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BEHAVIOUR OF SHORT REINFORCED CONCRETE COLUMNS REHABILITATED WITH EXTERNAL STEEL COLLARS

J. Liu[1], R.G. Driver[2] and A.S. Lubell[3]

ABSTRACT

A seismic rehabilitation technique is investigated for short reinforced concrete columns based on the use of external steel collars. The performance of this technique is evaluated through full-scale experiments and finite element analysis. Ten cantilever columns, including two control columns and eight rehabilitated columns confined externally by steel collars, have been constructed and tested under combined lateral and axial loading. Parameters considered in the experimental program include collar spacing, collar stiffness, longitudinal reinforcement ratio, axial compression ratio, pretension of collar bolts, and shear span-to-depth ratio. One control column was tested to failure and then repaired to study the feasibility of using external steel collars on previously damaged columns. The experimental results have shown excellent improvements in the ductility, strength, and energy dissipation capacity of the columns due to the presence of the collars. Three-dimensional finite element models were developed using the finite element program ABAQUS to further investigate the behaviour of these externally confined columns. Experimental results and finite element analysis have shown that this rehabilitation technique has great promise as an effective procedure for seismic rehabilitation of deficient reinforced concrete columns.

Introduction

The building stock and infrastructure around the world are aging and in constant need of maintenance, repair, and upgrading. Many reinforced concrete buildings built in high seismic regions before the 1970s are considered deficient due to reinforcing details that will not ensure ductile member response under severe seismic loading. Other life-cycle design considerations for structures, such as changes to the occupancy loading, may also make the original structure deficient, especially in earthquake-prone regions. Severe damage and even collapse of some structures in recent earthquakes due to column failures have highlighted the urgency and importance of rehabilitating seismically deficient structures to achieve an acceptable level of performance and life-safety.

Many research programs have been focussed on the seismic rehabilitation of existing structures, including deficient columns. The selection of a rehabilitation technique must consider the structural effectiveness, construction time, cost of materials, fabrication and installation, and the disruption to the building occupants during rehabilitation. A less intrusive rehabilitation scheme for reinforced concrete columns using external steel collars cut from rolled steel plate or fabricated from hollow structural sections has

[1]Graduate Research Assistant, Dept. of Civil & Envir. Engineering, University of Alberta, Edmonton, AB. Canada T6G 2W2
[2]Professor, Dept. of Civil & Envir. Engineering, University of Alberta, Edmonton, AB. Canada T6G 2W2
[3]Assistant Professor, Dept. of Civil & Envir. Engineering, University of Alberta, Edmonton, AB. Canada T6G 2W2

been proposed by Hussain and Driver (2001) and Driver *et al.* (2001). In this long-term research project, earlier phases showed through experimental and analytical studies that external collars can provide a very effective rehabilitation system. Collared reinforced concrete columns exhibited increases in both strength and ductility under concentric and eccentric axial loading and under flexure-dominant simulated seismic loading (Hussain and Driver 2001, 2003, 2005; Chapman and Driver 2006). This paper presents additional results from experiments and nonlinear finite element analysis for short reinforced concrete columns with steel collars under cyclic shear-dominant loading.

Experimental Program

Test Specimens

Fabrication and testing were conducted in the I.F. Morrison Structural Engineering Laboratory at the University of Alberta. Ten cantilever column specimens were constructed, including two control columns (CV0A, CV0B) and eight rehabilitated columns confined externally by steel collars (CV1 to CV8). Each column had a cross-section 400x400 mm and an overall height of 800 mm measured above an integral footing at the base measuring 1600x1000x570 mm. The footing was anchored to the strong floor by four prestressed 50 mm diameter high strength threaded rods. Column specimens had a shear span-to-depth ratio, M/(VD), of 1.63 or 0.88, where M, V, and D are the moment and shear at the base and the overall column dimension (400 mm), respectively, to create a high shear-to-moment ratio. A summary of the test specimen characteristics are provided as Fig. 1 and Table 1.

Columns CV1 to CV8 incorporated external collars, but no internal tie reinforcement. Control columns CV0A and CV0B incorporated conventional transverse reinforcement in the form of 10M closed ties at 400 and 100 mm spacing, respectively. CV0A was retested after being repaired and rehabilitated to investigate the feasibility of using external steel collars on previously damaged columns.

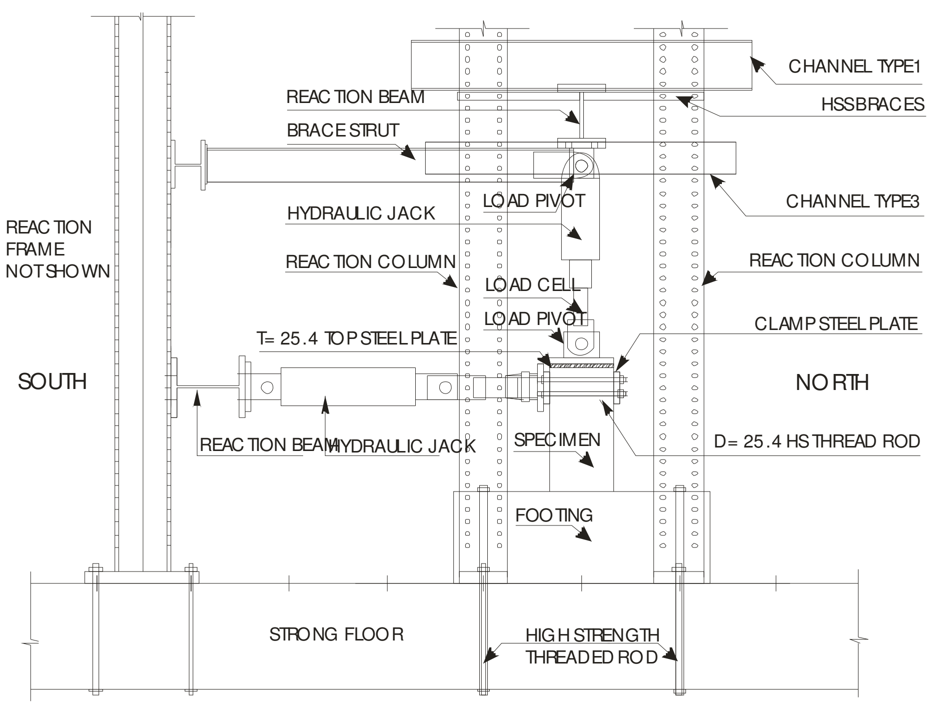


Figure 1. Test set-up.

Table 1. Summary of the test specimens.

Specimen	Aspect ratio	Collar cross-section*	Collar centre-centre spacing**	Axial compression index	Longitudinal reinforcement ratio***	Pretension in each collar bolt
	M/(VD)	mm × mm	mm	$P/(f_c^{'}A_g)$	Bars, ρ_t	(kN)
CV0A	1.63	—	400	0.3	Ten 25M,3.13%	—
CV0AR	1.63	30×50	150	0.3	Ten 25M,3.13%	10
CV0B	1.63	—	100	0.3	Ten 25M,3.13%	—
CV1	1.63	30×50	150	0.3	Ten 25M,3.13%	9
CV2	1.63	30×50	200	0.3	Ten 25M,3.13%	12
CV3	1.63	30×50	95	0.3	Ten 25M,3.13%	12
CV4	1.63	30×50	150	0.3	Ten 20M,1.88%	12
CV5	0.88	30×50	150	0.3	Ten 25M,3.13%	11
CV6	1.63	30×50	150	0	Ten 25M,3.13%	11
CV7	1.63	30×50	150	0.3	Ten 25M,3.13%	144
CV8	1.63	50×50	150	0.3	Ten 25M,3.13%	13

* Dimensions are perpendicular and parallel to the column longitudinal axis, respectively
** CV0A and CV0B values refer to the centre-centre spacing of conventional internal transverse ties
*** Total longitudinal reinforcement area divided by the gross cross-sectional area of the column

Material Properties, Instrumentation, and Loading Procedure

Columns and footings in the test program were constructed with concrete having a nominal design strength of 30 MPa at 28 days. The concrete contained nominally 20 mm maximum coarse aggregate. The compressive strength of the concrete on the day of each test was determined using standard 300 mm x 150 mm diameter cylinder tests, and the values are reported in Table 2.

Five types of measuring devices were used to monitor the performance of the specimens during experiments: load cells, LVDTs, clinometers, strain gauges, and mechanical dial gauges. Horizontal and vertical loads were measured with load cells on the loading rams. The horizontal displacements of the column were measured with LVDTs at the height of application of the horizontal load.

The direction of loading is defined as follows: the "push" direction relates to column deflection toward the north in the test set-up (see Fig. 1), while the "pull" direction relates to southward column deflection. One complete cycle consisted of a push-half-cycle followed by a pull-half-cycle, starting and ending at the initial vertical column alignment. The axial load, if applicable, was applied initially before any lateral loads and then kept constant throughout the remainder of test. The axial compressive load was established using an axial compression index value of 30% (see Table 1). Lateral loading was applied under force-control up to 75 percent of the force corresponding to the estimated first yield of the longitudinal reinforcement, followed by displacement-controlled loading. Five cycles were implemented at each level of force or displacement. For specimens CV0A and CV0AR, reduced peak force levels were utilized for the initial five cycles due to the large tie spacing.

Experimental Results

Results of the collared reinforced concrete column tests were compared with similar test specimens constructed with conventional internal tie reinforcement. Variations in ultimate lateral force, ductility, and deterioration due to the change of the test parameters were examined.

Table 2. Normalized peak lateral force, displacement ductility, and analytical results.

Specimen	f_c' (MPa)	Experiment V_{max} (kN)			$V_{max,n}= V_{max} / (SQRT(f_c')A_g)$	Average displacement ductility	Pushover analysis V_{max} (kN)	V_{max} Experiment / Analysis
		Push	**Pull**	**Average**				
CV0A	26.3	599	567	583	0.71	4.21	526	1.14
CV0AR	26.6	700	795	748	0.91	6.92	693	1.01
CV0B	26.9	675	730	702	0.85	4.57	754	0.90
CV1	33.3	791	838	815	0.88	5.54	768	1.03
CV2	25.5	701	747	724	0.90	5.31	544	1.29
CV3	22.0	746	774	760	1.01	7.76	701	1.06
CV4	30.8	689	752	721	0.81	9.36	700	0.99
CV5	29.5	1161	1227	1194	1.37	4.39	1027	1.13
CV6	31.5	555	654	604	0.67	5.58	689	0.81
CV7	29.1	889	944	916	1.06	5.77	769	1.16
CV8	27.4	731	804	767	0.92	8.02	708	1.03
						Mean		1.05
						Coefficient of Variation		0.13

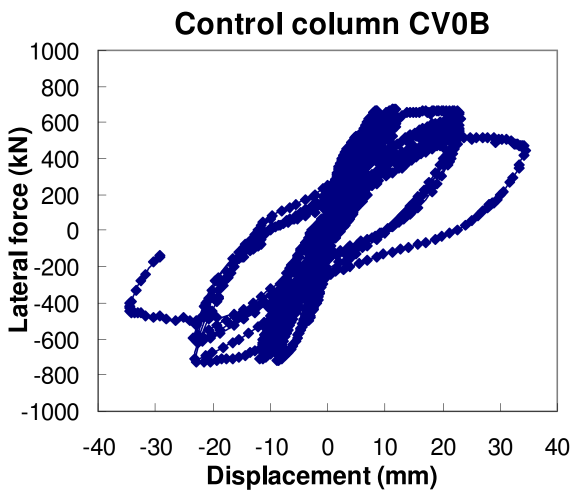
Even though the concrete crushed at the base of the column and some longitudinal reinforcing bars ruptured in tension during the tests, no slippage of the collars was observed. Collars deformed plastically outward to some degree, indicating confinement was being provided to the concrete. The collar legs on the north and south sides (the push and pull direction of the lateral load) deformed more than the west and east sides because of increased concrete dilation at the extreme compression fibre locations. At the end of each test, the steel collars were removed and the column was examined visually. No concrete spalling occurred under the collars, as shown in Fig. 2 which depicts specimen CV8 before adding steel collars, at cycle 2, and at the end of test. The steel collars allowed a more gradual degradation of strength at failure, as compared to the control columns without collars.

Figure 2. Specimen CV8 before adding steel collars, at cycle 2, and at the end of test.

Lateral Force-Displacement Response

Because the axial compressive load was applied through pinned connections (see Fig. 1), the horizontal and vertical components of the axial compressive load contribute increasingly to the moment at the critical section of the column at large lateral displacements. The lateral force discussed below accounts for the geometry, lateral load, and axial load within the system. Hysteresis loops for the lateral force to lateral displacement relationship were constructed for each test. In general, at later loading stages the shape of the loops became increasingly "pinched" toward the origin and the strength and stiffness of the specimen deteriorated at an increased rate. The experiments showed that collared columns have stable hysteresis behaviour with enhanced strength and ductility compared to columns without collars, as shown in Fig. 3 for specimens CV0B (closely spaced ties) and CV1. The collared system also maintained its integrity under repetitive reverse cyclic loading with large displacement amplitudes.

A load–deformation envelope curve for each column was obtained by connecting the peak points for the initial hysteresis loop obtained at each displacement level. The envelope curve indicates the stability of the overall hysteresis behaviour. It can be seen from Fig. 4 that collared columns, as compared to the control columns, exhibited increased peak lateral force values and the peak value was maintained for larger displacements. In general, a stable response was obtained through the use of external steel collars.



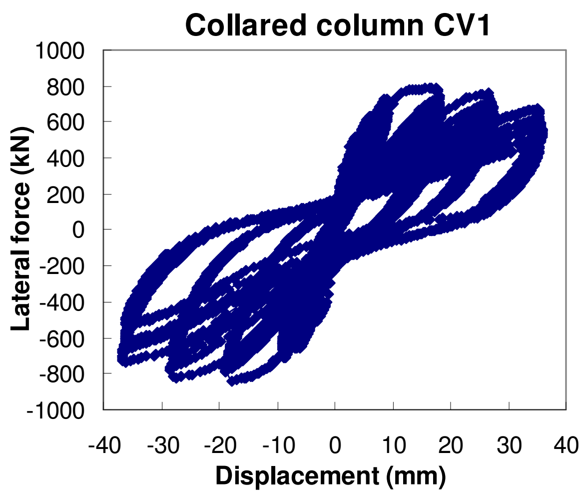


Figure 3. Lateral force–displacement hysteresis loops for test specimens CV0B and CV1.

To account for the variation of the strength of the concrete from specimen to specimen, the maximum lateral force was normalized with respect to the square root of the concrete compressive strength. Using procedures similar to other researchers (*e.g.*, Woodward and Jirsa 1984; Ghee *et al*. 1989; Priestley *et al*. 1994), normalization was conducted as follows:

$$V_{\max,n} = \frac{V_{\max}}{\sqrt{f_c'}A_g} \tag{1}$$

where $V_{\max,n}$ is the normalized maximum lateral force, $V_{\max}$ is the average peak lateral force obtained from the experiment, f_c' is the compressive strength of concrete, and A_g is the nominal gross cross-sectional area of the column, which is the same for all the specimens in the present study.

The normalized shear strength for each specimen is listed in Table 2, along with the peak lateral force applied to each specimen for the push and pull directions. The difference in peak lateral force between the push and pull directions is relatively small. The discrepancy arose partly due to small fluctuations in the horizontal force component from the hydraulic actuator used for introducing the axial load, which were less than 2% for all specimens, and small differences in the amplitudes of the displacement applied.

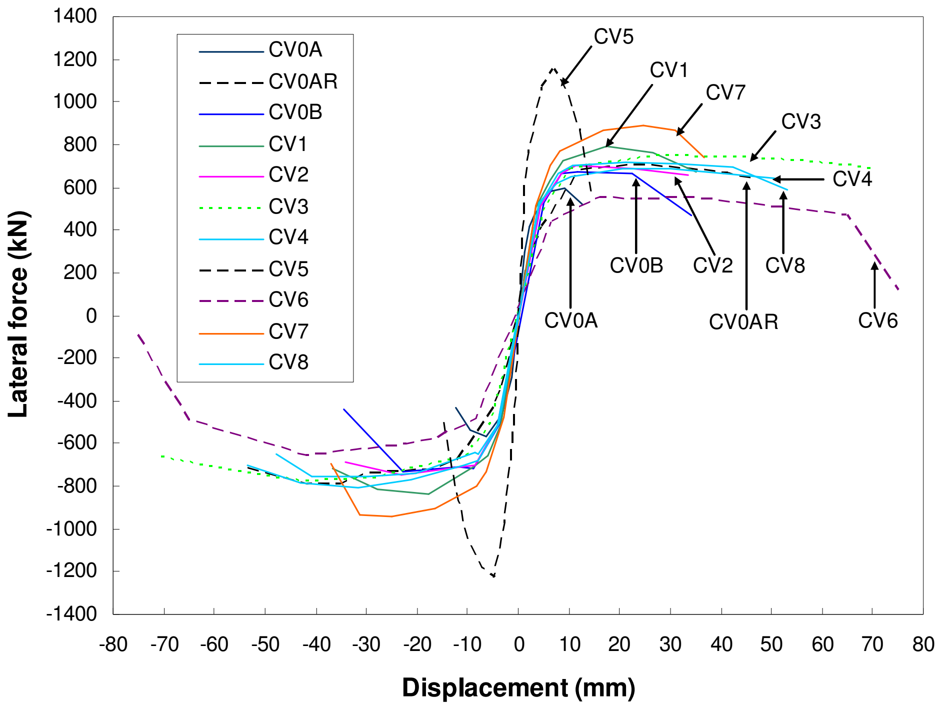


Figure 4. Lateral force–displacement envelope for test specimens.

Comparing the normalized capacity values in Table 2, it is seen that all collared columns having equivalent longitudinal reinforcement experienced peak lateral forces higher than those resisted by both CV0A and CV0B, with the exception of specimen CV6. The slight decrease in capacity of CV6 is attributed to the absence of an axial compressive load, which has been found to be beneficial to the shear capacity for reasonable axial load levels (*e.g.*, Woodward and Jirsa 1984). Neglecting specimen CV5, which had a smaller shear span, the greatest benefits in capacity over the control columns were achieved by prestressing the bolts (CV7) and reducing the collar spacing (CV3). Increasing the flexural stiffness of the collars (CV8) had a relatively small benefit in capacity, supporting the observation based on numerical studies that there is an optimal collar stiffness beyond which the strength benefits tend to diminish rapidly (Hussain and Driver 2001).

Ductility

Displacement ductility, μ, is defined as the ratio of the ultimate lateral displacement of the specimen, Δ_u, to the yield displacement of the specimen, Δ_y. Before conducting the experiments, a flexural sectional analysis was performed to predict the yield strength of each specimen. A value of 75% of the expected yield moment was used as guidance for the initial five force-controlled cycles, along with close visual observation of the load versus displacement curve during the test. This approach is similar to methods adopted by Saatcioglu and Baingo (1999). However, since lower loads were used in conducting the initial cycles for specimens CV0A and CV0AR, in an attempt to avoid possible failure in the initial cycles, the yield displacements for conducting these tests were smaller than they would have been according to the method used for the other specimens. Thus, an adjustment was needed to obtain the corresponding effective yield displacement for the calculation of displacement ductility values. A relatively accurate yield displacement can be determined from an idealized bilinear curve by extrapolating the first cycle of the lateral force versus displacement response linearly until it intersects the second line, which is set equal to the ultimate capacity of the specimen. In reinforced concrete column tests failing in shear, the ultimate displacement is typically defined as the displacement on the descending branch corresponding to the condition when the lateral force resistance drops to between 80% and 90% of the peak lateral force (Saatcioglu and Baingo 1999, Lacobucci *et al.* 2003, Memon and Sheikh 2005). In the current study, the

90% criterion was used. Displacement ductilities in the push and pull directions were calculated separately, with the average ductility value being used for comparisons in this paper.

From Table 2 it can be seen that the collared columns generally have better ductility than the control columns. The collared column with a wide collar spacing (CV2) had less ductility than those with narrower collar spacings (CV1 and CV3). The collared column with a smaller longitudinal reinforcement ratio (CV4) had higher ductility than the collared column with a larger longitudinal reinforcement ratio (CV1). The specimen with the smaller shear span-to-depth ratio (CV5) had less ductility than the equivalent specimen with the larger shear span-to-depth ratio (CV1), and the collared column with pretensioned bolts (CV7) had more ductility than the equivalent collared column with snug-tight bolts (CV1). The specimen with the larger size of collars (CV8) showed a higher ductility level than the specimen with the smaller collars (CV1). Although the axial load index had a significant effect on the lateral force capacity, it had a minimal effect on the displacement ductility, as can be seen by comparing specimens CV6 with no axial load applied and CV1 with an axial load index of 0.3. This phenomenon was also observed by Ahn *et al.* (2000) for column specimens regardless of concrete strength.

Energy Dissipation

The degree of energy dissipation is an important indicator of the performance of the column. Apart from enhancing the strength and ductility, the rehabilitation techniques would preferably also achieve significant levels of energy dissipation. The energy dissipation characteristics of the columns are influenced by various factors including the yield displacement, the axial load, and the number of load cycles applied. The total energy dissipated, which serves as an index of the energy dissipation characteristics of the system, was calculated and compared among the specimens.

The energy dissipated in a loading cycle can be considered as the area enclosed by the lateral force versus displacement hysteresis loop corresponding to that cycle. Wider and more stable hysteresis loops indicate higher energy dissipation. The total energy dissipated is determined by summing the energy dissipated in each cycle. It was found that the collared columns generally had better energy dissipation characteristics than the control columns, CV0A and CV0B, which dissipated 41 kN·m and 163 kN·m, respectively. The reference specimen, CV1, dissipated 295 kN·m. By comparison, the total energy dissipated by the column with a reduced collar spacing, CV3, was significantly higher (775 kN·m), the energy dissipated by the column with a reduced shear span-to-depth ratio, CV5, was lower (157 kN·m), and the energy dissipated by the column with prestressed collar bolts, CV7, was higher (444 kN·m).

Capacity Degradation

One indication of desirable cyclic behaviour is the stability of the hysteresis loops, *i.e.*, the tendency for the loops to achieve the same peak lateral force in subsequent cycles at a given displacement level. Woodward and Jirsa (1984) suggested that the degree of degradation can be quantified as the percentage reduction of peak lateral force from the first to the last cycle at each displacement level. The percentage reduction, although targeted as a comparison of the fifth cycle to the first, was adjusted as necessary for specimens that did not sustain five cycles at a given displacement level. The control columns (CV0A and CV0B) generally exhibited a larger percentage reduction in lateral force capacity (usually more than 20%, even at low displacement levels) than collared columns CV0AR, CV1, CV2, CV3, CV4, CV7, and CV8 (typically less than 20%, even at higher displacement levels). The specimen with a smaller shear span-to-depth ratio (CV5) had a larger reduction than the corresponding specimen with the larger shear span-to-depth ratio (CV1). The specimen without axial compressive load (CV6) exhibited similar degradation to specimen CV1 at low displacement levels, but had significantly higher degradation during the large displacement cycles. This can be attributed largely to the fact that specimen CV6 was intentionally loaded to very high displacement levels—even though the column should have been deemed as failed—in order to study whether collar slippage would occur under severe concrete spalling and crushing.

Specimen CV7 (with pretensioned bolts) showed much less degradation than specimen CV1 (with snug-tight bolts), with fewer and smaller cracks developing in CV7 due to the larger confinement provided to the concrete. Specimen CV2 (with the widest collar spacing) showed a degradation of more than 20% at large displacement levels and about 10% for the initial displacement level, whereas CV3 (with the narrowest spacing) exhibited a reduction of less than 10% for all displacement levels. Specimen CV8 (with the larger collar stiffness) showed slightly less degradation (around 10% for all displacement levels) than specimen CV1 (more than 10% at higher displacement levels). In general, collar configurations that provide higher degrees of confinement slowed the degradation in capacity from initial to later cycles.

Finite Element Analsyis

Finite Element Models

While it is valuable to conduct large scale tests on reinforced concrete columns with collars, it is not practical to consider a full range of geometric dimensions, loading conditions, or other parametric variations that may be encountered in practice. Therefore, three-dimensional finite element models were developed to simulate the response of collared reinforced concrete columns under shear-dominant cyclic loading using the commercial finite element program ABAQUS (ABAQUS 2003). ABAQUS/Explicit was utilized, incorporating a nonlinear explicit dynamic formulation. This allowed an efficient solution to the highly nonlinear quasi-static problem, which would otherwise experience severe numerical convergence difficulties using an implicit solution strategy. The rate of loading, defined through a smooth step amplitude function, and the corresponding mass scaling factor were selected based on the energy balance and the economy of the solution time for the whole system to facilitate the analysis.

The footing of each column was heavily reinforced to avoid failure prior to the column failure, which was confirmed through physical observation during the experiments. Thus, only the column was modelled in the finite element analysis and the nodes on the bottom surface of the column were restrained against all translation. Mesh sensitivity studies were conducted to find a reasonable mesh that would provide accurate results within a minimal computation time. Eight-node continuum elements with reduced integration (C3D8R) were used to model the concrete. Reduced integration elements were used to eliminate the concerns about shear locking under moment and to reduce analysis time, while still producing results similar to elements using full integration. A comparison study was conducted to determine the suitability of different element types to represent the internal reinforcement, including truss elements (T3D2) and beam elements (B31). It was found that models using beam elements, which take into account dowel action, provided better agreement with the test results than models using truss elements. Perfect bond between the reinforcement and concrete was assumed. Multi-point constraint (MPC) type BEAM was imposed between concrete nodes and the adjacent nodes on the steel collars. These constraints prevent relative movement between the steel collar and the concrete. This technique also ensured that the outward deformation of the steel collars was consistent with the lateral expansion of the concrete without slippage, even under extreme displacement. This was consistent with observations from the experiments that no relative sliding was found between the collars and the concrete.

The concrete damaged plasticity model was utilized, including both concrete compression hardening and tension stiffening definitions. Poisson’s ratio for the concrete was taken as 0.2. Steel reinforcement and the external collars were modelled as an isotropic elasto-plastic material satisfying the von Mises yield criterion. Material properties were established based on relevant cylinder or coupon test specimens from the experimental program. The axial compressive load, if applicable, was applied to the top steel bearing plate of the column through a uniformly distributed pressure and was kept constant during the whole analysis. Monotonic pushover loading in the “push” direction was conducted using a displacement boundary condition approach, by increasing the lateral displacement at the central node of the lateral loading plate. This technique permitted simulation of the post-peak behaviour. The lateral displacement was targeted to equal the maximum displacement level attained in the experiment. The pretension force applied to the steel collar bolts in specimen CV7 was generated in ABAQUS by applying a negative temperature change to the bolt as the initial load step of the analysis.

Verification of Finite Element Models

Finite element analysis and experimental results were compared in terms of lateral force versus lateral displacement relationships and peak lateral force. For comparison of the measured peak lateral force with the pushover analysis results, the peak lateral force in the push direction from the hysteresis curve envelope was used for consistency. Table 2 shows that the mean and coefficient of variation of the ratios of experimental–to-numerical peak lateral forces were 1.05 and 0.13, respectively, indicating that the finite element model captured the peak force with reasonable accuracy. The numerical simulations yielded accurate predictions of peak lateral forces for specimens CV0AR, CV1, CV3, CV4, and CV8. However, they overestimated the peak lateral forces for specimens CV0B and CV6 and underestimated them for specimens CV0A, CV2, CV5, and CV7. In general, somewhat stiffer lateral force–displacement responses were observed from the analysis compared to the experimental results, although the overall trend and shape of the lateral force–displacement curve from the analysis was similar to the experimental responses at a reasonable accuracy. Stiffer analytical predictions compared to experimental results were also reported by Cofer *et al*. (2002) in finite element analysis of reinforced concrete columns using ABAQUS. The higher modelled stiffness may result from possible imperfections in the test specimen that were not taken into account in the models, and discrepancies arising from variations in concrete material properties throughout the specimen. Degradation of response due to the cyclic loading in the test was not considered when performing pushover numerical analysis. Further work is ongoing to use this validated procedure to simulate reverse-cyclic loading and conduct additional parametric studies related to the behaviour of the collared reinforced concrete column under shear-dominant loading.

Conclusions

Based on both the experimental and analytical study of the behaviour of collared reinforced concrete columns under shear-dominant loading, along with previous research results, this rehabilitation technique is shown to be effective for seismic rehabilitation of deficient reinforced concrete columns.

No slippage of the collars was observed during the tests of collared columns, even when severe spalling and crushing of the cover concrete took place between them. This feature is beneficial for seismic rehabilitation. Collared columns showed ductile response with stable hysteresis loops and exhibited significantly improved energy dissipation capacity over the control columns without collars. The repaired specimen demonstrated significantly improved ductility, deformability, energy dissipation, and enhancement in strength over the original control column. These observations validate the feasibility and effectiveness of this rehabilitation technique.

Comparisons between a pushover finite element model developed using ABAQUS/Explicit and the experimental results showed that the model offered a reasonably reliable analytical tool that was capable of capturing the major performance characteristics of collared reinforced concrete columns under shear-dominant loading. Although the analytical models generally overestimated the column lateral stiffness in the initial part of the test, it was shown that the proposed model was capable of predicting the peak lateral force with reasonable accuracy. For the columns studied, the mean experiment-to-analysis capacity ratio was 1.05, with a coefficient of variation of 0.13.

Acknowledgments

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