Proceedings of the Tenth Pacific Conference on Earthquake Engineering
Building an Earthquake-Resilient Pacific
6-8 November 2015, Sydney, Australia

Seismic Performance of Steel Reinforced Concrete Composite Columns

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ABSTRACT: The experimental study presented in this paper addresses the seismic performance of steel reinforced concrete columns experiencing shear and flexural failures using different concrete grades and confinement details to mimic both existing buildings and modern tall buildings. Test specimens represent exterior columns modelled based on a typical seismic design of a 30-story prototype new core wall-frame tall building and a 20-story prototype gravity existing building. Test parameters are target failure mode, axial load ratio, percentage of longitudinal steel, concrete grade, and the transverse reinforcement volumetric ratio. The tests aim to establish criteria to classify the SRC column failure modes along with a preliminary attempt to establish backbone curve recommendations. The results show significant shear capacity of the tested columns that can be sustained by the composite section and a very satisfactory flexural performance up to a drift of 6.5%.

1 INTRODUCTION

Modern construction industry is witnessing a substantial increase in the number, heights and architectural irregularity of tall buildings. This, naturally, has led to exceeding the building code height or irregularity limitations, which has raised the need for using non-prescriptive design or performance-based engineering of these tall buildings. In addition, the real-estate developers increasingly demand smaller column and shear wall sections to maximize building usable and sellable space, particularly in mega-cities' business districts. Moreover, the existing building stock in many active seismic regions includes many seismically deficient buildings that were built before enforcing seismic details in the 1980s. Steel-reinforced concrete (SRC) composite columns and/or high strength concrete columns are being increasingly utilized in tall buildings to achieve these goals. Additionally, many existing buildings utilize SRC columns that are not seismically detailed. Practicing engineers face a major problem since performance-based earthquake nonlinear modeling and design of SRC columns are poorly informed by laboratory tests and nonlinear seismic design guidelines due to test scarcity. Literature reveals a serious lack of knowledge of the seismic behavior of SRC composite columns subjected to simulated seismic loading conditions. There are a small number of tests available to justify deriving seismic backbone curves for macro-modeling purposes. Numerical criteria to distinguish the seismic modes of failure of such columns are not available. In addition, no information on the residual axial capacity of composite columns following shear or flexural failure can be drawn from the few tests available in the literature due to premature test termination.

Rocles and Paboojian (1992), studied six composite column specimens to test lateral stiffness, transverse shear resistance, degree of concrete confinement to achieve adequate ductility, and the effectiveness of shear studs in resisting lateral loading. Chen et al (2007), conducted an experimental study on twenty six specimens to study the seismic behavior of steel-concrete composite members and their influence parameters. They used three steel section shapes and changed the parameters of axial load ratio, longitudinal steel ratio, steel section ratio, embedded steel section length, and transverse steel ratio. According to the results of these two studies, longitudinal bar buckling must be prevented to preserve the integrity of the member; the axial compression ratio is an important factor that affects the seismic behavior of steel concrete columns; stirrup ratio is also an important factor to affect the seismic behavior of steel concrete composite column; and the minimum value of the embedded depth of steel concrete composite column can be 2.5 times the section depth. No mode of failure criteria, recommendations for the backbone curves or performance acceptance criteria were made in these studies.

2 PROTOTYPE BUILDINGS

The current series of tests aims to establish more specific recommendations for the cyclic backbone curves of SRC sections. A high-rise building was used as a prototype building to obtain realistic demands on an exterior column of a modern tall building. The demands on the columns representing existing buildings were estimated based on the axial load ratios prevailing in older construction. Concrete strength used was f_c'=35 MPa and f_c'= 70 MPa for older and modern buildings, respectively. Table 1 shows the assumed parameters for the first prototype building.

Table 1: High rise prototype building parameters

Number of floors	30 floors
Ground floor height	3 m
Total height	90 m
Building area	1765 m^2
Live load	3 kN/m^2
Flooring cover	1.5 kN/m^2
Slab thickness	0.20 m
Load factors	1.4D.L+L.L+EQ
Location	San Francisco
Earthquake combination	100%Y direction + 30%X direction
Core Shear wall dimensions	6x6x0.3 m

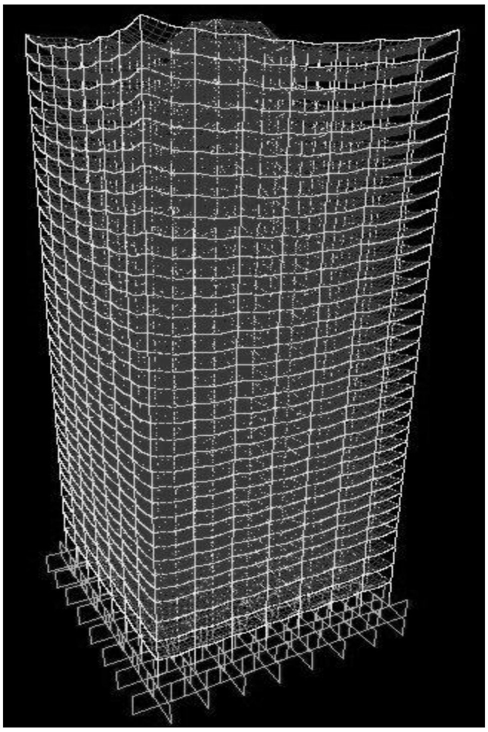

Figure 1: Prototype tall building numerical model

According to these parameters and using SAP2000 for modeling, the design axial load of the exterior column is 25,000 kN. Then, using the ACI 318-14 equation for designing composite column section which is:

$$P_u = A_a F_{ya} + A_s F_{ys} + A_c f_c'$$

P_u = Maximum axial load
A_a = Area cross section of steel shape = 1-6%A_c
F_{ya} = steel shape yield strength
A_s = Total area cross sections of longitudinal steel bars = 1-2%A_c
F_{ys} = longitudinal steel bars yield strength
A_c = cross section area of the column
f_c' = cylinder concrete strength

With this equation the primary parameters of the column were found which are listed in Table 2

Table 2: Column design parameters

Column section	0.75x0.75 m
Steel shape	WF 18x86, steel ratio 2.9%A_c
Longitudinal steel bars	20 dia. 25 mm, steel ratio 1.75%A_c

Another 20 story prototype building mimicking older construction was used to obtain the demands on the existing shear deficient exterior column. SAP2000 analysis results indicate design exterior column axial force of 9500 kN. According to ACI 318-63, the composite column equation design does not differ from the new equation of ACI 318-14 presented earlier. However, the main difference is in the steel shape ratio = 5-9% A_c in older code versus 1-3%A_c in modern ACI 318 codes; and the longitudinal steel ratio is 2-3% A_c instead of 1-2% A_c. The existing column section was designed as:

Table 5: Primary existing column parameters

Column section	0.45x0.45 m
Steel shape	WF 10x54 = 5%A_c
Longitudinal steel bars	12 dia. 25 mm = 2.9%A_c

3 TEST MATRIX

To deal with section experimentally, it had to be scaled to fit the lab dimensions. Scale factor of 0.5 was a very suitable scale to be easily built and tested within the available space. To represent a high-rise building, it was decided to build 9 specimens representing ground floor of the prototype building. Table 3 shows the overall specimen dimensions and reinforcement for modern tall building columns.

Table 3: Tall building specimen design

Test target	Number of specimens	f_c' MPa	Column dimensions (m)	Steel Reinforcement	Steel shape
Conventional concrete	7	35	0.3x0.3x1.5	8 dia 16 mm (Flexure failure)	HEB 120
High strength concrete	2	70		12 dia18 mm (Shear failure)	

The axial load ratios were calculated from this equation: $ALR = (P_u) / (A_c f_c')$
where P_u is maximum axial load and A_c is the gross section area

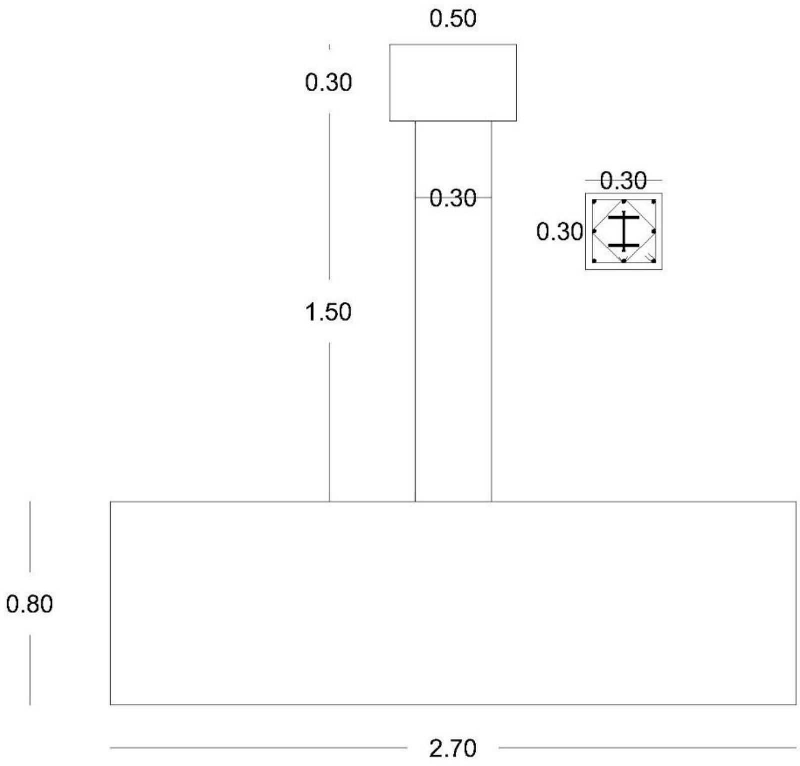

Figure 2: Test specimen

Figure 2 shows the test specimen dimensions and reinforcement details, while Table 4 presents the test matrix details for the specimens representing modern tall buildings. Table 5 shows the overall dimensions and reinforcement of exiting building specimens, while Table 6 shows a detailed test matrix for these columns.

Table 4: Test matrix (tall building specimens)

Specimens ID	***fc'*** (MPa)	**Target Failure Mode**	**ALR**	**Hoop Spacing**	**Steel Shape %**	**Reinf. Steel %** ρ
CSF10N	35	Flexure Ten.	0.10	S=7.5cm	3.7% (H120)	1.7% (8Φ16)
CSS15-N	35	Flex Ten.	0.15	S=25cm	3.7% (H120)	3% (12Φ18)
CSF45-N	35	Flexure Comp.	0.45	S=7.5cm	3.7% (H120)	1.7% (8Φ16)
CSS55-N	35	Flex Comp.	0.55	S=25cm	3.7% (H120)	3% (12Φ18)
CSF25-N	35	Flexure Comp.	0.25	S=7.5cm	3.7% (H120)	1.7% (8Φ16)
CSF55-N	35	Flexure Comp.	0.55	S=7.5cm	3.7% (H120)	1.7% (8Φ16)
CSF35-N	35	Flexure Comp.	0.35	S=7.5cm	3.7% (H120)	1.7% (8Φ16)
CSF15-H	70	Flexure Ten.	0.15	S=7.5cm	3.7% (H120)	1.7% (8Φ16)
CSF55-H	70	Flexure Comp.	0.55	S=7.5cm	3.7% (H120)	1.7% (8Φ16)

Table 5: Existing building specimen design

Number of specimens	f_c' **(MPa)**	**Base dimension**	**Column dimension**	**Reinforced Steel bars**	**Steel shape**
6	27	0.8x0.6x2.4 m	0.3x0.3x1.5 m	8 dia16 (Flexure failure) 12 dia18 (Shear failure)	HEB 160

Table 6: Test matrix (existing building specimens)

Specimens ID	*fc'* MPa	Target Failure Mode	ALR	Hoop Spacing	Steel Section Ratio	Reinforcement Steel Ratio
CSS10-E	27	Shear	0.10	S=30 cm	6% (H160)	3% (12Φ18)
CSS40-E	27	Shear	0.40	S=30 cm	6% (H160)	3% (12Φ18)
CSS20-E	27	Shear	0.20	S=30 cm	6% (H160)	3% (12Φ18)
CSS55-E	27	Shear	0.55	S=30 cm	6% (H160)	3% (12Φ18)
CSF15-E	27	Flexure Ten	0.15	S=7.5 cm	6% (H160)	1.7% (8Φ16)
CSF50-E	27	Flexure Comp	0.50	S=7.5 cm	6% (H160)	1.7% (8Φ16)

4 TEST SETUP

The test setup comprises a horizontal 5000 kN actuator with 120 mm tension and compression stroke supported to strong wall and applying lateral load on the top of the specimen. A vertical load cell connected to a jack that is attached to a loading frame and braced laterally to the reaction wall was used to apply the vertical load. A rolling mechanism was introduced to allow for sliding of the column top. The test was performed as displacement controlled. The test setup is shown in Fig. 3. The displacement protocol was derived based on multiples of the theoretical yield displacement and is shown in Fig. 5.

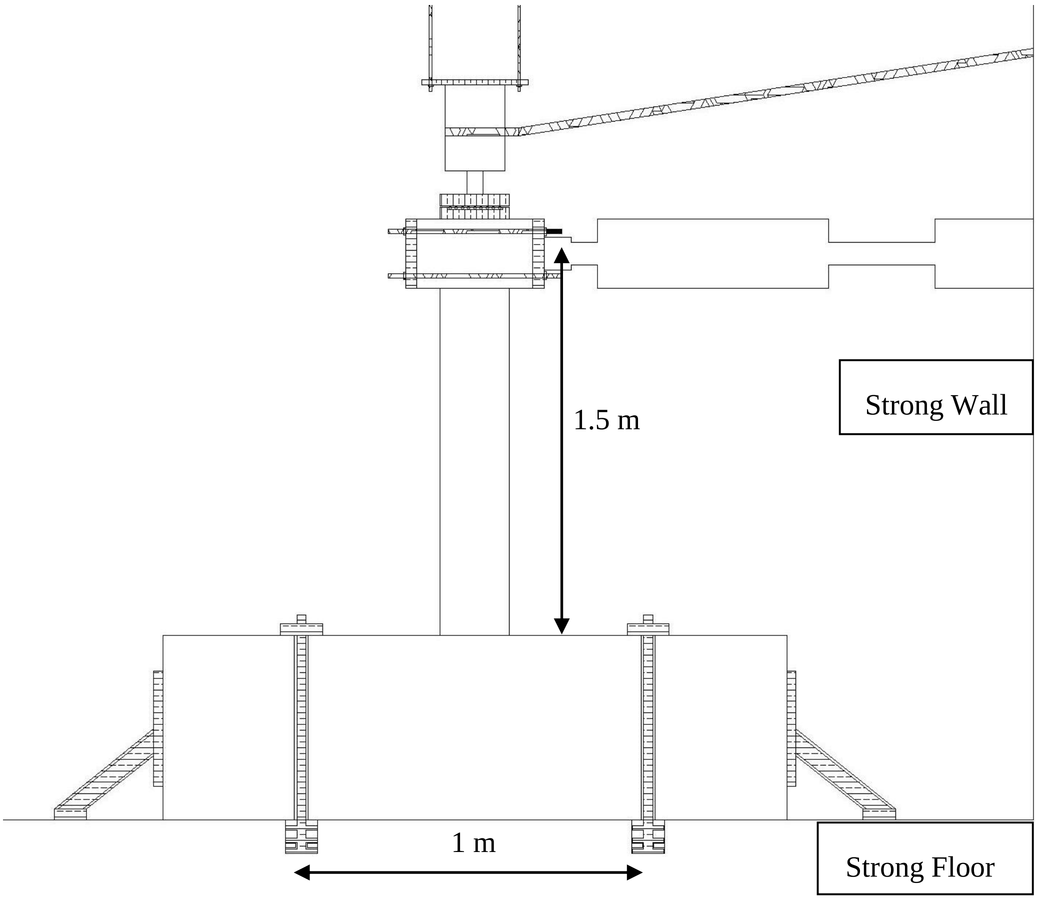

Figure 3: Test setup

Figure 4: A test specimen before testing

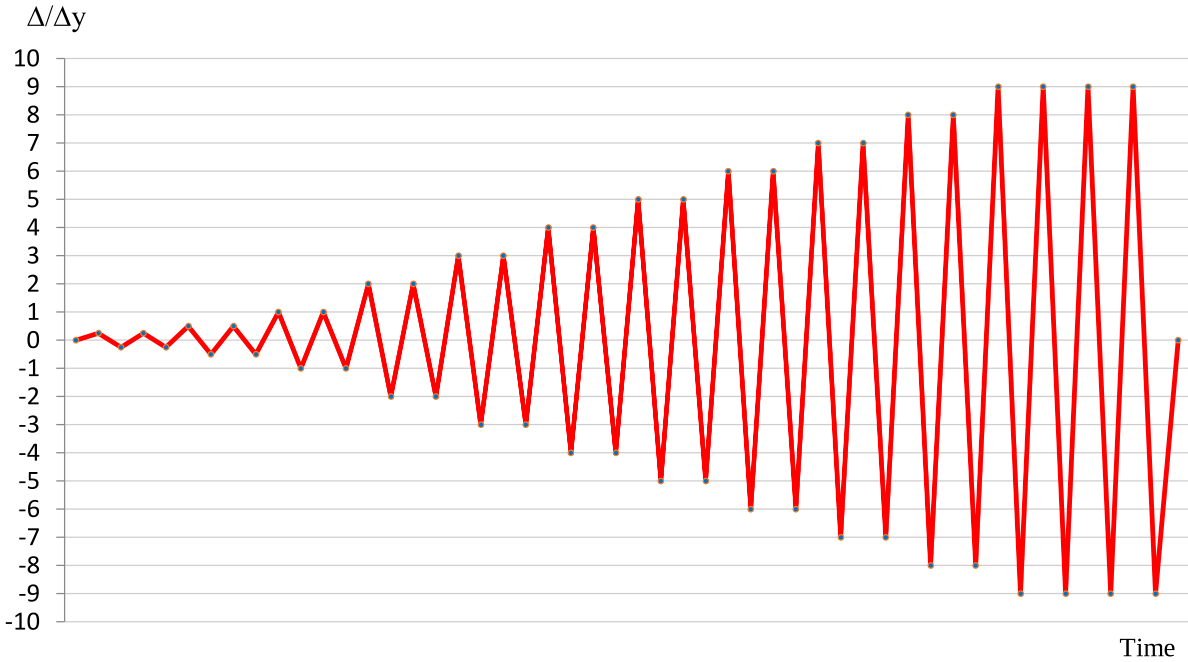

Figure 5: Displacement protocol

5 EXPERIMENTAL RESULTS

Figure 6 shows the shear force-drift ratio hysteresis response of specimen CSF10-N. The flexural nature of the response is clear through the cycles' shape which also show a strain hardening trend reaching a significant drift of about 6.5% without any strength degradation.

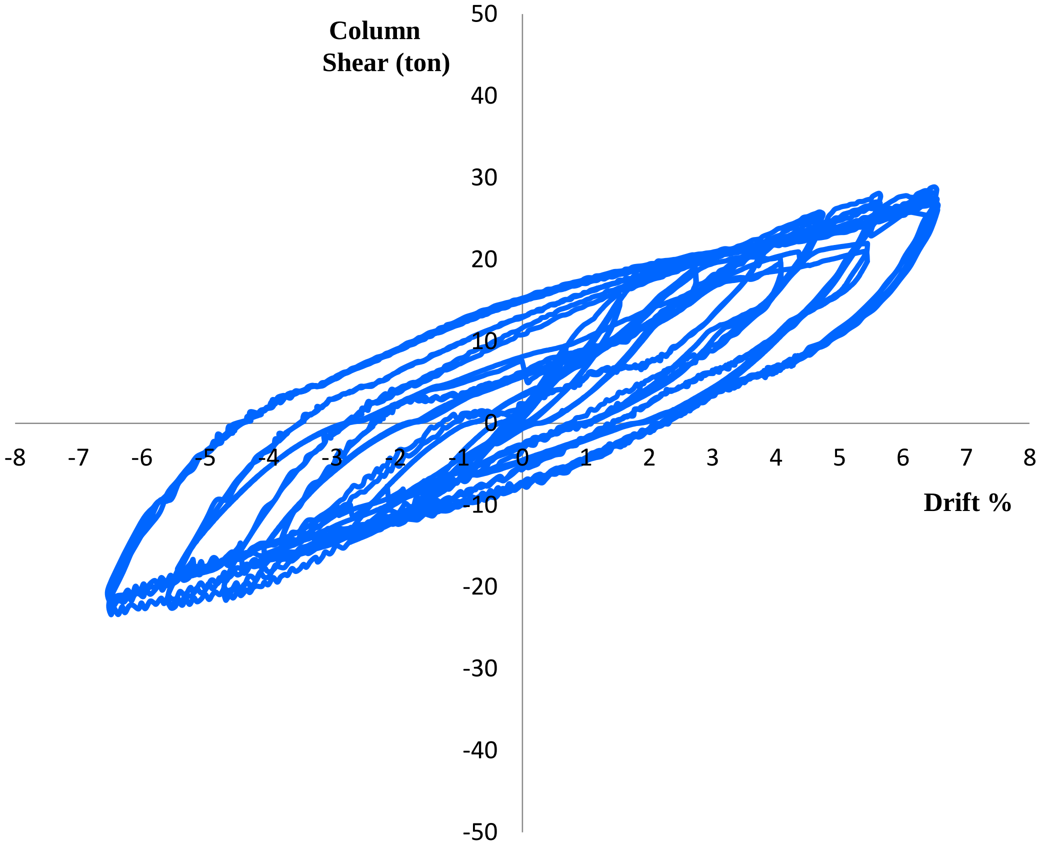

Figure 6: Shear force-drift hysteresis response of specimen CSF10-N

Figure 7 shows the failure mode of specimen CSF10-N under the effect of the applied displacement protocol. The specimen failed in a flexural tension failure mode as predicted by the theoretical analysis.

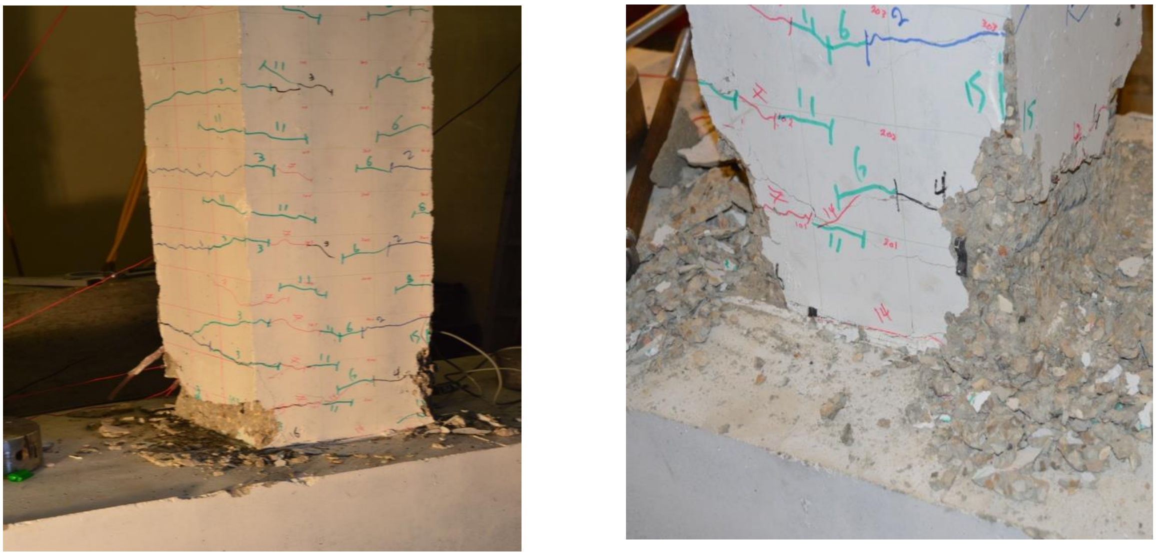

Figure 7: Failure mode of specimen CSF10-N

Figure 7 *(continued)*: Failure mode of specimen CSF10-N

Figure 8 shows the failure mode of specimen CSS15-N under the effect of the applied displacement protocol. The target failure mode of the specimen was flexural tension failure. According to ACI 318-14, the minimum hoop spacing used was 250 mm. The effect of hoop spacing on the backbone curve will be studied through comparing the response to that of specimen CSS55-N.

Figure 9 shows the shear-drift hysteresis response of specimen CSS15-N until the test termination drift ratio of about 6.5% due to actuator stroke capacity. A strain hardening trend reaching a significant drift of 6.5% without any strength degradation is evident. This drift ratio is believed to exceed any practical drift ratio corresponding to collapse prevention limit state. The peak shear value was 490 kN which excessively exceeded the predicted value of 190 kN that originally was believed to correspond to the flexural capacity of the section.

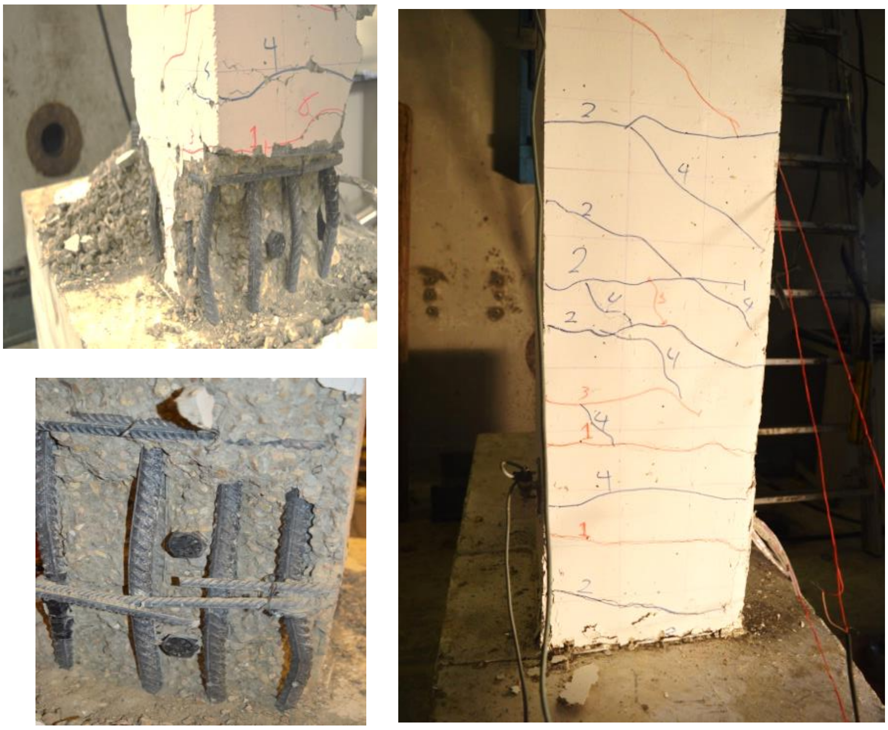

Figure 8: Failure mode of specimen CSS15-N

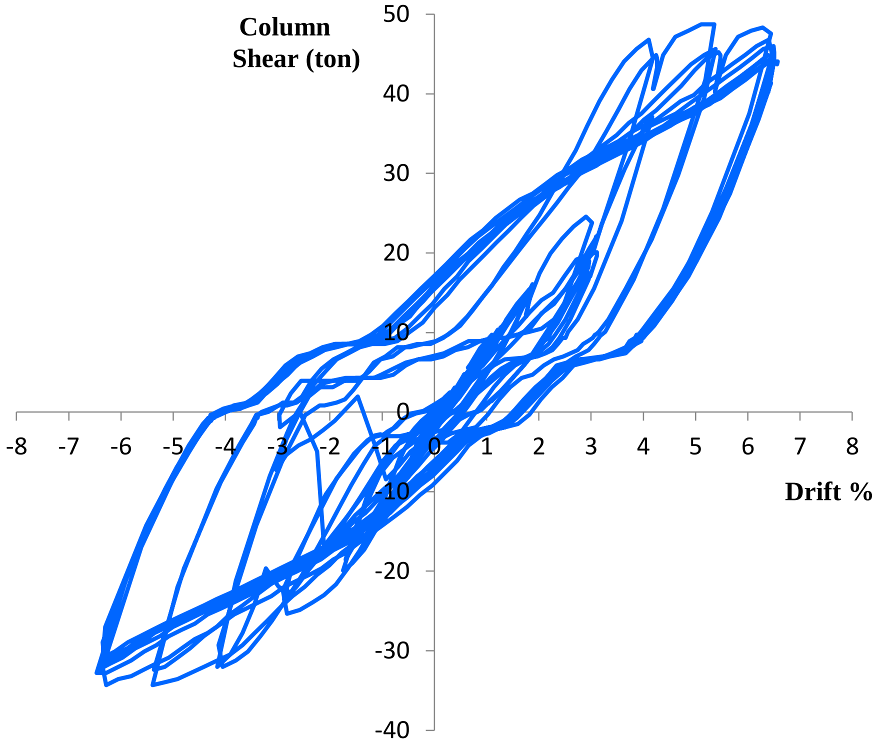

Figure 9: Shear force-drift hysteresis response of specimen CSS15-N

Figure 10 shows the failure mode of specimen CSS10-E under the effect of the applied displacement protocol. The target failure mode of the specimen was shear failure. The specimen was designed according to ACI 318-63. The hoop spacing was 300 mm. Some shear flexural cracks have appeared during the test. However, the specimen appears to have failed in flexure due to shear over-strength of the embedded steel section.

Figure 11 shows the shear-drift hysteresis response of specimen CSS10-N. The specimen initially yielded in flexure but at the large deformation resulting from strain hardening the 90 degree hooks of the hoops have opened and left the concrete in compression poorly confined which resulted in the crushing of concrete in compression.

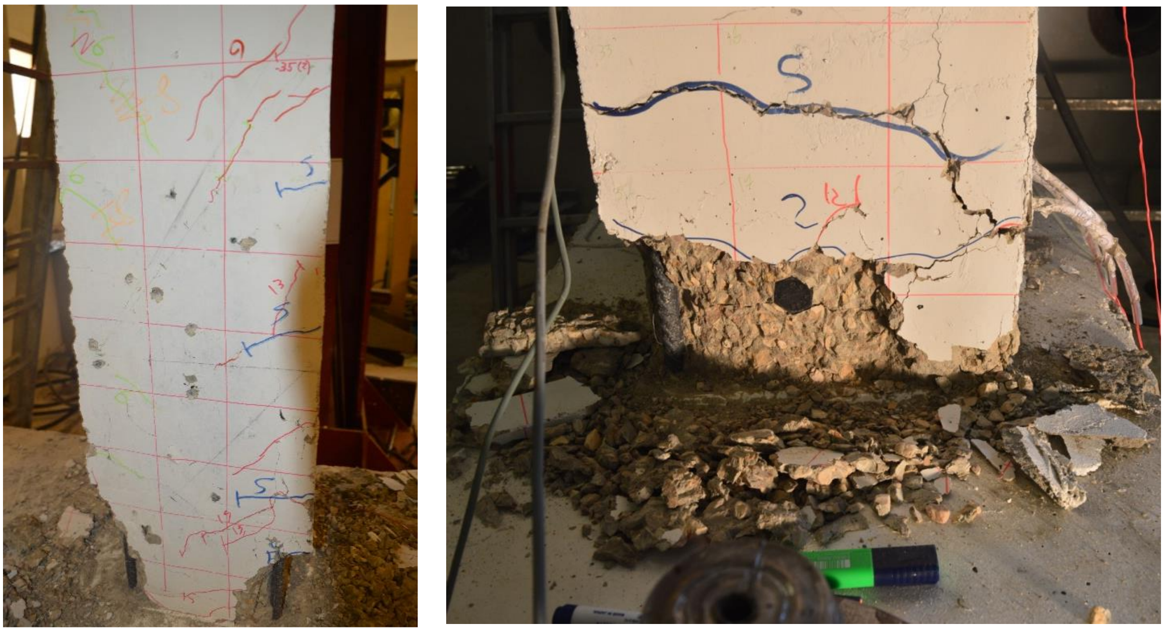

Figure 10: Failure mode of specimen CSS10-E

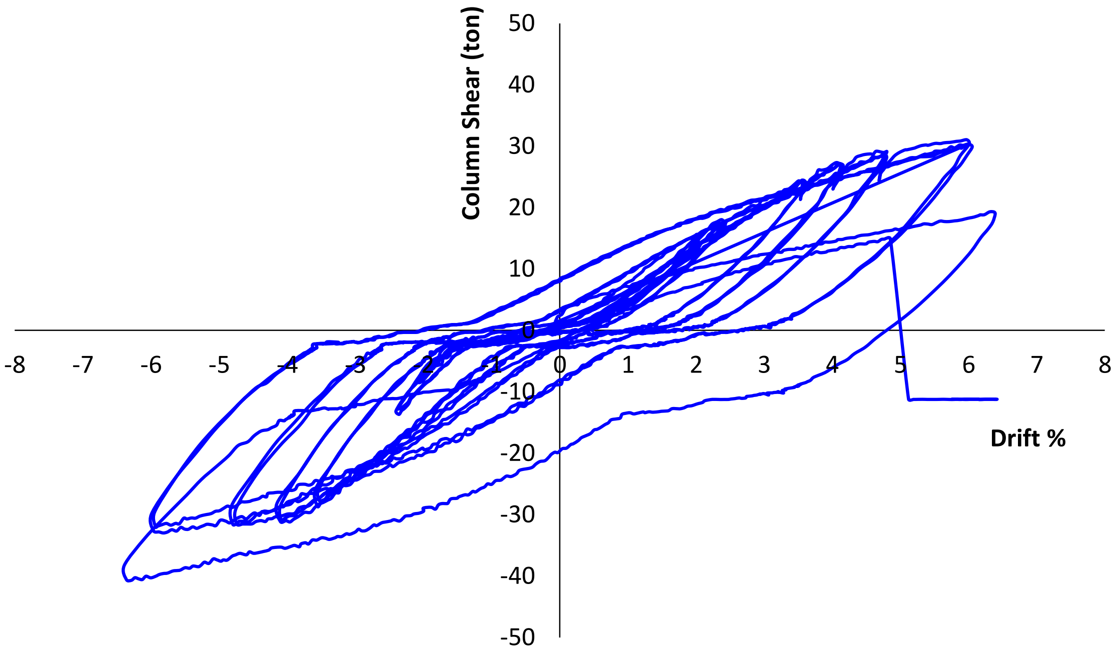

Figure 11: Shear force-drift hysteresis response of specimen CSS10-E

6 CONCLUSIONS

Three steel reinforced concrete composite columns were tested to simulate seismic action on modern tall buildings designed for lateral load resistance and typical existing buildings not designed to be seismically resistant. According to the test results, the following conclusions are made:

1- The steel section web and shear studs work to over-strength the column in shear. Thus, the shear failure of columns designed according to ACI 318-14 and AISC 341-2008 is not likely.

2- Both specimens representing the new tall building construction show very satisfactory flexural performance with no strength degradation until large drift ratio of 6.5% which exceeds any practical drift ratio for collapse prevention.

3- The existing building column did not fail in shear although poorly reinforced with transverse hoops, however, it sustained large flexural deformation and ended with opening the poorly detailed 90 degree hoops causing poorly confined concrete to crush at drift ratio of 4.9%.

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