Title

Numerical modeling and seismic retrofit for shear failure in reinforced concrete columns

Authors
Zimmerman, Reid
Panagiotou, Marios
Mahin, Stephen

Publication Date
2013-03-25

Report No.
UCB/SEMM-2013/03

Structural Engineering
Mechanics and Materials

NUMERICAL MODELING AND SEISMIC RETROFIT FOR SHEAR FAILURE IN REINFORCED CONCRETE COLUMNS

By

Reid B. Zimmerman
Marios Panagiotou
Stephen Mahin

March 2013

Department of Civil and Environmental Engineering
University of California, Berkeley

Table of Contents

List of Tables

List of Figures

ABSTRACT

This report investigates two related subjects, numerical techniques and seismic retrofit solutions for shear-critical reinforced concrete columns, utilizing test data of a reinforced concrete column with widely spaced transverse reinforcement. It primarily focuses on both the analysis method of nonlinear trusses and the retrofit option known as supplemental gravity columns. The bulk of the research has been broken up into seven investigations including sensitivity of material properties on computed response, engagement of supplemental columns in compression, detailing of supplemental columns to avoid composite action, and comparison between a distributed plasticity, fiber beam-column model and a nonlinear truss model. It is concluded that the nonlinear truss method can be effectively applied to analysis of shear-critical concrete columns and that supplemental gravity columns represent a viable retrofit alternative which have not been fully explored in either academia or professional structural engineering practice.

Keywords: reinforced concrete, structural columns, shear failure, seismic retrofit, nonlinear analysis, truss method, shear-flexure interaction

INTRODUCTION

Objectives

The objective of this study is two-fold. The first is to employ nonlinear truss elements in prediction of the behavior of reinforced concrete frame members in which shear failure is expected. Results of lateral force versus lateral displacement from a column test under constant axial load with widely spaced shear stirrups and given testing protocol will serve as a basis for assessing the accuracy of the analytical model [1]. Sensitivity of the concrete material parameters in tension will be conducted in addition to a comparison with a less-refined distributed plasticity, fiber beam-column model.

The second objective encompasses discussing and applying a seismic retrofit technique for shear-critical reinforced concrete columns achieved through the introduction of supplemental gravity support columns. Professional structural engineering practice has, on several occasions, employed this retrofit method in order to maintain the transfer of gravity load in the presence of significant degradation in lateral force capacity. An example calculation, investigation of the point at which supplemental columns become active and detailing to avoid tension, all serve to link the response of shear-critical concrete columns in the laboratory with a solution utilized in practical applications.

Topic Significance

The two focuses of this report collectively tie together aspects of earthquake engineering, structural engineering research and professional practice in high-seismic areas. The analytical model of the reinforced concrete column using the nonlinear truss method demonstrates how shear-critical members can be assessed under cyclic loading conditions. Though engineers understand axial-flexural interaction well, results of this simulation will hopefully help hone their intuition about the less adequately understood shear-flexural interaction problem. In conjunction, the seismic retrofit technique consisting of supplemental gravity supports represents one of the profession's answers to an existing building stock comprised of shear-critical structural members. It is the authors' intent that this report will be useful in advancing the knowledge of earthquake engineering for students, academia and professional practice.

Literature Review & Background Information

The method of nonlinear truss elements has been used by several researchers to accurately estimate the response of reinforced concrete elements under cyclic, laboratory testing. It is also referred to as a nonlinear strut-and-tie method in the existing literature. Although many inconsistencies and inadequacies of this modeling procedure are regularly pointed out, the consensus appears to be that the nonlinear truss method can capture expected behavior with sufficient accuracy when utilizing reasonable material properties and geometric discretization. No applications of the nonlinear truss method to columns with light transverse reinforcement were found during the literature review stage.

Park and Eom investigated short and slender coupling beams, cantilever beams, short and slender walls, and cantilever columns [2]. They discovered that the strut-and-tie method captured the global force-displacement relation well for nearly all structural element types except after the point at which rebar buckling or fracture was observed. Agreement between test and numerical results required that the concrete in compression include the effect of transverse strain to accurately estimate web crushing failure in shear-dominated members such as short walls.

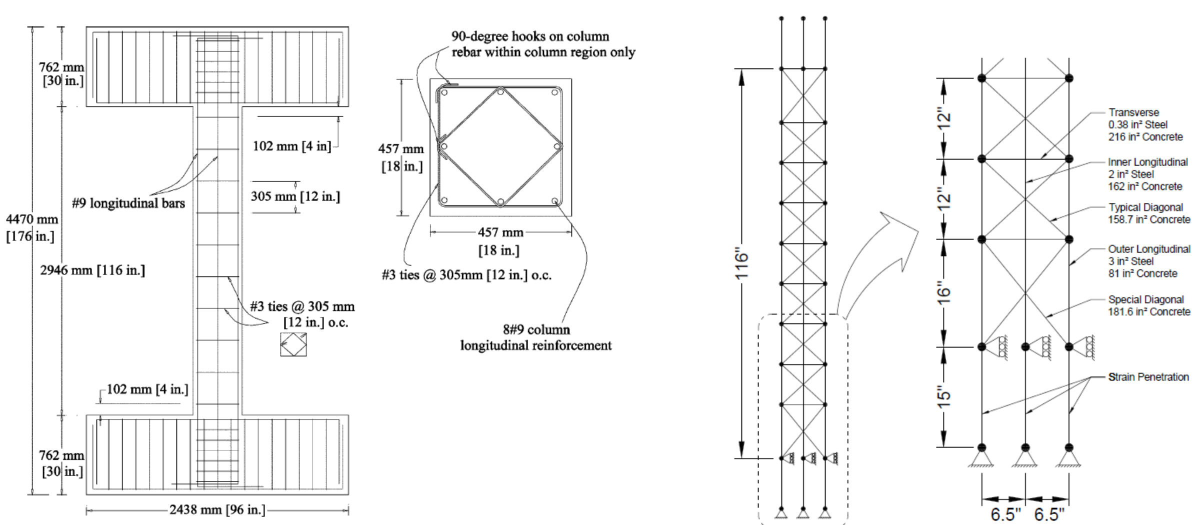
Panagiotou and Restrepo focused their research on five case studies of tested reinforced concrete walls [3]. In addition to incorporating the effect of transverse tension on compressive strength, they included size effects in modifying the softening curve of concrete in tension and a short parametric study on the angle of concrete diagonals. Finally, the authors concluded that the nonlinear truss method could be applied to several diverse situations encountered in professional practice and research such as walls with openings.

Extensive review of the pertinent literature by the lead author of this report in conjunction with personal correspondence between the lead author, several professors and practicing engineers uncovered no published documents on supplemental gravity support columns [4]. In addition to supplemental gravity support columns these devices are also often called crutches or preshoring. It is known that they have been successfully implemented in several projects on the University of California Berkeley campus, namely Wurster and Tolman Halls, by Rutherford + Chekene, a structural and geotechnical engineering firm in San Francisco. As a result of the lack of publicly available information on this topic, this report intends to record some of the important points the lead author has discovered through interacting with the structural engineering community.

APPROACH & INVESTIGATION OUTLINE

Model Description

A detailed diagram of the specimen elevation and cross section can be seen in Figure 1a, including relevant dimensions and reinforcement. This corresponds to Specimen 1 of Reference 1 and will hereafter be referred to as the test specimen. The light amount of transverse reinforcement is clear, shown spaced at 12" o.c. in Figure 1a with the first tie occurring 4" from the inner face of the beams. Figure 1b shows the analytical version of the test specimen utilized in this report where the three dimensional reality has been transferred into a two dimensional model under the assumption of plane stress. Open Software for Earthquake Engineering Simulation (OpenSees) provides the software platform for all analyses conducted in this report.



(a) (b)

Figure 1. Overview of (a) test specimen [1] and (b) analytical model

The location of truss elements was chosen based on a conceptual understanding of force flow through the column in a typical strut-and-tie method. See Figure 1b for specified dimensions and areas. Placing the outer and inner longitudinal trusses at the center of the outer and inner longitudinal reinforcing bars, respectively, intends to capture where the tension force centroid will occur after the concrete has cracked significantly, thus losing much of its tensile strength. With the transverse bars providing intuitive locations for the transverse trusses and pure concrete diagonal trusses between them, a complete model is formed. Due to the close offset between the first transverse reinforcing bar and the beams, it seems clear that the diagonal formed between the second transverse bar and the beam would carry most of the imposed shear. Thus, the first transverse bar has been left out of the analytical model. To account for strain penetration into the beams, shown to contribute significantly in Reference 1, and assuming that the steel strain varies linearly to zero at the outer edge of the beams, an additional truss half the beam depth is included both top and bottom. Half the beam depth with constant strain in the steel results in the same displacement or pullout as the full beam depth with a linearly varying strain.

Steel areas were assigned to the longitudinal trusses as either two or three longitudinal bars depending on whether they were outer or inner trusses. Transverse trusses included two #3 ties plus an equivalent area for the two #3 ties that form the diamond. Longitudinal concrete was distributed between the longitudinal trusses in a proportion such that half of the concrete area resided in the middle truss. Similarly, the transverse trusses are composed of a concrete area spanning the width of the member multiplied by the distance between transverse steel. An accepted methodology proposed by Panagiotou and Restrepo in Reference 3 was adopted to determine the concrete diagonal areas. The strain penetration elements are composed of the typical longitudinal steel area but have a larger concrete area to account for the significant stiffness of the beams under vertical compression. Concrete and steel trusses work in parallel in both the transverse and longitudinal directions.

Loading Protocol

The loading of the analytical model has been selected to match the force and displacement history experienced by the test specimen. Initially, the vertical force of 150k in compression is applied and held constant. Afterwards, the pseudo-static displacements shown in Figure 2 are imposed at the top, center node of the truss, see Figure 1b. Constraints are placed on the top six nodes such that they all translate horizontally together while the top three additionally translate vertically as one.

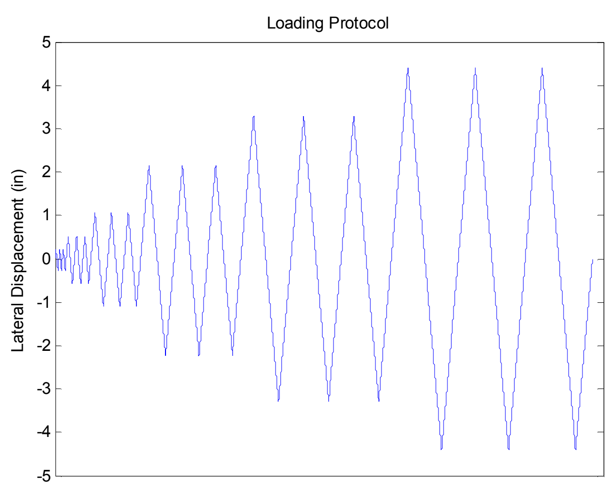


Figure 2. Imposed lateral displacements comprising loading protocol

Material Models

The concrete model shown in Figure 3 was selected from the available materials in OpenSees to capture strain softening in tension, in compression and varying tangent modulus during reversed cycles of loading. All parameters in Table 1, except fpc which was taken from Reference 1, were estimated using accepted correlations with compressive strength supplemented by knowledge of concrete under uniaxial loading conditions. For example, the stress and strain at transition in tension, ft0 and epst0 respectively, were selected to approximately account for tension stiffening and size effects. Tension stiffening, present in the longitudinal concrete but not in the diagonals and only slightly in the transverse concrete, helps to explain why the transition stress in these elements differ.

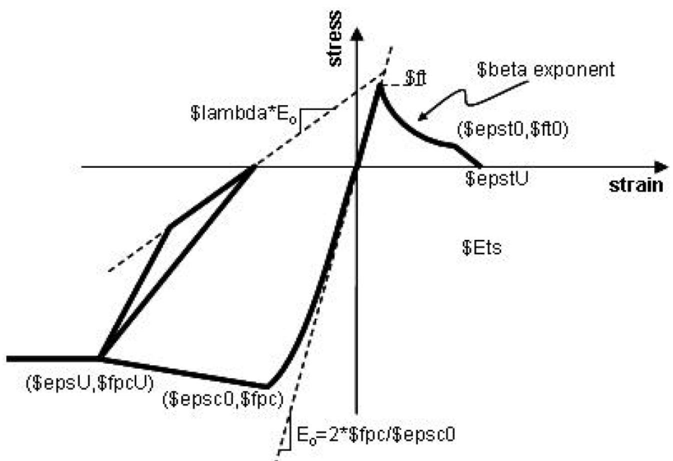


Figure 3. Concrete cyclic stress-strain diagram [5]

Table 1. Concrete material properties

Parameter	Units	Diagonal	Transverse	Longitudinal
fpc	ksi	- 3.06	-3.06	-3.06
epsc0	in/in	-0.002	-0.002	-0.002
fpcU/fpc	%	5	5	5
epsU	in/in	-0.007	-0.007	-0.007
lamda	-	0.2	0.2	0.2
ft	psi	220^{+}	220	220
epst	10^{-5} in/in	7.23	7.23	7.23
ft0/ft	%	5*	5*	12
epst0/epst	-	17*	17*	12
epstU/epst	x10^{3}	1	1	1

* properties modified in Investigation 2

$^{+}$ property modified in Investigation 3

The Guiffre-Menegotto-Pinto (GMP) model has been selected to represent reinforcing steel in both the transverse and longitudinal trusses. Reference 1 again provided the yield stresses from available tests, which differed between the #9 and #3 reinforcing bars, see Table 2. An estimate of the strain hardening ratio is proposed as 2% of the initial steel elastic modulus and an R-value of 18 was chosen. The R-value affects the transition from elastic to strain-hardening behavior and has been selected based on recommendations for reinforcing steel.

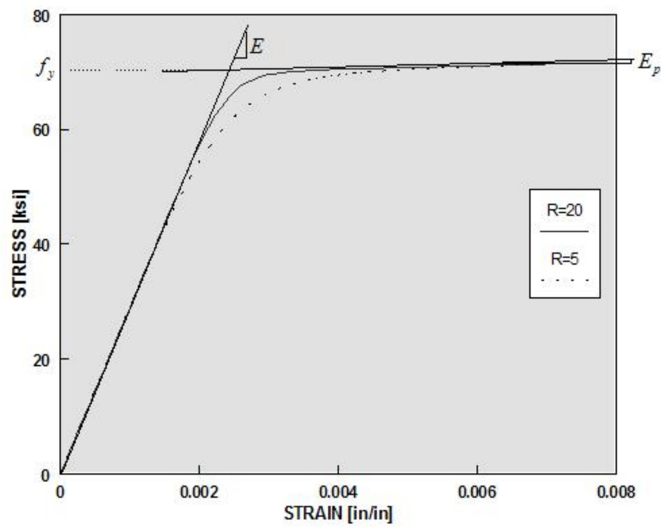


Figure 4. Steel stress-strain diagram [5]

Table 2. Steel material properties

Parameter	Units	Longitudinal	Transverse
Es	ksi	29000	29000
fy	ksi	63	69
b	%	2	2
R	-	18	18

RESULTS

Investigation Study 1

The purpose of Investigation 1 is to match the tested lateral force-displacement relation using the analytical model described in the previous section of this report. A graph comparing the two results can be seen in Figure 5. In general, the model captures yielding of the column and gradual strain hardening with fairly impressive accuracy. It further successfully predicts the displacement at the onset of softening and even provides an approximate intermediate, between peak and residual, strength at -3 in. The cycles exceeding 3 in are not captured well, yet this is an expected result since significant spalling and deterioration of concrete are beyond the refinement of the model.

Initially, the analytical model over-predicts the tangent stiffness due to double-counting of concrete areas. In the Model Description section above, concrete areas for the longitudinal, transverse and diagonal concrete were chosen to overlap substantially. This poses a problem when the concrete has not yet reached its tensile stress capacity, otherwise known as uncracked conditions, but diminishes quickly as the concrete becomes nonlinear, softens and cracks. Observations of the tangent slope to the curve after column yielding supports this conclusion. Another result of overlapping concrete is that the force capacity at yield will be overestimated as seen in Figure 5 at approximately 1 in displacement. Yet again, it is observed that this effect becomes much less pronounced as the model nears ultimate force.

The true power of the nonlinear truss model is not its ability to capture behavior up to yield but to explore response near failure. It is emphasized here that the analytical model provides an exceptionally accurate prediction in the vicinity of softening initiation as seen between 2 and 3in in either direction of Figure 5. The large reduction in capacity at around 3in is attributable to transverse concrete trusses reaching their tensile capacity and dropping to their low transition stress. As this process occurs, the transverse steel trusses accept more of the force and proceed to yield and strain harden. Additional cycling causes the transverse trusses to elongate substantially, especially the two located 16in from the inner face of the beams. This pronounced transverse strain would realistically reduce the compressive capacity of the diagonal concrete in compression and possibly cause crushing in these trusses. Since the analytical model does not account for compressive strength dependence on transverse strain, the analytical and test results are bound to drift apart when transverse strains become severe.

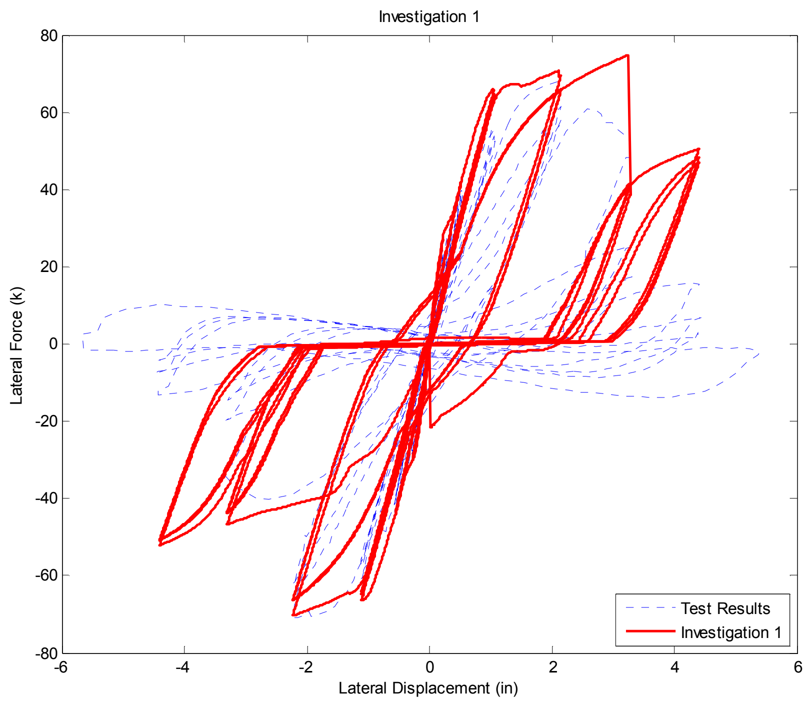


Figure 5. Lateral force versus lateral displacement for Investigation 1

Investigation Study 2

In an effort to bound the results and develop a sense of the sensitivity of the model to the estimated material parameters, the transition strain and stress in the diagonal and transverse concrete was varied as shown in Table 3. These variables were chosen after semi-rigorously investigating which inputs to the model were most uncertain in addition to which ones produced the greatest change in output. In this section, all other material properties not listed in Table 3 were left at their original values from Tables 1 and 2. Figure 6 presents the four cases considered in Table 3 compared with the results from Investigation 1.

Table 3. Modified concrete properties

Case	2A	2B	2C	2D
ft0/ft	5%	5%	10%	10%
epst0/epst	10	25	10	25

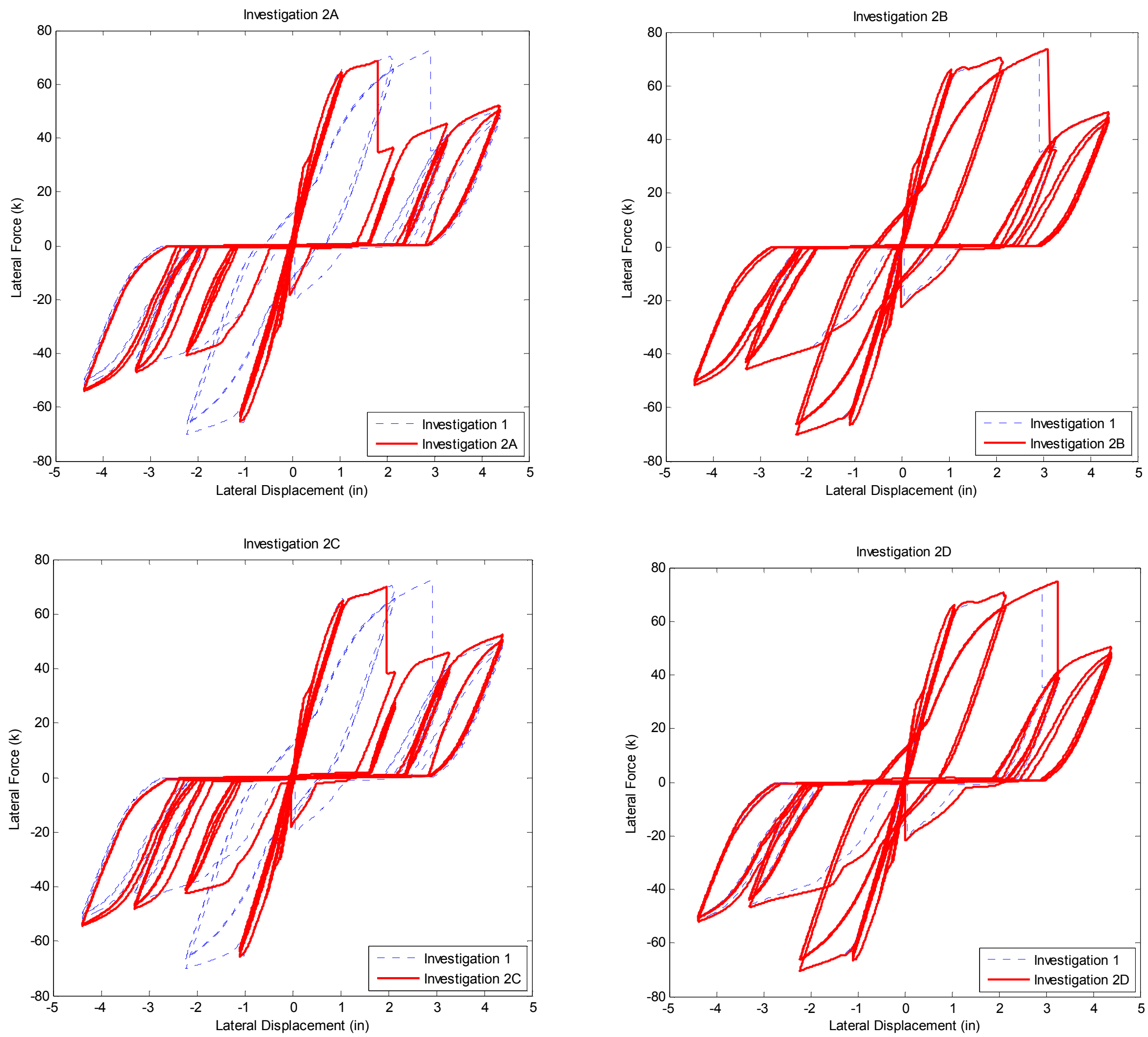


Figure 6. Lateral force versus lateral displacement for Investigations 2A-2D

Case 2A and Case 2C show nearly indistinguishable results from each other but observable difference between Investigation 1. They exhibit reduction in strength earlier, thus under predicting the displacement at softening, despite almost exactly matching Investigation 1 both up to and some time after strength loss. The strain at transition in tension therefore has a profound impact on the displacement at softening while having very little effect on strength outside of this crucial region of the response. From these two graphs, it appears as though the results are fairly insensitive to the transition stress of concrete in tension, at least in the practical range for pure concrete, such as the diagonals, or very lightly reinforced portions, namely the transverse trusses.

Case 2B and Case 2D display some resemblance in that they are closer to the results from Investigation 1. The increased strain at transition in tension appears to have no effect, with the slight difference from Investigation 1 too small to be of any consequence. Another interesting observation is that for Case 2A and 2C, the transition stress had almost no effect for a constant transition strain. Case 2B and 2D similarly reinforce this point as they exhibit nearly identical performance despite a varying concrete transition stress in tension. In general, however, the parameters investigated here are both difficult to quantify and element-size dependent such that an exact value cannot easily be selected.

Investigation Study 3

This investigation intends to assess the sensitivity of the computed response to the peak tensile stress of the diagonal concrete. To explore this concept more specifically, the diagonal concrete peak tensile stress was changed to its residual value in Table 1 in order to approximate a no-tension concrete case. No other material property was altered relative to Investigation 1. Figure 7 shows the results of this investigation plotted over those of Investigation 1.

A quick comparison highlights the fact that concrete in the diagonals contributes significantly to lateral force resistance. It is seen that the column begins to lose strength at a lower lateral force and lateral displacement pair than when concrete in tension is included. When the hysteretic curve again rejoins that of Investigation 1, it exhibits a higher lateral force resistance, likely owing to greater strain hardening with the concrete force taken instead by steel. Finally, the cyclic loops when the column is pushed to negative displacements demonstrate very little energy absorption capacity and essentially zero lateral resistance for several inches of displacement between reversals. This is likely because the diagonal that would normally be under compression still resides under tensile strain due to elongation of the transverse steel. The conclusion of this investigation is that tensile strength can contribute significantly in lightly reinforced members and cannot be neglected without major loss of accuracy.

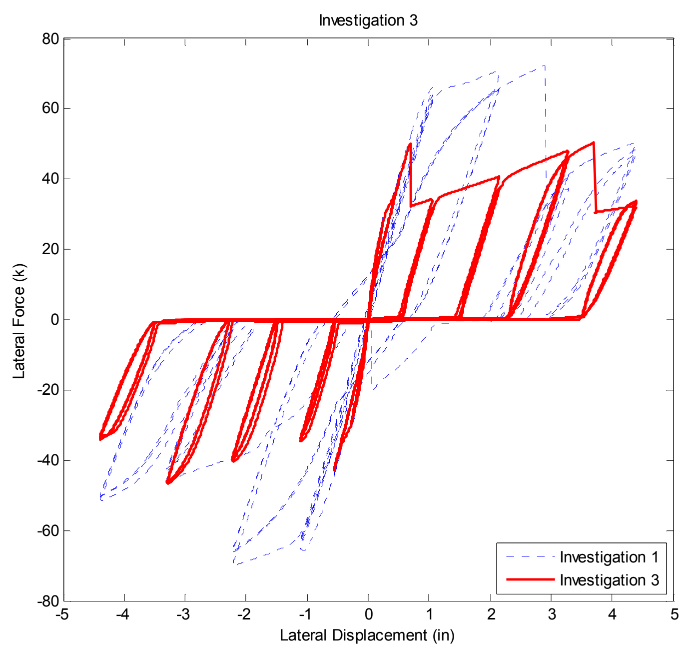


Figure 7. Lateral force versus lateral displacement for Investigation 3

Investigation Study 4

Due to the lack of published information on supplemental gravity columns it seems necessary to show their basic design and detailing as practiced by professional engineers. The intent of this investigation is thus to introduce these concepts and establish a common ground on which to build further discussion. A short section at the end of this investigation demonstrates a simple calculation to determine an approximate size for a supplemental gravity column consisting of a rectangular Hollow Structural Section (HSS).

Since these supports are typically installed one on either side of a shear-critical column, half of the load is assigned to each HSS post. Additionally, the current state of practice is to design the HSS sections to resist the maximum gravity forces under all ASCE 7-05 load combinations and to assume they have no lateral capacity. The issue of how the supplemental columns interact with the building's lateral resistance is considered in Investigation 5 and Investigation 6. Although conservative to use combinations such as

1.2D+1.6L, it seems more in the spirit of Load and Resistance Factor Design (LRFD) to only consider the vertical load expected at the time of an earthquake. This would correspond to a combination such as 1.2D + 0.5L. If the engineer seeks an additional safety margin, a knowledge factor could be introduced that increases the gravity load calculated depending on the degree of precision and information present about the distributed and concentrated dead and live loading within the building. Although such presentation appears to simply complicate the problem while arriving at the same solution as the simpler procedure, it more closely adheres to a transparent, consistent design methodology.

Figure 8 presents a typical elevation of supplemental gravity columns in a retrofit project. The HSS posts are shown on either side of a potentially shear-critical column, aligned between story levels. Aligning them vertically reduces the introduction of moment in the beam-slab system that would result from force eccentricity. Additionally, posts on one side of the reinforced concrete column are permitted to be offset a specified distance in order to avoid relocating existing utilities typically present along column lines. A design consideration worthy of noting is the vertical placement of supplemental gravity columns. When HSS posts are introduced at a specific story, they should be continued at every story below until either reaching a foundation or another structural element capable of transferring the gravity load to the earth. This presents a limitation of this retrofit technique for structures of significant height where shear-critical members are present not only in the lower floors. More advanced analysis may be able to substantiate relaxation of this requirement if it can be shown that lower floors are capable of resisting the imposed gravity load with sufficient redundancy.

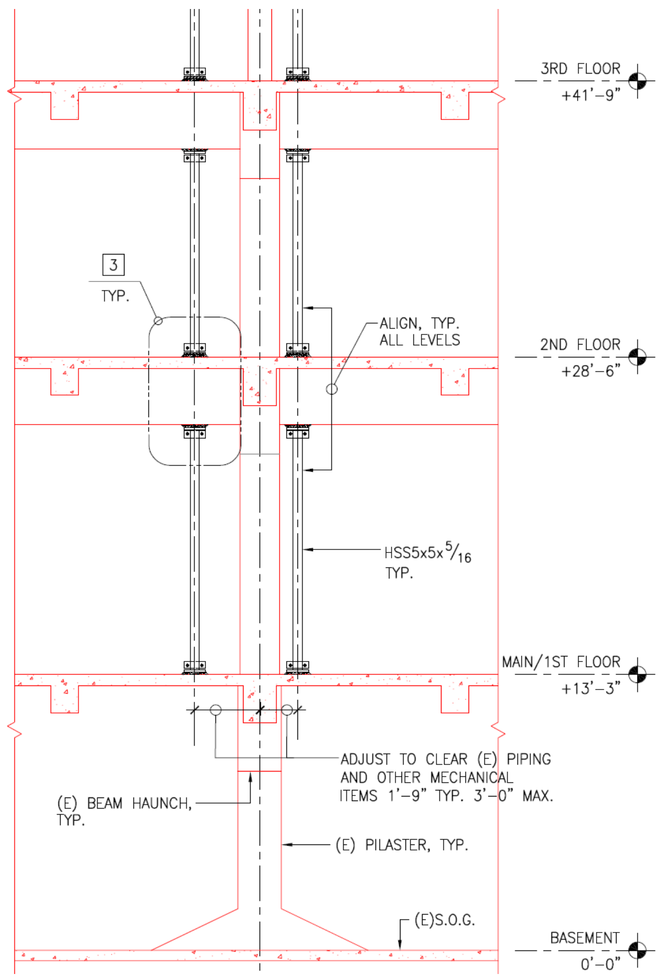


Figure 8. Typical HSS post elevation [4]

Preliminary HSS Post Design for Test Specimen Column

$P_u = 1.2D + 0.5L < 1.2(D+L) = 1.2(75k) = 90k$

Choose HSS 4x4x1/4

$$\frac{KL}{r} = \frac{(1.0)(116in)}{1.52in} = 76.3 \qquad \left(\frac{KL}{r}\right)_{\lim} = 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000ksi}{46ksi}} = 118.3$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29000ksi)}{76.3^2} = 49.14ksi$$ AISC 360-05 Equation E3-4

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{46ksi}{49.14ksi}}\right](46ksi) = 31.09ksi$$ AISC 360-05 Equation E3-2

$$\phi_c P_n = \phi_c F_{cr} A_g = (0.9)(31.09ksi)(3.37in^2) = 94.3k$$ AISC 360-05 Equation E3-1

Investigation Study 5

As discussed in Investigation 4, supplemental gravity columns are often designed for a specified force and not analyzed considering displacements. Additionally, since they are installed during a seismic retrofit, a gap or "slack" may exist between the grouted baseplate and the beam or slab to which it is attached. This investigation uses the test results from Reference 1 to determine the point at which the supplemental columns engage in axial load transfer for the hypothetical case that HSS posts with a certain gap were installed. Although this column was a laboratory specimen, it was selected and loaded to represent conditions of shear-critical columns in building systems during earthquakes. Thus it can be used to approximately represent the real behavior of such a column.

Three cases are presented in Figure 9 corresponding to an assumed gap between the supplemental column and the framing beams of 1/64", 1/32" and 1/16". These represent the amount of vertical displacement needed to engage the supplemental gravity columns in compression. It is important to note that these gaps are measured after the gravity load has been applied to the test specimen in order to replicate conditions during seismic retrofit where deflections due to gravity have already occurred. Figure 9 displays that the 1/64" and 1/32" gap occur quite close to one another, insinuating that once the column begins to displace vertically downward, the process becomes accelerated. The vertical displacement versus lateral displacement graph shows this most clearly.

Another important note brought to light by this investigation is the point on the hysteretic curve at which the supplemental columns engage. One might question when a shear-critical column begins to lose its vertical load-carrying capacity in relation to its loss of lateral force capacity. More crucial for the seismic application of supplemental gravity columns, however, is whether this retrofit technique will disturb the lateral response of the structural member. If the supplemental columns remove the axial load from the concrete column too soon, the parent column would lose shear resistance, by shear-friction reasoning, leading to worsened lateral behavior. Here it is seen that even for a very small gap, 1/64", the supplemental columns remain inactive in carrying gravity load until the reinforced concrete column has lost much of its lateral strength. Thus a lateral analysis of a building ignoring the supplemental columns appears to be acceptable as long as column axial failure in compression is not predicted.

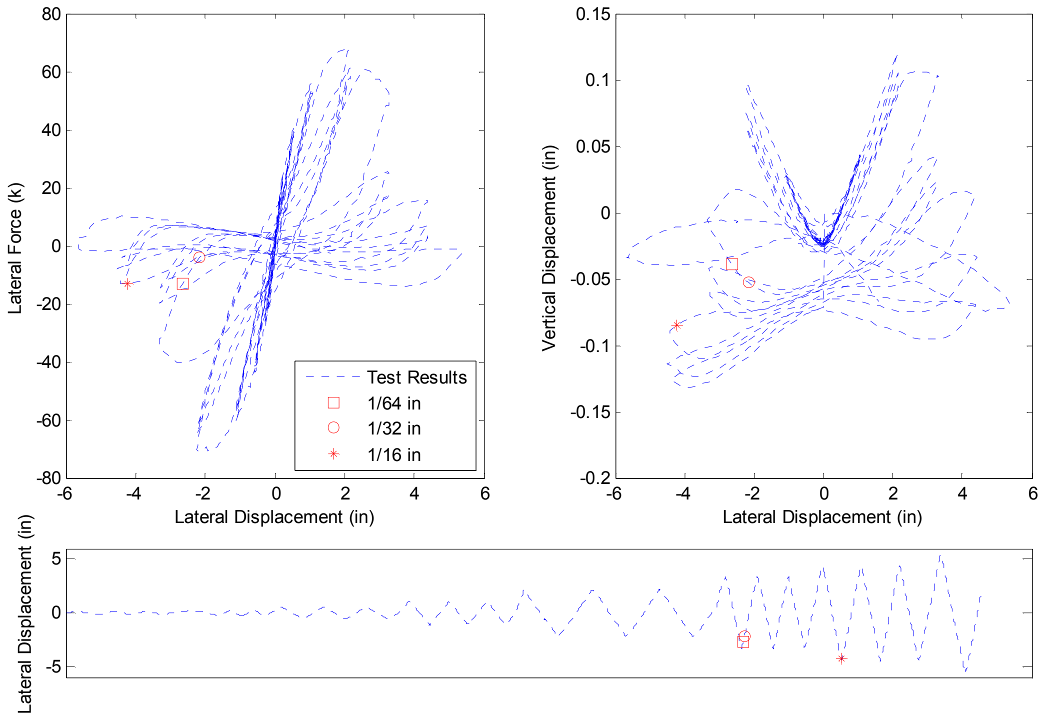


Figure 9. Engagement of HSS posts in compression for an assumed initial gap

Investigation Study 6

An additional way a supplemental gravity column could interact with its parent reinforced concrete column is through composite action. As shown in Investigation 5, the HSS posts remain inactive in compression until the shear-critical column has lost much of its lateral capacity and begins to shorten. This test, however, assumed that the beam or framing member above and below the column prevented rotation of the respective ends. A condition similar to that assumed could exist, for example, where a reinforced concrete column's length was shortened due to a parking garage ramp with a high relative stiffness. Another condition, where the ends of the reinforced concrete column are allowed to rotate by the presence of a similarly flexible member top and bottom, also represents a likely situation though. In this case, the HSS posts see additional axial displacements resulting from a rotation multiplied by the distance from the reinforced concrete column centerline. What makes this situation different from what has been investigated earlier is that these forces are opposite in sign on either side of the column. Therefore one HSS post may see upward displacement while its counterpart is compressed at a point during the earthquake record.

(a) (b)

Figure 10. Detailing to avoid composite action using (a) telescoping HSS and (b) isolator

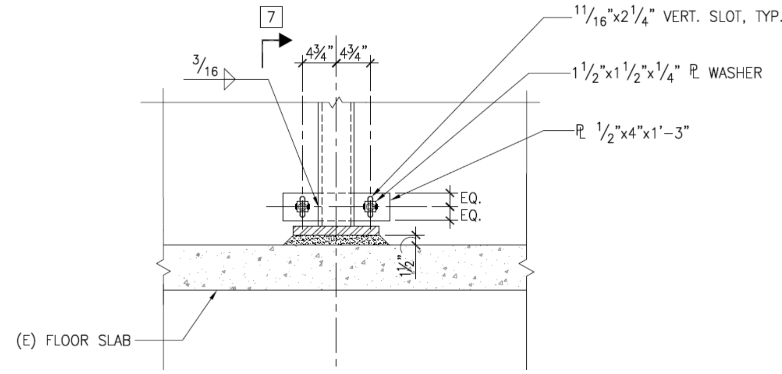


Figure 11. Detailing to avoid composite action using vertically slotted holes

Designs that seek to reduce the tensile capacity of the HSS posts have been implemented on several projects. Vertically-slotted holes, shown in Figure 11, can serve this purpose when it is possible to attach the column to an adjacent vertical element. When such a situation does not exist, a detail such as in Figure 10a also represents a constructible alternative. Here, the HSS 4x4x1/4 posts selected in Investigation 4 are fitted inside a short section of a larger HSS. This larger HSS is then welded to a baseplate, grouted and fixed to the floor slab above and below. When the inner HSS is subjected to elongation it simply slides upward within the "sleeve," thus avoiding any tensile force from being transferred. Proper inspection would be necessary to ensure that the inner HSS was sufficiently flush with the bottom baseplate to avoid an unnecessarily large gap as explained in Investigation 5.

Another solution to avoiding tension in the supplemental gravity columns was implemented in Wurster Hall on the University of California Berkeley campus by Rutherford + Chekene. The connection of interest is shown in Figure 10b. A small isolator was placed between the floor slab and the supplemental gravity support which served two purposes. First, it reduced the amount of moment transferred to the supplemental support and secondly, it essentially provided a tensile axial release. Isolators may therefore represent an economical alternative when gravity loads are high, such as in Wurster Hall, and thus the supplemental support sections become large.

Investigation Study 7

This section of the report focuses on analysis of the test specimen using a distributed inelasticity, fiber beam-column element with section discretization as shown in Figure 12. The intent is thus to compare the performance of the nonlinear truss model with a more well-established technique used in research and professional practice for nonlinear analysis. Fiber beam-column elements, unless specially modified, do not directly account for moment-shear-axial interaction. Thus we expect that the nonlinear truss method, which explicitly includes shear failure, should match the test results more precisely. It must be noted that the degree of refinement of the fiber model is not up to the same level as the nonlinear truss model owing to the fact that this report focuses primarily on the nonlinear truss method.

The core concrete has been subdivided into a five by five grid of rectangular fibers with the cover concrete split into quadrilaterals so as to match the core fiber vertices. Each longitudinal reinforcing bar fiber has been placed in the steel's actual position within the cross-section as determined in the Model Description section. All material properties and loading histories match those of Investigation 1 for longitudinal steel and concrete. As mentioned previously the transverse and diagonal properties cannot be accounted for in a typical fiber beam-column element as utilized here. Figure 13 presents the results for Investigation 7A on the left. Investigation 7A does not include the effect of strain penetration. Investigation 7B, shown on the right in Figure 13, attempts to include strain penetration by inserting a beam-column element at either end which has been calibrated to give the correct rotational stiffness.

It is clear, especially from Investigation 7B, that with greater time and energy spent, the fiber beam-column model could adequately capture the stiffness and strength of the test specimen up to yield. Less clear, however, is the ability of this analysis type to assess strength degradation in shear. After the column yields in flexure, the tensile steel begins to harden and thus controls the shape of the lateral force-lateral deformation curve as seen most clearly on the left in Figure 13 and to some extent on the right. Neither case therefore seems capable of losing strength with further cycling since, as demonstrated in Investigation 1, this would require some consideration of the transverse steel and transverse and diagonal concrete. The last point to make is that the fiber model, especially ignoring strain penetration, shows fuller loops than seen in the test specimen and thus unacceptably overestimates the energy dissipation capacity.

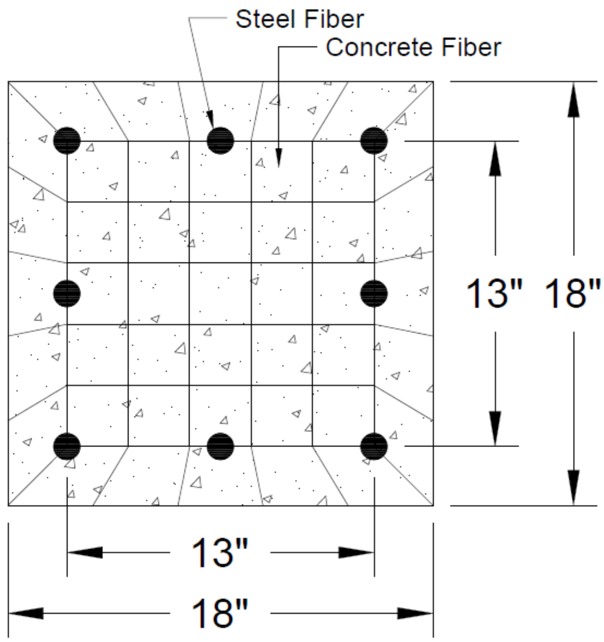


Figure 12. Fiber section discretization

Calibrated Strain Penetration Element Derivation

$\theta_1 + \frac{1}{2}\phi L = \theta_2$ Based on linear curvature diagram resulting from assumption of linear strain diagram

$\theta_1 = -\frac{\theta_2}{2}$ Using linear-elastic theory of uniformly prismatic element with moment release at end 1

$\theta_{sp} \cong \frac{\Delta_{sp}}{0.85d} = \frac{1}{0.85d}\left(\frac{1}{2}\varepsilon_s (30in)\right) \qquad \theta_2 = \frac{L}{3}\phi \cong \frac{L}{3}\left(\frac{\varepsilon_s}{0.85\,d}\right)$

$\theta_{sp} = \theta_2 \Rightarrow L = \frac{3}{2}(30in) = 45in$ Thus two elements were placed at each end to include strain penetration

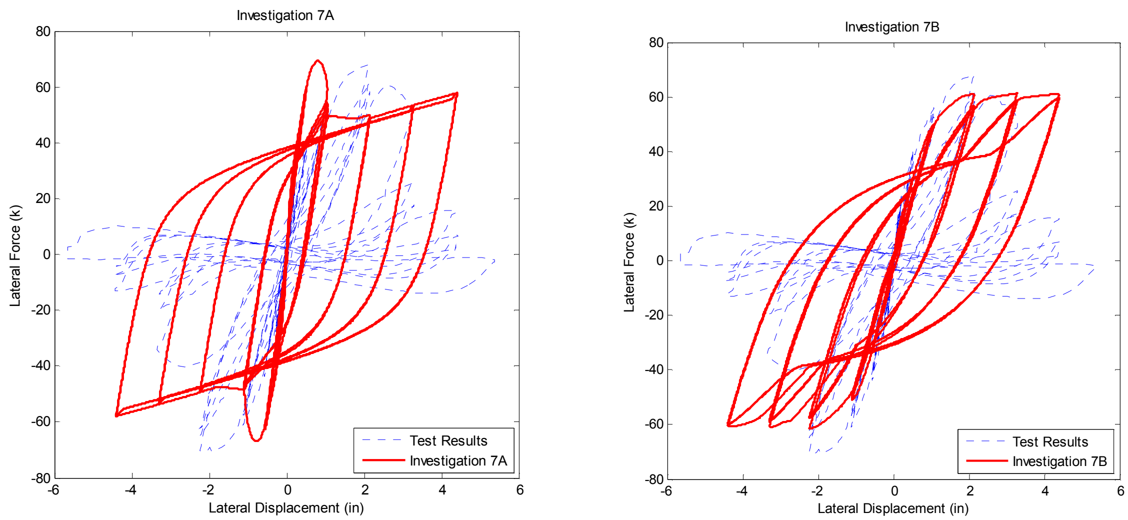


Figure 13. Lateral force versus lateral displacement for Investigations 7A and 7B

DISCUSSION

Numerical Modeling

Despite the precision of results in Investigation 1, the nonlinear trusses are still missing several crucial aspects that keep them from reflecting a more perfect reality. Of first importance is the effect of transverse tensile strain on concrete stress in compression. Transverse in this context refers to the direction normal to a specific truss and not to the direction normal to the column length. As the strain in the direction normal to a truss increases in tension one would expect the compressive capacity of the concrete at a given strain parallel to the truss to decrease. This phenomenon can be modeled in OpenSees through a specialized element which the lead author, unfortunately, did not have sufficient time to implement. By reducing the stress of concrete in compression, the material modulus would also similarly decrease and thus may help to reduce the overestimation of lateral force-lateral displacement stiffness up to yielding in flexure. Additionally, the reduced compressive stress would lower the moment at flexural yielding which would again improve the model's precision. After yielding and early into the strength degradation region, the analytical model was shown to match the tested response quite well. As imposed displacements increased, however, the results began to differ substantially. This may be owing to the fact that transverse tensile strains become exceedingly large in this region of response and are not accurately captured in the current model. The effect of transverse tensile strain on the global response could therefore produce significant improvements in the nonlinear truss method for such shear-critical columns.

Investigation 3 showed most clearly that concrete in tension contributes significantly to the behavior of this specimen and likely for most columns with light transverse reinforcement. Currently, the transverse steel provides the chosen locations for the horizontal trusses. This selection is based on intuition assuming that the transverse steel resists the majority of force in the tension tie of a strut-and-tie model. However, it was observed in Investigation 1 that up to the point of strength degradation, the transverse concrete resists a significant amount of that force. It subsequently cracks and initiates strength degradation of the global response. One might then rightfully question whether placing additional trusses, such as equally spaced vertically between transverse steel trusses, would improve the response up to strength degradation and better capture the initiation point. In the current model of Investigation 1, concrete in each transverse truss reaches its cracking stress at the same point since they all carry an equal force and have equal area. This leads to a model that is very sensitive to the properties of concrete in tension. By adding more transverse concrete trusses which are not located at the same level as the transverse steel, the precision up to and at strength degradation may be enhanced.

Seismic Retrofit

The reasoning behind the use of supplemental gravity columns stems from the belief that gravity causes the collapse of structures. Although the earthquake loading reduces the lateral resistance through cyclic degradation and damage, field reconnaissance of past earthquakes has tended to show that as long as the system which resists vertical forces remains intact, most occupants are able to leave the building safely. More specifically, buildings tend to fall vertically downwards during earthquakes rather than, with a few notable exceptions, tipping over. If structures collapsed solely as a result of loss of lateral resistance, one would expect more buildings to end up like a fallen domino set where the center of each floor was offset from the one below and above it while laying on the ground. Instead "pancaking" or one floor on top of another on the ground is more commonly observed.

Despite the discussion just presented, there are several reasons why supplemental gravity columns are an inadequate solution when applied alone. First and seemingly contradictory to what was just said before, supplemental gravity columns do nothing to prevent the loss of lateral resistance. This can be important in situations such as weak story formation where inelastic displacements concentrate due to non-uniform softening in one or several stories of a structure. For example, if a building that possessed a majority of shear-critical columns in one specific story was solely retrofitted with supplemental gravity columns it would still tend to develop a weak story at this height. Since that story drift would be amplified, P-Delta destabilizing effects would be more pronounced. The supplemental gravity columns have been designed not to interact with the lateral system and would therefore not be able to resist the P-Delta effects. This would likely lead to a story collapse. From this brief explanation it can be seen that supplemental gravity columns do not represent a panacea for all shear-critical structural members if used alone. On the other hand, they are thought to be highly effective when combined with other retrofit strategies such as fiber-reinforced polymer or steel jacketing of reinforced concrete columns.

CONCLUSIONS & FUTURE DIRECTIONS

Above all else, the intent of this report was very much accomplished. Although it is clear that more research is possible on both of these topics, the findings discussed here expand the current breadth of knowledge on shear-critical reinforced concrete columns in high-seismic regions. A few conclusions from the report bear repeating:

1. The nonlinear truss method can be applied to shear-critical reinforced concrete columns with quite accurate results when based off assumptions grounded in theory and enhanced by intuition.
2. Unlike modeling of squat, shear-dominated walls, the nonlinear truss method for shear-critical reinforced concrete columns can be fairly sensitive to material properties, especially those defining concrete in tension.
3. Despite the lack of research on this topic, supplemental gravity columns represent a simple and effective method of seismically retrofitting shear-critical reinforced concrete columns through supporting gravity load when the parent reinforced concrete column becomes unable.
4. Supplemental gravity columns appear not to engage in compression until the parent reinforced concrete column has lost a significant amount of lateral capacity, thus remaining inactive at lower levels of damage.
5. Proper detailing to avoid both moment and, more difficultly, tension transfer in supplemental gravity columns poses a challenge that can be addressed through several techniques.
6. General distributed inelasticity, fiber beam-column elements cannot capture shear interaction without special modification. Such modification seems unnecessary in the presence of the nonlinear truss method's accuracy.

Although the outline of this report intended to separate the discussion of the reinforced concrete column behavior from that of the supplemental column retrofit strategy, it soon became clear that addressing one was insufficient without the other. Earthquake engineers play a unique role in society as they are required not only to ask difficult questions about public safety and structural reliability but to somehow answer them. Therefore understanding the behavior of a shear-critical reinforced concrete column only addresses a portion of earthquake engineering responsibility. The research on the supplemental gravity columns strives to tackle a little more. Yet the findings and conclusions addressed in this report represent only a beginning, a step on which further inspection will hopefully benefit.

ACKNOWLEDGEMENTS

The authors would like to acknowledge Professor Halil Sezen from Ohio State University for providing electronic files of the tested specimen response for comparison in this report. Joe Maffei and Karl Telleen from Rutherford + Chekene also deserve recognition as great resources for information and design considerations about supplemental gravity columns.

REFERENCES

[1] Sezen, Halil and Jack Moehle, "Seismic Tests of Concrete Columns with Light Transverse Reinforcement," *ACI Structural Journal*, American Concrete Institute, Vol 103, No 6, p842-849, 2006.

[2] Park, Honggun and Taesung Eom, "Truss Model for Nonlinear Analysis of RC Members Subject to Cyclic Loading," *Journal of Structural Engineering*, ASCE, Vol 133, No 10, p1351-1363, 2007.

[3] Panagiotou, M., Restrepo, J. I., Schoettler, M. and Kim, G., "Nonlinear Cyclic Truss Model for Reinforced Concrete Walls," *ACI Structural Journal*, American Concrete Institute, Vol 109, No 2, p205-214, 2012.

[4] Maffei, Joe and Karl Telleen, Personal Correspondence, Office of Rutherford + Chekene, San Francisco, CA, April 7th, 2011.

[5] McKenna, Frank, "OpenSees Command Manual," Accessed April 15th, 2011. <http://opensees.berkeley.edu/wiki/index.php/Command_Manual>

[6] American Society of Civil Engineers, "ASCE 7-05: Minimum Design Loads for Buildings and Other Structures," Reston, VA, Jan. 2006.

[7] American Institute of Steel Construction, "AISC 360-05: Specification for Structural Steel Buildings," 13th Edition, March 2005.