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**State of Practice in Seismic Rehabilitation of Buildings and Fragility  
Analysis of RC Columns Retrofitted with FRP**

**APPROVED BY  
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**State of Practice in Seismic Rehabilitation of Buildings and Fragility**

**Analysis of RC Columns Retrofitted with FRP**

**by**

**Oscar Minor Garcia**

**Thesis**

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## **Dedication**

This thesis is dedicated to Jesús, a person I met on September 19<sup>th</sup> 2017 when an earthquake shocked Mexico city, causing extensive damage in buildings and the collapse of several structures. Like many other people, I approached to the site of one of the collapsed buildings willing to help. The army, navy, red-cross, professional rescue groups (Topos) and many other people were all already there. I met Jesus there, he worked as a guard overnight in a building nearby. While we were waiting to see if we were going to be useful there, he was listening the radio news in his little phone. After several hours in which we could do nothing because professionals were already taking care of rescue for surviving people, he told me he heard about another collapsed building in which help was required. We decided to move there. On the way, a taxi offered us to get us closer. It was heartbreaking to see the amount of people on the streets helping unconditionally, donating, and collecting water, food, tools, or equipment. Some streets were closed so we continued walking until we got to the collapsed site. It was a 5 story RC building with flat slabs used as a cloth factory. The five slabs were piled up just like paper sheets. It was already dark, but the structure was lighted by portable light towers that accentuated the shadows of concrete debris and the bended reinforcing steel. Heavy machinery was already there carefully removing concrete blocks. We helped throughout the night putting debris apart carrying it in buckets or demolishing concrete blocks, so that it was easy to transport them out. We took breaks occasionally, and he would notify me with a movement of his elbow when another human chain to remove debris was being formed. In the morning I decided I was of more help surveying buildings than doing physical work, so I told this to Jesus. We shook hands, and that moment is when I asked his name. I do not know anything else about him but his inspiring willingness to help.

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I would like to acknowledge my advisor Dr. Murcia for all the help and motivation throughout this master, in class and during the development of this work. For his precise and concrete observations and comments from which I have learned to a great extent.

I would like to also acknowledge the help and constructive comments provided by Dr. Clayton. Her enthusiasm in every class is one of the best memories that I take from this master.

Finally, I would like to thank IIE and COMEXUS for providing me the Fulbright-Garcia Robles scholarship, which made this master's possible.

## **Abstract**

# **State of Practice in Seismic Rehabilitation of Buildings and Fragility Analysis of RC Columns Retrofitted with FRP**

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The University of Texas at Austin, 2020

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The first part of this thesis is a report intended to document the type of works for seismic building retrofits that are taking place in the state of California, where the legislation mandates retrofits of existing infrastructure so that minimum levels of safety are met. These retrofits are mainly on hospitals, but other type of structures such as non-ductile concrete buildings are also included. Five case studies are presented along with the description of the structure, the retrofit techniques, and illustrations to visualize where and how the works have been implemented. Regulations include not only structural but also non-structural elements; therefore, these kinds of works are similarly described.

The second part of this thesis concentrates on the determination of the lateral deformation capacity and damage fragility of circular reinforced concrete columns retrofitted with FRP jackets. A database of 56 tests found in the literature is compiled, and it is used to assess the accuracy of two different methods proposed in the literature to estimate the lateral deformation capacity of FRP-jacketed columns. New analytical

expressions are developed to compute the ultimate curvature and the plastic hinge length. These equations are used to compute the lateral deformation capacity of the columns in the database, obtaining good estimations. Finally, fragility curves are proposed based the results of tests included in the database. Three damage states are defined: longitudinal reinforcing yielding, discoloration in the FRP wrap and failure of the FRP. Two groups of fragility curves are proposed: group A utilizes story-drift ratio as a demand parameter, and Group B uses story-drift ratio normalized to the theoretical ultimate story-drift ratio. The fragility curves can be utilized in the performance-based evaluation of circular columns retrofitted with FRP.

## Table of Contents

|                                                                                                |          |
|------------------------------------------------------------------------------------------------|----------|
| List of Tables .....                                                                           | xi       |
| List of Figures .....                                                                          | xii      |
| <b>CHAPTER 1: FORMAT OF THESIS .....</b>                                                       | <b>1</b> |
| <b>CHAPTER 2: SEISMIC REHABILITATION OF REINFORCED CONCRETE BUILDINGS IN CALIFORNIA.....</b>   | <b>3</b> |
| 1. Introduction.....                                                                           | 3        |
| 2. Overview of seismic mandates for existing buildings in California.....                      | 6        |
| 2.1 Evolution of seismic mandates for hospitals .....                                          | 8        |
| Prior to 1971 .....                                                                            | 8        |
| 1971 San Fernando Earthquake and Hospital Seismic Safety Act .....                             | 8        |
| 1994 Northridge earthquake and Senate Bill 1953 .....                                          | 9        |
| Deadline extensions and SPC modifications .....                                                | 12       |
| 2.2 2016 California Administrative Code (California Code of Regulations Title 24, part 1)..... | 15       |
| 3. Case studies.....                                                                           | 26       |
| 3.1 Building A. Elevator tower retrofit.....                                                   | 27       |
| 3.1.1 Building description.....                                                                | 27       |
| 3.1.2 Retrofit goal .....                                                                      | 29       |
| 3.1.3 Design data and analysis of the retrofit.....                                            | 29       |
| 3.1.4 Strengthening Plan.....                                                                  | 30       |
| 3.2 Building B. Steam plant.....                                                               | 34       |
| 3.2.1 Building description.....                                                                | 34       |
| 3.2.2 Retrofit goal .....                                                                      | 35       |

|       |                                    |    |
|-------|------------------------------------|----|
| 3.2.3 | Design data and analysis .....     | 36 |
| 3.2.4 | Strengthening Plan .....           | 36 |
| 3.3   | Building C. Hospital offices ..... | 48 |
| 3.3.1 | Building description.....          | 48 |
| 3.3.2 | Retrofit goal .....                | 49 |
| 3.3.3 | Design data and analysis .....     | 50 |
| 3.3.4 | Strengthening plan .....           | 52 |
| 3.4   | Building D. Hotel.....             | 65 |
| 3.4.1 | Building description.....          | 65 |
| 3.4.2 | Retrofit goal .....                | 68 |
| 3.4.3 | Design data and analysis .....     | 68 |
| 3.4.4 | Strengthening plan .....           | 69 |
| 3.5   | Building E. Hotel .....            | 76 |
| 3.5.1 | Building description.....          | 76 |
| 3.5.2 | Retrofit goal .....                | 77 |
| 3.5.3 | Design data and analysis .....     | 77 |
| 3.5.4 | Retrofit plan .....                | 78 |

|                                                                                       |            |
|---------------------------------------------------------------------------------------|------------|
| 4. Conclusions.....                                                                   | 85         |
| Acknowledgments.....                                                                  | 88         |
| References.....                                                                       | 88         |
| <b>CHAPTER 3. LATERAL DEFORMATION CAPACITY AND DAMAGE FRAGILITY</b>                   |            |
| <b>ANALYSIS OF CIRCULAR RC COLUMNS RETROFITTED WITH FRP JACKETS .....</b>             | <b>90</b>  |
| Abstract.....                                                                         | 90         |
| 1. Introduction.....                                                                  | 91         |
| 2. Database of cyclic tests on circular RC columns with FRP jackets .....             | 93         |
| 3. Analysis of the lateral deformation capacity of FRP-jacketed circular columns..... | 104        |
| 3.1 Predictions using ACI 440.2R-17 and Lam and Teng (2003) equations.....            | 107        |
| 3.2 Results using Gu et al. (2015) equations .....                                    | 110        |
| 3.3 Proposed model.....                                                               | 111        |
| 4. Damage fragility functions for FRP-jacketed circular columns .....                 | 119        |
| 4.1 Damage state classification.....                                                  | 119        |
| 4.2 Structural demand parameters .....                                                | 122        |
| 4.3 Development of fragility functions.....                                           | 123        |
| 5. Conclusions.....                                                                   | 127        |
| References.....                                                                       | 129        |
| <b>CHAPTER 4. CONCLUSIONS .....</b>                                                   | <b>132</b> |
| <b>REFERENCES.....</b>                                                                | <b>134</b> |

## **List of Tables**

|                                                                               |     |
|-------------------------------------------------------------------------------|-----|
| Table 2.1 SPC Classification, deadline, and retrofit criteria.....            | 10  |
| Table 2.2 SPC Classification .....                                            | 17  |
| Table 3.1. Characteristics of column specimens included in the database ..... | 101 |
| Table 3.2. Results for experimental and calculated displacement .....         | 118 |
| Table 3.3. Damage States for Circular RC Columns retrofitted with FRP .....   | 121 |
| Table 3.4. Median $\theta$ and dispersion $\beta_r$ of fragility curves ..... | 126 |

## List of Figures

|                                                                                                                                                                                                                       |    |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----|
| Figure 2.1. Codes that regulate evaluation of existing hospitals and design of retrofits<br>of buildings in California .....                                                                                          | 7  |
| Figure 2.2. Evolution of legislation on existing hospitals and the original 36-year plan<br>on SB 1953.....                                                                                                           | 12 |
| Figure 2.3. Fault Activity Map of California (2010), with details of San Francisco and<br>Los Angeles. Source: <a href="http://maps.conservation.ca.gov/cgs/fam/">http://maps.conservation.ca.gov/cgs/fam/</a> . .... | 26 |
| Figure 2.4. General localization of structure A.....                                                                                                                                                                  | 27 |
| Figure 2.5. Floor plan and Elevation of building A.....                                                                                                                                                               | 28 |
| Figure 2.6. Construction sequence in Building A.....                                                                                                                                                                  | 29 |
| Figure 2.7. Strengthening plan for building A.....                                                                                                                                                                    | 30 |
| Figure 2.8. Details for anchoring new to existing reinforced concrete walls .....                                                                                                                                     | 32 |
| Figure 2.9. Details for FRP strengthening .....                                                                                                                                                                       | 33 |
| Figure 2.10. General plan of building B (Power plant). ....                                                                                                                                                           | 34 |
| Figure 2.11. General view of the power plant: chiller facility (left building) and steam<br>plant (right building).....                                                                                               | 35 |
| Figure 2.12. Original classification for the different structures of the power plant.<br>These structures are being upgraded to SPC-4D and NPC-5 standards .....                                                      | 36 |
| Figure 2.13. 3D model of the steam plant of building B. ....                                                                                                                                                          | 37 |
| Figure 2.14. Strengthening plan for building B. ....                                                                                                                                                                  | 38 |
| Figure 2.15. View of the wall attached to the foundation and the infills in the shear<br>walls. ....                                                                                                                  | 40 |
| Figure 2.16. Details of wall infills, increase of wall thickness and FRP strengthening<br>of beams .....                                                                                                              | 41 |

|                                                                                                                                                                                          |    |
|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----|
| Figure 2.17. General view of the new piles, the footing and the walls prepared for connection to the new foundation .....                                                                | 42 |
| Figure 2.18. Details of connection of new foundation to existing structure .....                                                                                                         | 43 |
| Figure 2.19. Works on the roof: Equipment bolted to the slab (top left), cable bracing of pipes in cantilever (top right) and bracing of parapets with diagonal elements (bottom). ..... | 44 |
| Figure 2.20. Details of the connection of the bracing in the parapet. ....                                                                                                               | 45 |
| Figure 2.21. Detachment of masonry wall from the structure. ....                                                                                                                         | 46 |
| Figure 2.22. Details of the detachment of a masonry wall from the structure.....                                                                                                         | 46 |
| Figure 2.23. Different details of the bracing and bolting of the equipment in the plant....                                                                                              | 47 |
| Figure 2.24. General localization of building C. ....                                                                                                                                    | 48 |
| Figure 2.25. First Floor framing plan.....                                                                                                                                               | 49 |
| Figure 2.26. 3D view of building C with the main retrofit actions.....                                                                                                                   | 50 |
| Figure 2.27. Extruded view of Building C. Note the high irregularities in floor and elevation. ....                                                                                      | 51 |
| Figure 2.28. Elevation of new wall on grid line X.....                                                                                                                                   | 53 |
| Figure 2.29. Details of connections of new wall on grid line X. ....                                                                                                                     | 54 |
| Figure 2.30. Details of connections of new wall on grid line H and wall on grid line 4. ..                                                                                               | 56 |
| Figure 2.31. Elevation on grid line U. Wall strengthened with FRP. ....                                                                                                                  | 58 |
| Figure 2.32. Details of connections of wall on grid line F and steel column strengthening.....                                                                                           | 59 |
| Figure 2.33. Details for collectors.....                                                                                                                                                 | 61 |
| Figure 2.34. Details of connections of new with existing foundation. ....                                                                                                                | 62 |
| Figure 2.35. Portion of the wall that is to be increased with an additional 8 in. RC layer.....                                                                                          | 63 |

|                                                                                                                                                           |    |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------|----|
| Figure 2.36. Detail of an existing concrete footing totally embedded in the new footing.....                                                              | 64 |
| Figure 2.37. Construction of the new footing with the existing footing embedded .....                                                                     | 64 |
| Figure 2.38. Typical floor plan of building C.....                                                                                                        | 65 |
| Figure 2.39. Typical elevation of building C.....                                                                                                         | 66 |
| Figure 2.40. Details of typical existing moment connections.....                                                                                          | 67 |
| Figure 2.41. Details of connections of bracings of strips and HSS .....                                                                                   | 68 |
| Figure 2.42. Increasing in the size of the foundation of one column.....                                                                                  | 69 |
| Figure 2.43. Details of strengthening in beams where buckling restrained braced are placed (left) and strengthening of the webs in columns (right). ..... | 70 |
| Figure 2.44. Details of strengthening in columns with welded HSS elements; beams with additional plates and the slab with FRP.....                        | 70 |
| Figure 2.45. Details of strengthening in columns with welded HSS elements; beams with additional plates and the slab with FRP.....                        | 71 |
| Figure 2.46. FRP strengthening of circular column in the basement.....                                                                                    | 72 |
| Figure 2.47. Column with capital strengthened with FRP, already covered. ....                                                                             | 73 |
| Figure 2.48. Plan with FRP strips to strengthen the diaphragm and the area around the pool at the roof level. ....                                        | 74 |
| Figure 2.49. Application of FRP strips to strengthen the area around the pool at the roof level .....                                                     | 74 |
| Figure 2.50. Structure for the pool on the roof. Bottom view (left) and top view (right). ....                                                            | 75 |
| Figure 2.51. 3D model of building E. ....                                                                                                                 | 76 |
| Figure 2.52. Basement and first floor plan of Building E.....                                                                                             | 77 |
| Figure 2.53. 3D model of strengthening plan for building E.....                                                                                           | 79 |

|                                                                                                                                                     |     |
|-----------------------------------------------------------------------------------------------------------------------------------------------------|-----|
| Figure 2.54. Shear Walls A, E1 and E2 (Elevation) .....                                                                                             | 80  |
| Figure 2.55. Shear Walls 4 and 9D (Elevation) .....                                                                                                 | 81  |
| Figure 2.56. Jacketing of columns .....                                                                                                             | 82  |
| Figure 2.57. View of Shear Wall E1 .....                                                                                                            | 82  |
| Figure 2.58. Construction of shear walls .....                                                                                                      | 83  |
| Figure 2.59. Construction of roof deck and penthouse .....                                                                                          | 83  |
| Figure 2.60. Parapet bracing .....                                                                                                                  | 84  |
| Figure 3.1 Distribution of main characteristics of columns included in the database .....                                                           | 103 |
| Figure 3.2. Plastic-hinge model for cantilever column: (a) distribution of moment,<br>curvature, and deflection, (b) moment-curvature diagram ..... | 105 |
| Figure 3.3. Material laws for concrete (left) and steel (right).....                                                                                | 107 |
| Figure 3.4. FRP-confined concrete stress-strain models .....                                                                                        | 108 |
| Figure 3.5. Ultimate Curvature parameters.....                                                                                                      | 114 |
| Figure 3.6. Ultimate curvature .....                                                                                                                | 115 |
| Figure 3.7. Effect of confinement in plastic hinge length.....                                                                                      | 117 |
| Figure 3.8. Comparison of measured vs. analytical displacement.....                                                                                 | 117 |
| Figure 3.9. Physical appearance of DS2 (above) and DS3 (below). a) to d) from refs.<br>[2], [14], [11], [10], respectively.....                     | 121 |
| Figure 3.10. Group A Fragility curves for circular RC columns retrofitted with FRP .....                                                            | 124 |
| Figure 3.11. Group B Fragility curves for circular RC columns retrofitted with FRP .....                                                            | 125 |

## **Chapter 1: Format of Thesis**

In the last three decades, major earthquakes in different locations have encouraged extensive research on understanding the behavior and damage mechanisms of structural elements subjected to cyclic loads. Results from research have yielded to the development of retrofitting techniques aimed not only to prevent failures but also to decrease the costs associated with damage in structures. The knowledge accumulated based on both research and the experience through the observation has been reflected in more strict building regulations for new structures. However, the question of what to do with existing infrastructure that may not conform to current standards and regulations and may require seismic retrofit arises. The state of California, for instance, has developed a code that requires existing buildings to be retrofitted to minimum levels of safety. Consequently, several retrofitting projects for existing buildings have been under course. Reporting the type of techniques used in retrofits can serve for future investigation to evaluate the effectiveness of such techniques. Part of this thesis is focused on document current practice in retrofit techniques.

In parallel, new methodologies for analysis have been developed, intended to evaluate not only the ultimate capacity of the structure but also the performance of the structure when subjected to different levels of demand. These methodologies have concentrated in the design of new structural elements; however, it is equally important to evaluate the performance of retrofitted elements. The second part of this thesis focuses on the development of fragility curves for circular reinforced concrete (RC) columns retrofitted with fiber reinforced polymer (FRP). FRP jacketing has proved to significantly improve the capacity of columns in terms of both strength and ductility. These types of

fragility curves are used in performance-based evaluation of a structure after being retrofitted.

This thesis is organized in four chapters. Chapter 1 provides an introduction and explains the intent and organization of this thesis. Chapter 2 contains a report titled *Seismic Rehabilitation of Reinforced Concrete Buildings in California*, in which a review of current practice of seismic retrofits of buildings in the state of California is presented. This report was developed as part of a collaborative project between University of Texas at Austin and Universidad Nacional Autónoma de México with support from The University of Texas System and CONACYT through the ConTex program. Chapter 3 contains an article intended for publication titled *Lateral Deformation Capacity and Damage Fragility Analysis of Circular RC Columns Retrofitted with FRP Jackets*, in which the deformation capacity and damage fragility of FRP-jacketed columns is studied based on experimental data collected from the literature. Lastly, Chapter 4 draws general conclusions based on the findings from the two topics of this thesis.

## **Chapter 2: Seismic Rehabilitation of Reinforced Concrete Buildings in California**

Authors: Oscar Minor Garcia and Juan Murcia-Delso.

Contribution: Reviewed literature and project documentation, and conducted field investigations. Wrote the report, including the overview of seismic mandates and the description of the case studies, and created the graphics of retrofits of each building.

Presented as Appendix C3 in ConTex Collaborative Research Project:  
“Assessment of Seismically Retrofitted Concrete Buildings Following the 2017 Earthquake in Mexico City”

### **1. Introduction**

Earthquake engineering is a relatively young science; most of the advances in this field have been made in the twentieth century. The first strong ground motion recorded in the United States was Long Beach in 1933; however, the ground motion obtained in El Centro in 1940 provided the basics for dynamic study of structures. Particularly, significant development in seismic design was done in the last 50 years with the advance in technology and enhanced by large seismic events. For instance, Northridge in California in 1994 and Mexico City in 1985 highlighted the importance of the detailing of reinforcing steel in concrete structures to withstand cyclic loads like those produced by earthquakes. Such lessons motivated abundant research as well as changes in engineering standards and legislation to prevent the numerous failures observed.

Codes have been constantly updated to increase the safety of structures in active seismic areas, but typically, the regulations apply to the design of new structures. The question arises on what to do with the existing infrastructure that might not comply with

the increasingly strict codes. In this sense, the state of California has had a leading role in mandating retrofits of certain group of vulnerable structures to be upgraded to minimum safety levels. This report concentrates in the practical applications of retrofits in existing building structures.

There are countless types of structures, and codes do not exhaustively cover every one of them. Actually, the purpose of most of the regulations is to cover the maximum types of structures and to assign levels of safety to each group. In seismic design, there is normally a classification on the importance of the structure that differentiates those that are essential to the function and recovery of a city after a major earthquake, namely roads, schools, and hospitals. In the specific case of California, failures in these three types of structures in past earthquakes has guided legislation not only on new buildings but on seismically upgrading existing buildings and infrastructures so that they can remain functional after a major earthquake.

This report is part of a collaborative effort between the University of Texas at Austin and the Institute of Engineering of the Universidad Nacional Autónoma de México, aimed to document and report works on seismic retrofits of buildings in Mexico and the United States in order to characterize challenges and opportunities that are shared by both countries. A similar collaboration was made in 1989 where a group of researchers from different universities in the United States traveled to Mexico to observe and discuss with local researches and professionals about the retrofits done in Mexico City after the 1985 earthquake. The result of that workshop is a report prepared by Prof. James O. Jirsa and his collaborators in 1996 [1] where valuable information on the characteristics of the earthquake, the local code, and several cases study are described in detail. This collaboration also helped to start rehabilitation research projects that eventually led to an ample number of evaluation and rehabilitation manuals, guidelines, and standards. The

present work is intended to continue and renew the efforts of previous researchers ([1], [2]), which provided first-hand, detailed information about the rehabilitation of concrete buildings.

The specific objective of this report is to document recent ongoing retrofit interventions in existing concrete buildings in the United States. There is currently an intense seismic retrofit activity in the West Coast, especially in California, where efforts are being undertaken to assess and upgrade existing non-ductile concrete constructions and buildings that are vulnerable to soft-story failures and pose a high risk to the population. The case studies included in this report are entirely from buildings located in California.

Nowadays, there is significant laboratory data about the seismic retrofits, and comprehensive guidelines and specifications are available to perform seismic assessment and retrofit of existing buildings. The interventions in this report are analyzed in the scope of state-of-the-art knowledge, technologies, and design methods. This effort will identify challenges concerning upgrading of vulnerable buildings and it will also provide valuable data for the assessment of current practices.

Section 2 of this report is an overview of seismic mandates in California for upgrading existing hospital buildings, as well as a summary of the non-ductile concrete building ordinances in the city of Los Angeles. Section 3 presents five case studies of seismic retrofit projects in California, which include three hospital buildings and two hotel buildings. Finally, Section 4 provides a summary and conclusions from these case studies.

Information from the case studies was obtained from professionals involved in the design and construction of the retrofit project. Site visits were also conducted in 2019 for first-hand observation of the rehabilitation works.

## **2. Overview of seismic mandates for existing buildings in California**

The following overview is not an exhaustive presentation of seismic regulations in the state of California. Rather, it reviews the evolution of seismic mandates for rehabilitation of existing buildings, with particular emphasis on hospital buildings since most of the case studies presented in this report fit in this category. Seismic mandates for hospitals also exemplify how earthquakes have guided changes in legislation.

Other types of constructions have their own regulations. For instance, there has been a thorough action on highway infrastructure and retrofits on bridges by Caltrans, starting after the 1971 Sylmar earthquake and continuing after the 1989 Loma Prieta and 1994 Northridge earthquakes [3]. The seismic retrofit was completed in 2004 with over 2000 bridges retrofitted [4].

In the case of elementary schools, after the Long Beach earthquake in 1933 legislators enacted the Field Act, requiring new schools to be seismically designed. No further work was done for the rest of the twentieth century until 1999, when Assembly Bill AB 300 was enacted mandating a survey of the State's public-school buildings. The survey was completed in 2002 with a final report which concludes that "Public Schools in California are the safest in the nation" and "since the passage of the Field Act, in 1933, no school has collapsed due to a seismic event, and there has been no loss of life" [5] [6]. Besides this, no further action has been taken.

California building regulations are contained within Title 24 of the California Code of Regulations (CCR). Regulations for seismic retrofits for hospitals are contained mainly in two sections of Title 24: part 1, California Administrative Code (CAC) [7] and part 2 volume 2, California Building Code (CBC) [8]. CAC contains seismic evaluation procedures for hospitals and the definition of Structural Performance Categories (SPC) in

its chapter 6. CBC includes specific requirements for each SPC in chapter 34A. Title 24 of CCR also contains a set of requirements for historical buildings in part 8, and similarly, for existing buildings in part 10.

In addition to state legislation, cities have ordinances that address specific building characteristics, such as the non-ductile concrete buildings ordinance in the city of Los Angeles.

Figure 2.1 shows the main regulations in the state of California related with the retrofit of existing hospitals and ordinances of the city of Los Angeles related with non-ductile buildings.

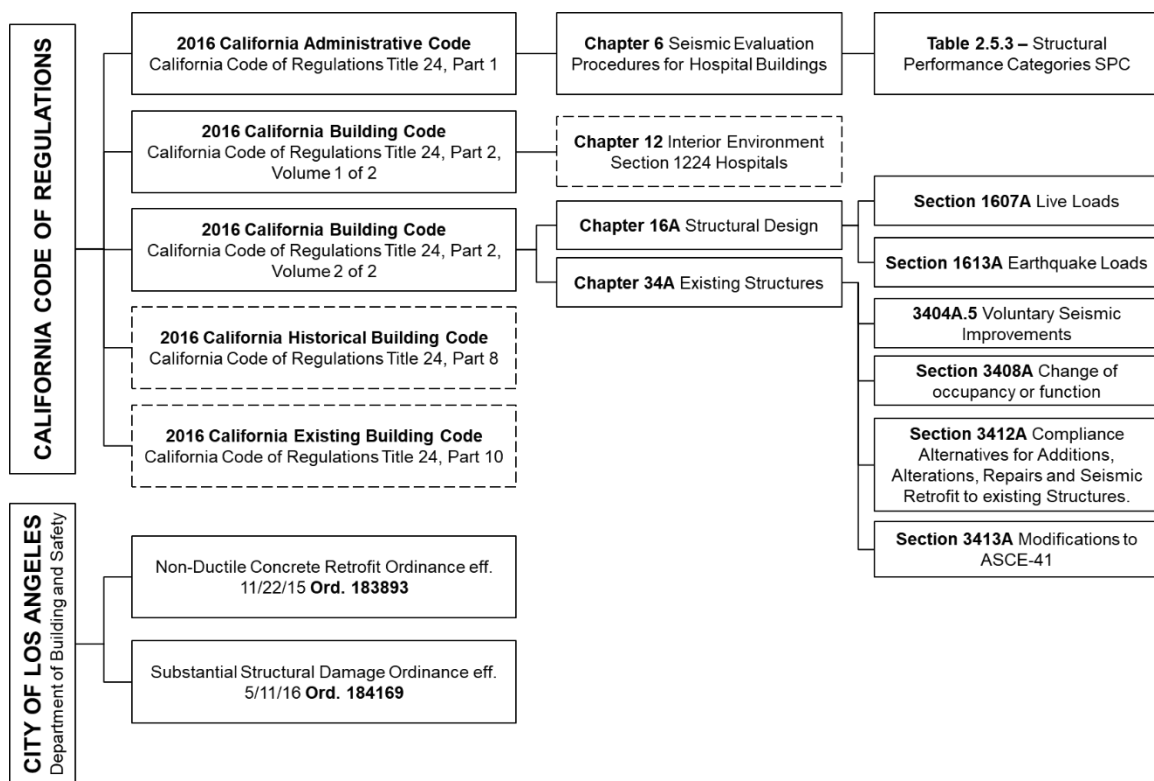


Figure 2.1. Codes that regulate evaluation of existing hospitals and design of retrofits of buildings in California

## **2.1 EVOLUTION OF SEISMIC MANDATES FOR HOSPITALS**

### **Prior to 1971**

The state of California is known for its high seismicity associated with strip-like faults that run across the state. Spanish explorers funded the first missions on the region in 1769, and as part of their reports they documented ground movements and their effects in the constructions. Missions were secularized in 1834, marking the end of these reports; however, another source of earthquake history can be found in written press during the rest of 1800's and the beginning of the 1900's [9]. Formal methodological study on the field in the United States was not made until the twentieth century, endorsed by the 1906 San Francisco earthquake; though, this event did not trigger any action in the building code.

The first seismic regulation in the United States was included as an appendix in the first edition of the Uniform Building Code of 1927, although the application was voluntary. The 6.3-magnitude Long Beach earthquake of 1933 encouraged legislation on seismic design of buildings with the passing of two acts, the first of which was the Riley Act that required every building of any character to be designed and constructed to resist, "in so far as possible, future earthquakes" [10]. The second regulation was the Field Act focused on seismic design of schools [11]. Both acts applied to new structures; they did not address existing buildings.

### **1971 San Fernando Earthquake and Hospital Seismic Safety Act**

The 6.6-magnitude Sylmar earthquake of February 9, 1971 in the San Fernando Valley caused the collapse of two hospitals: the Veterans Administration Hospital, a building from 1925, causing the loss of 49 lives; and some structures of the Olive View Medical Center, a new hospital dedicated one month prior to the earthquake that experienced the collapse of basements, lower stories and a carport [11] [12]. Four other

major hospitals suffered severe damage and a dozen of freeway bridges collapsed. The effect of this earthquake prompted legislators to pass the Alfred E. Alquist Hospital Facilities Seismic Safety Act (Hospital Seismic Safety Act) of 1973, whose intent was that “essential facilities such as hospitals should remain operational after an earthquake” [13]. Again, these regulations did not specifically address existing structures.

Legislators expected that the organic replacement of aging buildings would make hospital infrastructure to comply with the act within a time frame of 25 years. Nonetheless, hospitals were not being replaced at the rate anticipated, resulting in a continuously aging non-conforming infrastructure [14].

### **1994 Northridge earthquake and Senate Bill 1953**

Twenty years after the Hospital Seismic Safety Act, another major earthquake hit California: the 6.7-magnitude Northridge earthquake in 1994 where twelve hospitals built prior to the 1973 Act were severely damaged. Hospitals built in accordance with the 1973 Act resisted with minimal structural damage, but with significant nonstructural damage (plumbing and ceiling systems) that limited normal operations. [14]

As a result, new ordinances were formulated in the Senate Bill 1953 (SB 1953), enacted in 1994, which included mandates on both, structural and nonstructural performance of existing hospitals. SB 1953 is a 36 year plan which establishes deadlines for different stages of compliance of existing hospitals: an initial structural evaluation by 2002 to classify the structures into performance categories; all acute care service buildings must be able to survive earthquakes without collapse (life safety) by 2008; and the same group of buildings must have been structurally evaluated and retrofitted to fully comply with the act by 2030. The entity in charge of observing the compliance of these mandates is the Office of Statewide Health Planning and Development (OSHPD).

SB 1953 includes a classification of Structural Performance Criteria (SPC) that range from 1 to 5, with SPC-1 assigned to buildings posing significant risk of collapse following a strong earthquake and SP-5 assigned to buildings reasonably capable of providing services to the public following a strong earthquake. SPC-2 is assigned to a building posing a low risk of collapse, while SPC-3 and SPC-4 are assigned to buildings with pre-Northridge steel moment-resisting frames and built with a permit obtained through OSHPD. This last group of buildings may experience damage after a strong earthquake, inhibiting the possibility to continue service to the public. Table 2.1 summarizes the SPC-Categories with their expected performance.

Table 2.1 SPC Classification, deadline, and retrofit criteria

| <b>SPC Rating</b> | <b>Expected Performance</b>                       | <b>Permitted Code Year</b>        | <b>Good Until</b>      | <b>Retrofit Criteria (ASCE 41)</b> |
|-------------------|---------------------------------------------------|-----------------------------------|------------------------|------------------------------------|
| SPC-1             | Significant Risk of Collapse                      | Pre-1973                          | 2020 (with extensions) | N/A                                |
| SPC-2             | Low Risk of Collapse                              | Pre-1973                          | 2030                   | LS@BSE-1E                          |
| SPC=3             | Damaged, but may provide service after EQ         | 1973 – 1994 (Pre-Northridge MF's) | Beyond 2030            | N/A                                |
| SPC-4             | Some damage, but may provide service after EQ     | 1973-1989                         | Beyond 2030            | N/A                                |
| SPC-4D*           | Some damage, but may provide service after EQ     | Retrofits of Pre-1973             | Beyond 2030            | DC@BSE-1E<br>CP@BSE-2E             |
| SPC-5             | Reasonably capable of providing services after EQ | 1989-Present                      | Beyond 2030            | IO@BSE-1N<br>LS@BSE-2N             |

Similarly to the Seismic Performance Categories, SB 1953 establishes a classification for Nonstructural Performance Category ranging from NPC-1 assigned to buildings with equipment that does not meet anchorage and bracing requirements, to NPC-5 which is assigned to structures in which the different systems and critical equipment is braced or anchored, and that includes water, wastewater and fuel tanks so that the facility can remain operational for at least 72 hr. after a strong earthquake. The objective of these

categories is to reduce nonstructural damage in buildings that may prevent the hospital from providing service to the public after a strong earthquake.

The plan on SB 1953 was that by 2008, all hospitals were required to meet the criteria for an SPC-2 classification, that is, to survive earthquake without collapsing or posing the threat of significant loss of life. These SPC-2 buildings must be upgraded to SPC-5 by January 1<sup>st</sup> 2030 or be removed from general acute care service; in other words, these buildings can no longer be used as hospitals with patient service, but they can still be used as offices or nursery if a change in occupancy is obtained from OSHPD.

SPC-3 and SPC-4 are existing buildings that meet specific requirements. No building can be upgraded to SPC-3 or SPC-4; hence, there is only a finite set of buildings that fit in these categories. These SPC-3 or SPC-4 buildings are allowed to provide general acute care service up until 2030 and beyond.

By 2030, all critical care facilities must fully comply with the Act, that is, they must be SPC-3, SPC-4 or upgraded to SPC-5.

Figure 2.2 summarizes the time sequence of most relevant changes in legislation and occurrence of major earthquakes. As shown, the 1971 Sylmar earthquake led to the Hospital Seismic Safety act in 1973; later, the 1994 Northridge resulted in SB 1953 with the 36-year plan: SPC-2 for 2008 and SPC-5 to 2030.

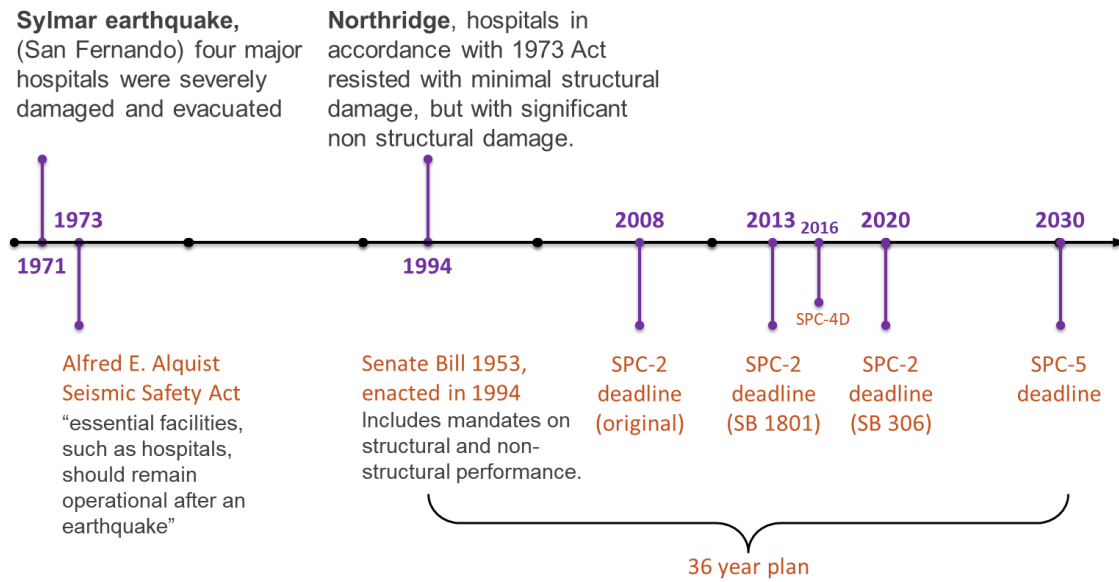


Figure 2.2. Evolution of legislation on existing hospitals and the original 36-year plan on SB 1953

### Deadline extensions and SPC modifications

By the time SB 1953 was enacted in 1994, most hospital buildings in California had been constructed prior to the Hospital Seismic Safety Act of 1973. Hence, a SPC-1 was assigned to these buildings. The rate at which hospitals were being retrofitted made difficult in many cases to meet the 2008 SPC-2 deadline. OSHPD allowed for extensions on this deadline on a case-by-case basis when the hospitals proved work on progress for compliance but encountered financial obstacles or when the works resulted in a reduced capacity for service to the community. In 2000, SB 1801 provided hospitals with a five-year extension; hence the SPC-2 deadline was moved to 2013. In 2001, evaluation of hospitals was completed classifying each building into a SPC category, providing a whole picture of the situation of hospitals in the state. In 2007, SB 306 authorized an additional seven-year extension for addressing SPC-1 buildings, resulting in a SPC-2 deadline for

2020. More recently, AB 2190 allowed for further extensions to 2022 or 2024 for hospitals that are working towards the SPC-2 [12].

These series of extensions have been made considering that in certain cases new construction is better than retrofit, allowing hospital owners to make plans for new buildings to directly comply with SPC-5 by 2030. The SPC-2 deadline has been changing; however, the 2030 deadline for full compliance with the act (SPC-5) has not been changed.

For the NPC categories the deadlines were similar to those for SPC. All SPC-1 buildings must obtain the NPC-2 for nonstructural category by 2020. Higher SPC structures must comply with the NPC-3 by 2024 if the seismic design category is F, or they must comply with NPC-4 and NPC-5 by 2030.

In 2016, a new Seismic Performance Category SPC-4D was introduced to the California Administrative Code, offering an option for non-conforming buildings to be upgraded below SPC-5. The reasoning behind this decision is that for SPC-1 buildings, upgrading to SPC-5 would result in performance higher than those considered compliant in cases of SPC-3 and SPC-4 structures. The performance level of a SPC-4D hospital is damage control, therefore, buildings conforming to SPC-4D will show a performance similar to SPC-3 and SPC-4 but return to functionality will not be as quick as that for SPC-5 buildings. Buildings upgraded to SPC-4D are allowed to operate beyond 2030.

There has been wide discussion about the real possibility for hospital owners to meet the objectives of the SB 1953 ( [14], [12], [11]) There is, clearly, an important economic aspect that has to be addressed to be able to comply with such mandates, especially because the legislation was silent about the origin of resources for the retrofits. Hospital owners were expected to cover the cost of retrofits with their own resources. By the time SB 1953 was enacted in 1994, most hospitals were making profits. This situation

changed in the late 1990's, where more than 80% of health care organizations were losing money, even without considering the cost of retrofits [11].

Hospitals have a limited number of options for compliance: retrofit of old existing infrastructure, construction of new buildings, or change the use of the building to a non-acute care service facility. In some cases, the cost of structural and nonstructural retrofit can be up to half the cost of a new building, thus it might be attractive for some hospitals to opt for new construction, but the time required to plan, get approval from OSHPD and construction is limited, and the cost is significant. There are some other constraints in certain cases due to the way hospitals have evolved throughout time, for example, when a non-conforming building was posteriorly expanded with a new structure built after the Hospital Safety Act of 1973, considered compliant under SB 1953. Such a situation complicates the retrofit project of the non-conforming building as it might involve works on conforming buildings as well. The option for new construction is also constrained by the physical space where new structures can be placed. Another point that owners must consider is that the costs associated with disruption and reduction of hospital activity can often be greater than the direct construction cost for seismic improvements [14]. Stopping service to demolish and build a new hospital is not an attractive option, and this condition has caused cases where the retrofits are being executed while the building is in service, which complicates the retrofit intervention.

## **2.2 2016 CALIFORNIA ADMINISTRATIVE CODE (CALIFORNIA CODE OF REGULATIONS TITLE 24, PART 1)**

Chapter 6 of the California Administrative Code (CAC) contains the seismic evaluation procedures for hospital buildings needed to implement the seismic retrofit requirements of the Alfred E. Alquist Hospital Facilities Seismic Safety Act of 1983. These procedures apply to all general acute care facilities. Conforming buildings are those built in compliance with the requirements of the 1973 of the California Building Code or classified as SCP-4D. All hospital buildings are assigned a Seismic Performance Category that ranges from 1 to 5. An evaluation must be done in order to assign the corresponding classification for each building according to the special characteristics that are described in the chapter. All SPC-1 buildings are to be removed from acute care service by 2008 unless the owner gets a special permit from the office. Change in the SPC or NPC for a hospital is only achieved either by collapse probabilistic assessment [HAZUS<sup>1</sup>] (1.4.5.1.2) or by retrofit analysis (1.4.5.1.3.). Unreinforced masonry shear walls or precast concrete buildings are not eligible to be re-classified as SPC-4D. The intent is that by 2030, all acute care hospital building must meet the requirements for SPC-3, SPC-4, SPC-4D or SPC-5, and the nonstructural requirements for NPC-5.

For the evaluation, besides a report with a general description of the building, the “General Sets of Evaluation Statements” have to be included. These sets of evaluation statements include a checklist of building characteristics. The statements are a summary of the requirements contained within the chapter.

Article 2 shows the load combination that should be used in the design ( $Q = 1.1Q_D + Q_L + Q_S \pm Q_E$  or  $Q = 0.9Q_D \pm Q_E$ ) and the basic base shear ( $V = C_s \cdot W$ ),

---

<sup>1</sup> HAZUS is a standardized methodology for estimating the potential losses from earthquakes, floods and hurricanes including physical damage, economic loss, and social impact. It is based on a probabilistic approach where different performance categories are analyzed qualitatively.

where  $C_s$  is the seismic coefficient which is a function of the peak acceleration coefficient or velocity coefficient, site coefficient, response modification factor and the fundamental period of the structure. The article provides equations to compute an approximate fundamental period. The force distribution along the height of the building is calculated with a linear distribution for short period structures and a quadratic distribution for long period structures. Torsional effects must be considered including a five percent of the dimensions at that level perpendicular to the directions of the applied force. For overturning, the counterbalancing moment is reduced by 75%. Diaphragm should be designed for a force of  $0.5A_v$  plus the shear force at that level ( $A_v$  is a velocity-related acceleration coefficient established in the code for each county). Parts and portions of the building are analyzed with a lateral force  $F_p$  which is a function of a velocity-related acceleration coefficient, a coefficient  $C_c$  and the weight of the component. The “quick check” of strength and stiffness provides basic checks of the overall story shear at every level (which is a function of the number and weights of the stories at that level and above), and from this value, a drift ratio can be computed considering the stiffness of columns, beams, modulus of elasticity and the corresponding deflection amplification factor. The shearing stress is checked with equations that take into consideration the sum of the areas resisting shear (all columns on a story, walls, or braces) and comparing it to the shear stress due to earthquake at that level. The element capacities are checked against the 1994 NEHRP Recommended Seismic Provisions [16].

Dynamic analysis is required for buildings 240 ft or more in height, with vertical irregularities, or where lateral force distribution deviates from the assumptions for the equivalent lateral force procedure. The response of all modes should be considered, thus, the participation of at least 90% of the mass must be included in the analysis. If the dynamic analysis results in lower base shear forces than those obtained from the equivalent lateral

force method (ELFM), the base shear must be scaled to equal the results of the ELM. Similarly, if the base shear force is greater, it can be scaled down to match those of the ELM. The basic acceptance criterion for each element is that the demand  $Q$  must be smaller than the capacity  $C$ . The element with the larger ratio is the critical element and an evaluation of whether this element will jeopardize the structure must be made, in case it will not jeopardize the structure, the element with the next ratio is the critical element. The SPC classification is made with these considerations. There are some deficiencies that classify buildings in categories 1, 3 or 4 without calculations. As mentioned, the SPC classification can be obtained using a probabilistic analysis. Table 2.2 provides the description of each of the SPC Classifications as it is included in the chapter.

Table 2.2 SPC Classification

| SPC   | Description                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                              |
|-------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| SPC-1 | Buildings posing significant risk of collapse and a danger to the public. These buildings must be brought up to the SPC 2 level by January 1, 2008, or be removed from acute care service. Where the Office has performed a collapse probability assessment, buildings with Probability of Collapse greater than 1.20% shall be placed in this category.                                                                                                                                                                                                                                                                                                                                                                                                                 |
| SPC-2 | Buildings in compliance with the pre-1973 California Building Standards Code or other applicable standards, but not in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act. These buildings do not significantly jeopardize life, but may not be repairable or functional following strong ground motion. These buildings must be brought into compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, its regulations, or its retrofit provisions by January 1, 2030, or be removed from acute care service. Where the Office has performed a collapse probability assessment, buildings with Probability of Collapse less than or equal to 1.20% shall be placed in this category |
| SPC-3 | Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, utilizing steel moment-resisting frames in regions of high seismicity as defined in Section 4.2.10 and constructed under a permit issued prior to October 25, 1994. These buildings may experience structural damage which does not significantly jeopardize life, but may not be repairable or functional following strong ground motion. Buildings in this category will have been constructed or reconstructed under a building permit obtained through OSHPD. These buildings may be used to January 1, 2030, and beyond.                                                                                                                              |
| SPC-4 | Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, but may experience structural damage which may inhibit ability to provide services to the public following strong ground motion. Buildings in this category will have been constructed or reconstructed under a building permit obtained through OSHPD. These buildings may be used to January 1, 2030, and beyond                                                                                                                                                                                                                                                                                                                                         |

|        |                                                                                                                                                                                                                                                                                                                                                                                                                 |
|--------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| SPC-4D | Nonconforming hospital buildings satisfying the requirements of Section 1.4.5.1.3 and the CBC 2016 Section 3412A.2.3 or equivalent provisions in later editions of the CBC. These buildings may experience structural damage which may inhibit ability to provide services to the public following strong ground motion. These buildings may be used to January 1, 2030 and beyond.                             |
| SPC-5  | Buildings in compliance with the structural provisions of the Alquist Hospital Facilities Seismic Safety Act, and reasonably capable of providing services to the public following strong ground motion. Buildings in this category will have been constructed or reconstructed under a building permit obtained through OSHPD. These buildings may be used without restriction to January 1, 2030, and beyond. |

Source: CAC Table 2.5.3.

The structure must have a load path to transfer all the inertia forces from the diaphragms to the lateral force resisting system down to the foundation. Redundancy is checked to guarantee the stability of the structure even if one critical element fails. Irregularities are classified in vertical and horizontal. Irregularities include the weak and soft stories (in terms of strength and stiffness, correspondingly), setbacks and reduction in 30% of the dimension in the lateral force resisting system, the reduction of 50% of mass in one story to the next, vertical discontinuities in elements that does not connect down to the foundation; and torsion, limited to 20% of the width of the structure, of eccentricity between center of rigidity and center of mass. The immediately adjacent building limit for any structure that is adjacent, with half the number of stories or with floors that do not match with the building analyzed, is 2 inches [5 cm] times the number of the stories of the building. Captive columns are defined as those with height to depth ratio of 75% of that of typical columns at the level considered. As for materials, deficiencies that are accounted towards the classification of the structure are: a) rusting or corrosion for steel; b) decay, shrinkage, splitting or loose metal accessories for wood; c) cracks of less than 1.0 mm without the X pattern in walls and columns for concrete, and d) easily scrapped mortar in the joints of masonry walls or cracks in the masonry infills.

- Moment-resisting frames

The critical features of this type of structures are the connections. A detailed frame joint evaluation has to be done if the building has experienced ground motions accelerations greater than 0.3g or an earthquake magnitude 6.5 that produced accelerations greater than 0.25g; and if significant damage was observed in one or more welded steel moment frames within 1 km around the building studied. Also, infill walls should be isolated from structural elements.

The following statements must be true to classify the structure to SPC-4D:

- For steel moment frames, openings in beams are less than  $\frac{1}{4}$  the beam depth, all the moment connections are with a full-penetration flange weld and a bolted or welded web connection. Column splices connect both, flanges, and web. Web thickness in the joint is capable of resisting shear. There must be continuity plates of the girder flanges in the joint. Strong column/weak beam is at least in half of the joints in each story and no less than  $\frac{1}{3}$  in every line of moment frame.
- For concrete buildings, no prestressed elements are included; the eccentricity in the joint is at least 20 % of the column dimensions. The shear capacity is larger than the moment capacity in frame member elements. The moment capacity of columns is at least 1.2 that of the beams. Hooks or stirrups end in  $135^\circ$  and are spaced  $d/4$  inches in columns or  $d/2$  inches in beams or 8db in plastic hinges zones or in splices zones, whose length is required to be at least 35db and they need to be located at the center half of the beams. Special attention receives the shear strength of the joint in which the joint shear is obtained with a free body diagram at a horizontal section at mid-height of the joint, and the forces developed by the expected flexural capacities of the beams on the left or the right. Flat slabs are assigned SPC-1 unless the building was designed and constructed with the 1989 or a later edition of Part

2, Title 24; or a detailed analysis is performed. In precast buildings the connections should be able to develop the capacity of the connected members. All these conditions are checked to keep the classification of conforming buildings to SPC 5 or classify them as SPC-4.

- Shear walls

This section focuses on shear walls subjected to in-plane effects. All walls that are not isolated from the structure are considered to act as shear walls.

For concrete shear walls, the height to length ratio should be less than 4. The overturning resistance (righting moment or wall flexural capacity) should be greater than 0.75 the moment at the base. Stirrups in coupling beams should be spaced less than  $d/2$  and must end in hooks of 135 degrees. Column splices should develop their tensile strength in the boundaries with shear wall. In case of partially welded splices, seismic load should be amplified to check adequacy of strength. Walls should include boundary elements according to ACI-318, for those with height to length ratio greater than 2, the boundary elements should be confined with spiral or ties spaced less than 8db. Rebar percentage in walls should be larger than 0.0025 in both directions, and maximum rebar distance should be less than 18 inches [45 cm]. If the rebar percentage is less than 0.0015 or distance between bars is larger than 18db, the wall should be ignored for shear. All openings should include special reinforcement.

Precast concrete shear walls are permitted but special analysis should be made for the connections to ensure the proper ductility. These connections should be able to allow the development of the yield level and capacity of the walls. Opening in walls in the perimeter should be less than 75% of the length, and the resulting wall piers height to length ratio should result in less than 2. When more than half of the lateral displacement is due to flexure, or the width of the piers is less than 5 times the thickness, the wall should be

analyzed as a moment frame. Collectors should be used in corner walls with opening larger than a typical panel.

For reinforced masonry walls, the sum of vertical and horizontal reinforcement should be greater than 0.002 with a minimum of 0.0007 in either direction. The spacing should be less than 48 inches [122 cm] and all vertical bars should extend to the top of the wall. Trim reinforcing should be included in wall openings. Unreinforced masonry walls structures are classified as SPC-1 unless the structure meets the probability assessment requirements. The allowable stress for solid masonry is 10 psi [69 kPa], for hollow unit masonry is 6 psi [41 kPa] and for grouted masonry is 12.5 psi [86 kPa]. The height to thickness is limited to 15 for single story buildings, and 13 for multistory buildings except for the top story which is limited to 9.

For unreinforced masonry infill walls in frames, the height to thickness limits are 14 for single story and 20 for multistory buildings, except for the last story, for which the limit is 9. Infill walls should be continuous and encompass the frame in the entire perimeter.

- Braced frames

Braces compressive ratio  $KL/r$  is required to be less than 120. Chevron V or K braces should be checked in detail, for instance, the beam should be a single element and able to resist gravity forces without the support of the braces, and the system should be analyzed for buckling of one of the legs of the bracing.

Concentrically braces are preferred for which the connection should be able to develop the yielding capacity of the diagonal. Columns splices in braced frames should be able to develop the column yield capacity. Concrete braces make the structure to be classified as SPC-1 due to ductility of the braced frame.

For eccentrically braced frames, the link beams should not be connected to the columns, due to a deficiency in the ductility of the connection.

- Diaphragms

Diaphragms are the horizontal system that distributes the lateral force to the lateral force resisting systems. These elements are treated as a horizontal beam where the shear is resisted by the web and the flexural forces are resisted by the edges of the slab in an analogy to the beam flanges. There should be enough tensile capacity in reentrant corners of diaphragms. Cross ties are beams or girders that span the width of the diaphragm, accumulate the wall loads and transfer them into the next bay. Cross ties should be continuous between diaphragm chords. Focus is made in the openings, which should be less than 25% the wall length or 25% the length of the bracing. For masonry shear walls the openings should be less than 8 ft [2.4 m] long. Reinforcing should be included if the opening is larger than 50% the building width.

For wood diaphragms, no straight sheathing or span to depth ratio greater than 2 is looked. Plywood or diagonal sheathing is required for spans larger than 24 ft [7.3 m].

- Connections

The connections covered in this chapter are primarily between walls and framing, and between diaphragm and foundation, whose failure would jeopardize the system. Walls must be properly anchored for out of plane loads. For anchors between diaphragm and masonry walls, the spacing should be less than 4 ft [1.2 m]. Tilt-up are connected with steel anchors embedded to the concrete and at least two connectors per panel are used. Diaphragms must possess sufficient capacity and must be connected for transferring the shear loads to the walls or steel frame. For frames, all columns must be properly anchored into the foundation, as well as longitudinal reinforcement of walls. Frame girders should not be welded to corbels and the bearing length should be larger than 3 inches [7.6 cm].

- Foundation

Evaluation of the foundation and of the capacity of the soil to withstand earthquake forces should be included for all seismic evaluations. The foundation must be checked for movements or settlements, visual examination of lower level walls, grade beams, and visible footings for cracks, yielding or buckling. Also, the foundation must be checked for deterioration including corrosion, sulphate attack, material breakdown that would decrease the strength of the element. For overturning, the ratio of the effective length of the foundation to the building height should exceed  $1.4A_v$  ( $A_v$  is the velocity-related acceleration coefficient). The vertical bearing pressure of the soil under seismic loading conditions due to gravity forces and overturning must exceed two times the allowable static pressure.

The capacity of the foundation to resist lateral forces must be larger than the base shear. All pile caps must be able to transfer the loads of the structure to each pile and piles or piers should be able to transfer all lateral forces due to seismic loads to the soil. Piers must have adequate confinement and flexural strength. The grade difference from one side to the other of the building is not higher than half of the story.

Geological investigation must be made to detect geological hazards and if present, that they do not threaten the integrity of the structure. The building must not be on soil prone to liquefaction for a depth of 50 ft [15 m] under the building, and it must be sufficiently remote from soil with potential slope failure and surface faults rupture are not anticipated.

- Elements not part of the Lateral Force Resisting system

This section covers nonstructural elements that are related with life safety concerns. Nonstructural walls that are not part of the lateral force resisting system, if not properly detached, can become load bearing. Unbraced unreinforced masonry partition walls should

not be in critical care areas. Partitions in exit corridors must have seismic or control joints. Veneers should be properly connected to the structure with no possibility for falling elements. Cladding should be connected with at least two or four connections for each panel. Metal studs for backup systems should be properly connected and corrosion of veneer, ties, and studs should be minimal. If veneer is 30 ft [9 m] above the ground, it must be supported with shelf angles. Parapets, cornices or ornaments must be properly anchored. Unsupported unreinforced masonry parapets or cornices must not exceed 1.5 heights to thickness ratio. Canopies must be anchored and braced.

- Critical Nonstructural Components

Article 11 covers nonstructural components and systems critical to patient care. Each building must be assigned a Nonstructural Performance Category based upon the degree of anchorage or bracing of nonstructural elements, including architectural, mechanical, electrical and hospital addition, as well as piping and machinery. Table 11.1 includes the nonstructural performance categories and the systems that are checked for each category. There are 6 categories for NPC, each one including the previous and additional systems to be braced. NPC-2 includes bracing and anchorage of communication systems, emergency power supply, bulk medical gas systems, fire alarm systems and emergency lighting equipment and signs in the means of egress. NPC-5 includes, besides the bracing and anchorage of systems, onsite supplies of water, holding tanks for sewage and liquid waste, and onsite emergency systems that are capable for 72 hours of emergency operations.

For a system to comply, it must have been installed under a permit issued by OSHPD or reviewed and approved by the Department of General Services, Office of Architecture and Construction, Structural Safety Section. If the system is braced but does not meet the previous requirements, it must be determined if the bracing is sufficient to

meet the requirements in Table 11.1. The bracing capacity must be determined based on structural drawings, and where they are not available or incomplete, the evaluation must be made based on the as-built condition, with the capacity of the fastener determined by testing. For anchors bracing pipes, ducts, or conduit, 20% of the anchors must be tested to three times the design load but no less than 500 lb [2.2 kN]. Shell-type anchors must be tested to four times the design load. A minimum of one anchor in a group of four must be tested. If one anchor fails testing, 20 anchors in the vicinity must be tested prior to resuming to the 20% anchor testing. For anchors used for bracing equipment and other components, a minimum of one anchor of a given size must be tested to three times the design load. The testing apparatus must not prevent the formation of the cone failure in concrete or masonry. Failure is defined as when the tension produces a slip of 1/8 inch [3 mm] or 1/16 [1.5 mm] for shell type anchors, a shear cone failure, concrete spitting, or fracture of the steel anchor prior to the target load.

### 3. Case studies

This section presents five cases of building retrofits recently conducted in California. The case studies include three buildings in hospital facilities and two hotel buildings. One of the case studies is located in the bay area, while the rest are located in greater Los Angeles. Figure 2.3 presents the approximate location of the buildings along with the fault activity map of the state of California. As shown, the buildings are located in areas with high seismic activity.

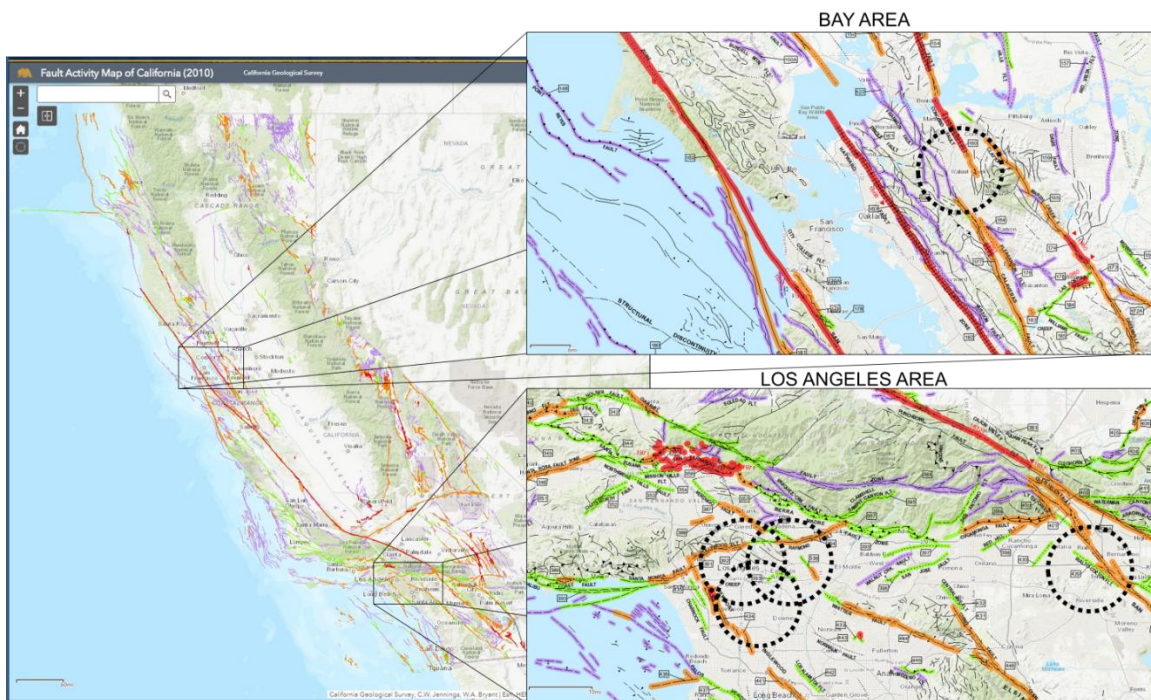


Figure 2.3. Fault Activity Map of California (2010), with details of San Francisco and Los Angeles. Source: <http://maps.conservation.ca.gov/cgs/fam/>.

### 3.1 BUILDING A. ELEVATOR TOWER RETROFIT

#### 3.1.1 Building description

Building A is located in the East Bay area of California. It is a six-story reinforced concrete tower housing the stairs and elevators of a medical center. The tower is adjacent but not directly connected with two buildings of the medical center (see Figure 2.4). The floor plan is a rectangle with dimensions 28'-5" [8.66 m] in the north-south direction, and 27'-10" [8.48 m] in the east-west direction. The total height of the structure is 61'-10" [18.85 m] (see Figure 2.5). The first two stories were constructed prior to 1959, and additional four stories (including a mechanical story) were constructed afterwards, as shown in Figure 2.6.



Figure 2.4. General localization of structure A

The foundation of the original structure comprises column footings which are typically 2'-4" [71.1 m] wide.

The lateral force resisting system comprises reinforced concrete shear walls in both directions. The strengthening plan includes the addition of new reinforced concrete shear walls as well as increasing the thickness of existing walls.

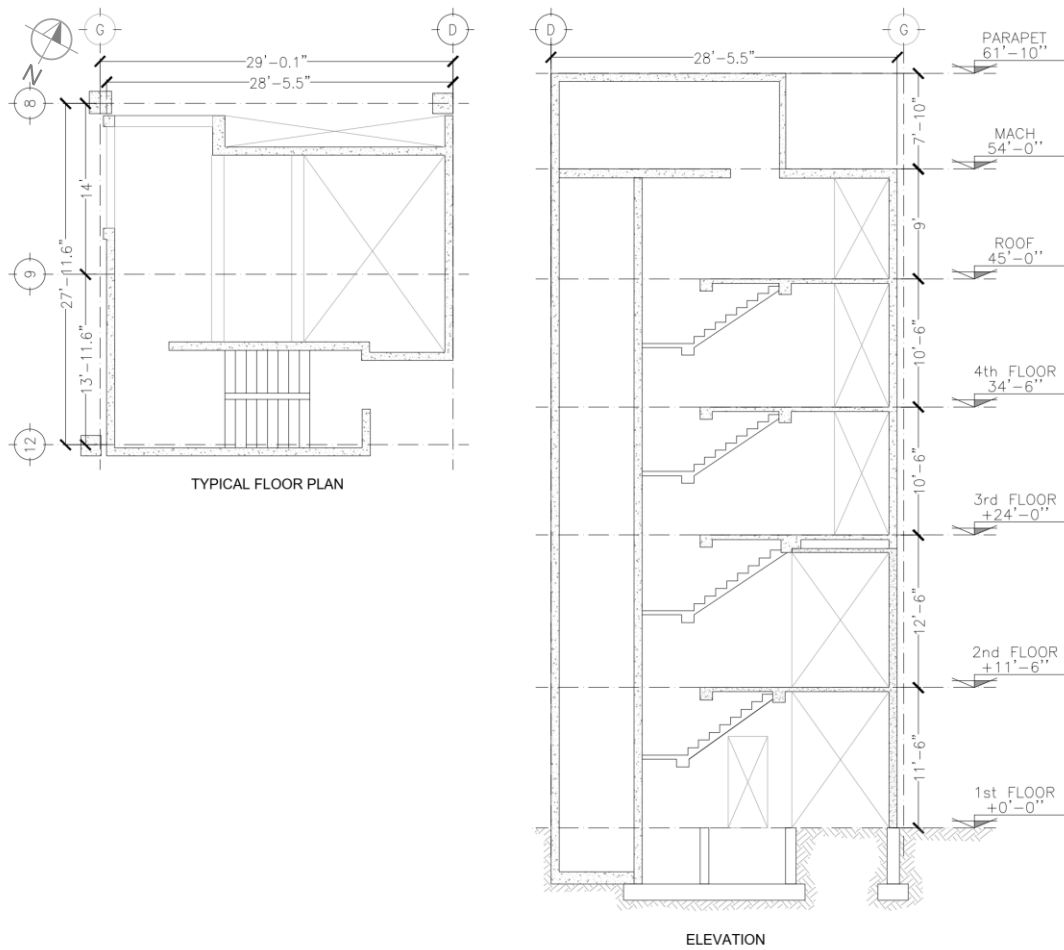


Figure 2.5. Floor plan and Elevation of building A

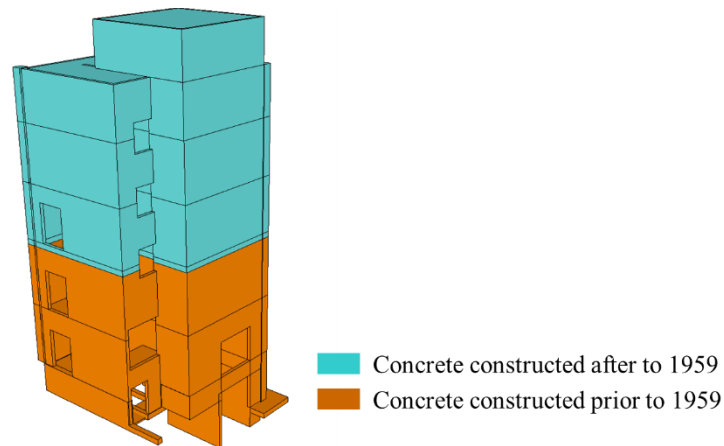


Figure 2.6. Construction sequence in Building A

### 3.1.2 Retrofit goal

The retrofit of the structure resulted from the application of the evaluation requirements in Chapter 6 of the California Building Code (SPC-2 upgrade, CAN 1-6-1.4.5.1).

### 3.1.3 Design data and analysis of the retrofit

For the seismic design of the retrofit, the seismic importance factor was taken as 1.0, with an occupancy category of IV per CBC Table 1604A.5: IV. Site class is D, Mapped Spectral Response Accelerations are  $S_s = 1.94 \text{ g}$  and  $S_1 = 0.70 \text{ g}$ . Spectral Response Coefficients are  $S_{ds} = 1.29 \text{ g}$  and  $S_{d1} = 0.70 \text{ g}$ . Finally, Seismic design category considered is D.

A three-dimensional computer model of the 6-story building was developed to demonstrate that the stair tower is stable laterally under seismic loads and does not overturn. Linear elastic elements with reduced cracked stiffness were used for the structure, and nonlinear compression only gap elements were used to model the behavior of the foundation. Time-history dynamic analyses were performed using seven sets of time

histories of near-fault motions with a maximum magnitude of 6.9 and a  $v_s = 30$  from 203 to 371 m/s. The time histories were spectrally matched to the site-specific BSE-I spectrum.

### 3.1.4 Strengthening Plan

The strengthening of the building is in compliance with 2010 California Building Code Title 24. The intervention consisted primarily of the infill of existing wall openings, addition of reinforced concrete walls, and strengthening of concrete members with FRP. For the stairs, additional steel beams were also introduced. Figure 2.7 shows the location of the main works on the structure.

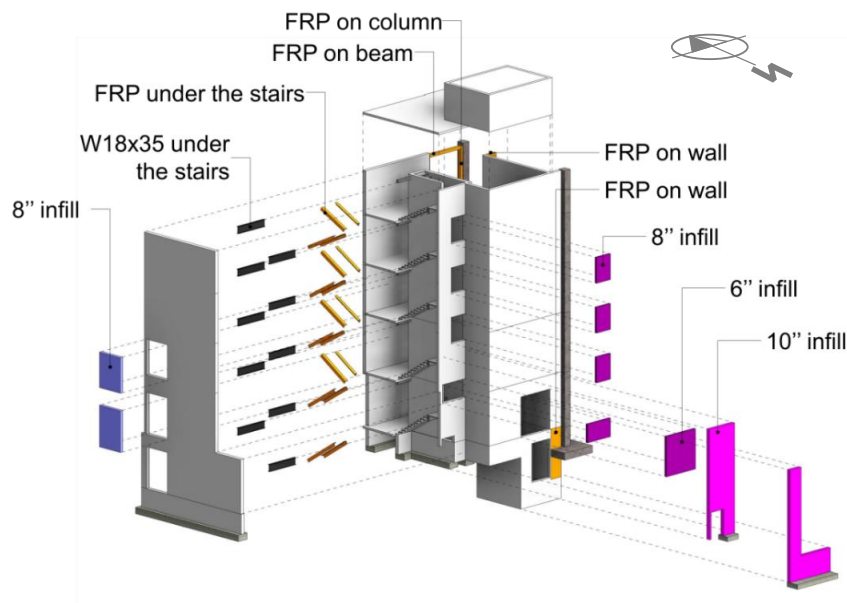


Figure 2.7. Strengthening plan for building A.

For the additional reinforced concrete walls, a footing with a typical width of 22'' [56 cm] and a height of 16'' [40 cm] was constructed. The surface of the existing wall was roughened and a combination of epoxied dowels and threaded bars was used to ensure

bonding between existing and new concrete (see Figure 2.8). The anchors were made of #4 or #5 bars with a spacing of 1' to 1'-3'' [30 to 38 cm]. The compressive strength of the new concrete was specified as 5,000 psi [34.47 MPa] for the wall and 4,000 psi [27.58 MPa] for the footings. Reinforcing steel was Grade 60.

The infills of existing openings also included #6 epoxied dowels embedded 1'-9'' [53 cm] into the existing wall. Dowels were located a minimum of 2'' [5 cm] and maximum of 3'' [7.6 cm] from the surface and spaced at 12'' [30 cm]. Where it was not possible to provide sufficient development length, threaded bars were used instead.

All the fiber-reinforced polymer (FRP) composites were specified to be Tyfo SCH-41 (ICC ESR-2103) with minimum design values as follows:

$$E = 11,900 \text{ ksi (82,048 MPa)}$$

$$F_{tu} = 121 \text{ ksi (834 MPa)}$$

$$t = 0.04 \text{ in/ply (1 mm/ply)}$$

FRP strengthening of beams followed a U-shaped configuration, with the fibers oriented vertically (see Figure 2.9). Half-inch-diameter ( $\Phi=12.7$  mm) steel anchor rods were used transversally to the member and plate washers were used on the sides of the beam to confine the FRP.

The 11'' [28 cm] square columns were strengthened by wrapping the element with two layers of FRP with no anchors (see Figure 2.9). For wall elements, the corners were grinded to create a round corner with a minimum radius of 1'' [25 mm] (see detail 4 in Figure 2.9). Carbon fiber-reinforced anchors (TYFO) were used to provide continuity when there was an interfering wall.

Stair risers were also reinforced with FRP in an L-shaped configuration. In this case, Carbon fiber-reinforced anchors (TYFO) anchors were used in both surfaces with 3'' [76 mm] embedment (see detail 7 in Figure 2.9).

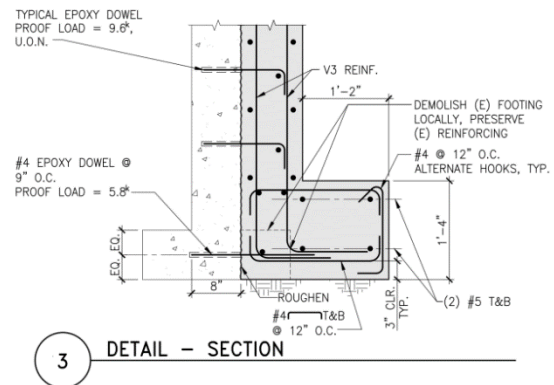
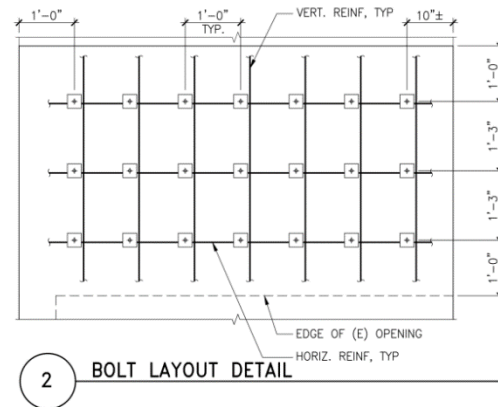
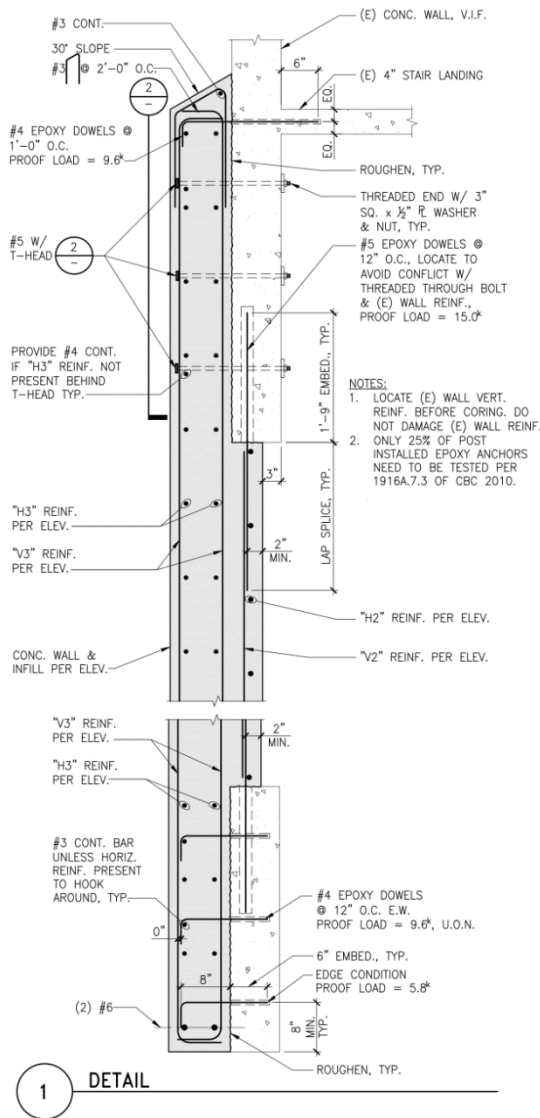


Figure 2.8. Details for anchoring new to existing reinforced concrete walls

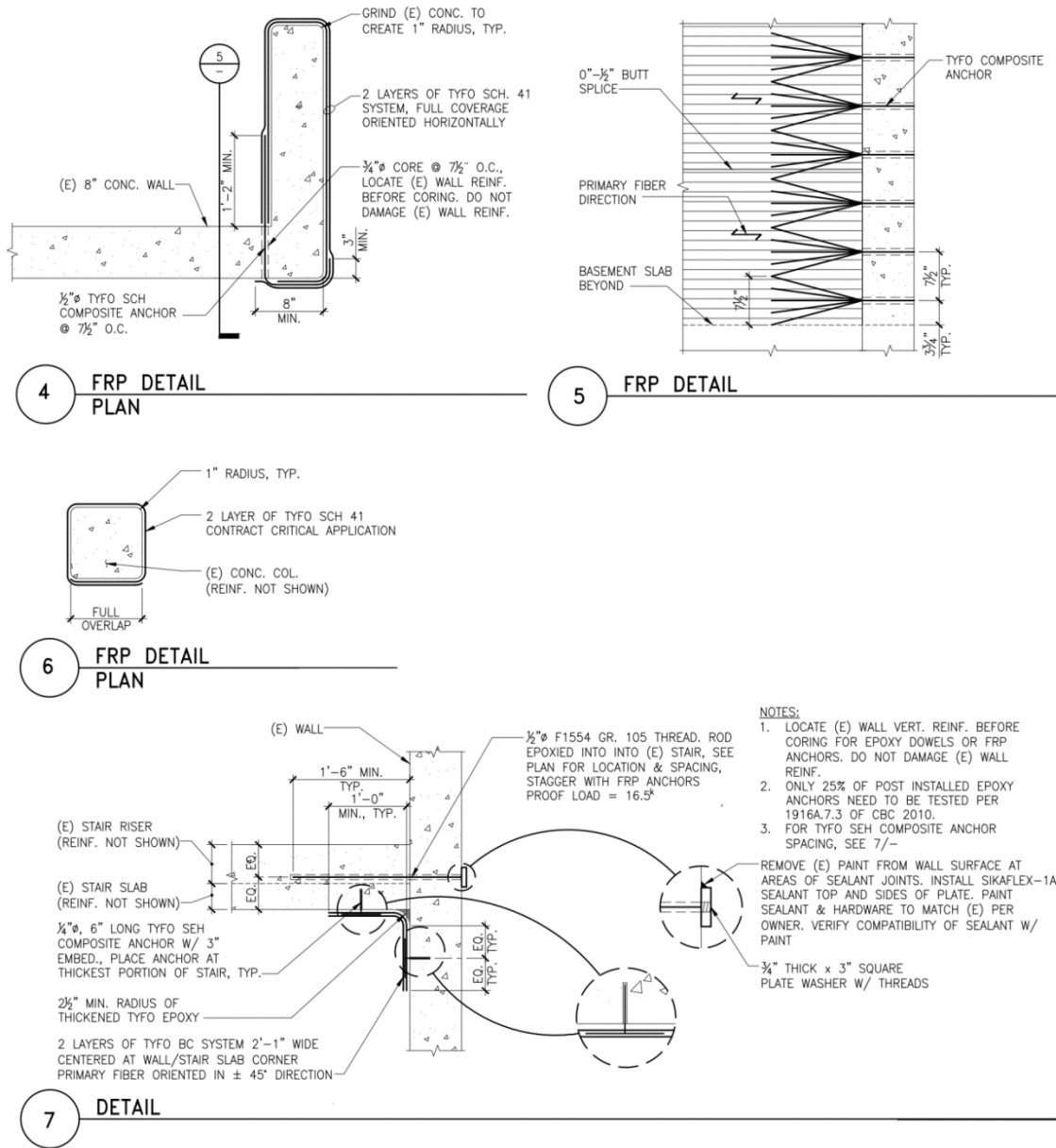


Figure 2.9. Details for FRP strengthening

## 3.2 BUILDING B. STEAM PLANT

### 3.2.1 Building description

Building B is a steam plant that serves a medical center in the San Bernardino area in Southern California. The original structure has a rectangular plan with four reinforced concrete walls in the perimeter. Figure 2.10 presents a general plan of the power plant complex with the steam plant in the bottom right corner. Figure 2.11 presents a general view of the power plant.

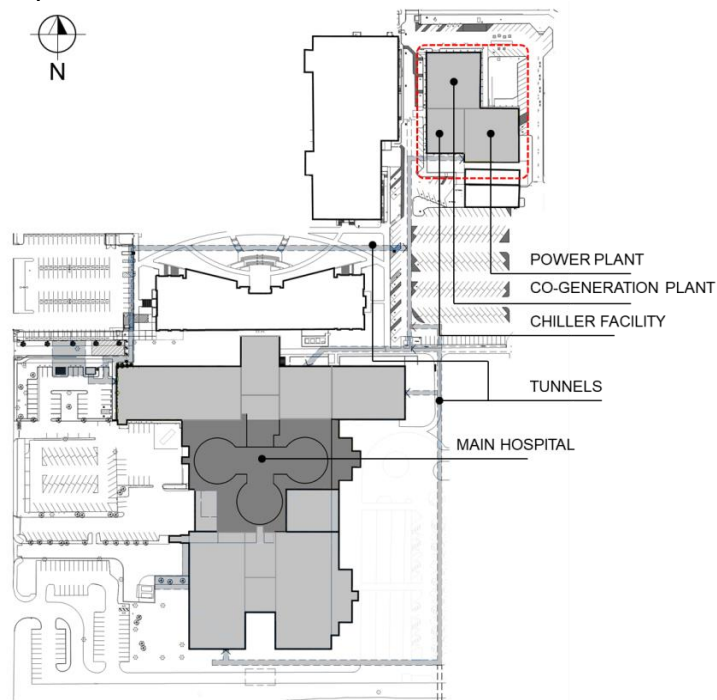


Figure 2.10. General plan of building B (Power plant).



Figure 2.11. General view of the power plant: chiller facility (left building) and steam plant (right building)

The steam plant is a single-story structure with the roof level located at a height of 26' [7.92 m] but with parapets in the perimeter walls that extends up to 42' [12.80 m].

### **3.2.2 Retrofit goal**

The steam plant structure was classified as SCP-1 (significant risk of collapse) and NPC-2 (only emergency systems are braced or anchored) for nonstructural components based on the OSHPD rating. The other two structures in the power plant complex were classified as SPC-4 (expected to experience damage but may provide service after an earthquake) and NPC-2 for nonstructural components. OSHPD requires this type of structures to comply with the same standard as the hospitals they are serving, hence, an upgrade plan to bring all the structures of the central plan to an SPC-4D level and NPC-5 was implemented (See Figure 2.12).

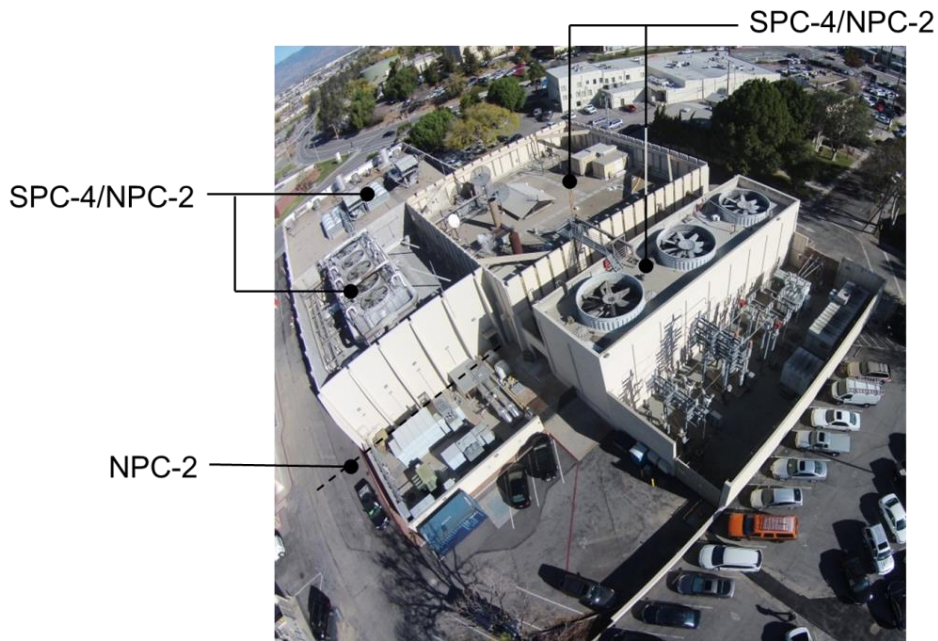


Figure 2.12. Original classification for the different structures of the power plant. These structures are being upgraded to SPC-4D and NPC-5 standards

### 3.2.3 Design data and analysis

The building was analyzed and retrofitted to meet Damage Control at BSE-1E and Collapse Prevention at BSE 2-E in accordance with ASCE 41-13. Linear Static Analysis was used to evaluate the superstructure using a fixed-base model developed in SAP2000. A separate foundation analysis was conducted in SAP2000 using nonlinear spring elements representing each of the existing and new pile's vertical and lateral stiffness, including group action effects.

### 3.2.4 Strengthening Plan

The steam plant is a rectangular structure with reinforced concrete panel walls in the perimeter, with small windows in the gaps between panels. These walls did not have the sufficient strength to withstand the shear forces resulting from the seismic action as

determined from analysis. The strengthening plan included infills of the wall openings to provide continuity between concrete elements so that the required shear strength could be reached (see Figure 2.13, Figure 2.14 and Figure 2.15).

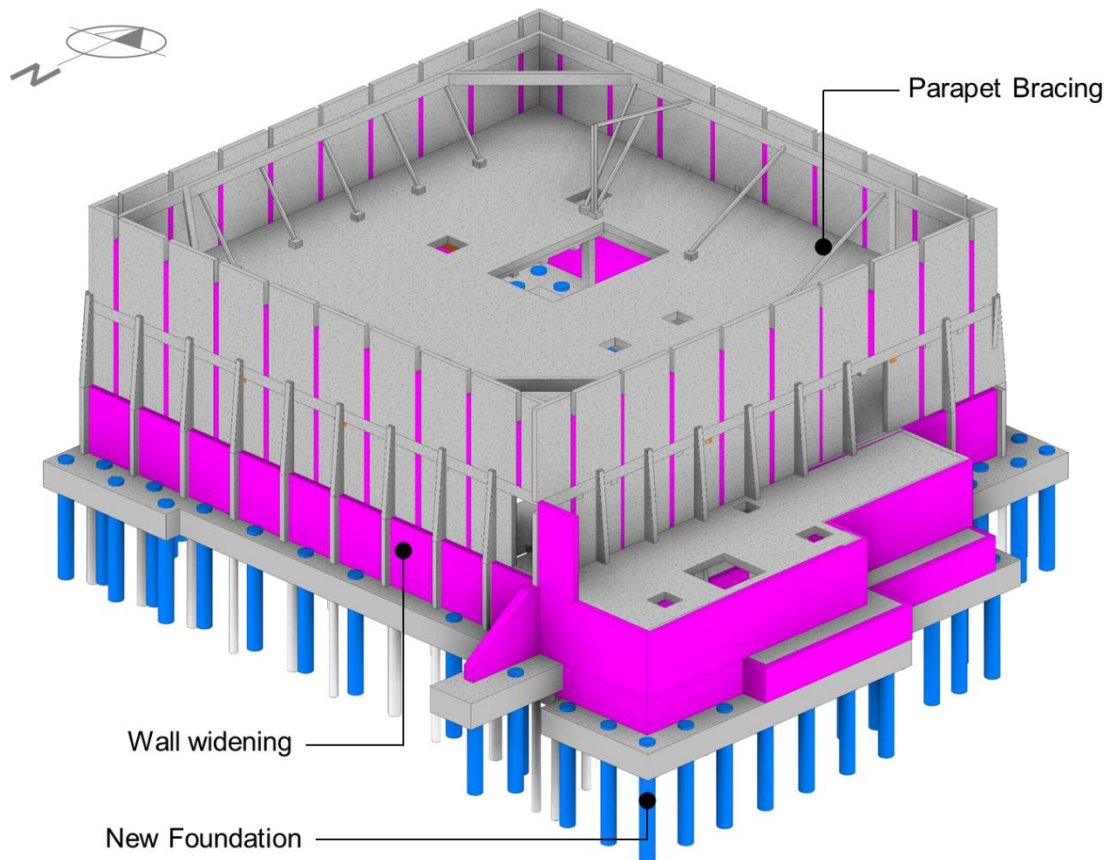


Figure 2.13. 3D model of the steam plant of building B.

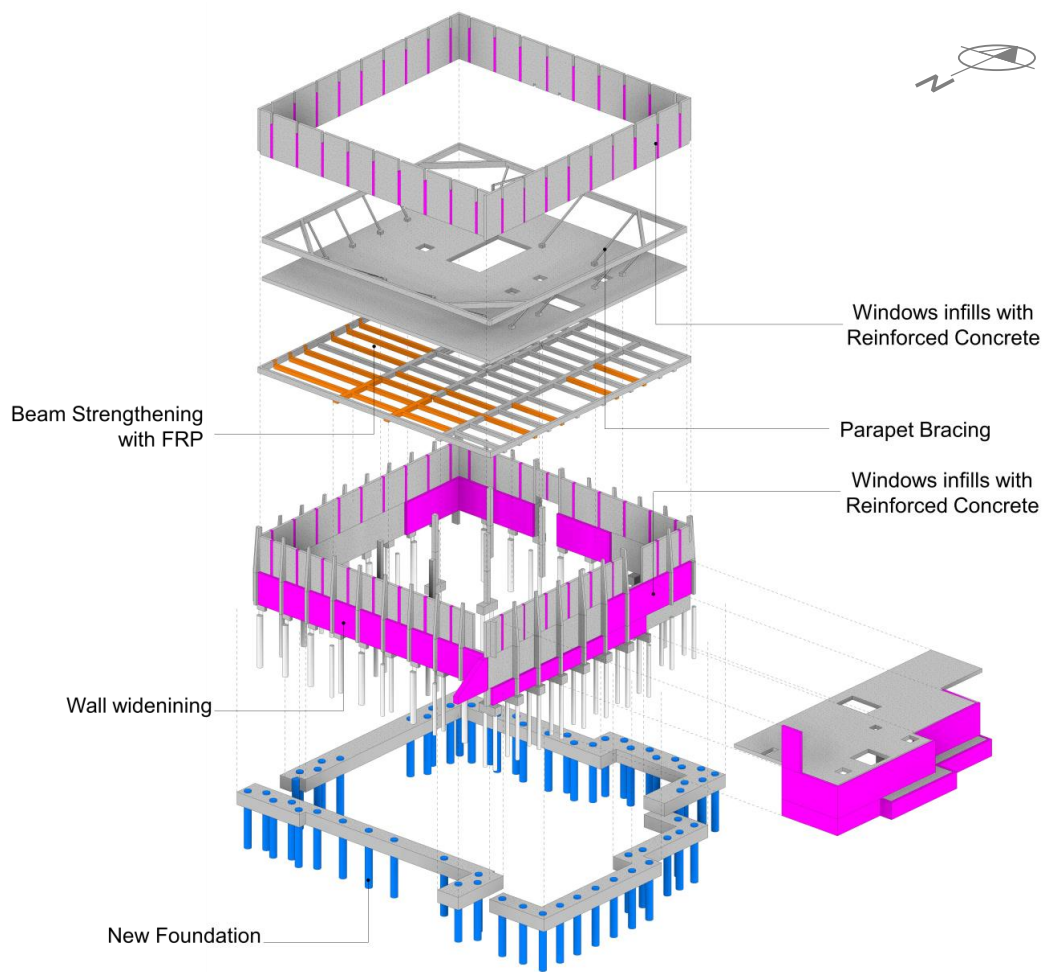


Figure 2.14. Strengthening plan for building B.

The infills procedure started by removing 1' [30 cm] or less of the existing wall at both ends of the opening. Two different approaches were used to provide continuity of reinforcement: the first consisted of placing horizontal rebars matching the size and spacing of existing rebar and welding them to existing bars. The second approach used mechanical rebar couplers to connect new and existing rebar at the opening ends and splicing the new bars at the center of the infill (Figure 2.16).

Additionally, the walls in the perimeter at the basement level were widened with new concrete walls casted against the existing elements. Number 5 bars spaced a typical distance of 6 in [15 cm] were used as dowels to the existing wall.

The shear walls in the perimeter of the steam plant will introduce higher forces to the existing foundation, which was an old and brittle structure. New piles were required to carry these forces. The strengthening plan includes a wall in the perimeter at the level of the foundation, for the connection of the building with the new piles. Figure 2.17 shows the distribution of piles (highlighted in red) in the steam plant, and the extent of the new footing (highlighted in blue) that connects them to the structure. Note that the foundation of the east wall is modified with respect to the building perimeter and moved to the right due to the existence of an electrical room in the basement. To correct the lack of number of piles right under the wall, a buttress was projected in the bottom right corner of the building.



Figure 2.15. View of the wall attached to the foundation and the infills in the shear walls.

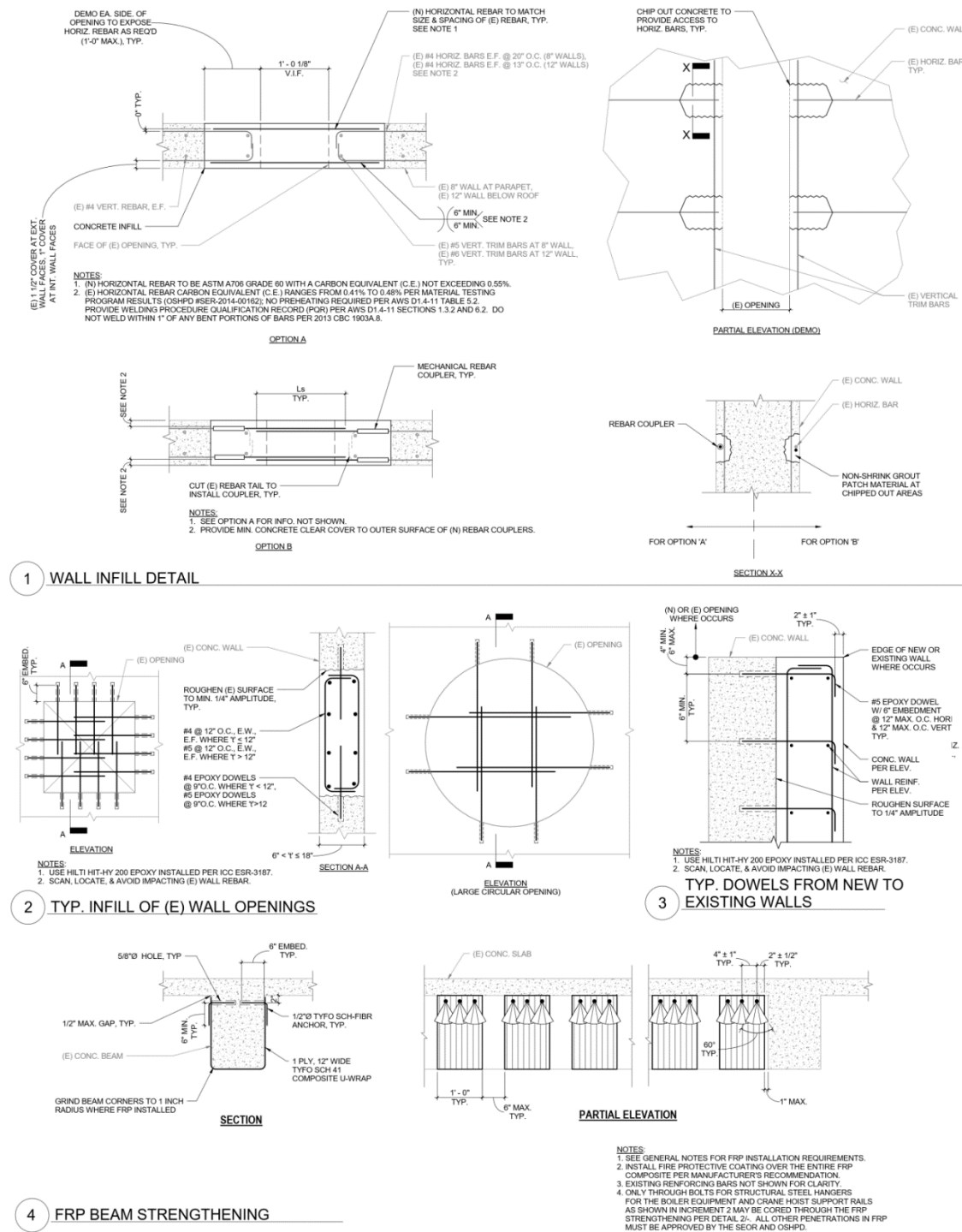
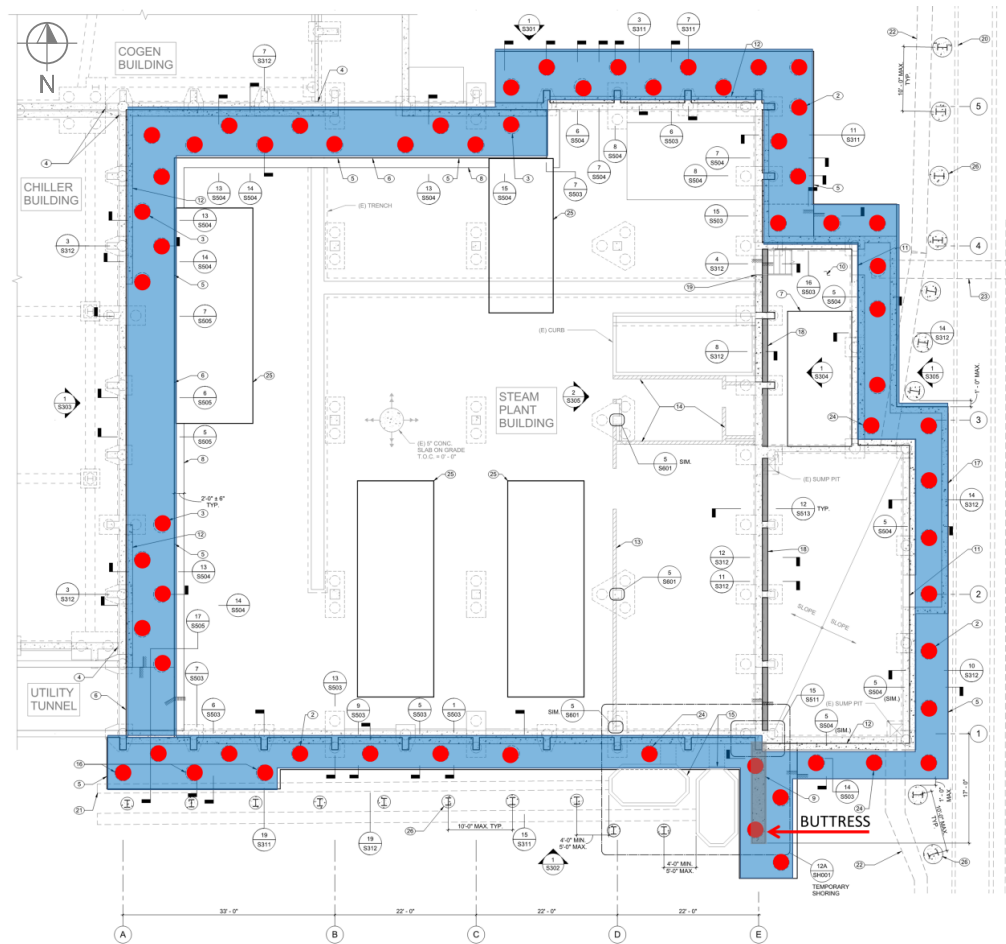


Figure 2.16. Details of wall infills, increase of wall thickness and FRP strengthening of beams



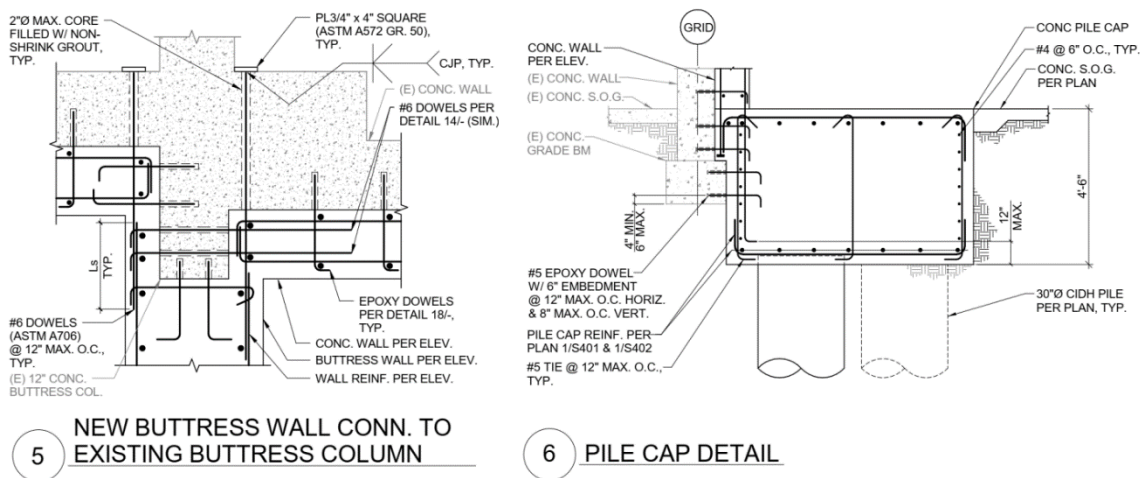


Figure 2.18. Details of connection of new foundation to existing structure

The parapets of the structure were considerably tall. To prevent damage under earthquake forces, the original structure included a coupling beam in the entire perimeter. In the new analysis, the contribution of this beam was determined insufficient and additional bracing was required. The bracing consists of diagonal HSS members bolted to the coupling beam and to the roof slab (see Figure 2.19). These members are made of galvanized steel as they are exposed to the outdoor environment and steam coming from the plant. The connection of the bracing to the parapet is executed with a plate with stainless steel anchors embedded 7.5" [190 mm] into the reinforced concrete beam. The connection to the slab is done with two plates at both sides of the slab connected with 1" [25 mm] diameter thru bolts (see details in Figure 2.20). In the roof, all the equipment and pipes are bolted down to the roof slab.

The beams in the roof were strengthened with FRP laminates to account for additional equipment weight with the detail shown in Figure 2.16. In addition, a large portion of the roof slab was repaired as corrosion on the steel bars was found while working on that area.



Figure 2.19. Works on the roof: Equipment bolted to the slab (top left), cable bracing of pipes in cantilever (top right) and bracing of parapets with diagonal elements (bottom).

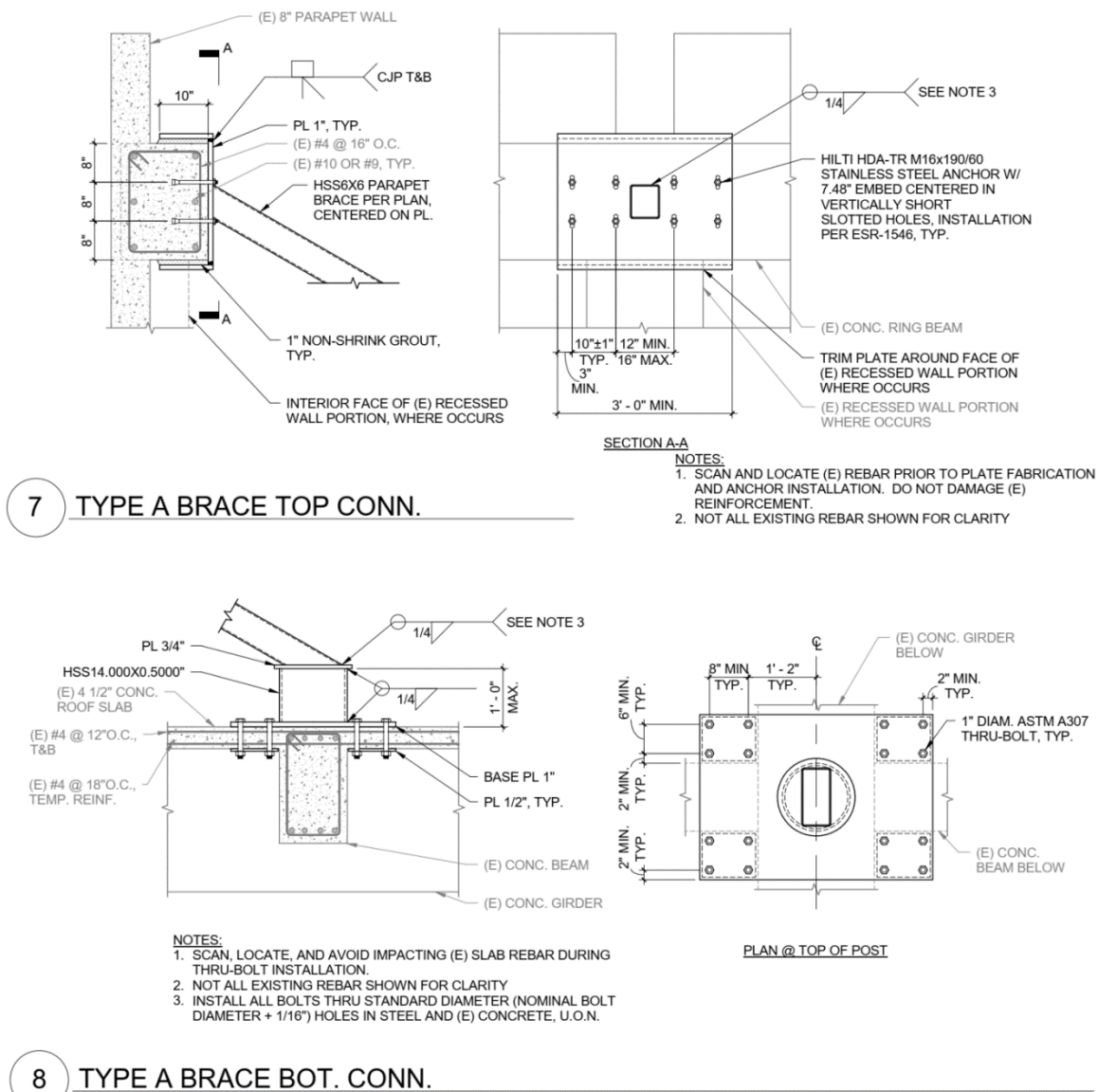


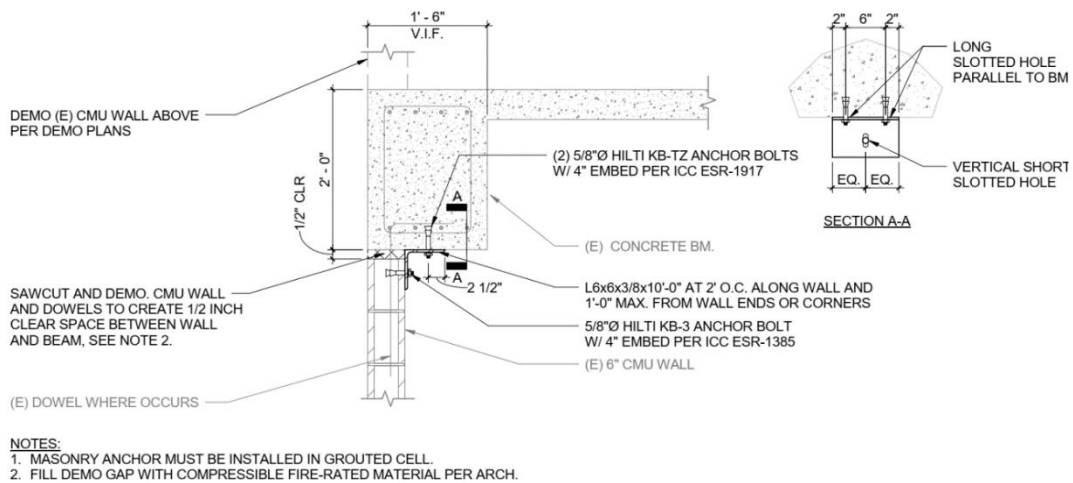
Figure 2.20. Details of the connection of the bracing in the parapet.

Inside the plant, there were a number of unreinforced masonry walls that were never intended nor designed to sustain shear load but could attract shear loads according to the evaluation. Such walls were demolished or, in cases where the new project required division walls, detached from the structure, as shown in Figure 2.21. Epoxy fire rated

material was used to fill in the new gap in order to maintain its fire rating. Plates with through anchors were placed to prevent out-of-plane failures (see detail in Figure 2.22).



Figure 2.21. Detachment of masonry wall from the structure.



## 9 DETAIL

Figure 2.22. Details of the detachment of a masonry wall from the structure

For the nonstructural elements, all the utilities including equipment and pipes were bolted and braced (see Figure 2.23).

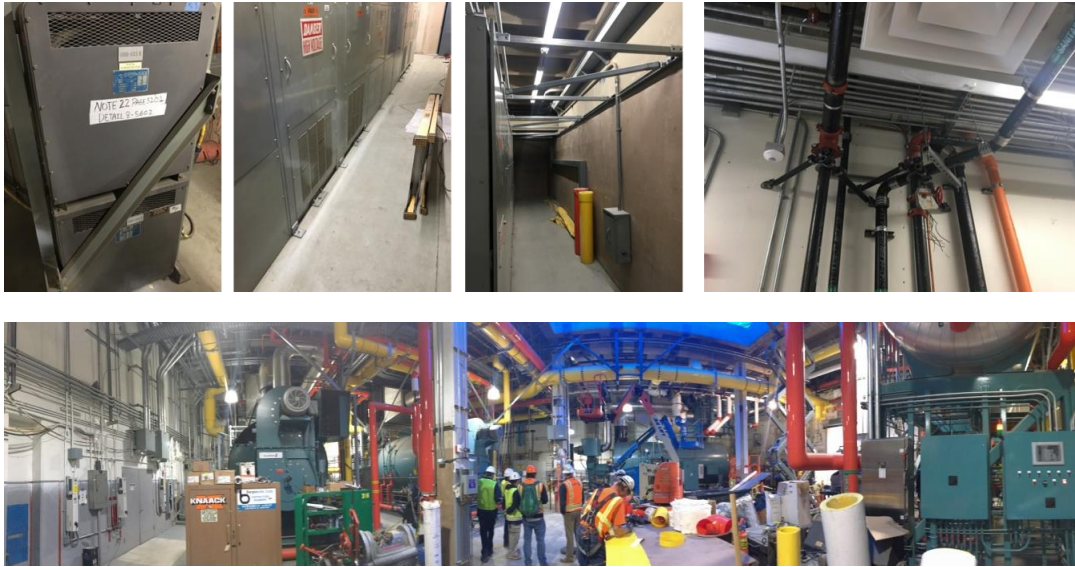


Figure 2.23. Different details of the bracing and bolting of the equipment in the plant

One of the main constraints of this project is that the hospital had to remain in service during the intervention. Hence, the retrofit work had to be made with the entire equipment in operation at all times.

### 3.3 BUILDING C. HOSPITAL OFFICES

#### 3.3.1 Building description

This structure allocates offices for a hospital in Los Angeles County. To the west, the structure is bordered by two buildings built in 1921 and 1938 as shown in Figure 2.24. A service building communicates with Building C at the southern section. No structures are located at the other two sides of the building. The building is irregular in both, floor plan and elevation, with a foundation at different levels and with typical floor plans presenting a T shape. The total length of the building is 344' [104.85 m] and in the transversal direction the typical dimension is 96' [29.26 m] with 200' [61 m] in the widest section. In elevation, it consists of a basement, ground floor, four typical floors, roof level and a penthouse. The story height is 12' [3.66 m] up to the second floor, 11'-8 1/2" [3.57 m] in the next floors and 15' [4.57 m] in the pent house, reaching a total height of 86'-1 1/2" [26.25 m] from the basement to the top.



Figure 2.24. General localization of building C.

The basement floor is rectangular and has reinforced concrete wall around its entire perimeter. The ground floor includes an addition to the north and to the west. First, second

and third floor plans maintain the similar T shape. For the fourth floor, the addition to the west is cut so that the floor plan looks like a rectangle. Finally, the penthouse is made of two bodies with steel frames (see Figure 2.26 and Figure 2.27).

The structural system is formed by pre-Northridge moment frames with W shape steel columns and beams. Some of the steel columns are encased with concrete. Concrete walls are also located in an irregular pattern in the limit and around the elevators.

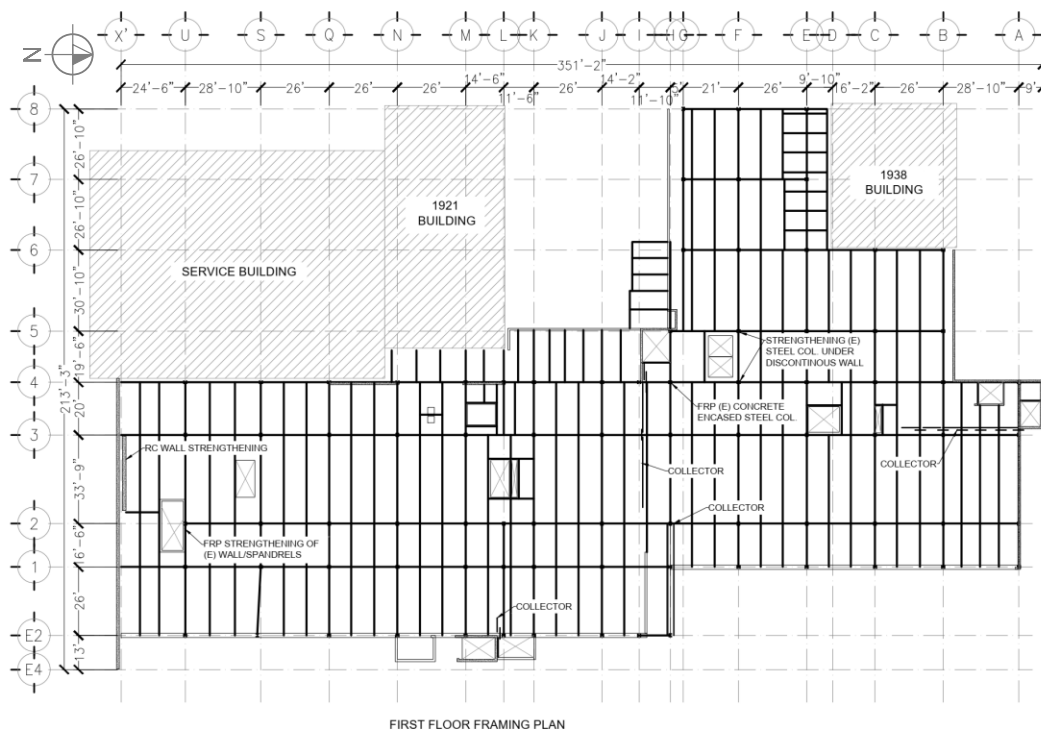


Figure 2.25. First Floor framing plan

### 3.3.2 Retrofit goal

The intent of the strengthening plan was to mitigate torsional irregularities encountered in the building. The strengthening was voluntary; as a consequence, no SPC specification is

targeted. Additional seismic evaluation is required to upgrade the building to a SPC-2 specification.

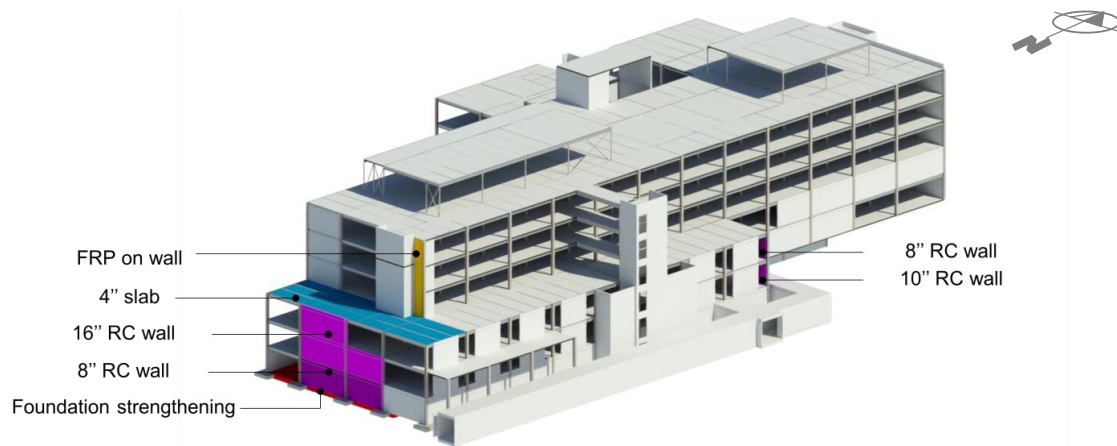


Figure 2.26. 3D view of building C with the main retrofit actions

### 3.3.3 Design data and analysis

The seismic design parameters for the retrofit were: risk category IV, seismic importance factor was taken as 1. Site class is C, seismic design category is F. The structure was analyzed with a linear dynamic method.

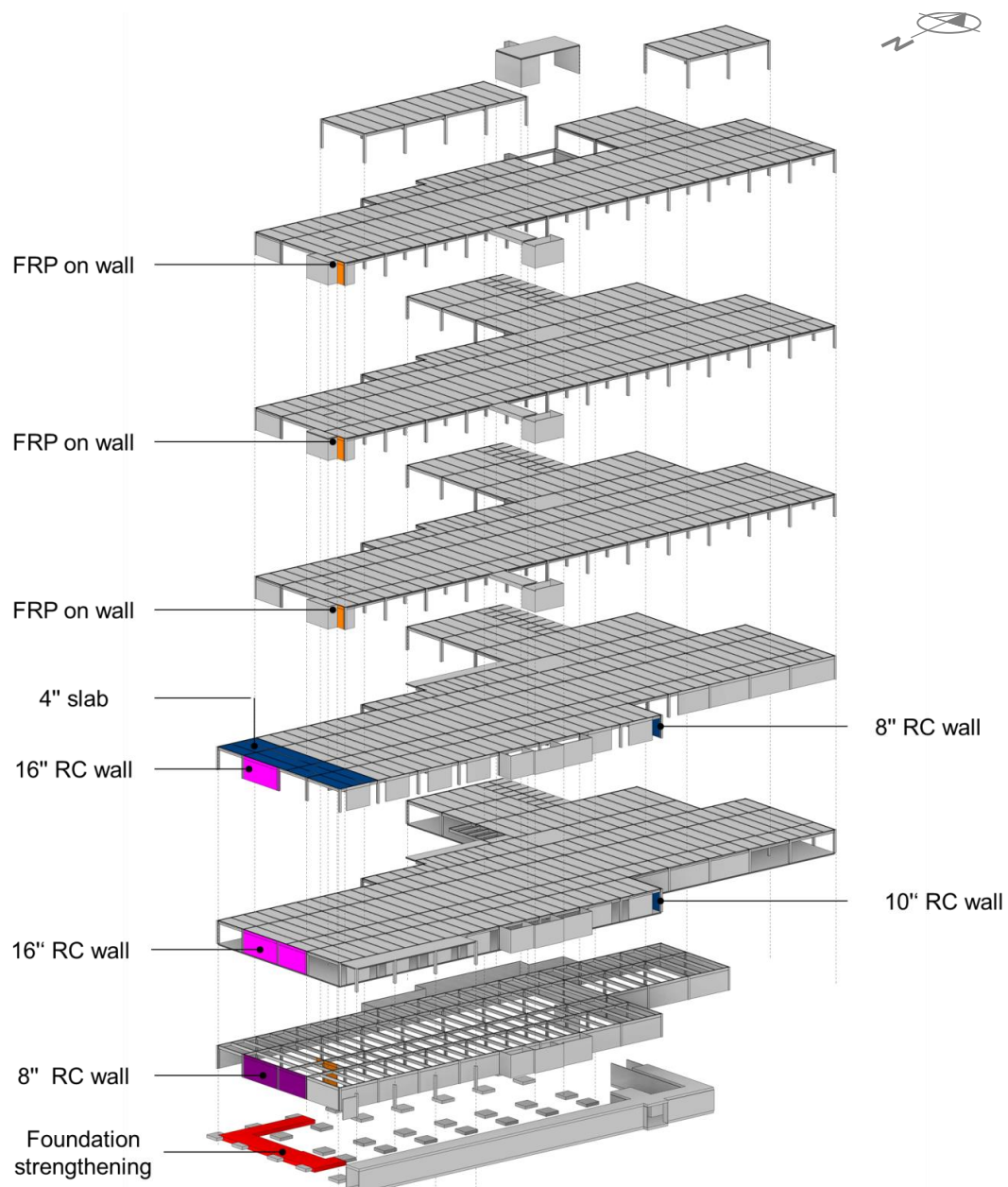


Figure 2.27. Extruded view of Building C. Note the high irregularities in floor and elevation.

### 3.3.4 Strengthening plan

The addition of new structural elements mainly focuses on two types of actions. First, the construction of a reinforced concrete wall on the southern portion of the building to resist lateral forces and to reduce torsional eccentricity (see Figure 2.26). The second, the construction of a series of collectors used to transmit lateral forces from the diaphragms to the lateral force resisting system (Figure 2.25).

The compressive strength of concrete specified for new elements is 6,000 psi [41.36 MPa] for footings, structural toppings, slabs, and walls. ASTM A706 reinforcing bars with 80 ksi [551.58 MPa] were specified for the foundation. The rest of the reinforcement was Grade 60 steel.

On grid line X', in the extreme southern portion of the building, there is an existing 8'' [0.20 m] thick reinforced concrete wall that is to be increased with a 8'' [0.20 m] overlay so that the total width of the wall from the ground to the first floor will be 16'' [0.40 m] (see Figure 2.28). The reinforced concrete overlay will be connected to the existing wall with #4 hooked dowels from which 25% are specified to be tested to 4540 lb [20.2 kN]. The existing wall is roughened to a 1/4'' [6.35 mm] amplitude. The wall overlay will be reinforced with #9@9'' each way. Details of connection at the floor level include the addition of a beam with reinforcing bars welded to the existing steel girder (see detail 3 in Figure 2.29). From the ground floor to first floor, the wall will be completely new with a constant thickness of 16 in [0.40 m] with steel #8@9 in. From first to second floor, the wall continues with same thickness and reinforcing but with smaller length (28.5 ft. [8.68 m]).



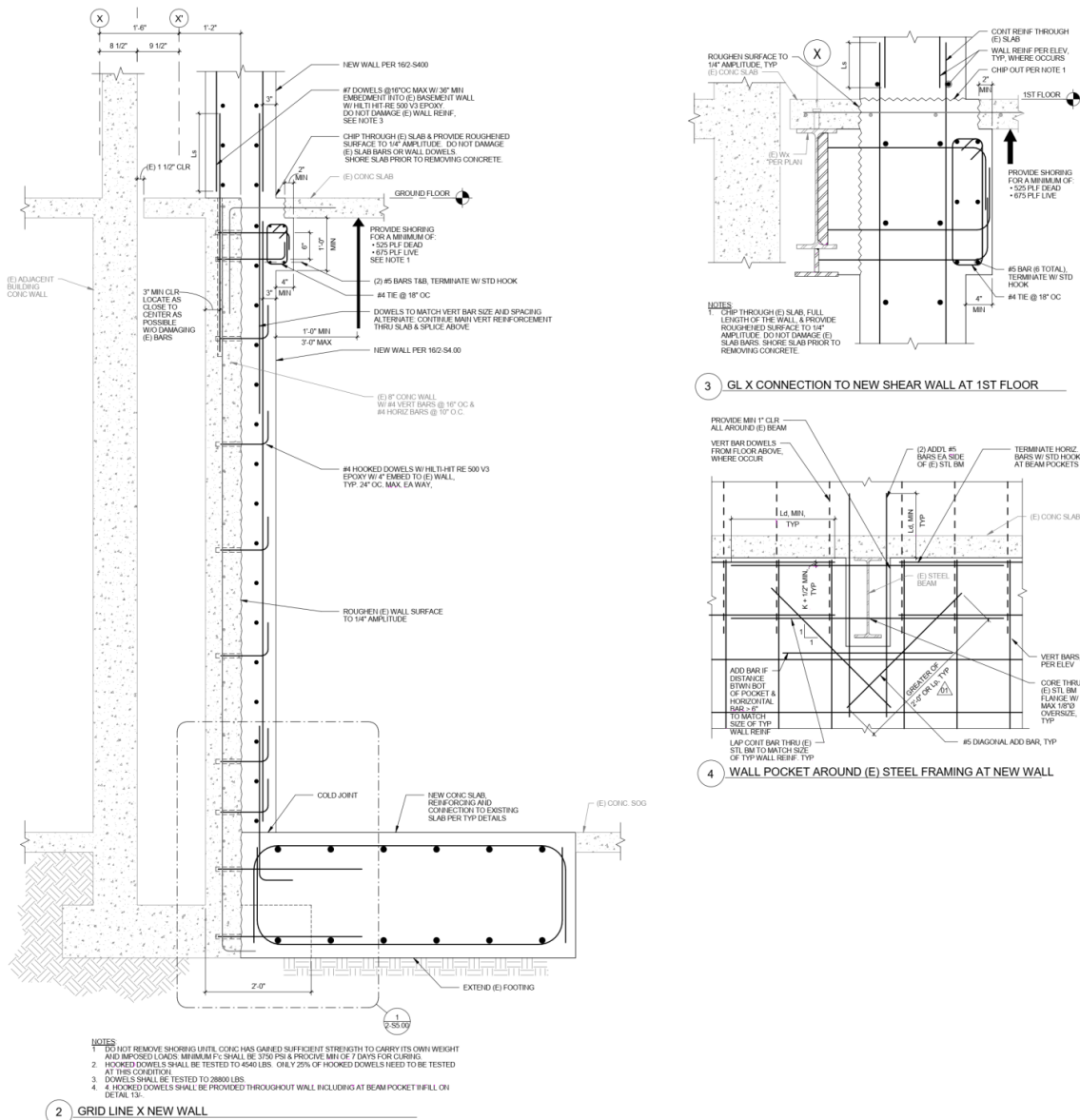


Figure 2.29. Details of connections of new wall on grid line X.

A second wall is going to be completed on grid line H. From the basement to ground floor there is an existing 10'' [0.25 m] concrete wall. From the ground to first floor, a new wall with thickness of 10'' [0.25 m] is planned with an opening of 8'x8' [2.43x2.43 m]. The reinforcement for this section is with #6@12. Finally, from first to second floor an 8''

[0.20 m] concrete wall is planned to replace an existing brick wall. The same type of reinforcement will be used. Figure 2.30 shows details on how the steel bars are passed through the concrete column (detail 6) and how steel bars are welded to the existing encased steel column (detail 7).

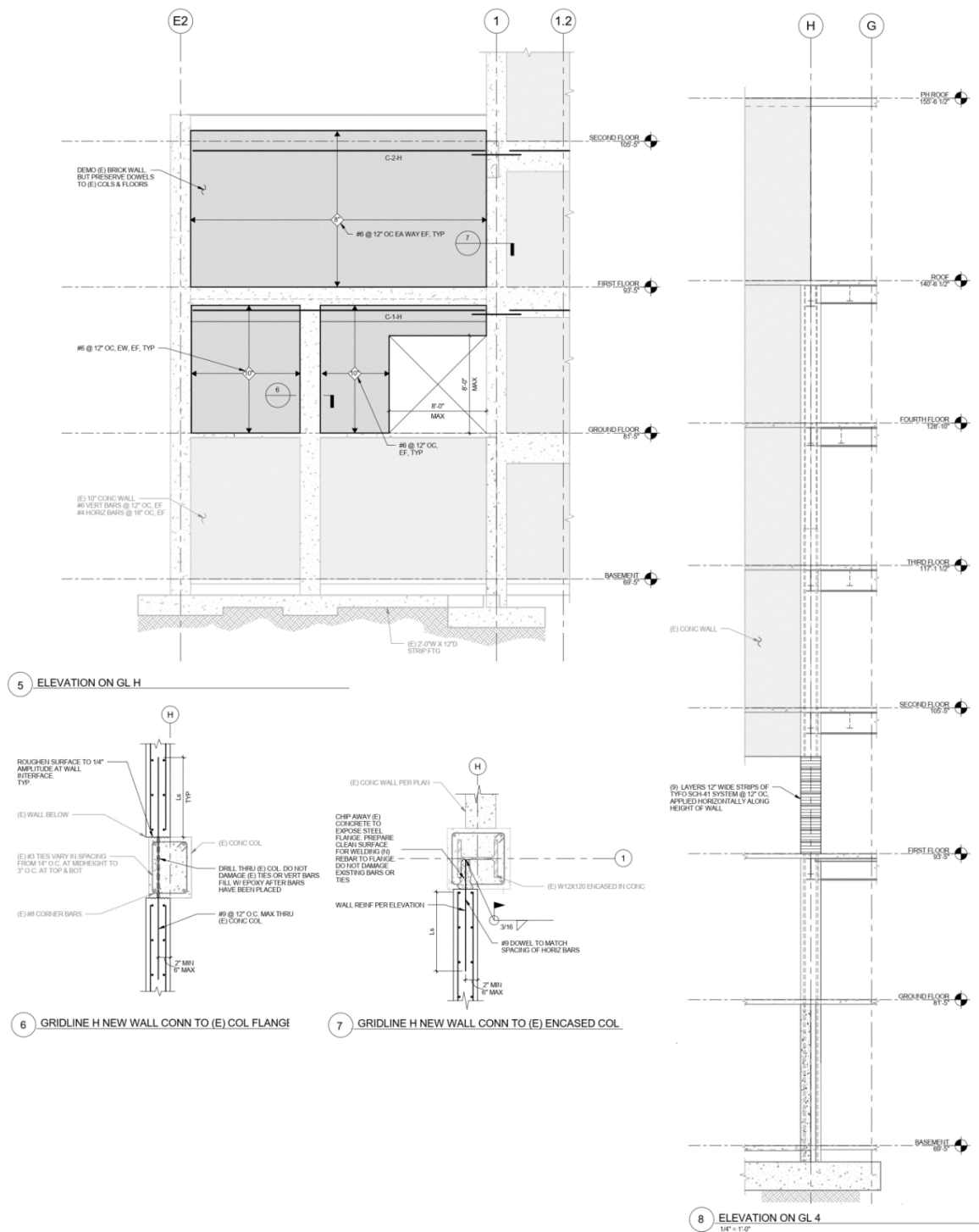


Figure 2.30. Details of connections of new wall on grid line H and wall on grid line 4.

On gridline 4 there is an existing short concrete wall that starts in the second floor and ends in the roof. The column below will be encased in FRP with 12'' [0.30 m] wide strips to provide a better transition of the forces attracted by this stiff element. See Figure 2.30, detail 8.

On grid line U, shown in Figure 2.31, there is also an existing concrete wall along the entire building height that will be strengthened with FRP. On the same grid there is also a discontinuous wall that starts on second floor up to the roof. A similar discontinuous wall that starts at third floor up to the roof exists on gridline F. For these cases, the steel columns located at lower levels are strengthened with steel plates welded to the flanges as it can be observed in detail 11 of Figure 2.32.



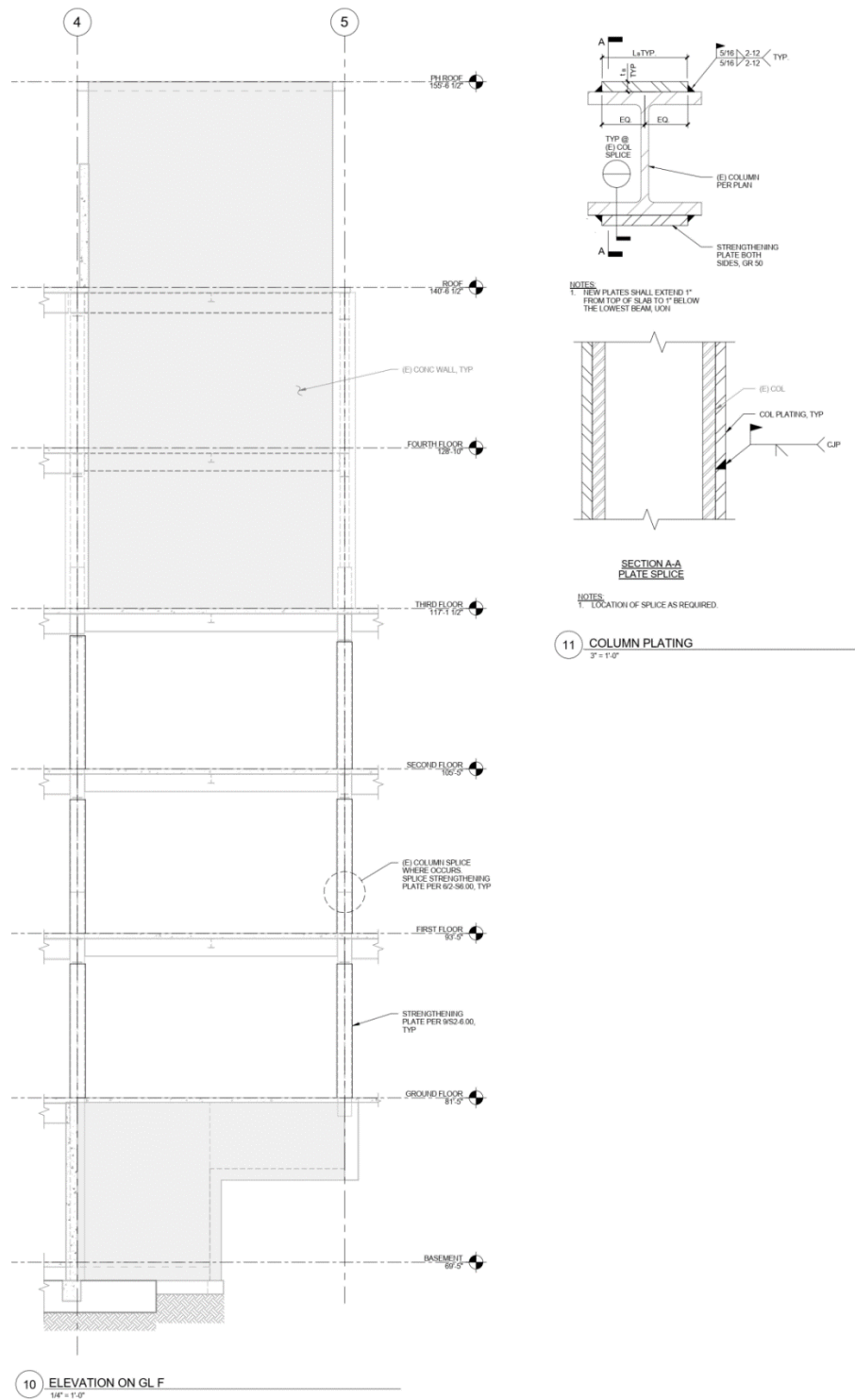


Figure 2.32. Details of connections of wall on grid line F and steel column strengthening.

The floor plan presented in Figure 2.25 shows the localization of the collectors at this level. Figure 2.33 shows different details of the configuration and connections of such elements. The collectors in gridlines 3.1, L.1 and I, are made from an HSS section welded to a plate that it is connected with expansion anchors to the lower face of the slab. Detail 14 in Figure 2.33 shows how continuity is achieved in the intersection with steel beams. In these cases, both sections of the collector are connected by tensile rods passing through holes drilled in the web of the existing steel beam. The rods are bolted to “buckets” made from HSS sections welded at each side of the collector as shown in detail 12 of Figure 2.33. The “buckets” are specified in an alternate configuration to prevent saturation. Detail 15 in Figure 2.33 shows the connection between the collector and the shear wall in grid line 3.1. Here, the action line of the collector intersects with a concrete wall and a steel beam encased by concrete. Similar to the previous detail, holes are drilled in the concrete and in the web of the beam with a maximum diameter of 2’’ [51 mm]. The holes are filled with non-shrink grout after the rods are in place. The rods are connected to buckets welded to a 1’’ [25 mm] thick plates anchored to the RC shear wall with  $\phi=1/4$ ’’ [63 mm] through bolts spaced 24’’ [60 cm]. Detail 13 in Figure 2.33 shows the collector on grid line H, consisting of a plate attached to both sides of the existing steel beam encased by concrete. Note that the steel plate is also welded to plates connected to the lower face of the slab. The connection of this collector to the new shear wall is shown in detail 16. Finally, detail 17 in Figure 2.33 shows the connection of the collector on grid line L.1 to the concrete wall adjacent to the stairs.

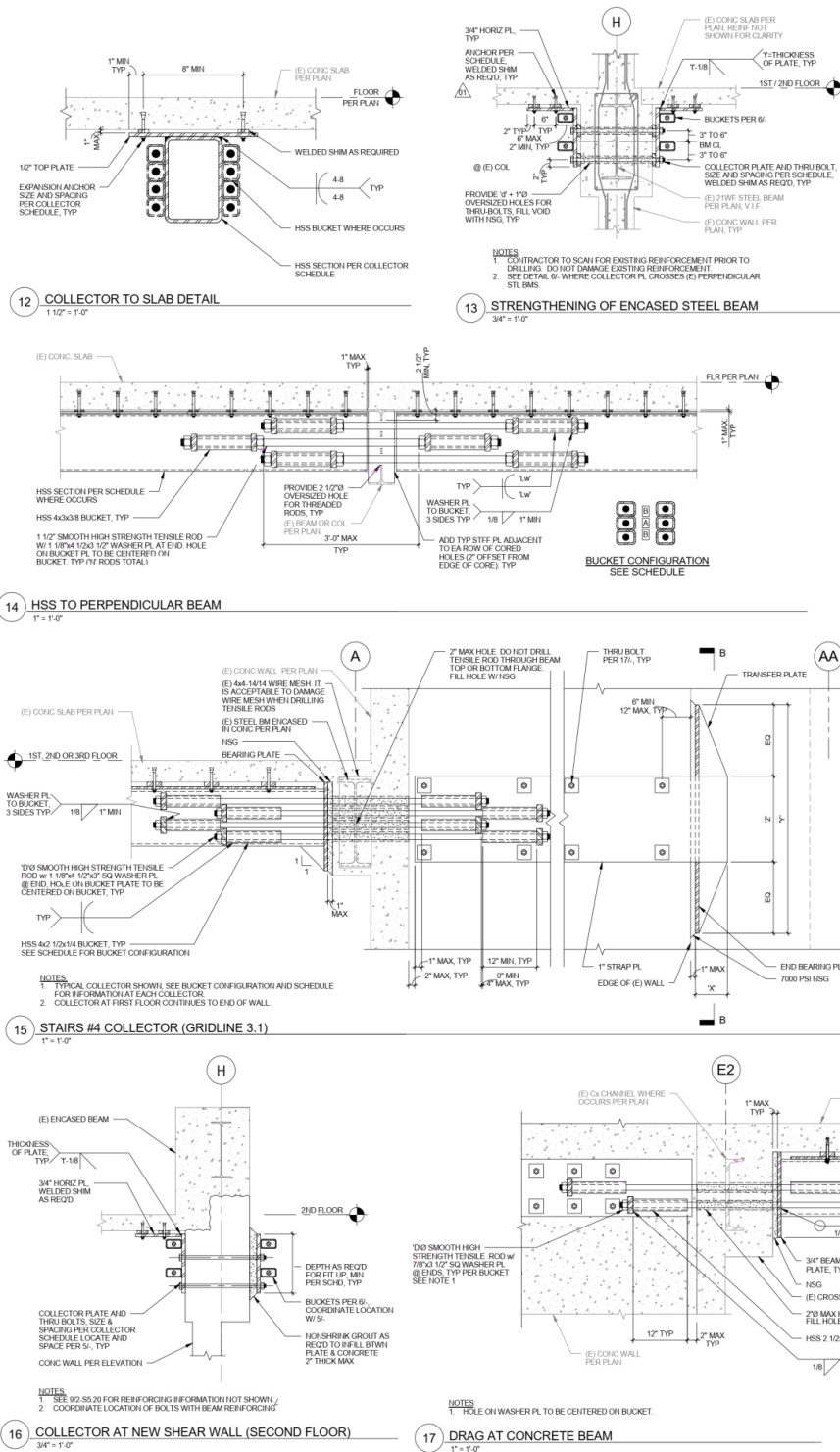


Figure 2.33. Details for collectors

The construction of the wall on grid line X will attract a large portion of the lateral force; hence, a new foundation is required to transmit such forces to the ground. As a consequence, a new footing with a L shape in floor plan is constructed, specified to match the existing foundation with a minimum depth of 3'-6" [1.07 m] below the top of the slab in that level. The new footing will be connected laterally to the different existing footings with dowels anchored with epoxy. The reinforcing of the wall footing consists of combination of #11 bars at different spacing from 12" to 6" [0.30 to 0.15 m]. See Figure 2.34. Figure 2.35 shows the existing wall on grid line X at the basement with preparations for the construction of the new foundation. Note the reinforcement of the existing wall marked to prevent interferences while drilling for installation of dowels.

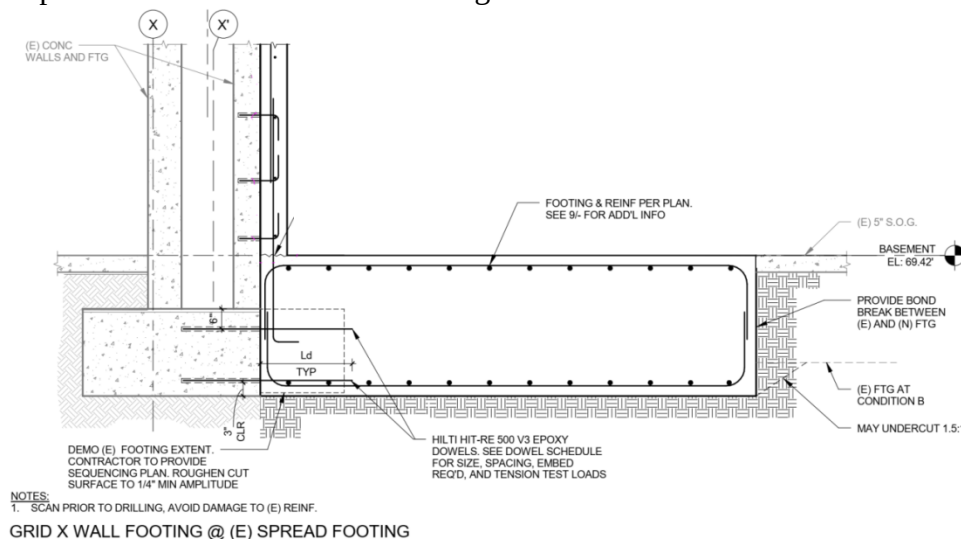


Figure 2.34. Details of connections of new with existing foundation.



Figure 2.35. Portion of the wall that is to be increased with an additional 8 in. RC layer.

A special detail is included where an existing footing will be totally embedded in the new footing, for which the new footing bars will be passed through holes previously drilled in the existing footing as shown in Figure 2.36. The construction of this footing is shown in Figure 2.37. Observe the steel passing through the existing footing. In order to prevent the removal of the entire support for the existing foundation, the construction of the new foundation is specified in sequential stages.

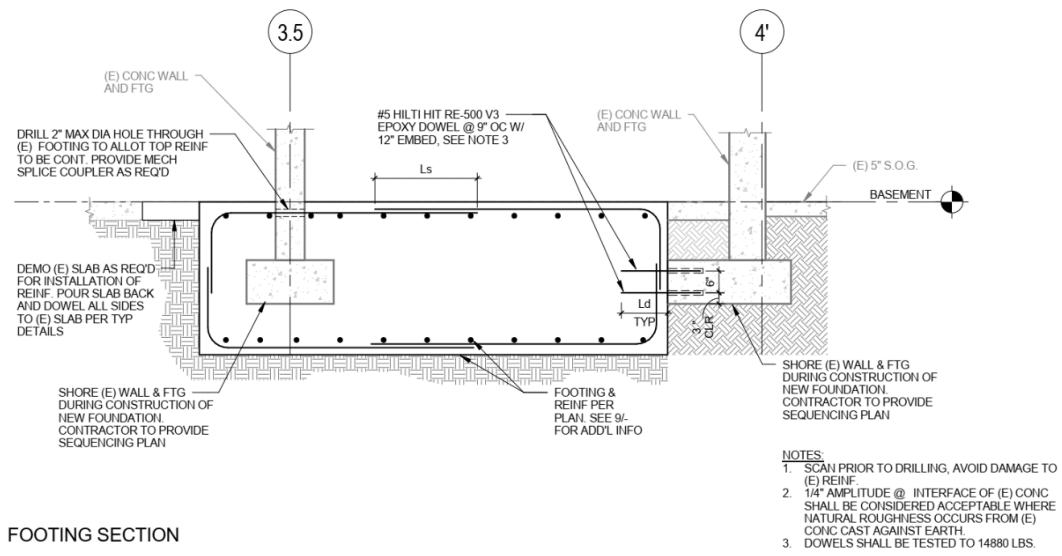


Figure 2.36. Detail of an existing concrete footing totally embedded in the new footing.



Figure 2.37. Construction of the new footing with the existing footing embedded

### 3.4 BUILDING D. HOTEL

#### 3.4.1 Building description

Building D is located in Los Angeles County. It was designed and constructed in 1966 and before the retrofit it was used as a hotel that allocated 750 hotel rooms in total with a total area of 34000 square foot per floor [315.9 m<sup>2</sup>]. The structure is completely isolated from other structures and consists of four basements, a mezzanine, sixteen levels and roof as shown in Figure 2.39. The total height of the structure is 160' [48.8 m] above the street level. In floor plan it is curved, with a total length of 465' [142 m] at the center line, and a constant width of 62' [18.90 m]. (Figure 2.38)

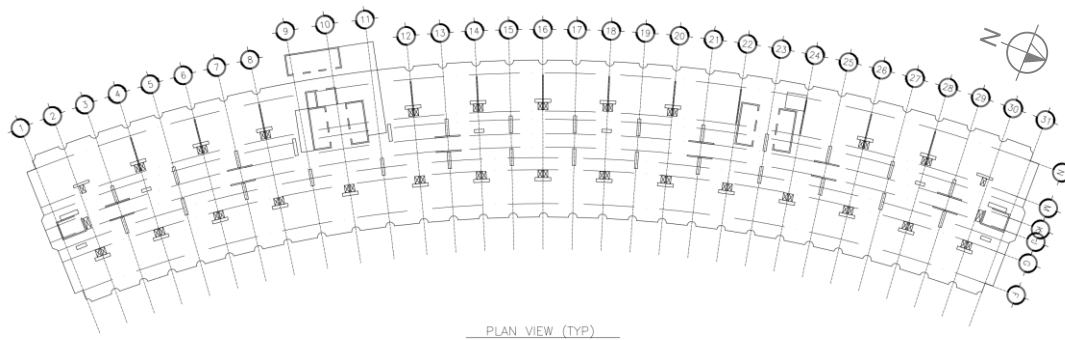


Figure 2.38. Typical floor plan of building C.

The original structure consists of concrete core shear walls from the basement level B4 to the level B2; steel moment frames in each bay from B1 to the third floor; and steel frames with strap bracing in alternate bays from the fourth floor to the roof level. The floor system is a two-way slab. The type of foundation used was square footings. No major modifications to the structure were made prior to the retrofit.

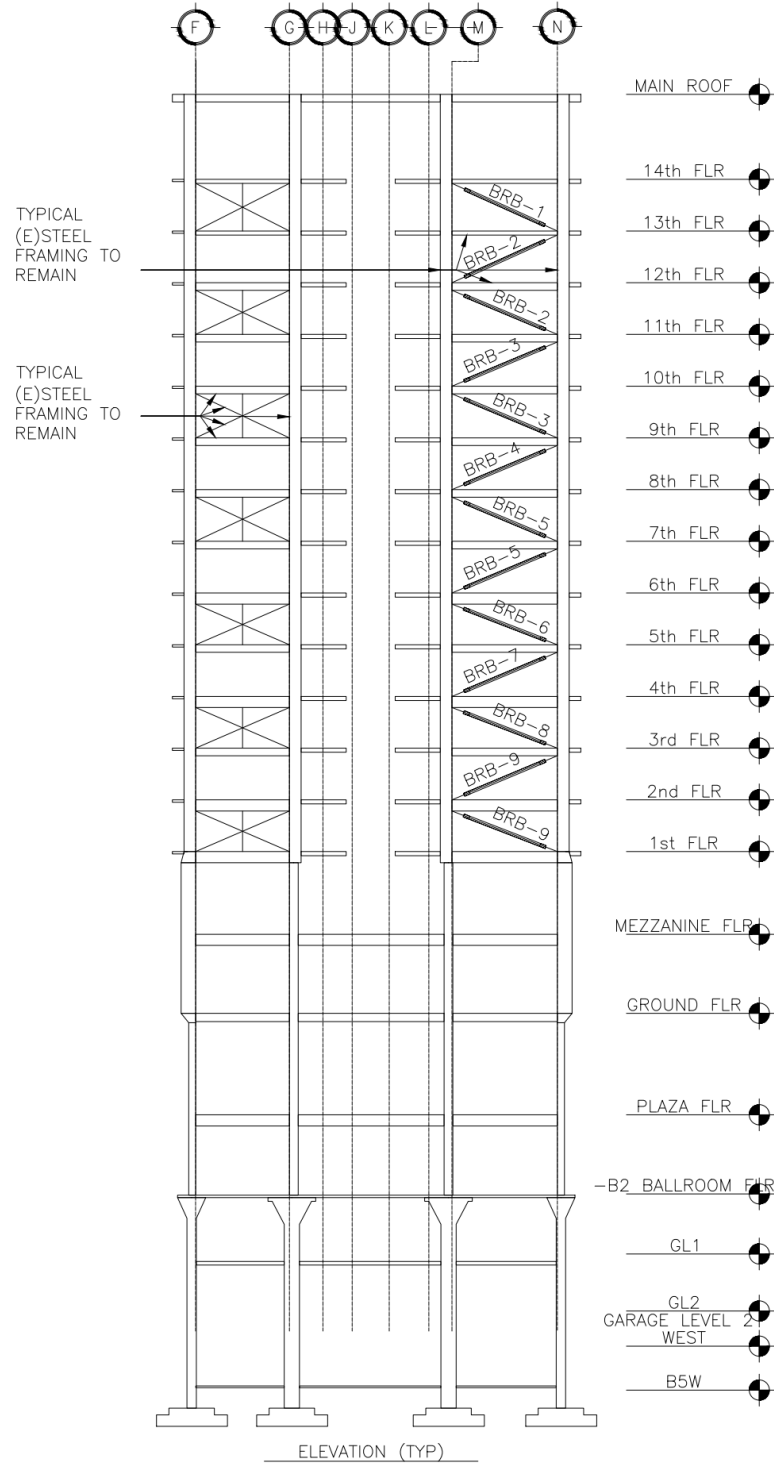


Figure 2.39. Typical elevation of building C

Up to level 4, the moment frames are made of wide flange steel sections. For upper levels, the sections were made of three A-36 steel plates welded into an “I” shape with a vast variation in shapes and dimensions. The typical moment connection was made with welded cover plates and a shear plate connection with three bolts as shown in Figure 2.40. Two types of bracing were observed in the building: a) bracing made of strips of 2’’x1/4’’ [51x6.3 mm] steel plates and b) bracing made of square HSS of 2’’ [51 mm] in width. The connections of such braces were made with large gusset plates to which the strips or HSS were welded as shown in Figure 2.41.

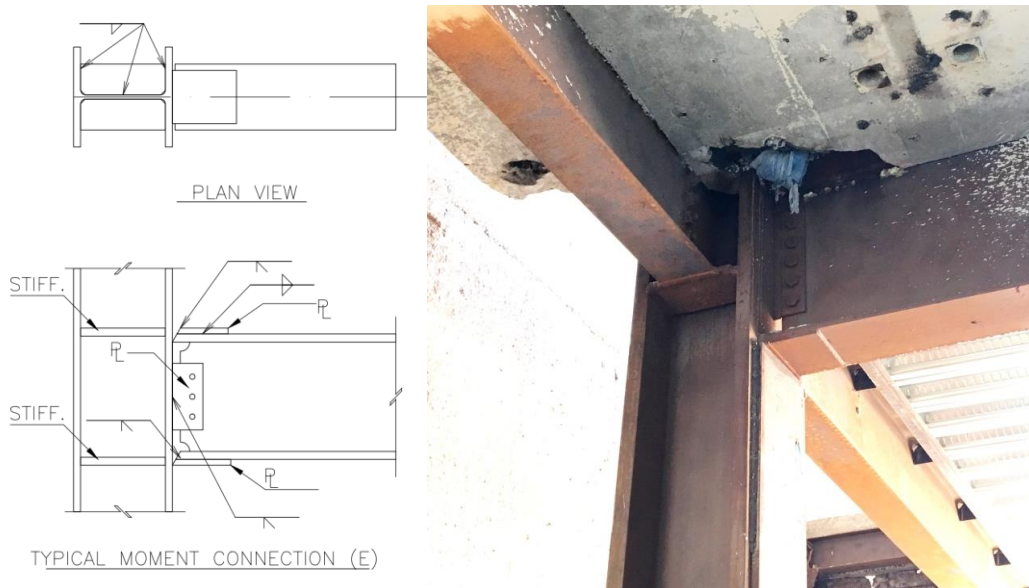


Figure 2.40. Details of typical existing moment connections.



Figure 2.41. Details of connections of bracings of strips and HSS

The upgrade and renovation of this building is part of a new development of the hotel complex which will also include two new high-rise buildings. The existing building is planned to allocate 500 hotel rooms and 63 new condominiums in the upper levels with double height spaces. New amenities are to be constructed in the rooftop, including a pool. Also included are the construction of drive ramps, and the infill of a sunken plaza in front of the hotel.

### 3.4.2 Retrofit goal

This building was a voluntary seismic upgrade. Different retrofit schemes were considered such as the use of steel plated shear walls and dampers, but the final selected retrofit scheme was the use buckling restrained braced (BRB) frames as the main lateral force resisting system.

### 3.4.3 Design data and analysis

The retrofit design was made in compliance with LABC Chapter 34 and ASCE 7-10. The structural performance of the retrofitted building was evaluated using nonlinear static analysis procedures. The new lateral force resisting system consisting of BRB were

placed into the existing beam-column frames. The existing beams and columns were strengthened to ensure the new BRB's were the yielding mechanisms and the existing elements remain elastic. The existing connections were tested and verified to meet the requirements for the new construction.

#### **3.4.4 Strengthening plan**

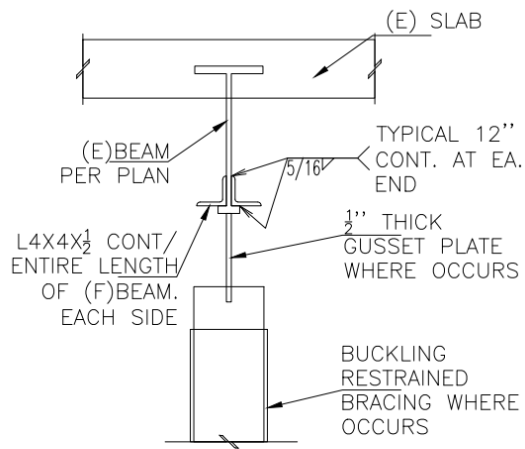
The foundation was strengthened by increasing the size of the reinforced concrete footings as shown in Figure 2.42.



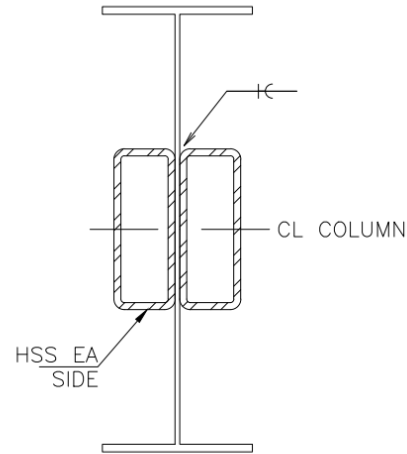
Figure 2.42. Increasing in the size of the foundation of one column.

For the gravity system, added mass or altered slabs forced the localized strengthening of certain elements. Most of the localized strengthening was made by welding new elements to the steel columns, as shown in the detail of Figure 2.43(right) and Figure 2.44 (left). Note that in Figure 2.45, instead of the HSS an angle was welded to the

web of the column. For the beams, especially when new BRB were placed, some plates or angles were welded to the existing element to achieve the required strength (Figure 2.43 left and Figure 2.44).



TYPICAL BEAM STRENGTHENING



TYPICAL COLUMN STRENGTHENING

Figure 2.43. Details of strengthening in beams where buckling restrained braced are placed (left) and strengthening of the webs in columns (right).



Figure 2.44. Details of strengthening in columns with welded HSS elements; beams with additional plates and the slab with FRP.



Figure 2.45. Details of strengthening in columns with welded HSS elements; beams with additional plates and the slab with FRP.

Fiber Reinforced Polymer (FRP) was used to strengthen slabs with insufficient tension steel, as shown in Figure 2.44 and Figure 2.45. Similarly, some reinforced concrete columns in the first floors were wrapped with FRP (see Figure 2.46). Figure 2.47 shows one squared column with a pyramidal capital which was similarly strengthened. In this case, the corners were rounded prior to the application of the FRP wrap.

FRP strips were also installed to strengthen the reinforced concrete diaphragm at the roof level to supplement the strength loss due to the new openings made for the pool.

The strips were oriented in the longitudinal direction of the building to improve the diaphragm behavior of the slab at this level as shown in Figure 2.48. Note that FRP is also used to strengthen the slab around the pool area with strips oriented in the transversal direction. Figure 2.49 shows the application of this FRP on the roof level.

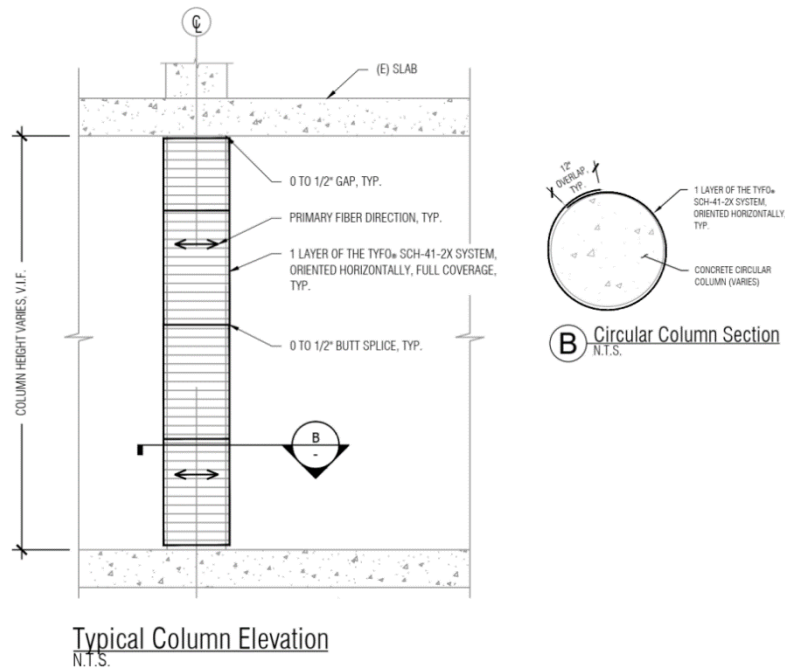


Figure 2.46. FRP strengthening of circular column in the basement

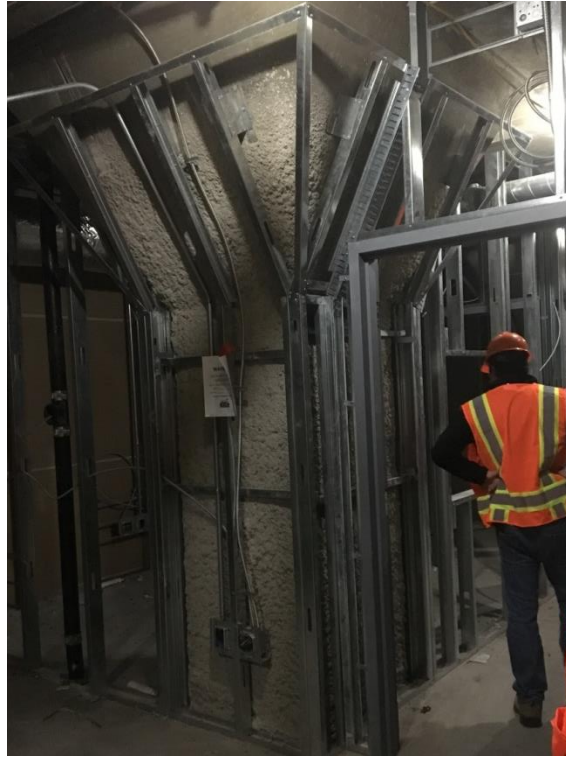


Figure 2.47. Column with capital strengthened with FRP, already covered.

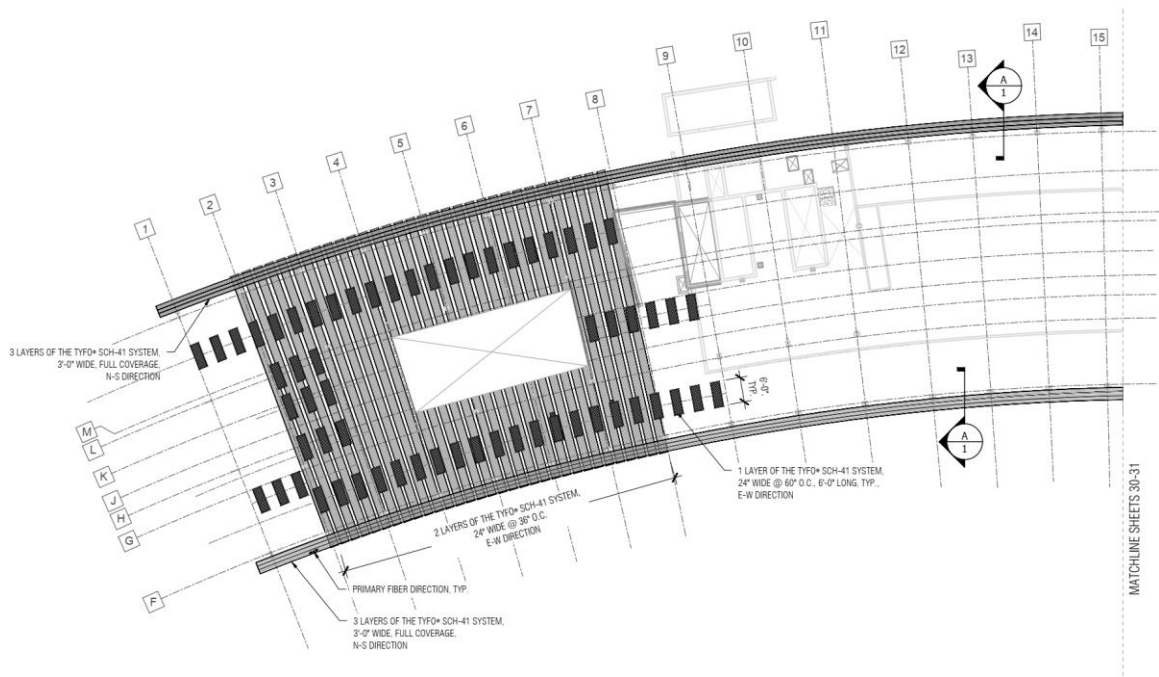


Figure 2.48. Plan with FRP strips to strengthen the diaphragm and the area around the pool at the roof level.

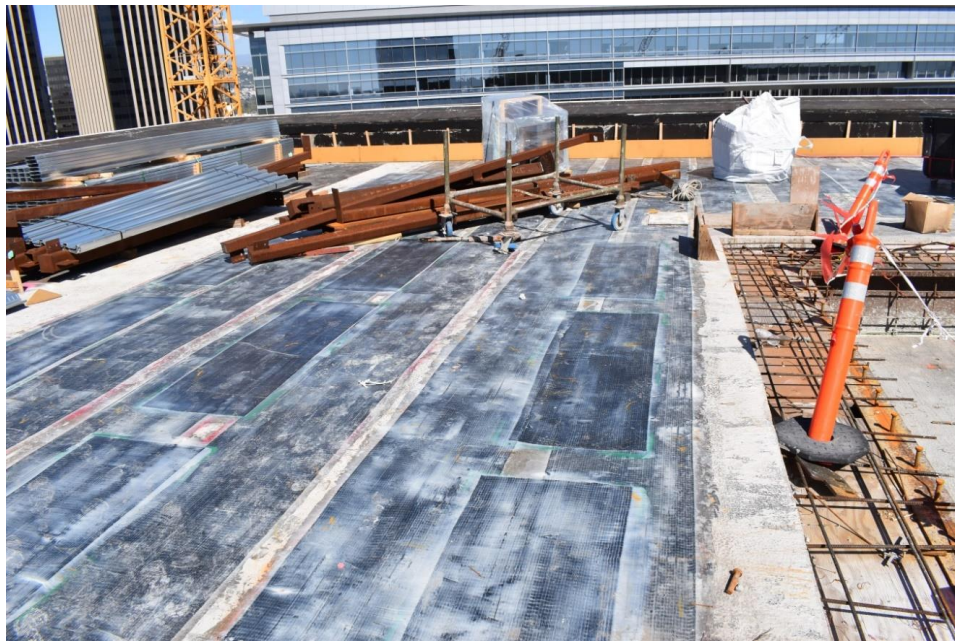


Figure 2.49. Application of FRP strips to strengthen the area around the pool at the roof level

Finally, a new pool was included among the new amenities of the building remodeling. As a consequence, a section of the roof was cut, and a supporting structure formed by steel beams was welded to the existing structure. A steel deck was used to support the pool, as shown in the left picture of Figure 2.50.



Figure 2.50. Structure for the pool on the roof. Bottom view (left) and top view (right).

### 3.5 BUILDING E. HOTEL

#### 3.5.1 Building description

Building E is located in Los Angeles. It was constructed in 1924 and it was originally used for commercial and office spaces. The building has thirteen stories with a total height of 150' [45.72 m], and a rectangular floor plan (see Figure 2.51 and Figure 2.53). The building has eight vertical bays in the east-west direction and three vertical bays in the north-south direction. The north and west facades are clad with red brick and terra cotta elements. In 1935, a five-foot section was removed from the center of the building in the east-west (long) direction to accommodate a sidewalk widening project. The building was reconnected by moving the west half to the east. In 2016, the building was rehabilitated and converted into a hotel.

The original structure comprised RC frames in both directions of the building. The 2016 retrofit consisted mainly of the addition of RC shear walls in the two orthogonal directions of the building, as shown in Figure 2.54. Several columns were also strengthened with RC jackets. A new roof deck and penthouse were also constructed using steel frames to house a new rooftop bar and pool.

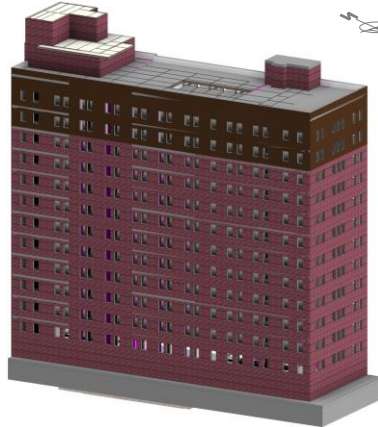


Figure 2.51. 3D model of building E.

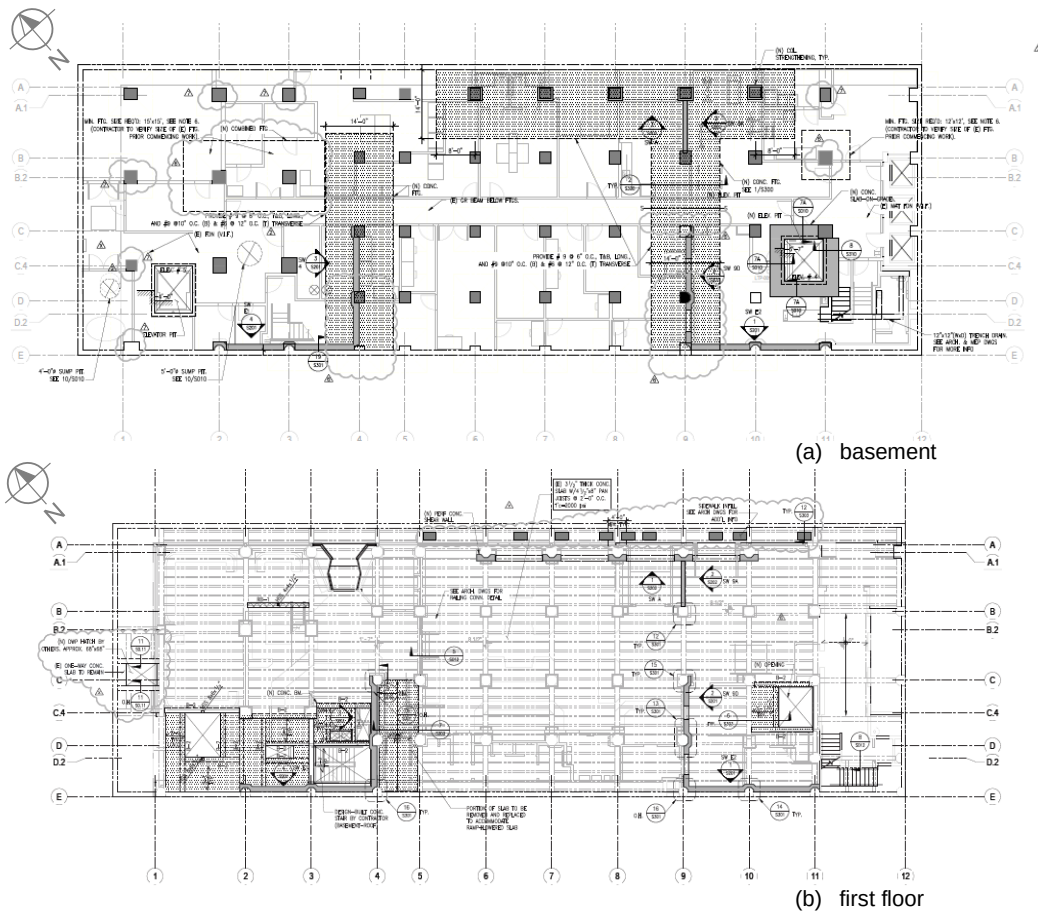


Figure 2.52. Basement and first floor plan of Building E

### 3.5.2 Retrofit goal

The retrofit project was triggered by a change in occupancy from office building to a hotel building. The building retrofit was designed based on the 2013 Los Angeles Building Code (LABC) and 2013 California Historical Building Code.

### 3.5.3 Design data and analysis

The building retrofit was designed as a seismic upgrade and renovation in accordance with the 2013 LABC and 2013 California Historical Building Code. Existing

elements were evaluated following ASCE 41-06. All new construction has been designed to meet the seismic provisions of the current code.

The analysis and design considered an Occupancy Category II, Site Class D, and Seismic Design Category D. The seismic force resisting system of the main building structure consists of special RC shear walls.

#### **3.5.4 Retrofit plan**

The retrofit plan mainly comprised the addition of RC shear walls to provide lateral load resistance (see Figure 2.53). In the east-west direction, a perforated shear wall (Wall A) was built next to the north face of the building and two shear walls were built on the south face of the building (Walls E1 and E2), as shown in Figure 2.52b and Figure 2.54. Three shear walls were constructed in the north-south direction (Walls 4, 9A and 9D), as shown in Figure 2.52b and Figure 2.55.

The piers and spandrel beams of Wall A have a thickness of 16'' [40 cm], except the upper two stories which are 8'' [20 cm] thick. Instead of extending the pier into the basement level, the existing columns at this level were jacketed with reinforced concrete, as shown in Figure 2.52b and Figure 2.56. Walls E1 and E2 have a thickness of 12'' [30 cm]. Walls 4 and 9A have a thickness of 10'' [25 cm]. Wall 9D has a thickness of 12'' [30 cm]. The new walls were aligned with the existing frames and were casted against existing columns (see Figure 2.57 and Figure 2.58). The specified concrete compressive strength of the walls was 6,000 psi [41.4 MPa].

In the upper level, a new roof deck and penthouse were constructed with steel frames (Figure 2.59). Existing brick parapets were braced with steel elements as shown in Figure 2.60.

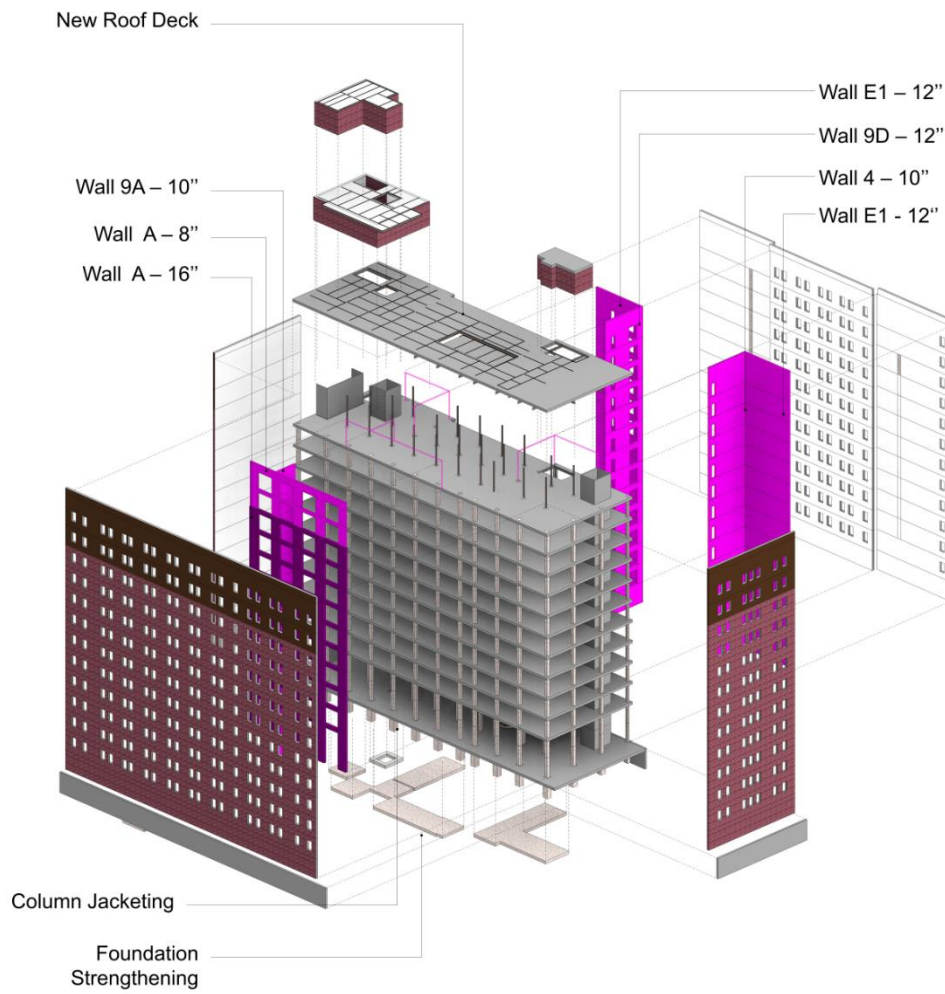


Figure 2.53. 3D model of strengthening plan for building E.

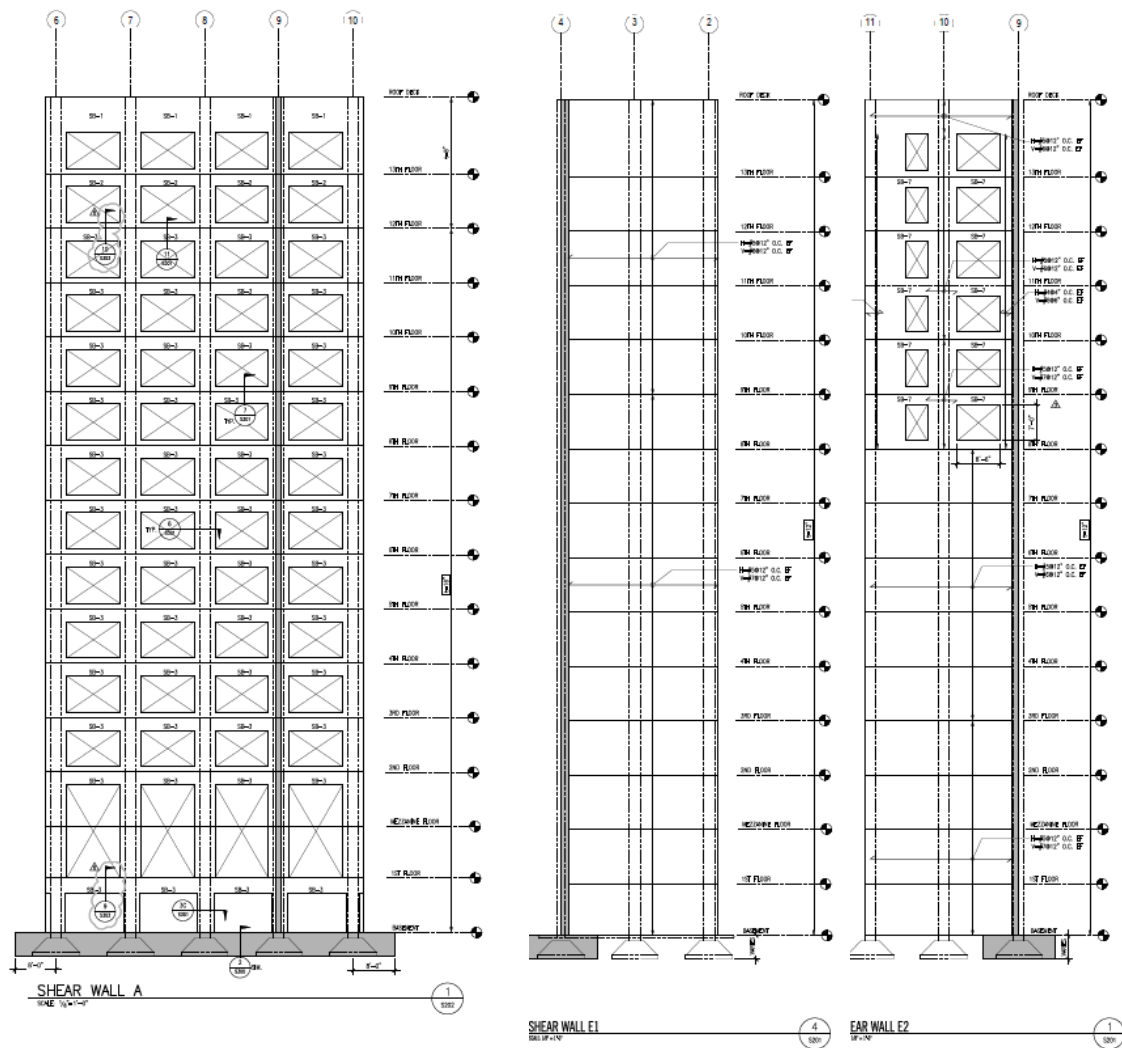


Figure 2.54. Shear Walls A, E1 and E2 (Elevation)

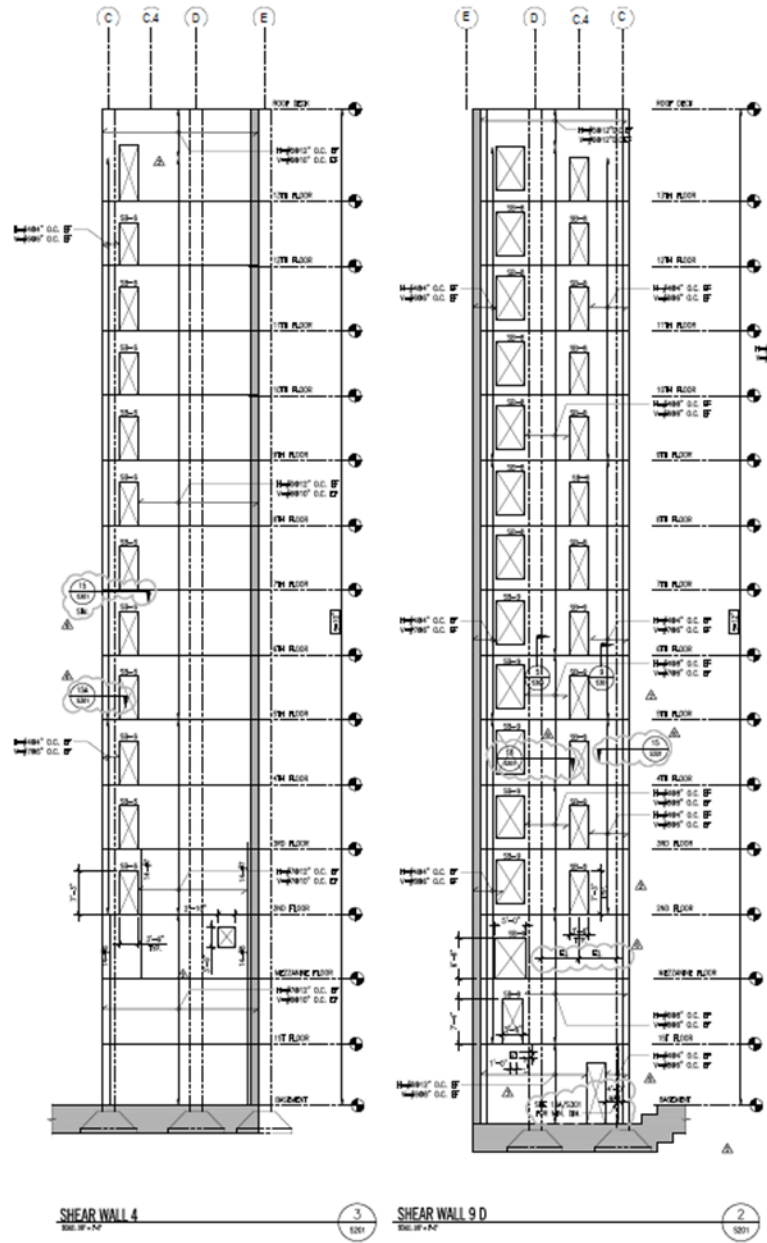


Figure 2.55. Shear Walls 4 and 9D (Elevation)



Figure 2.56. Jacketing of columns



Figure 2.57. View of Shear Wall E1



Figure 2.58. Construction of shear walls

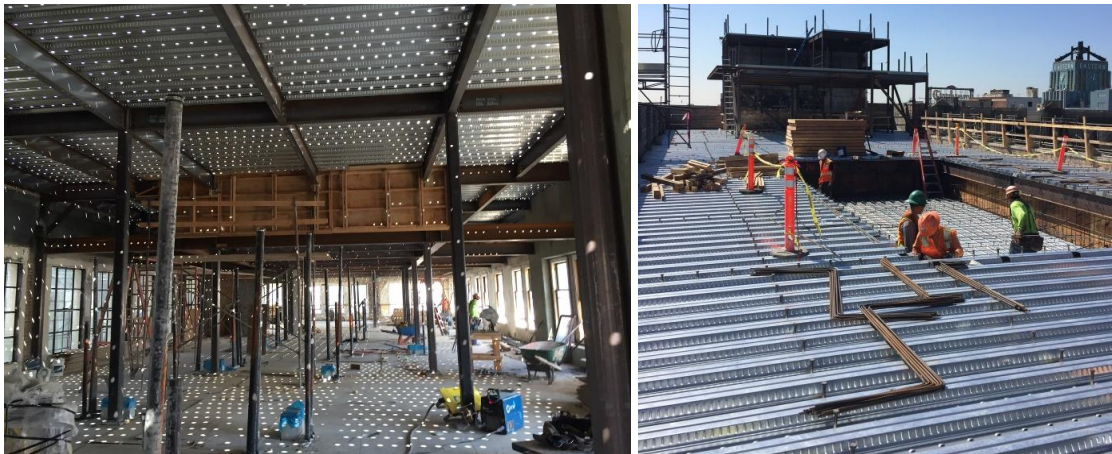


Figure 2.59. Construction of roof deck and penthouse



Figure 2.60. Parapet bracing

## **4. Conclusions**

Our understanding of earthquakes and their effect on structures is constantly evolving. After an important earthquake event, lessons learned from the observation of the behavior of structures have turned into legislation with the aim to obtain safer structures. This applies to both the design of new structures and retrofit of existing structures.

In California, Senate Bill 1953 (SB 1953) enacted in 1994 after the Northridge earthquake represented a significant step towards increasing the safety of existing constructions. The importance of SB 1953 is that it does not only address new building design but mandates on the analysis and the retrofit of existing building infrastructure to obtain minimum levels of safety, with a progressive long-term plan. This plan included an initial categorization of the hospital buildings according to the seismic structural and nonstructural performance, and deadlines for compliance to a minimum safety level (SPC-2) established originally by 2008; and a target safety level (SPC-3, SPC-4, SPC-4D and SPC-5) to be completed by 2030. There are, certainly, challenges related with design constraints and funding for such retrofits. Significant progress has been made after 25 years; still, efforts must be done in order to meet the objectives of the 36-year plan of such regulations.

The success in changes of legislation related with seismic design has been proved in previous examples. In the city of Los Angeles, for instance, a significant reduction in failures was observed in 1994 in Northridge, for hospitals in compliance with the 1973 Hospital Safety Act, in contrast with the failures observed in structures built under previous regulations. Hence, it is important to legislate not only in new buildings but also on the existing infrastructure.

In recent years, attention has also been placed in nonstructural elements, whose damage can be high in buildings with costly specialized equipment such as in hospitals, besides the cost associated with disruption of activities in the facility. The nonstructural classification in SB 1953 is intended to reduce this type of damage. To achieve such goals, interventions mainly consist of the bracing and anchoring of pieces equipment and conduits. This type of actions has been observed in in the hospital building retrofits presented in this report. For non-hospitals buildings, bracing and anchoring of nonstructural elements posing significant risk, such as parapets, is also required and commonly observed.

The change in legislation typically results in larger design forces, which translates into larger demands on the lateral force resisting system. These forces must be transmitted to the foundation. As a consequence, work on increasing the capacity of the foundation is required in many cases. Enlargement of footings or introduction of new piles are among the types of solutions used in the case studies presented in this report.

Reinforced concrete shear walls are widely used as lateral force resisting systems in concrete structures. The increase of design forces may also require additional strength in these systems. Infills on openings to provide continuity and widening existing concrete elements are among the solutions observed in the case studies to increase wall capacity. In some cases, shear strengthening of walls has also been accomplished with composite materials (FRP). FRP have also been used to wrap existing reinforced concrete columns.

The characteristics of the works for retrofits change for each particular case. Hospitals are complex structures that have evolved through time. This complexity may result in combination of conforming and non-conforming structures, with plenty of irregularities. Such irregularities can be mitigated by introducing stiff elements, for instance concrete walls, to reduce torsional eccentricity. One of the examples included the

construction of new walls or widening of existing ones with RC overlays. New reinforced concrete elements and overlays are commonly connected to the existing ones using dowels.

The inertial forces generated in an earthquake are concentrated in the slabs at each floor level, and they must be properly transmitted to the lateral force resisting system. Hence, diaphragms are also essential elements of the seismic resisting systems. A number of case studies also included retrofit interventions of the diaphragms. One of the cases added collectors to transmit the forces from the slab to the lateral force resisting system. The new collectors consisted of a series of steel rods and widening of the concrete slab. In another case, FRP bands were placed in the longitudinal direction of the diaphragm. The function of these FRP bands is analogous to that of tension steel in a concrete beam.

## Acknowledgments

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### **Chapter 3. Lateral Deformation Capacity and Damage Fragility Analysis of Circular RC Columns Retrofitted with FRP Jackets**

Authors: Oscar Minor Garcia and Juan Murcia Delso.

Contribution: Conducted a literature review, collected test database, performed analytical and statistical analysis, developed fragility curves, and wrote the article.

#### **Abstract**

This paper presents a study on the lateral deformation capacity and damageability of circular RC columns retrofitted with FRP jackets. An experimental database composed by 56 tests is compiled from the literature, and it is used to assess the accuracy of two different methods proposed in the literature to estimate the lateral deformation capacity of FRP-jacketed columns. A new equation for the ultimate curvature is derived based on moment-curvature analyses of the columns in the database and a unified expression for the plastic hinge length is also proposed. These provide good estimations of the lateral displacement capacity of the columns in the database. Lastly, two sets of seismic fragility functions for three different levels of damage are developed based on the analysis of experimental data. One set uses story-drift ratio as demand parameter, and the other uses a demand parameter normalized to the estimated ultimate deformation capacity.

Keywords: deformation capacity, fragility analysis, concrete columns, fiber reinforced polymer, reinforced concrete, seismic retrofit.

## 1. Introduction

Fiber reinforced polymer (FRP) composites are commonly used to strengthen reinforced concrete (RC) columns in bridge and building structures. FRP jackets provide additional confinement to column sections, resulting in improvements in the axial load capacity, flexural and shear strengths, flexural ductility, and/or performance of lap splices (Balaguru et al. [1]). This type of technique is particularly attractive to enhance the seismic performance of columns because it limits or precludes non-ductile damage mechanisms (shear and lap-splice failures) and favors the formation of flexural plastic hinges with increased ductility capacity.

A significant number of experimental studies have been conducted in the last three decades to characterize the seismic performance of FRP jacketed columns (e.g., [2-16]). Based on test data, analytical methods have been developed to estimate their lateral deformation capacity. Priestley et al. [17] proposed using a simple plastic hinge model to calculate the ultimate lateral displacement capacity of FRP-jacketed columns with circular and rectangular sections. As compared to conventional columns, the proposed model uses a modified effective plastic hinge length which assumes that the jacket limits the spread of plasticity. The ultimate curvature corresponding to the rupture of the jacket is obtained based on an empirically-derived equation for the ultimate compressive strain for concrete, which depends on the amount of passive confinement provided by FRP. The design guidelines for FRP strengthening of concrete structures developed by ACI 440 Committee [18] recommends using the same effective plastic hinge length equation but a different ultimate strain value for concrete, which is based on the stress-strain relations proposed by Lam and Teng [19]. More recently, a number of research studies have proposed alternative equations to calculate the effective plastic hinge length and/or ultimate curvature based on

experimental evidence (Gu et al. [20], Yousff et al. [21]) or results from numerical simulation (Binici [22] Yousff et al. [23]).

New performance-based seismic design and assessment methods require analytical tools to predict the response of structures at different levels of demand, and not only their capacity at ultimate conditions. Current seismic evaluation standards and guidelines following performance-based engineering principles, such as the ASCE/SEI 41-17 [24] and FEMA P-58 [25], provide guidance on how to predict the inelastic response and damageability of structures. The ASCE/SEI 41-17 standard for buildings uses backbone force-deformation curves to model structural response and a set of acceptance criteria linked to different performance goals. Modeling parameters and acceptance in this standard are provided for unretrofitted structural components, such as RC columns, and no specific information is provided for retrofitted elements. Backbone curve parameters and acceptance thresholds following the ASCE/SEI 41-17 format were recently proposed by Alvarez et al. [26] for FRP- and steel-jacketed RC columns. The proposed values were empirically adjusted based on a database comprising 116 tests on circular and rectangular sections. The FEMA P-58 probabilistic seismic evaluation framework for buildings presents a similar limitation for retrofitted structures. This framework uses fragility functions to relate structural response, measured using demand parameters such as story-drift ratio or floor acceleration, with the probable damage experienced by structural and non-structural components. While default fragility parameters are provided for conventional structural components, no specific guidelines are given for retrofitted elements such as jacketed columns. To date, few studies have been conducted on the damage fragility of FRP-jacketed columns. Billah and Alam [27] and Stefanidou and Kappos [28] conducted fragility analyses of RC bridges with FRP-jacketed columns using numerical modeling. In each of these studies, fragility curves were derived for a single

bridge case study and the probability of exceeding different damage states were defined as a function of the peak ground acceleration (PGA). Similarly, Tabandeh and Gardoni [29] developed fragility functions based on probabilistic formulation and numerical simulation to assess the effectiveness of FRP jacketed columns. Damage states were defined as bond failure and flexural failure, and these were related to story-drift demand in the fragility curves. However, there is currently a lack of experimentally-derived fragility data to directly relate the response of FRP-jacketed columns to their level of damage.

This paper presents a study on the lateral deformation capacity and damageability of circular RC columns retrofitted with FRP jackets. An experimental database compiled from tests reported in the literature is presented. This database is used to assess the accuracy of two approaches proposed in the literature to calculate the ultimate lateral displacement capacity of FRP-jacketed circular columns. A new equation for the ultimate curvature is also derived based on sectional analyses of the columns included in the database. Seismic fragility functions for three different levels of damage are developed based on the analysis of experimental data. Two different sets of fragilities are proposed: one uses drift ratio as demand parameter, and the other uses a demand parameter normalized to the estimated ultimate deformation capacity.

## **2. Database of cyclic tests on circular RC columns with FRP jackets**

Experimental results have been compiled from the literature to develop a database of 56 FRP-jacketed circular RC columns tested under cyclic lateral loading. Specifically, the studies carried out by Xiao et al. [2] Sheik and Yau [3], Li and Sung [4] Haroun and Elsanadedy [5], Ozbakkaloglu and Saatcioglu [6] Shan et al. [7] Ghosh and Sheik [8], Gu et al. [9] Desprez et al. [10] Idris and Ozbakkaloglu [11], Paultre et al. [12] Seible et al.

[13], Yousff et al. [14] Xiao and Ma [15] and Haroun and Elsanadedy [16] are considered. In these studies, column specimens were subjected to a constant axial load and cyclic lateral loading of increasing displacement amplitude. Most of the columns were tested as cantilevers (single curvature); however, there are 13 columns that were tested under fixed-fixed conditions (double curvature). In general, the column specimens in these studies presented design and detailing characteristics typical of pre-1971 non-ductile practices, including low transverse reinforcement or insufficient lap-splice. FRP jackets were installed at the column end(s) or along the entire height of the column. The FRP jackets were made of carbon fibers (CFRP) in most of the cases, but glass (GFRP) and aramid (AFRP) fibers were also used in some specimens. One case used ultra-high molecular weight polyethylene (Dyneema, DFRP). Most of the specimens had internal confining steel reinforcement, but few specimens confined by FRP tubes only have been included in the database.

Other studies on FRP columns are not considered due to lack of information regarding the type of failure or due to uncertainty on the damage state (Saadatmaneh [30] Wang and Wang [31] Breña and Schlick [32])

The experimental studies considered to develop the database are briefly reviewed here:

- Xiao et al. [2] tested one as built and two columns strengthened with GFRP along the entire height. One of the retrofitted specimens included 3-layer continuous fibers while the other was strengthened with four individual single-layer shells with a longitudinal slit for each layer. Specimens were tested in double curvature. Only the continuous wrap showed FRP rupture for the outer layer, though fine horizontal cracks were observed in both specimens. The results show little difference between continuous or individual shells and agreement in the more ductile behavior as compared to the as-built specimens.

- Sheik and Yau [3] tested twelve cantilever columns: six as built and six retrofitted columns with GFRP or CFRP. In the specimens retrofitted with GFRP, the jacket extended approximately  $0.5H$  from the base, where  $H$  is the effective column height; however, crushing in concrete outside the wrap was observed during loading in one case. The load was reduced, and the column was repaired applying wrap in the entire column and retested until failure. The other two specimens were fully wrapped. One of the specimens strengthened with CFRP had four 100-mm-wide composite bands with a clear spacing of 100 mm instead of a continuous wrap. This specimen is not included in the database because it may present a different spread of plasticity. Change in color was observed in the jackets during early stages of inelastic deformation, indicating a separation of FRP fabric from concrete. Failure occurred by FRP rupture along with buckling of longitudinal reinforcing bars. The results show ductility deterioration as the axial load level increases.

- Li and Sung [4] tested three cantilever columns including one as built and two columns strengthened with CFRP along the entire column height. In both retrofitted columns fiber rupture was observed. The strengthening system changed the failure mode from shear to flexural dominated. The authors also proposed a design methodology for FRP jackets to prevent shear failures.

- Haroun and Elsanadedy [5] tested two as-built, four retrofitted, and one repaired circular column under double curvature. CFRP or GFRP jackets were applied over the entire column height but with more layers in the plastic hinge zones (over  $D$  or  $1.5D$  at the top and bottom, where  $D$  is the column diameter). Although the addition of FRP jackets changed the failure mode from brittle shear to ductile flexural, failure was due to concrete crushing within the plastic hinge region for all specimens.

- Ozbakkaloglu and Saatcioglu [6] tested four circular cantilever columns confined by external CFRP tubes and no internal transverse steel. Discoloration of the CFRP was observed at initial stages of deformation. This effect was attributed to the separation between concrete and FRP. The failure of the specimens was caused by the rupture of the tube fibers.

- Shan et al. [7] tested eight cantilever columns, two of which were tested in as-built conditions and the remaining six with CFRP or GFRP. The intent of this study was to investigate long-term loading effect. To this end, four of the strengthened columns were analyzed for long-term axial load after being subjected to certain level of damage due to lateral load cycles, while the other two of retrofitted specimens were subjected to lateral loading only. Only the latter are included in the database. The extent of the jacketing was about  $0.53H$  for CFRP and  $0.4H$  for GFRP. Rupture in the FRP was observed for the GFRP-jacketed specimen. Bar rupture was observed for the CFRP-jacketed specimen.

- Ghosh and Sheik [8] tested six circular columns: three as-built and three strengthened with FRP. The columns had lap splices at the base. The specimens were tested as cantilevers under different levels of axial load. CFRP wraps were installed in the potential plastic hinge regions ( $0.3H$ ) while the remaining portion of the columns were covered with GFRP. Slippage of the spliced longitudinal bars was observed for the columns with low axial load ratio, and rupture of the FRP was observed for one specimen with a high axial load ratio  $P/P_o = 0.27$ , where  $P_o = 0.85f'_c(A_g - A_s) + A_s f_y$  (included in the database). However, this high axial load resulted in a considerable reduction in the ductility and energy dissipation capacity of the specimen.

- Gu et al. [9] tested 17 circular cantilever columns with different aspect ratios. Three specimens corresponded to as-built conditions, seven specimens were tested at low axial load ratios and the remaining six, were tested at high axial load ratios. All

columns were entirely wrapped FRP jackets and some specimens included additional confinement with FRP strips. Two types of carbon fibers and one type of ultra-high molecular weight polyethylene (Dyneema) fibers were used. Eleven specimens were subjected to cyclic loading until failure. The remaining six specimens were subjected to load reversals to a certain extent after which monotonical increase in load was applied until failure. All retrofitted columns failed in a flexural manner with FRP rupture. It was observed that higher axial load ratio decreased the drift ratio capacity. Also, larger aspect ratios increased drift ratio capacity. Monotonical load showed larger column drift capacity. The amount of confinement increased the drift ratio capacity until a certain point after which the opposite effect was observed. This behavior was attributed to the increase in frictional bond in the longitudinal steel provided by the FRP, which derives in higher variation of strains in the reinforcing steel, resulting in the reduction of the plastic hinge length for high confinement levels.

- Desprez et al. [10] tested four as-built and four retrofitted cantilever columns with constant CFRP thickness along their entire height. Results showed significant improvements in strength and ductility for the retrofitted columns. In all cases, rupture in the FRP was observed. For low level of axial load ratio, the gain in strength is small compared with the gain in ductility. The opposite effect was observed for high axial load ratio, with smaller gain in ductility but higher increment of the strength.

- Idris and Ozbakkaloglu [11] tested one cantilever circular column among other square section columns. All specimens were made with Aramid FRP tubes. No transverse reinforcement was included. High strength concrete was used. Discoloration in the wrap was observed around 5% lateral drift. Failure was due to FRP rupture. The circular column developed lower energy-dissipation but higher lateral drift capacities than the square specimens.

- Paultre et al. [12] tested four circular columns in as-built conditions, and four columns with CFRP jackets in a cantilever configuration. CFRP was applied along the entire height of the columns. Transverse reinforcement yielding was observed for all jacketed columns; however, no rupture of reinforcement nor CFRP was observed.
- Seible et al. [13] tested four circular columns, including one as-built, one retrofitted with steel jacketing and two columns retrofitted with CFRP jacketing. The CFRP was extended for about  $0.3H$  from the base and all specimens were tested in cantilever configuration. Retrofitted columns showed significant improvement in changing the shear failure into a more ductile one. One specimen, however, experienced a lap-splice failure due to insufficient CFRP confinement. The other specimen achieved larger ductility, and the failure mode changed to bar rupture.
- Yousff et al. [14] tested five cantilever circular columns including one as-built and one retrofitted column with CFRP in the entire length. The other three specimens were made with concrete with rubber particles, two of which were retrofitted with CFRP. These specimens were included in the database. Failure mode in all specimens was due to rupture in the most stressed longitudinal bar. No significant change in behavior was observed between rubberized concrete and normal concrete.
- Xiao and Ma [15] tested one as-built and two retrofitted circular columns in cantilever configuration. The GFRP was extended to  $\sim 0.5H$  except for one specimen, which was retrofitted almost entirely but with fewer layers in the upper half. Columns were tested until bond deterioration in the lap splices was observed with the observed maximum moment strength being 70-80% of the theoretical flexural strength of the section. Fine horizontal cracks on the surface at a high ductility demand were observed. No delamination or rupture occurred in the jacketing. Brittle failure due to bond deterioration in the as-built

columns was changed to a more stable with gradual degradation failure for the retrofitted columns.

- Haroun and Elsanadedy [16] tested two as-built and six columns retrofitted with CFRP and GFRP in a cantilever configuration. The jackets were applied over the entire column height but with more layers in the plastic hinge region. Failure in all the retrofitted specimens was due to lap-splice slippage, however, these columns presented a more ductile behavior as compared to the as-built specimens.

In summary, the addition of FRP jackets in the above-mentioned studies proved to significantly increase the displacement ductility capacity of columns, and for the most part change the failure mode from shear-dominated or lap-splice failures to one dominated by flexure. The most common type of flexural failure was governed by rupture of the FRP, initiated in some cases by buckling of the reinforcing bars. Other types of flexural failures were crushing in the concrete within the plastic hinge region and rupture of longitudinal reinforcing bars.

From the above-mentioned studies, only circular columns strengthened with FRP have been included in the database. The main characteristics of the 56 column specimens comprising the database are shown in Table 3.1. Figure 3.1 presents frequency plots for of the column diameter, aspect ratio, longitudinal reinforcement ratio, and volumetric FRP confinement ratio for the column specimens included in the database. Column diameters vary between 150 mm and 760 mm. The typical aspect ratio ( $H/D$ ) varies between 2.0 and 6.6, with a maximum of 7.4 and a minimum of 1.5 (the latter corresponding to one of the tests in double curvature where  $H$  is half the total length of the specimen). 50% of the tests were performed applying an axial load ratio  $P/(f'_c \cdot A_g)$  between 5% and 10%. 16 tests had low axial load ratio (5% or less) and two tests had considerably high axial load ratio (>60%). More variability is observed with respect to the FRP confinement ratio  $\lambda_f = 2 \cdot$

$E_f \cdot t_f \cdot \varepsilon_f / (D \cdot f'_c)$ , where the minimum was 6.7% and the maximum was 84.7%, with most of the columns being retrofitted with 10% to 30% FRP confinement ratio. With respect to materials, the typical concrete compressive strength varies between 25 to 35 MPa, and the typical steel yield strength was is 400 MPa with a minimum of 300 MPa and a maximum of 600 MPa.

Table 3.1. Characteristics of column specimens included in the database

| ID | Name    | Author                                  | D    | H    | $f_c$ | $f_y$ | $P/(f_c \cdot A_g)$ | $\rho = A_s/A_g$ | $\rho_{vol}$ | Jacket      | Type of failure |
|----|---------|-----------------------------------------|------|------|-------|-------|---------------------|------------------|--------------|-------------|-----------------|
|    |         |                                         | [mm] | mm   | MPa   | MPa   | %                   | %                | %            |             |                 |
| 1  | CSJ_RT  | Xiao et al. (1999) [2]                  | 610  | 915  | 35.9  | 303   | 6.2                 | 2.4              | 0.15         | GFRP        | FRP rupture     |
| 2  | ST_2NT  | Sheik and Yau (2002) [3]                | 356  | 2010 | 40.4  | 550   | 67.0                | 4.2              | 0.31         | GFRP        | FRP rupture     |
| 3  | ST_3NT  |                                         | 356  | 2010 | 40.4  | 550   | 67.0                | 4.2              | 0.31         | CFRP        | FRP rupture     |
| 4  | ST_4NT  |                                         | 356  | 2010 | 44.8  | 550   | 32.4                | 4.2              | 0.31         | CFRP        | FRP rupture     |
| 5  | ST_5NT  |                                         | 356  | 2010 | 40.8  | 550   | 33.4                | 4.2              | 0.31         | CFRP        | FRP rupture     |
| 6  | FCS_1   | Li and Sung (2004) [4]                  | 760  | 1750 | 18.6  | 426   | 16.6                | 2.2              | 0.14         | CFRP        | FRP rupture     |
| 7  | FCS_2   |                                         | 760  | 1750 | 18.6  | 426   | 16.6                | 2.2              | 0.14         | CFRP        | FRP rupture     |
| 8  | CS_R4   | Haroun and Elsanadedy (2005) [5]        | 610  | 1219 | 37.6  | 480.7 | 5.9                 | 2.3              | 0.18         | CFRP        | FRP rupture     |
| 9  | RC_1    | Ozbakkal oğlu and Saatcioglu (2006) [6] | 270  | 2000 | 90.1  | 500   | 30.6                | 4.2              | 0.00         | CFRP        | FRP rupture     |
| 10 | RC_2    |                                         | 270  | 2000 | 75.2  | 500   | 34.4                | 4.2              | 0.00         | CFRP        | FRP rupture     |
| 11 | RC_3    |                                         | 270  | 2000 | 49.7  | 500   | 52.0                | 4.2              | 0.00         | CFRP        | FRP rupture     |
| 12 | CCR-2   | Shan et al. (2006) [7]                  | 375  | 1500 | 30.9  | 372   | 20.0                | 1.8              | 0.65         | GFRP        | FRP rupture     |
| 13 | CAF1_5N | Ghosh and Sheikh (2007) [8]             | 356  | 2010 | 25.1  | 465   | 27.0                | 2.3              | 0.31         | CFRP        | FRP rupture     |
| 14 | J1      | Gu et al. (2010) [9]                    | 300  | 850  | 28    | 400   | 5.1                 | 6.7              | 0.32         | DFRP        | FRP rupture     |
| 15 | J2      |                                         | 300  | 850  | 28    | 400   | 5.1                 | 6.7              | 0.32         | CFRP        | FRP rupture     |
| 16 | J3      |                                         | 300  | 850  | 28    | 400   | 5.1                 | 6.7              | 0.32         | CFRP        | FRP rupture     |
| 17 | J4      |                                         | 300  | 850  | 28    | 400   | 5.1                 | 6.7              | 0.32         | DFRP        | FRP rupture     |
| 18 | J5      |                                         | 300  | 850  | 28    | 400   | 5.1                 | 6.7              | 0.32         | CFRP        | FRP rupture     |
| 19 | J6      |                                         | 300  | 850  | 28    | 400   | 5.1                 | 6.7              | 0.32         | CFRP + DFRP | FRP rupture     |
| 20 | J7      |                                         | 300  | 850  | 28    | 400   | 5.1                 | 6.7              | 0.32         | CFRP + DFRP | FRP rupture     |
| 21 | J8      |                                         | 300  | 850  | 28    | 400   | 5.1                 | 6.7              | 0.32         | CFRP + DFRP | FRP rupture     |
| 22 | CH1     |                                         | 360  | 1100 | 34.9  | 382   | 33.8                | 8.4              | 0.28         | +DFRP       | FRP rupture     |
| 23 | CH2     |                                         | 360  | 1100 | 34.9  | 382   | 33.8                | 8.4              | 0.28         | DFRP        | FRP rupture     |
| 24 | CH3     |                                         | 360  | 1100 | 34.9  | 382   | 33.8                | 8.4              | 0.28         | CFRP        | FRP rupture     |
| 25 | CL1     |                                         | 360  | 800  | 34.9  | 382   | 33.8                | 8.4              | 0.28         | DFRP        | FRP rupture     |
| 26 | CL2     |                                         | 360  | 800  | 34.9  | 382   | 33.8                | 8.4              | 0.28         | CFRP        | FRP rupture     |
| 27 | CL3     |                                         | 360  | 800  | 34.9  | 382   | 33.8                | 8.4              | 0.28         | CFRP        | FRP rupture     |
| 28 | P1C     | Desprez et al. (2013) [10]              | 300  | 2000 | 35.8  | 415   | 8.9                 | 3.6              | 0.70         | CFRP        | FRP rupture     |
| 29 | P2C     |                                         | 300  | 2000 | 34.9  | 415   | 35.1                | 3.6              | 0.70         | CFRP        | FRP rupture     |
| 30 | P3C     |                                         | 300  | 2000 | 34.4  | 415   | 10.2                | 3.6              | 0.35         | CFRP        | FRP rupture     |
| 31 | P4C     |                                         | 300  | 2000 | 34.3  | 415   | 33.2                | 3.6              | 0.35         | CFRP        | FRP rupture     |
| 32 | CFFT6   | Idris and Ozbakkal                      | 150  | 1000 | 95    | 600   | 43.2                | 4.3              | 0.00         | AFRP        | FRP rupture     |

| ID               | Name      | Author                           | D    | H    | f <sub>c</sub> | f <sub>y</sub> | P/<br>(f <sub>c</sub> ·A <sub>g</sub> ) | ρ=A <sub>s</sub> /<br>A <sub>g</sub> | ρ <sub>vol</sub> | Jacket | Type of failure   |
|------------------|-----------|----------------------------------|------|------|----------------|----------------|-----------------------------------------|--------------------------------------|------------------|--------|-------------------|
|                  |           |                                  | [mm] | mm   | MPa            | MPa            | %                                       | %                                    | %                |        |                   |
| oglu (2013) [11] |           |                                  |      |      |                |                |                                         |                                      |                  |        |                   |
| 33               | S75P10C1  | Paultre et al. (2016) [12]       | 305  | 2000 | 35.8           | 415            | 10.0                                    | 4.0                                  | 2.22             | CFRP   | Not available     |
| 34               | S75P35C1  |                                  | 305  | 2000 | 34.9           | 415            | 35.0                                    | 4.0                                  | 2.22             | CFRP   | Not available     |
| 35               | S150P10C1 |                                  | 305  | 2000 | 34.2           | 415            | 10.0                                    | 4.0                                  | 1.11             | CFRP   | Not available     |
| 36               | S150P35C1 |                                  | 305  | 2000 | 34.2           | 415            | 35.0                                    | 4.0                                  | 1.11             | CFRP   | Not available     |
| 37               | T2        | Seible et al. (1997) [13]        | 610  | 3658 | 34.4<br>5      | 303.1          | 17.7                                    | 2.9                                  | 0.18             | CFRP   | Bar rupture       |
| 38               | CCR-1     | Shan et al. (2006) [7]           | 375  | 1500 | 30.9           | 372            | 20.0                                    | 1.8                                  | 0.65             | CFRP   | Bar rupture       |
| 39               | CCF2      | Yousff et al. (2015) [14]        | 240  | 1500 | 60             | 550            | 4.1                                     | 2.2                                  | 1.69             | CFRP   | Bar rupture       |
| 40               | CRCF2     |                                  | 240  | 1500 | 46.4           | 550            | 5.2                                     | 2.2                                  | 1.69             | CFRP   | Bar rupture       |
| 41               | ISJ_RT    | Xiao et al. (1999) [2]           | 610  | 915  | 35.9           | 303            | 6.2                                     | 2.4                                  | 0.15             | GFRP   | No failure        |
| 42               | CS_R3     | Haroun and Elsanadey (2005) [5]  | 610  | 1219 | 34.2           | 480.7          | 6.5                                     | 2.3                                  | 0.18             | GFRP   | No failure        |
| 43               | CS_R1     |                                  | 610  | 1219 | 40.8           | 299.1          | 5.4                                     | 2.3                                  | 0.18             | CFRP   | Concrete crushing |
| 44               | CS_R2     |                                  | 610  | 1219 | 39.2           | 299.1          | 5.6                                     | 2.3                                  | 0.18             | CFRP   | Concrete crushing |
| 45               | T1        | Seible (1997) [13]               | 610  | 3658 | 34.4<br>5      | 303.1          | 17.7                                    | 2.9                                  | 0.18             | CFRP   | Lap Splice        |
| 46               | C2-RT4    | Xiao and Ma (1997) [15]          | 610  | 2440 | 44.8           | 462            | 5.4                                     | 2.6                                  | 0.19             | GFRP   | Lap Splice        |
| 47               | C3-RT5    |                                  | 610  | 2440 | 44.8           | 462            | 5.4                                     | 2.6                                  | 0.19             | GFRP   | Lap Splice        |
| 48               | C4-RP4    |                                  | 610  | 2440 | 44.8           | 462            | 5.4                                     | 2.6                                  | 0.19             | GFRP   | Lap Splice        |
| 49               | CF-R1     | Haroun and Elsanadey (2005) [16] | 610  | 3660 | 36             | 299.1          | 9.9                                     | 2.3                                  | 0.18             | CFRP   | Lap Splice        |
| 50               | CF-R2     |                                  | 610  | 3660 | 36.9           | 299.1          | 9.9                                     | 2.3                                  | 0.18             | CFRP   | Lap Splice        |
| 51               | CF-R3     |                                  | 610  | 3660 | 32.8           | 299.1          | 9.9                                     | 2.3                                  | 0.18             | GFRP   | Lap Splice        |
| 52               | CF-R4     |                                  | 610  | 3660 | 37.7           | 299.1          | 9.9                                     | 2.3                                  | 0.18             | CFRP   | Lap Splice        |
| 53               | CF-R5     |                                  | 610  | 3660 | 39.7           | 299.1          | 9.9                                     | 2.3                                  | 0.18             | GFRP   | Lap Splice        |
| 54               | CF-R6     |                                  | 610  | 3660 | 33.1           | 299.1          | 9.9                                     | 2.3                                  | 0.18             | CFRP   | Lap Splice        |
| 55               | CAF1_2N   | Ghosh and Sheikh (2007) [8]      | 356  | 2010 | 24.9           | 465            | 5.0                                     | 2.3                                  | 0.31             | CFRP   | Lap Splice        |
| 56               | CBF1_6N   |                                  | 356  | 2010 | 26.5           | 465            | 5.0                                     | 2.3                                  | 1.16             | CFRP   | Lap Splice        |

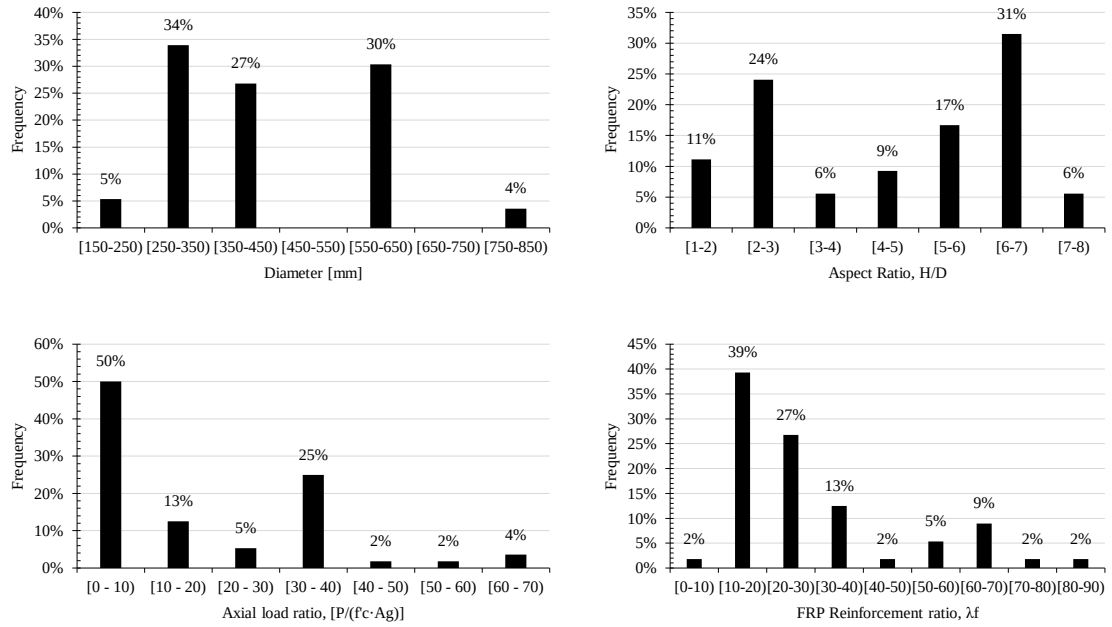


Figure 3.1 Distribution of main characteristics of columns included in the database

### 3. Analysis of the lateral deformation capacity of FRP-jacketed circular columns

The lateral deformation capacity of flexure-dominated cantilever columns is computed with the analytical plastic hinge model presented in Figure 3.2. For this model, the yield displacement of the column, i.e. the top displacement when the column base reaches its effective yield curvature  $\phi_y$ , is obtained with Eq. (1).

$$\Delta_y = \frac{\phi_y \cdot H^2}{3} \quad (1)$$

At ultimate conditions, it is assumed that plastic deformations concentrate along an effective plastic hinge length  $l_p$ , as shown in Figure 3.2a. The ultimate displacement of the column, i.e. the displacement when the column base reaches ultimate curvature  $\phi_u$ , is then obtained with Eq. (2).

$$\Delta_u = \Delta_y + \Delta_p = \frac{\phi_y \cdot H^2}{3} + (\phi_u - \phi_y)(l_p) \cdot (H) \quad (2)$$

The values of  $\phi_y$  and  $\phi_u$ , needed to characterize the deformation behavior of the cantilever column are obtained from an idealized moment-curvature like that presented in Figure 3.2b. An approximate value for  $\phi_y$ , equal to  $2.45\varepsilon_y/D$  was proposed by Priestley et al. [17] for conventional RC columns. The factor was later refined by Priestley et al. [33] based on parametric moment-curvature analyses resulting in the following expression:

$$\phi_y = 2.25 \cdot \frac{\varepsilon_y}{D} \quad (3)$$

Since the value of  $\phi_y$ , is independent of the amount of confinement level, Eq. (3) is used here for FRP-jacketed columns.

For columns loaded under fixed-fixed conditions, a similar model can be derived assuming plastic hinges form at both the top and bottom of the column. By integrating their double-curvature distribution, the ultimate displacement of these column is obtained as:

$$\Delta_u = \Delta_y + \Delta_p = 2 \frac{\phi_y \cdot (0.5H)^2}{3} + 2(\phi_u - \phi_y)(l_p) \cdot (0.5H) \quad (4)$$

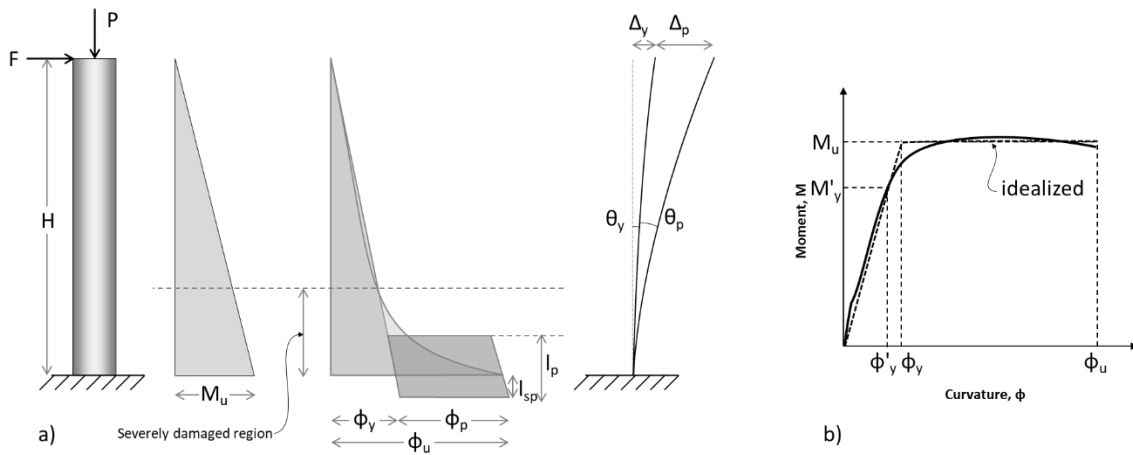


Figure 3.2. Plastic-hinge model for cantilever column: (a) distribution of moment, curvature, and deflection, (b) moment-curvature diagram

Two different approaches to predict the ultimate displacement capacity of FRP-jacketed columns are evaluated against the experimentally-obtained deformation capacities of the columns in the database failing in flexure due to FRP rupture, which is the most common type of failure. Both approaches are based on the plastic hinge model described above but employ different equations to calculate  $\phi_u$  and  $l_p$ . The first one follows ACI 440.2R-17 design recommendations [18] to compute  $l_p$ , and to determine  $\phi_u$  from moment-curvature analysis. The second one uses equations for  $\phi_u$  and  $l_p$  proposed by Gu et al. [20].

A third alternative approach, which uses a new equation for  $\phi_u$  derived from moment-curvature analyses of the columns in the database, is also proposed here.

The ultimate curvature is the ratio between the ultimate strain and the depth of the neutral axis in compression,  $c$ .

$$\phi_u = \varepsilon_{cu}/c \quad (5)$$

The ultimate strain in the concrete is defined by the concrete model used (i. e. Lam and Teng [19]), hence, the unknown is the depth of the neutral axis,  $c$ , which depends on the axial load, longitudinal steel and transverse reinforcement; and can be obtained directly from moment curvature analysis.

The plastic hinge length defined in Figure 3.2a, includes one term that accounts for the strain penetration  $l_{sp}$ , which is usually taken as:

$$l_{sp} = 0.022 \cdot f_{ye} \cdot d_{bl} \quad (6)$$

where  $f_{ye}$  is the steel yield strength and  $d_{bl}$  is the diameter of longitudinal bar reinforcement. Strain penetration occurs in both, the column and the foundation, hence, a minimum value for the plastic hinge length is determined by two times  $l_{sp}$ .

A moment-curvature analysis was performed for each of the specimens collected in the database using OpenSees software developed by PEER [34]. Concrete 02 material with tensile strength as shown in Figure 3.3 is used. The parameters for the compression part were calibrated with concrete models defined subsequently. This model considers a residual strength in concrete that remains constant after  $\varepsilon_{cu}$ , therefore, the results after this point were not accounted.

For the longitudinal reinforcing steel, a bilinear elastic-plastic behavior was used as shown in Figure 3.3. Parameter  $b$  was fixed to a value of 0.01, to account for the post-yield linear stress increase to represent strain hardening. The effect of the FRP material was considered implicitly in the concrete model as it exclusively provides confinement to the column.

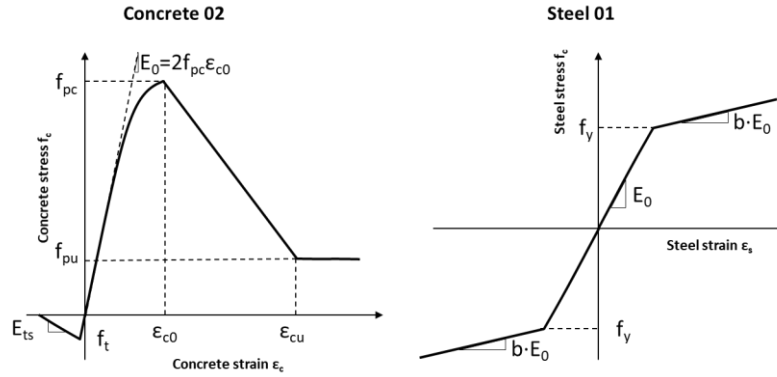


Figure 3.3. Material laws for concrete (left) and steel (right)

### 3.1 PREDICTIONS USING ACI 440.2R-17 AND LAM AND TENG (2003) EQUATIONS

The ultimate displacement of the FRP columns of the database is predicted using the plastic hinge model in Figure 3.2 and the approach recommended in ACI 440.2R-17 [18] to determine the effective plastic hinge length and ultimate curvature capacity. The guidelines refer to the stress-strain law for FRP-confined concrete proposed by Lam and Teng [19]. This material model was developed on the basis that the FRP remains elastic until failure, therefore, the confinement provided by this material is constantly increasing, unlike the confinement provided by reinforcing steel which remains constant after the bars yield. This model ignores the contribution of the transverse reinforcement. ACI 440.2R-17 [18] incorporates this model with minimal adjustments to design the retrofit of existing circular columns with FRP. Later, Teng et al. [35] proposed a revised version of Lam and Teng [19] model with adjustments for the behavior at ultimate conditions, shown in Figure

3.4b. Finally, Lin et al. [36], proposed new modifications to previous model, considering the contribution of the transverse steel reinforcement. Such model is presented in Figure 3.4c. Figure 3.4 also includes the equations to obtain the confined concrete strength,  $f'_{cc}$ , the post-yield modulus of elasticity,  $E_2$ , and the ultimate strain  $\epsilon_{cu}$  for each of the models discussed.

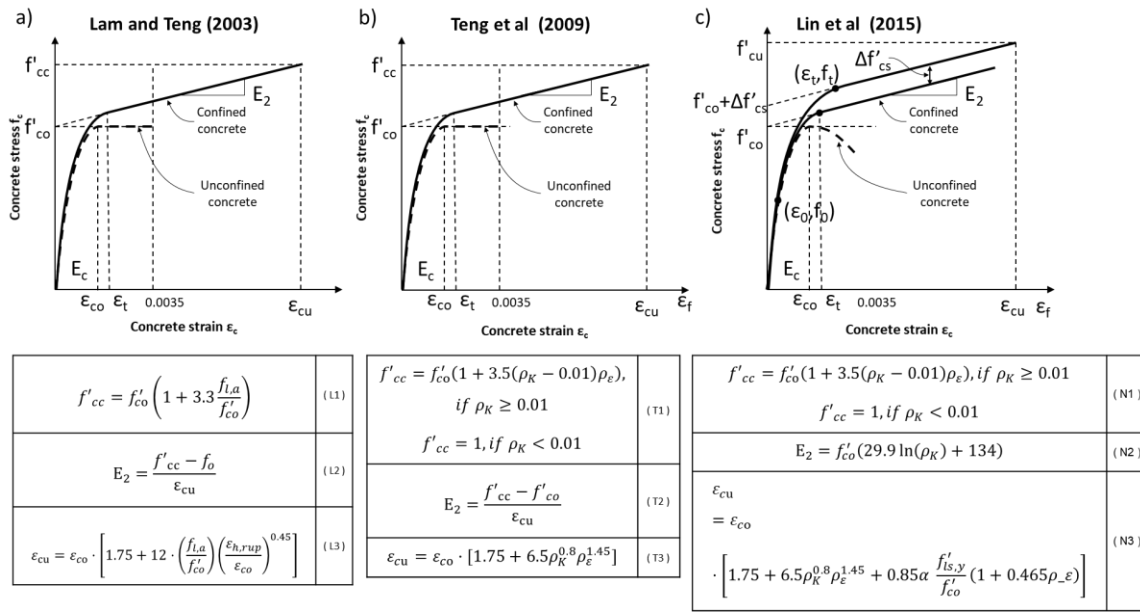


Figure 3.4. FRP-confined concrete stress-strain models

The stress-strain behavior can be characterized by two points: the end of first stage defined by  $f'_c$  at a strain  $\epsilon_t$ ; and the ultimate conditions defined by  $f'_{cu}$  at  $\epsilon_{cu}$ . The unconfined concrete strength  $f'_c$  is specified, and the initial strain  $\epsilon_{co}$  is usually taken as 0.002. Some authors (Gu et al. [20]) recognize that moment-curvature analysis is insensitive to the model used in terms of strength, however, the strain at ultimate conditions influences greatly in the ultimate curvature of the section. Equations proposed by Lam and Teng [19] (Eq. L-3 in Figure 3.4) and Teng et al. [35] (Eq. T-3 in Figure 3.4) results in similar ultimate

strain values. When reinforcement steel is considered, larger strains are achieved, considering the additional terms in Eq. N-3 in Figure 3.4 for the model proposed by Lin et al. [36].

Priestley et al. [33] proposed the following expression to compute the plastic hinge length for conventionally reinforced concrete columns:

$$l_p = k \cdot H + l_{sp} > 2 \cdot l_{sp} \quad (7)$$

where  $k = 0.2(f_u/f_y - 1) \leq 0.08$ . For columns retrofitted with steel jackets or FRP jackets, Priestley et al. [17] proposed using a different equation to account for a more limited spread of plasticity caused by the jackets.

$$l_p = g + 0.044f_{ye}d_{bl} \quad (8)$$

where  $g$  is the dimension of the gap between the base and the start of the FRP wraps, when provided, and the second term corresponds to  $2 \cdot l_{sp}$ . Equation (8) is used in ACI 440.2R-17 to compute the plastic hinge length for columns retrofitted with FRP.

In this section, the results of moment curvature analysis using Lam and Teng [19] concrete model (Eqs. L-1 to L-3 in Figure 3.4) are shown. Based on a large test database, Lam and Teng [19] found that the material ultimate strain and the actual hoop rupture strain can be related by an efficiency factor of 0.586, to account for premature failure of FRP due to concentration of stresses in cracks. This factor was used for the effective ultimate strain of the FRP ( $\varepsilon_{fe} = 0.586\varepsilon_{fu}$ ). As mentioned earlier, this model was incorporated by ACI 440.2R-17 [18] with minimal adjustments for design.

Figure 3.8 shows the comparison of the measured vs. computed top displacement for the column specimens that failed with FRP rupture only (31 specimens). The top

displacement was computed using Eqs. (2), (3), and (8). The ultimate curvature was obtained from the moment-curvature analysis as described above. The mean of the ratio of the measured to the calculated displacement is 2.65, with a COV of 40.34%. Hence, the use of equations proposed in ACI 440.2R-17 tend to a clear underestimation of the lateral deformation capacity of columns and a relatively high dispersion. As shown in Figure 3.8, all the predictions resulted in  $\Delta_{u\_me} / \Delta_{u\_calc}$  larger than 1.

### 3.2 RESULTS USING GU ET AL. (2015) EQUATIONS

Gu et al. [20] proposed a method to calculate the ultimate curvature of circular columns retrofitted with FRP which does not require moment-curvature analysis. As shown in Eq. (5), the ultimate curvature depends both on the ultimate strain of concrete and the depth of neutral axis,  $c$ . Both parameters are a function of the level of passive confinement. Gu et al. [20] derived an equation for the ratio of  $c/D$ , based on regression of parametric numerical analyses of one cross-section with different axial load ratios and different confinement ratios. The resulting expression is:

$$\frac{c}{D} = 0.19 + (0.72 - 0.67 \cdot \lambda_f) \cdot n \quad (9)$$

where  $\lambda_f$  is the FRP confinement ratio and  $n$  the axial load ratio, defined as:

$$\lambda_f = \frac{2 \cdot E_f \cdot t_f \cdot \varepsilon_f}{D \cdot f'_c} \quad (10)$$

$$n = \frac{N}{A_g f'_c} \quad (11)$$

Equation (9) expresses that the depth of the neutral axis at ultimate conditions is 19% of the section depth plus a factor that is a function of the level of confinement and

axial load. By replacing (9) in (5), and conducting a regression analysis with ultimate curvatures reported in a limited number of column tests completed by Sheik and Yau [3], Gu et al. [20] proposed the following expression for the ultimate curvature:

$$\phi_u = \frac{0.008 + 0.09 \cdot \lambda_f}{(0.19 + (0.72 - 0.67 \cdot \lambda_f) \cdot n)D} \quad (12)$$

Similarly, based on Eq. (7), and by regression analysis of ultimate drift obtained from a test database collected by the authors, Gu et al. [20] noted a relationship between the plastic hinge length and the confinement ratio, and obtained an equation for the  $k$  value in terms of the confinement parameter  $\lambda_f$ , resulting in the following equation for the plastic hinge length:

$$l_p = (3.028 + 0.08\lambda_f)H + 0.022 \cdot f_{ye} \cdot d_{bl}, \quad \text{for } 0 < \lambda_f < 0.1 \quad (13)$$

$$l_p = (0.59 - 2.30\lambda_f + 2.28\lambda_f^2)H + 0.022 \cdot f_{ye} \cdot d_{bl}, \quad \text{for } \lambda_f > 0.1 \quad (14)$$

Eq. (13) is suggested by Gu et al. [20] given the lack of tests with small confinement ( $\lambda_f < 0.1$ ) and to match with the  $k$  value proposed by Priestley et al. [16] for columns with no confinement.

Using Eqs. (2), (3), (12 – 14), the ultimate displacement is computed and plotted in Figure 3.8. The mean of the ratio  $\Delta_{u\_me} / \Delta_{u\_calc}$  is 1.02, with a COV of 24.96%.

### 3.3 PROPOSED MODEL

The results presented in Section 3.1 have indicated that the approach recommended in ACI 440.2R-17 [18] to compute the ultimate deformation capacity of FRP-jacketed columns clearly underestimates the lateral displacement capacity of the column specimens

in the database. This is attributed here to the use of a confined concrete model which was derived from tests on concentric loading. As discussed previously, models based on Lam and Teng [19], for instance Lin et al. [36], have provided larger ultimate strains for concrete. Consequently, larger curvatures are expected if more refined material models are used. Besides, all these models have considered concentrically loaded circular columns. Lin and Teng [37] proposed a new constitutive model to account for the variation in stress distribution along the section of columns subjected to eccentric loading, i. e. columns with axial forces in combination with a flexural moment. With this model, the ultimate strain in concrete is obtained by multiplying the ultimate strain from a concentric model (i. e.  $\varepsilon_{cu,con}$  calculated with Lin et al. [36]), by a factor which is a function of the depth of the neutral axis, as follows:

$$\varepsilon_{cu,ecc} = \varepsilon_{cu,con} \left[ 1 + 0.253 \frac{D}{c} + 0.0227 \left( \frac{D}{c} \right)^2 \right] \quad (15)$$

The slope in the second portion of the stress-strain is also modified with a similar factor, also a function of  $D/c$ :

$$E_{2,ecc} = E_{2,con} \left[ 1 - 0.0808 \frac{E_{2,con}}{|E_{2,con}|} \frac{D}{c} \right], \quad \frac{D}{c} \leq 12.4 \quad (16)$$

Given that the depth of the neutral axis is not known a priori, the application of this model in a moment-curvature analysis requires iteration.

A new set of moment-curvature analyses were performed using Lin and Teng [37] eccentrically load approach with Lin et al. [36] constitutive model. For each column section, a first iteration of moment-curvature was performed using the concentric model by Lin et al. [36]. Once  $D/c$  values were obtained, the parameters for eccentric loading

were computed (Eqs. (15) and (16)) and these values were iteratively updated with moment-curvature analyses until the difference in  $D/c$  was less than 1%. The ultimate concrete strains obtained from these analyses are considerable larger than those obtained in Section 3.1. The average  $D/c$  value obtained from the 31 column section analyses was approximately 4.65, which results in an average factor of 2.6 for the ultimate strain corresponding to concentric loading case.

Based on the results of the moment-curvature analyses, a simplified equation has been derived to compute ultimate curvature. This equation follows the format of Eq. (5) but the depth of the neutral axis is defined as a fraction of the column diameter,  $f \cdot D$ . Furthermore,  $\varepsilon_{cu}$  is taken as that computed using concentric model (Eq. N-3 in Figure 3.4). The reason for this is that the eccentric strain  $\varepsilon_{cu}$  is also a function of  $D/c$ , and the moment-curvature is required to obtain such parameter. Consequently, the factor considering the eccentric load will be included within the value of  $f$ .

$$\phi_u = \frac{\varepsilon_{cu}}{f \cdot D} \cdot \quad (17)$$

To calibrate  $f$ , an analysis of the influence of different parameters in the ultimate curvature values obtained from moment-curvature analysis is conducted. Figure 3.5 shows  $\phi_u$  plotted against the confinement ratio  $\lambda_f$ , the axial load ratio  $P/(f'_c \cdot A_g)$ , and the longitudinal reinforcing ratio  $\rho$ . As shown in Figure 3.5, there is no clear correlation between factor  $f$  and the confinement ratio,  $\lambda_f$ . The reason is that the effect of confinement on the ultimate curvature is already considered in the computation of  $\varepsilon_{cu}$  (see Eq. N-3 in Figure 3.4). Similarly, no correlation can be observed between the longitudinal ratio and the ultimate curvature. Finally, a defined trend is observed for the axial load ratio,  $P/(f'_c \cdot A_g)$ , with an inverse linear relation, that is, the influence decreases as the axial load ratio

increases. This behavior is explained by considering that an increase of the axial load will increase the depth of the neutral axis at ultimate conditions, hence decreasing the ultimate curvature.

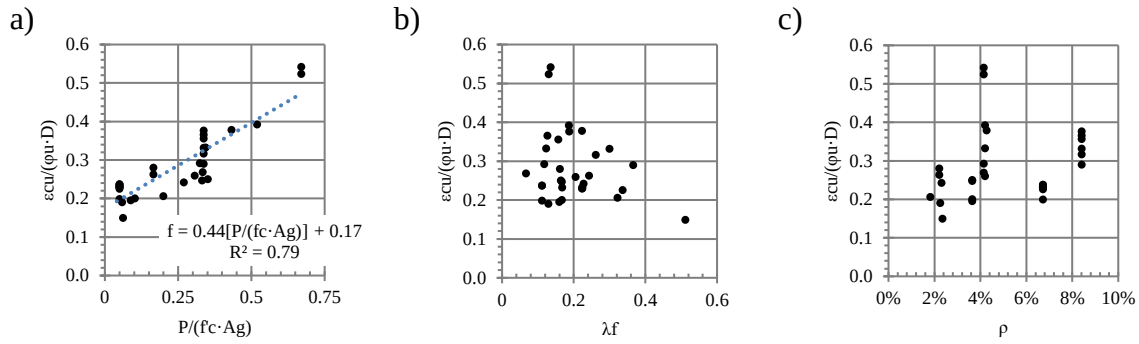


Figure 3.5. Ultimate Curvature parameters

Based on the results of this analysis, an equation to estimate the ultimate curvature is presented in Eq. (18), where  $\varepsilon_{cu}$  was computed using Eq. N-3 from Figure 3.4.

$$\phi_u = \frac{\varepsilon_{cu}}{\left(0.44 \frac{P}{f_c' \cdot A_g} + 0.17\right) D} \quad (18)$$

The values obtained with such equation are plotted in Figure 3.6 against the results from the ultimate curvature obtained from moment-curvature analysis for those tests in the database that failed by FRP Rupture (31 tests). The mean of the ratio  $\phi_{u,mom.curv} / \phi_{u,eq18}$  is 1.0 and the coefficient of variation is 16 %. Figure 3.6 also presents results for the ultimate curvature computed with Eq. (12) proposed by Gu et al. [20]. This data has a mean of 0.93 and a coefficient of variation of 16% for the ratio  $\phi_{u,mom.curv} / \phi_{u,eq12}$ .

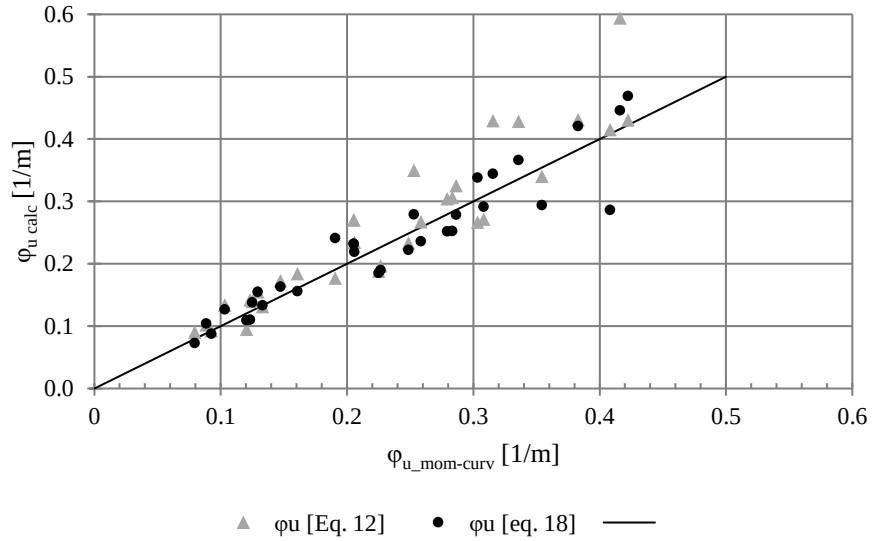


Figure 3.6. Ultimate curvature

The ultimate top displacement  $\Delta_u$ , can be then computed using Eq. (2), with  $\phi_y$  and  $\phi_u$  from Eqs. (3) and (18) respectively, and  $l_p$  from Eq. (8). The values are plotted in Figure 3.8 tagged as P1. The mean of the ratio  $\Delta_{u\_me} / \Delta_{u\_calc}$  is 1.08, with a COV of 32.47%. Clearly, there is an improvement in the estimation as compared with the prediction using ACI 440.2R-17 specifications [18], but still not as accurate as the prediction using the approximation by Gu et al. [20].

A better estimate can be achieved by revisiting the equation for the plastic hinge length. Plastic hinge length depends on several variables such as the axial load ratio, the moment gradient, longitudinal and transverse reinforcement, concrete strength, and level of confinement (Bae and Bayrak [38]). Using the measured ultimate displacement from tests and the aforementioned equations for yield and ultimate curvature, the ultimate displacement in Eq. (2) can be solved for the value of  $l_p$ , then, the value of  $k$  is obtained by subtracting the gap between FRP and the base, and the strain penetration term corresponding to the base, and then dividing by the height. Figure 3.7 shows the  $k$  values

as a function of the confinement ratio. By regression analysis and setting the intercept in 0.08, which corresponds to the unconfined case, the following expression is derived:

$$l_p = (-4.45\lambda_f^2 + 1.23\lambda_f + 0.08) \cdot H + l_{sp} + g > 2 \cdot l_{sp} + g \quad (19)$$

The above equation is consistent with the value of  $l_p$  proposed by Priestley [33] for unretrofitted columns (i. e.  $\lambda_f = 0$ ). The behavior governed by the quadratic equation is also consistent with the expression proposed by Gu et al. [20] shown in Eqs. (13) and (14). This increase and later decrease in the plastic hinge length is explained by Gu et al. [20] considering that the extent of the plastic hinge increases for small confinement values ( $\lambda_f < 0.15$ ) due to the increase in the flexural strength of the section, which allows a larger portion of the column to develop plasticity. However, if larger confinement in the section is applied, additional bond stresses in the longitudinal bars are developed, resulting in an increase in the variation of strains along the bar, which in turn concentrates the deformation within a smaller region. For large confinement ratios ( $\lambda_f > 0.36$ ), the quadratic term in Eq. (19) results in negative values, however, for these cases, the second portion of the equation controls, resulting always a positive plastic hinge length.

The ultimate top displacements computed using eq. (19) are plotted in Figure 3.8 tagged as P2. The mean of the measured to the calculated top displacement is 1.01 with a COV of 26.44%. These results are comparable with those obtained when using the approximation proposed by Gu et al. [20]. The advantage of this equation is that in a single expression the contribution of the confinement is accounted, it is compatible with unretrofitted models and the expression still considers the minimum value due to strain penetration happening in both, the column length and in the base.

Table 3.2 presents a summary of the lateral displacement values for each of the approaches described above.

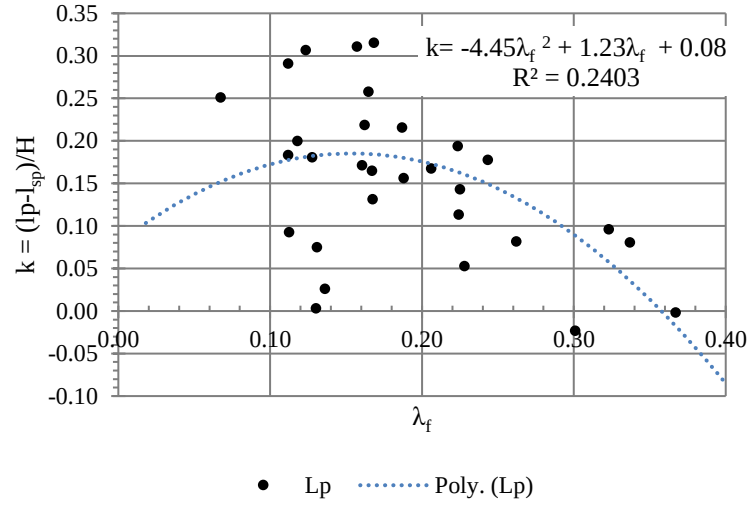


Figure 3.7. Effect of confinement in plastic hinge length

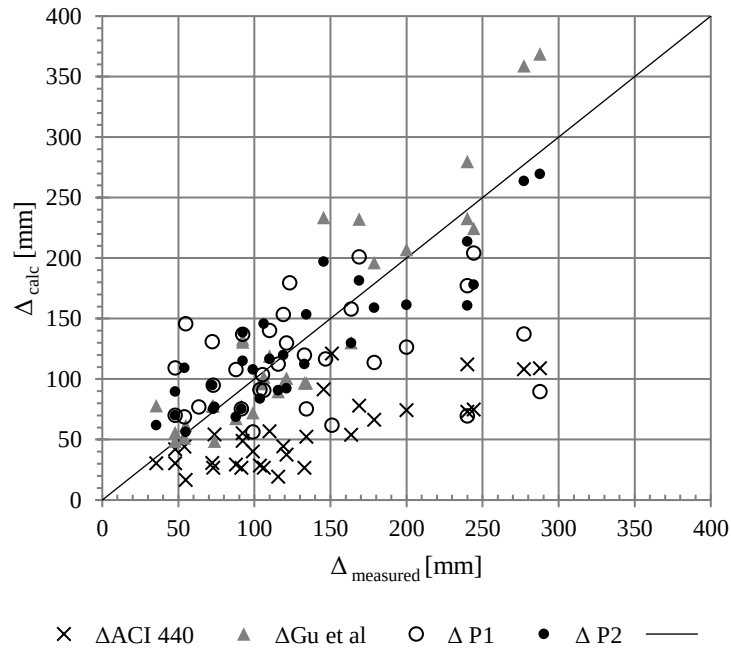


Figure 3.8. Comparison of measured vs. analytical displacement

Table 3.2. Results for experimental and calculated displacement

| ID | Spec.   | Ref  | $\lambda_f$ | $\Delta_{me}$ | $l_{sp}$ | ACI 440.2R-17 |            | Gu et al. |            | P1   |            | P2   |            |
|----|---------|------|-------------|---------------|----------|---------------|------------|-----------|------------|------|------------|------|------------|
|    |         |      |             |               |          | lp            | $\Delta$   | lp        | $\Delta$   | lp   | $\Delta$   | lp   | $\Delta$   |
|    |         |      |             | [mm]          | [mm]     | [mm]          | [mm]       | [mm]      | [mm]       | [mm] | [mm]       | [mm] | [mm]       |
| 1  | CS_R4   | [18] | 0.13        | <b>36</b>     | 201      | 427           | <b>31</b>  | 602       | <b>78</b>  | 427  | <b>62</b>  | 427  | <b>62</b>  |
| 2  | CAF1_5N | [21] | 0.23        | <b>146</b>    | 194      | 409           | <b>92</b>  | 564       | <b>234</b> | 409  | <b>180</b> | 434  | <b>190</b> |
| 3  | CSJ_RT  | [2]  | 0.51        | <b>74</b>     | 127      | 272           | <b>54</b>  | 136       | <b>49</b>  | 272  | <b>77</b>  | 272  | <b>77</b>  |
| 4  | ST_2NT  | [3]  | 0.14        | <b>92</b>     | 303      | 625           | <b>55</b>  | 943       | <b>133</b> | 625  | <b>137</b> | 631  | <b>138</b> |
| 5  | ST_3NT  |      | 0.13        | <b>92</b>     | 303      | 625           | <b>49</b>  | 962       | <b>131</b> | 625  | <b>114</b> | 632  | <b>115</b> |
| 6  | ST_4NTb |      | 0.12        | <b>179</b>    | 303      | 625           | <b>66</b>  | 1006      | <b>196</b> | 625  | <b>158</b> | 631  | <b>159</b> |
| 7  | ST_5NT  |      | 0.07        | <b>164</b>    | 303      | 625           | <b>54</b>  | 874       | <b>130</b> | 625  | <b>130</b> | 625  | <b>130</b> |
| 8  | FCS_1   | [17] | 0.24        | <b>121</b>    | 178      | 376           | <b>38</b>  | 467       | <b>101</b> | 376  | <b>91</b>  | 376  | <b>91</b>  |
| 9  | FCS_2   |      | 0.16        | <b>104</b>    | 178      | 376           | <b>29</b>  | 663       | <b>96</b>  | 376  | <b>70</b>  | 458  | <b>83</b>  |
| 10 | RC_1    | [19] | 0.21        | <b>240</b>    | 176      | 372           | <b>112</b> | 601       | <b>280</b> | 372  | <b>177</b> | 449  | <b>208</b> |
| 11 | RC_2    |      | 0.12        | <b>240</b>    | 176      | 372           | <b>73</b>  | 857       | <b>233</b> | 372  | <b>127</b> | 503  | <b>161</b> |
| 12 | RC_3    |      | 0.19        | <b>200</b>    | 176      | 372           | <b>74</b>  | 656       | <b>207</b> | 372  | <b>131</b> | 473  | <b>158</b> |
| 13 | J1      | [22] | 0.11        | <b>72</b>     | 167      | 354           | <b>31</b>  | 473       | <b>78</b>  | 354  | <b>95</b>  | 354  | <b>95</b>  |
| 14 | J2      |      | 0.11        | <b>73</b>     | 167      | 354           | <b>27</b>  | 474       | <b>78</b>  | 354  | <b>75</b>  | 354  | <b>75</b>  |
| 15 | J3      |      | 0.11        | <b>92</b>     | 167      | 354           | <b>27</b>  | 474       | <b>78</b>  | 354  | <b>75</b>  | 354  | <b>75</b>  |
| 16 | J4      |      | 0.23        | <b>134</b>    | 167      | 354           | <b>52</b>  | 327       | <b>97</b>  | 354  | <b>154</b> | 354  | <b>154</b> |
| 17 | J5      |      | 0.22        | <b>119</b>    | 167      | 354           | <b>45</b>  | 328       | <b>97</b>  | 354  | <b>120</b> | 354  | <b>120</b> |
| 18 | J6      |      | 0.22        | <b>133</b>    | 167      | 354           | <b>27</b>  | 328       | <b>97</b>  | 354  | <b>113</b> | 354  | <b>113</b> |
| 19 | J7      |      | 0.17        | <b>116</b>    | 167      | 354           | <b>19</b>  | 394       | <b>90</b>  | 354  | <b>91</b>  | 354  | <b>91</b>  |
| 20 | J8      |      | 0.34        | <b>106</b>    | 167      | 354           | <b>27</b>  | 230       | <b>102</b> | 354  | <b>146</b> | 354  | <b>146</b> |
| 21 | CH1     |      | 0.13        | <b>55</b>     | 210      | 440           | <b>17</b>  | 577       | <b>62</b>  | 440  | <b>56</b>  | 440  | <b>56</b>  |
| 22 | CH2     |      | 0.19        | <b>99</b>     | 210      | 440           | <b>40</b>  | 472       | <b>72</b>  | 440  | <b>108</b> | 440  | <b>108</b> |
| 23 | CH3     |      | 0.16        | <b>88</b>     | 210      | 440           | <b>29</b>  | 523       | <b>67</b>  | 440  | <b>69</b>  | 440  | <b>69</b>  |
| 24 | CL1     |      | 0.30        | <b>54</b>     | 210      | 440           | <b>44</b>  | 294       | <b>51</b>  | 440  | <b>109</b> | 440  | <b>109</b> |
| 25 | CL2     |      | 0.26        | <b>48</b>     | 210      | 440           | <b>30</b>  | 325       | <b>49</b>  | 440  | <b>70</b>  | 440  | <b>70</b>  |
| 26 | CL3     |      | 0.37        | <b>48</b>     | 210      | 440           | <b>42</b>  | 252       | <b>56</b>  | 440  | <b>90</b>  | 440  | <b>90</b>  |
| 27 | P1C     | [23] | 0.16        | <b>288</b>    | 174      | 367           | <b>109</b> | 732       | <b>369</b> | 367  | <b>204</b> | 492  | <b>266</b> |
| 28 | P2C     |      | 0.16        | <b>244</b>    | 174      | 367           | <b>75</b>  | 719       | <b>225</b> | 367  | <b>137</b> | 489  | <b>176</b> |
| 29 | P3C     |      | 0.17        | <b>277</b>    | 174      | 367           | <b>108</b> | 712       | <b>359</b> | 367  | <b>201</b> | 488  | <b>260</b> |
| 30 | P4C     |      | 0.17        | <b>169</b>    | 174      | 367           | <b>78</b>  | 710       | <b>232</b> | 367  | <b>140</b> | 487  | <b>179</b> |
| 31 | CCR-2   | [20] | 0.32        | <b>119</b>    | 106      | 246           | <b>62</b>  | 234       | <b>115</b> | 233  | <b>104</b> | 233  | <b>104</b> |
|    |         |      |             |               |          | Mean          | 2.65       |           | 1.02       |      | 1.08       |      | 1.01       |
|    |         |      |             |               |          | COV (%)       | 40.34      |           | 24.96      |      | 32.47      |      | 26.44      |

## **4. Damage fragility functions for FRP-jacketed circular columns**

### **4.1 DAMAGE STATE CLASSIFICATION**

The progression of damage in structural elements can be discretized in different levels, known as damage states. Each of these damage states is associated to specific repair activities required to restore the element to its pre-earthquake state. For example, FEMA P-58 [39] fragilities for RC moment frames are defined based on six different damage states which are related to damage mechanisms in RC beams, columns and beam-column joints. These damage mechanisms include:

1. Concrete cover cracking
2. Longitudinal and transverse steel reinforcement yielding
3. Concrete core crushing and spalling
4. Buckling or rupture of longitudinal reinforcement.

Damage states for a single element such as RC columns retrofitted with FRP can be defined in a similar way as for RC columns in those moment frames but considering the changes in behavior due to the FRP wrap. For instance, damage states related with spalling, and concrete core crushing, are generally prevented with sufficient FRP confinement, therefore, such type of failures can be ignored.

Longitudinal steel yielding can be considered to correspond to slight damage state (DS1) for RC columns retrofitted with FRP. Before reaching this damage state the member is essentially elastic, and this point marks the initiation of the plasticity mechanisms and the existence of flexural cracks with residual crack widths. Visible residual cracks at this phase can occur in the concrete near the base if a gap is left between the base and the start of the wrap, or beyond the retrofitted section if only a partial wrap is used. Repair of these cracks is required to protect the reinforcement for durability concerns.

At the other extreme of damage intensity, a severe damage state (DS3) is defined for FRP rupture, which is the most typical failure mode observed in the tests that conform the database presented in section 2. In some cases, this failure occurred together with buckling of the longitudinal bar [3], [21], condition that made difficult to determine whether one failure caused the other or vice versa. FRP rupture was often localized at a certain distance from the base, attributed to the additional confinement effect provided by the base. A gradual degradation of the lateral strength with increasing drift demands was observed at this stage; however, specimens were still able to resist a residual portion of the lateral load when tests were terminated. The damage state corresponding to the failure of the FRP would require full reparation of the element, including the removal and replacement of the existing wrap, crushed concrete, and buckled steel.

An intermediate damage state (DS2) is also defined based on moderate FRP damage. Some authors have reported a change in color in the FRP, attributed to “separation of the FRP fabric from concrete along the circumference [3]”, or explained by the lack of fibers in the longitudinal direction. None of the tests included fibers in the longitudinal direction, which is also typical for FRP applications in the field. Some other authors [2], [9], [15] describe “fine horizontal cracks” appearing in cycles succeeding longitudinal bar yielding. This discoloration or horizontal cracks in the FRP are classified in an intermediate damage state between bar yielding and FRP rupture, as a physical indicator that the FRP is not functioning as expected and may have compromised the total capacity of the retrofitted section. Hence, replacement of the wrap is recommended.

Based on these considerations, the three damage states proposed for circular RC columns retrofitted with FRP are summarized in Table 3.3, together with the repairing activities proposed for each damage states. Figure 3.9 shows visual examples of the specimens that reach DS2 [a) and b)] and specimens at DS3 [c) and d)].

Table 3.3. Damage States for Circular RC Columns retrofitted with FRP

| Damage State   | Damage characteristics                                              | Repair activity                                              |
|----------------|---------------------------------------------------------------------|--------------------------------------------------------------|
| DS1 - Slight   | Residual flexural cracks/ longitudinal reinforcement yielding       | None/Epoxy Inject Concrete                                   |
| DS2 - Moderate | FRP damage (discoloration/ cracking)                                | Replace FRP                                                  |
| DS3 - Severe   | FRP Rupture, concrete crushing, possibly steel buckling or fracture | Replace Reinforcing Steel, replace concrete and replace FRP. |

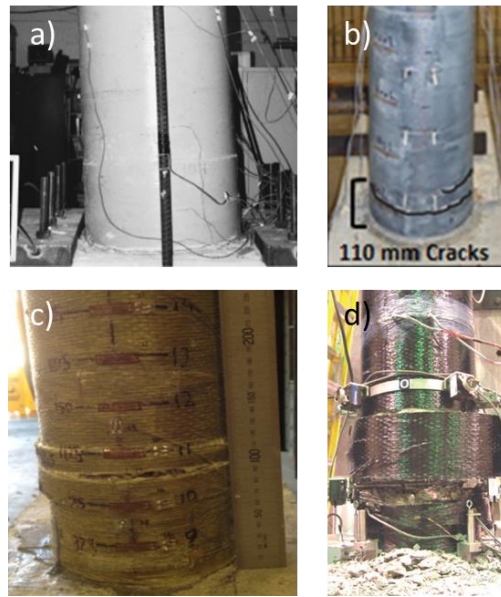


Figure 3.9. Physical appearance of DS2 (above) and DS3 (below). a) to d) from refs. [2], [14], [11], [10], respectively

## 4.2 STRUCTURAL DEMAND PARAMETERS

Two types of fragility curves are developed using two different types of parameters representing demands in a structure. For columns and other structural components, it is common to use the lateral drift ratio (or story-drift ratio in multi-story structures), defined as the lateral displacement divided by the height of the element (or story).

$$\delta = \Delta/H \quad (20)$$

The first type of fragility functions, Group A, have been derived using story-drift ratio as demand parameter. The drift ratio can be computed using Eq. [20] for both the cantilever and fixed-fixed configurations of the columns in the database. The advantage of using story-drift ratio is that it can be directly obtained from structural analysis. However, this demand parameter does not take into account the different deformation capacity of elements presenting different designs (aside from the height) and loading conditions.

The second type of fragilities, Group B, are derived using a normalized drift demand as demand parameter. This parameter normalized the story-drift ratio demand by the theoretical value of the ultimate drift ratio as follows:

$$\delta' = \frac{\delta_{me}}{\delta_u} = \frac{\Delta_{me}/H}{\Delta_u/H} = \frac{\Delta_{me}}{\Delta_u} \quad (21)$$

where  $\Delta_{me}$  is the measured displacement (demand) and  $\Delta_u$  is the theoretical ultimate displacement of the column calculated using the plastic hinge model and newly proposed equations for ultimate curvature and effective plastic hinge length in Section 3.3 (Equations (2), (3), (18), and (19)). The normalization by  $\Delta_u$  allows the demand parameter to implicitly take into account design and loading characteristics, such as amount of longitudinal and transverse reinforcement or axial load, which affect the lateral response of the column.

### 4.3 DEVELOPMENT OF FRAGILITY FUNCTIONS

Fragility curves were developed based on experimental data in accordance with the procedures described in FEMA-P58 [25]. The fragility function is assumed to take the form of a lognormal distribution defined as:

$$F_i(D) = \Phi \left( \frac{\ln \left( \frac{D}{\theta_i} \right)}{\beta_i} \right) \quad (22)$$

where  $\Phi$  is the standard normal (Gaussian) cumulative distribution function,  $D$  is the demand parameter,  $\theta$  is the median value of the probability distribution and  $\beta$  is the logarithmic standard deviation. The total dispersion  $\beta$  is computed as:  $\beta = \sqrt{\beta_r^2 + \beta_u^2}$ , where  $\beta_r$ , represents the random variability of the test data; and  $\beta_u$  accounts for the uncertainty that the test configuration represents actual conditions. For all three damage states, sufficient test data was available, and  $\beta_u=0.1$  was used based on the values recommended in FEMA-P58 [25].

The two groups of fragility curves were derived using actual demand measurements obtained from the tests considered in the database presented in Section 2. For DS1, the demand values were those at which the first yielding of longitudinal steel was explicitly reported. A total of 32 elements were collected for these fragility curves. For DS2, the population consists of 20 points from the tests in which the change in color or fine horizontal cracks in the FRP were explicitly indicated. For simplification, some authors described the observations of only one representative specimen when the individual behavior of a group of columns was similar. In such cases, the reported deformation was used for the set of tests within the alike behavior group. Finally, the points for DS3 were collected from the measurements obtained at ultimate conditions for those 31 tests that failed due to FRP rupture.

Group A fragility curves using story-drift ratio as demand parameter are shown in Figure 3.10. Likewise, Figure 3.11 shows Group B fragility curves using normalized story-drift demand. Both empirical and adjusted lognormal functions using  $\beta_r$  only are plotted.

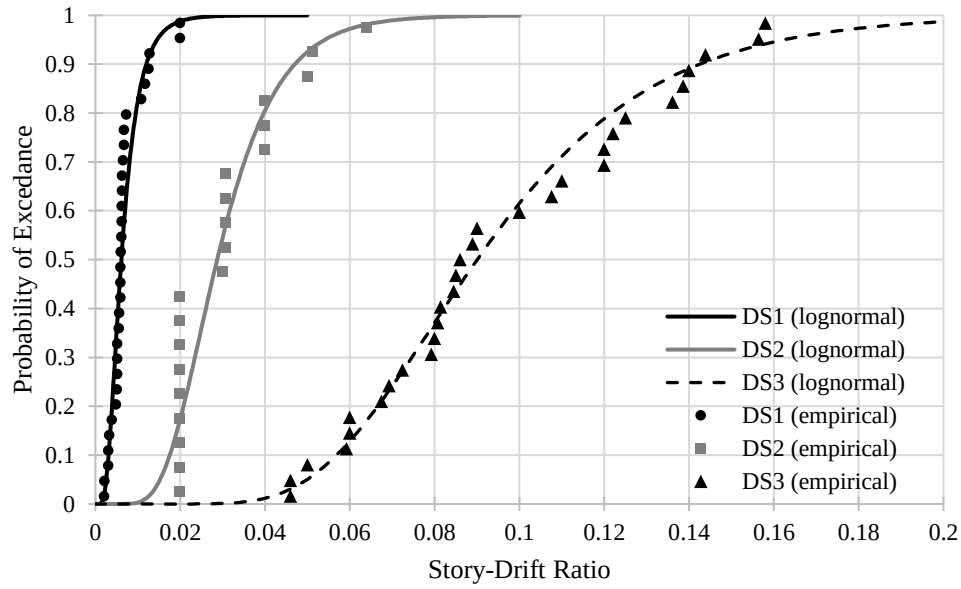


Figure 3.10. Group A Fragility curves for circular RC columns retrofitted with FRP

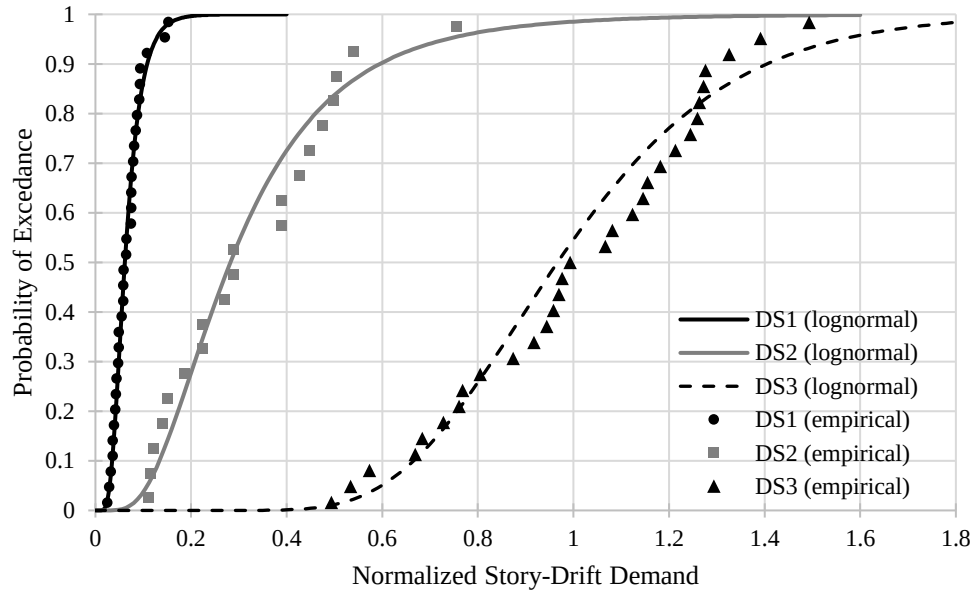


Figure 3.11. Group B Fragility curves for circular RC columns retrofitted with FRP

Goodness-of-fit for the adjusted lognormal functions were calculated using the Lilliefors method described in FEMA-P58 [25]. For Group A, only DS3 lognormal functions pass the goodness-of-fit test at 5% significance, DS1 and DS2 fail with just few empirical points deviating excessively from the derived fragility. As for fragility functions in group B, the three curves pass the goodness-of-fit test at 5% significance.

Table 3.4 summarizes the values of median and dispersion for both groups of fragility curves. Both types of fragility curves show a similar behavior in terms of distribution of median and dispersion for each of the damage states. DS1 occurs at very early stages with a median of 0.6% story-drift ratio, which corresponds to an average of 6% of the ultimate story-drift ratio. DS2 corresponding to the discoloration and fine horizontal cracks occurs at a median story-drift ratio of 2.8%, which is around 28% of the ultimate story-drift ratio. This level of story-drift is slightly above the maximum drift allowed by ASCE 7-16 for RC buildings with four stories or less and of risk category I or

II (2.5%). At this level of drift, the probability of DS2 being exceeded is around 36%. This implies that if the building is properly design to such limit, the probability of exceeding DS2 will not be high. Even with the degradation in the stiffness due to cyclic loads, it is unlikely that the structure would observe higher story-drifts than those for DS2. However, there is still a 5% probability that DS2 will be exceeded at a drift of 1.5%. DS3 occurs at a median of 9% story-drift ratio, and a 2% probability that DS3 is exceeded will occur at a drift of about 4.2%. These values show how much ductility is gained with an FRP retrofiting system, however, in a real building, these extreme story-drift ratios are very unlikely to preclude other failure mechanisms in the rest of structural elements.

In both groups, DS1 shows the largest dispersion with 52.69% and 43.97% for group A and group B, respectively. DS2 shows larger dispersion in group B than group A fragility curves, going from to 38.49% in the latter to 58.17% in the former. DS3 shows a smaller dispersion in group B than group A, with 29.14% and 35.85%, respectively.

Table 3.4. Median  $\theta$  and dispersion  $\beta_r$  of fragility curves

|     | Group A  |           | Group B  |           |
|-----|----------|-----------|----------|-----------|
|     | $\theta$ | $\beta_r$ | $\theta$ | $\beta_r$ |
| DS1 | 0.0061   | 52.69%    | 0.0615   | 43.97%    |
| DS2 | 0.0287   | 38.49%    | 0.2822   | 58.17%    |
| DS3 | 0.0900   | 35.85%    | 0.9670   | 29.14%    |

## 5. Conclusions

A study on the lateral deformation response and damageability of circular RC columns retrofitted with FRP has been presented. The study has been conducted using a database of 54 column tests collected from available literature.

The following conclusions can be drawn from this research.

1. The predictions of the ultimate displacement using the equations according to ACI 440.2R-17 [18] tend to underestimate the lateral deformation capacity of RC columns retrofitted with FRP. To apply such approach, moment curvature needs to be performed to obtain the ultimate curvature. The mean of the ratio of  $\Delta_{u\_me}/\Delta_{u\_calc}$  is 2.6. This is so because ACI 440.2R-17 [18] considers a concentric concrete model that limits the maximum strain in the concrete, reducing the ultimate curvature of the section and thus, the plastic deformation of the section.
2. Gu et al. [20] equations for lateral deformation capacity are simple and accurate since they include analytical equations to evaluate ultimate curvature (moment-curvature analysis is not required) and they result in a mean of the ratio  $\Delta_{u\_me}/\Delta_{u\_calc}$  equal to 1.01 with a COV of 25%.
3. The proposed equation for analytical ultimate curvature (Eq. 18) provides a good agreement with the results from moment-curvature analysis, with a mean of the ratio  $\phi_{u,mom.curv}/\phi_{u,eq18}$  equal to 1.0. This equation together with a new unified expression for plastic hinge length as a function of the confinement ratio, results in an estimate of the lateral deformation of RC columns with  $\Delta_{u\_me}/\Delta_{u\_calc}$  equal to 1.0 and with a COV of 26%, comparable with the results from Gu et al. The advantage of the proposed approach is that the confinement

effect provided by the FRP is taken into account in a single expression, which is also compatible with unretrofitted columns.

4. The proposed sets of fragility functions include three damage states based on measurements on tests: first yield of longitudinal reinforcement, discoloration or fine horizontal cracks in the FRP wrap; and the failure in FRP which may occur simultaneously with reinforcement buckling. The use of the normalized story-drift ratio as a demand parameter in Group B fragility curves, instead of the story-drift ratio used in Group A fragility curves, reduced the dispersion for DS1 and DS3. Goodness of fit also improved in Group B fragility curves.

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## **Chapter 4. Conclusions**

Key findings from the seismic rehabilitation of RC buildings in California include:

- To obtain minimum levels of safety, it is important to mandate not only on new structures but also on existing building infrastructure. Seismic retrofit plans require several technical and economic resources; hence, a progressive long-term plan can be applied to meet the goals of building regulations. Such is the example of the state of California with Senate Bill 1953 (SB 1953).
- Seismic retrofits included work on increasing the capacity of the existing foundation. Addition of piles or enlargement of footings are among the techniques used.
- Seismic analysis usually resulted in higher seismic forces, for which higher strength was required. Construction of new RC walls, or widening of existing RC walls was a common technique to provide strength. Localized strengthening was achieved using fiber reinforced polymers (FRP).
- Floor diaphragms are an important part of the structure to transmit inertial forces to the lateral force resisting system. Two methods for increasing the capacity of diaphragms were seen, using FRP in concrete slabs and adding collectors formed by steel elements.
- Non-structural elements can cause significant economic loss. Some seismic retrofits of buildings in the case studies include works on non-structural elements consisting of bracing and bolting equipment.

Key findings from the lateral deformation capacity and damage fragility assessment include:

- Deformation capacity computed according to ACI 440.2R-17 underestimate the capacity of circular RC columns retrofitted with FRP.
- The simplified equation proposed in this study to compute the ultimate curvature produce results comparable with results from moment-curvature analysis.
- Lateral deformation capacity evaluated with the proposed ultimate curvature and plastic hinge length equations are in good agreement with results from tests reported in the literature.
- The use of the story-drift ratio normalized to the theoretical ultimate deformation capacity as demand parameter decreased the dispersion of fragility curves in two of the three damage states. Goodness of fit check also improved with the normalization.

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