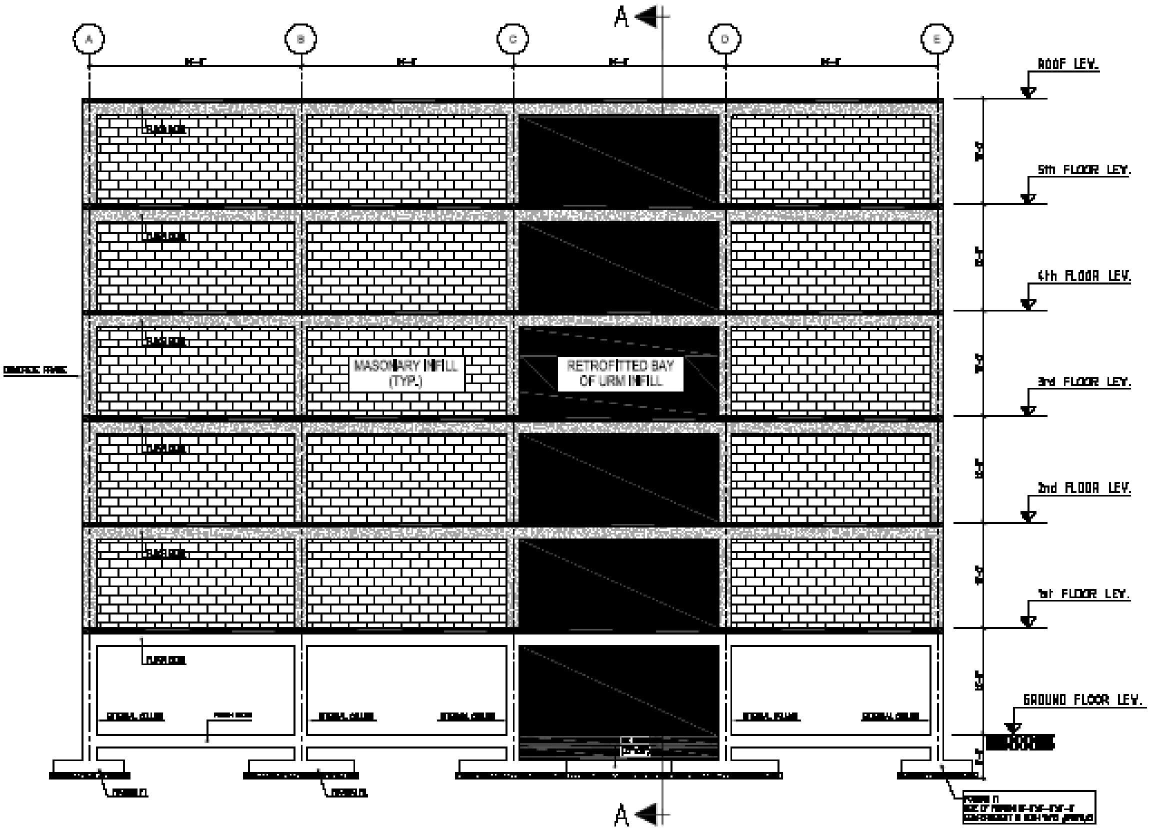

ELEVATION OF RETROFITTED URM INFILLED FRAME

6-Storey Mixed Use Building in Karachi

A Pilot Case Study
of Seismic Assessment and Retrofit Design

GEOHAZARDS INTERNATIONAL
A Nonprofit Working Toward Global Earthquake Safety

Supported by the Pakistan-US Science and Technology Cooperation Program

THE NATIONAL ACADEMIES
Advisers to the Nation on Science, Engineering, and Medicine

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Summary

In order to learn how to analyze typical reinforced concrete buildings, understand their seismic behavior and to learn how guidelines such as ASCE 41, ATC-40 and FEMA could apply to buildings in Pakistan, the project team idealized a typical Karachi residential-commercial mixed use building as the pilot case study building. For simplicity, the team investigated the behavior of two-dimensional frame models with and without infill walls, and simplified certain structural details. A separate report describes a study of the three dimensional model of the building.

The building upon which the idealized case study structure is based is located in Gulistan-e-Johar, a densely populated area in Karachi. This building consists of reinforced concrete framed building with five storeys including the ground floor. The building has shops located at the ground floor, while the above floors have residential apartments. The building was constructed before the 2005 Kashmir Earthquake. Project participants selected this building as the pilot case study because it has several seismic vulnerabilities common to mixed-use residential buildings in Karachi: a weak story created by open shop fronts at the ground floor, an eccentrically located reinforced concrete core, and heavy, stiff unreinforced masonry infill walls that were not considered during the structural design of the building.

The case study team assessed the building's potential seismic vulnerabilities using the US Federal Emergency Management Agency (FEMA) Prestandard 310 Tier 1 Checklist modified for Pakistan conditions, as well as the American Society of Civil Engineers (ASCE) Standard 31 Tier 2 and 3 analyses and acceptance and modeling criteria from ASCE 41. The building was found to be inadequate for seismic zone 4 and requires retrofitting to rectify the soft storey at the base and provide lateral stability to the building.

The team examined a number of potential retrofit solutions for both seismic performance and economic considerations. In order to provide a cost-effective and minimally intrusive retrofit, the team selected a rocking spine retrofit solution. A spine of existing infill panels reinforced with shotcrete above a reinforced concrete wall at the open ground storey prevents the building from collapsing. The spine provides stability and strength without extensive foundation work. This retrofit solution promises to be an innovative and cost-effective alternative for buildings in Pakistan.

About the Project

NED University of Engineering (NED) and Technology and GeoHazards International (GHI), a California based non-profit organization that improves global earthquake safety, are working to build capacity in Pakistan's academic, public, and private sectors to assess and reduce the seismic vulnerability of existing buildings, and to construct new buildings better. The project is part of the Pakistan-US Science and Technology Cooperation Program, which is funded by the Pakistan Higher Education Commission (HEC) and the National Academies through a grant from the United States Agency for International Development (USAID). Together, the NED and GHI project teams are assessing and designing seismic retrofits for existing buildings typical of the local building stock, such as the one described in this report, in order to provide case studies for use in teaching students and professionals how to address the earthquake risks posed by existing building. The teams are also improving the earthquake engineering curriculum, providing professional training for Pakistani engineers, and strengthening cooperative research and professional relationships between Pakistani and American researchers.

Case Study Participants

This report was compiled by Dr. Rashid Khan, Associate Professor, Department of Civil Engineering, NED University of Engineering and Technology, and Dr. Janise Rodgers, Project Manager, GeoHazards International.

This building was investigated by Mr. Aslam Faqeer Mohammad, Ms. Najmus Sahar Zafar, and Ms. Tehimna Ayub, Assistant Professors, from the Department of Civil Engineering, NED University of Engineering and Technology.

The case study team and authors wish to express their gratitude for the technical guidance provided by Dr. Gregory G. Deierlein, Professor, Department of Civil and Environmental Engineering, Stanford University; Dr. S.F.A. Rafeeqi, Pro Vice Chancellor, NED University of Engineering and Technology; Dr. Khalid M. Mosalam, Professor and Vice-Chair, Department of Civil and Environmental Engineering, University of California, Berkeley; Dr. Sarosh H. Lodi, Professor and Dean, Faculty of Engineering and Architecture, NED University Engineering and Technology; Dr. Selim Gunay, Post-doctoral Researcher, Department of Civil and Environmental Engineering, University of California, Berkeley; Mr. David Mar, Principal and Lead Designer, Tipping Mar, and Mr. L. Thomas Tobin, Senior Advisor, GeoHazards International.

Contents

Introduction

This pilot case study provided participants with the opportunity to utilize analytical techniques and procedures from international standards on a simplified version of a real building with major seismic vulnerabilities. The FEMA 310 Tier 1 vulnerability assessment exercise provided an opportunity to evaluate a real building with all the physical constraints. On the basis of the vulnerabilities found through the Tier 1 assessment, Tier 2 (linear static structural analysis) and Tier 3 (nonlinear static structural analysis) assessments were carried out to assess the vulnerabilities and potential solutions in more detail. This gave the members a chance to do hands-on practice on ETABS and understand the ASCE/SEI 31-03, ASCE/SEI 41-06 and FEMA documents.

Building Information

The building is a six storey (ground plus five) mixed use apartment building with shops at the ground floor. The building's overall dimensions 39' wide by 48' long and it is approximately 53 feet tall. The building has a reinforced concrete moment frame structural system with unreinforced concrete block infill walls and an eccentrically located reinforced concrete core. The concrete block infill walls are 6 inches thick. The foundations are reinforced concrete spread footings. The building is old and some repairs with no condition assessments have been made.

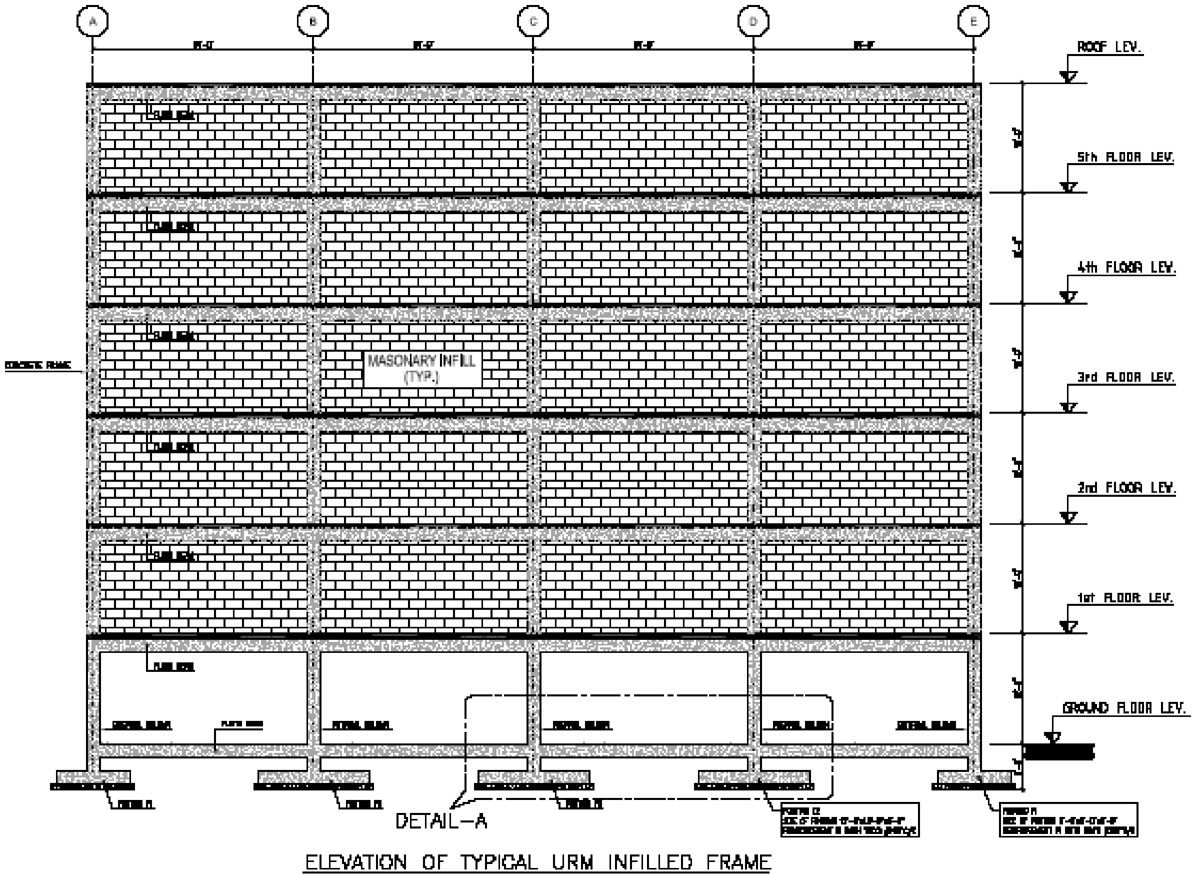


Figure 1. Structural elevation of the idealized building

For the linear and non linear finite element analysis, the building was idealized as a 2-D reinforced concrete frame with and without infill walls. Figure 2 shows the idealized 2-D frame model created in ETABS, V9.7.0. As original design calculations were not available, therefore ACI-99 was used to design the frame elements and earthquake analysis was carried out using UBC-97.

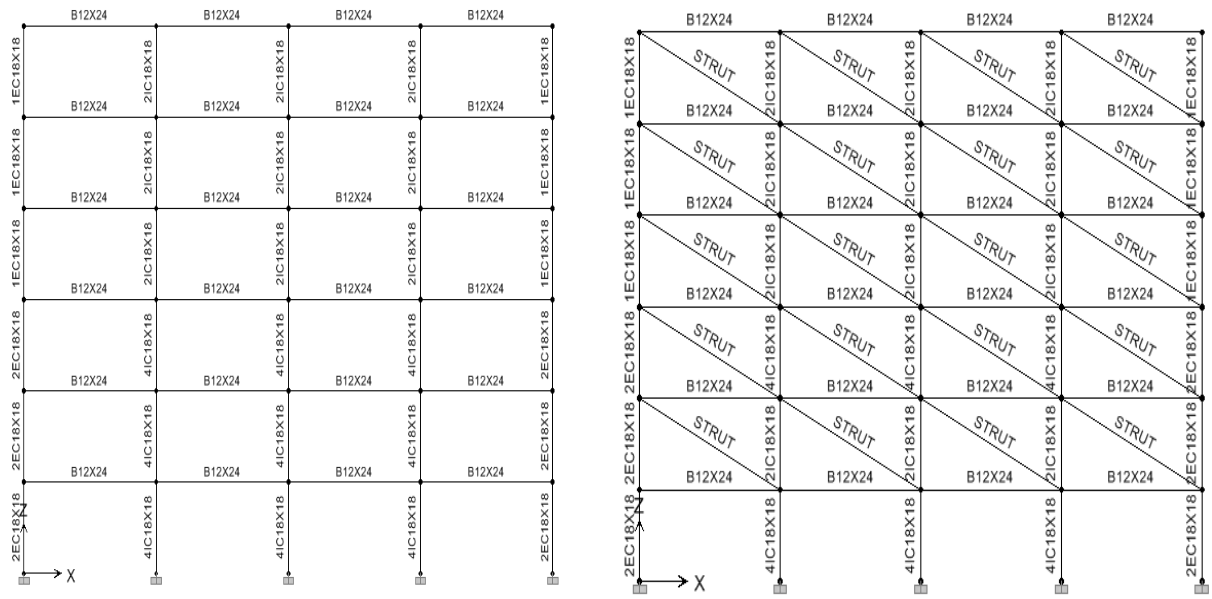


Figure 2. Idealized 2-D model in ETABS – Without infill walls (left) with infill walls (right)

The typical spacing between the columns was taken as 24 feet and beam sizes were taken as 12”x24”. The column heights were taken as 11 feet and column sizes from ground to roof were 18”x18”. Moreover, for the exterior columns a total 2% area of steel was considered from foundation to second floor and 1% for up to the roof level. Similarly, for interior columns, a 4% area of steel was considered for column between foundation and second floor level and 2% for up to the roof level. Figure 3 shows assumed section properties and reinforcement. The slab thickness was assumed to be 6 inches. For Concrete f’c was taken as 3000psi and for steel fy was taken as 60000 psi. Figure 4 shows the stress-strain curves for the two materials.

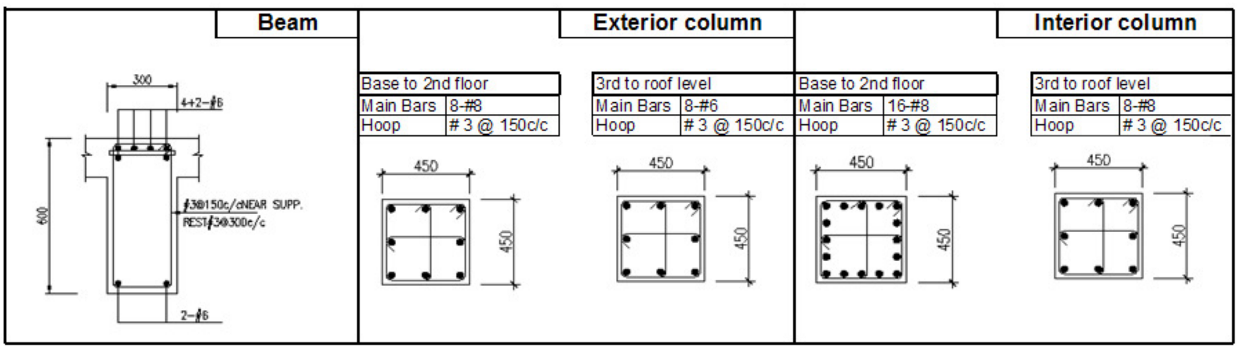


Figure 3. Structural details of typical beam and columns

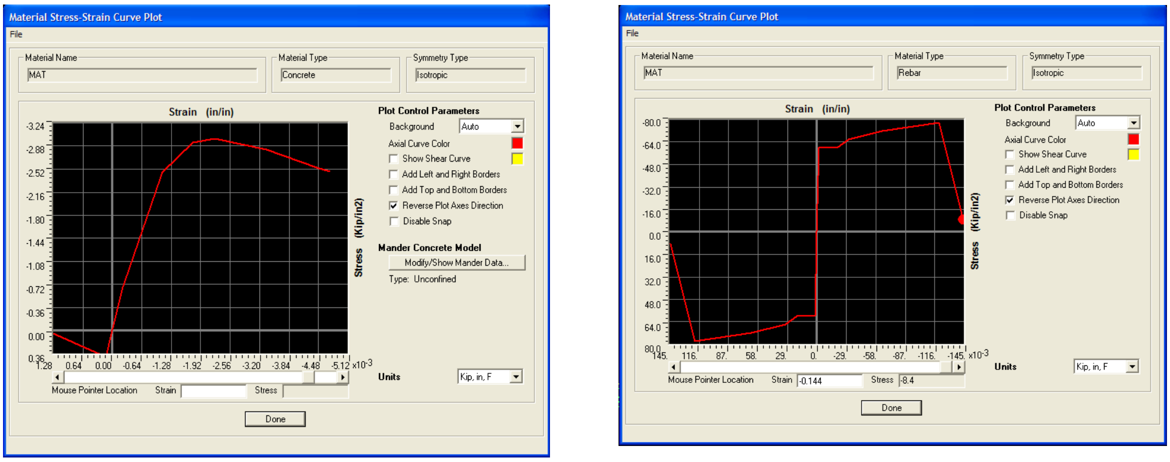


Figure 4. Stress-Strain Curves – Concrete (left); Steel (right)

Site Information

The building is located in an area with firm soil, where bedrock outcrops are often found close to the surface. No known active faults pass through or near the site. The bearing capacity of the soil is 2.0 tons per square foot (tsf). The Uniform Building Code soil type used in the analysis is S_B.

Hazard Information

Karachi's current seismic zoning under the National Building Code of Pakistan is Zone 2B. However, there is currently significant uncertainty regarding the severity of the city's seismic hazard. For this reason, the building is being evaluated for Zone 4 of the 1997 Uniform Building Code with seismic coefficients C_a=0.4, C_v=0.4. The site is not located near any known active faults so near-source factors are not applicable.

Initial and Linear Evaluations of Existing Building

Checklist-based Evaluation

The case study team used a checklist evaluation form based on FEMA 310, the precursor to ASCE/SEI 31-03 *Seismic Assessment of Existing Buildings,* modified for Pakistan conditions, to perform an initial evaluation of the building. This evaluation identified the following non-compliant items, all of which indicate a potential seismic hazard:

- Adjacent building that may pound against the building
- Torsion
- Soft/weak storey
- Shear stress check of columns show shear stresses too high
- Unreinforced masonry infill walls present
- Proportions of infill walls
- Overall construction quality is fair to poor
- Captive columns

Linear Evaluation

Figure 2 shows the 2-D model of the building generated in ETABS Nonlinear version 9.7.0. The beams and columns were modeled with linear beam-column elements, and the infill walls were modeled with single linear compression struts. The linear static analysis shows that there are a number of columns at various levels with demand/capacity ratios (DCRs) greater than one for frame without the infill walls Figure 5.

However, with infill walls present the deformation concentrates at the ground storey and the demand/capacity ratios for all the columns above the ground level becomes less than one as shown in Figure 6. This shows that the building is expected to respond in the nonlinear range, and furthermore, to have a soft storey at the ground level. The linear analysis shows that it is important to model the infill walls in order to gain a basic understanding of the potential for adverse seismic behavior, such as soft stories, due to the arrangement of infill walls in the building. However, linear analysis does not present the full picture. Nonlinear analysis is necessary to better understand the building behavior in detail and to help select and design a good retrofit solution.

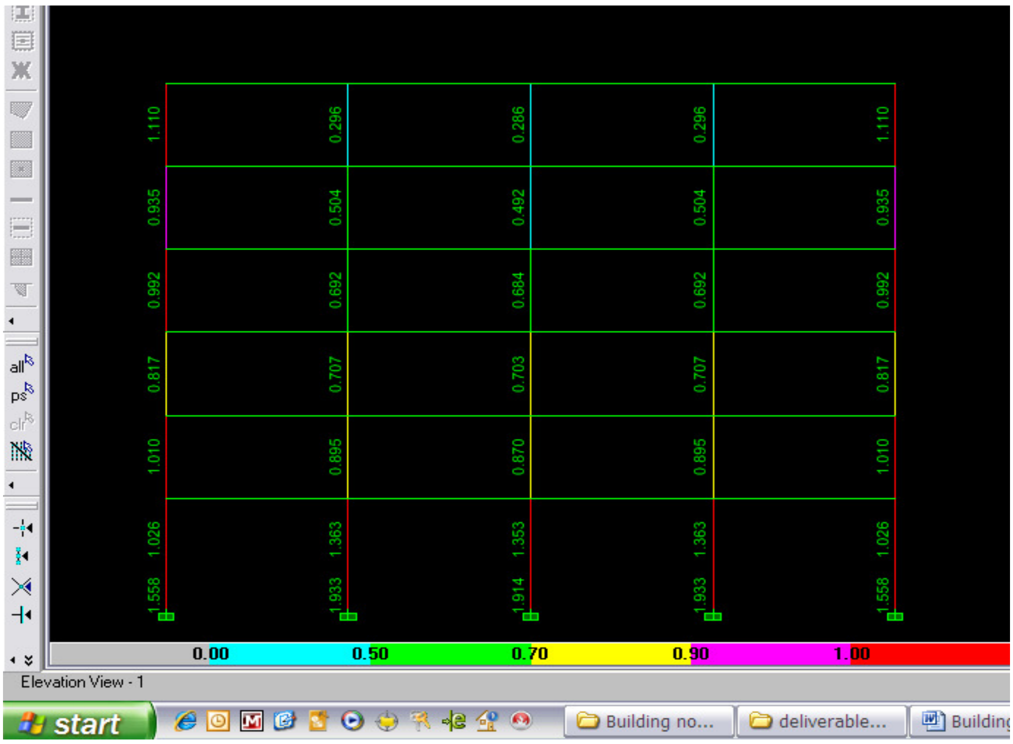


Figure 5. Demand/capacity ratios for bare frame

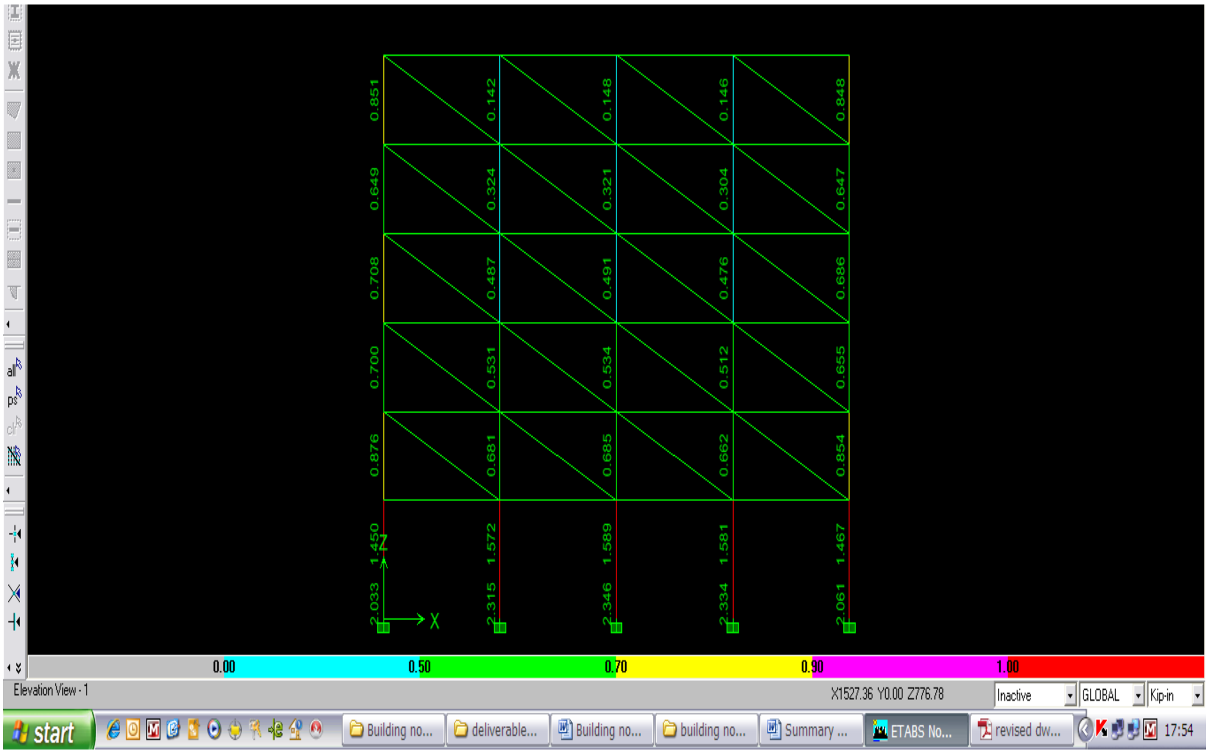


Figure 6. Demand/capacity ratios for frame with infill walls modeled with compression struts

Detailed Evaluations of Existing Building

Through linear static analysis of this building, Tier-2, it was also observed that many columns had DCR > 1. This indicates that the building is expected to respond in the nonlinear range. In Tier 3 (non linear analysis), nonlinear static pushover analysis according to ATC-40 and FEMA-356 criteria was adopted. Both 2-D frames, with and without infill walls, were evaluated.

Analytical Models

The building was modeled using discrete plastic hinge elements (i.e., a lumped plasticity model) in locations expected to experience nonlinear behavior, such as beam and column ends and the midpoint of compression struts. ASCE/SEI 41-06 standard (Seismic Rehabilitation of Existing Buildings) was adopted to determine the plastic hinge properties for compression struts, beams and columns. Figure 7 shows how plastic hinge force-deformation relations are defined in ASCE/SEI 41-06. IO, LS and CP are the Immediate Occupancy, Life Safety and Collapse Prevention performance levels, respectively.

Infill walls were modeled using equivalent compression struts defined using procedure in Section 7.5.2 of FEMA 356. The hinge properties for compression struts were computed using lower bound unreinforced masonry properties given in Table 7-1 (ASCE/SEI 41-06). For evaluation of plastic hinges for beams and columns, values given in Table 6-7 and Table 6-8 (Supplement 1 for ASCE/SEI 41-06) were used, respectively. ETABS Nonlinear (version 9.7.0) was used to create the models and perform the pushover analysis. Table 1 gives the geometric and material properties used in the model.

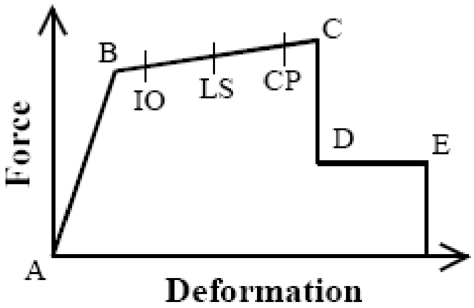


Figure 7. Force-deformation relation for hinges (reprinted from public domain document FEMA 356, the precursor to ASCE/SEI 41-06) showing the definition of acceptance criteria and performance states

Table 1. Properties of nonlinear model

Geometric Properties			
Beam	Width = 12 in		
	Depth = 24 in		
	@ Support	Top R/F Bottom R/F	As' = 2.64 in^2 As = 0.88 in^2
	Length = 24 ft		
Column	Width = 18 in		
	Depth = 18 in		
	Exterior Column	from base to 2ND floor from 3RD floor to roof	As = 8 # 8 = 6.3 in^2 As = 8 # 6 = 3.5 in^2
	Interior Column	from base to 2ND floor from 3RD floor to roof	As = 16 # 8 = 12.6 in^2 As = 8 # 8 = 6.3 in^2
	Height = 12 ft		
Ordinary Infill Wall Strut	Width = 6in		
	Depth = 36.6 in		
Material Properties			
	fc' = 3000psi for beam and column		
	E_{con} = 3144 ksi for beam and column		
	For ordinary strut fc' = 300 psi		
	E_{mas} = 214.5 ksi		

Loading and Performance Criteria

Table 2 shows the ETABS input values for gravity and earthquake loading, as well as key assumptions. The UBC-97 was used for the seismic demands. As mentioned in the Seismic Hazard section, the building was evaluated for Zone 4 seismic loads due to the current uncertainty in the seismic hazard. The case study team considered both inverted triangular and uniform lateral load distributions. For the pushover analysis, the team used restart using secant stiffness for member unloading method with P-Delta effects for geometric nonlinearity. A life safety performance criterion was selected for the study building, because it is a regular residential building and such buildings are typically evaluated for life safety.

Table 2. ETABS loading input parameters

Gravity loads:	Slab loads transferred to beam were manually calculated and applied to each of the beams in the 2-D model. 18'-0" span distance between frames assumed.
	Dead load: Self wt of frame + 6" thick slab + 2" thick finishes + 50psf wall load
	Live load: 50psf on floor and 30psf on roof
Earthquake load:	
Z	0.4g
R	5.5
C_a	$0.4N_a$ (Ref: Table 16-Q (UBC 97)) with N_a = 1.0
C_v	$0.4N_v$ (Ref: Table 16-R (UBC 97) with N_v = 1.0
Soil type	S_B (Ref: Table 16-J UBC-97)

Analysis Results

Bare Frame (Infill Walls not Modelled)

Figure 8 shows the load-deformation curve, or *pushover* curve for the bare frame (i.e., infill walls not modeled). In Figure 9 the pushover curve, a measure of the building's capacity, is converted into a capacity spectrum and compared with the estimated demand using the capacity spectrum method. This figure shows the performance level where the demand and capacity spectra intersect each other, at the point called the *performance point* where it is necessary to see the condition of the structure, and whether it is fulfilling the demand or not.

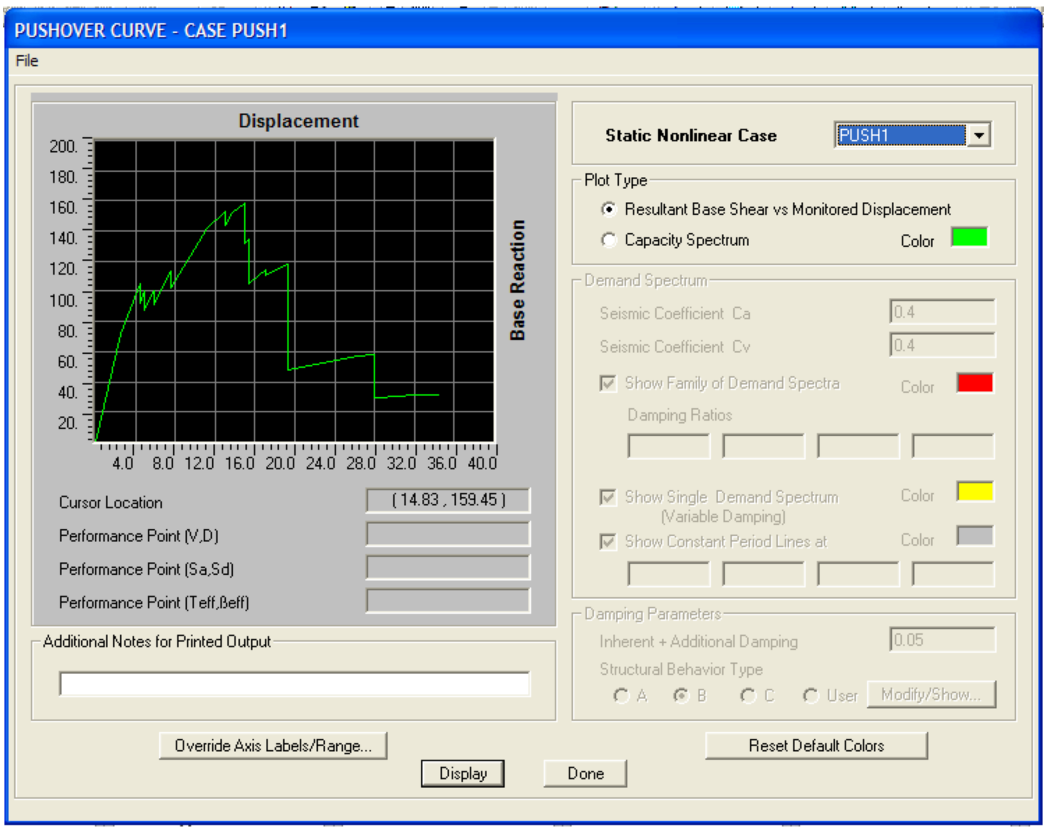


Figure 8. Pushover curve for bare frame

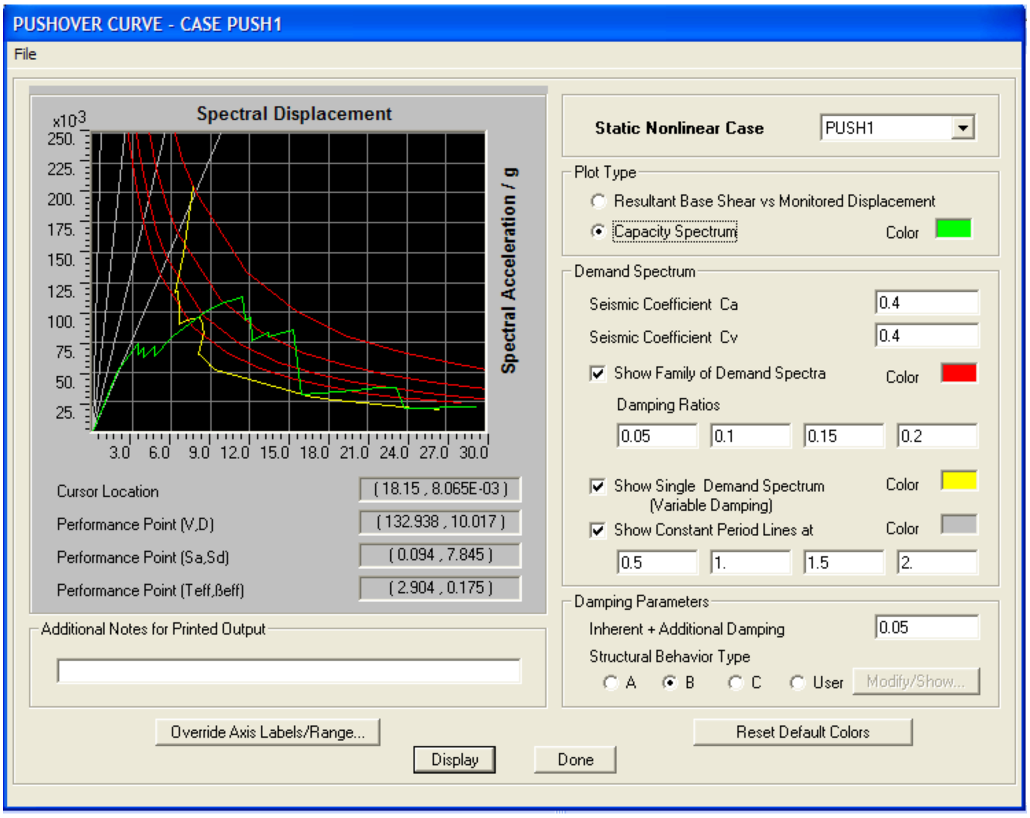


Figure 9. Performance level for bare frame

Figure 10 shows the deformed shape of the building at the performance point, with the state of the plastic hinges indicated by coloured circles.

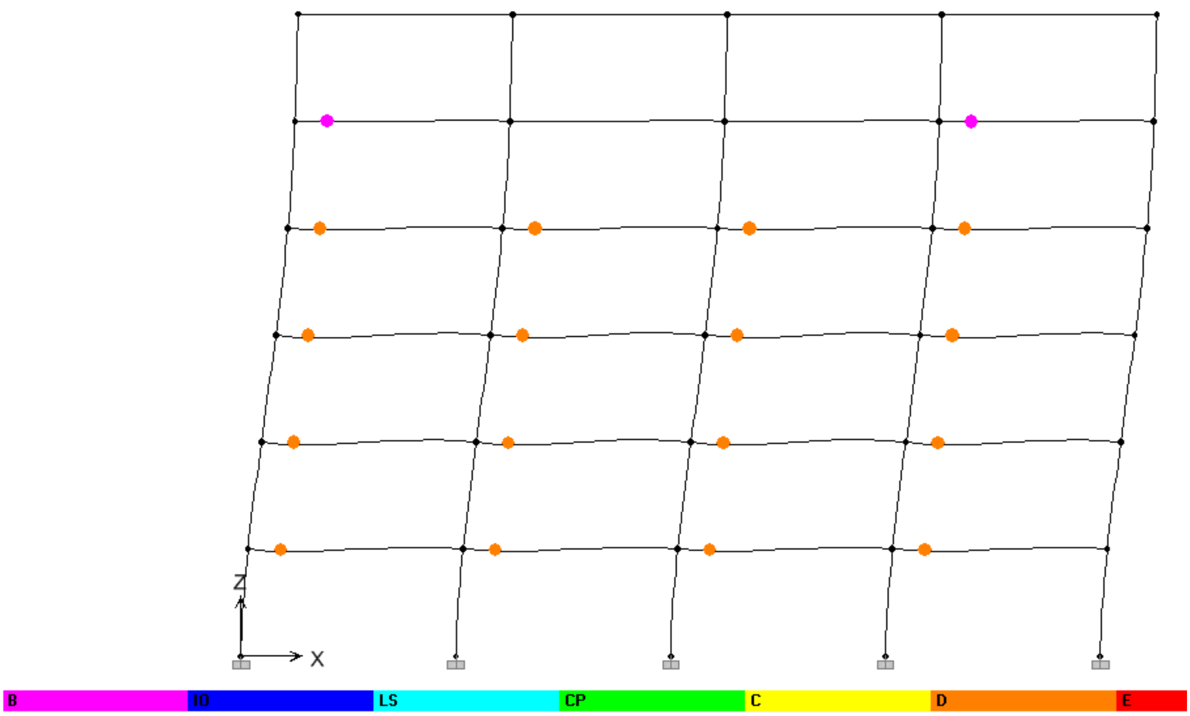


Figure 10. Hinge deformation vs. acceptance criteria

There are 16 plastic hinge locations, all in beams, where the rotations have exceeded the life safety criteria. The orange color indicates that the hinges have experienced large plastic rotations and have exceeded the collapse prevention criteria. Despite this, the plastic deformations and energy dissipation are well-distributed throughout the structure, and the building appears well-behaved in the nonlinear range. There is no indication that the building will form a single-storey collapse mechanism. As the next set of analyses will demonstrate, it is necessary to model the infill walls in order to see the true nonlinear behavior of the building.

Frame with Infill Walls Modelled

Figure 11 shows the load-deformation curve for the 2-D frame with infill walls modeled using uniaxial compression struts. Figure 12 shows the performance level where demand spectra and capacity spectra intersect each other.

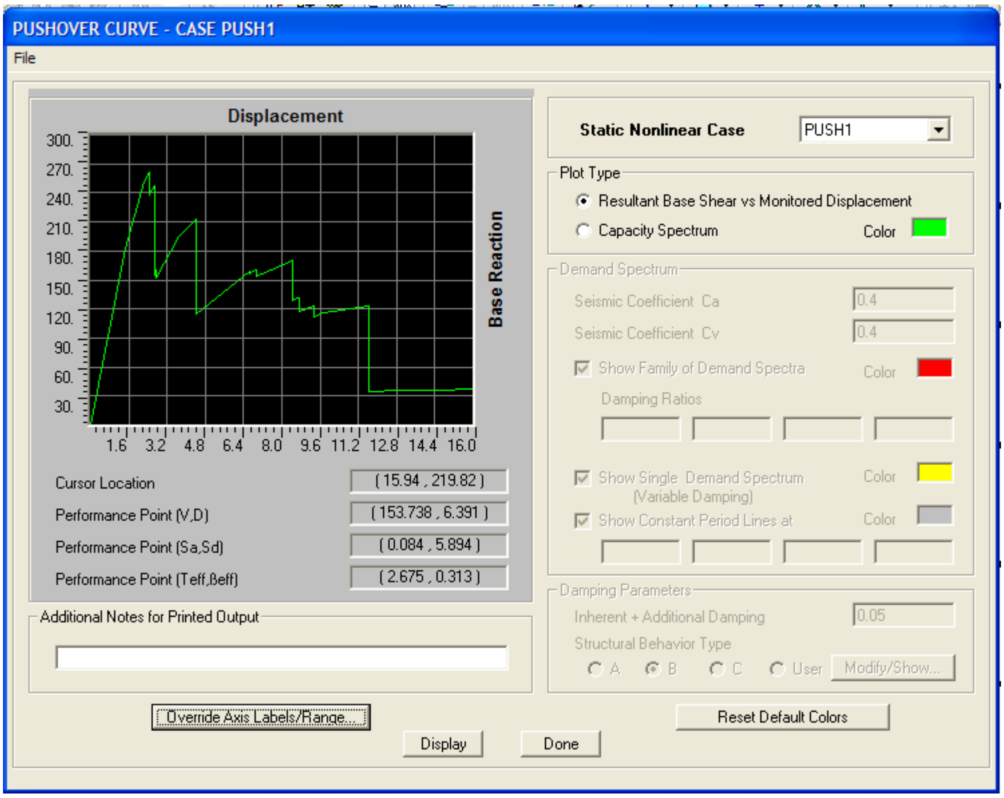


Figure 11. Pushover curve for frame with infill walls modelled

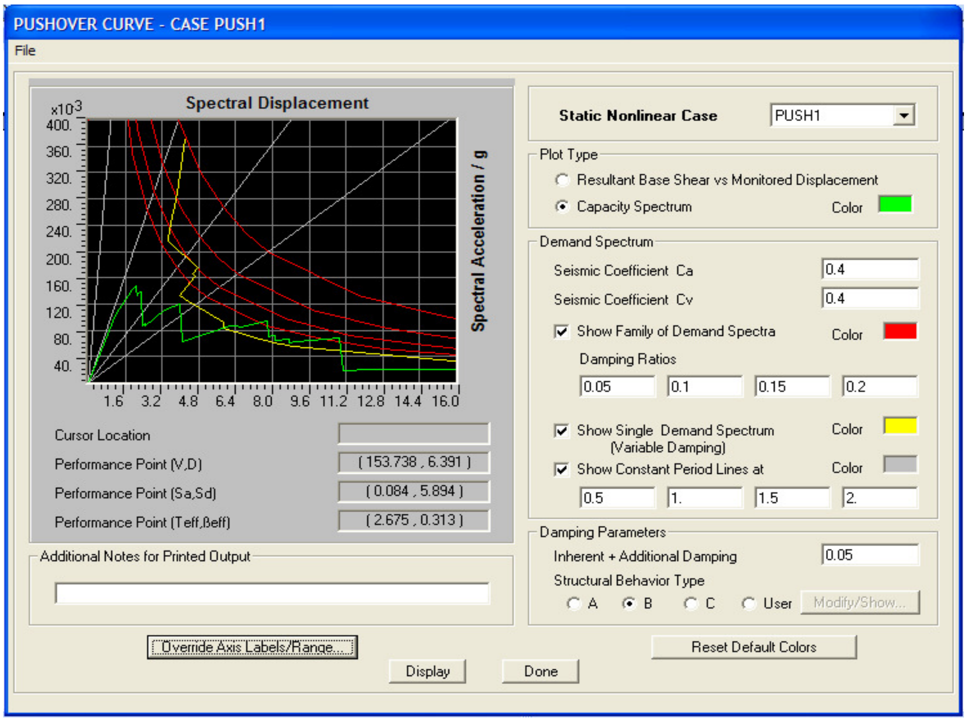


Figure 12. Performance level for frame with infill walls modeled

Figure 13 shows the state of the plastic hinges at the performance point. This shows that retrofitting is needed to achieve stability and to prevent failure at the acceptance level (life safety).

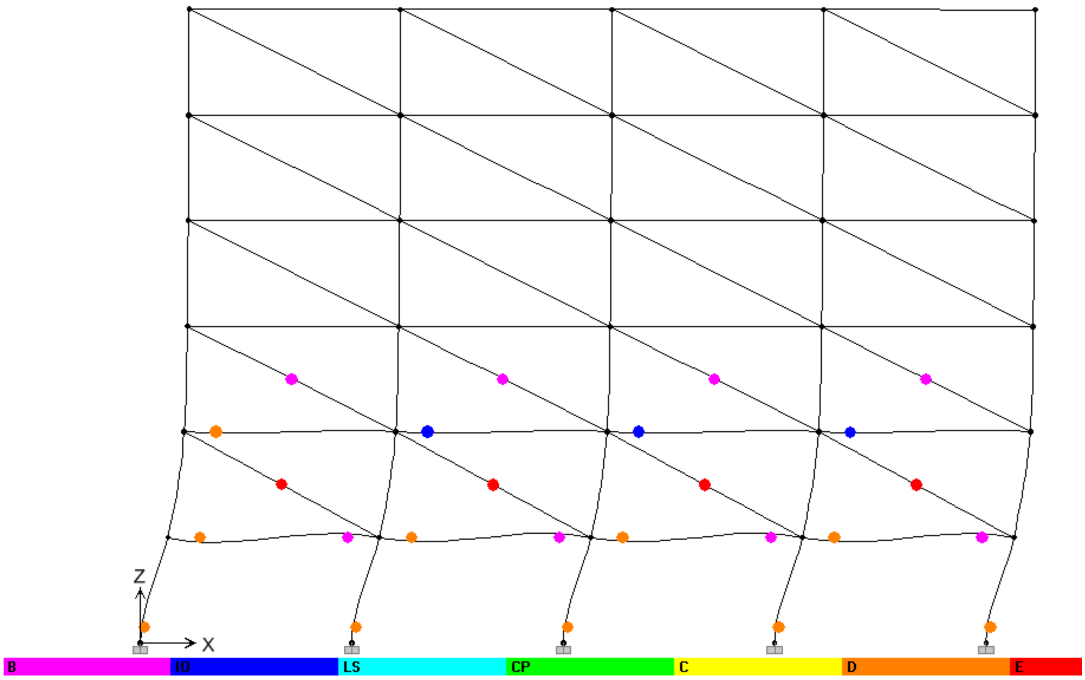


Figure 13. Hinge deformation vs. acceptance criteria, frame with infill

Figure 14 shows that the deformation concentrates in the ground storey, forming a weak storey collapse mechanism. The strength and stiffness of the infill walls in the upper storeys significantly reduces the interstorey drifts in the first storey and above. The comparison of the bare frame and infilled frame in Figure 14 shows that the presence of the infill walls increases the strength and stiffness of the building, but decreases the ductility and creates a weak storey. These plots show that the infill walls, which are often considered as architectural rather than structural elements, can drastically alter the performance of the building during an earthquake. These results show that if buildings contain infill walls, these walls **must** be modeled in order to understand the true behavior. Infill walls can be beneficial as long as they are properly taken into consideration in the design process and the failure mechanism is controlled. However, failing to consider infill walls during structural design can lead to deadly weak story collapses.

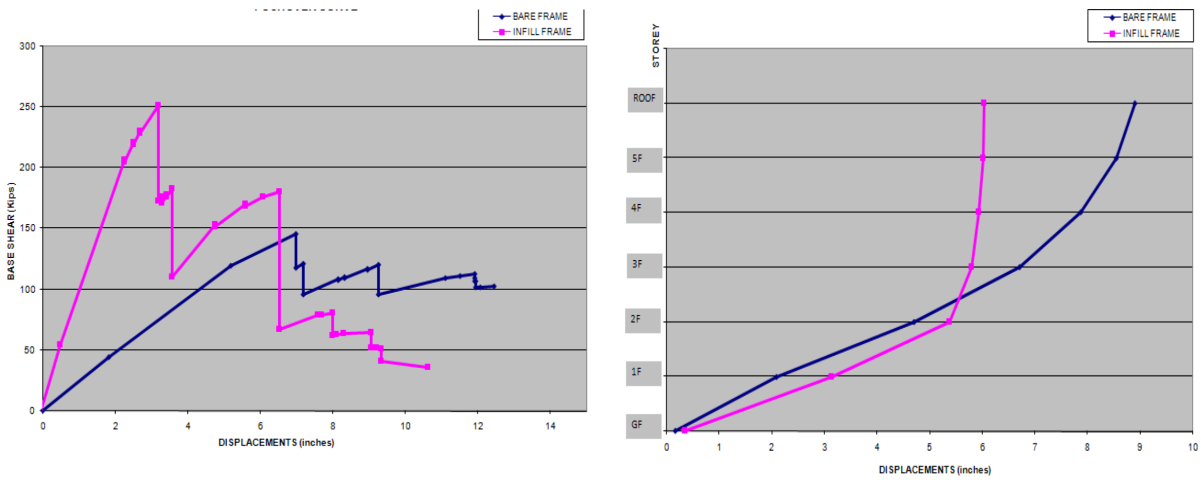


Figure 14. Comparison between bare and infilled frames - Pushover curve (left), Storey displacements (right)

Retrofit Solution

Conceptual Solutions Considered

The building presented several challenges. It is a residential building, so the solution should be low-cost and minimally intrusive in the upper floor apartments. Also, the shops at the ground floor need to be preserved to the extent possible. Figure 15, the result of a working session in Kathmandu, Nepal, shows some early options the participants considered, including column wrapping, braces and shear walls. Project participants first considered a retrofit solution in the ground storey only, in order to alleviate the weak and soft storey, but the addition of walls in the ground storey forced the failure to occur in the first storey instead. For this reason, walls needed to extend into the upper floors, high enough to prevent a failure in the storey above. RC shear walls were considered but deemed too expensive because of the foundation work that would be required.

The also team examined a rocking spine solution, which ended up as the final recommended retrofit solution and is described in further detail in the Recommended Retrofit Scheme section below.

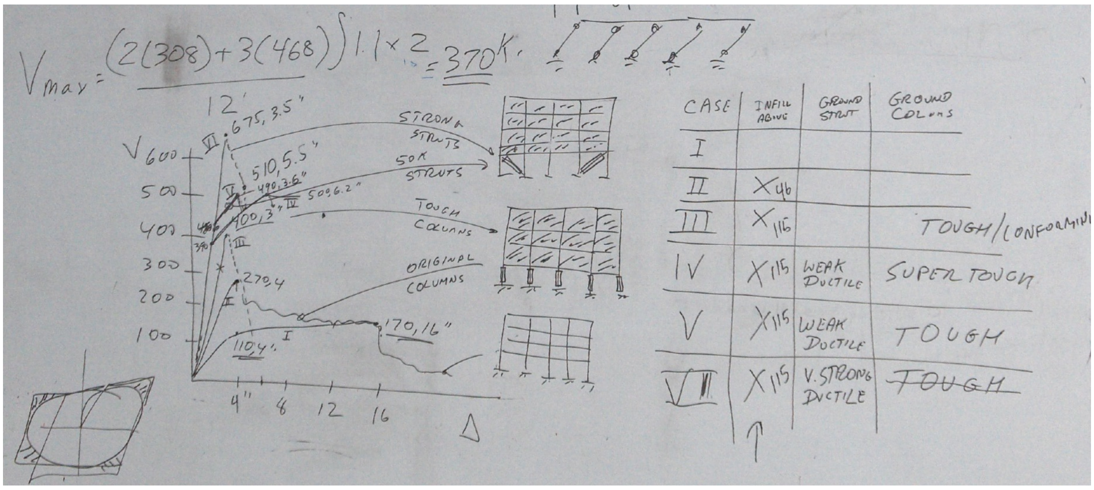


Figure 15. Some early conceptual retrofit schemes evaluated during a working session in Kathmandu, Nepal

Retrofit Analysis Results

Nonlinear analysis with retrofit measures added to the model was essential to evaluate different retrofit schemes. The team considered many different options as described above, and then many different versions of a spine solution.

The analyses confirmed that the retrofit solution provided satisfactory performance, and helped determine the properties of the new elements. Figure 16 shows a comparison between the pushover curves for the initial set of conceptual retrofit options explored during the working session in Kathmandu. Figure 17 compares the inter-storey drift profiles for the initial set of retrofit options. It is clear that the spine or walls must extend high up into the building (to at least the 3rd floor) to prevent a dangerous single-storey collapse mechanism from forming. The spine solutions prevent a single storey mechanism from forming.

Details of the modeling assumptions and methods for strut modeling are covered in greater detail in *A Practical Guide to Nonlinear Static Analysis of Reinforced Concrete Frames with Masonry Infill Walls*, available from NED University of Engineering and Technology and GeoHazards International.

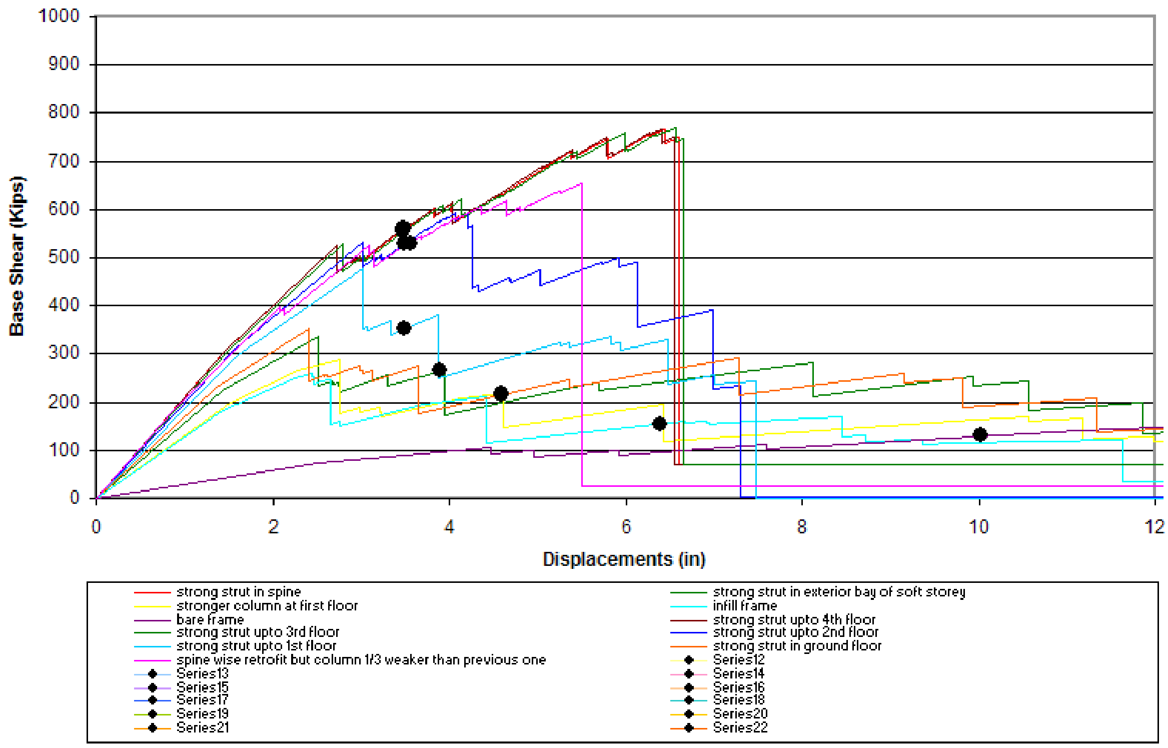


Figure 16. Comparison of pushover curves for initial set of conceptual retrofit solutions; black diamonds indicate the performance point

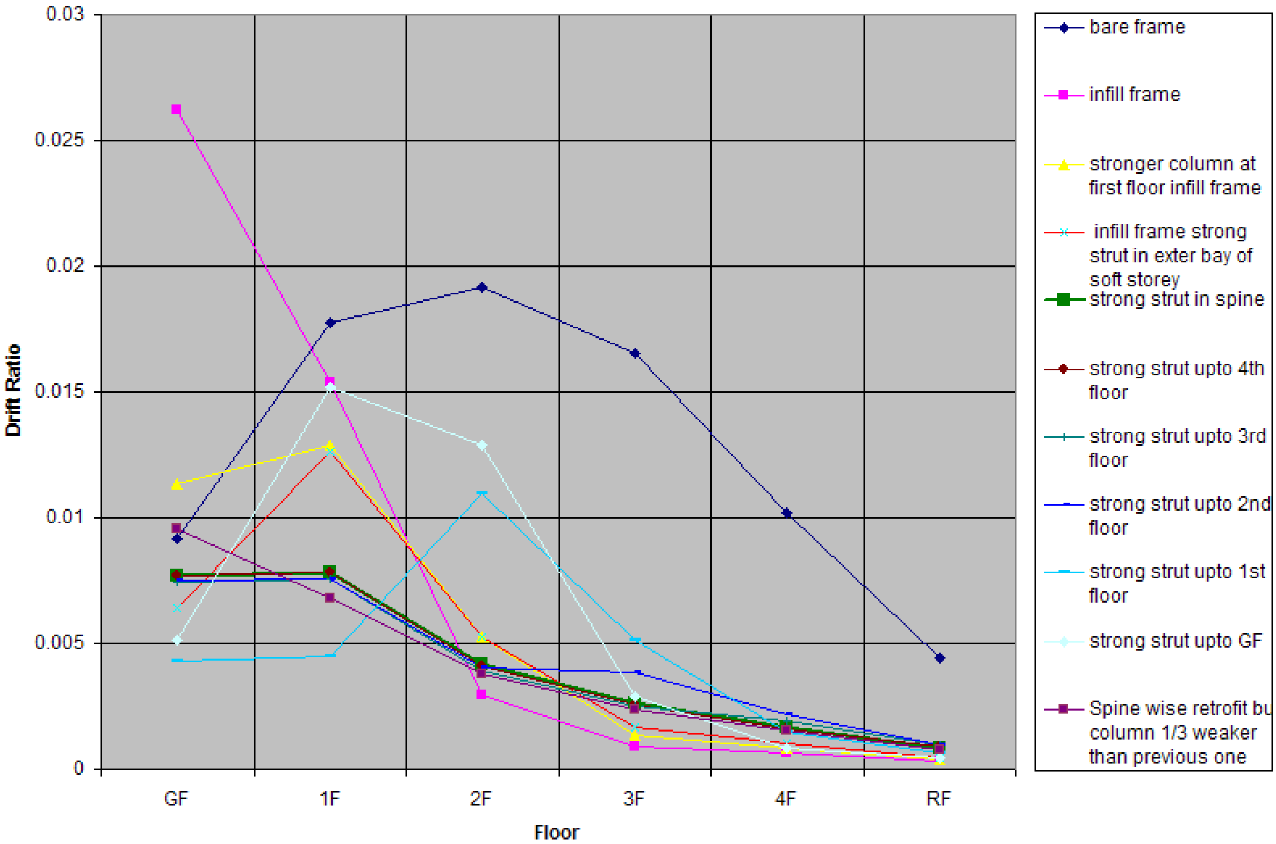


Figure 17. Comparison of inter-storey drifts for initial set of conceptual retrofit solutions

Recommended Retrofit Solution

The team selected a rocking spine solution, shown in Figure 18, to prevent a weak storey from forming and to keep costs down. Rocking spines were developed by project participant David Mar, who was inspired by traditional Chinese pagodas. A spine acts as a sort of splint to control the deformation of the structure by forcing it to respond primarily in the first vibration mode, which is typically well behaved seismically. The spine is allowed to rock at the base, which reduces removes large overturning moments from the foundation. Spines can be constructed of many different materials, and this spine would be constructed with strengthened infill panels in the first storey and above and a new RCC shear wall in the ground storey and plinth.

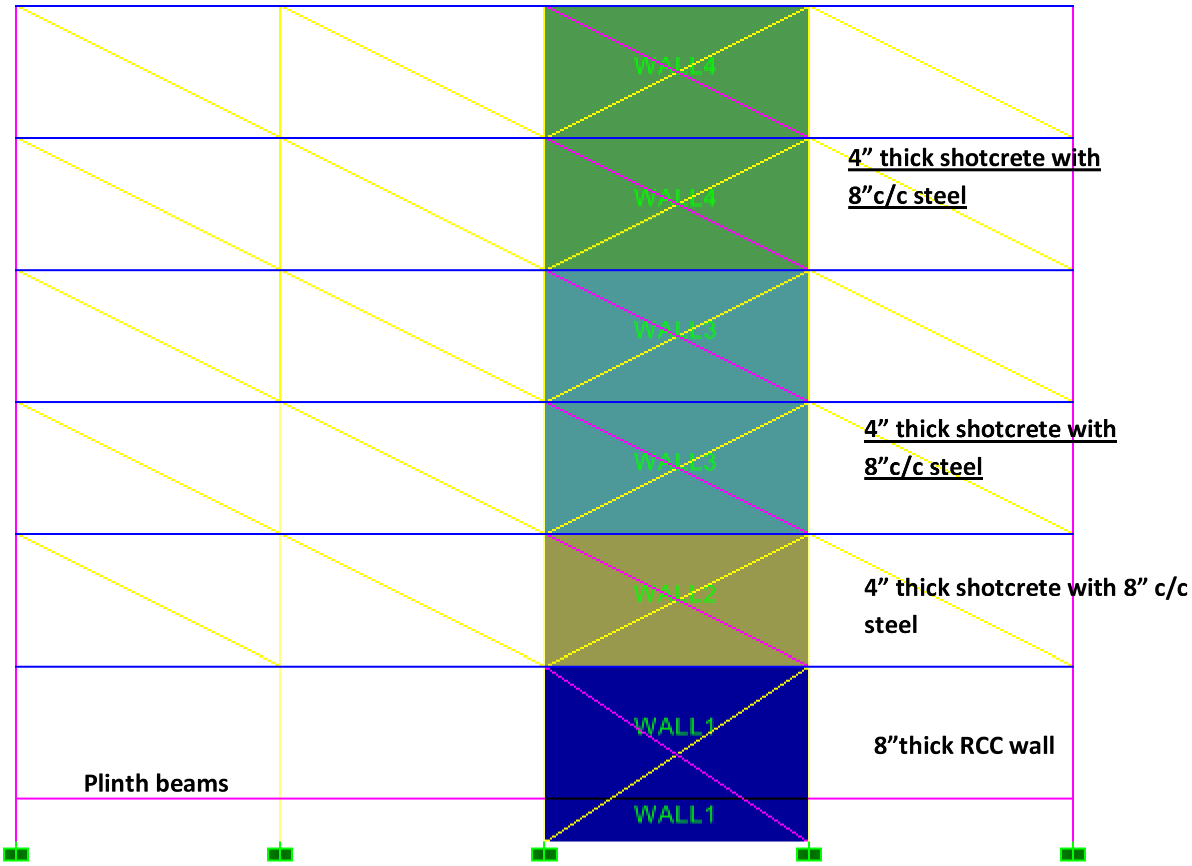


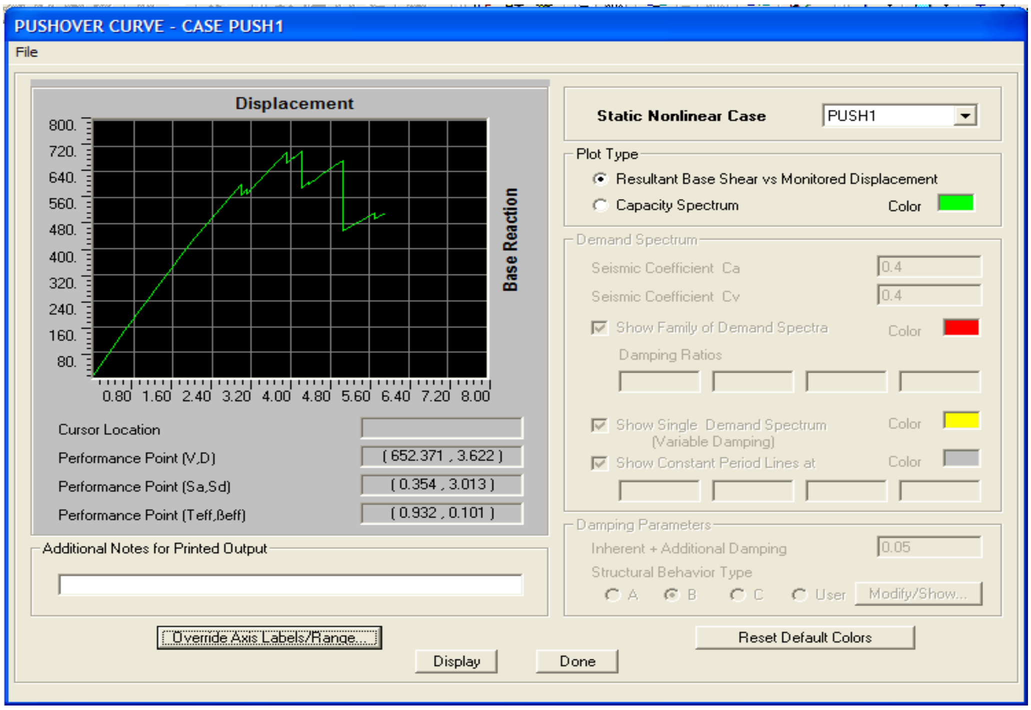
Figure 18. Concept drawing of the recommended retrofit solution

The existing plinth beam was removed and the RCC wall extended down to the top of the footings to properly transfer the shear. A new tie beam connects the two footings on either side of the spine in order to spread out the load from the spine. The spine was modeled with compression struts and tension ties with the geometric properties in Table 3. Material properties are in Table 1.

Table 3. Properties of retrofit elements

Retrofitted strut	Width of compression strut = 20 in
	Depth of compression strut = 36.6 in
Retrofitted tie	Width of tension tie = 6 in
	Depth of tension tie = 36.6 in

The performance of the final retrofit solution is shown in Figure 19, with the deformed shape at the performance point shown in Figure 20.



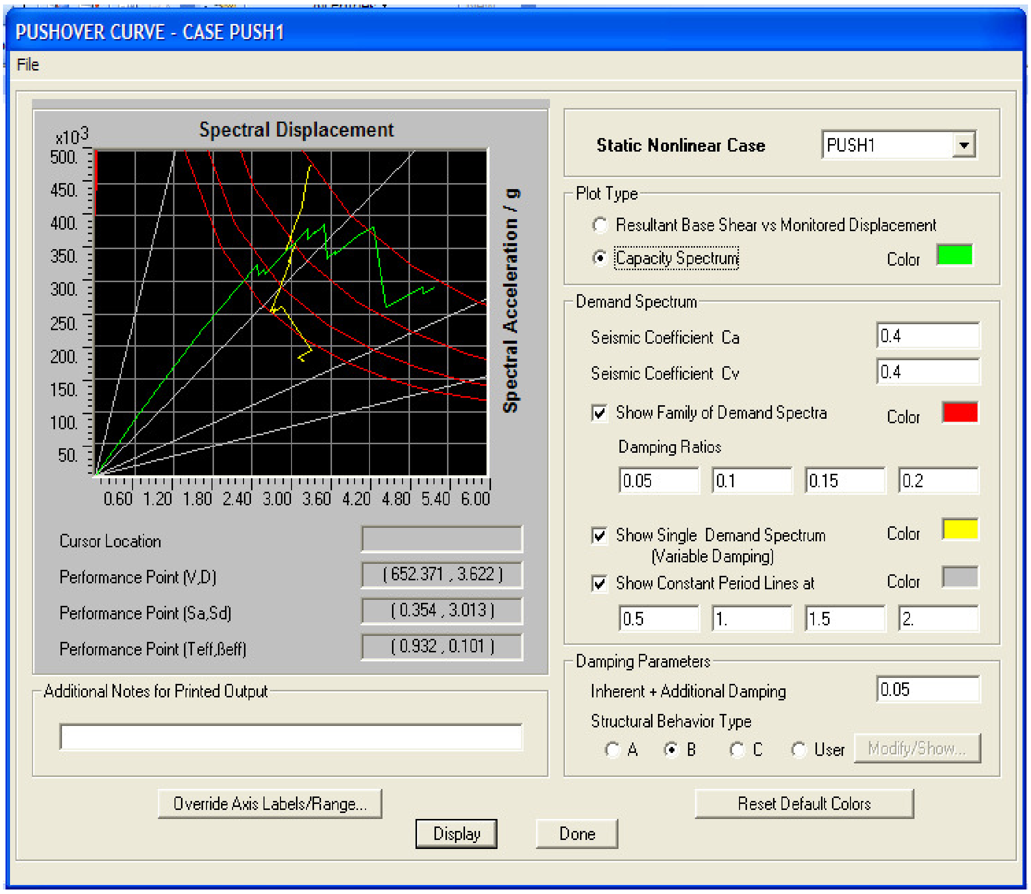


Figure 19. Pushover curve (top) and Performance level for the retrofitted building (bottom)

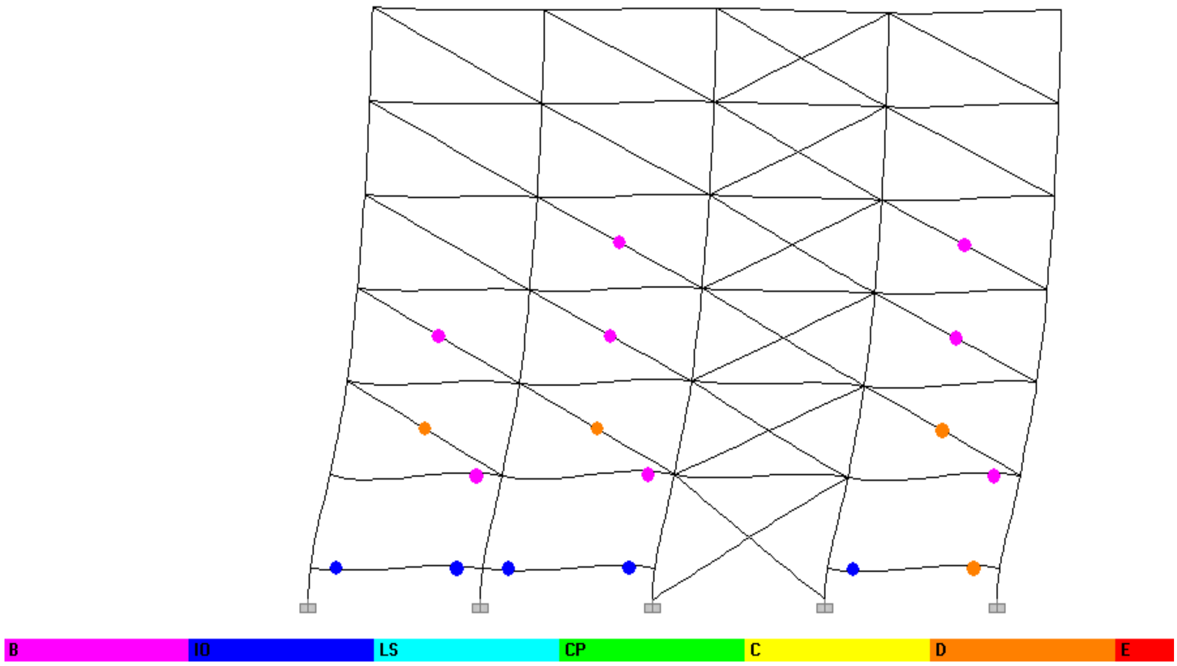


Figure 20. Hinge deformation vs. acceptance criteria for recommended spine retrofit solution

Because of the added RCC walls, footing sizes had to be checked for additional loads, and the results showed the sizes to be inadequate. Therefore, it was decided to tie the two footings together with tie beams where the RCC wall was proposed. This allowed the pressure under the footings to be spread over a larger area.

Hand Calculation Checks

At beam-column joint, joint shear was checked using the ACI method. The two joints shown in Figure 21 were selected and checked as follows:

Beam size = 12" x 24" A_s = 2 #6 A_s' = 4 #6
d = 24 -2.5 = 21.5 in
a = (2.64 x 1.0 x 60) / (0.85 x 3 x 12) = 5.18 in
M_p = 2.64 x 1.0 x 60 (21.5 – 5.18/2) = 2995 k-in
V_{col} = M_p / L_{col} = 2995/ 144 = 20.8 kips
Joint shear V_u = T_u – V_{col} = 2.64 x 1.0 x 60 – 20.8 = 137.6 kips

Joint shear strength = V_n

$$V_n = \gamma \sqrt{f_c'} b_j h_c \text{ (psi)}$$

Where:
γ = 15 h_c = 18 in b_j = 15 in
ΦV_n = 0.85 x 15 x (3000)^0.5 x 15 x18/1000 = 188.5 kips
Joint shear demand is less than joint shear strength. $V_u < \Phi V_n$ for joint 1. Calculations show similar results for joint 2.

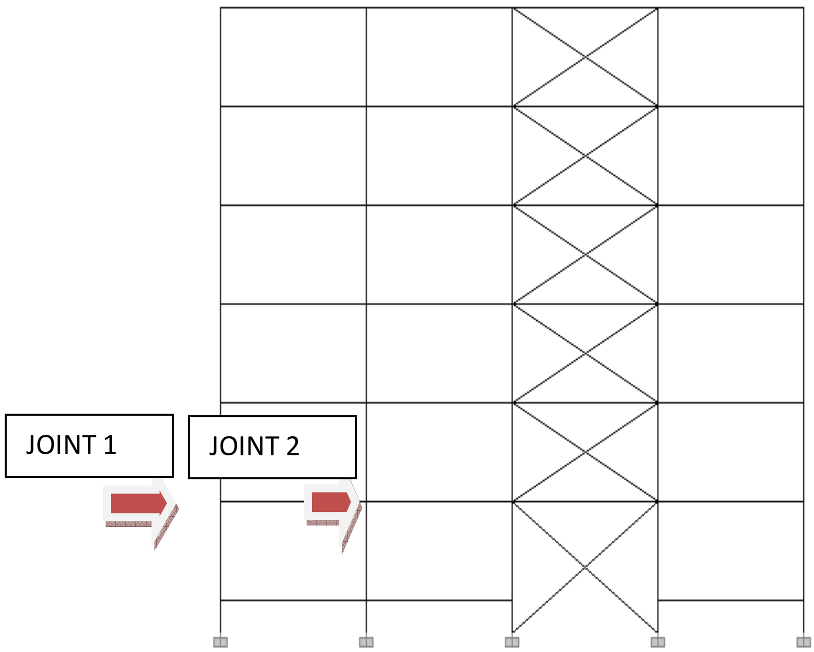


Figure 21. Selected joints for joint shear check using ACI method

Design and Detailing of Spine

Engineering drawings containing the details of the retrofit solution are shown below. Figure 22 shows an elevation of the retrofitted building. Appendix A shows the full set of drawings.

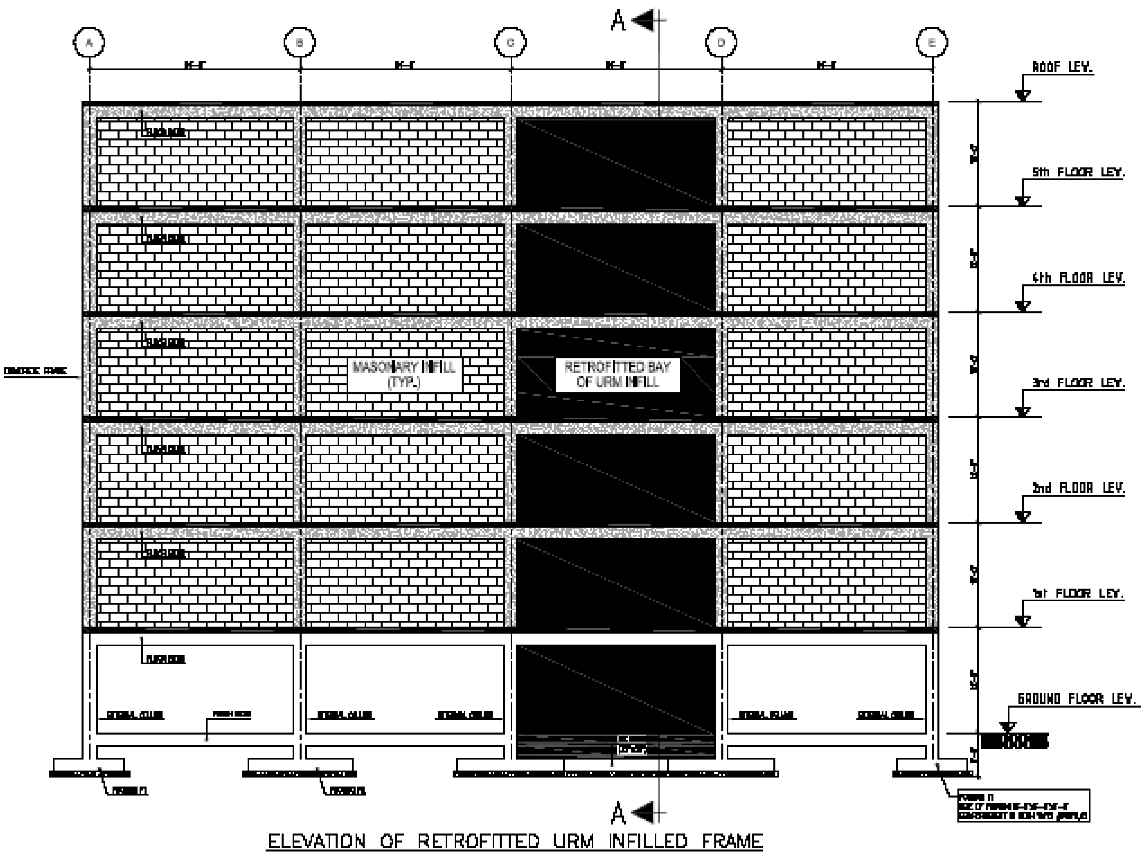


Figure 22. Elevation of frame retrofitted with spine

Figure 23 through Figure 25 show details of the spine construction using shotcrete (spray-applied concrete, also called gunite) applied to existing infill wall panels, the RCC shear wall at the ground storey, and the new tie beam connecting the footings on either side of the spine.

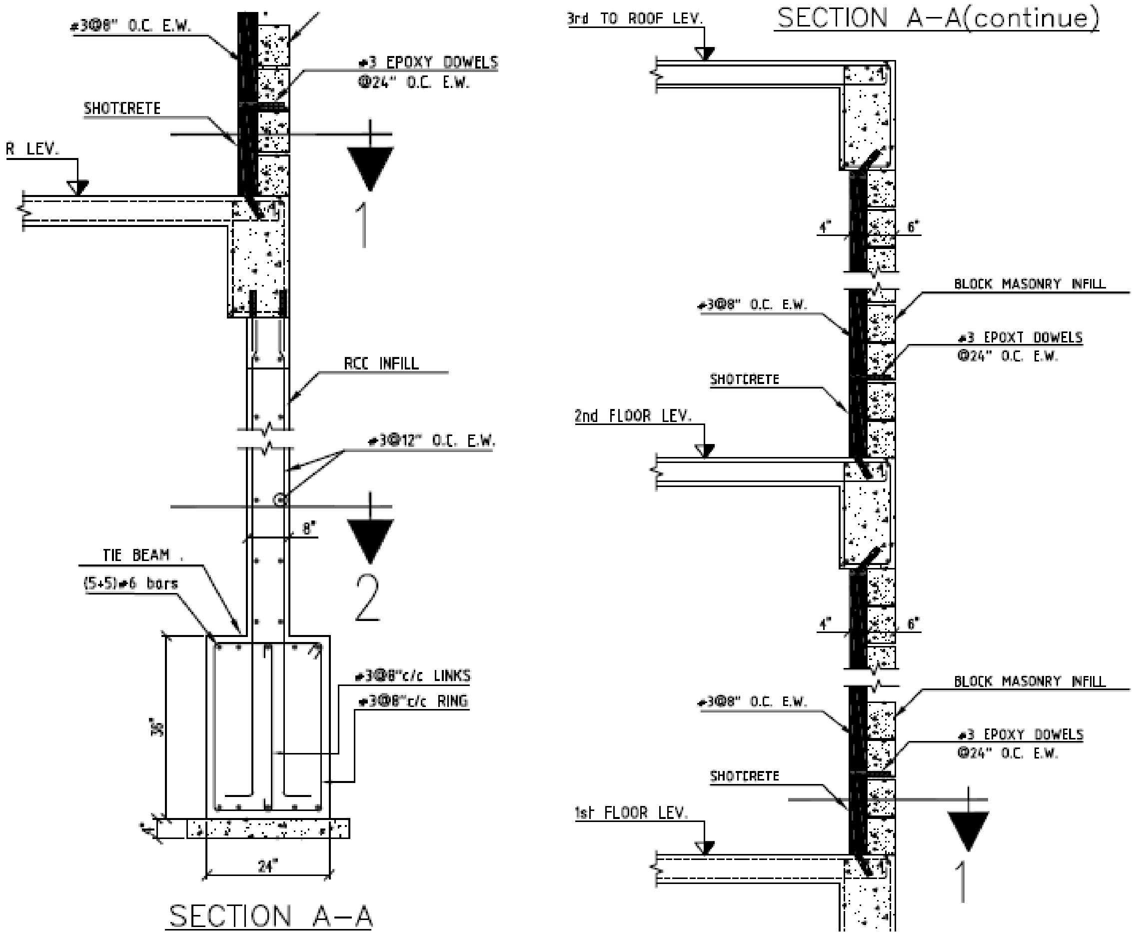


Figure 23. Vertical section showing connection of spine to existing beams and new tie beam

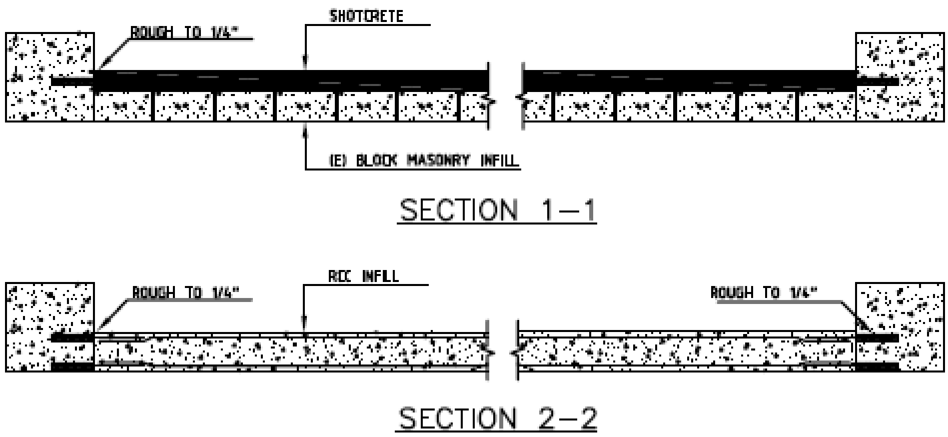


Figure 24. Horizontal sections showing connection of spine to columns

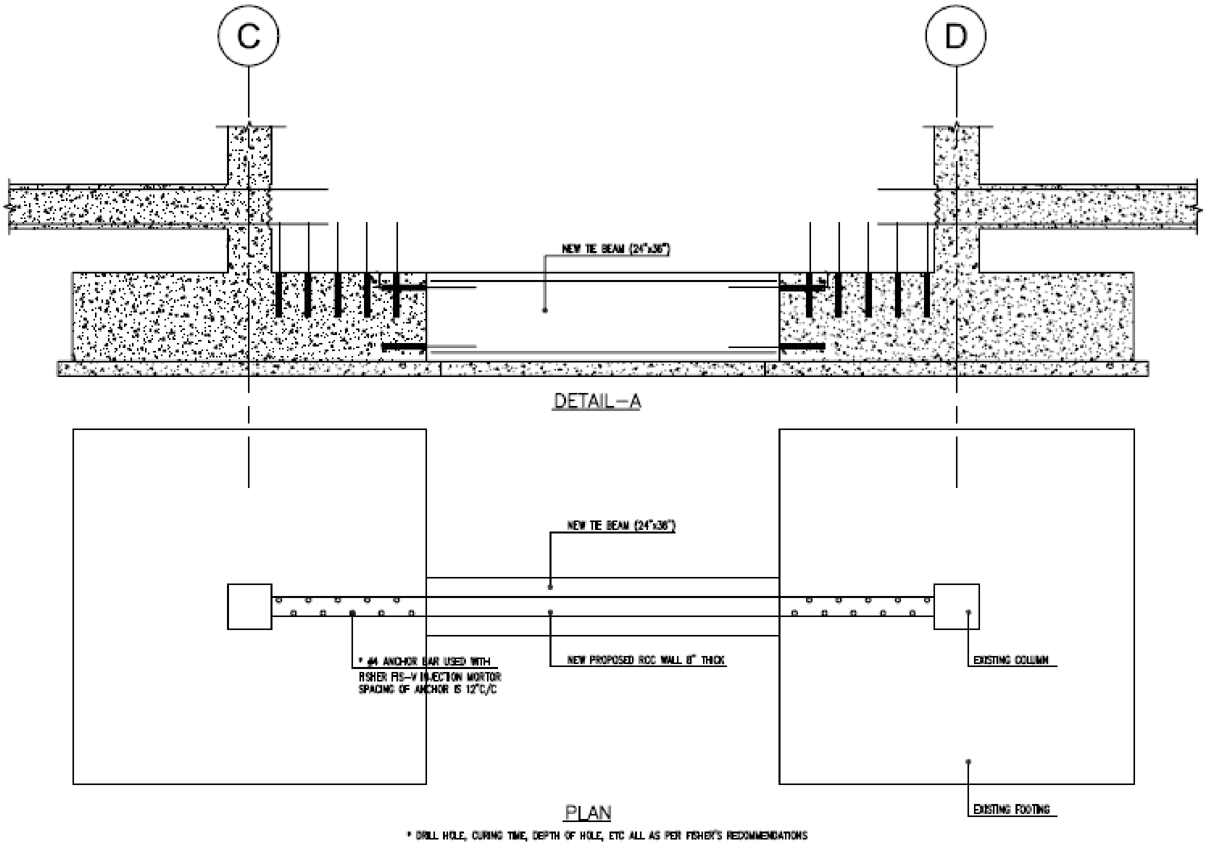


Figure 25. Elevation detail showing removal of existing grade beam and new tie beam below spine

Observations and Future Work

Considering the severity of the detrimental effects of infill – which can cause collapse – proper modeling of URM infill walls within RC frames is essential for seismic evaluation and consequently for the selection of adequate retrofits solutions to reduce damage and its consequences. The teams also analyzed a 3-D model of the building is also completed as a separate case study. The ETABS modeling techniques and workings of ASCE-41, ATC-40 and FEMA standards for evaluation of seismic venerability of buildings are being transferred to other practicing engineers, students and faculty members through workshops and seminars and publishing of a pushover analysis guide.

Appendix A: Retrofit Drawings