SEISMIC SHEAR STRENGTH OF COLUMNS WITH INTERLOCKING SPIRAL REINFORCEMENT

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SUMMARY

The behavior of four 1/4 scale shear-critical rectangular reinforced concrete columns, with interlocking spirals is investigated. Three units were tested under cyclic inelastic lateral displacements with axial load ratios $P/f_c^{'}A_g$ of 0.0, 0.35,-0.1, in double bending. The fourth unit was subjected to vertical loads varying as a function of the applied horizontal loads from axial load ratios of -0.1 to 0.35. Specific emphasis has been devoted to the analysis of the shear force carried by the transverse reinforcement. Among others the approach of considering the column cross section as combination of two circular sections and the UCSD shear model, adapted from circular section to this special configuration, have been investigated. Comparisons between experimental and expected shear response, from different current equations, are provided.

INTRODUCTION

Very few codified procedures or recommendations for column reinforced with interlocking spirals are available. The first and probably only code referring directly to this transverse reinforcement configuration is the California Department of Transportation (Caltrans) Bridge Design Manual [Caltrans, 1991]. The Caltrans specifications are, however, limited to the maximum allowable value for center-to-center spacing of adjacent spirals (0.75 times the diameter of the cage) and to the minimum number of longitudinal bars to be placed in the interlock region (4 bars). The available database of experimental tests completed on interlocking spiral columns is limited as well. The main interest of the present study was to investigate the performance of shear dominated interlocking spirals columns, under different conditions of axial load.

Several models have been used in order to estimate, from geometry and material characteristics, the shear capacity of the four specimens. Their equations are here applied and compared with the experimental results.

PREVIOUS RESEARCH

Tanaka and Park [Tanaka and Park, 1993] performed cyclic horizontal loading tests on three columns with interlocking spiral and, for comparison, on one column with rectangular hoops and cross ties. Columns were rectangular or near-rectangular with dimensions 400 x 600 mm with aspect ratio equal to 3 and designed using provisions for columns with single spirals from the New Zealand Design Code. Axial load was constant. The measured hysteresis loops showed very good energy dissipation and limited reduction in strength. All the units exceeded displacement ductility of 10. Yielding of interlocking spirals occurred at a displacement ductility of 3 to 4 in all tested columns and the measured shear deformation accounted for 10 to 30 percent of the column deflection. The authors also identified the need for specifications related to the minimum quantity of transverse reinforcement, the distance between the centers of adjacent spirals and the appropriate size and spacing of longitudinal bars in the interlocking region.

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Buckingham, McLean and Nelson [Buckingham, McLean and Nelson, 1993] tested eight columns under constant axial load and cycled inelastic lateral displacement. Specimens were approximately 1/5 scale and were designed based on results of an extensive test program on 1/25-scale units. The parameters studied included spiral overlap percentage, the use of nominal reinforcement in the interlock zone, comparison between interlocking spirals and ties, variation in flexural detailing and cross-sectional shape. The increased spiral overlap percentage improved energy dissipation characteristics and reduced lateral load degradation. The opposite effect was observed in case of incorporation of nominal interlocking bars. Specimens reinforced with ties did not performed as well as the ones with spirals, despite a 50% higher content of steel. The last specimen performed well under torsional load but the authors recommended further investigation on the torsional behavior.

DESIGN CONSIDERATIONS

The main attention of this project was dedicated to the shear strength of the columns for the case of variable axial load. In design practice several equation are available to define the contributions to the shear capacity of columns, even though no specific indications are provided about their use in the case of double spirals. In United States usually ACI and ASCE/ACI Committee 426 nominal shear strength definitions are applied, in both the approximate and refined formulation. For space limitation reasons, these equations are not reported here but will be addressed as comparison shear models for the test result analyses.

"Two Columns" Original approach

This predictive approach consists on the application of the shear strength model, developed by Priestley et al. [4] for circular columns. In order to match the actual geometry of the specimens, the model was applied assuming the column cross section as obtained by the combination of two circular sections. The truss mechanism strength is so calculated as :

$$V_s = 2 * \left(\frac{\pi}{2} A_{sp} f_s \frac{D'}{s} \cot \vartheta \right) \qquad (1)$$

were the multiplier 2 refers to the previously mentioned assumption. The concrete contribution is given by:

$$V_c = 0.8 A_g K \sqrt{f_c'} \qquad (2)$$

where K is a function of the curvature ductility μ_ϕ, and A_g is the gross section area.

The axial load P is taken into account by a separate component

$$V_p = P \tan \alpha \qquad (3)$$

where α is the angle between the column axis and the axial force strut.

"Two Columns" Modified Approach

The truss model proposed by Kowalsky et al. [Kowalsky, Priestley and Seible, 1995], modifies the steel component of the previous model, by taking into account the effect of neutral axis depth (c). In the hypothesis of combination of two circular sections, Equation 1 is so modified as:

$$V_s = 2 * \left(\frac{\pi}{2} A_{sp} f_s \frac{D' - c}{s} \cot \vartheta \right) \qquad (4)$$

with the contribution from concrete and axial load unchanged from the previous approach.

UCSD Original

In this approach, the three-component assessment of the column shear capacity, briefly described above, was combined with the definition of equivalent transverse section indicated in Tanaka and Park. In this case the portion of the transverse reinforcement located at the interlocking region is essentially omitted in the steel contribution to the shear capacity. The transverse reinforcement taken into account is shown in Figure 2.

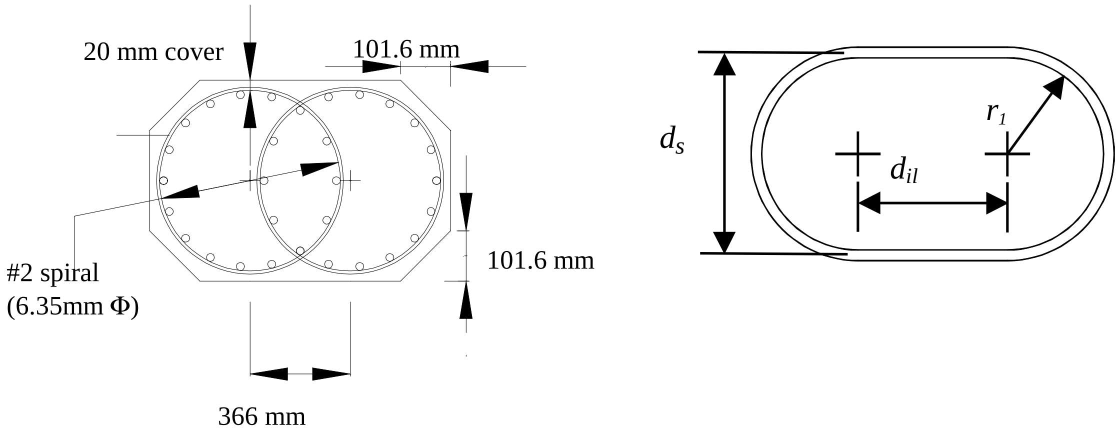

Figure 2. Cross Section of test units and equivalent transverse reinforcement.

The truss mechanism strength is calculated as:

$$V_s = \frac{\pi}{2} A_{sp} f_s \frac{d_s}{s} \cot\vartheta + 2A_{sp} f_s \frac{d_{il}}{s} \cot\vartheta \qquad (5)$$

UCSD Modified

The shear carried by the spirals V_s, as defined in Equation (5) is modified, in this model, in order to take into account the effect of neutral axis depth (c). Based on the extent of c, the V_s contribution becomes:

$$V_s = \frac{\pi}{2} A_{sp} f_s \frac{(\frac{d_s}{2} - c)}{s} \cot\vartheta + 2A_{sp} f_s \frac{d_{il}}{s} \cot\vartheta + \frac{\pi}{2} A_{sp} f_s \frac{d_s}{2s} \cot\vartheta \qquad \text{for } c \le \frac{d_s}{2} \qquad (6a)$$

$$V_s = \frac{\pi}{2} A_{sp} f_s \frac{d_s}{2s} \cot\vartheta + 2A_{sp} f_s \frac{(d_{il} + \frac{d_s}{2} - c)}{s} \cot\vartheta \qquad \text{for } \frac{d_s}{2} < c < \frac{d_s}{2} + d_{il} \qquad (6b)$$

$$V_s = \frac{\pi}{2} A_{sp} f_s \frac{(d_s + d_{il} - c)}{s} \cot\vartheta \qquad \text{for } c \ge \frac{d_s}{2} + d_{il} \qquad (6c)$$

TEST UNIT DIMENSIONS

Because of the lack of test results applicable to the seismic response of columns with interlocking transverse reinforcement, a limited test program of four columns was carried out. The four columns were identical reinforced concrete bridge columns, 1/4 scale, designed to be shear critical. Test units were 2.44 m high with rectangular cross section 0.6m wide and 0.4m deep, with 0.1m chamfers, and were constructed by interlocking two circular spirals with center to center distance of about half the spiral diameter. They were tested in double bending to give M/VD=2.0 where D is the overall section depth. The first and third column (INTER1 and INTER3) were designed to be tested under constant compressive axial load, corresponding to $0.022 f_c' A_g$ and $0.35 f_c' A_g$, respectively. The goal of the second unit (INTER2) was to assess the shear capacity for a column with constant tensile axial load, in order to acquire a limit situation for the unit to be tested under variable axial load regimes. The applied axial load ratio for test unit INTER2 was equal to -0.1. The fourth column (INTER4) was subjected to a variable axial load, as a function of the applied horizontal load. The axial load range was defined assuming the loading conditions of unit 2 and 3 as extreme limits. The axial load ratio ranged from -0.1 to 0.35. . Column details are summarized in Table 1.

Table 1. Dimensions and material strengths for model columns

	INTER 1	INTER 2	INTER 3	INTER 4
Width	0.6 m	0.6 m	0.6 m	0.6 m
Depth	0.4 m	0.4 m	0.4 m	0.4 m
Chamfer	0.1 m	0.1 m	0.1 m	0.1 m
Diameter internal column	0.46 m	0.46 m	0.46 m	0.46 m
Column height	2.44 m	2.44 m	2.44 m	2.44 m
Cover to longitudinal rebars	20.3 mm	20.3 mm	20.3 mm	20.3 mm
Center to center distance	203 mm	203 mm	203 mm	203 mm
Longitudinal steel	G60 30#5 (15.87 mm ϕ)	G60 30#5 (15.87 mm ϕ)	G60 30#5 (15.87 mm ϕ)	G60 30#5 (15.87 mm ϕ)
Transverse steel	G40 Spiral #2 (6.35 mm ϕ) 89 mm pitch	G40 Spiral #2 (6.35 mm ϕ) 89 mm pitch	G40 Spiral #2 (6.35 mm ϕ) 89 mm pitch	G40 Spiral #2 (6.35 mm ϕ) 89 mm pitch
Axial load	180 kN	-801 kN	2788 kN	Variable
Axial load ratio $P/f_c^{'}A_g$	0.022	-0.1	0.35	-0.1~0.35
Longitudinal steel ratio ρ_1	0.026	0.026	0.026	0.026
$f_c^{'}$ (concrete)	35.17 Mpa	34.48 Mpa	35.17 Mpa	36.55 MPa
f_{yh} (spiral)	448.24 Mpa	448.24 MPa	448.24 Mpa	448.24 MPa
f_{yl} (rebars)	442.03 Mpa	442.03 MPa	442.03 Mpa	442.03 MPa

As shown in Figure 2, transverse reinforcement consisted of plain round spiral organized in two circular spirals with 203 mm center-to-center spacing. This distance corresponds to 1.11 r_1 (the upper limit in Caltrans Specifications is $1.5\,r_1$) were r_1 is the radius of the spiral reinforcement. Eight longitudinal bars were located into the interlock region to prevent spiral separation (minimum Caltrans requirement = 4 bars). The test setup is presented in Figure 3. After initial dead load and unbalance moment compensation, the columns were subjected to cycles of force reversals, under force control. The force amplitude was cyclically increased to a maximum value corresponding to the theoretical moment required to induce yield strain in the extreme longitudinal tension bar (first yield). The test was subsequently driven under displacement control to increasing displacement ductility levels, with three cycles at every stage.

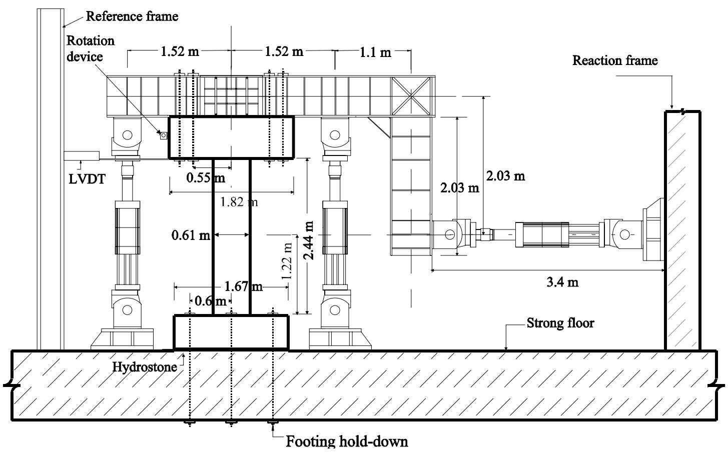

Figure 3 Test Setup

The variable regime of vertical loads, for unit INTER4, was based on the experimental results of INTER2 (P = -0.1 $f_c^' A_g$) and INTER3 (P = +0.35 $f_c^' A_g$). The function between vertical and horizontal load was derived, for a typical two column bent, fully restrained at the base, subjected to horizontal load H. The overall lateral force H corresponds to the sum of the shear capacity of the column under compressive axial load and the one with tensile vertical load, assumed equal to the maximum experimental shear forces for unit 2 and 3, respectively.

EXPERIMENTAL RESULTS

Unit INTER1 (axial load ratio=0.022):

The first cycle at μ_Δ =2 was accompanied by crushing at the column ends under flexural compression and wide crack distribution. Noticeable widening of shear cracks occurred at the first cycle of μ_Δ =2.8, in "pull" direction, in the upper column region, on both east and west sides. The main crack showed a 30° initial inclination to the vertical axis, reduced to about 40° in the interlocking region. Cycling at the same ductility level failure of several spiral loops on the upper column region was noted. The lateral force-displacement response of Figure 4 indicates stable response up to the second cycle at ductility 2.6, when failure occurred. Maximum experimental shear force and corresponding displacement were $V_{\exp}$ = 540.4 kN and $\Delta_{\max}$ = 35.07 mm, respectively.

Unit INTER2 (axial load ratio=-0.1):

Horizontal cracks developed uniformly along the column high at the stage of tensile axial load application.

Significant inclination of the cracks happened at about 300 kN and it visibly extended during the cycles at ductility μ_Δ =1. The unit achieved μ_Δ =4.0 without appreciable degradation of strength and at the first cycle to μ_Δ =6 experienced transverse reinforcement failure. The maximum experimental shear force $V_{\exp}$ was equal to 414.2 kN and the corresponding displacement was $\Delta_{\max}$ =63.79mm.

Unit INTER3 (axial load ratio=0.35):

Shear effect on the crack distribution appeared at early test stages (450 kN), with very simmetric distribution between the top and bottom portion of the column. The crack pattern mainly extended during the following cycles, without a significant development of new crack. Brittle shear failure occurred at displacement ductility μ_Δ =1.8, in the push direction, along a major diagonal crack at the column top. The maximum lateral force $V_{\exp}$ =731.7 kN was reached at displacement $\Delta_{\max}$ = 19.65mm.

Unit INTER4 (axial load ratio=-0.1~0.35)

The cracking pattern of this test unit was characterized by a change in inclination of the diagonal cracks at the interlocking area. The cracks evolved into two vertical cracks on both the east and west side of the column, with a relative distance practically corresponding to the dimension of the interlocking region in the loading direction. These two vertical cracks opened progressively during the test, along the column height. The confining effect of the foundation and loading block prevented the extension of these cracks at the top and bottom 380mm of the column height. Failure is noticeable, from Figure 4, at μ_Δ =2 in push direction.

The Maximum shear forces for positive and negative axial load were 676.0 kN and 377.9 kN respectively.

The displacement measured at maximum lateral load was equal to 17.14mm for compressive axial load and 31.18mm for tensile vertical load.

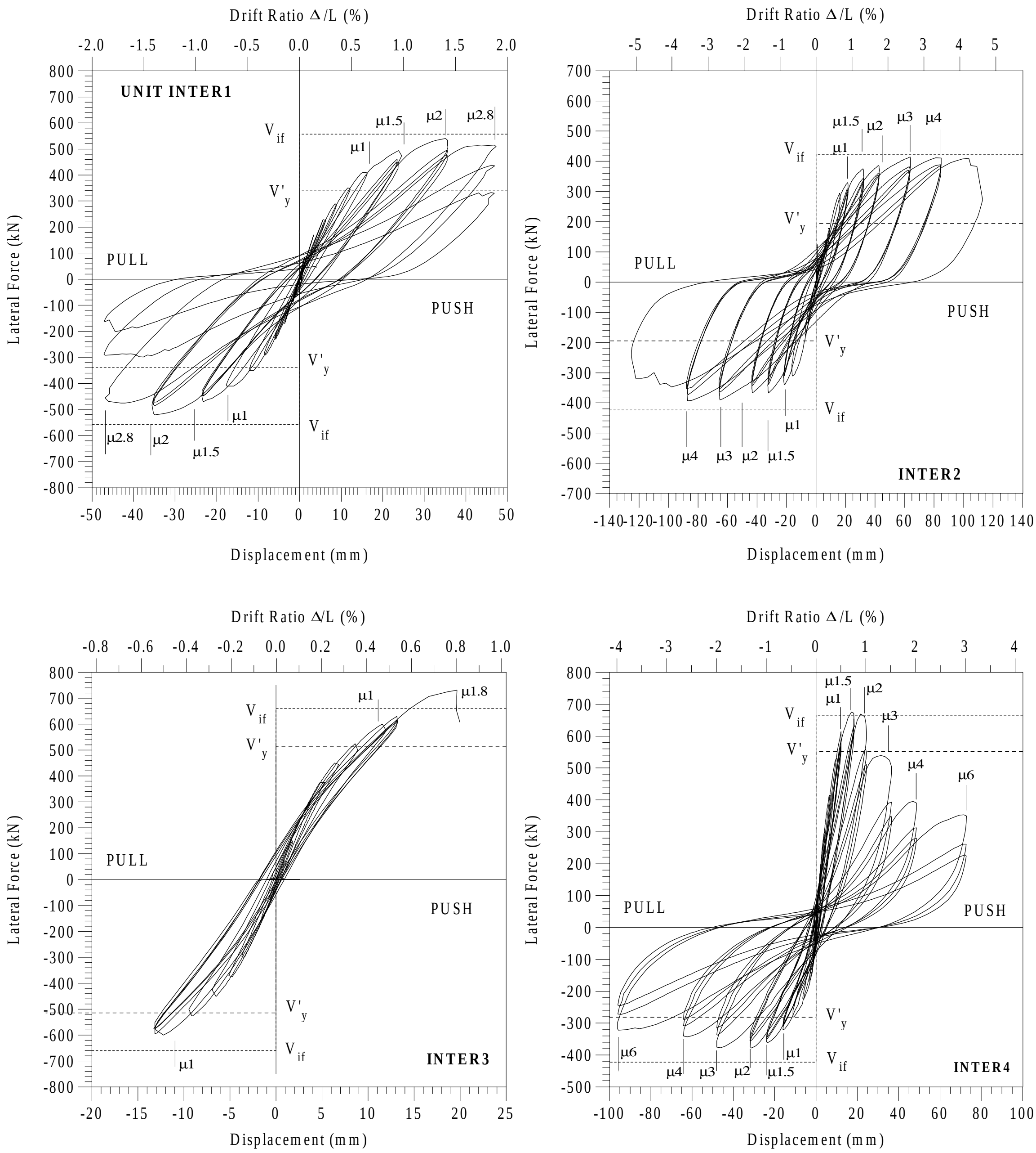

Figure 4. Lateral Force-Displacement response

SHEAR STRENGTH

Different predictions for shear strength of the four tested columns are coupled with experimental results in the following figures.

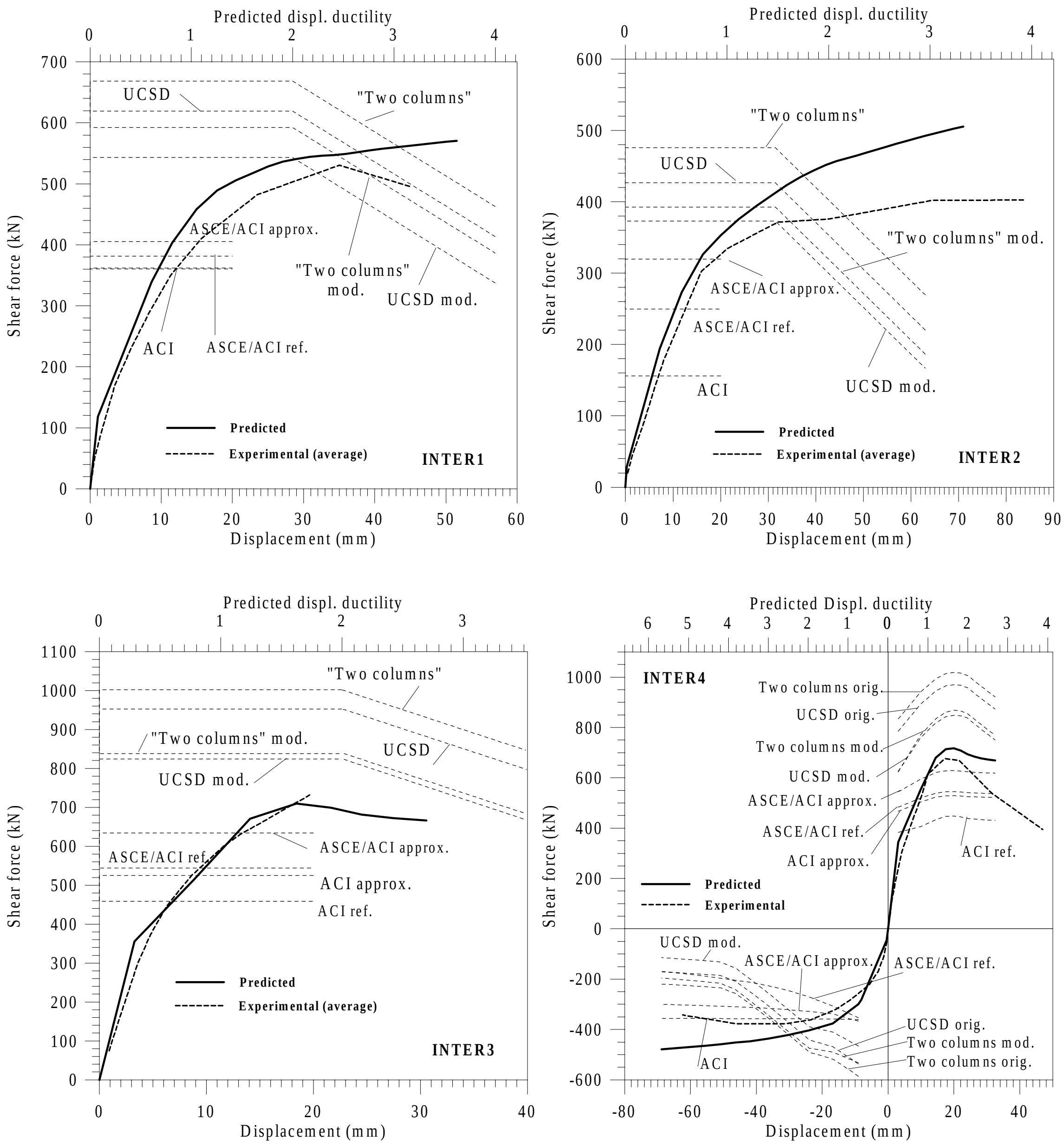

Figure 5 Comparison between experimental and predicted shear capacity

The predicted shear capacity (V_n), calculated as the intersection between predicted flexural response and shear strength envelope curve is reported in Table 2. When intersection between curves is not expected, V_n is assumed equal to the lateral load at maximum predicted displacement. The ratio between maximum measured shear force and nominal shear capacity $V_{\exp}/V_n$ is presented in brackets. It is noticeable the overall significant under-estimate of the ACI shear strength equations, particularly severe for the unit under tensile axial load. The same trend is visible for ASCE/ACI refined approach. The corresponding approximate method shows instead a better agreement with the experimental results, particularly for the test unit under variable axial load. The "two-column" and the UCSD equations are very similar in terms of assessment capacity but the improvement, introduced taking into account the neutral axis depth, is clearly visible for both the methods.

Table 2 comparison between predicted and measured shear strength

	INTER1	INTER2	INTER3	INTER4	INTER4
ACI 318-89 Approx. V_n (V_{exp}/V_n)	362.25 ***(1.492)***	See Ref.	524.80 **(1.394)**	492.0 ***(1.374)***	See Ref.
ACI 318-89 Ref. V_n (V_{exp}/V_n)	360.68 ***(1.498)***	155.78 ***(2.658)***	458.15 ***(1.597)***	388.0 ***(1.742)***	358.00***(1.055)***
ASCE/ACI 426 Appr. V_n (V_{exp}/V_n)	405.29 ***(1.333)***	319.79 ***(1.295)***	634.20 ***(1.154)***	602.0 ***(1.123)***	349.00 ***(1.082)***
ASCE/ACI 426 Ref. V_n (V_{exp}/V_n)	381.27 ***(1.417)***	249.83 ***(1.658)***	544.27 ***(1.344)***	510.0 ***(1.325)***	333.00 ***(1.135)***
"Two columns" Orig. V_n (V_{exp}/V_n)	560.59 ***(0.964)***	437.17 ***(0.947)***	* 927.86 ***(0.788)***	* 921.82 ***(0.733)***	421.00 ***(0.897)***
"Two columns" Mod. V_n (V_{exp}/V_n)	548.01 ***(0.986)***	418.50 ***(0.989)***	* 764.15 ***(0.957)***	* 768.70 ***(0.879)***	419.00 ***(0.901)***
UCSD Orig. V_n (V_{exp}/V_n)	552.55 ***(0.978)***	419.29 ***(0.987)***	* 878.59 ***(0.832)***	* 873.20 ***(0.774)***	411.00 ***(0.919)***
UCSD Mod. V_n (V_{exp}/V_n)	539.25 ***(1.002)***	375.16 ***(1.104)***	* 749.73 ***(0.976)***	* 750.58 ***(0.900)***	395.00 ***(0.956)***

CONCLUSIONS AND RECOMMENDATIONS

The interlocking spiral construction technique proved to provide a significant level of performance for shear critical columns. The ratio between experimental and ideal flexural lateral load was never below 97%. The three component shear model ($V_n = V_c + V_s + V_p$) provided a satisfactory prediction of the measured shear capacity for both the "two column" and UCSD formulations. Good agreement with experimental results was achieved under the assumption that only the transverse reinforcement on the tensile side of the neutral axis provides contribution to the shear capacity.

Differential slippage experienced between the two spirally reinforced sections, suggest further analytical and experimental investigation, particularly focused on the extent of the interlocking region and its content of reinforcement.

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