**7.99**	*81.09*	**1.35**	*13.75*	**0.51**	*5.17*	***9.30***	***0.30***
Specimen S2	3	**3.05**	*0.55*	**2.25**	*73.82*	**0.55**	*17.95*	**0.25**	*8.23*	***2.27***	***0.17***
	4	**2.75**	*0.5*	**1.89**	*68.72*	**0.69**	*25.00*	**0.17**	*6.28*	***2.27***	***0.17***
	7	**6.53**	*1.19*	**4.64**	*70.97*	**1.26**	*19.35*	**0.63**	*9.68*	***4.67***	***0.47***
	8	**5.74**	*1.05*	**3.82**	*66.63*	**1.37**	*23.84*	**0.55**	*9.53*	***4.67***	***0.47***

The aforementioned deformation model have been used to construct envelope curves for hysteretic force-deformation behaviour of the columns (refer to Figure 2). The model is shown to provide good predictions for the ascending curve for column S1 but some discrepancies between experimental results and experimental predictions are evident for column S2. This is explained by the additional contributions to rotation of the column by shear cracking. It has also been found that the model over-predicts post-peak deformation.

Whilst column deformation could be increased by shear cracking, the available column ductility could be compromised by shear strength degradation. In this study, shear strength of column specimens have been checked against recommendations by ATC-40 [1996], Priestley et al. [1994] and Moehle et al. [2002]. Both columns are not expected to fail in brittle shear mode but there is a possibility that shear strength degradation may lead to shear failure in column S2.

The model based on shear-friction theory developed by Moehle et al. [2002] has also been used to estimate the ultimate limit of deformation (at which the axial load carrying capacity of the column will subside). This model for predicting limiting drift is only valid for columns that are expected to fail in shear (i.e. column S2). Using this model, the limiting drift was found to be 2% based on a crack angle of 65 degree whereas the limiting drift is approximately 3.6% when the crack angle varies between 35 and 65 degree. It is evident that this model is overly conservative in predicting limiting drift. Furthermore, the prediction is very sensitive to the assumed crack angle (which is very sensitive to changes in height within the column).

5. CLOSING REMARKS

Results from experimental investigation into the cyclic behaviour of half-scaled reinforced concrete columns have been reported. For both columns, deformation at yield and 20% loss of lateral strength was found to be 1% and 2.7% drift respectively. Compression bar buckling has resulted in the axial failure of column specimen S1 at a drift limit of 2.7%. In contrast, column specimen S2 was able to maintain the full axial load up to a drift limit of 3.6% (at which the column failed by flexural shear). The test revealed that additional rotation at shear cracks increased deformation capacity of columns. The ability of columns to sustain axial load at large deformations was improved due to this effect. The deformation model provided satisfactory predictions for the ascending envelope curve but over-predicted post-peak deformation. An analytical model that can accurately and reliably estimate the limiting drift of a column (at the threshold of loss in axial load carrying capacity) has yet to be developed. Further column tests with different detailing and shear span-to-depth ratios will be undertaken to validate the model. Subsequently, the model will be part of the displacement-based methodology that can be used in predicting the performance of soft-storey buildings in regions of low to moderate seismicity.

6. ACKNOWLEGMENTS

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