STRENGTH AND DUCTILITY OF CIRCULAR HOLLOW COLUMNS WITHOUT INSIDE CONFINEMENT UNDER CYCLIC LOADING

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The flexural behavior of two circular hollow reinforced concrete columns, with different longitudinal reinforcement ratios, under cyclic loading is investigated through a discussion of experimental studies. Test results showed that the columns were significantly failed when the inside concrete was crushed. It was also found that confinement-induced transverse strain at the plastic hinge region did not reach the yield strain before failure. Furthermore, moment-curvature analyses were carried out for the test units, to evaluate the lateral force-displacement response. The stress-strain curve of the confined concrete was calculated with an assumption that the maximum effective lateral pressure occurred when the spiral strain reached a maximum useful confinement-induced strain of 0.001. As a result, the theoretical force-displacement relation showed a good agreement with the experimental response.

Key Words *: Hollow Columns, Flexural Behavior, Cyclic Loading Tests, Confinement, Ductility*

1. INTRODUCTION

A hollow concrete section is often used for column design, particularly for very tall bridge columns in seismic areas including California, New Zealand, Japan and Italy et al., for reducing the mass and therefore minimizing the self-weight contribution to the inertial mode of vibration during an earthquake. The hollow columns also enable to reduce foundation dimensions and thus save the construction cost substantially. In the hollow circular section for bridge columns, some layers of longitudinal and transverse steel are placed near both outside and inside faces and they are tied through the wall thickness by cross ties. However, the transverse steel placed near inside face and the cross ties may not significantly contribute to the confinement of concrete wall in the hollow section. This report presents the results of experimental studies conducted to establish the parameters controlling the available ductility of the hollow columns with one layer of steel placed only near inside face of the section.

2. DETAILS OF LOADING TESTS

The overall test setup is illustrated in Fig. 1. Column section up to 3480mm height from the base was constructed by reinforced concrete with hollow section, and a loading steel tube was connected to the top of the column for the extension of the column height. Two columns (called herein HF1 and HF2) with different longitudinal reinforcement ratios were constructed and tested in this program. The test columns have 34 bundles of 2 #4 bars (HF1) or #6 bars (HF2) in one layer distributed evenly with a constant cover. The longitudinal reinforcement ratios to the net area of the concrete are 1.4% (HF1) and 3.3% (HF2), respectively. The transverse reinforcement is a W5 wire (6.35mm diameter) spiral with 35mm pitch in the range of plastic end region. A vertical load was applied to the units through the use of four dywidag high strength bars and two rocker beam assemblies. A total vertical load of 2913kN (HF1) or 2997kN (HF2), corresponding to an axial load ratio ($P / f_c' A_g$) of nearly 0.13, was applied and maintained throughout the test.

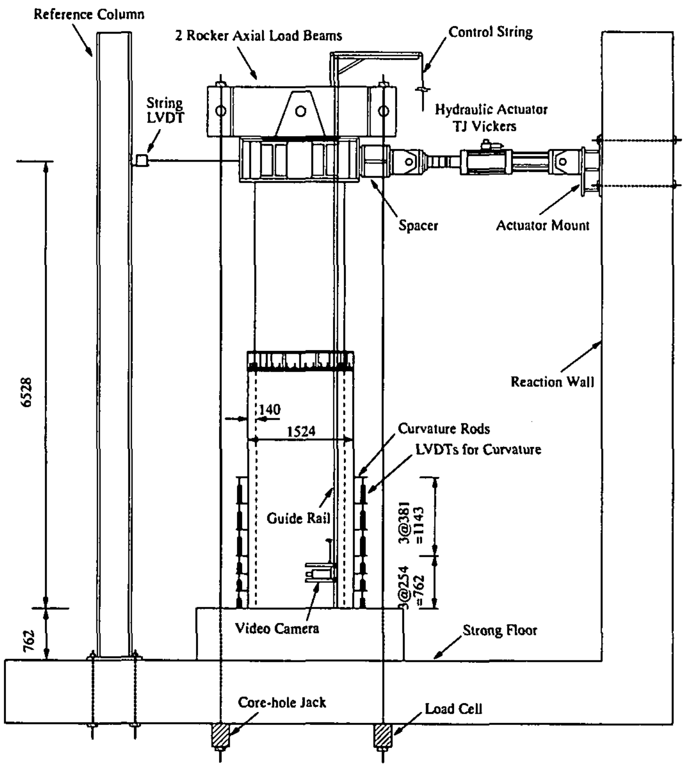
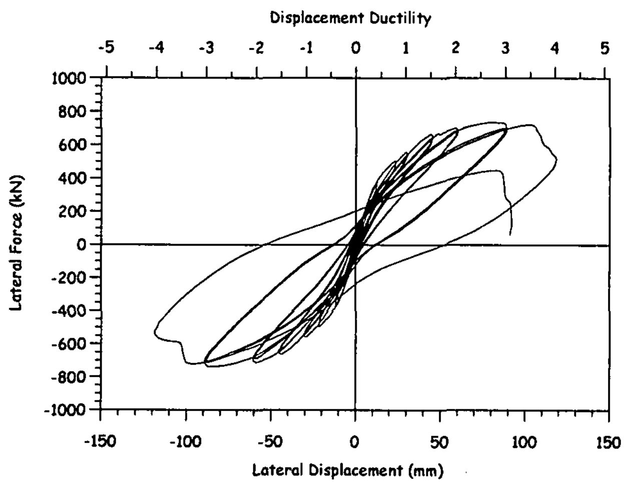


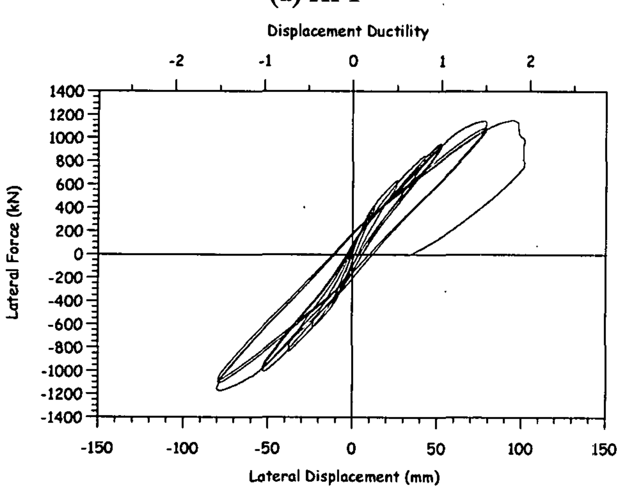
Fig. 1 Test Setup

3. TEST OBSERVATIONS AND STRUCTURAL RESPONSES

The lateral force-displacement hysteretic responses of the test unit are shown in Fig. 2. It is interesting that both columns exhibited the crush of the inside face concrete at the maximum useful ductility of 3.5 (HF1) or 1.8 (HF2), which caused the significant degradation of the lateral force. At the ductility 4.0 in the HF1 unit, even though no longitudinal reinforcement buckled and the core concrete seems to be still confined, the inside face concrete significantly spalled-off. Fig. 3 shows longitudinal strain profiles in the plane section of 146mm height form the column base where the significant plastic curvature developed. It is noted that the strains of near inside face concrete at the maximum useful ductility of 3.5 (HF1) and 1.8 (HF2) seem to be nearly 5000 microstrains, indicating that the ultimate compression strain of the inside face concrete may be at most 5000 micorstrains.

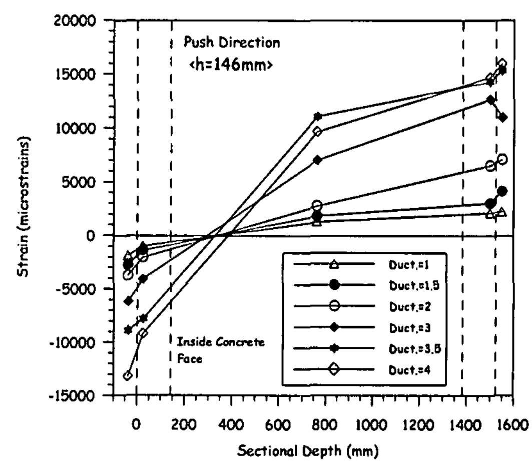


(a) HF1

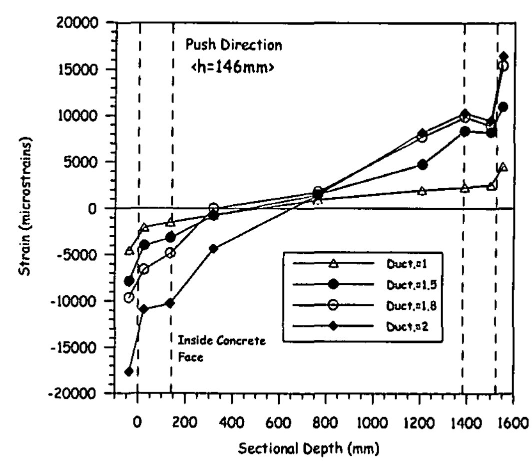


(b) HF2

Fig. 2 Lateral Force-displacement Response



(a) HF1



(b) HF2

Fig. 3 Longitudinal Strain Profiles in Section

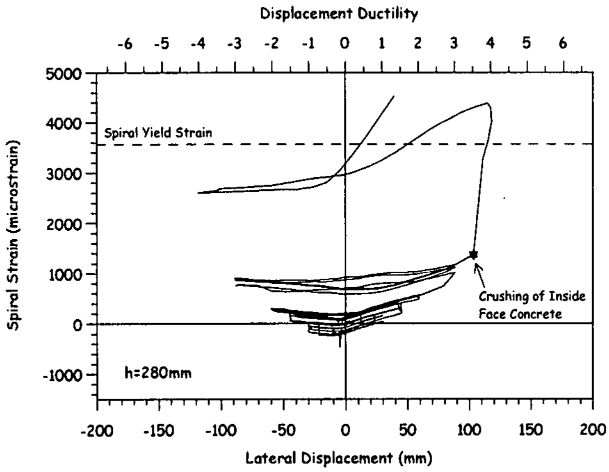


Fig. 4 Confinement-induced Strain Response

Fig. 4 shows the confinement-induced strain hysteresis response measured at 280mm height of the HF1 unit. The spiral strain has reached about 1380 microstrains at the interesting ductility 3.5 and the spiral steel has yielded immediately after crushing of the inside face concrete. Similar strain behaviors were observed at other sections within the plastic hinge. It should be remarked that the confinement-induced strains at the maximum useful ductility seems to be around 1000 microstrains averaged over the anticipated plastic hinge region and therefore still do not reach their yield strains in a moment of crushing of the inside face concrete. Based on the test results from the units of HF1 and HF2, the strain of 0.001 would represent the effective confinement-induced strain in the circular hollow section.

4. ANALYSES OF COLUMN RESPONSES

The transverse steel placed near outside face in the plastic hinge region activates in tension when the section performs in a ductile manner in the inelastic range, and the resulting steel stress then applies the confinement pressure to the concrete core. It should be noted that the concrete wall in the circular hollow section is subjected to biaxial compression in such situation as illustrated in Fig. 5, since the confinement pressure arises in the only circumferential direction, other than in the radial direction. It is known that the biaxial compression applying to the core results in poorer confinement than the triaxial compression [1]. Accordingly, this confinement loss should be evaluated in the analytical considerations.

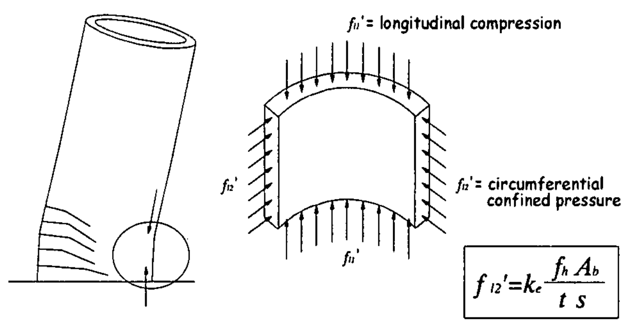


Fig. 5 Confinement in Circular Hollow Section

The flexural response was estimated based on a conventional monotonic moment-curvature analysis, where the effect of the biaxial compression can be estimated through a stress-strain curve of the confined concrete proposed by Mander, Priestley and Park [1]. In the model, the confinement effect is represented by the confined strength ratio f_{cc}'/f_c', where f_{cc}' = compressive strength of confined concrete and f_c' = compressive strength of unconfined concrete. The circumferential confining stress f_{l2}' in the hollow section is written as

$$f_{l2}' = k_e \frac{f_h A_b}{t\ s} \tag{1}$$

where, k_e = a confinement effectiveness coefficient relating the minimum area of the effectively confined core to the nominal core area; f_h = confinement-induced transverse steel stress; A_b = area of transverse steel; t = concrete wall thickness inside the transverse steel; and s = spacing of transverse steel. An appropriate value of k_e for the hollow section may be 0.6 because it is recommended for wall sections. [1] As described, test results showed that the transverse steel strain induced by confinement was nearly 1000 microstrains averaged over the plastic hinge region before column failure, the confinement-induced transverse steel stress f_h can be then estimated as (175000 MPa × 0.001) = 175 MPa.

To establish the stress-strain curve of concrete and thus determine the ultimate compression strain, an equivalent solid circular section is assumed in terms of the confinement effect, that is, the confined strength ratio. The effective lateral pressure in the equivalent solid circular section (f_l') was read out in accordance with the confinement effect chart as shown in Fig. 6 [1], which notes the procedure for obtaining the effective lateral pressure f_l'.

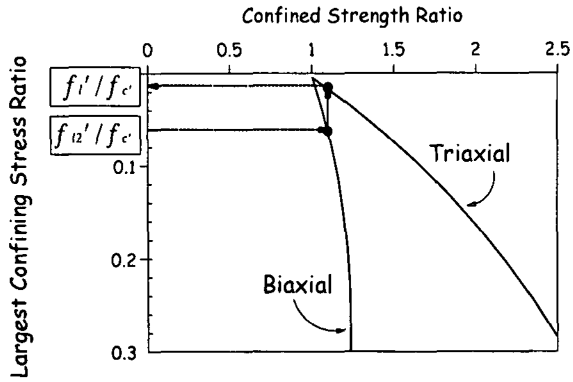


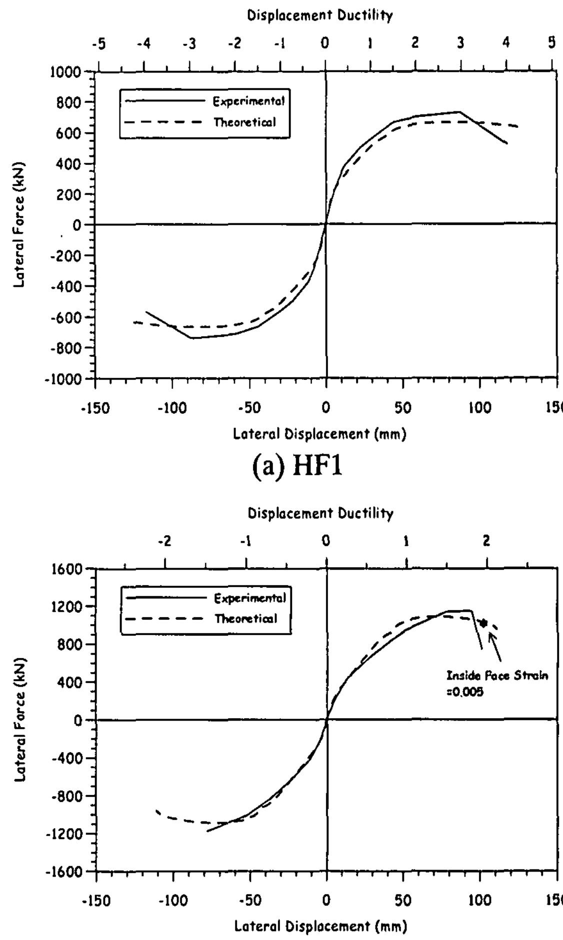
Fig. 6 Equivalent Confined Pressure

Moment-curvature analyses were carried out for critical sections and theoretical lateral force-displacement responses were calculated based on the idealized curvature profile, where deformation developed in the loading steel tube was taken into account. The plastic hinge length L_p was assumed here as [2]

$$L_p = 0.08L + 0.022 f_y d_{bl} \qquad (2)$$

where f_y = yield strength of longitudinal reinforcement (MPa) and d_{bl} = diameter of longitudinal reinforcement.

The measured envelopes of the lateral force-displacement responses are compared with theoretical results in Fig. 7. The experimental responses show good agreements with the theoretical results. It is also interesting in the analytical result of the HF2 that the inside face compression strain exceeded 5000 microstrains before the outside face strain reached the ultimate strain. This predicts that crushing of the inside face concrete occurs before failure of the outside face concrete, which coincided well with the test observation of the HF2.



(a) HF1

(b) HF2

Fig. 7 Comparison of Measured and Analytical Lateral Force-displacement Envelops

It should be also noted that the confinement-induced strain was nearly 0.001 averaged over the plastic hinge region and did not exceed its yield strain before column failure.

5. CONCLUSIONS

The inside face concrete compression strain is one of the most important parameters to control the ductility capacity of the hollow columns. Test results showed that the inside face concrete was crushed when the compression strain reached nearly 0.005 even though a sufficient amount of transverse steel was placed near outside face.

The flexural strength and the ductility capacity of the circular hollow columns were evaluated with use of the conventional moment-curvature analyses, where the confinement effect in the hollow section was converted to that in the equivalent solid circular section. Ultimate compression strains of concrete were defined at both outside and inside faces of the section, to estimate the response at ultimate limit state. It was noted that analytical results coincided well with experimental force-displacement behavior.

References

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