

EXPERIMENTAL BEHAVIOUR OF DUCTILE HOLLOW
REINFORCED CONCRETE COLUMNS

J.B. Mander (I)

SUMMARY

This paper presents an experimental investigation into the seismic performance of ductile hollow reinforced concrete columns. Four 3.2 m high specimens, 750 mm square with a 120 mm wall thickness containing 60 longitudinal steel bars and different arrangements of confining steel in the plastic hinge zone were subjected to a constant axial load and cyclic lateral displacements of increasing ductility amplitudes. Conclusions are made regarding the applicability of the New Zealand Concrete Design Code (Ref. 1) when detailing the plastic hinge regions of hollow concrete columns.

INTRODUCTION

The construction of tall bridge piers using rectangular hollow reinforced concrete columns is an attractive means by which the superstructure weight, and hence seismic loading, is minimised. The column strength and stiffness is maintained whilst significantly reducing the construction cost. There is currently a reluctance amongst bridge designers to specify the use of ductile hollow columns for tall bridge piers, due to the unknown performance of the plastic hinge regions under severe seismic disturbances.

The seismic resistance of bridge structures is normally achieved by designing the piers for both strength and ductility. Ductile piers dissipate seismic energy by the formation of plastic hinges. The ductility capacity of plastic hinges should be greater than the ductility demand under the design ground motions. In New Zealand a structure ductility factor of six is typically required for bridge piers. The ductility capability of reinforced concrete members is achieved by providing sufficient transverse reinforcement in the form of rectangular hoops or spirals to adequately confine the concrete, to prevent buckling of the longitudinal reinforcement, and to prevent shear failures.

Solid reinforced concrete elements detailed in accordance with the provisions of the New Zealand Concrete Design Code, NZS 3101, should be capable of sustaining member ductilities of at least eight (Ref. 1). However, hollow columns under seismic loading can lose a significant proportion of their cross section when the cover concrete on the outside and inside faces spall. Hence careful detailing of the core concrete is necessary to maintain flexural strength at high levels of ductility. This paper describes an experimental research programme in which the applicability of the NZS 3101 code provisions for the detailing of rectangular hollow reinforced concrete columns was investigated. Full details of this work summarised herein may be seen elsewhere (Ref. 2).

(I) Former Graduate Student, University of Canterbury, Christchurch,
New Zealand.

PRELIMINARY COMPRESSION TESTING

Firstly, 16 one-half full size wall specimens, that modelled the flange portion of a 1900 mm square hollow column with a 300 mm wall thickness, were constructed. Each wall had a section width of 150 mm and length of 700 mm, and the overall height was 1200 mm. The wall specimens were subjected to a concentric axial compression load to failure (hoop fracture) at either low or high strain rates. The specimens were tested in a 10 MN capacity DARTEC Universal Testing Machine, and a support frame was used to prevent out-of-plane (lateral) buckling of the walls. The principal variable in this preliminary test series was the amount of transverse confining reinforcement (ρ_s). Different volume ratios of transverse reinforcement ρ_s , were tested in order to cover a typical range of columns confined in accordance with Ref. 1.

Fig. 1 presents the test results of Walls 5, 6 and 8, each specimen being cast from the same batch of concrete ($f'_c = 28$ MPa), and tested at the low strain rate ($\dot{\epsilon} = 0.0001/\text{sec}$). Wall 8 was a plain concrete (unreinforced) specimen which had a compressive strength of 26 MPa. Wall 5 had a minimum amount of transverse steel ($\rho_s = 0.0162$) which was governed by the antibuckling requirement of Ref. 1 that the hoop set spacing should not exceed six longitudinal bar diameters. Wall 6 had a maximum amount of transverse steel ($\rho_s = 0.0708$), typical of prototype columns with the maximum level of axial load permitted by Ref. 1. (i.e. $P_e = 0.7f'_c A_c$).

From the test results presented in Fig. 1, it can be seen that due to the presence of the confining reinforcement, the strength of the core concrete f'_c , was increased by 32% and 100% above the control cylinder strength f'_c , for the low and high levels of transverse reinforcement, respectively. Also due to confinement, the maximum useable strain in the concrete core was increased from about 0.004 (see the unreinforced Wall 8), to the respective hoop fracture strains (ϵ_{cu}) of 0.028 and 0.055 for the low and high levels of transverse reinforcement. Thus it is evident that the energy dissipation capability of confined concrete, represented by the area beneath the stress-strain graph, is markedly increased due to confinement of the concrete.

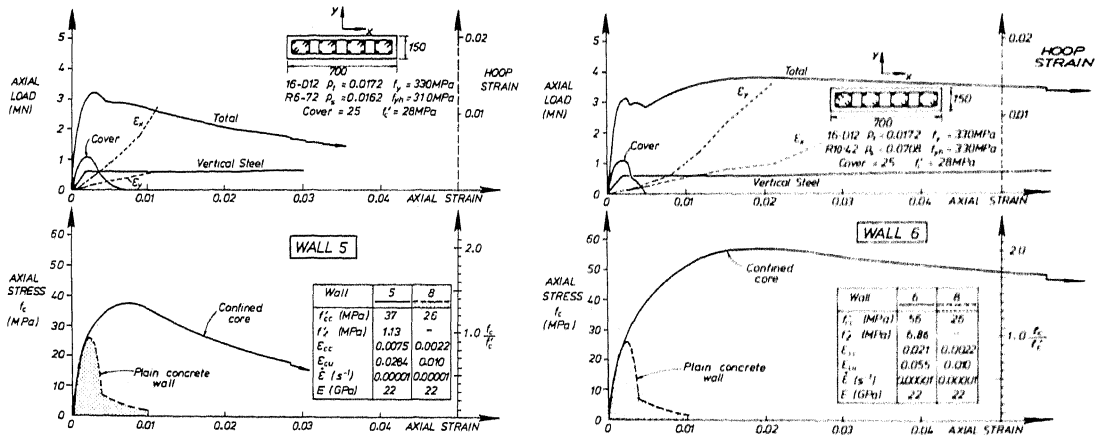


FIG. 1 RESULTS OF THE 1200 MM HIGH WALL TESTS.

DESCRIPTION OF THE HOLLOW COLUMN SPECIMENS

Four hollow column specimens, designated Columns A, B, C and D, were constructed. The specimens were 40% scale models of a 1900 mm prototype square hollow column with 300 mm thick walls. The dimensions of the test specimens are shown in Fig. 2. Each specimen had an overall height of 4.1 m, a 750 mm square hollow cross section with a 120 mm wall thickness. The choice of a relatively large-scale test specimen meant that ordinary reinforcing bars with normal (unscaled) concrete could be used in the construction. The longitudinal reinforcement consisted of sixty 10 mm diameter mild steel deformed bars, equally spaced about the external and internal faces of the hollow section giving a steel ratio of 0.0155. The transverse hoop steel was from 6 mm diameter mild steel plain round bars.

Normal weight ordinary Portland cement concrete was used, and the measured compressive strengths at the time of testing were $f'_c = 30$ MPa for Columns A and B, and $f'_c = 29$ MPa for Columns C and D. The measured yield strengths of the longitudinal and transverse reinforcement were $f_y = 335$ MPa and $f_{yh} = 320$ MPa, respectively.

The main variables during the tests were the level of axial load applied to the specimen, P_e , and the corresponding amount of transverse reinforcement present in the plastic hinge region. The applied axial load level, as expressed by the ratio $P_e / f'_c A_g$, where A_g is the gross area of concrete section, was 0.1, 0.5, 0.3 and 0.3 for Columns A, B, C and D respectively. The plastic hinge region in the hollow column was located from 0 to 750 mm above the solid concrete base, and the centre-to-centre spacing of the hoop sets was 60 mm, 30 mm, 40 mm and 60 mm for Columns A, B, C and D, respectively. Outside the plastic hinge zone the hoop set spacing was 120 mm for each column specimen.

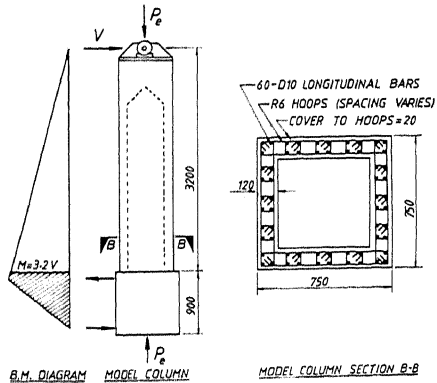


FIG. 2 HOLLOW COLUMN SPECIMENS

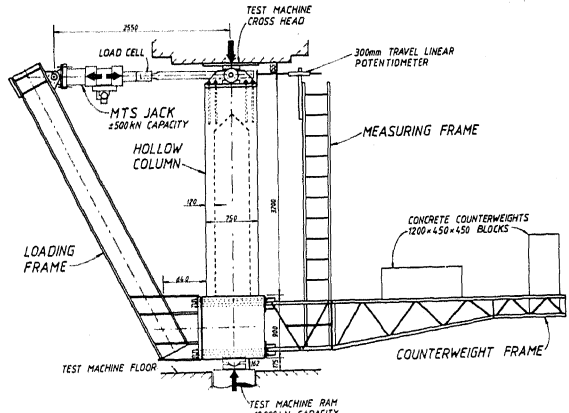


FIG. 3 TEST RIG SET UP.

TESTING OF THE HOLLOW COLUMN SPECIMENS

The general set up of the test rig and hollow columns is illustrated in Fig. 3. The 10 MN capacity DARTEC universal testing machine was used to apply the compressive axial load to the column specimen. The loading frame was bolted to the solid concrete base of the test unit. A 500 kN capacity double acting MTS servo-hydraulic loading jack was used to apply a lateral load at the top of the cantilever arm of the loading frame and at the top of the hollow column. Hence, bending moment, shear force and axial load were applied to the member, with the critical section being just located above the base. The jack displacements could be applied cyclically to reverse the direction of the lateral load and hence reverse the moment and shear.

An initial lateral load cycle to approximately 75% of the ideal column strength was applied in both the positive and negative directions. From the resulting lateral load-displacement graph an experimental value for the yield displacement, Δ_y , was obtained by extrapolating a straight line from the origin through the peak lateral load-displacement coordinate to the lateral load at ideal strength, $V_i = 3.2 M_i$. Note the ideal moment capacity of the hollow section M_i was calculated using the NZS 3101 approach where an ultimate concrete strain of 0.003, together with a rectangular concrete stress block are assumed (Ref. 1). Subsequent testing, in a quasi-static fashion, consisted of two complete cycles to displacement ductility factors each of $\mu = \pm 2, \pm 4, \pm 6$ and ± 8 , unless premature failure of the specimen caused a halt to the test programme.

RESULTS OF THE HOLLOW COLUMN TESTS

Lateral load-displacement hysteresis loops are presented in Fig. 4 for each hollow column specimen. Superimposed on the measured lateral load-displacement curves of these graphs is the ideal lateral load capacity of the specimen plotted as a dashed line. Note that the lines drop as the displacement increases owing to the secondary P-delta moment effect of the axial load. Where the axial load level is high (e.g. Column B) the secondary moments from the P-delta effect become more significant.

Column A:
$$P_e = 0.1f'_c A_g$$

This specimen was provided with a minimum amount of transverse reinforcing steel in the plastic hinge zone. The amount was governed by the spacing of transverse steel required to prevent buckling of the longitudinal steel, and a spacing of six longitudinal bar diameters was used as recommended by Ref. 1.

The low level of axial load applied to the specimen throughout testing was intended to represent an average level of load expected on a prototype pier. At the completion of the testing at $\mu = \pm 8$, the cover concrete had spalled over a height of 150 mm above the solid concrete base, and buckling of the four external corner longitudinal bars had commenced. Nevertheless, the lateral load at $\mu = \pm 8$ was still 10% in excess of the ideal strength, and there was little degradation of column strength and stiffness due to cyclic loading.

Column B: $P_e = 0.5f'_c A_g$

For the design and detailing of plastic hinge regions in columns, the length of the region over which the closely spaced hoops are placed is normally taken as the largest section dimension or from where the moment exceeds 80% of the maximum moment at the end of the member, whichever is greater (Ref. 1). When the axial load P_e is greater than $0.3f'_c A_g$, Ref. 1 requires the potential plastic hinge length to be increased by 50%.^g Hence the length of the confined concrete should have been not less than $1.5 \times 750 = 1125$ mm for Column B. However the length of the confined hinge zone of Column B was only 750 mm, that is, one section width. Consequently this specimen suffered a premature failure, resulting from a spread of compression steel yield above the extent of confinement due to the high strength of the confined hinge region. Failure was due to longitudinal bar buckling just above the confined hinge region, 800 mm above the column base where the transverse hoop spacing was 12 longitudinal bar diameters. This test clearly demonstrates the need to increase the length of confinement and anti-buckling restraint of the longitudinal bars in columns with high levels of axial load.

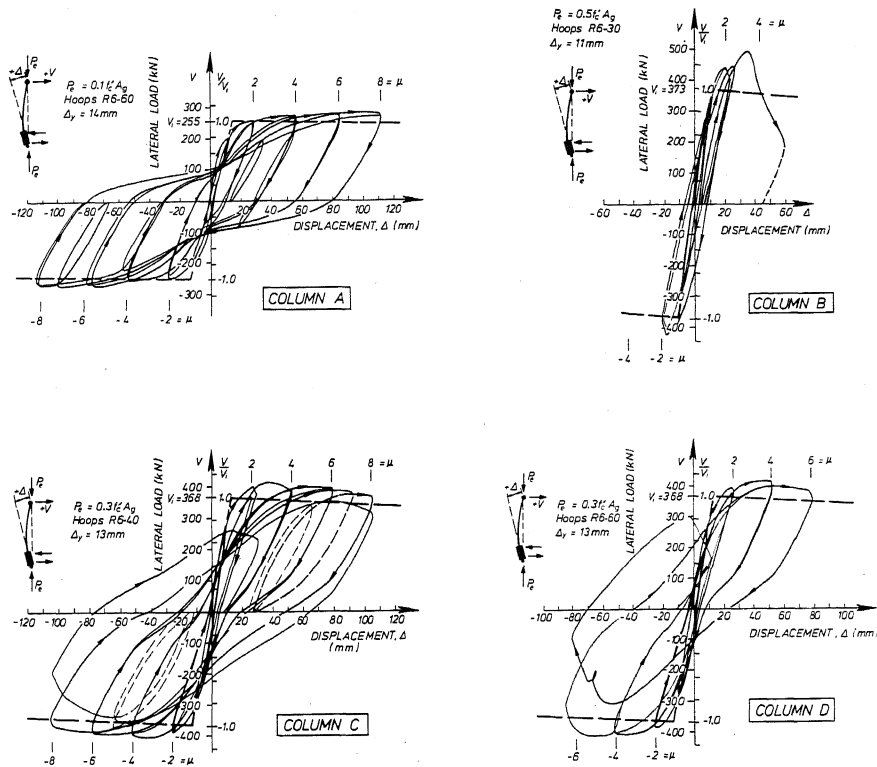


FIG. 4 EXPERIMENTAL LATERAL LOAD-DISPLACEMENT GRAPHS FOR THE HOLLOW COLUMNS

Columns C and D: $P_e = 0.3f'_c A_c g$

The intensity of axial load applied to Columns C and D was intended to represent the maximum level expected in the design of hollow columns for prototype bridge piers. To examine the effect of confinement on member ductility, different amounts of transverse reinforcement were used in these two specimens. The respective quantities of transverse hoop steel present in the plastic hinge regions of Columns C and D were 83% and 55% of the amount required by NZS 3101 for full ductility (Ref. 1). As in Column A, the maximum allowable centre-to-centre hoops spacing of six longitudinal bar diameters was used for Column D. Both Columns C and D performed well under cyclic loading with the lateral load between 10% and 15% in excess of the ideal strength. Column C eventually suffered a confinement failure when a number of the small transverse flange hoops fractured on the final load cycle at $\mu = -8$. Column D behaved in a similar fashion to Column C up to $\mu = \pm 4$. However, due to the reduced level of confinement, the failure of Column D occurred earlier than for Column C. Fracture of some of the small transverse flange hoops occurred as the displacement was increased from $\mu = -5.8$ to $\mu = -6.5$.

General Deflection Behaviour

In general, the hysteresis loops show very good energy dissipation characteristics and stability. With increasing displacement ductility some stiffness degradation of the unloading curves is apparent. Very little degradation of strength occurred on the second complete cycle at constant ductility factors.

Some "pinching" of the load-displacement hysteresis loops is apparent for Columns C and D and particularly for Column A. Pinching is a characteristic of hysteresis loops for columns with low to medium levels of axial load. Pinching is due to the compressive axial load causing the closure of open cracks in the compressed flange. The open cracks remain open from the tensile extension of the steel which occurred from the cycle of loading in the previous direction. Note the absence of pinching for Column B which had the high level of axial load.

ANALYSIS OF THE HOLLOW COLUMN TEST RESULTS

In this section the results of Columns A, C and D are presented in Table 1 and discussed. Results of Column B have been ignored due to the nature of the premature failure.

TABLE 1 : EXPERIMENTAL RESULTS

Column	$\frac{P_e}{f'_c A_c g}$	$\frac{A_{sh} \text{ (provided)}}{A_{sh} \text{ (code)}}$	μ	$\frac{L_p}{h}$	θ_p (radians)	Spalling Strains	Max. Core Strain
A	0.1	0.74	8+	0.37	0.032+	0.008	0.029
C	0.3	0.83	8	0.40	0.030	0.009	0.050
D	0.3	0.55	6	0.41	0.021	0.009	0.028

Equivalent Plastic Hinge Lengths (L_p)

Equivalent plastic hinge lengths were calculated from the maximum experimental curvatures measured in the hollow column adjacent to the solid base. The non-dimensional plastic hinge lengths L_p/h (where h is the column width) listed in Table 1 were calculated for the final load cycle. It appears that the average equivalent plastic hinge length can be taken as about 0.4 of the column width.

Plastic Hinge Rotation (θ_p)

Also listed in Table 1 are values for the experimental plastic hinge rotation which was determined either at the end of quasi-static testing or at first fracture of the flange hoops. From these results it was found that the attainable rotational capacity of column hinges will not be less than

$$\theta_p = 0.035 A_{sh}(\text{provided})/A_{sh}(\text{code}) \text{ radians} \quad (1)$$

This equation implies that a rotational capacity of at least 0.035 radians is available for column hinges with standard Code (Ref. 1) detailing. Thus for the design of ductile structures, Eq. 1 could be used to indicate the available plastic hinge rotation of column hinges.

Flange Strains

Constant visual examination of the specimens during the early stages of testing enabled identification of the strain at which spalling of the cover concrete first occurred. The results tabulated in Table 1 show that the minimum spalling strain of 0.008 is well in excess of the value of 0.003 generally used for ultimate strength calculations, and also in excess of 0.004, which is the value often assumed in many moment-curvature analyses.

Table 1 also lists the maximum concrete compression strains at the edge of the confined concrete core, which were measured either at the end of testing (Column A) or just prior to first hoop fracture (Columns C and D). From these results it is evident that the transverse hoops effectively confined the core concrete enabling compressive concrete strains many times greater than the spalling strain of unconfined concrete to be achieved.

CONCLUSIONS

From the results of the reinforced concrete specimens tested in the series the following conclusions are listed:

1. It appears reasonable to use the New Zealand Concrete Design Code (Ref. 1) to detail the flanges of hollow columns by applying the transverse reinforcement provisions in the same manner as for solid column members. The tests indicated that member ductilities of at least eight can be achieved with standard Code detailing.
2. The tests indicated that the full quantity of hoop steel recommended by the Code may be excessive if only limited ductility ($1 < \mu < 8$) is required. Thus the use of a reduced quantity of confining reinforcement

appears justified when only limited ductility is required. The available plastic hinge rotation could be taken as

$$\theta_p = 0.035 A_{sh}(\text{provided})/A_{sh}(\text{code}) \text{ radians}$$

3. The hysteretic performance of the hollow columns tested herein demonstrated very good energy dissipation characteristics. Little sign of strength degradation was shown prior to column failure by fracturing of the transverse hoops in the flanges.

4. For the purposes of ductility calculations it appears that the equivalent plastic hinge length for hollow columns with a height to depth aspect ratio of about four can be taken as 40% of the column depth.

ACKNOWLEDGEMENT

This research work was carried out by the author at the University of Canterbury as part of a Ph.D. project and was supervised by Dr. M.J.N. Priestley and Professor R. Park.

REFERENCES

1. NZS 3101, "The Design of Concrete Structures", Part 1 : Code of Practice, Part 2 : Commentary, Standards Association of New Zealand, Wellington, New Zealand, 1982.
2. MANDER, J.B., "Seismic Design of Bridge Piers", Doctor of Philosophy Thesis, University of Canterbury, Christchurch, New Zealand, 1983.

NOTATION

- A_g = gross area of concrete section
 A_{sh} = area of rectangular hoops plus supplementary cross ties
 f'_c = compressive strength of concrete control cylinders
 f_y = yield strength of longitudinal reinforcement
 f_{yh} = yield strength of transverse reinforcement
 M_i = ideal flexural strength of section
 P_e = compression load on column
 V = lateral load or shear force applied to the column
 V_i = ideal lateral load capacity (when bending moment = M_i)
 Δ = lateral displacement of column
 Δ_y = yield displacement measured during testing
 μ = displacement ductility factor of column = Δ/Δ_y
 ρ_s = ratio of hoop reinforcement to volume of concrete core