Earthquake Performance of Non-engineered Construction in Nicaragua

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SUMMARY:
Low-rise concrete and masonry structures can provide excellent seismic resistance when they are designed by an engineer, are made of quality materials, and are built by well-trained workers in conformance with building codes. Unfortunately, this is not the way many of the structures are being built. Property owners themselves are building low-rise non-engineered structures, paying little attention to building codes or seismic resistance. Adding to the problem, building with concrete and masonry construction it is possible to have relatively long spans, large openings, and irregular shapes all of which impact their earthquake performance. These non-engineered buildings are deceptive because they seem safe, they perform well under gravity loads and they do not sag or lean. In this study, several typical concrete and masonry Nicaraguan low-rise structures were modeled and subjected to seismic loads. These models were then manipulated to determine which low-cost changes will have the greatest effect on earthquake performance.

Keywords: Non-engineered Construction, Earthquake Performance, Concrete, Infill Masonry Bricks

1. INTRODUCTION

In recent times, the trend in many parts of the world has been to build with higher strength materials such as concrete and masonry instead of lower strength materials such as earthen materials. Property owners in many regions are often building low-rise non-engineered structures, paying little attention to building codes or seismic resistance. This was not previously possible with low-strength earthen materials; however, the non-engineered concrete and masonry buildings are deceptive because they seem safe, they perform well under gravity loads and they do not sag or lean. The buildings are also relatively heavy which adds to the illusion of safety. However, there often is no consideration given to lateral loads, the type of loads they will experience during an earthquake.

In Nicaragua there are many types of low-rise residential buildings. Some are vernacular and made with local materials, while others are made of engineered products seen elsewhere around the world. In this study, several typical Nicaraguan concrete low-rise buildings were modeled and subjected to seismic loads in order to predict their earthquake response. These models were then manipulated to determine which low-cost changes will have the greatest effect on earthquake performance. The two most common types of construction were chosen: concrete shear wall systems and concrete frames with infill masonry bricks.

2. ANALYSIS OF NON-ENGINEERED CONCRETE SHEAR WALL SYSTEMS

The use of concrete in Nicaragua becomes more common every day. The design of concrete buildings is a well-studied area of structural engineering. However, the area of non-engineered (and perhaps poorly-detailed) concrete deserves further study.

To investigate less than perfectly designed and constructed concrete buildings, a typical Nicaraguan concrete building was chosen from the town of Rivas (Figure 2.1). This structure seems typical, the size is average, the openings are representative, and it is a simple design made from concrete.

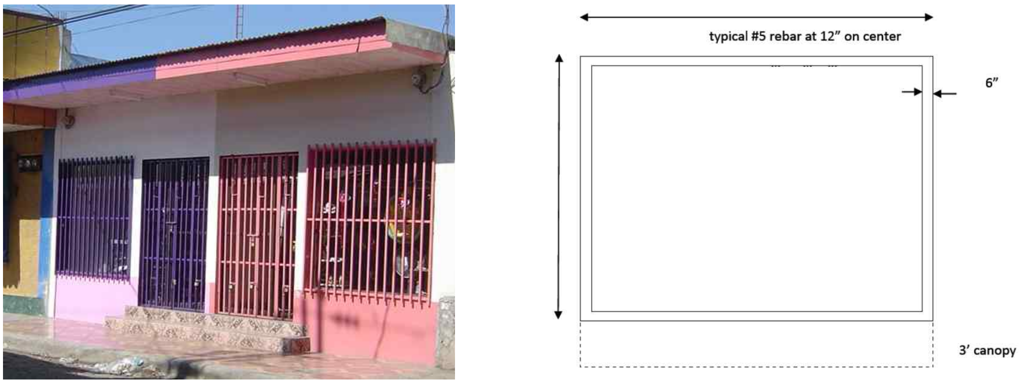


Figure 2.1. Typical concrete building chosen for analysis

Several variations of this building were analyzed. The first variation was the building without windows, doors, or a canopy to serve as a control structure that could be analyzed as a baseline. The variations were analyzed to determine not only how these variations effect the structural adequacy of the building, but also to extend the analysis to buildings of other geometries so as to determine how the geometry changes the structural adequacy.

The variations also included:

- The building as it is seen (as an actual building in Nicaragua).
- The same building without a canopy (only windows and doors). This variation was analyzed to get a better understanding of how openings affect the overall performance of the building.
- The same building but longer in one direction (rectangular). This variation was analyzed to determine how the shape of a structure affects the performance and also to apply conclusions to rectangular buildings.
- The same building but taller. This variation was analyzed to determine how the height of a structure affects the performance and also to generalize conclusions to taller buildings.
- The same building with increased steel.
- The same building with increased concrete strength.

The last two variations were analyzed to determine which might be more beneficial and therefore which would be worth spending additional resources, if any.

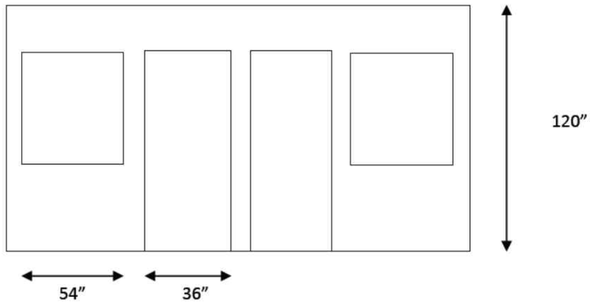
2.1. Geometry

The building was scaled from the photograph to the extent possible, but the geometry that was unknown was assumed. For instance the front of the building was scaled from the picture. The common height of a door was used and then that length determined the scale used to measure the rest of the front of the building. All geometry behind the front face was assumed based on experience entering this type of building. The geometry that was analyzed is shown in Figures 2.1 and 2.2.

2.2. Material Properties

The material properties were assumed as indicated in Table 2.1. Steel is generally produced consistently around the world, so steel properties were not reduced. However, concrete is mixed locally and its properties can vary greatly, so concrete properties were reduced accordingly. The modulus of elasticity (E) for concrete was calculated from $E = 57{,}000\sqrt{f'_c}$, and the shear modulus (G) was estimated as $G = 1{,}187$ ksi, where G was calculated from $G = E/[2(1 + \nu)]$ and ν (Poisson's ratio)

= 0.2 for concrete. Because the concrete Poisson's ratio is generally taken as 0.1 to 0.2, while steel is 0.27 to 0.3, the v of 0.2 was used for the combined system to account for the steel in the concrete.



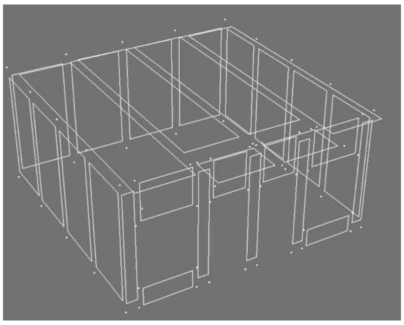


Figure 2.2. Front view of concrete building and nonlinear 3D model with canopy and openings

Table 2.1. Material Properties Used in Concrete Model

Property	Value used for steel	Value used for concrete	Value used for shear wall
F_u (Ultimate strength)	50 ksi	2.5 ksi	0.2 ksi
E (Modulus of elasticity)	29,000 ksi	2,850 ksi	
G (Shear modulus)		1,187 ksi	2,000 ksi

The ultimate strength of the inelastic shear material (F_u) was calculated to be 5% of compressive strength which gives F_u = 0.125 but this value was increased to 0.2 to account for steel reinforcing in concrete. Also the shear modulus (G) was assumed higher than for plain concrete to account for the steel.

The total building weight was estimated to be 90,000 lbs without a canopy and 94,500 lbs with the canopy. The calculations for the building weight are shown in Table 2.2. Based on concrete density of 150 pcf, 6 in. concrete walls and roof, the concrete would weigh 75 psf.

Table 2.2. Calculations Used to Determine the Weight of the Concrete Building

Member	Calculation	Weight
Walls	(75 psf)(20' x 10')(4)	60,000 lbs
Roof	(75 psf)(20' x 20')(1)	30,000 lbs
Canopy	(75 psf)(3' x 20')	4,5000 lbs

2.3. Nonlinear Model

The models were created by setting up a system of nodes and then creating elements between the nodes. To create the nodes, all the dimensions were laid out on a grid system and the points that create the geometry were specified. Elements were then defined as regions between the nodes as seen in Figure 2.2. Once the elements were created, they were assigned the material properties listed in Table 2.1. The five models were created in much the same fashion. The models with openings were created with additional nodes and smaller elements to simulate the openings. The selected resulting frames are shown in Figure 2.2.

The weight of the building was applied evenly to the top nodes. The forces appear upward because that is the only direction the arrows will display in Perform 3D. The direction is determined by the negative sign.

The models with the canopy had an additional 4,500 lbs distribute to the structural model. When this load was applied over the windows the structure failed under dead load. Failure in this sense means the deflections were large and went into the non-linear zone and therefore the program stops applying load. This model did not have any additional reinforcement beams over the windows and this might have been too harsh an assumption to consider the load above the windows. This assumes the load path applied the roof load above the windows without any additional header beams above the window, which is probably unrealistic to assume, so the load was moved to nodes away from the windows to the nodes creating the jambs of the windows.

For the pushover analysis lateral load was applied at two corner nodes. Simulations of each of the buildings were made with the lateral load distributed to all the nodes of two sides of the building and the results were similar. So for sake of simplicity, the loads were applied at the two corners.

The building was fixed at its base at all node locations. During an early analysis the building was fixed only at the corners and this allowed in-plane bending in the walls and gave results that did not agree with hand calculations, so the model was fixed at intermediate node locations as shown, to better simulate the actual connection to the foundation.

The 4 corners of the roof plane were tied together to create a diaphragm. Not all roof nodes were tied together to create a diaphragm because buildings in Nicaragua are not always well tied to their roof diaphragms and this connection creates a model that is closer to the actual condition of these structures.

2.4. Nonlinear Analysis

Prior to the nonlinear analysis the following mode shapes in Table 2.3 were determined.

Table 2.3. Period of Vibration for Modes 1 to 4

Model #1	Period	Description of mode shape
1st period of vibration	0.1378 sec	Vertical deformation
2nd period of vibration	0.1378 sec	Lateral deformation
3rd period of vibration	0.1373 sec	Torsional deformation
4th period of vibration	0.1373 sec	Shrink and swell
Model #4 (with canopy and openings)		
1st period of vibration	0.7552 sec	Vertical deformation
2nd period of vibration	0.7552 sec	Lateral deformation
3rd period of vibration	0.5713 sec	Torsional deformation
4th period of vibration	0.1367 sec	Shrink and swell

The first and second periods are identical, from this it seems the building is likely to be excited laterally and vertically almost at the same frequency. Also notice the period increases greatly for the structure when the canopy and openings are added, as the structure becomes much more flexible.
The selected, deflected shape can be seen in the following image:

The object of the pushover analysis is to determine performance points, which are usually defined in terms of drift ratios, and these performance points are then correlated to static loads. This method gives several (usually 3) static loads for which a building can be expected to respond at different levels of performance. These levels of performance describe the post-earthquake damage state that remains. Immediate occupancy suggests the building will have only minor architectural damage and will be fully functional after an earthquake. Life safety implies the building will require architectural repairs but will remain safe. And collapse prevention implies the building is on the verge of collapse and is not safe. FEMA356 suggests the following performance drift ratios for reinforced concrete buildings:

- Immediate occupancy – negligible
- Life safety - 0.005
- Collapse prevention – 0.02

These limits do no relate well to the model. The model fails before it reaches the collapse prevention limit, suggesting the limits are too large for this non-engineered concrete structure. Taking the more generalized approach, the pushover curve for model #1 was chosen as the standard curve to set the values of immediate occupancy, life safety, and structural stability. On this curve the roof displacement at the general first yield point was determined and set as the point of Immediate Occupancy (IO). Also the roof displacement at general collapse (or loss of stiffness) was chosen as the collapse prevention (CP). Then the point half way between IO and CP was set as life safety (LS). These points were then set as the performance points for all the variations of the concrete building models. The limits are given by:

- Immediate occupancy – 0.0005 (occurs at 550 kips)
- Life safety - 0.0023 (occurs at 580 kips)
- Collapse prevention (occurs at 590 kips)

The representative pushover curve, with the performance points overlaid, is shown in Figure 2.3. The models can then be compared by holding the same performance points for each of the models. The results are summarized in Table 2.4.

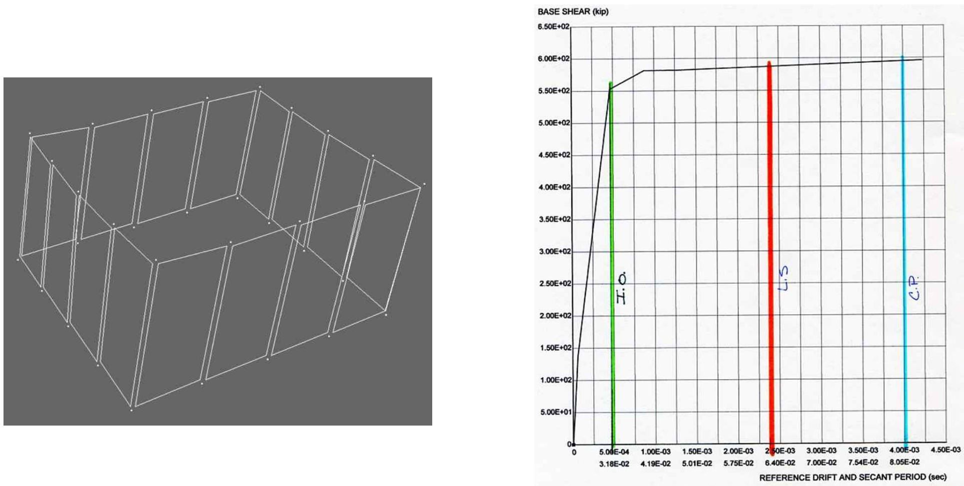


Figure 2.3. Model #1 – Mode shape for the 1st period of vibration and pushover curve

Table 2.4. Load at Performance Points for Each Model

Model	Load at immediate occupancy	Load at life safety	Load at collapse prevention
#1	550 kips	580 kips	590 kips
#2 (with windows)	80 kips	270 kips	310 kips
#3 (with canopy and windows)	85 kips	240 kips	300 kips
#4 (taller)	260 kips	575 kips	590 kips
#5 (with windows)	310 kips	725 kips	740 kips

There are several things worth noticing in this table. First, the doors and windows dramatically reduce the load at immediate occupancy. The reduction is due to the loss of material stiffness in the shear walls. They are however necessary, but it would be best if they are not all located on one wall. This reduces greatly the shear capacity in this wall and creates a weak link created by a reduction in shear strength in that wall. It would increase structural capacity if the openings could be distributed better throughout the building. It would also improve structural performance if the roof load was supported by the sidewalls instead of the weak front walls. However it is most convenient to span the slab in the direction of the front wall since the steel in the roof could continue past the front wall to create a canopy. It would take more effort to ensure the load path was directed to the sidewalls instead (e.g., as shown in Fig. 2.4).

Second, the taller building (12' tall rather than 10' tall) had a lower capacity at IO but almost the same capacity at CP. This leads the conjecture that within some average the height of a floor is ultimately not very important in determining the structural capacity. The taller wall deflected more quickly, which is what one would expect. The increased deflection and reduced load at immediate occupancy show that a building with taller walls will sustain more architectural damage and require more repairs after an earthquake. However, ultimately the shear walls performed similarly and the load at collapse prevention would be similar.

The longer building (40’ long rather than 20’ long) had some reduced capacity early in the pushover curve but had increased capacity at life safety and collapse prevention. This seems reasonable because this is a shear wall system and the strength of shear walls has increased with the greater length. However the shear wall’s length has doubled and the capacity has not doubled, so it is not a proportional increase in capacity, but the general conclusion is that more shear walls is better than less shear capacity.

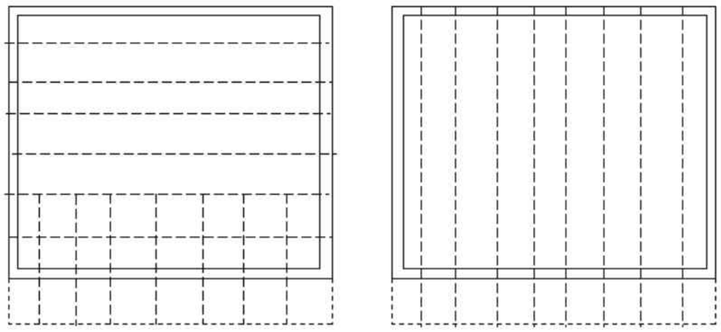


Figure 2.4. Possible reinforcement options

Increasing the strength of the concrete increased the performance of the building at both immediate occupancy and collapse prevention more so than adding additional steel. However, adding steel adds ductility gives the occupants more warning before the building collapses.

2.5. Possible Improvements

To build a concrete building in Nicaragua that will perform better during an earthquake, this study makes the following recommendations:

- Height should be restricted to reduce deflections and cracking.
- Building openings (windows and doors) should not be concentrated in one area, where they may create a weak wall or soft story. It is best if windows and doors are not excessive in size and are well distributed around the building.
- Columns should have sufficient ties. Insufficient ties have been observed on jobsites on many occasions. It seems that ties are not considered structural elements, by local construction personnel, but their purpose in Nicaragua instead is to merely hold the longitudinal reinforcement in place.
- Special attention should be paid to inter-element ties. Structural elements should be well tied to one another. For example walls and should be well tied to the foundation and roof.
- Higher strength concrete increases the performance of the building both at immediate occupancy level and collapse prevention level of performance. This requires having strict mixing and pouring standards and also using high quality sand and aggregate and avoiding the use of local pierda pomez aggregate.

3. ANALYSIS OF NON-ENGINEERED CONCRETE FRAMES WITH INFILL BRICKS

In recent years, concrete frames with brick infills have become a popular method of construction in Nicaragua. These types of buildings have proven to hold up better than earthen buildings in earthquakes and are relatively easy to construct. However, building with these modern materials without engineering advice can lead to dangerous building designs. The establishment of basic guidelines regarding concrete reinforcement, maximum spans, maximum heights and detailing would help minimize such dangers.

3.1. Assumptions

Paulay and Priestley (1992) reported that confined masonry has four failure modes: 1) Tension in the column resulting from overturning moments; 2) Sliding shear failure; 3) Compression failure of the diagonal strut; and 4) Flexural or shear failure of the column. Of these four failure modes, two are a result of the columns that surround the masonry (tension in the column, and flexural or shear failure of the column) and two are a failure of the masonry. Using Perform 3D, the frame that surrounds the masonry was analyzed and the masonry infill was analyzed as a strut. The strut capacity was determined as the lower capacity of the two failure modes (sliding shear failure or compression failure of the diagonal strut).

Paulay and Priestley's formula for sliding shear failure simplifies to:

$$R_s = \frac{0.03 f'_m}{1 - 0.3\left(\frac{h}{l}\right)} d_m t \qquad (3.1)$$

where d_m is the diagonal length (d_m = 72.11 in the example shown in Figure 3.1), t is the thickness, h is the height (h = 40 ft in this example) , and l is the length (l = 60 ft in this example). The effective width of the diagonal strut is 0.25(d_m) = 18" and the thickness is 4 in. This gives $R_s = 10.82 f'_m$.

Figure 3.1. Non-engineered concrete frames with infill bricks in Rivas, Nicaragua

The formula for compression failure of the diagonal strut is:

$$R_c = \frac{2}{3} Z f'_m \sec\theta \qquad (3.2)$$

where Z is expressed by:

$$Z = \sqrt[4]{\frac{\pi}{2}\left(\frac{4 E_c I_g h_m}{E_m t \sin 2\theta}\right)} = 19.33 \qquad (3.3)$$

where θ is 33.69°, t is 4", E_m is 600f'_m (E_m = 150,000 psi with f'_m of 250 psi), H_m is 40", and E_c is 2,850,000, in this example. The column's I_g is $6^4/12$ (I_g = 108 in^4). If R_s and R_c are compared, R_c is the lowest value (R_c =10.82 x 250 psi = 2,705 lbs) and controls. Therefore, the ultimate stress F_u is determined as 37.6 psi [F_u = 2,705 lbs / (18” x 4”)].

3.2. Geometry

The geometry of the building was scaled from the photograph and was assumed to be as shown in Figure 3.2.

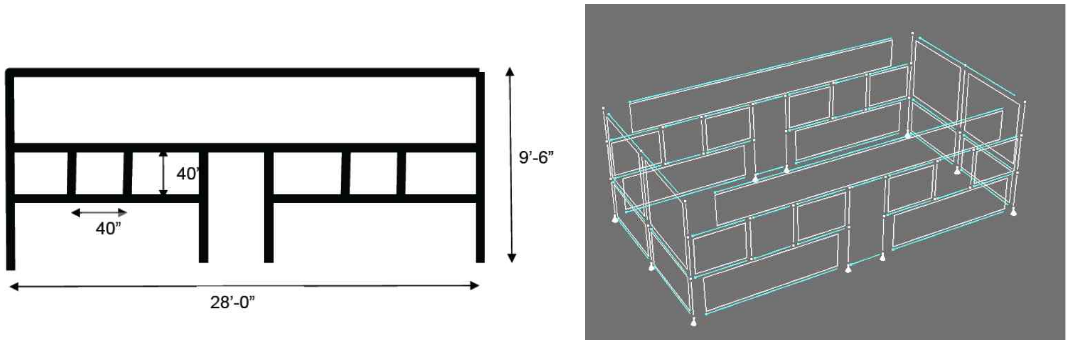


Figure 3.2. Front view of concrete frames and as-built computer Model #2 with beams at the top and bottom

The concrete frame (shown in bold lines) was assumed to be made of 6” x 6” concrete beams and columns, each with 4#5 bars. Model #1 was modeled as shown in figure 3.1. Model #2 was modeled with beams at the top and bottom. Model #3 was modeled without beams at the top and bottom. Model #4 was created with more distance between the beams and Model #5 had less distance between the columns.

3.3. Material Properties

The properties used for the model are slightly lower than US standards because materials in Nicaragua have been observed to be generally less consistent in material quality. This was done by reducing the concrete strength and the strength of the masonry infill. The material properties were assumed to be as indicated in Table 3.1.

Table 3.1. Material Properties Used in Concrete and Masonry Model

Property	Value used for steel	Value used for concrete	Value used for shear wall	Value for infill brick wall
F_u (Ultimate strength)	50 ksi	2.5 ksi	0.2 ksi	37.6 psi
E (Modulus of elasticity)	29,000 ksi	2,850 ksi		
G (Shear modulus)		1,187 ksi	2,000 ksi	

For the shear walls, E was calculated from $E = 57{,}000f'_c$, where f'_c was assumed to be 2.5 ksi, and G was calculated to be 1,187 ksi. This value was then increased to 2,000 ksi to account for the increased capacity from the steel in the shear wall.

3.4. Building Weight

The total building weight was estimated to be 51,471 lbs. The total weight was calculated as follows:

- Assumed wall weight = 63 psf
- Building overall dimensions: 28’ x 15’ x 9.5’
- The wall weights are then [(28’ x 9.5’ x 2) + (15’ x 9.5’ x 2)] x 63 psf = 51,471 lbs. The weight of the roof was ignored because of its relatively low weight compared to the weight of the walls.

3.5. Nonlinear Model

The selected building as-built computer model is shown in Figure 3.2. Model #1 has a continuous beam at the top but not the bottom. Notice in Figure 3.2 the supports are located only at column locations and the weight is applied at the four corners. Similarly, the diaphragm at the top is only connected at the column locations.

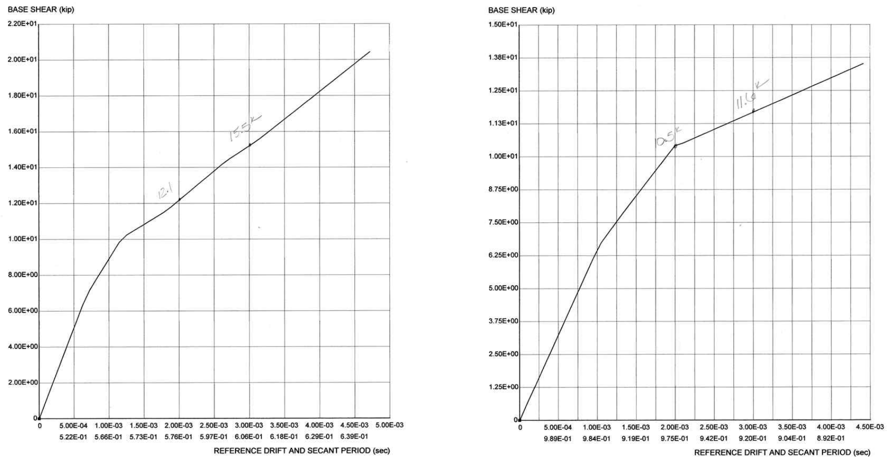


Figure 3.3. Pushover analysis results for Model #2 and Model #3

3.5. Nonlinear Pushover Analysis

The pushover analysis terminated when the model either reached the maximum deflection or a member failed. The points for immediate occupancy, life safety, and collapse prevention were evaluated as discussed earlier in this paper. The selected pushover charts are shown in Figure 3.3. Table 3.2 shows the results.

Table 3.2. Comparison of Model Performances

Model	Load at immediate occupancy	Load at life safety	Load at collapse prevention
#1	11.3 kips	11.3 kips	13.5 kips
#2 (w/ beams at top & bottom)	12.1 kips	12.1 kips	15.5 kips
#3 (w/o beams at top & bottom)	10.5 kips	10.5 kips	11.6 kips
#4 (w/ greater distance btwn beams)	3.2 kips	3.2 kips	4.5 kips
#5 (w/ less distance btwn columns)	6 kips	6 kips	10 kips

It has been noted with shear wall systems the importance of having a structural ring around the top and bottom to tie the system together (Tolles el al., 2000; Cao and Watanabe, 2004; May, 1984; Holliday et al., 2012). This ring acts in the same way a steel ring holds a wooden barrel together. As expected, adding beams at the top and the bottom increased the load capacity. Adding a beam at the top and adding a beam at the bottom are both equally important and both make an equal contribution to the building load capacity. However, doing so did not increase the capacity as much as expected. With no beams the load at collapse prevention was found to be 11.6 kips, with one beam the capacity was 13.5 kips and with beams at the top and bottom the load was found to be 15.5 kips.

Increasing the distance between the beams dramatically decreased the capacity resulting in a decrease of nearly two-thirds. This was an unexpected result and further investigations into this case will be carried out in subsequent research efforts. Additionally, it was expected that the capacity of the building would increase with more columns and yet the capacity went down. This decrease was possibly the result of the increase in rigidity caused by adding more columns. However, the columns did increase the ductility of the building.

3.6. Possible Improvements

The following changes are recommended to improve the earthquake performance of non-engineered concrete frames with infill masonry bricks:

- A structural ring around the top and bottom are most important to increasing the structural capacity in the event of an earthquake. This ring should consist of a continuous reinforced beam with adequate longitudinal reinforcement, sufficient ties, and sufficient development lengths.
- In addition to structural rings, additional beams should be located no more than 5' on center. Where possible these beams should be continuous. In every case, the beams should have adequate longitudinal reinforcement, sufficient ties, and sufficient development lengths.
- Infill bricks should be reinforced. If not possible they should be tied to the frames surrounding them.
- Tall walls should be avoided as they create large deflections.

4. PRELIMINARY CONCLUSIONS

Based on the procedure described in the preceding sections, pushover analysis was used to determine performance graphs for two common buildings in Nicaragua. These two models were then varied to determine the effect of changes in materials and geometry has on the seismic performance.

Based on the analysis of a concrete shear wall building, increasing the concrete strength improves the overall capacity of the building to resist lateral loads. Concrete strength increases the building capacity more so than increasing the percentage of steel. However, steel creates ductility in the building and should not be reduced. It was also found that wall opening sizes and locations can have a considerable effect on the building lateral load resisting capacity.

In the case of the concrete frame building with masonry infill walls, the study showed that the beams pay a large role in the earthquake performance of the building even more so than the columns. It was also shown that a beam at the top and bottom of the building (often considered a confining ring) are important elements and increase the building's earthquake capacity.

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