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Seismic Behaviour of Columns Made of High-Strength Concrete

F. Légeron[1] and P. Paultre[2]

ABSTRACT

Large scale reversed cyclic flexural loading of six high-strength concrete (HSC) columns is presented. The columns were subjected to constant axial loads corresponding to 15, 25 and 40 % of the columns axial load capacity. It is shown that tie spacing and therefore tie volumetric ratio and axial load level have significant effects on the flexural behaviour of HSC columns. The need to include the level of axial load in code requirements for confinement reinforcement is pointed out. Different stress-strain proposed models are used to predict the moment-curvature responses of the specimens and are compared with test results.

INTRODUCTION

Structural engineers are still reluctant towards the use of high-strength concrete (HSC) in seismic areas because of its more fragile behaviour as opposed to normal-strength concrete (NSC). This can be explained by the limited number of cyclic flexural tests of columns under significant axial compression loading. Nevertheless, some authors (Bing, 1994 and Azizinamini *et al.*, 1994) have shown that HSC columns can present ductile behaviour under flexion and axial loads when sufficiently confined with transverse ties. Data on ductility of HSC columns with axial load in the range of 20 to 40 % of column axial load capacity are limited. This paper presents the experimental results from six HSC columns which were subjected to significant constant axial loads (below, at about and above the balanced loads) and reversed cyclic flexure. Theoretical moment-curvature predictions from different stress-strain models are compared to experimental results.

EXPERIMENTAL PROGRAM AND INSTRUMENTATION

The specimens tested represent a ground-floor HSC column in a typical building. The specimens are 2.15 m high with a 305 × 305 mm cross section. The load is applied at 2.00 m from the bottom of the columns, and assuming the point of contraflexure to be located at about the middle of the column, they represent a 4.00 m column. The column is connected to a massive stub simulating a rigid member of a foundation (see Fig. 1). Figure 2 shows the reinforcement of a specimen with the tie configuration used.

The effects of axial load level and tie spacing were the two parameters studied in this research program. Three levels of axial load (15, 25 and 40 % of $A_g f'_c$), which for a specified concrete strength of 100 MPa, represent three points below, at about and above the balanced point. Two different tie spacings, 60 mm and 130 mm, were used. The 60 mm spacing was chosen to obtain ductile behaviour, and the 130 mm was controlled by shear design and minimum spacings. In this case, a 60 mm tie spacing represents a 4.26 % volumetric ratio, which is about 66 % of the volumetric ratio recommended in the ACI and CSA Codes for seismic design. A 130 mm spacing represents a 1.96 % volumetric ratio.

[1]Research Assistant, Dept. of Civil Eng., Université de Sherbrooke, Sherbrooke, Québec, J1K 2R1
[2]Professor, Dept. of Civil Eng., Université de Sherbrooke, Sherbrooke, Québec, J1K 2R1

Figure 1 shows the experimental setup. The axial load was applied with four high-strength 35 mm diameter Dywidag bars tensioned by two 1000 kN jacks and two 1500 kN jacks. Each Dywidag bar is instrumented with strain gauges and calibrated to give the actual tension in the bars. The transverse load was applied with two 100 kN MTS actuators with displacement and force control capabilities. The columns were subjected to reversed cyclic horizontal loading, while the axial load was maintained constant. The maximum transverse load in the first cycle reached 0.75 of the expected yield load. The second cycle reached the yield load. Thereafter, each cycle was under displacement control with the maximum displacement equal to 1.5, 2, 3, ..., times the yield displacement up to failure. Except for the first cycle, all subsequent cycles were repeated twice.

The longitudinal bars were instrumented with 16 electrical strain gauges above and in the stub (see Fig. 2). Four sets of ties just above the stub were instrumented with 16 electrical strain gauges. Curvatures were calculated from the strain gauges measurements for the first two specimens tested (C100B60N15 and C100B130N15). For the last four specimens, curvatures were also calculated from the readings of two sets of four LVDTs placed close to the concrete surface.

MATERIAL PROPERTIES

The specified 100 MPa concrete was mixed in the concrete laboratory at the University of Sherbrooke. The concrete formulation was based on a water-cement ratio of 0.25. One batch of 1000 kg was needed to cast about 75 % of the base stub. A second batch of 1000 kg was used to complete the base stub and the column. The same formwork was reused to cast all the specimens. The concrete compressive strength was determined from standard compressive tests on at least three 150 x 300 mm cylinders. Complete stress-strain curves were obtained from at least three 100 x 200 mm cylinders tested at a very slow rate in a very stiff machine. Actual average strength ranged from 92.4 to 104.3 MPa, as reported in Table 1. The Young modulus ranged from 34 300 to 41 000 MPa. The cracking strength of concrete was estimated from modulus of rupture tests on at least three 400 x 100 x 100 mm prisms for each specimen. The cracking strength of the specimens ranged from 7.8 to 9.0 MPa.

Deformed No. 15 and No. 20 Grade 400 were used for longitudinal reinforcement. Transverse ties were realized with deformed No. 10 Grade 400. All characteristic values such as yield stress, commencement of strain hardening, ultimate strain and stress were measured for the two batches of steel used in this research program. Yield stresses of No. 10 bars were 391 and 418 MPa, and the ultimate stresses were 637 and 675 MPa. No. 15 bars yielded at 494 and 467 MPa, and the ultimate stresses were 729 and 722 MPa. No. 20 bars yielded at 451 and 430 MPa, and the ultimate stresses were 716 and 661 MPa. These values represent the average of at least three tests.

TEST RESULTS

At the end of the test, all the columns were heavily damaged just above the base stub. As can be seen from Figure 3, the length of the damaged region increases with the axial load level. This is also confirmed by strain readings on the longitudinal bars. It is important to note that fracture occurred at a section 180 mm above the interface between the column and the base stub. Sheikh and Khoury (1993) and Bing (1994) have made the same observations and attributed this primarily to confinement provided by the base stub to the sections in its immediate vicinity, thereby increasing their moment capacity.

All experimental data were stored at predetermined steps and at special events such as cracking, yielding, peak loads, zero crossing, etc. Acquisition was performed by increments of force or displacement depending on the type of test control that was in effect. Figure 4 shows the uncorrected transverse load *vs.* tip displacement for all the specimens. Also reported on the graphs are the $P-\Delta$ effects. A strength gain is indicated when the response curve, in absolute term, is lying above the $P-\Delta$ line and a strength loss when the reponse curve is under the $P - \Delta$ line.

Table 1 summarizes the main results of the tests. The term M_{max} is calculated by multiplying the applied lateral load by two meters, the distance between the point of application of the load to the base of the column. As mentioned previously, failure occurred at a section about 180 mm away from the stub face. The moment at this section is defined as M'_{max}. μ_Δ is the structural ductility and μ_ϕ is the curvature ductility. These parameters are defined with respect to the moment at which the column resists 80 % of the maximum experienced moment. The corresponding compressive strain in the concrete is denoted ε_{c80u}. Structural ductility ranged from 1.6 for the specimen with low confinement and the larger axial load to 8.8 for the more confined specimen with the lower axial load level. Curvature ductility ranged from 3.3 to 26.9 for only four specimens. For the other two specimens, not instrumented with LVDTs, it was not possible to calculate the complete moment-curvature response from the strain gauges readings.

ANALYSIS OF RESULTS

From the results reported in Table 1 it can be seen that all columns exhibited a ductile behaviour with a structural ductility of at least 4.0, except for specimens C100B130N25 and C100B130N40. These two specimens were the least confined and were subjected to the larger level of axial loads. The level of axial load influences considerably the ductility of columns. Structural ductility drops from 8.8 to 4.7 for specimens with ties spaced at 60 mm, and from 4.4 to 1.6 for specimens with ties spaced at 130 mm, when the axial load level is increased from 15 to 40 % of the axial load capacity. This emphasizes the importance of the axial load level in the design of ductile columns. This variable is not considered in the ACI-318 89 (1989) and the CSA A23.3 (1984) provisions for confinement. A 60 mm tie spacing of the configuration shown in Figure 2 results in 66 % of the required transverse reinforcement in the ACI and Canadian Codes. However, all specimens with this tie spacing displayed a ductile behaviour for the range of axial loads used in this study. It can be concluded that the ACI and Canadian Codes are too conservative for 100 MPa concrete and the level of axial load tested. Neglecting the level of axial load can lead to overdesign, when the axial load level is low, and underdesign, when the axial load level is high. The same conclusions have been reached by Azizinamini *et al.* (1994) and Bing (1994). It is worth pointing out that the New-Zealand concrete code includes the effect of axial load in the confinement reinforcement requirements.

Watson *et al.* (1994) have proposed an equation giving the required transverse reinforcement ratio in terms of axial load level, tie yield strength and concrete strength among other variables. All the specimens which satisfy this equation displayed a ductile behaviour. Because of the use of the tie yield strength in the proposed equation, its application to high-strength transverse reinforcement is questionable. Indeed, Cusson and Paultre (1995) have shown that the full strength of high-strength transverse reinforcement might not be used. The determination of the actual stress in transverse reinforcement, as proposed by Cusson and Paultre (1995), can lead to more realistic transverse reinforcement ratios.

PREDICTION OF MOMENT vs. CURVATURE RESPONSE

To predict the behaviour of all six specimens, three stress-strain models proposed by Park, Priestley and Gill (1982), Thorenfeld, Tomaszewicz and Jensen (1987) and Cusson and Paultre (1995) were implemented in a sectional analysis computer program. The predicted responses for the three different models are plotted together with the measured response of the specimens in Figure 5. The Park, Priestley and Gill model was developed for NSC and overestimates the confinement gain of the member. The Thorenfeld, Tomaszewicz and Jensen model (implemented in the computer program RESPONSE, Collins and Mitchell 1991) was developed for unconfined HSC and significantly underestimates the confinement effect. The Cusson and Paultre model was developed for confined HSC tied columns and predicts well the behaviour of the columns.

CONCLUSION

High-strength concrete columns can be designed to behave in a ductile manner. The lateral steel required for ductile behaviour depends on the axial load level. The equation proposed by Watson *et al.* (1994) is very useful to determine the amount of lateral reinforcement for normal-strength transverse reinforcement. Its application to high-strength transverse reinforcement is questionable. The Cusson and Paultre (1995) stress-strain model seems to be reasonably accurate to predict the behaviour of HSC columns.

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Table 1. Main results

Specimens	f'_c MPa	s mm	ρ_s %	f_{yh} MPa	$P/A_g f'_c$ %	M_{max} kN.m	M'_{max} kN.m	μ_Δ	μ_ϕ	ε_{c80u} mm/mm
C100B60N15	92.4	60	4.26	391	14	250.8	235.8	8.8	-	-
C100B60N25	93.3	60	4.26	391	28	333.0	316.3	8.2	26.9	0.026
C100B60N40	98.2	60	4.26	418	39	385.1	337.0	4.7	7.6	0.026
C100B130N15	94.8	130	1.96	391	14	229.6	215.8	4.4	-	-
C100B130N25	97.7	130	1.96	391	26	341.8	299.1	2.3	3.3	0.015
C100B130N40	104.3	130	1.96	418	37	380.2	334.5	1.6	4.3	0.014

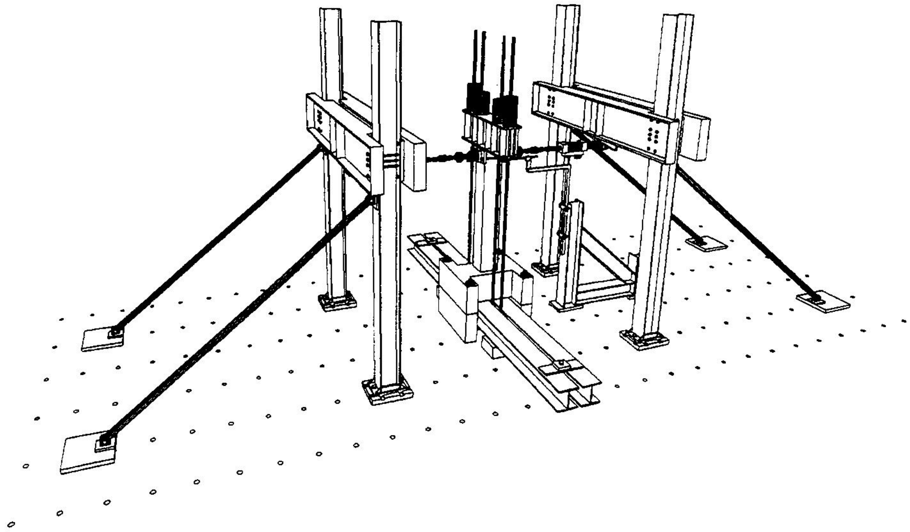

Figure 1. Experimental frame with a specimen

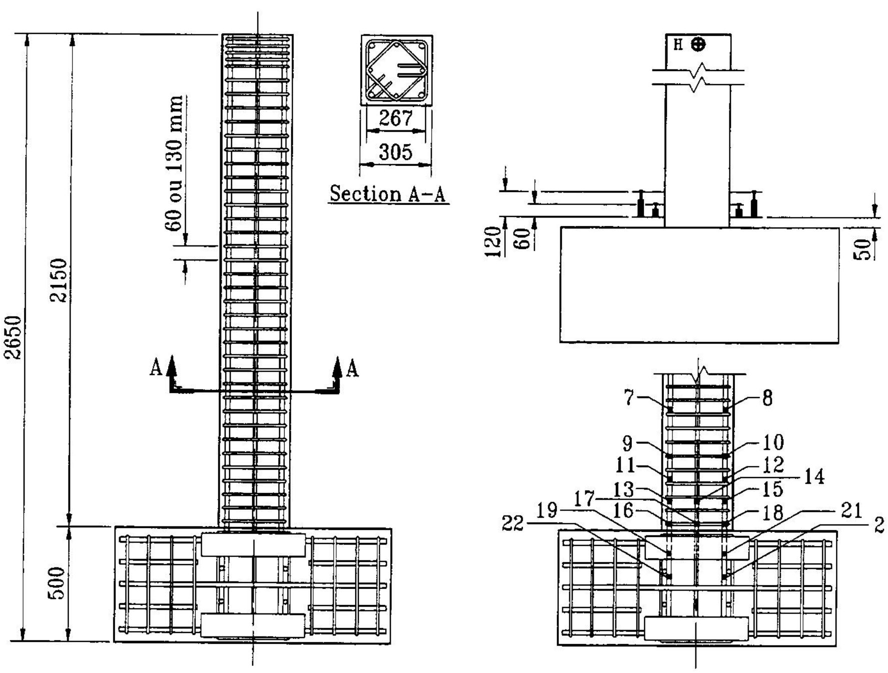

Figure 2. Reinforcing cage and instrumentation details

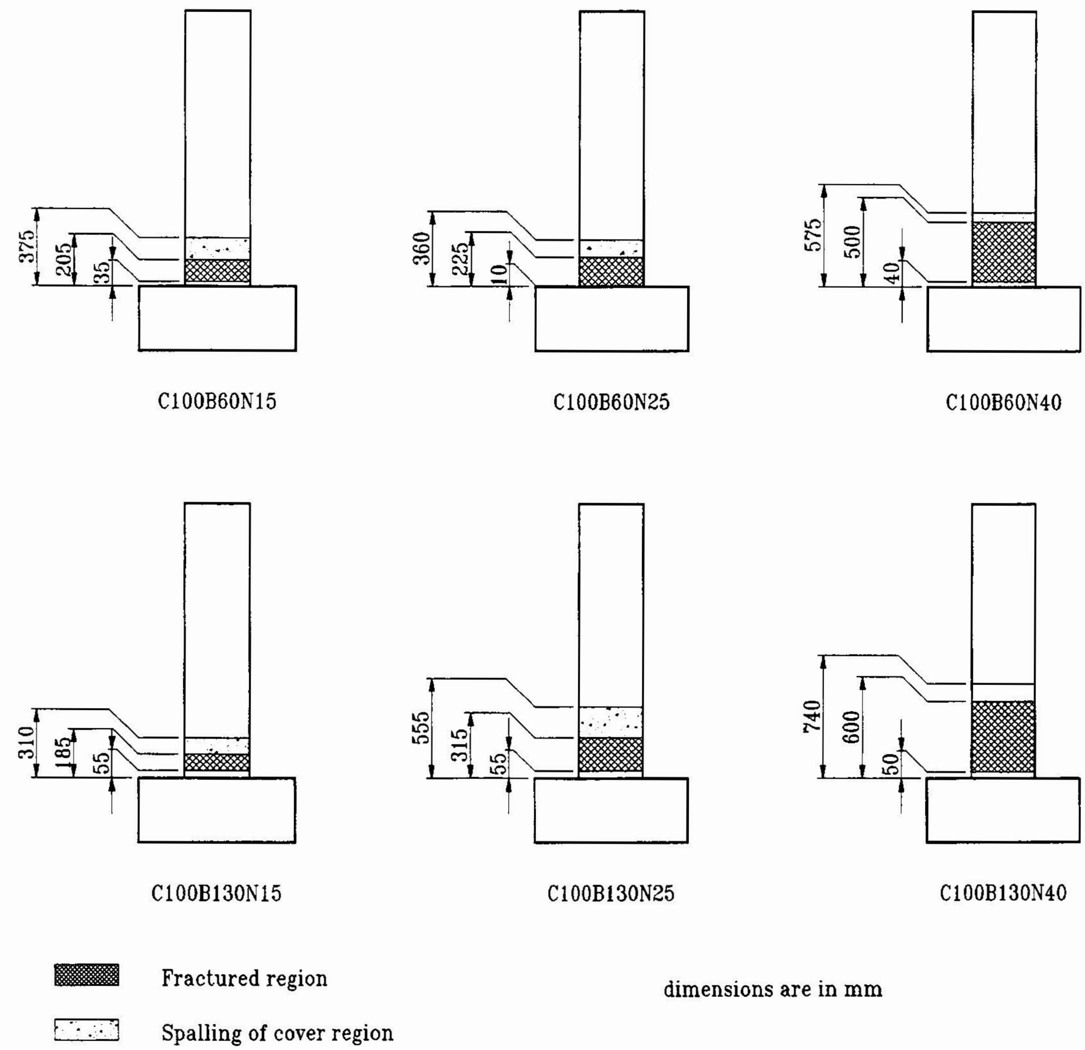

Figure 3. Sketches of most damaged regions of specimens

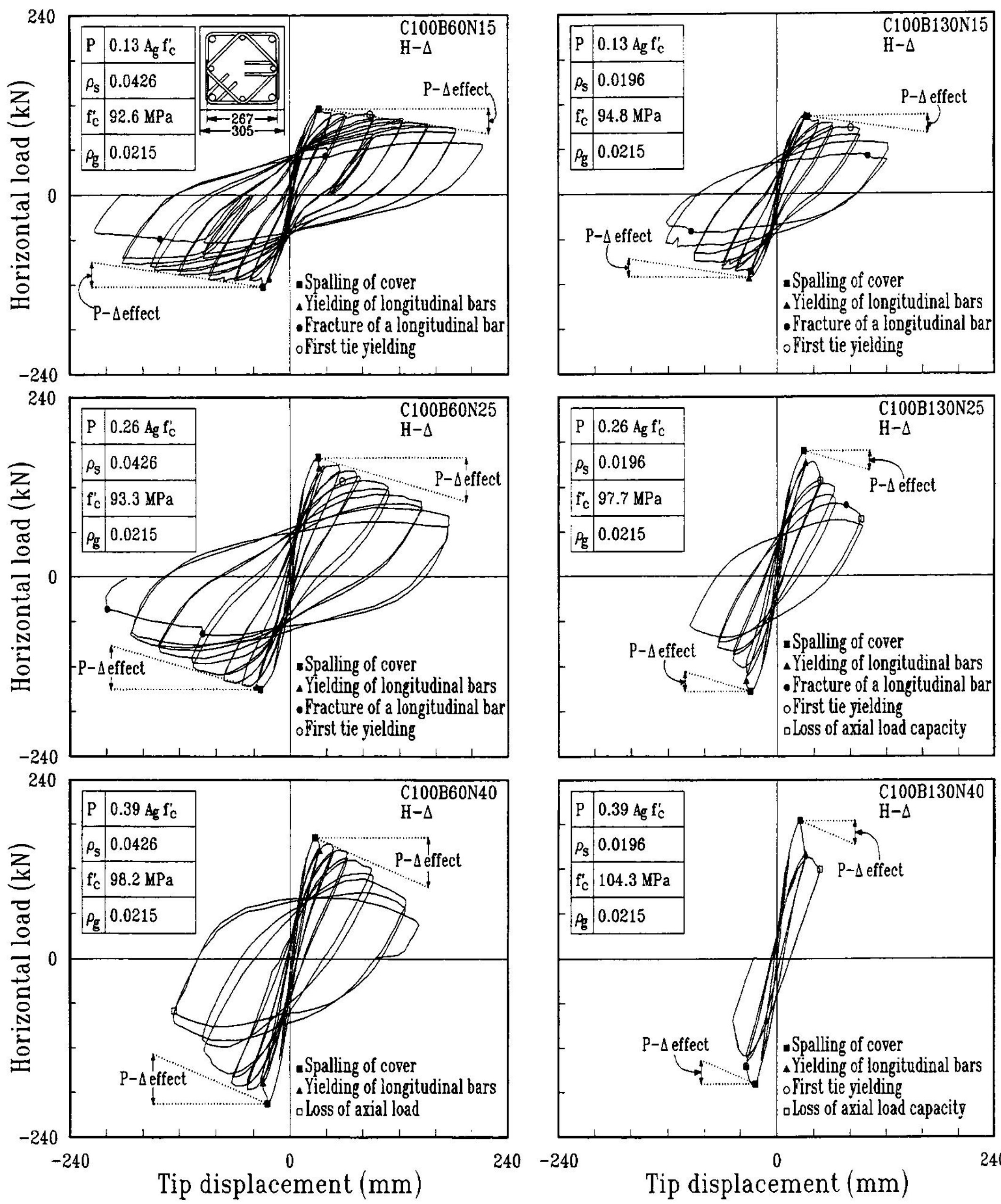

Figure 4. Tip displacement *vs.* applied load

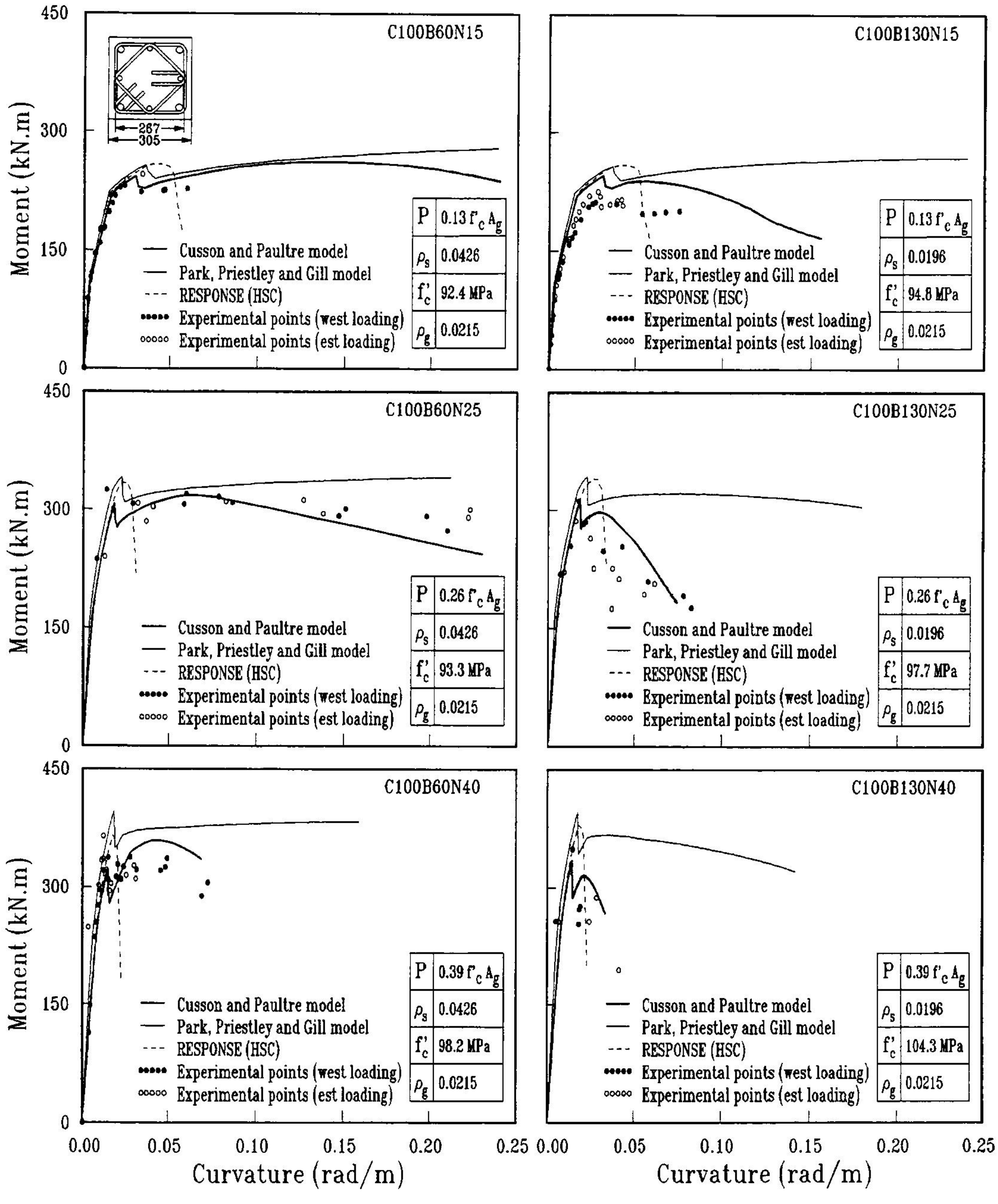

Figure 5. Prediction of moment-curvature responses