Solid Mechanics and Its Applications

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Computer Aided Seismic and Fire Retrofitting Analysis of Existing High Rise Reinforced Concrete Buildings

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Solid Mechanics and Its Applications

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Synopsis of the Book

This book aims at detailed analysis and structural design of high rise buildings with real-life examples. The knowledge of design of high buildings for gravity and seismic loading is highly essential for upcoming engineers to work in the actual field efficiently. The various structural analysis and design aspects for high rise buildings are taught in the undergraduate level but not in full depth. Therefore, the objective of this book is to understand the analysis and design of high rise buildings for gravity and seismic analysis. Since designing for gravity and seismic is usual in the actual field but retrofitting of high rise building is less common, it is highly important for study. In this report, retrofitting techniques are also proposed for high rise buildings taking into account the gravity and seismic loads. The understanding of current software for design is the need of the hour, especially for design structural engineers. Therefore, ETABS was used for analysis of gravity and for earthquake loads. The gravity analysis and design is based on ACI-99 and earthquake analysis is based on UBC-97. Furthermore, FRP retrofitting and fire retrofitting are also discussed in detail by manual approach.

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Preface

Earthquakes have long been feared as one of nature's most terrifying phenomena due to its destruction power. Without any prior warning, the earthquake can and does, in a few seconds, create a level of death and destruction that can only be equaled by the most extreme weapons of war. The earthquake damage is almost entirely associated with man-made structures such as buildings, dams, bridges, etc.

In the current context, prevention of disasters caused by earthquake has become significantly important. Disaster prevention includes the reduction of seismic risk through retrofitting existing buildings in order to meet seismic safety requirements. To make alterations to existing buildings in comparison to new construction, the planning has to be different. The existing construction must be taken as the basis for all planning and building actions.

In developed countries, the codes were changed and new codes were prepared, which contain the provision of lateral loads. The new structures can be built sufficiently earthquake resistant by adopting these new design methodologies and controlling the construction quality at a higher level. However, existing old structures, which have mostly been planned without considering this important aspect, stands at enormous seismic risk.

The objective of this book is to understand the analysis and design of high-rise building for gravity and seismic analysis. The designing of structure for gravity and seismic is usual in the real field. However, retrofitting of high-rise building is less common, though it is very important to study. In this analysis, retrofitting techniques are also proposed for high-rise buildings taking into account the gravity and seismic loads. The understanding of the current software for design is the need of the hour, especially for design structural engineers. In this analysis ETABS was used for gravity and earthquake loads. The gravity analysis and design is based on ACI-99 and earthquake analysis is based on UBC-97. Furthermore, FRP retrofitting and fire retrofitting is also discussed in detail by manual approach.

This book covers all features of assessment of the retrofitting of existing structures, and the authors of the book believe that it will be of immense use to the student community, academicians, consultants, practicing professional engineers/scientists involved in planning, design, execution, inspection and supervision for proper retrofitting of buildings.

The authors will be pleased to hear from the readers with positive feedback to improve the book quality, and also to erase the errors and misprints. This will be highly regarded and will be acknowledged accordingly.

Contents

Notations

ACI	American Concrete Institute
AC/FC	Air condition/False ceiling
AISC	American Institute of Steel Construction
Ac	Actual concrete to be provided in the jacket
As	Actual steel to be provided in the jacket
Ac′	Concrete values obtained for jacket after deducting the existing concrete
Ag	Area gross
As′	Steel values obtained for the jacket after deducting the existing steel
ASTM	American Society for Testing of Material
A_g	Gross area of cross-section
A_S	Area of steel
A_{nt}	Net area subjected to tension
A_{nv}	Net area subjected to shear
b	Width of compression zone at extreme fibers
b_f	Width of flange
b_o	Perimeter of critical section of punching shear
Ca, Cv	Seismic coefficient
CAPO	Cut And Pull Out
C_v	Numerical coefficient dependent upon moment gradient
C_w	Warping constant
d	Effective depth
dh	Diameter of stirrup
D.L	Dead load
e	Eccentricity
E	Modulus of Elasticity
E_c	Modulus of Elasticity of concrete
E_{cc}	E_c of internal core of column
E_{cj}	E_c of jacketed concrete of column
fck	Cube strength of concrete

ETABs	Extended 3D Analysis of Building System
f′c	Compressive strength of concrete
f_y	Yield strength of steel
F′c	Compressive strength of concrete
FRP	Fiber-Reinforced Polymers
f'_c	Compressive strength of concrete
F_r	Compressive residual stress in flange
F_u	Tensile strength
F_y	Yield strength
f_{frpu}	Ultimate FRP strength
G	Shear Modulus
h	Height of web
h_n	Height above the base to level “n”
h n	Height above the base to level n
I	Importance factor
Im	Moment of Inertia
J	Torsional constant
L	Length of beam
L.L	Live load
L_b	Unbraced Length
L_p	Limiting laterally unbraced length for full plastic flexural strength
L_r	Limiting laterally unbraced length for inelastic lateral torsional buckling
LTB	Lateral Torsional Buckling
M	Moment
M_n	Nominal Ultimate moment
M_p	Plastic bending moment
M_r	Limiting buckling moment
Mu2	Minor axis moment
Mu3	Major axis moment
n	Number of Stories
n	Transformation factor
NDE	Non-Destructive Evaluation
N_g	Number of FRP layers
P	Axial load
PI	Plasticity index
Pu	Ultimate load
pn	Nominal load
Po	Load with no eccentricity
Q	Static moment about the neutral axis of a portion of section through a line parallel to the neutral axis
R	Ductility and over-strength coefficient
R	Factor that accounts for the ductility and over strength of the structural system
R_n	Block Shear design strength

r_y	Radius of gyration about y-axis
S	Spacing
S	Elastic Section Modulus
SA, SB, SC, SD, SE, SF	Soil profile types
t	Time period
T	Torsion
t_f	Thickness of flange
t_w	Thickness of web
tj	Thickness of jacket
t_{frp}	FRP thickness
UBC	Uniform Building Code
V2	Major axis shear
V3	Minor axis shear
Vc	Shear in concrete
Vs	Shear in steel
Vf	Shear in FRP
Vx	Story shear
v	Shear resistance
V	Base Shear
V_u	Factored Shear Force
w	Weight of structure
W	Seismic dead load
W_u	Factored Load
X_1 and X_2	Beam buckling factors defined by LRFD specification equations F1-8, F1-9
z	Plastic Section Modulus
Z	Seismic zone factor
γ	Radius of gyration
μ	Coefficient of friction
$\varnothing$	Capacity reduction factor
Ø	Resistance factor
Δ	Deflection
$\varnothing_{frp}$	Concrete strength
α_{pc}	Performance coefficient

Chapter 1
Introduction

Abstract An earthquake is a sudden and transient motion of the earth's surface, which is believed to be happening since the beginning of the earth. Earthquake has caused a significant amount of loss in lives and infrastructure, which is a predominant concern for the engineers in many cities of the world. The earthquake that took place earlier century in different cities of Japan and in the world, led to the significant changes in the practice of seismic design, particularly in the high risk seismic regions. Moreover, it also triggered changes in the codes in developed countries and the new codes were prepared with the provision of lateral loads.

1.1 General

The earthquake at California and in different cities of Japan led to the significant changes in the practice of seismic design, particularly in the high risk seismic regions. After disastrous earthquakes, engineers realized that lateral loads should be considered in the design in order to remove destruction and loss of lives. In developed countries the codes were changed and the new codes were prepared which contains the provision of lateral loads. Engineers also worked and came up with the solution for the already existing buildings in the form of retrofitting techniques. These may include energy dissipation techniques (dampers or base isolators), column jacketing, use of carbon fiber reinforced plastic, fiber reinforced polymers etc. But in under developing countries still the buildings are designed for gravity loads only either because of lack of awareness or negligence by the concerned legislation.

The earthquakes had clearly demonstrated that not only non-engineered rural houses are vulnerable to earthquakes; the engineered multistoried buildings in big cities are also mostly vulnerable due to faulty design and construction. Considering the large number of people and high fatality in RC buildings; the seismic retrofitting of the existing buildings has to be undertaken to make these unsafe buildings safe to resist future earthquakes, thereby reducing the number of casualties significantly.

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1.2 Damages to Reinforced Concrete Structures and Lives in Past Earthquake

Earthquakes have not only claimed hundreds of thousands of lives in the last 100 years but also numerous reinforced buildings were totally collapsed by these earthquakes and improvements in technology have only slightly reduced the death toll and other infrastructures damages. If we look at the past history we can find that in so many places of the world, there were catastrophic impacts of the earthquakes. San Francisco was hit by a series of violent shocks which lasted up to a minute. Between 700 and 3,000 people died either from collapsing buildings or in the subsequent fire on 18 April 1906 (BBC 2013). The Great Kanto earthquake, with its epicentre just outside Tokyo, claimed the lives of 142,800 people in the Japanese capital on 1 September, 1923 (BBC 2013). Up to 10,000 people were killed in the Nicaraguan capital Managua by an earthquake that measured 6.5 on the Richter scale. The devastation caused by the earthquake was blamed on badly built high-rise buildings that easily collapsed, 23 December 1972 (BBC 2013). Mexico City was shaken by a huge earthquake which razed buildings and kills 10,000 people, 19 September 1985 (BBC 2013). An earthquake measuring 6.9 on the Richter scale devastated north-west Armenia, killing 25,000 people, 7 December 1988 (BBC 2013). Italy was traumatised by the loss of an entire class of children, killed in the southern village of San Giuliano di Puglia when their school building collapsed on them, 31 October 2002. Up to 300 people were killed in the Pakistani province of Balochistan after an earthquake of 6.4 magnitude struck 70 km (45 miles) north of Quetta, 29 October 2008. The most recent earthquake was on 12 January, 2010 about 230,000 die in and around the Haitian capital Port-au-Prince, as a 7.0 magnitude earthquake struck the city. These all damages necessitate the more detailed understanding of the Seismic and Retrofitting Analysis for the new structural engineers.

Most of the old existing buildings were designed for gravity loads, as there were no provisions for lateral loads earlier. The buildings in which seismic loads are not incorporated in their design are obviously more vulnerable to earthquake failures. The most common weaknesses are the discontinuity of the longitudinal reinforcement in beams and columns, beam and column failures, shear cracking in shear walls, and failures of beam-column and slab-column connections. Sometimes foundations have to be strengthened for Seismic loads. Columns are critical elements in any structural system and their performance during a seismic event can dominate the overall performance of the structure.

1.3 Objectives

This book is intended to serve the upcoming Structural design engineers to get them familiarize with the computer aided analysis and design of real existing high rise buildings by using ACI code for application of the gravity loads, UBC- 97 for seismic analysis and retrofitting analysis by computer models. The verification of these computer models is also mentioned in this book in detail by doing manual calculations using common reinforced concrete structural analysis, the objective is to build good analytic skills in upcoming engineers so that once they come in the field they could have the good knowledge of computer aided design of high rise buildings.

1.4 Scope and Limitation

The scope of this book limits the use of ETABS for the design and evaluation of existing structure. The theme is to evaluate performance of the building in the presence of seismic loads and applying retrofitting techniques. In the later chapters of this fire retrofitting is also introduced and some manual calculations are done to help building good understanding of the fire retrofitting as well which is nowadays an issue of concern for most of the consultants and researchers. The techniques considered for retrofitting in details employing software are:

1. Proposal of new shear wall panels in between existing columns.
2. Proposal of Bracings in between existing columns.
3. Proposal of Jacketing (CFRP (carbon fiber reinforced plastic) /FRP (fiber reinforced polymer and R.C.C column jacketing).
4. Strengthening by FRP of beams and columns.
5. Examples of calculations for fire Retrofitting.

The buildings are analyzed for zone 2B and UBC 97 code is used for seismic evaluations with the aid of computer software.

1.5 Methodology

The structure will be studied in detail using ETABS (Extended three dimensional analysis of building system) software which is universally accepted best software for the analysis of high rise buildings. Lateral load will be applied to these structures using ETABS. Those building which are deficient in resisting lateral loads will be retrofitted by suitable techniques which will be studied in detail in upcoming chapters.

15 Story High Rise Existing Building is considered for seismic study in this book. The building software model is first designed for gravity loads considering the actual existing building designed condition. The building is then analyzed for lateral loads to study the earthquake impacts. For Seismic analysis building is put in zone 2B as per UBC 97 for the analysis. The members which are deficit in shear, flexure and biaxial moment; member level and structural level techniques are applied for retrofitting to make the structure safe.

Beside these analyses of the ETABS models, strengthening by FRP is discussed in detail with examples to make this technique understandable for the new structural engineers. Furthermore to make structural engineers familiar with the Fire Retrofitting of the high rise building some important manual calculations are also mentioned in this book.

Reference

BBC (2013) History of deadly earthquakes. http://www.bbc.com/news/world-12717980. Accessed Feb 2015

Chapter 2
Related Reviews

Abstract The modification of existing structures to enhance the resistant to seismic activity, ground motion or soil failure due to earthquakes is called Seismic Retrofitting. The force that causes the damage of the structure due to earthquake is termed as F-Inertial which is the internal force generated by the weight of the structure. Several factors are associated for the design of a structures to be seismic resistant such as torsion, ductility and so on. In this chapter a brief overview of many new technologies that are rapidly becoming more prevalent in the seismic design of building structures are presented. Retrofitting seismically deficient structures to reach an acceptable level of performance is highly regarded issues in the current context. To achieve this, methodology such as reducing the load effect input to the existing structures, or by improving the strength, stiffness, and/or ductility of the existing structures can be taken into considerations.

2.1 Definition of Retrofitting

Seismic Retrofitting is the modification of existing structures to make them more resistant to seismic activity, ground motion or soil failure due to earthquakes. Retrofitting techniques are applied when there is a danger of damage to the structure by some catastrophe. For example; if a bridge is being damaged by any disaster and maintenance is required to make up the structure for its existing capacity then this is re-strengthening as the design remains the same and it is only repaired. If the structure is designed in the year 2005 and in year 2010 it is required to increase the load carrying capacity of that structure in order to meet the demands, then this is retrofitting. Retrofitting techniques are applied where we have to increase the capacity of the reinforced concrete structure.

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2.1.1 *Background*

The seismic measures are used to calculate forces that earthquakes impose on buildings. Ground shaking (pushing back and forth, sideways, up and down) generates internal forces within buildings called the Inertial Force (F-Inertial), which in turn causes most seismic damage.

$$\text{F-Inertial} = \text{Mass (M)} \times \text{Acceleration (A)}$$

The greater the mass (weight of the building), the greater the internal inertial forces generated. Lightweight construction with less mass is typically an advantage in seismic design. Greater mass generates greater lateral forces, thereby increasing the possibility of columns being displaced, out of plumb, and/or buckling under vertical load (P delta Effect).

2.1.2 *Seismic Design Factors*

The following factors affect the design of the building. It is important that the design team understands these factors and deal with them prudently in the design phase.

Torsion
When the centre of rigidity and centre of mass do not coincide with each other the torsion is produced in the structure. Torsion is produced when the earthquake resistive vertical members are non-uniformly distributed in plan.

Ductility
The earthquake forces develop greatly depend upon the type of lateral load resisting system provided in a building which is also the measure of ductility present in the structure. Good ductility can be achieved with carefully detailed beam column joints in R.C.C.

Building Configuration
This term defines a building's size and shape, and structural and nonstructural elements. Building configuration determines the way seismic forces are distributed within the structure, their relative magnitude, and problematic design concerns (Fig. 1.1).

Redundancy
The ability of a structure to redistribute seismic loads to other elements, if a particular element fails or is damaged. If a particular element of a building's lateral load resisting system should fail or be damaged during an earthquake, it is very desirable that the seismic load be redistributed to other elements and thereby avoids collapse. This is called redundancy.

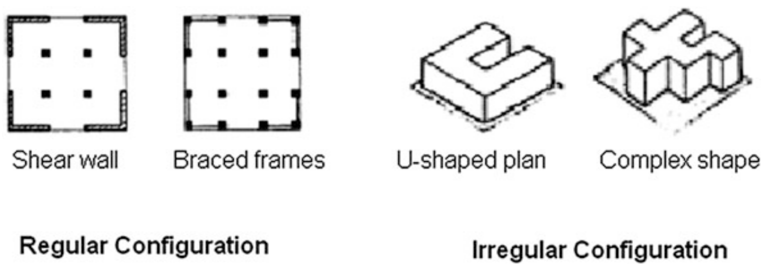

Fig. 1 Regular and irregular configuration of a building

Knowledge of the building's period, torsion, damping, ductility, strength, stiffness, and configuration can help one determine the most appropriate seismic design devices and mitigation strategies to employ.

2.2 Retrofitting Techniques

The purpose of this chapter is to provide a brief overview of many new technologies that are rapidly becoming more prevalent in the seismic design of building structures, and to provide guidance for the consideration and evaluation of the use of these systems in selected buildings. Recent earthquakes in all parts of the world have highlighted the urgency and importance of retrofitting seismically deficient structures to achieve an acceptable level of performance. This can be achieved, in part, by reducing the load effect input to the existing structures. Or by improving the strength, stiffness, and/or ductility of the existing structures. Over the past 20 years, significant advancements have been made in the research and development of innovative materials and technologies for improving the seismic performance of existing structures through rehabilitation processes.

2.2.1 Structure System-Level Rehabilitation

Structure system-level rehabilitations are commonly used to reduce lateral drift and decrease ductility requirement and to enhance the lateral resistance of existing structures. Such rehabilitations for RC buildings include adding infill walls, steel braces, post-tensioned cables, steel plate shear walls, and base isolators. The methods described below are commonly used when implementing a structure system-level rehabilitation technique. The studies show that there is no unique solution and that several difference retrofit schemes can be designed to provide effective seismic resistance. Satisfactory response was obtained only for schemes that adequately controlled inter-story lateral drifts.

Structure System-Level Rehabilitation Include the following Techniques

- Supplementary Energy Dissipation
- Addition of RC Structural Walls
- Addition of Steel Plate Shear Wall
- Use of Steel Bracing
- Ferrocement

2.2.1.1 Seismic Isolation and Energy Dissipation Systems

Seismic isolation and energy dissipation systems involve the use of special details or specific device to alter or control the dynamic behaviour of buildings. The structural systems that utilize these technologies can be broadly categorized as passive, active, or hybrid control systems. Definitions of these terms are provided below, although the primary focus is on passive control systems. Additional guidelines and design provisions for base isolation systems are provided in FEMA 302. Similar guidance for energy dissipation systems is provided in FEMA 273. The definitions of the passive, active and hybrid control system are mentioned below.

Passive Control Systems

These systems are designed to dissipate a large portion of the earthquake input energy in specialized devices or special connection details that deform and yield during an earthquake. Since the deformation and yielding are concentrated in the device, damage to other elements of the building may be reduced. These systems are passive in that they do not require any additional energy source to operate, and are activated by the earthquake input motion. Seismic isolation and passive energy dissipation are both examples of passive control systems. It is interesting to note that many of these devices can be used at the base of a structure as part of an isolation system or in combination with braced frames or walls as energy dissipation devices.

(a) Seismic isolation systems: The objective of these systems is to decouple the building structure from the damaging components of the earthquake input motion, i.e., to prevent the superstructure of the building from absorbing the earthquake energy. The entire superstructure must be supported on discrete isolators whose dynamic characteristics are chosen to uncouple the ground motion. Some isolators are also designed to add substantial damping. Displacement and yielding are concentrated at the level of the isolation devices, and the superstructure behaves very much like a rigid body.

(b) Passive energy dissipation systems: The objective of these systems is to provide supplemental damping in order to significantly reduce structural response to earthquake motions. This may involve the addition of viscous damping through the use of viscoelastic dampers, hydraulic devices or lead extrusion systems; or the addition of hysteretic damping through the use of friction-slip devices, metallic yielding devices, or shape-memory alloy devices. Using these systems, a building will dissipate a large portion of the earthquake energy through inelastic deformations or friction concentrated in the energy dissipation devices, thereby protecting other structural elements from damage.

Active Control Systems

These systems provide seismic protection by imposing forces on a structure that counter-balance the earthquake-induced forces. These systems are active in that they require an energy source and computer-controlled actuators to operate special braces or tuned-mass dampers located throughout the building. Active systems are more complex than passive systems, since they rely on computer control, motion sensors, feedback mechanisms, and moving parts that may require service or maintenance. In addition, these systems need an emergency power source to ensure that they will operate during a major earthquake and any immediate aftershocks.

2.2.1.2 Hybrid Control Systems

These systems combine features of both passive and active control systems. In general, they have reduced power demands, improved reliability, and reduced cost when compared to fully active systems. In the future, these systems may include variable friction dampers, variable viscous dampers, and semi-active isolation bearings.

It is important to note that the passive energy dissipation systems described above are new technologies when applied to civil engineering structures, but have been used in mechanical engineering for many years. There are numerous situations where dampers, springs, torsion bars, or elastomeric bearings have been used to control vibration or alter the dynamic behavior of mechanical systems. Several examples include vehicular shock absorbers, spring mounts that provide vertical vibration isolation for mechanical equipment, and hydraulic damping devices that utilize fluid flow through an orifice to provide shock isolation for military hardware. Many of these devices have been in use for decades and have performed well in situations where they are subjected to millions of cycles of loading; many more than would be required for seismic resistance. The immediate challenge is therefore not to develop new technologies, but to develop guidelines that will enable us to adapt existing technologies to civil/structural engineering applications.

2.2.1.3 Addition of RC Structural Walls

Adding structural walls is one of the most conventional structure system-level rehabilitation methods to strengthen existing structures. This approach is effective in controlling global lateral drifts and in reducing damage in frame members. Generally, repair of an existing. In order to save time, cost and ensure the quality, panels can be prefabricated and assembled in-site. Many researches into structural walls have been conducted, and findings corresponding to detailed rehabilitation have been reported. Addition of new shear walls will normally result in better seismic performance, however, its side-effect on the existing structure and its relatively intrusive and disrupt construction style are disadvantages. Previous research shows that the infilling process tends to stiffen the existing structure such that the base shear can increase accordingly. Along with the addition of the weight, in turn the gravity load effect, gives more pressure to the existing foundation. Therefore, strengthening of the existing foundation is usually required when this technique is used, while commonly upgrading existing foundations under the new walls is expensive.

2.2.1.4 Use of Steel Bracing

Steel moment-resisting frames are susceptible to large lateral displacements during severe earthquake ground motions, and require special attention to limit damage to non-structural elements as well as to avoid problems associated with P-D effects and brittle or ductile fracture of beam to column connections (FEMA 2000). As a consequence, engineers in the US have increasingly turned to concentrically braced steel frames as an economical means for resisting earthquake loads. However, damage to concentrically braced frames in past earthquakes, such as the 1985 Mexico (Osteraas 1989), 1989 Loma Prieta (Kim 1992), 1994 Northridge (Tremblay 1995; Krawinkler 1996), and 1995 Hyogo-ken Nanbu (Hisatoku 1995) earthquakes, raises concerns about the ultimate deformation capacity of this class of structure. Individual braces often possess only limited ductility capacity under cyclic loading (Tang 1989). Brace hysteretic behavior is unsymmetric in tension and compression, and typically exhibits substantial strength deterioration when loaded monotonically in compression or cyclically. Because of this complex behavior, actual distributions of internal forces and deformations often differ substantially from those predicted using conventional design methods (see, for example, Jain 1979 and Khatib 1987). Design simplifications and practical considerations often result in the braces selected for some stories being far stronger than required, while braces in other stories have capacities very close to design targets. This variation in story capacity, together with potential strength losses when some braces buckle prior to others, tend to concentrate earthquake damage a few "weak" stories. Such damage concentrations place even greater burdens on the limited ductility capacities of conventional braces and their connections. It has also been noted that lateral buckling of braces may cause substantial damage to adjacent nonstructural elements.

2.2.1.5 Ferrocement

Ferro-cement is a thin wall type composite; having a total thickness ranging between 12 to 30 mm. It is composed of hydraulic cement mortar reinforced with a minimum two layers of continuous and relatively small diameter orthogonally woven wire mesh separated by 4–6 mm dip galvanized spacer wires. The cement mortar is admixed with plasticizers and polymers of sealing pores. The wire mesh is mechanically connected to the parent surface by U-shaped nails fixed with suitable epoxy bonding system. The mesh may be made up of hot dip galvanized MS wire or some other metallic or suitable material. Special technique for compacting Ferro-cement layer is used with the help of orbital vibrators to ensure proper encapsulation of wire mash in mortar.

This repair technique is used for providing protective reinforced membrane for rehabilitation of distressed RCC structures. This acts as a protective layer against the vagaries of the environment. It is also used as a water proofing technique over reinforced concrete shell structures and RCC slabs as it provides impermeable thin membrane, which prevents seepage and leakage of water.

Although there are many obstacles to the greater use of ductility is attracting researchers to exploit its potential. Ferro-cement have no apparent advantages over any other type of reinforced concrete either in direct tension or flexure, but it has a high level of control over cracking provided by the spacing and specific surface area of the wire reinforcing mesh.

2.3 Rehabilitation

2.3.1 Member-Level Rehabilitation

A critical region in an RC frame is the beam-column connection, which ACI-ASCE 352 R (2002) defines as interior, exterior or corner connection depending on its vertical and horizontal position within the structure, and where different constructive details can originate local failures, such as the shear collapse of the panel due to the lack of transverse reinforcement. The member-level rehabilitation approach can provide a more cost-effective strategy than structure system-level rehabilitation because only those components needed to effectively enhance the seismic performance of the existing structure are selected and strengthened. The member-level rehabilitation approaches include the addition of concrete, steel, composite material (FRP) jackets or pretension tendons/wires in confining RC columns, beams and joints. This would involve jacketing the columns, beams and joint regions to improve flexural and shear strength, and concrete confinement.

Member Level Rehabilitation can be done by number of techniques. Some of them are as follows

- Column Jacketing
- External Reinforcement in the form of Steel Plate Bonding and External-Bonded (FRP Composites)
- Over slabbing
- Steel plate bonding

2.3.1.1 Column Jacketing

Columns are critical elements in any structural system and their performance during a seismic event can dominate the overall outcome of the structure. Failure of these older RC columns in shear usually takes place at low deformations and is associated with a large and sudden drop in lateral load resistance. Moreover, the shear strength of a column tends to degrade faster than its flexural strength with cycling of the lateral load. Column failures have caused the most significant failures of RC structures. So column rehabilitation is often critical to the seismic performance of a structure. To prevent the column failure mechanism during earthquakes, columns should never be the weakest components in the whole structure. The response of a column in a structure is controlled by its combined axial, flexural, and shear load. Therefore, column jacketing may be used to increase column shear and flexural strength so that columns are not damaged.

2.3.1.2 External Reinforcement in the Form of FRP or Steel

Strengthening of concrete structures by means of externally bonded reinforcements is generally done using either steel plates or CFRP laminates. Each material has its specific advantages and disadvantages. Steel plates have been used for many years due to their simplicity in handling and applying and to their effectiveness for strengthening. The properties and behavior of the steel adhesive- concrete combination are well known. Steel plates are very effective to be used as bending reinforcement. The high tensile strength and stiffness lead to an increase in bending capacity and a reduction of the deformations. Steel plates can also be used as external shear reinforcement. However, labour costs might rise quickly. Steel stirrups have to be bent or welded and very often anchored with bolts in the concrete compression zone. When several stirrups per meter are needed, these costs can make this technique economically less interesting.

CFRP sheets and laminates have very high tensile strength and stiffness. Nevertheless, they cannot be used in every strengthening situation. When used in bending, the active stresses in the CFRP laminates have to be kept small in order to prevent the internal steel reinforcement from yielding. This means that the high strength properties of the CFRP sheets are not used effectively, except if the laminates are pre-stressed. However, pre-stressing increases the cost and decreases the ease of application, which both has negative effect on the economy of the technique. Besides pre-stressing, the required increase of bearing capacity can only be reached by adding a considerable number of sheets or a thick laminate, which increases the material and labour costs. For limiting the deflections, CFRP sheets are not very effective. Due to their very small cross sectional area per sheet, the moment of inertia will only slightly increase and so the deformation decrease will only be marginal. In these cases, steel plates offer very often a better alternative. On the other hand, CFRP sheets are more appropriate for shear strengthening than steel plates. An orthogonal net of carbon fibres bonded at both sides of a beam is very well able to take shear forces. The applying of the CFRP sheets is very easy. Even complex shapes and geometries can be done. Labour costs are considerably lower for CFRP than for externally bonded steel stirrups.

To combine the features of the two materials, a hybrid steel/CFRP strengthening method can be developed. The additional longitudinal reinforcement consists of externally bonded steel plates, whereas the shear reinforcement consists of externally bonded CFRP sheets.

2.3.1.3 Over-Slabbing

In this technique, a plain or reinforced concrete slab is overlaid on the top of the existing slabs or beams to increase the section dimension in order to increase flexural strength. To ensure the composite action between the two dowels or shear studs may be installed. This method in particular may be advantageous when the member needing strengthening possesses reinforcement near or equal to balanced steel. However, as the flexural strength of a reinforced concrete member is usually limited by the capacity of the reinforcement rather than by the capacity of concrete, over-slabbing may therefore, is of no significant value. Any strength increase may be offset by the increase in dead load.

2.3.1.4 Steel Plate Bonding

Plate bonding is an inexpensive, versatile and advanced technique for rehabilitation, upgrading of concrete structures by mechanically connecting MS plated by bolting and gluing to their surfaces with epoxy. Plate bonding cans substantially increased strength, stiffness, ductility and stability of the reinforced concrete elements and can be used effectively for seismic retrofitting. The behavior of resulting composite system largely depends upon the inter-layer bond between concrete and plate. The thickness of plate needs special attention on relatively thick plates can initiate horizontal cracking and plate separation. With increase width, there is a risk of defect in adhesive, and with increasing thickness of the adhesive; the slip between the reinforcing element and concrete becomes greater. Anchor plates are needed where the width-thickness ratio of the plate is less than 50:1 due to production of high stresses near the ends of the plates leading to premature failure. This technique cannot be used where the member shows any sign of reinforcement corrosion.

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Chapter 3
Introduction to ETABS

Abstract The structural analysis software plays an important role to carry out the seismic calculation for the infrastructures. In this modern period of time, where computer has reached every phase of life, using the traditional book system for analytical development of the students, which is no doubt necessary, is no longer sufficient. Moreover, construction and design has become so much competitive in this world that using computers became mandatory. In this chapter, an introduction to the structural analysis software ETABS has been presented. Different features of the software have been highlighted which was used to carry out the analysis work presented in this book.

3.1 Introduction

In this modern world it is highly important for the new structural engineers to be well familiar with the current softwares of design so that when they come in the field they will not find difficulties in understanding these softwares. In undergraduate engineering standard generally analytical development of the students is done by using the tradition book system which is no doubt necessary but now in this competitive world of construction and design everything is done by using computers. Therefore the need of the hour is the proper understanding of the current design softwares. There are many softwares available for designing. But in this book ETABS is used as a design tool for high rise buildings gravity and seismic analysis. ETABS undoubtedly the most commonly used software for design in the whole world. Therefore it is highly important to know ETABS analytical and designing features. Below some of the features of ETABS are mentioned with brevity.

3.2 Building Model

- Multiple simultaneous rectangular and cylindrical grid systems
- Story definitions using the concept of similar Stories

R.R. Hussain et al., *Computer Aided Seismic and Fire Retrofitting Analysis of Existing High Rise Reinforced Concrete Buildings*,
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- Building modeled as Area, Line and Point objects
- Common labeling of Objects between similar Stories
- Area objects for: Walls, Slabs/Decks, Openings, Springs, Mass, Loads
- Line objects for: Columns, Beams, Braces, Links, Springs, Mass, Loads
- Point objects for: Supports, Springs, Mass, Loads
- Rigid Diaphragm definitions
- Built-in database of steel sections
- Graphical Section Designer for defining custom sections

3.3 Building Loads

- No limit on number of independent load cases
- Gravity loads specified as point, line or area loads
- Automatic wind load generation: UBC, BOCA, ASCE, NBCC
- Automatic seismic load generation: UBC, BOCA, NBCC
- Built-in response spectrum and time history input
- Temperature and thermal-gradient loads
- Algebraic, absolute, SRSS, and enveloping load combinations
- Mass directly specified or calculated from gravity loads

3.4 Analytical Options

- Static and dynamic analysis
- Automatic meshing of frame members into analysis elements
- Automatic transfer of loads on decks/slabs to beams and walls
- Automatic meshing of decks/slabs for flexible diaphragm analysis
- P-delta analysis with either static or dynamic analysis
- Automated center-of-rigidity calculations
- Integrated output forces for walls/slabs/decks for all loads
- Explicit Panel-zone deformations
- Automatic tributary-area calculations for Live-Load reduction factors
- Construction sequence loading analysis
- Eigen and load-dependent Ritz vector determination
- Multiple Response Spectrum cases
- Modal combination by SRSS, CQC or GMC (Gupta) method
- Combination of three directions by ABS or SRSS method
- Static and dynamic response combinations and envelopes
- Multiple Time History cases
- Sequential Time History cases
- Seismic acceleration or displacement excitation

- Wind-load forcing functions
- Transient or steady-state excitation
- Envelope or step-by-step design for Time-History loads

3.5 Analysis Output Options

- Deformed and Undeformed geometry in 3D perspective
- Loading diagrams
- Bending-Moment and Shear-Force diagrams for Frames
- Stress contours for Shells
- Integrated-force diagrams for Wall Piers and Spandrels
- Interactive Section-force results using Groups
- Animation of deformed shapes
- Time-History deformed shapes as real time AVI files
- Displays of nodal and element time-history records
- Time History displays of function vs. time or function vs. function
- Response spectrum curves for any joint from Time History response
- Instantaneous on-screen results output with right-button click on element
- Selective or complete tabulated output for all output quantities
- Graphics output to screen, printer, DXF file, or Windows Metafile
- Tabulated output to screen, printer, or Access Database

3.6 ETABS Nonlinear Features

ETABS Nonlinear extends the capabilities of the PLUS version to include the following static and dynamic nonlinear analysis options
Static Nonlinear Analysis Options
Large displacement option
Sequential loading option
Plastic Hinge Element

- Used as Spring, Link, Panel zone or inside Frame Elements
- Axial, flexural, shear and torsional behavior
- Axial-load/biaxial-moment interaction
- Multilinear behavior including softening
- Tabulated and Graphical display of hinge status

3.7 Specialization for Static Pushover Analysis

- FEMA 273, ATC-40
- Automated force-deformation relations for steel and concrete hinges
- Modal, uniform, or user-defined lateral load patterns
- Start from applied gravity load
- Capacity Spectrum conversions
- Effective damping calculation
- Demand Spectrum comparisons
- Performance point calculation
- Summary reports including plastic-hinge deformations

3.8 Dynamic Nonlinear Analysis Options

The nonlinear dynamic analysis option extends the capabilities of the Linear Time History option of the ETABS Plus by allowing for nonlinearity in predefined nonlinear elements.

3.8.1 Nonlinear Link Element

- Used with the Dynamic Nonlinear Analysis option
- Used as Link, Spring or as Panel zone
- Viscous damper with nonlinear exponent on velocity term
- Gap (compression only) and Hook (tension only)
- Uniaxial plasticity (all 6 degrees of freedom)
- Base isolator with biaxial-plasticity behavior
- Base isolator with friction and/or pendulum behavior
- Force or displacement vs. time plots
- Force versus deformation plots

3.8.2 The Wilson FNA Method

The ETABS nonlinear time history analysis uses the new numerical integration technique known as the Wilson FNA (Fast Nonlinear Analysis) Method. The procedure uses an iterative vector superposition algorithm that is extremely efficient for analyzing structures with predefined, localized nonlinearity. The method has demonstrated significant reductions in processing times when compared with other nonlinear analysis methods.

3.9 The Element Library

Underlying the ETABS object-based building models is a comprehensive analysis engine comprised of the following element types.

3.9.1 The 3D Beam/Column/Brace (Frame) Element

- Axial, bending, torsional and shear deformations
- Multiple non-prismatic segments over element length
- Ends offset from reference nodes in any direction
- Automated evaluation of offsets for joint size
- Moment and shear releases and partial-fixity
- Point, uniform and trapezoidal loading in any direction
- Temperature and thermal-gradient loading

3.9.2 The 3D Wall/Slab/Deck (Shell) Element

• Shell, plate or membrane action

- Thick-shell option
- General quadrilateral or triangular element
- Orthotropic materials
- Six degrees of freedom per joint
- Uniform load in any direction
- Temperature and thermal-gradient loading

The Joint Element

- Support
- Coupled or uncoupled grounded springs
- Force loads
- Ground-displacement loads

The Link Element

- Two node linear spring with 6 degrees of freedom
- Can be used to model Panel-zone deformations

3.10 Design Options

The following design options are fully integrated with analysis in the ETABS® graphical user interface.

Steel Frame Design

- Fully integrated steel frame design
- AISC-ASD, AISC-LRFD, UBC, Canadian and Euro Codes
- Design for static and dynamic loads
- Grouping for design envelopes
- Optimization for strength and lateral drift
- Seismic design of special moment-resisting frames
- Seismic design of concentric and eccentric braced frames
- Check of panel zones for doubler and continuity plates
- Graphical display of stress ratios
- Interactive design and review
- Summary and detailed reports including database formats

3.10.1 Concrete Frame Design

- Fully integrated concrete frame design
- ACI, UBC, Canadian and Euro Codes
- Design for static and dynamic loads
- Seismic design of intermediate/special moment-resisting frames
- Seismic design of beam/column joints
- Seismic check for strong-column/weak-beam design
- Graphical Section Designer for concrete rebar location
- Biaxial-moment/axial-load interaction diagrams
- Graphical display of reinforcement and stress ratios
- Interactive design and review
- Summary and detailed reports including database formats

3.10.1.1 Composite Beam Design

- Fully integrated composite beam design
- AISC-ASD and AISC-LRFD Specifications
- Automatic calculation of effective slab widths
- Numerous user-specified constraints
- Shored and unshored design
- Optimal design for strength and deflections
- Camber calculations
- Floor vibration analysis
- Graphical display of all design quantities
- Interactive design and review
- Summary and detailed reports including database formats

3.10.1.2 Concrete Shear Wall Design

- Fully integrated wall pier and spandrel design
- ACI, UBC and Canadian Codes
- Design for static and dynamic loads
- Automatic integration of forces for piers and spandrels
- 2D wall pier design and boundary-member checks
- 2D wall spandrel design
- 3D wall pier check for provided reinforcement
- Graphical Section Designer for concrete rebar location.
- Graphical display of reinforcement and stress ratios.
- Interactive design and review.
- Summary and detailed reports including database formats.

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Chapter 4
Structural Evaluation for Gravity Loads

Abstract This chapter discusses briefly the ACI code used for gravity analysis and stepwise methodology. The considered structure for this analytical study is a reinforced concrete structure, consisting 15 storys each having a height of 10 ft. The different range of gravity and thermal loads may be applied for seismic analysis. In this analytical work lateral loads considered are automated UBC, IBC 2000 and NBCC seismic and wind load. ETABS software was used to carry out the seismic analysis considering two different combinations of loads. The results obtained from the ETABS analysis were cross validated using the manual calculations for a single column with the approximate catchment area technique. The analysis yields reliable results, as the estimated load is almost 90 % of the load calculated by the software.

4.1 Introduction

Loads that act on the structures can be divided into three broad categories: dead loads, live loads, and environmental loads.

Dead Loads are those that are constant in magnitude and fixed in location throughout the lifetime of structure. Usually the major part of the dead load is the weight of the structure it's self. For buildings, floor fill, finish floors, piping, lighting fixtures and plastered ceilings are usually included as super imposed dead loads.

Live loads consist of occupancy loads of a building. They may be either fully or partially in place or not at all and may also change in location. Their magnitude and locations at any given time are uncertain. The live loads for which the floor and roof must be designed are usually specified in the building code that governs at the site of construction. ACI code is used for the design of gravity loads in this project.

Environmental loads consist mainly of snow loads, wind pressure, earthquake loads, soil pressure on subsurface portion of the structure, and forces caused by temperature differentials. Like live loads, environmental loads at any given time are uncertain both in magnitude and distribution.

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Solid Mechanics and Its Applications 222, DOI 10.1007/978-94-017-7297-6_4

Earthquake load is a type of load that is acting in the lateral direction on the structure. UBC 97 code is used for the analysis of earthquake loads. In this chapter the design of gravity loads will be discussed in detail and the analysis for lateral loads will be discussed in the later chapter.

4.2 Building Description

The structure that is taken under study is a reinforced concrete structure. It consist of 15 storys each having a height of 10 ft. Basement height is 12 ft and height of the OHWT is 8 ft above the top roof level. It is 173′ × 47.25′ plot. Its covered area is 7760 sq-ft. Overall height of the structure is 160 ft. Driveway width is 19.85 ft. It is a residential building of a mega city.

Architectural drawings are taken from a consultant. The building has been designed using computer aided software ETABS (2008) keeping in view the given architectural plan. The modifications that were done in it are as follows:

1. Front elevation of the building is curved. It was transformed into straight elevation for convenience.
2. Building is consisting of two blocks joint by a lobby. Front elevation of both the blocks consists of a wide balcony with servant bath, laundry and a duct, which is not, included the modified plan.
3. From back elevation, some portion of bedrooms and baths are deducted and balcony is entirely removed.

4.3 Pictorial View of the Model

3-D View:

See Fig. 4.1.

Plan of the Model:

See Fig. 4.2.

4.4 Software Used for Analysis and Designing

There are several softwares available for the structural designing. Particularly for a building design ETABS is more efficient and widely used. ETABS is the abbreviation of 'Extended three dimensional analysis of building system'.

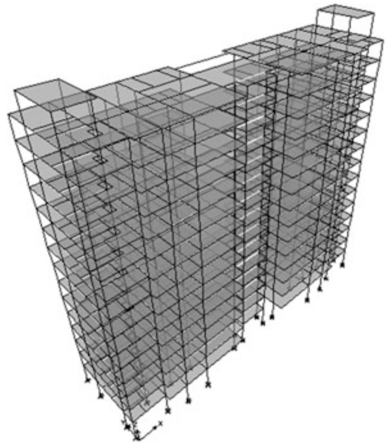


Fig. 4.1 3-D View of model

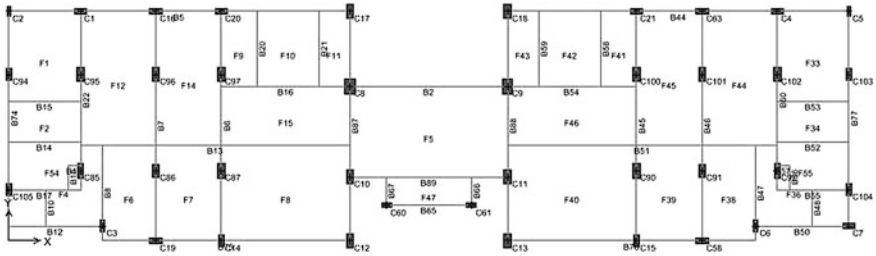


Fig. 4.2 Architectural plan of the model

4.4.1 Introduction of ETABS

ETABS offers sophisticated analysis and design for steel, concrete masonry multistory building structure as official building apartment and hospital. Its analysis is based upon direct stiffness formulation and finite element techniques. The input, output, and numerical solution techniques of ETABS are specifically designed to take advantages of the unique physical and numerical characteristics associated with building type structures. A complete suite of window graphical tools and utilities are included with the base package, including a modeler and a post processor for viewing all results including forces diagrams and shapes.

4.4.2 Fundamental Concepts

ETABS v 8 works on an integrated database. The basic concept is that only one model consisting of the floor systems and the vertical and lateral-framing system to analyze and design the whole building is created. Everything needed is created into one versatile analysis and design system with one user interface. There are no external modules to maintain and no worries about data transfer between modules. The effects on one part of the surface from changes in another part are instantaneous and automatic.

4.4.3 Modeling Options

ETABS can analyze any combination of 3-d frame and shear wall systems and provides complete interaction between the two. The shear wall element is specially formulated for ETABS. The output produced is in the form of wall forces and moments rather than stresses.

4.4.4 Loading Options

A wide range of gravity and thermal loads may be applied for analysis. Lateral loads include automated UBC, IBC 2000 and NBCC seismic and wind. Lateral/Environmental load can be defined as user input or it may be auto-generated using the built-in codes library in ETABS.

4.4.5 Design Options

Steel frames, concrete frame and concrete/masonry shear wall can be design easily by this software.

4.4.6 Advantages

Advantages of ETABS are as follows:

- Most buildings are of simple geometry with horizontal beams and vertical columns. A simple grid system define by lines option can establish such geometry with minimal input.

- Many of the frames and shear walls are typical. Most general programs do not recognize this fact, therefore inputs may be large and some internal calculations may be complicated.
- The loading in building systems is of a restricted form. Loads in general are either vertically down (dead or live) or lateral (wind or seismic). The vertical loads are generated at the floor levels.
- It is desirable to have a building analysis computer output printed in a special format i.e. in terms of a particular frames, storey columns and beam. Also special output, such as lateral storey displacement and internal storey drifts, time periods etc. may be desirable.

All of the above mentioned characteristics of building system are recognized by ETABS, making it ideally suitable for specific application.

4.5 Analysis and Design Procedure Using ETABS Software

First the tutorial of ETABS is studied in detail and a small problem is analyzed in order to develop the design concepts. After studying working of ETABS the modified model is prepared. Following steps are performed before analysis for gravity loads;

1. Building plan grid system is defined with the help of architectural drawing.
2. 15 numbers of stories and typical 10ft height has been defined except for the basement and roof top which was 12ft and 8ft respectively.
3. After specifying nos. of stories, frame properties are defined.
4. Column and beam sizes (cross-section) and their material are defined.
5. 5″ Slab thickness and its type (plate) is specified.
6. Now, columns are assigned on the grid system, then beam and slab respectively.
7. For applying loads, static load cases are defined. Dead load, live load and super imposed dead load are defined for gravity analysis.
8. Now these loads are assigned on the area object (slab) as 60 lb/ft2 for live load and 20 lb/ft2 for super imposed dead load considering 2″ thick finishes. Dead load is automatically calculated by the software using defined cross-sections. Load of water tank was assigned on the slab using the height of the tank and unit weight of water that is 62.41 lb/ft3.
9. Wall load is assigned on the line object (beam) using 7 ft high concrete wall having the specific weight of 144 lb/ft2. Wall thickness is 5″ and the load is 350 lb/ft.
10. Model is analyzed in the end.

11. Following load combinations were automatically generated by the software;

    - Combo 1

      1.4 D.L + 1.4 SDEAD

    - Combo 2

      1.4 D.L + 1.7 L.L + 1.4 SDEAD

12. From input table mode column reactions are noted which are maximum for the combo 2 at the bottom story. With the help of these reactions approximate column sizes are calculated using formula:

    - $Ag = Pu/1.4\ f'c$

13. Some of the columns and beams have been found overstressed because at first column sizes are roughly estimated. Torsion is found to be the major problem, which caused some of the beams over, stressed.
14. By changing the orientation of the column this problem has been removed successfully.
15. After assigning the cross-sections as per calculation (step 12), the building is designed for gravity loads.
16. Several trials are made until we got the final beam and column sizes.
17. The bending, torsion and rebar percentage have been found under control.
18. Typical Column sizes are as follows;

    - CA = 10×24 inch^2
    - CB = 12×30 inch^2
    - CC = 15×30 inch^2
    - CD = 15×36 inch^2
    - CE = 18×36 inch^2
    - CF = 24×36 inch^2

19. Typical Beam sizes are as follows;

    - concbeam = 8×42 inch^2
    - Concbeam 8×30 = 8×30 inch^2
    - Concbeam 8×48 = 8×48 inch^2

4.6 Analysis Inputs

4.6.1 Grid Data

The grid data is provided below (Table 4.1).

Table 4.1 Grid data

GridDir	GridID	GridCoord	GridType	GridHide	BubbleLoc	SortID
X	1	0	Primary	No	Default	1
X	1A	44.16	Primary	No	Default	2
X	1B	91.248	Primary	No	Default	3
X	1C	147	Primary	No	Default	4
X	2	177	Primary	No	Default	5
X	3	231	Primary	No	Default	6
X	4	363	Primary	No	Default	7
X	5	525	Primary	No	Default	8
X	5A	613.92	Primary	No	Default	9
X	5B	765.96	Primary	No	Default	10
X	6	843	Primary	No	Default	11
X	6f	874.56	Primary	No	Default	12
X	6a	906.96	Primary	No	Default	13
X	7	933	Primary	No	Default	14
X	6b	982.92	Primary	No	Default	15
X	6c	1082.88	Primary	No	Default	16
X	8	1143	Primary	No	Default	17
X	6d	1158.84	Primary	No	Default	18
X	8A	1201.44	Primary	No	Default	19
X	6e	1222.92	Primary	No	Default	20
X	9	1233	Primary	No	Default	21
X	10	1551	Primary	No	Default	22
X	11	1713	Primary	No	Default	23
X	12	1845	Primary	No	Default	24
X	14	2076	Primary	No	Default	26
Y	A	0	Primary	No	Switched	27
Y	B	8.496	Primary	No	Switched	28
Y	B1	12.492	Primary	No	Switched	29
Y	B2	36.576	Primary	No	Switched	30
Y	col2	78.84	Primary	No	Default	31
Y	C	87.84	Primary	No	Switched	32
Y	C1	126.084	Primary	No	Switched	33
Y	D	157.692	Primary	No	Switched	34
Y	D1	187.08	Primary	No	Switched	35
Y	D2	235.692	Primary	No	Switched	36
Y	D3	243.684	Primary	No	Switched	37
Y	D4	343.92	Primary	No	Switched	38
Y	D5	381.684	Primary	No	Switched	39
Y	E	395.94	Primary	No	Switched	40
Y	EB	445.44	Primary	No	Switched	41
Y	col	474.12	Primary	No	Default	43
Y	E1	511.932	Primary	No	Switched	44

4.6.2 Load Cases

See Table 4.2.

4.6.3 Load Combinations

See Table 4.3.

4.7 Analysis Result

4.7.1 Column Forces

See Table 4.4.

Column Key:
Approximate sections are assigned according to rough calculations (Table 4.5).

4.7.2 Beam Forces

See Table 4.6.

Table 4.2 static load cases

Case	Type	SW multiplier
LIVE	LIVE	0
SDEAD	SUPER DEAD	0
DEAD	DEAD	1

Table 4.3 Load combos

Combo	Type	Case	Factor	Case type	Sort id
COMB1	ADD	SDEAD	1.4	Static	1
COMB1		DEAD	1.4	Static	2
COMB2	ADD	SDEAD	1.4	Static	3
COMB2		DEAD	1.4	Static	4
COMB2		LIVE	1.7	Static	5

Table 4.4 Column forces

Story	Column	Load	Loc	P	V2	V3	T	M2	M3
1	C1	COMB2	0	−543.97	1.26	0.68	0	32.633	70.879
1	C1	COMB2	55.5	−542.35	1.26	0.68	0	−5.354	1.14
1	C1	COMB2	111	−540.73	1.26	0.68	0	−43.341	−68.599
1	C14	COMB2	0	−993.67	−3	−0.47	0	−22.521	−104.37
1	C14	COMB2	55.5	−991.24	−3	−0.47	0	3.804	61.858
1	C15	COMB2	0	−927.43	0	1.19	0	61.652	−0.28
1	C15	COMB2	55.5	−925	0	1.19	0	−4.67	−0.067
1	C15	COMB2	111	−922.57	0	1.19	0	−70.993	0.146
1	C16	COMB2	0	−940.87	0.41	1.18	0	60.96	18.327
1	C16	COMB2	55.5	−938.44	0.41	1.18	0	−4.555	−4.408
1	C16	COMB2	111	−936.02	0.41	1.18	0	−70.07	−27.144
1	C17	COMB2	0	−1322.16	−7.5	1.45	−0.001	77.854	−337.80
1	C17	COMB2	51	−1319.37	−7.5	1.45	−0.001	4.076	44.889
1	C17	COMB2	102	−1316.58	−7.5	1.45	−0.001	−69.701	427.583
1	C19	COMB2	0	−920.91	0.11	−1.28	0	−55.879	4.745
1	C19	COMB2	55.5	−918.48	0.11	−1.28	0	15.066	−1.253
1	C19	COMB2	111	−916.05	0.11	−1.28	0	86.011	−7.251
1	C20	COMB2	0	−1018.36	−2.56	0.79	0	37.433	−95.993
1	C20	COMB2	51	−1016.12	−2.56	0.79	0	−2.606	34.313
1	C20	COMB2	102	−1013.89	−2.56	0.79	0	−42.645	164.619
1	C43	COMB2	0	−543.98	1.26	−0.68	0	−32.586	70.847
1	C43	COMB2	55.5	−542.36	1.26	−0.68	0	5.364	1.125
1	C43	COMB2	111	−540.74	1.26	−0.68	0	43.315	−68.597
1	C55	COMB2	0	−993.67	−3	0.48	0	22.628	−104.43
1	C55	COMB2	55.5	−991.24	−3	0.48	0	−3.778	61.835
1	C55	COMB2	111	−988.82	−3	0.48	0	−30.184	228.114
1	C57	COMB2	0	−1018.62	−2.55	−0.78	0	−37.394	−95.992
1	C57	COMB2	51	−1016.39	−2.55	−0.78	0	2.631	34.297
1	C57	COMB2	102	−1014.16	−2.55	−0.78	0	42.657	164.586
1	C58	COMB2	0	−920.93	−0.11	−1.28	0	−55.887	−5.201
1	C58	COMB2	55.5	−918.5	−0.11	−1.28	0	15.062	1.068
1	C58	COMB2	111	−916.08	−0.11	−1.28	0	86.012	7.337
1	C60	COMB2	0	−506.93	−2.06	−2.5	0	−116.822	−94.97
1	C60	COMB2	55.5	−505.31	−2.06	−2.5	0	21.82	19.354
1	C60	COMB2	111	−503.69	−2.06	−2.5	0	160.461	133.679
1	C61	COMB2	0	−512.57	2.02	−2.51	0	−117.501	92.928
1	C61	COMB2	55.5	−510.95	2.02	−2.51	0	21.931	−19.057
1	C61	COMB2	111	−509.33	2.02	−2.51	0	161.362	−131.04
1	C62	COMB2	0	−1322.21	7.5	1.45	−0.001	77.849	337.348
1	C62	COMB2	51	−1319.42	7.5	1.45	−0.001	4.07	−45.131
1	C62	COMB2	102	−1316.63	7.5	1.45	−0.001	−69.709	−427.61

(continued)

Table 4.4 (continued)

Story	Column	Load	Loc	P	V2	V3	T	M2	M3
1	C63	COMB2	0	−940.88	−0.41	1.18	0	60.948	−18.744
1	C63	COMB2	55.5	−938.45	−0.41	1.18	0	−4.558	4.242
1	C63	COMB2	111	−936.02	−0.41	1.18	0	−70.064	27.228
1	C64	COMB2	0	−927.43	0	1.19	0	61.638	−0.121
1	C64	COMB2	55.5	−925	0	1.19	0	−4.674	−0.104
1	C64	COMB2	111	−922.57	0	1.19	0	−70.987	−0.086
1	C85	COMB2	0	−1677.31	−6.7	−0.48	−0.001	−22.473	−227.02
1	C85	COMB2	72	−1672.59	−6.7	−0.48	−0.001	11.837	255.464
1	C85	COMB2	144	−1667.86	−6.7	−0.48	−0.001	46.147	737.948
1	C86	COMB2	0	−1381.83	−3.43	−0.26	−0.001	−12.279	−116.41
1	C86	COMB2	72	−1377.89	−3.43	−0.26	−0.001	6.487	130.66
1	C86	COMB2	144	−1373.95	−3.43	−0.26	−0.001	25.252	377.729
1	C87	COMB2	0	−2063.64	−7.65	2.72	−0.001	127.548	−255.24
1	C87	COMB2	144	−2052.3	−7.65	2.72	−0.001	−263.674	846.011
1	C90	COMB2	0	−2063.94	−7.65	−2.72	−0.001	−127.316	−255.24
1	C90	COMB2	72	−2058.27	−7.65	−2.72	−0.001	68.168	295.303
1	C90	COMB2	144	−2052.6	−7.65	−2.72	−0.001	263.652	845.855
1	C91	COMB2	0	−1381.82	−3.43	0.26	−0.001	12.392	−116.47
1	C91	COMB2	72	−1377.88	−3.43	0.26	−0.001	−6.436	130.643
1	C91	COMB2	144	−1373.95	−3.43	0.26	−0.001	−25.265	377.76
1	C92	COMB2	0	−1677.31	−6.7	0.48	−0.001	22.633	−227.12
1	C92	COMB2	72	−1672.58	−6.7	0.48	−0.001	−11.79	255.406
1	C92	COMB2	144	−1667.86	−6.7	0.48	−0.001	−46.212	737.931
1	C94	COMB2	0	−906.45	4.68	1.43	0	67.819	241.205
1	C94	COMB2	72	−903.3	4.68	1.43	0	−34.91	−96.061
1	C94	COMB2	144	−900.15	4.68	1.43	0	−137.64	−433.32
1	C95	COMB2	0	−1742.28	7.43	−0.37	−0.001	−17.451	403.363
1	C95	COMB2	72	−1736.61	7.43	−0.37	−0.001	9.438	−131.59
1	C95	COMB2	144	−1730.94	7.43	−0.37	−0.001	36.327	−666.54
1	C96	COMB2	0	−1460.48	4.46	−0.57	−0.001	−26.915	260.561
1	C96	COMB2	72	−1455.75	4.46	−0.57	−0.001	14.161	−60.674
1	C96	COMB2	144	−1451.03	4.46	−0.57	−0.001	55.237	−381.90
1	C97	COMB2	0	−2058.58	9.33	7.8	−0.001	366.167	486.303
1	C97	COMB2	72	−2052.91	9.33	7.8	−0.001	−195.617	−185.39
1	C97	COMB2	144	−2047.24	9.33	7.8	−0.001	−757.401	−857.09
1	C100	COMB2	0	−2058.73	9.33	−7.8	−0.001	−365.949	486.231
1	C100	COMB2	72	−2053.06	9.33	−7.8	−0.001	195.719	−185.43
1	C100	COMB2	144	−2047.39	9.33	−7.8	−0.001	757.387	−857.10
1	C101	COMB2	0	−1460.48	4.46	0.57	−0.001	27.042	260.476
1	C101	COMB2	72	−1455.75	4.46	0.57	−0.001	−14.102	−60.714
1	C101	COMB2	144	−1451.03	4.46	0.57	−0.001	−55.245	−381.90
1	C102	COMB2	0	−1742.28	7.43	0.38	−0.001	17.682	403.233

(continued)

Table 4.4 (continued)

Story	Column	Load	Loc	P	V2	V3	T	M2	M3
1	C102	COMB2	72	−1736.61	7.43	0.38	−0.001	−9.342	−131.65
1	C102	COMB2	144	−1730.94	7.43	0.38	−0.001	−36.367	−666.54
1	C103	COMB2	0	−906.48	4.68	−1.43	0	−67.756	241.13
1	C103	COMB2	72	−903.33	4.68	−1.43	0	34.931	−96.082
1	C103	COMB2	144	−900.18	4.68	−1.43	0	137.619	−433.29
1	C104	COMB2	0	−1027.96	−4.13	−2.93	−0.001	−138.278	−148.09
1	C104	COMB2	55.5	−1024.92	−4.13	−2.93	−0.001	24.186	81.379
1	C104	COMB2	111	−1021.89	−4.13	−2.93	−0.001	186.65	310.848
1	C105	COMB2	0	−1027.93	−4.13	2.93	−0.001	138.43	−148.09
1	C105	COMB2	55.5	−1024.9	−4.13	2.93	−0.001	−24.128	81.424
1	C105	COMB2	111	−1021.86	−4.13	2.93	−0.001	−186.686	310.857
1	C112	COMB2	0	−737.18	4.36	−0.2	0	−4.637	196.106
1	C112	COMB2	55.5	−734.75	4.36	−0.2	0	6.454	−45.903
1	C112	COMB2	111	−732.32	4.36	−0.2	0	17.546	−287.91
1	C113	COMB2	0	−737.14	−4.36	−0.2	0	−4.623	−196.48
1	C113	COMB2	55.5	−734.71	−4.36	−0.2	0	6.46	45.701
1	C113	COMB2	111	−732.28	−4.36	−0.2	0	17.543	287.883
1	C135	COMB2	0	−1657.22	−7.74	1.69	−0.001	79.604	−259.27
1	C135	COMB2	55.5	−1652.85	−7.74	1.69	−0.001	−14.468	170.272
1	C135	COMB2	111	−1648.48	−7.74	1.69	−0.001	−108.54	599.821
1	C136	COMB2	51	−1280.98	−3.56	−6.47	−0.001	26.466	104.865
1	C136	COMB2	102	−1276.96	−3.56	−6.47	−0.001	356.329	286.538
1	C137	COMB2	0	−1662.23	−7.74	−1.78	−0.001	−83.244	−259.36
1	C137	COMB2	55.5	−1657.86	−7.74	−1.78	−0.001	15.311	170.269
1	C137	COMB2	111	−1653.49	−7.74	−1.78	−0.001	113.866	599.898
1	C138	COMB2	0	−1288	−3.58	6.46	−0.001	303.403	−77.592
1	C138	COMB2	51	−1283.98	−3.58	6.46	−0.001	−26.3	104.973
1	C138	COMB2	102	−1279.96	−3.58	6.46	−0.001	−356.002	287.537
1	C139	COMB2	0	−2469.48	7.06	−6.51	−0.002	−299.702	413.515
1	C139	COMB2	55.5	−2463.65	7.06	−6.51	−0.002	61.472	21.581
1	C139	COMB2	111	−2457.83	7.06	−6.51	−0.002	422.645	−370.35
1	C141	COMB2	0	−2471.25	7.08	6.51	−0.002	300.294	414.435
1	C141	COMB2	55.5	−2465.43	7.08	6.51	−0.002	−61.184	21.298
1	C141	COMB2	111	−2459.6	7.08	6.51	−0.002	−422.661	−371.83
1	C142	COMB2	0	−1530.52	4.7	−9.2	−0.001	−431.825	284.242
1	C142	COMB2	51	−1526.5	4.7	−9.2	−0.001	37.625	44.359
1	C142	COMB2	102	−1522.49	4.7	−9.2	−0.001	507.075	−195.52
1	C143	COMB2	0	−1531.11	4.71	9.21	−0.001	432.063	284.348
1	C143	COMB2	51	−1527.09	4.71	9.21	−0.001	−37.495	44.318
1	C143	COMB2	102	−1523.07	4.71	9.21	−0.001	−507.054	−195.71

Table 4.5 Columns assignments

Col. ID	Sec Assigned	Length	Width
C1	CA	24	10
C14	CB	36	18
C15	CB	36	18
C16	CB	36	18
C17	CC	30	15
C19	CB	36	18
C20	CB	36	18
C43	CA	24	10
C55	CB	36	18
C57	CB	36	18
C58	CB	36	18
C60	CA	24	10
C61	CA	24	10
C62	CC	30	15
C63	CB	30	12
C64	CB	30	12
C85	CD	36	15
C86	CC	30	15
C87	CE	36	18
C90	CE	36	18
C91	CC	30	15
C92	CD	36	15
C94	CB	30	12
C95	CE	36	18
C96	CD	36	15
C97	CE	36	18
C100	CE	36	18
C101	CD	36	15
C102	CE	36	18
C103	CB	30	12
C104	CC	30	15
C105	CC	30	15
C112	CB	30	12
C113	CB	30	12
C135	CE	36	18
C136	CE	36	18
C137	CE	36	18
C138	CE	36	18
C139	CE	36	18
C141	CE	36	18
C142	CE	36	18
C143	CE	36	18

Table 4.6 Beam forces

Story	Beam	Load	Loc	P	V2	V3	T	M2	M3
1	B6	COMB2	15	0	−17.38	0	51.167	0	12.399
1	B6	COMB2	31.069	0	−14.98	0	51.167	0	272.356
1	B6	COMB2	47.138	0	−12.57	0	51.167	0	493.678
1	B6	COMB2	47.138	0	−4.05	0	8.039	0	499.296
1	B6	COMB2	70.708	0	−0.52	0	8.039	0	553.174
1	B6	COMB2	94.277	0	3	0	8.039	0	523.936
1	B6	COMB2	94.277	0	16.16	0	−85.857	0	519.438
1	B6	COMB2	117.846	0	19.68	0	−85.857	0	97.114
1	B6	COMB2	141.415	0	23.21	0	−85.857	0	−408.325
1	B6	COMB2	141.415	0	35.13	0	−15.159	0	−455.273
1	B6	COMB2	157.054	0	37.47	0	−15.159	0	−1022.93
1	B6	COMB2	172.692	0	39.81	0	−15.159	0	−1627.18
1	B6	COMB2	172.692	0	−77.7	0	3.227	0	−3465.13
1	B6	COMB2	188.554	0	−75.33	0	3.227	0	−2251.48
1	B6	COMB2	188.554	0	−86.55	0	294.312	0	−2049.63
1	B6	COMB2	212.123	0	−83.02	0	294.312	0	−51.357
1	B6	COMB2	235.692	0	−79.49	0	294.312	0	1863.804
1	B6	COMB2	235.692	0	−2.81	0	65.4	0	1830.642
1	B6	COMB2	253.941	0	−0.08	0	65.4	0	1857.003
1	B6	COMB2	272.19	0	2.65	0	65.4	0	1833.536
1	B6	COMB2	272.19	0	2.79	0	61.486	0	1826.098
1	B6	COMB2	283.013	0	4.41	0	61.486	0	1787.084
1	B6	COMB2	283.013	0	9.42	0	60.321	0	1763.374
1	B6	COMB2	295.851	0	11.34	0	60.321	0	1630.159
1	B6	COMB2	308.688	0	13.26	0	60.321	0	1472.287
1	B6	COMB2	308.688	0	15.4	0	48.889	0	1438.021
1	B6	COMB2	330.334	0	18.64	0	48.889	0	1069.646
1	B6	COMB2	330.334	0	20.26	0	31.269	0	999.165
1	B6	COMB2	345.186	0	22.49	0	31.269	0	681.709
1	B6	COMB2	345.186	0	29.93	0	−5.431	0	609.765
1	B6	COMB2	361.421	0	32.36	0	−5.431	0	104.087
1	B6	COMB2	377.655	0	34.79	0	−5.431	0	−441.026
1	B6	COMB2	377.655	0	7.38	0	−198.60	0	−562.558
1	B6	COMB2	381.684	0	7.98	0	−198.60	0	−593.511
1	B6	COMB2	381.684	0	89.92	0	−473.96	0	−713.482
1	B6	COMB2	396.312	0	92.11	0	−473.966	0	−2044.89
1	B6	COMB2	410.94	0	94.3	0	−473.96	0	−3408.31
1	B6	COMB2	410.94	0	−29.06	0	−418.59	0	−1104.73
1	B6	COMB2	424.977	0	−26.96	0	−418.59	0	−711.545
1	B6	COMB2	424.977	0	−3.07	0	−266.858	0	−661.345

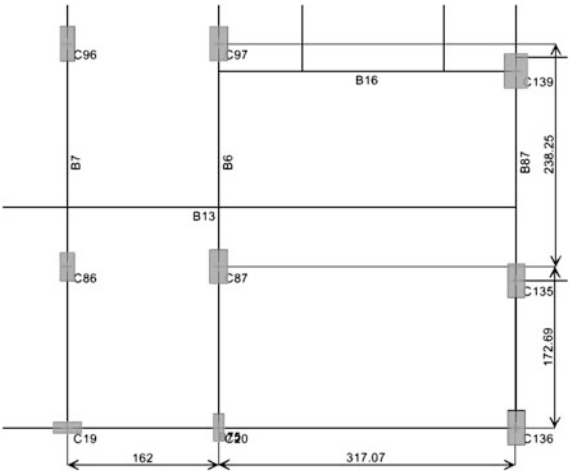
(continued)

Table 4.6 (continued)

Story	Beam	Load	Loc	P	V2	V3	T	M2	M3
1	B6	COMB2	427.998	0	−2.62	0	−266.858	0	−652.754
1	B6	COMB2	427.998	0	−22.44	0	−37.772	0	−541.522
1	B6	COMB2	450.148	0	−19.13	0	−37.772	0	−81.112
1	B6	COMB2	472.298	0	−15.81	0	−37.772	0	305.891
1	B6	COMB2	472.298	0	−0.74	0	16.223	0	327.708
1	B6	COMB2	474.312	0	−0.44	0	16.223	0	328.899
1	B6	COMB2	474.312	0	−4.81	0	59.45	0	335.037
1	B6	COMB2	496.965	0	−1.42	0	59.45	0	405.694
1	B6	COMB2	519.619	0	1.97	0	59.45	0	399.569
1	B6	COMB2	519.619	0	0.01	0	53.778	0	400.315
1	B6	COMB2	520.626	0	0.16	0	53.778	0	400.229
1	B6	COMB2	520.626	0	10.68	0	40.485	0	389.873
1	B6	COMB2	540.033	0	13.58	0	40.485	0	154.522
1	B6	COMB2	559.44	0	16.48	0	40.485	0	−137.18

4.7.3 Validation of Software Analysis

To check the software analysis result, a column, having the column ID C87, is considered and the manual calculations using the approximate catchment area technique, has done, as shown below,



4.7.4 Dead Load of a Single Story

Weight of slab:

Weight of slab = Area of slab under catchment area of the column* Thickness of slab* Unit weight of R.C.C

$$= [(81 + 160)/12] * [(86 + 119)/12] * (5/12) * 150$$
$$= 21443.142 \text{ lb.}$$

Weight of finishes:

$$= \text{Area} * \text{Thickness} * \text{Unit weight of finishes}$$
$$= [(81 + 160)/12] * [(86 + 119)/12] * (2/12) * 120$$
$$= 6862 \text{ lb.}$$

Weight of beams:

$$= [8 * 30 * (81 + 160)] + [10 * 42 * (86 + 119)]/12^3 * 150$$
$$= 12494.8 \text{ lb.}$$

Wight of Walls:

$$= [5 * 7 * (81 + 160 + 86 + 119) * 120]/144$$
$$= 13008.33 \text{ lb.}$$

Self Weight of Columns:

$$= [(18 * 36) * 10 * 150/144]$$
$$= 6750 \text{ lb.}$$

Total Dead Load:

$$= \text{Weight of slab} + \text{Finishes} + \text{Beams} + \text{wall} + \text{Columns}$$
$$= 60.6 \text{ Kips}$$

This load is for one story,

for 15 stories, the load will be

$$= 60.6 * 15$$
$$= 908.37 \text{ Kips}$$

4.7.5 Live Load

$$= [(81+160)/12] * [(86+119)/12] * 60$$
$$= 20.585416 \text{ kips.}$$

This load is foe one story, for fifteen stories; it will be 15 times of it.

$$= 308.775$$

4.7.6 Total Load

$$= 1.7 \text{ L.L } + 1.4 \text{ D.L}$$
$$= 1800 \text{ kips}$$

4.7.7 Comparison

The estimated load is almost 90 % of the load calculated by the software i.e. equal to 2050 kips. As shown in the result attached.

Reference

ETABS Software (2008) Version 9.2, Computer and structures, Inc. Berkeley, California

Chapter 5
Seismic Analysis

Abstract The decision to strengthen an existing structure vastly depends the building's seismic resistance to select an appropriate rehabilitation scheme, an accurate evaluation of the condition and seismic performance of the structure. In this chapter, the strength of existing building is evaluated for the given seismic zone; then adopting a practice to apply the earthquake load using Design Code (UBC-97), whether the building is safe enough to bear the earthquake load for the required zone '2B' was determined. The earthquake provisions considered in this study is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function. After analyzing the building for lateral loads, all the columns were found to be inadequate to support the interaction of axial force and biaxial moment. Moreover, some of the beams were failed in shear. It was also observed that sway of the building and story drift was under control.

5.1 General

A higher degree of damage in a building is expected during an earthquake if the seismic resistance of the building is inadequate. The decision to strengthen it before an earthquake occurs depends on the building's seismic resistance to select an appropriate rehabilitation scheme, an accurate evaluation of the condition and seismic performance of an existing structure is necessary. Based on this evaluation, practitioners can choose the most effective rehabilitation scheme among the various rehabilitation techniques and optimize the enhancement in seismic performance for an existing structure. Seismic deficiencies should first be identified through a seismic evaluation of the structure. The selection of an appropriate rehabilitation technique based on the structural condition and its deficiencies is the most important step in the whole rehabilitation process. Seismic evaluation consists of gathering as-built information and obtaining the results of a structural analysis based on collected data.

R.R. Hussain et al., *Computer Aided Seismic and Fire Retrofitting Analysis of Existing High Rise Reinforced Concrete Buildings*,
Solid Mechanics and Its Applications 222, DOI 10.1007/978-94-017-7297-6_5

In the previous chapter, the design of a high rise building for gravity load is explained. In this chapter, our objective is to evaluate the strength of existing building for the given seismic zone; we adopt a practice, to apply earthquake load using Design Code (UBC-97) and find out, whether the building is safe enough to bear the earthquake load of our required zone '2B'. UBC will be summarized in the next section.

5.2 Design Code for Earthquake Load

5.2.1 Introduction

The purpose of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function.

UBC stands for Uniform Building code. We follow UBC-97 edition in our analysis, now let us discuss important terminologies, necessary for defining earthquake loads.

5.2.2 Definitions

Base

Base is the level at which the earthquake motions are considered to be imparted to the structure or the level at which the structure as a dynamic vibrator is supported.

Base shear

V is the total design lateral force or shear at the base of a structure.

Design basis ground motion

It is that ground motion that has a 10 % chance of being exceeded in 50 years as determined by a site-specific hazard analysis or may be determined from a hazard map. A suite of ground motion time histories with dynamic properties representative of the site characteristics shall be used to represent this ground motion. The dynamic effects of the Design Basis Ground Motion may be represented by the Design Response Spectrum.

Design seismic force

It is the minimum total strength design base shear, factored and distributed.

Story shear

V_x is the summation of design lateral forces above the story under consideration.

Strength

It is the capacity of an element or a member to resist factored load.

5.3 Basis for Design

The procedures and the limitations for the design of structures shall be determined considering seismic zoning, site characteristics, occupancy, configuration, structural system and height in accordance with this section. Structures shall be designed with adequate strength to withstand the lateral displacements induced by the Design Basis Ground Motion, considering the inelastic response of the structure and the inherent redundancy, over strength and ductility of the lateral-force- resisting system. The minimum design strength shall be based on the Design Seismic Forces determined in accordance with the static lateral force procedure of Section 1630 (of UBC-97), except as modified by Section 1631.5.4(of UBC-97). Where strength design is used, the load combinations of Section 1612.2(of UBC-97) shall apply. Where Allowable Stress Design is used, the load combinations of Section 1612.3(of UBC-97) shall apply. Allowable Stress Design may be used to evaluate sliding or overturning at the soil-structure interface regardless of the design approach used in the design of the structure, provided load combinations of Section 1612.3(of UBC-97) are utilized.

5.3.1 Occupancy Categories

For purposes of earthquake- resistant design, each structure shall be placed in one of the occupancy categories listed in Table 16-K of chapter no 16 of UBC. Table 16-K assigns importance factors, *I,* and structural observation requirements for each category.

5.3.2 Site Geology and Soil Characteristics

Each site shall be assigned a soil profile type based on properly substantiated geotechnical data using the site categorization procedure set forth in Division VI, Section 1636(of UBC-97) and Table 16-J.

Exception: When the soil properties are not known in sufficient detail to determine the soil profile type, Type *SD* shall be used. Soil Profile Type *SE* or *SF* need not be assumed unless the building official determines that Type *SE* or *SF* may be present at the site or in the event that Type *SE* or *SF* is established by geotechnical data.

5.3.3 Soil Profile Type

Soil Profile Types SA, SB, SC, SD and SE are defined in Table 16-J and Soil Profile Type SF is defined as soils requiring site-specific evaluation as follows:

1. Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.
2. Peats and/or highly organic clays, where the thickness of peat or highly organic clay exceeds 10 feet (3048 mm).
3. Very high plasticity clays with a plasticity index, PI > 75, where the depth of clay exceeds 25 feet (7620 mm).
4. Very thick soft/medium stiff clays, where the depth of clay exceeds 120 feet (36 576 mm).

5.3.4 Seismic Zone

Each structure shall be assigned a seismic zone factor *Z,* in accordance with Table 16-I.

5.3.5 Seismic Response Coefficients

Each structure shall be assigned a seismic coefficient, *Ca,* in accordance with Table 16-Q and a seismic coefficient, *Cv,* in accordance with Table 16-R.

5.4 Earthquake Loads and Modeling Requirements

5.4.1 Design Base Shear

The total design base shear in a given direction shall be determined from the following formula.

$$V = (C\,v\,I)/(R\,T) * W \tag{5.1}$$

The total design base shear need not exceed the following:

$$V = 2.5\,C\,a\ \ I/R * W \tag{5.2}$$

The total design base shear shall not be less than the following:

$$V = 0.11\, C a I W \tag{5.3}$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$\mathrm{V} = 0.8\ \mathrm{Z\ N\ v\ I/R} * \mathrm{W} \tag{5.4}$$

5.4.2 *Structure Period*

The value of T shall be determined for all buildings. The value T may be approximated from the following formula,

$$\mathrm{T = Ct\ (hn)^{3/4}} \tag{5.5}$$

where:

Ct = 0.035 (0.0853) for steel moment-resisting frames.
Ct = 0.030 (0.0731) for reinforced concrete moment-resisting frames and eccentrically braced frames.
Ct = 0.020 (0.0488) for all other buildings.
h n = height above the base to level n
(Ref. 9)

5.5 Analysis Inputs

5.5.1 *Zone Factor for Which Building Is Situated According to UBC-97 Division*

According to UBC classification Karachi is located in zone 2B of seismic zone classification UBC. Therefore, the earthquake loads for zone 2B are applied to model, to analyze and check. Let's have a brief introduction of basis of Zoning System.

In order to understand the Seismic Zoning method and how it pertains to the Monolithic Dome, we must first understand what effective peak ground acceleration means and how it is measured against gravity.

First let's discuss what is meant by ground acceleration. It is not unlike the feeling we have when a car takes off at high speed, or when we ride the roller coasters at Disneyland. In order to measure the acceleration of an earthquake, it must be measured against Gravity (or 1.0 g). Gravity is the rate at which an object falls when dropped from being at rest in a vacuum. It is quite a high rate of acceleration. It is approximately the same as a car traveling 100 m from rest in just 4.5 s.

There are studies that show that much of the damage done in earthquakes is, perhaps, due rather to the velocity of the back and forth movements of the earth, rather than to the ground acceleration. However, the mean and peak ground accelerations do have much to do with the intensity of damage a building may have to withstand. Consequently, engineers and designers rely a great deal on the measure of the peak ground acceleration, as compared to gravity, to determine how strong an earthquake force a new building may have to withstand.

There are instruments called accelerographs that measure ground acceleration against the value of gravity. (Acceleration in g/10) These values are gathered from all parts of the nation to create a seismic-risk map, which is used by engineers and builders when designing earthquake resistant structures for different parts of the country.

5.5.2 Soil Conditions

Due to unavailability of actual data, soil type has supposed to be SD.

5.5.3 Ductility/Over-Strength Factor

The value of R is taken for intermediate moment resisting frame as 5.5 according to UBC-97

5.6 Analyze Outputs

5.6.1 Columns Forces

See Table 5.1.

5.6.2 Beam Forces

See Table 5.2.

Table 5.1 Columns Forces (*Units* Kips-in)

Story	Column	Load	Loc	P	V2	V3	T	M2	M3
1	C1	DCON4	0	−741.6	1.14	11.09	0.357	795.643	86.436
1	C1	DCON4	55.5	−740.06	1.14	11.09	−0.357	180.219	23.36
1	C1	DCON4	111	−738.52	1.14	11.09	−0.357	−435.204	−39.716
1	C14	DCON4	0	−999.77	5.4	21.39	−0.782	1597.901	310.379
1	C14	DCON4	55.5	−997.46	5.4	21.39	−0.782	410.828	10.555
1	C14	DCON4	111	−995.16	5.4	21.39	−0.782	−776.245	−289.268
1	C15	DCON4	0	−988.61	−77.88	0.9	−0.782	53.828	−7110.3
1	C15	DCON4	55.5	−986.31	−77.88	0.9	−0.782	4.082	−2787.83
1	C15	DCON4	111	−984	−77.88	0.9	−0.782	−45.664	1534.651
1	C16	DCON4	0	−892.97	−84.04	1.09	−0.782	61.181	−7387.44
1	C16	DCON4	55.5	−890.66	−84.04	1.09	−0.782	0.957	−2723.16
1	C16	DCON4	111	−888.35	−84.04	1.09	−0.782	−59.267	1941.116
1	C17	DCON4	0	−929.77	−98.29	2.39	−1.4	129.077	−8930.24
1	C17	DCON4	51	−927.12	−98.29	2.39	−1.4	7.056	−3917.22
1	C17	DCON4	102	−924.47	−98.29	2.39	−1.4	−114.966	1095.79
1	C19	DCON4	0	−775.64	−94.01	−0.82	−0.782	−29.535	−7866.86
1	C19	DCON4	55.5	−773.33	−94.01	−0.82	−0.782	16.19	−2649.33
1	C19	DCON4	111	−771.02	−94.01	−0.82	−0.782	61.915	2568.209
1	C20	DCON4	0	−747.74	−2.24	18.36	−0.782	1282.379	−66.86
1	C20	DCON4	51	−745.62	−2.24	18.36	−0.782	346.032	47.32
1	C20	DCON4	102	−743.5	−2.24	18.36	−0.782	−590.315	161.5
1	C43	DCON4	0	−200.97	0.94	9.89	−0.357	738.543	29.248
1	C43	DCON4	55.5	−199.43	0.94	9.89	−0.357	189.603	−22.972
1	C43	DCON4	111	−197.89	0.94	9.89	−0.357	−359.338	−75.191

(continued)

Table 5.1 (continued)

Story	Column	Load	Loc	P	V2	V3	T	M2	M3
1	C55	DCON4	0	−736.92	−10.69	22.23	−0.782	1638.092	−501.334
1	C55	DCON4	55.5	−734.61	−10.69	22.23	−0.782	404.079	91.69
1	C55	DCON4	111	−732.3	−10.69	22.23	−0.782	−829.935	684.715
1	C57	DCON4	0	−1026.4	−2.3	17.02	−0.782	1218.63	−108.594
1	C57	DCON4	51	−1024.28	−2.3	17.02	−0.782	350.493	8.556
1	C57	DCON4	102	−1022.16	−2.3	17.02	−0.782	−517.643	125.705
1	C58	DCON4	0	−832.4	−94.22	−1.42	−0.782	−69.375	−7876.46
1	C58	DCON4	55.5	−830.09	−94.22	−1.42	−0.782	9.16	−2647.09
1	C58	DCON4	111	−827.79	−94.22	−1.42	−0.782	87.695	2582.285
1	C60	DCON4	0	−578.76	−38.54	0.31	−0.357	16.79	−3325.51
1	C60	DCON4	55.5	−577.22	−38.54	0.31	−0.357	−0.25	−1186.44
1	C60	DCON4	111	−575.68	−38.54	0.31	−0.357	−17.289	952.634
1	C61	DCON4	0	−322.77	−34.91	−4.48	−0.357	−212.065	−3158.19
1	C61	DCON4	55.5	−321.23	−34.91	−4.48	−0.357	36.374	−1220.64
1	C61	DCON4	111	−319.7	−34.91	−4.48	−0.357	284.814	716.906
1	C62	DCON4	0	−1378.45	−85.17	0.05	−1.4	1.024	−8339.62
1	C62	DCON4	51	−1375.8	−85.17	0.05	−1.4	−1.604	−3995.97
1	C62	DCON4	102	−1373.2	−85.17	0.05	−1.4	−4.231	347.672
1	C63	DCON4	0	−749.12	−84.75	0.92	−0.782	41.453	−7419.4
1	C63	DCON4	55.5	−746.82	−84.75	0.92	−0.782	−9.451	−2715.7
1	C63	DCON4	111	−744.51	−84.75	0.92	−0.782	−60.355	1987.96
1	C64	DCON4	0	−628.35	−77.85	1.14	−0.782	50.227	−7108.7
1	C64	DCON4	55.5	−626.04	−77.85	1.14	−0.782	−12.815	−2788.2
1	C64	DCON4	111	−623.73	−77.85	1.14	−0.782	−75.857	1532.31

(continued)

Table 5.1 (continued)

Story	Column	Load	Loc	P	V2	V3	T	M2	M3
1	C85	DCON4	0	−1430.9	−5.3	27.79	−1.807	2673.37	−80.033
1	C85	DCON4	72	−1426.4	−5.3	27.79	−1.807	672.252	301.889
1	C85	DCON4	144	−1421.9	−5.3	27.79	−1.807	−1328.9	683.81
1	C86	DCON4	0	−1184.5	−2.04	19.77	−1.4	2067.68	−16.617
1	C86	DCON4	72	−1180.8	−2.04	19.77	−1.4	644.09	130.17
1	C86	DCON4	144	−1177	−2.04	19.77	−1.4	−779.5	276.957
1	C87	DCON4	0	−1732.4	−6.4	41.19	−2.904	4283.1	−146.71
1	C87	DCON4	72	−1727	−6.4	41.19	−2.904	1317.2	314.254
1	C87	DCON4	144	−1721.6	−6.4	41.19	−2.904	−1648.7	775.222
1	C90	DCON4	0	−1865.4	−7.16	36.5	−2.904	4062.97	−324.27
1	C90	DCON4	72	−1860.1	−7.16	36.5	−2.904	1434.86	190.957
1	C90	DCON4	144	−1854.7	−7.16	36.5	−2.904	−1193.2	706.183
1	C91	DCON4	0	−1225.8	−4.03	20.23	−1.4	2089.26	−197.98
1	C91	DCON4	72	−1222	−4.03	20.23	−1.4	632.786	92.141
1	C91	DCON4	144	−1218.3	−4.03	20.23	−1.4	−823.69	382.265
1	C92	DCON4	0	−1504	−6.45	28.67	−1.807	2714.84	−332.55
1	C92	DCON4	72	−1499.5	−6.45	28.67	−1.807	650.535	132.15
1	C92	DCON4	144	−1495	−6.45	28.67	−1.807	−1413.8	596.848
1	C94	DCON4	0	−1040.1	5.93	12.54	−0.782	1174.32	346.555
1	C94	DCON4	72	−1037.1	5.93	12.54	−0.782	271.188	−80.185
1	C94	DCON4	144	−1034.1	5.93	12.54	−0.782	−631.94	−506.93
1	C95	DCON4	0	−1562	7.46	39.03	−2.904	4173.21	508.001
1	C95	DCON4	72	−1556.6	7.46	39.03	−2.904	1363.05	−29.365
1	C95	DCON4	144	−1551.2	7.46	39.03	−2.904	−1447.1	−566.73

(continued)

Table 5.1 (continued)

Story	Column	Load	Loc	P	V2	V3	T	M2	M3
1	C96	DCON4	0	−1290.3	5.21	21.34	−1.807	2363.44	357.533
1	C96	DCON4	72	−1285.8	5.21	21.34	−1.807	827.196	−17.826
1	C96	DCON4	144	−1281.3	5.21	21.34	−1.807	−709.05	−393.18
1	C97	DCON4	0	−1771.6	12.27	51.46	−2.904	4756.59	668.848
1	C97	DCON4	72	−1766.2	12.27	51.46	−2.904	1051.22	−214.52
1	C97	DCON4	144	−1760.9	12.27	51.46	−2.904	−2654.2	−1097.9
1	C100	DCON4	0	−1820	3.64	37.97	−2.904	4123.64	147.462
1	C100	DCON4	72	−1814.6	3.64	37.97	−2.904	1389.55	−114.89
1	C100	DCON4	144	−1809.2	3.64	37.97	−2.904	−1344.5	−377.24
1	C101	DCON4	0	−1257	2.37	22.32	−1.807	2409.79	74.91
1	C101	DCON4	72	−1252.5	2.37	22.32	−1.807	802.914	−96.008
1	C101	DCON4	144	−1248	2.37	22.32	−1.807	−803.96	−266.93
1	C102	DCON4	0	−1478.7	5.3	39.66	−2.904	4202.99	170.903
1	C102	DCON4	72	−1473.3	5.3	39.66	−2.904	1347.13	−210.96
1	C62	DCON4	51	−1375.8	−85.17	0.05	−1.4	−1.604	−3995.97
1	C102	DCON4	144	−1467.87	5.3	39.66	−2.904	−1508.73	−592.813
1	C103	DCON4	0	−529	2.08	10.08	−0.782	1057.228	60.473
1	C103	DCON4	72	−526.01	2.08	10.08	−0.782	331.507	−89.191
1	C103	DCON4	144	−523.02	2.08	10.08	−0.782	−394.213	−238.855
1	C104	DCON4	0	−547.39	−4.75	25.74	−1.4	2350.522	−255.188
1	C104	DCON4	55.5	−544.5	−4.75	25.74	−1.4	921.874	8.565
1	C104	DCON4	111	−541.62	−4.75	25.74	−1.4	−506.774	272.318
1	C105	DCON4	0	−1247.61	−2.48	30.89	−1.4	2593.987	−11.693
1	C105	DCON4	55.5	−1244.72	−2.48	30.89	−1.4	879.364	125.819

(continued)

Table 5.1 (continued)

Story	Column	Load	Loc	P	V2	V3	T	M2	M3
1	C105	DCON4	111	−1241.84	−2.48	30.89	−1.4	−835.259	263.331
1	C112	DCON4	0	−366.77	−56.24	−0.5	−0.782	−28.968	−6165.16
1	C112	DCON4	55.5	−364.46	−56.24	−0.5	−0.782	−1.315	−3043.94
1	C112	DCON4	111	−362.15	−56.24	−0.5	−0.782	26.339	77.285
1	C113	DCON4	0	−924.27	−63.84	0.13	−0.782	19.146	−6507.29
1	C113	DCON4	55.5	−921.96	−63.84	0.13	−0.782	11.705	−2964.11
1	C113	DCON4	111	−919.65	−63.84	0.13	−0.782	4.264	579.079
1	C135	DCON4	0	−1523.24	−6.55	51.05	−2.904	4746.229	−198.188
1	C135	DCON4	55.5	−1519.09	−6.55	51.05	−2.904	1912.817	165.584
1	C135	DCON4	111	−1514.93	−6.55	51.05	−2.904	−920.596	529.357
1	C136	DCON4	0	−1007.11	−4.01	47.2	−2.904	4571.054	−87.081
1	C136	DCON4	51	−1003.3	−4.01	47.2	−2.904	2163.826	117.47
1	C136	DCON4	102	−999.48	−4.01	47.2	−2.904	−243.401	322.021
1	C137	DCON4	0	−1355.03	−7	48.23	−2.904	4613.74	−272.415
1	C137	DCON4	55.5	−1350.88	−7	48.23	−2.904	1937.044	115.813
1	C137	DCON4	111	−1346.73	−7	48.23	−2.904	−739.653	504.04
1	C138	DCON4	0	−1234	−2.28	58.28	−2.904	5090.971	−66.569
1	C138	DCON4	51	−1230.19	−2.28	58.28	−2.904	2118.616	49.839
1	C138	DCON4	102	−1226.37	−2.28	58.28	−2.904	−853.74	166.247
1	C139	DCON4	0	−2230.49	9.25	114.61	−5.884	10834.75	521.708
1	C139	DCON4	51	−2225.4	9.25	114.61	−5.884	4989.799	49.74
1	C139	DCON4	102	−2220.31	9.25	114.61	−5.884	−855.152	−422.229
1	C141	DCON4	0	−2040.27	5.5	123.6	−5.884	11248.98	284.304
1	C141	DCON4	51	−2035.18	5.5	123.6	−5.884	4945.571	3.933

(continued)

Table 5.1 (continued)

Story	Column	Load	Loc	P	V2	V3	T	M2	M3
1	C141	DCON4	102	−2030.09	5.5	123.6	−5.884	−1357.84	−276.438
1	C142	DCON4	0	−1188.93	5.62	37.3	−2.904	4086.5	333.545
1	C142	DCON4	51	−1185.11	5.62	37.3	−2.904	2184.243	46.98
1	C142	DCON4	102	−1181.3	5.62	37.3	−2.904	281.986	−239.585
1	C143	DCON4	0	−1457.89	1.85	54.26	−2.904	4882.198	113.937
1	C143	DCON4	51	−1454.07	1.85	54.26	−2.904	2115.052	19.589
1	C143	DCON4	102	−1450.25	1.85	54.26	−2.904	−652.095	−74.759

Table 5.2 Beam forces (*Units* Kips-in)

Story	Beam	Load	Loc	P	V2	V3	T	M2	M3
1	B6	DCON4	15	0	−16.66	0	−230.581	0	−132.448
1	B6	DCON4	31.069	0	−14.54	0	−230.581	0	118.27
1	B6	DCON4	47.138	0	−12.41	0	−230.581	0	334.835
1	B6	DCON4	47.138	0	−5.58	0	−371.183	0	340.548
1	B6	DCON4	70.708	0	−2.46	0	−371.183	0	435.312
1	B6	DCON4	94.277	0	0.66	0	−371.183	0	456.604
1	B6	DCON4	94.277	0	11.17	0	−643.509	0	453.64
1	B6	DCON4	117.846	0	14.29	0	−643.509	0	153.632
1	B6	DCON4	141.415	0	17.4	0	−643.509	0	−219.847
1	B6	DCON4	141.415	0	26.91	0	−140.313	0	−255.813
1	B6	DCON4	157.054	0	28.98	0	−140.313	0	−692.765
1	B6	DCON4	172.692	0	31.04	0	−140.313	0	−1162.06
1	B6	DCON4	172.692	0	−68.15	0	−55.048	0	−3065.65
1	B6	DCON4	188.554	0	−66.05	0	−55.048	0	−2001.36
1	B6	DCON4	188.554	0	−76.58	0	1390.608	0	−1825.82
1	B6	DCON4	212.123	0	−73.47	0	1390.608	0	−57.554
1	B6	DCON4	235.692	0	−70.35	0	1390.608	0	1637.237
1	B6	DCON4	235.692	0	−3.71	0	−12.289	0	1618.84
1	B6	DCON4	253.941	0	−1.3	0	−12.289	0	1664.609
1	B6	DCON4	272.19	0	1.11	0	−12.289	0	1666.332
1	B6	DCON4	272.19	0	2.05	0	−38.395	0	1661.158
1	B6	DCON4	283.013	0	3.48	0	−38.395	0	1631.276
1	B6	DCON4	283.013	0	7.08	0	−68.426	0	1613.23
1	B6	DCON4	295.851	0	8.78	0	−68.426	0	1511.452
1	B6	DCON4	308.688	0	10.48	0	−68.426	0	1387.877
1	B6	DCON4	308.688	0	12.48	0	−110.971	0	1360.325
1	B6	DCON4	330.334	0	15.35	0	−110.971	0	1059.087
1	B6	DCON4	330.334	0	16.3	0	−178.278	0	1001.536
1	B6	DCON4	345.186	0	18.26	0	−178.278	0	744.911
1	B6	DCON4	345.186	0	24.24	0	−269.154	0	686.303
1	B6	DCON4	361.421	0	26.38	0	−269.154	0	275.411
1	B6	DCON4	377.655	0	28.53	0	−269.154	0	−170.341
1	B6	DCON4	377.655	0	5.85	0	−650.067	0	−271.718
1	B6	DCON4	381.684	0	6.38	0	−650.067	0	−296.344
1	B6	DCON4	381.684	0	91.54	0	−1189.3	0	−465.318
1	B6	DCON4	396.312	0	93.47	0	−1189.3	0	−1818.46
1	B6	DCON4	410.94	0	95.41	0	−1189.3	0	−3199.89
1	B6	DCON4	410.94	0	−20.46	0	−1042.74	0	−461.98
1	B6	DCON4	424.977	0	−18.61	0	−1042.74	0	−187.793
1	B6	DCON4	424.977	0	−2.42	0	−578.513	0	−153.61

(continued)

Table 5.2 (continued)

1	B6	DCON4	427.998	0	−2.03	0	−578.513	0	−146.887
1	B6	DCON4	427.998	0	−13.8	0	101.379	0	−54.885
1	B6	DCON4	450.148	0	−10.87	0	101.379	0	218.424
1	B6	DCON4	472.298	0	−7.94	0	101.379	0	426.844
1	B6	DCON4	474.312	0	−0.19	0	190.593	0	440.499
1	B6	DCON4	474.312	0	0.71	0	259.621	0	442.13
1	B6	DCON4	496.965	0	3.7	0	259.621	0	392.193
1	B6	DCON4	519.619	0	6.7	0	259.621	0	274.383
1	B6	DCON4	519.619	0	0.05	0	171.95	0	271.008
1	B6	DCON4	520.626	0	0.19	0	171.95	0	270.887
1	B6	DCON4	520.626	0	13.26	0	70.633	0	259.968
1	B6	DCON4	540.033	0	15.83	0	70.633	0	−22.326
1	B6	DCON4	559.44	0	18.4	0	70.633	0	−354.433

5.6.3 Point Displacement

See Table 5.3.

Table 5.3 Point displacement (inch)

Story	Point	Load	UX	UY	UZ	RX	RY	RZ
15	1	EQX	3.5407	−0.0633	0.1558	0.00012	0.00054	0.00006
15	1	EQY	0	2.2306	−0.1464	−0.00073	−0.00006	0
15	2	EQX	3.5407	−0.0119	−0.0214	−0.00007	0.00047	0.00006
15	2	EQY	0	2.2306	−0.1708	−0.00081	0.00002	0
15	13	EQX	3.5657	−0.0119	0.0093	0.00006	0.00044	0.00006
15	13	EQY	0	2.2306	0.0443	−0.00068	0.00003	0
15	14	EQX	3.5731	−0.0492	0.0333	−0.00012	0.00027	0.00006
15	14	EQY	0	2.2306	0.1117	−0.00075	0.00003	0
15	15	EQX	3.5407	−0.0525	0.069	0.00035	0.0006	0.00006
15	15	EQY	0	2.2306	−0.1371	−0.00062	−0.00001	0
15	16	EQX	3.5407	−0.0412	0.0167	0.00009	0.00053	0.00006
15	16	EQY	0	2.2306	−0.139	−0.00072	0.00003	0
15	17	EQX	3.5407	−0.0313	−0.0439	−0.00018	0.00054	0.00006
15	17	EQY	0	2.2306	−0.1528	−0.00071	0.00004	0
15	19	EQX	3.5753	−0.0412	−0.0095	0	0.00047	0.00006
15	19	EQY	0	2.2306	0.1503	−0.00072	−0.00007	0
15	20	EQX	3.5753	−0.0313	−0.0376	0.00009	0	0.00006
15	20	EQY	0	2.2306	0.1653	−0.00076	−0.00006	0
15	21	EQX	3.5753	−0.0119	−0.0158	0.00009	0.00037	0.00006

(continued)

Table 5.3 (continued)

15	21	EQY	0	2.2306	0.1643	−0.00084	0.00002	0
15	42	EQX	3.5407	0.0119	0.0214	0.00007	0.00047	0.00006
15	42	EQY	0	2.2306	−0.1708	−0.00081	−0.00002	0
15	43	EQX	3.5407	0.0633	−0.1558	−0.00012	0.00054	0.00006
15	43	EQY	0	2.2306	−0.1464	−0.00073	0.00006	0
15	50	EQX	3.5657	0.0119	−0.0093	−0.00006	0.00044	0.00006
15	50	EQY	0	2.2306	0.0443	−0.00068	−0.00003	0
15	55	EQX	3.5731	0.0492	−0.0333	0.00012	0.00027	0.00006
15	55	EQY	0	2.2306	0.1117	−0.00075	−0.00003	0
15	56	EQX	3.5753	0.0119	0.0158	−0.00009	0.00037	0.00006
15	56	EQY	0	2.2306	0.1643	−0.00084	−0.00002	0
15	57	EQY	0	2.2306	0.1653	−0.00076	0.00006	0
15	58	EQX	3.5753	0.0412	0.0095	0	0.00047	0.00006
15	58	EQY	0	2.2306	0.1503	−0.00072	0.00007	0
15	60	EQX	3.57	−0.0064	0.0459	−0.00063	0.0006	0.00006
15	60	EQY	0	2.2306	0.0735	−0.00052	0	0
15	61	EQX	3.57	0.0064	−0.0459	0.00063	0.0006	0.00006
15	61	EQY	0	2.2306	0.0735	−0.00052	0	0
15	62	EQX	3.5407	0.0313	0.0439	0.00018	0.00054	0.00006
15	62	EQY	0	2.2306	−0.1528	−0.00071	−0.00004	0
15	63	EQX	3.5407	0.0412	−0.0167	−0.00009	0.00053	0.00006
15	63	EQY	0	2.2306	−0.139	−0.00072	−0.00003	0
15	64	EQX	3.5407	0.0525	−0.069	−0.00035	0.0006	0.00006
15	64	EQY	0	2.2306	−0.1371	−0.00062	0.00001	0
15	67	EQX	3.5753	−0.0492	0.0388	−0.00014	0.00033	0.00006
15	67	EQY	0	2.2306	0.1395	−0.00076	−0.00007	0
15	68	EQX	3.5609	−0.0492	−0.0093	−0.0001	0.00017	0.00006
15	68	EQY	0	2.2306	0.0077	−0.00038	−0.00002	0
15	69	EQX	3.5676	−0.0525	0.007	−0.00002	0.00053	0.00006
15	69	EQY	0	2.2306	0.0745	−0.00071	−0.00018	0
15	70	EQX	3.5609	−0.0525	0.008	0.00003	0.00049	0.00006
15	70	EQY	0	2.2306	0.0064	−0.00026	−0.00002	0
15	71	EQX	3.5731	−0.0577	0.0643	−0.00024	0.00039	0.00006
15	71	EQY	0	2.2306	0.1174	−0.00067	0.00001	0
15	72	EQX	3.5676	−0.0577	0.0472	−0.00005	0.00051	0.00006
15	72	EQY	0	2.2306	0.0566	−0.00071	−0.0002	0
15	75	EQX	3.5731	−0.0633	0.1184	0.00008	0.00079	0.00006
15	75	EQY	0	2.2306	0.1172	−0.0008	−0.00001	0
15	76	EQX	3.5609	−0.0119	0.0109	0	0.00016	0.00006
15	76	EQY	0	2.2306	0.0102	−0.00031	0.00003	0
15	77	EQX	3.5605	−0.0633	0.1297	0.00002	0.00079	0.00006

(continued)

Table 5.3 (continued)

15	77	EQY	0	2.2306	0.002	0	−0.00002	0
15	78	EQX	3.5605	−0.0525	0.0082	0.00002	0.00047	0.00006
15	78	EQY	0	2.2306	0.0046	−0.00021	−0.00002	0
15	79	EQX	3.5543	−0.0633	0.1321	0.00002	0.00073	0.00006
15	79	EQY	0	2.2306	0.0015	−0.00022	0.00005	0
15	80	EQX	3.5543	−0.0525	0.0093	0.00005	0.00091	0.00006
15	80	EQY	0	2.2306	−0.0045	−0.00022	0.00001	0
15	81	EQX	3.552	−0.0313	−0.0145	−0.00004	0.00026	0.00006
15	81	EQY	0	2.2306	−0.0323	−0.00051	0.00003	0
15	82	EQX	3.552	−0.0119	0.007	−0.00005	0.00043	0.00006
15	82	EQY	0	2.2306	−0.0422	−0.00068	0	0
15	83	EQX	3.5676	−0.0633	0.1264	0.00006	0.00155	0.00006
15	83	EQY	0	2.2306	0.0423	−0.00096	−0.0001	0
15	84	EQX	3.5676	−0.0543	0.0224	0.00008	0.00048	0.00006
15	84	EQY	0	2.2306	0.0684	−0.0007	−0.00021	0
15	85	EQX	3.5639	−0.0543	0.0343	0.00017	0.00095	0.00006
15	85	EQY	0	2.2306	0.0266	−0.00066	−0.00004	0
15	86	EQY	0	2.2306	0.0276	−0.00071	−0.00003	0
15	87	EQX	3.552	−0.0259	−0.0154	−0.0002	−0.00017	0.00006
15	87	EQY	0	2.2306	−0.0365	−0.00062	0.00005	0
15	88	EQX	3.5407	−0.0259	−0.0553	−0.00022	−0.00016	0.00006
15	88	EQY	0	2.2306	−0.1573	−0.00067	0.00006	0
15	89	EQX	3.552	−0.0166	0.018	−0.00014	−0.00007	0.00006
15	89	EQY	0	2.2306	−0.0416	−0.00065	0.00002	0
15	90	EQX	3.5407	−0.0166	−0.0118	−0.00017	−0.00012	0.00006
15	90	EQY	0	2.2306	−0.1674	−0.0007	0.00006	0
15	119	EQX	3.5753	0.0492	−0.0388	0.00014	0.00033	0.00006
15	119	EQY	0	2.2306	0.1395	−0.00076	0.00007	0
15	120	EQX	3.5609	0.0492	0.0093	0.0001	0.00017	0.00006
15	120	EQY	0	2.2306	0.0077	−0.00038	0.00002	0
15	121	EQX	3.5731	0.0577	−0.0643	0.00024	0.00039	0.00006
15	121	EQY	0	2.2306	0.1174	−0.00067	−0.00001	0
15	122	EQX	3.5676	0.0577	−0.0472	0.00005	0.00051	0.00006
15	122	EQY	0	2.2306	0.0566	−0.00071	0.0002	0
15	125	EQX	3.5731	0.0633	−0.1184	−0.00008	0.00079	0.00006
15	125	EQY	0	2.2306	0.1172	−0.0008	0.00001	0
15	126	EQX	3.5609	0.0119	−0.0109	0	0.00016	0.00006
15	126	EQY	0	2.2306	0.0102	−0.00031	−0.00003	0
15	127	EQX	3.5609	0.0525	−0.008	−0.00003	0.00049	0.00006
15	127	EQY	0	2.2306	0.0064	−0.00026	0.00002	0
15	128	EQX	3.5605	0.0525	−0.0082	−0.00002	0.00047	0.00006

(continued)

Table 5.3 (continued)

15	128	EQY	0	2.2306	0.0046	−0.00021	0.00002	0
15	129	EQX	3.5605	0.0633	−0.1297	−0.00002	0.00079	0.00006
15	129	EQY	0	2.2306	0.002	0	0.00002	0
15	130	EQX	3.5543	0.0525	−0.0093	−0.00005	0.00091	0.00006
15	130	EQY	0	2.2306	−0.0045	−0.00022	−0.00001	0
15	131	EQX	3.5543	0.0633	−0.1321	−0.00002	0.00073	0.00006
15	131	EQY	0	2.2306	0.0015	−0.00022	−0.00005	0
15	132	EQX	3.552	0.0119	−0.007	0.00005	0.00043	0.00006
15	132	EQY	0	2.2306	−0.0422	−0.00068	0	0
15	133	EQX	3.552	0.0313	0.0145	0.00004	0.00026	0.00006
15	133	EQY	0	2.2306	−0.0323	−0.00051	−0.00003	0
15	134	EQX	3.5676	0.0525	−0.007	0.00002	0.00053	0.00006
15	134	EQY	0	2.2306	0.0745	−0.00071	0.00018	0
15	135	EQX	3.5676	0.0633	−0.1264	−0.00006	0.00155	0.00006
15	135	EQY	0	2.2306	0.0423	−0.00096	0.0001	0
15	136	EQX	3.5676	0.0543	−0.0224	−0.00008	0.00048	0.00006
15	136	EQY	0	2.2306	0.0684	−0.0007	0.00021	0
15	137	EQX	3.5639	0.0543	−0.0343	−0.00017	0.00095	0.00006
15	137	EQY	0	2.2306	0.0266	−0.00066	0.00004	0
15	138	EQX	3.5639	0.0525	−0.0054	−0.00004	0.00108	0.00006
15	138	EQY	0	2.2306	0.0276	−0.00071	0.00003	0
15	139	EQX	3.552	0.0259	0.0154	0.0002	−0.00017	0.00006
15	139	EQY	0	2.2306	−0.0365	−0.00062	−0.00005	0
15	140	EQY	0	2.2306	−0.1573	−0.00067	−0.00006	0
15	141	EQX	3.552	0.0166	−0.018	0.00014	−0.00007	0.00006
15	141	EQY	0	2.2306	−0.0416	−0.00065	−0.00002	0
15	142	EQX	3.5407	0.0166	0.0118	0.00017	−0.00012	0.00006
15	142	EQY	0	2.2306	−0.1674	−0.0007	−0.00006	0
15	145	EQX	3.5657	0.0064	0.0027	0.00066	0.00001	0.00006
15	145	EQY	0	2.2306	0.0396	−0.00048	−0.00004	0
15	146	EQX	3.5657	−0.0064	−0.0027	−0.00066	0.00001	0.00006
15	146	EQY	0	2.2306	0.0396	−0.00048	0.00004	0
15	148	EQX	3.5609	−0.0412	−0.0108	0.00002	0.00001	0.00006
15	148	EQY	0	2.2306	0.0116	−0.00024	−0.00003	0
15	149	EQX	3.5609	−0.0313	−0.0155	0.00005	0.00004	0.00006
15	149	EQY	0	2.2306	0.015	−0.00037	−0.00001	0
15	179	EQX	3.5609	0.0412	0.0108	−0.00002	0.00001	0.00006
15	179	EQY	0	2.2306	0.0116	−0.00024	0.00003	0
15	180	EQX	3.5609	0.0313	0.0155	−0.00005	0.00004	0.00006
15	180	EQY	0	2.2306	0.015	−0.00037	0.00001	0
15	203	EQX	3.5648	−0.0525	0.0054	0.00002	0.00144	0.00006

(continued)

Table 5.3 (continued)

15	203	EQY	0	2.2306	0.0386	−0.0009	−0.00003	0
15	204	EQX	3.5648	−0.0412	−0.0104	0.00001	0.00062	0.00006
15	204	EQY	0	2.2306	0.0371	−0.00066	−0.00002	0
15	205	EQX	3.5648	−0.0313	−0.0192	0.00006	0.00064	0.00006
15	205	EQY	0	2.2306	0.0457	−0.00069	−0.00001	0
15	208	EQX	3.5648	0.0313	0.0192	−0.00006	0.00064	0.00006
15	208	EQY	0	2.2306	0.0457	−0.00069	0.00001	0
15	209	EQX	3.5648	0.0412	0.0104	−0.00001	0.00062	0.00006
15	209	EQY	0	2.2306	0.0371	−0.00066	0.00002	0
15	210	EQX	3.5648	0.0525	−0.0054	−0.00002	0.00144	0.00006
15	210	EQY	0	2.2306	0.0386	−0.0009	0.00003	0
15	212	EQX	3.5503	−0.0633	0.1341	0.00008	0.00211	0.00006
15	212	EQY	0	2.2306	−0.0326	−0.00092	0	0
15	213	EQX	3.5503	−0.0525	0.0151	0.0001	0.0016	0.00006
15	213	EQY	0	2.2306	−0.0368	−0.00089	0	0
15	214	EQX	3.5503	−0.0412	0.0032	0.00006	0.00065	0.00006
15	214	EQY	0	2.2306	−0.0319	−0.00072	0.00001	0
15	215	EQX	3.5503	−0.0313	−0.016	−0.00005	0.00053	0.00006
15	215	EQY	0	2.2306	−0.049	−0.00068	0.00002	0
15	218	EQX	3.5503	0.0313	0.016	0.00005	0.00053	0.00006
15	218	EQY	0	2.2306	−0.049	−0.00068	−0.00002	0
15	219	EQX	3.5503	0.0412	−0.0032	−0.00006	0.00065	0.00006
15	219	EQY	0	2.2306	−0.0319	−0.00072	−0.00001	0
15	220	EQX	3.5503	0.0525	−0.0151	−0.0001	0.0016	0.00006
15	220	EQY	0	2.2306	−0.0368	−0.00089	0	0
15	221	EQX	3.5503	0.0633	−0.1341	−0.00008	0.00211	0.00006
15	221	EQY	0	2.2306	−0.0326	−0.00092	0	0

5.7 Seismic Calculation Formulas Used by Softwares

$\mathrm{Ta} = \mathrm{Ct}\ (\mathrm{hn}^{3/4})$

If $Z \geq 0.35$(Zone 4) then: If Tetabs ≤ 1.30 Ta then T = Tetabs, else T = Ta

If $Z < 0.35$(Zone 1, 2 or 3) then: If Tetabs ≤ 1.40 Ta then T = Tetabs, else T = Ta

$$V = (\mathrm{Cv\ I\ W})/(\mathrm{R\ T}) \tag{5.6}$$

$$V \leq 2.5\ \mathrm{CaI\ W\ /\ R} \tag{5.7}$$

$$V \geq 0.11 \text{ CaI W} \tag{5.8}$$

If T $\leq$ 0.7 sec, then Ft = 0
If T $>$ 0.7 sec, then Ft = 0.07 T V $\leq$ 0.25 V

5.7.1 Auto Seismic Calculation Results

Ta = 1.3748 s
T Used = 1.9248 s
W Used = 46927.52 lb.
V (Eq. 5.6) = 0.0302 W
V (Eq. 5.7) = 0.1091 W
V (Eq. 5.8) = 0.0264 W
V (Eq. 5.4) = 0.0465 W
V Used = 0.0302 W = 1418.51
Ft Used = 191.12

5.8 Conclusions on the Basis of Structural Analysis

After analyzing the building for lateral loads (earthquake), all the columns were failed because of the interaction of axial force and biaxial moment and some of the beams were failed in shear. It was also observed that sway of the building and story drift was under control. The sway of the building was 2.4 in which is less than 0.025 times the story height (3.15 in) of the structure as defined in UBC 97. The time period of the building is 1.3 s which is greater than 0.7 s. Therefore the story drift of the structure is also checked which is 0.006 in and is less than 0.020 times the story height (2.52 in.) as defined in UBC 97.

The structural members which are deficit will be retrofitted by suitable technique, which are discussed in detail in Chap. 6. The analysis outputs of column C-1 and beam B6 are attached to the chapter.

Reference

AISC (American Institute of Steel Construction) (1997) Seismic Provisions for Structural Steel Buildings, Chicago

Chapter 6
Retrofitting of the Building for Earth Quake

Abstract The considered structure yields dispersed at member level throughout the building in the analysis for the earthquake loads. The intensity of failure is found to be highest at ground level and diminishes at above stories. Moreover, it is found in the analysis that the lateral earthquake forces have induced huge axial loads in cuts and large shear forces in beams, as a result of which certain beams are found overstressed in shear. Different strengthening techniques have been proposed in this chapter for the considering structure to improve its seismic resistance. In the first solution use of cross bracings are proposed in order to reduce the effect of lateral loads in the building. After providing the cross bracings, many columns are found over stressed. Hence, reinforced concrete jacketing is proposed as it improves column Axial as well as flexural strength and ductility. In another proposed solution, addition of new reinforced concrete shear walls is considered as it provides the best option of strengthening an existing structure for improved seismic performance. It adds significant strength and stiffness to framed structures.

6.1 Introduction

The analysis of building for the earthquake loads yields dispersed at member level throughout the building. The intensity of failure is maximum at ground and vanishes at above stories. The lateral earthquake forces have induced huge axial loads in cuts and huge shear forces in beams, as a result of which certain beams are found overstressed in shear. In this chapter, strengthening techniques has been proposed for the structure.

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Solid Mechanics and Its Applications 222, DOI 10.1007/978-94-017-7297-6_6

6.2 Retrofitting Solution 1

6.2.1 Use of Cross Bracings Together with Column Strengthening

In order to reduce the effect of lateral loads in the building, use of cross bracings are proposed. This can be done by utilizing available free bays for cross bracings. In the building i.e. is under consideration, bracings are proposed in the bays marked in the Fig. 6.1.

The maximum axial force is found to be 407 kips in bracings along grid-B2 between column C113 and C14, as shown in the elevation given (Fig. 6.2).

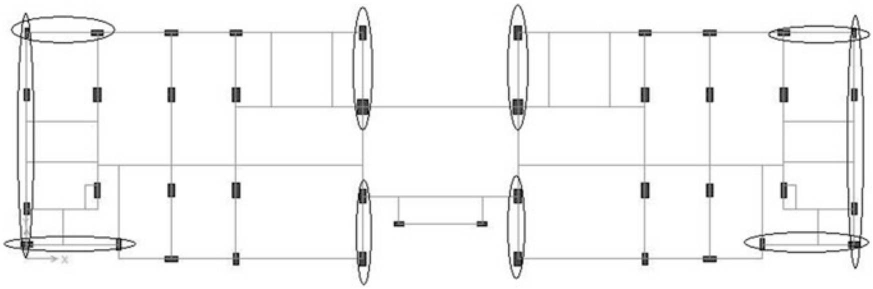
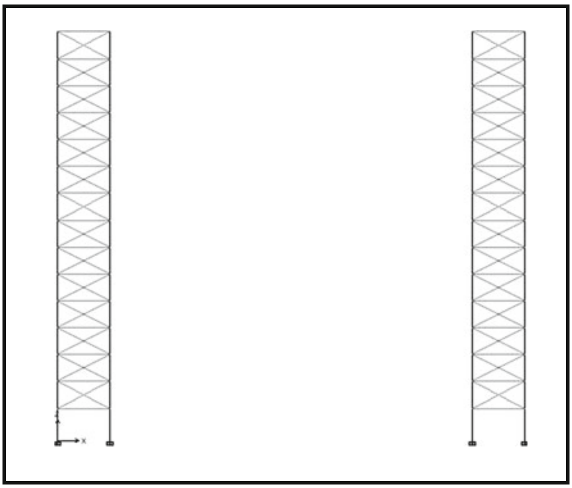


Fig. 6.1 Proposed bays for the application of bracings

Fig. 6.2 Elevation grid B2



6.2.1.1 Design of Bracing

Yield strength of bracings = 40 ksi.
Area of steel bracing = axial load/∅ * yield strength
And is the capacity reduction factor, ∅ = 0.9
Area of steel bracing = 407/0.9*40
=11.1667 in^2
for consideration of area reduction due to holes at joints and connections, the area of steel bracing is enhanced by 1.33
11.1667 * 1.33 = 15 in^2
A channel section would be a suitable section, for bracing due to ease in connection with gusset plate,
From AISC (1997) Table 1–5 two C-section 10 × 30 having a cross-section area of 8.81 in^2, thickness of 0.673 in and depth = 10 in. would be suitable.

So the area of bracing will become 2 × 8.81 = 17.62 in^2. This area can be applied for all required bracings but for economical reason, it would be beneficial to determine required area for every bracing (Figs. 6.3 and 6.4).

6.2.1.2 Functioning of Bracings

Bracings are attached to the beam and column joint through gusset plates by welding or through bolts. A gusset plate assembly is attached with the plate with which the bracing is connected. These plates are then attached with columns and beams. This is illustrated in the Fig. 6.5.

When the lateral force acts, all of these bracings are in tension. By knowing the force in the bracing and its angle of inclination, connection can be analyzed for their connecting shear stressed. The vertical component of the force will act as a shear stress in column and its horizontal component will act as a shear stress in beam.

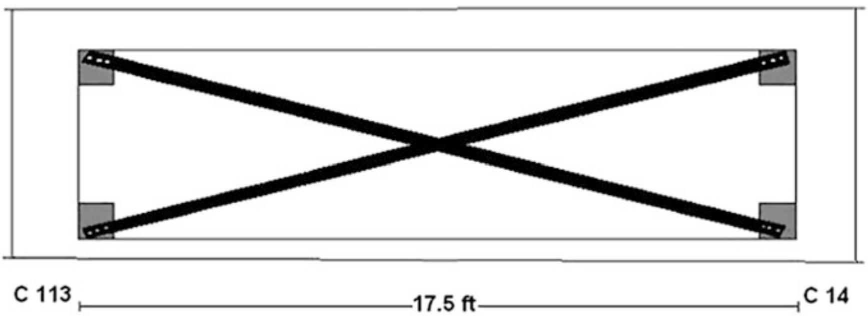


Fig. 6.3 Elevation of bay

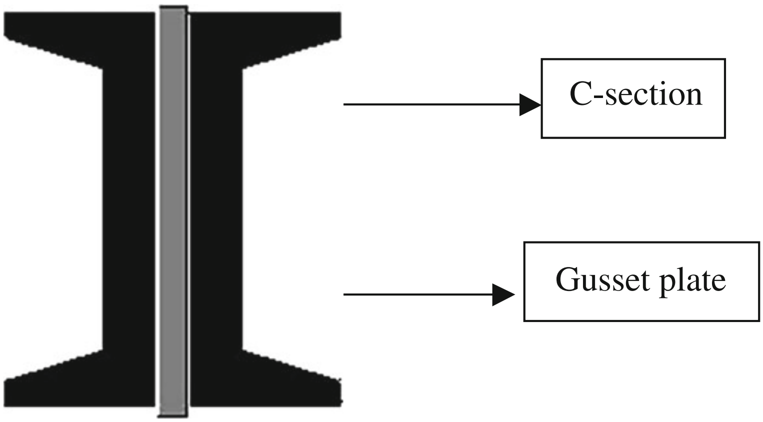


Fig. 6.4 Two C-sections with gusset plate

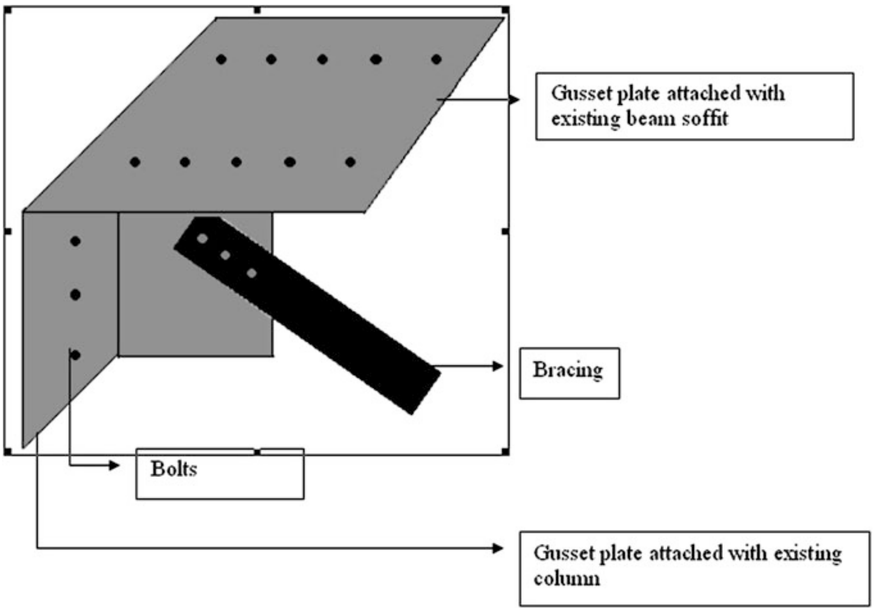


Fig. 6.5 Gusset plate attached with existing column

6.2.2 Column Strengthening

After providing the cross bracings, many columns of lower stories (story1-story5), are found over stressed and thus required strengthening. The failure of the columns are the result of interaction axial load and biaxial bending, having eccentricities in both principal axes of the section.

Column jacketing would be suitable solution for these columns. R.C or steel jackets may be used for strengthening of these existing columns. Jacket design of column C-1 is given below. However the technique would be the same for all columns.

6.2.2.1 Analysis Outputs

Pu = 660 kips
Mu2 = −594 Kips-in (Minor axis moment)
Mu3 = 2101.785 Kips-in (Major axis moment)
Section assigned = 10 × 24
Rebar Percent = 4 %

6.2.2.2 Column Capacity Using Reciprocal Load Method

$$\begin{aligned} &1/\mathrm{Pn} = 1/\mathrm{Pnx} + 1/\mathrm{Pny} - 1/\mathrm{Po} \\ &\mathrm{ex} = \mathrm{Mu3}/\mathrm{Pu} \quad = 0.9\,\mathrm{in} \\ &\mathrm{ey} = \mathrm{Mu2}/\mathrm{Pu} \quad\quad = 3.2\,\mathrm{in} \\ &\gamma = 0.8 \end{aligned} \tag{6.1}$$

Bending about y-axis:

$$\begin{aligned} &\mathrm{ey/h} = 3.18/24 = 0.1325 \\ &\mathrm{Puy}/\emptyset\mathrm{Ag\,fc'} = 1.06125(\text{from interaction charts}) \end{aligned} \tag{6.2}$$

$$\mathrm{Puo}/\emptyset\,\mathrm{Ag\,fc'} = 1.26(\text{from interaction charts}) \tag{6.3}$$

Bending about x-axis:

$$\begin{aligned} &\mathrm{Pux}/\emptyset\mathrm{fc'\,Ag} = 1.1(\text{from interaction charts}) \\ &\emptyset = 0.75 \end{aligned} \tag{6.4}$$

Putting these values in Eq. 6.1

Ag = 355 in^2
Area of existing column = 10 × 24 = 240 in^2
The load carrying capability of existing column for given eccentricities is comes out to be
Pu = 595.6 kips

6.2.2.3 RC Jacketing of Columns

Reinforced concrete jacketing improves column Axial and flexural strength and ductility. Closely spaced transverse reinforcement provided in the jacket improves the shear strength and ductility of the column. The procedure for reinforced concrete jacketing is:

(i) The seismic demand on the columns, in terms of axial load (*P*) and moment (*M*) is obtained.
(ii) The column size and section details are estimated for *P* and *M* as determined above.
(iii) In the jacket, the existing column size is deducted from the estimated column size, to obtain the amount of concrete and steel to be provided in addition to the already provided column.
(iv) larger section is selected and steel to be provided as follows to account for losses.

$$\mathrm{Ac} = 3/2\ \mathrm{Ac}' \tag{6.5}$$

$$\mathrm{As}' = 4/3\,\mathrm{As}' \tag{6.6}$$

(v) The spacing of ties to be provided in the jacket in order to avoid shear failure of column and provide adequate confinement to the longitudinal steel along the jacket is given as:

$$\mathrm{S} = \frac{\mathrm{Fy\,dh}^2}{\sqrt{\mathrm{fck\,tj}}} \tag{6.7}$$

(vi) If the transfer of axial load to new longitudinal steel is not critical then friction present at the interface can be relied on for the shear transfer, which can be enhanced by roughening the old surface.
(vii) Dowels that are epoxy grouted and bent into 90° hook can also employed to improve the anchorage of new concrete jacket.
(viii) In order to transfer the additional axial load from the old to the new longitudinal reinforcement, bent-down bars intermittent lap welded to bars of jacket and longitudinal bars in the existing column can be used. Moreover, bent-down bars help in good anchorage between existing and new concrete shown in Fig. 6.6.

6.2.2.4 The Minimum Specifications for Jacketing of Columns

i. Strength of the new materials must be equal or greater than those of the existing column. Concrete strength should be at least 5 MPa greater than the strength of the existing concrete.

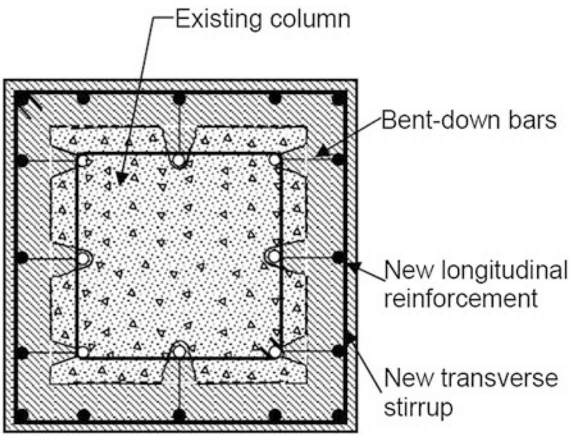


Fig. 6.6 Cross-section of jacketed column

ii. For columns where extra longitudinal reinforcement is not required, a minimum of 12∅ bars in the four corners and ties of 8∅ @ 100 c/c should be provided with 1350 bends and 10∅ leg lengths.
iii. Minimum jacket thickness should be 100 mm.
iv. Lateral support to all the longitudinal bars should be provided by ties with an included angle of not more than 135°.
v. Minimum diameter of ties should be 8 mm and not less than 1/3 of the longitudinal bar diameter.
vi. Vertical spacing of ties shall not exceed 200 mm, whereas the spacing close to the joints within a length of ¼ of the clear height should not exceed 100 mm. Preferably, the spacing of ties should not exceed the thickness of the jacket or 200 mm whichever is less.

6.2.2.5 Calculations of Concrete Jacket

From Sect. 6.2.2.1 (Analysis output) area of jacket required = 355 − 240 = 115 in^2
fcu = 3.5 ksi
Consider a 2″ thick jacket around the existing column
Area of jacket = (area of existing column + area of jacket) − (Area of existing column)
$= [(24 + 2 + 2) \times (10 + 2 + 2)] - (10 \times 24)$
$= 152\,\text{in}^2$
Using equation (Eq. 6.5) Ac = 152/1.5 = 101 in^2
Using 4 % area of steel in jacket
Area of steel = 0.04 × 152 = 6 in^2
Using equation (Eq. 6.6) As = 6 × 3/4 = 4.5 in^2
Now, the available total cross-sectional area of column would become

Ag = Area of existing column + Ac
=100 + 240 = 340 in^2
As = 4.5 + 0.04 × 240 = 14.1 in^2
Percentage of steel would become = 4 %
Now, again using the reciprocal load method, the capacity of the new section (having concrete jacket) for the given eccentricities, comes out to be,
Pn = 1344 kips
The spacing of new transverse stirrups is comes out to be from Eq. (6.7)
S = 5 in
The area of dowels required, to transfer the shear between old and new concrete, can be calculated as,
Area of Dowels = Additional load on jacket/fy
$= [1344 - 595.6/\emptyset]/40$
$= [1344 - 595.6/0.75]/40$
$= 14\,\text{in}^2$
Which should be evenly distributed (Fig. 6.7).

After the application of above proposal, the structure would be safe enough to bear the earthquake load without any global system strengthening, but many of the beams will require to be strengthened for shear, which will be discussed in the end of the chapter as a separate topic.

6.3 Retrofitting Solution 2

6.3.1 Use of R.C.C Shear Wall

Addition of new reinforced concrete shear walls provides the best option of strengthening an existing structure for improved seismic performance. It adds significant strength and stiffness to framed structures. The design of shear wall, for an existing structure can e made using ETABS (2008), but the proper consideration of its joint with existing structure would be very important.

The design of shear wall has done using software by inserting structural walls in 8 available free bays in outer perimeter of the building as shown in the Fig. 6.8,

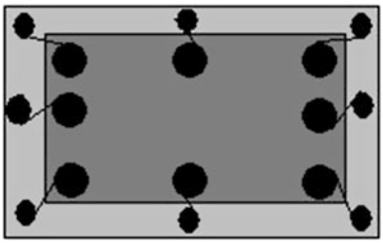


Fig. 6.7 Section of column

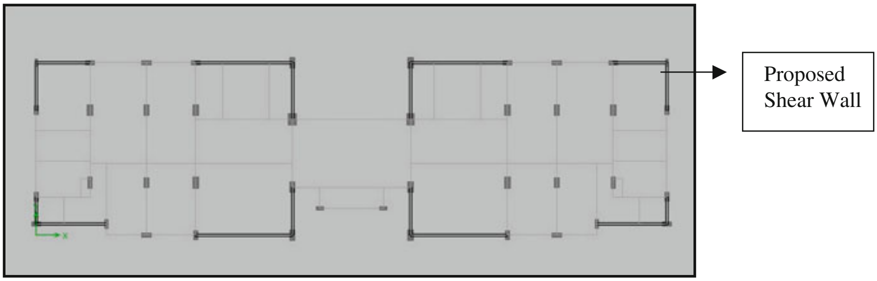


Fig. 6.8 Typical plan with shear walls

6.3.2 Analysis of Building with Proposed Shear Wall

The wall having marked red i.e. the pier label of W12 is the most critical member. Load combination COMB6 comprises of; Dead + live + Earthquake load (Table 6.1).

Given below is the location of W12 (Fig. 6.9).

6.3.3 Design Output for W-12

Story ID: 1 Pier ID: W12 X Loc: 1392 Y Loc: 0 Units: Kip-in

Flexural Design for P-M2-M3 (RLLF = 1.000)

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	Pu	M2u	M3u	Pier Ag
Top	0.0025	0.0044	COMB6	1443.466	0.000	24306.458	2544.000
Bottom	0.0025	0.0044	COMB6	1487.983	0.000	14016.634	2544.000

Shear Design

Station Location	Rebar in^2/ft	Shear Combo	Pu	Mu	Vu	Capacity phi Vc	Capacity phi Vn
Top Leg 1	0.312	COMB3	2180.157	71704.441	478.058	180.606	478.058
Bot Leg 1	0.312	COMB3	2224.674	140544.861	478.058	180.606	478.058

Boundary Element Check

Station Location	B-Zone Length	B-Zone Combo	Pu	Mu	Vu	Pu/Po
Top Leg 1	Not Checked	COMB4	3154.796	74367.511	469.227	0.3978
Bot Leg 1	Not Checked	COMB4	3199.313	141936.190	469.227	0.4034

Design Inadequacy Message: Pier Pu/Po < -0.35 !!

The design of proper connection between new shear wall and existing structure should be such that, it has capability of transferring shear from members to Wall.

Table 6.1 Shear wall forces (*units* kips-in)

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
1	**W12**	**COMB6**	**Top**	**−1443.47**	**−71.46**	**1.78**	**−50.536**	**39.629**	**24306.46**
1	**W12**	**COMB6**	**Bottom**	**−1487.98**	**−71.46**	**1.78**	**−50.536**	**295.512**	**14016.63**
1	W11	COMB4	Top	−701.36	−68.16	−1.49	−12.31	67.354	8846.05
1	W11	COMB4	Bottom	−723.44	−68.16	−1.49	−12.31	−146.686	−968.663
1	W12	COMB5	Top	−468.83	−62.63	2.69	−9.655	−47.502	21643.39
1	W12	COMB5	Bottom	−513.34	−62.63	2.69	−9.655	340.456	12625.31
1	W11	COMB3	Top	−213.34	−55.26	−1.5	−9.849	68.908	7291.961
1	W11	COMB3	Bottom	−235.42	−55.26	−1.5	−9.849	−147.607	−666.039
1	W5	COMB5	Top	−3401.63	−49.39	6.62	−164.197	−424.607	17643.66
1	W5	COMB5	Bottom	−3446.15	−49.39	6.62	−164.197	529.25	10531.91
1	W4	COMB6	Top	−1476.17	−47.55	1	−31.293	57.165	10042.86
1	W4	COMB6	Bottom	−1508.51	−47.55	1	−31.293	201.026	3195.928
1	W5	COMB6	Top	−4396.42	−42	8.13	−232.522	−570.461	15042.38
1	W5	COMB6	Bottom	−4440.94	−42	8.13	−232.522	600.978	8994.732
1	W4	COMB5	Top	−778.88	−40.78	1.48	−16.215	11.456	8708.375
1	W4	COMB5	Bottom	−811.22	−40.78	1.48	−16.215	224.649	2836.209
1	W16	COMB5	Top	−1613	−30.71	3.19	−45.01	−188.69	4809.165
1	W16	COMB5	Bottom	−1637.78	−30.71	3.19	−45.01	270.51	387.533
1	W11	COMB2	Top	−1389.92	−30.36	−0.11	−8.954	10.631	3904.12
1	W11	COMB2	Bottom	−1412	−30.36	−0.11	−8.954	−5.034	−467.729
1	W7	COMB2	Top	−1382.8	−30.27	0.1	8.53	−9.027	3916.681
1	W7	COMB2	Bottom	−1404.88	−30.27	0.1	8.53	4.795	−442.553
1	W3	COMB4	Top	−362.72	−30.03	−0.14	5.813	−29.609	3044.167
1	W3	COMB4	Bottom	−375.25	−30.03	−0.14	5.813	−49.35	−1280.7
1	W15	COMB3	Top	−1365.18	−28.57	−1.55	7.213	72.22	2391.776
1	W3	COMB3	Top	−96.85	−25.24	−0.34	3.537	−10.359	2630.766
1	W3	COMB3	Bottom	−109.38	−25.24	−0.34	3.537	−59.057	−1004.01
1	W16	COMB6	Top	−2094.48	−24.95	3.83	−62.551	−250.408	3914.961
1	W16	COMB6	Bottom	−2119.26	−24.95	3.83	−62.551	300.52	322.074
1	W15	COMB4	Top	−1796.99	−21.96	−1.95	16.431	111.259	1346.169
1	W15	COMB4	Bottom	−1818.83	−21.96	−1.95	16.431	−170.001	−1816.02
1	W6	COMB3	Top	−1941.4	−20.09	−1.87	1.853	88.976	4016.826
1	W6	COMB3	Bottom	−1967.33	−20.09	−1.87	1.853	−180.311	1123.381
1	W12	COMB2	Top	−2758.42	−19.73	−2.15	−96.251	205.072	6057.781
1	W12	COMB2	Bottom	−2802.94	−19.73	−2.15	−96.251	−103.868	3217.203
1	W9	COMB2	Top	−2964.81	−18.22	3.59	161.839	−345.426	5661.749
1	W9	COMB2	Bottom	−3009.33	−18.22	3.59	161.839	171.381	3038.466
1	W7	COMB1	Top	−899.93	−17.5	0.13	6.487	−12.157	2367.06
1	W7	COMB1	Bottom	−922	−17.5	0.13	6.487	6.215	−152.711
1	W11	COMB1	Top	−901.9	−17.47	−0.13	−6.492	12.185	2350.032
1	W11	COMB1	Bottom	−923.98	−17.47	−0.13	−6.492	−5.955	−165.106
1	W4	COMB2	Top	−2044.88	−16.25	−1.2	−37.678	114.455	3259.021
1	W4	COMB2	Bottom	−2077.22	−16.25	−1.2	−37.678	−57.725	919.388
1	W1	COMB2	Top	−1405.15	−15.12	1.48	41.183	−141.9	2222.026

(continued)

Table 6.1 (continued)

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
1	W1	COMB2	Bottom	−1429.93	−15.12	1.48	41.183	70.576	44.492
1	W14	COMB2	Top	−783.52	−11.37	−0.55	−6.455	52.897	997.711
1	W14	COMB2	Bottom	−796.05	−11.37	−0.55	−6.455	−26.289	−639.385
1	W3	COMB2	Top	−783.8	−11.14	0.55	6.439	−52.96	1000.246
1	W3	COMB2	Bottom	−796.33	−11.14	0.55	6.439	26.639	−603.442
1	W12	COMB1	Top	−1783.78	−10.89	−1.23	−55.369	117.94	3394.711
1	W12	COMB1	Bottom	−1828.3	−10.89	−1.23	−55.369	−58.925	1825.875
1	W9	COMB1	Top	−1972.26	−10.57	2.08	93.58	−199.563	3320.113
1	W9	COMB1	Bottom	−2016.77	−10.57	2.08	93.58	99.826	1798.337
1	W4	COMB1	Top	−1347.6	−9.48	−0.71	−22.599	68.745	1924.533
1	W4	COMB1	Bottom	−1379.94	−9.48	−0.71	−22.599	−34.102	559.669
1	W1	COMB1	Top	−921.92	−9.31	0.84	23.618	−80.17	1378.857
1	W1	COMB1	Bottom	−946.7	−9.31	0.84	23.618	40.296	38.134
1	W14	COMB1	Top	−517.89	−6.47	−0.35	−4.17	33.657	586.284
1	W14	COMB1	Bottom	−530.42	−6.47	−0.35	−4.17	−16.751	−345.309
1	W3	COMB1	Top	−517.93	−6.34	0.35	4.162	−33.71	586.845
1	W3	COMB1	Bottom	−530.46	−6.34	0.35	4.162	16.932	−326.752
1	W6	COMB4	Top	−2544.55	−6.01	−2.2	6.218	120.844	1574.962
1	W6	COMB4	Bottom	−2570.48	−6.01	−2.2	6.218	−196.134	709.355
1	W15	COMB1	Top	−803.65	6.99	−0.52	11.497	49.816	−915.987
1	W15	COMB1	Bottom	−825.49	6.99	−0.52	11.497	−24.824	89.897
1	W2	COMB1	Top	−804.37	7.09	0.52	−11.524	−49.754	−865.801
1	W2	COMB1	Bottom	−826.21	7.09	0.52	−11.524	24.961	155.113
1	W7	COMB4	Top	−2062.85	7.33	−1.28	5.17	47.608	−1020.46
1	W14	COMB4	Top	−1215.91	7.5	−1.24	−7.07	76.186	−1047.96
1	W14	COMB4	Bottom	−1228.44	7.5	−1.24	−7.07	−102.247	32.396
1	W16	COMB1	Top	−920.25	9.26	0.83	−23.6	−80.161	−1425.3
1	W16	COMB1	Bottom	−945.02	9.26	0.83	−23.6	40.021	−91.397
1	W13	COMB1	Top	−1356.26	10.06	−0.69	21.387	65.873	−2241.06
1	W13	COMB1	Bottom	−1388.6	10.06	−0.69	21.387	−33.01	−792.836
1	W5	COMB1	Top	−1974.81	10.31	2.08	−93.665	−199.59	−3545.01
1	W5	COMB1	Bottom	−2019.33	10.31	2.08	−93.665	100.021	−2060.55
1	W8	COMB1	Top	−1788.4	10.52	−1.22	55.081	117.646	−3879.65
1	W8	COMB1	Bottom	−1832.92	10.52	−1.22	55.081	−58.597	−2364.53
1	W14	COMB3	Top	−950.28	12.4	−1.04	−4.784	56.946	−1459.39
1	W14	COMB3	Bottom	−962.81	12.4	−1.04	−4.784	−92.709	326.473
1	W15	COMB2	Top	−1235.46	13.59	−0.92	20.715	88.854	−1961.59
1	W15	COMB2	Bottom	−1257.29	13.59	−0.92	20.715	−44.251	−3.98
1	W2	COMB2	Top	−1237.12	13.78	0.93	−20.769	−88.722	−1859.21
1	W2	COMB2	Bottom	−1258.96	13.78	0.93	−20.769	44.537	124.989
1	W16	COMB2	Top	−1401.72	15.02	1.47	−41.142	−141.879	−2319.51
1	W16	COMB2	Bottom	−1426.5	15.02	1.47	−41.142	70.03	−156.856
1	W13	COMB2	Top	−2061.8	17.3	−1.15	35.449	109.155	−3886.07

(continued)

Table 6.1 (continued)

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
1	W13	COMB2	Bottom	−2094.14	17.3	−1.15	35.449	−55.755	−1395.28
1	W5	COMB2	Top	−2969.61	17.7	3.59	−161.99	−345.444	−6146.3
1	W5	COMB2	Bottom	−3014.12	17.7	3.59	−161.99	171.749	−3597.73
1	W8	COMB2	Top	−2763.26	19.09	−2.14	95.75	204.479	−7095.4
1	W8	COMB2	Bottom	−2807.78	19.09	−2.14	95.75	−103.213	−4346.41
1	W7	COMB3	Top	−1579.98	20.11	−1.25	3.126	44.478	−2570.08
1	W7	COMB3	Bottom	−1602.06	20.11	−1.25	3.126	−135.392	325.439
1	W10	COMB1	Top	−1179.49	21.26	0.19	2.405	−18.116	−3327.67
1	W10	COMB1	Bottom	−1205.42	21.26	0.19	2.405	9.16	−266.701
1	W6	COMB1	Top	−1177.88	21.35	−0.19	−2.452	18.106	−3322.57
1	W6	COMB1	Bottom	−1203.82	21.35	−0.19	−2.452	−8.951	−248.284
1	W1	COMB6	Top	−2098.3	24.84	3.83	62.604	−250.452	−4028.01
1	W1	COMB6	Bottom	−2123.08	24.84	3.83	62.604	301.169	−451.759
1	W1	COMB5	Top	−1615.07	30.65	3.19	45.039	−188.722	−4871.17
1	W1	COMB5	Bottom	−1639.85	30.65	3.19	45.039	270.888	−458.117
1	W10	COMB2	Top	−1784.87	35.27	0.52	−2.004	−49.977	−5777.64
1	W10	COMB2	Bottom	−1810.8	35.27	0.52	−2.004	25.202	−698.988
1	W6	COMB2	Top	−1781.04	35.43	−0.52	1.912	49.975	−5764.44
1	W6	COMB2	Bottom	−1806.97	35.43	−0.52	1.912	−24.774	−662.31
1	W9	COMB6	Top	−4390.95	41.36	8.13	232.345	−570.429	−15588.7
1	W9	COMB6	Bottom	−4435.47	41.36	8.13	232.345	600.541	−9632.44
1	W13	COMB5	Top	−790.1	41.64	1.52	14.47	7.335	−9145.25
1	W13	COMB5	Bottom	−822.44	41.64	1.52	14.47	226.25	−3149.72
1	W2	COMB3	Top	−244.35	42.66	−0.51	−15.809	−27.352	−4196.72
1	W13	COMB6	Top	−1495.64	48.88	1.06	28.532	50.618	−10790.3
1	W13	COMB6	Bottom	−1527.98	48.88	1.06	28.532	203.504	−3752.16
1	W9	COMB5	Top	−3398.39	49.01	6.62	164.086	−424.566	−17930.3
1	W9	COMB5	Bottom	−3442.91	49.01	6.62	164.086	528.987	−10872.6
1	W2	COMB4	Top	−677.1	49.35	−0.1	−25.054	−66.321	−5190.13
1	W2	COMB4	Bottom	−698.94	49.35	−0.1	−25.054	−81.212	1916.663
1	W8	COMB5	Top	−474.62	62.07	2.7	9.288	−47.852	−22269
1	W8	COMB5	Bottom	−519.14	62.07	2.7	9.288	340.873	−13330.3
1	W10	COMB3	Top	−414.98	62.56	−1.49	6.714	52.748	−10679.7
1	W10	COMB3	Bottom	−440.91	62.56	−1.49	6.714	−162.197	−1670.98
1	W8	COMB6	Top	−1449.49	70.64	1.79	49.957	38.98	−25484.8
1	W8	COMB6	Bottom	−1494	70.64	1.79	49.957	296.257	−15312.2
1	W10	COMB4	Top	−1020.36	76.57	−1.16	2.304	20.887	−13129.6
1	W10	COMB4	Bottom	−1046.29	76.57	−1.16	2.304	−146.155	−2103.27
1	W14	COMB6	Top	−841.16	109.76	−0.47	−4.705	45.561	−3026.72
1	W14	COMB6	Bottom	−853.69	109.76	−0.47	−4.705	−22.596	12778.08
1	W3	COMB6	Top	−841.65	110.04	0.48	4.687	−45.634	−3024.87
1	W3	COMB6	Bottom	−854.18	110.04	0.48	4.687	23.002	12820.79
1	W14	COMB5	Top	−575.53	114.65	−0.27	−2.42	26.321	−3438.15

(continued)

Table 6.1 (continued)

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
1	W14	COMB5	Bottom	−588.06	114.65	−0.27	−2.42	−13.057	13072.15
1	W3	COMB5	Top	−575.78	114.83	0.28	2.41	−26.384	−3438.27
1	W3	COMB5	Bottom	−588.31	114.83	0.28	2.41	13.295	13097.48
1	W1	COMB4	Top	−1535.56	208.39	1.24	27.359	−115.228	9340.101
1	W1	COMB4	Bottom	−1560.34	208.39	1.24	27.359	63.69	39348.49
1	W1	COMB3	Top	−1052.33	214.2	0.6	9.794	−53.498	8496.933
1	W1	COMB3	Bottom	−1077.11	214.2	0.6	9.794	33.41	39342.13
1	W16	COMB3	Top	−790.57	232.78	1.07	−37.38	−106.75	5692.875
1	W16	COMB3	Bottom	−815.35	232.78	1.07	−37.38	46.769	39212.57
1	W16	COMB4	Top	−1272.04	238.53	1.7	−54.921	−168.468	4798.671
1	W16	COMB4	Bottom	−1296.82	238.53	1.7	−54.921	76.778	39147.12
1	W15	COMB5	Top	−615.84	248.72	−0.53	16.593	51.012	4645.239
1	W15	COMB5	Bottom	−637.68	248.72	−0.53	16.593	−25.395	40460.22
1	W2	COMB5	Top	−616.66	248.86	0.53	−16.631	−50.932	4711.487
1	W2	COMB5	Bottom	−638.5	248.86	0.53	−16.631	25.578	40547.48
1	W11	COMB6	Top	−1421.79	252.34	−0.05	−10.259	5.322	4768.005
1	W11	COMB6	Bottom	−1443.87	252.34	−0.05	−10.259	−2.335	41105.58
1	W7	COMB6	Top	−1414.03	252.41	0.04	9.833	−3.717	4787.862
1	W7	COMB6	Bottom	−1436.1	252.41	0.04	9.833	2.185	41134.89
1	W15	COMB6	Top	−1047.65	255.32	−0.94	25.812	90.051	3599.633
1	W15	COMB6	Bottom	−1069.49	255.32	−0.94	25.812	−44.822	40366.34
1	W2	COMB6	Top	−1049.41	255.55	0.94	−25.876	−89.901	3718.082
1	W2	COMB6	Bottom	−1071.25	255.55	0.94	−25.876	45.154	40517.35
1	W7	COMB5	Top	−931.16	265.18	0.07	7.79	−6.847	3238.241
1	W11	COMB5	Top	−933.77	265.24	−0.07	−7.798	6.876	3213.916
1	W11	COMB5	Bottom	−955.85	265.24	−0.07	−7.798	−3.256	41408.2
1	W4	COMB4	Top	−2067.98	289.15	−1.34	−32.932	133.352	28190.45
1	W4	COMB4	Bottom	−2100.32	289.15	−1.34	−32.932	−58.997	69828.74
1	W4	COMB3	Top	−1370.69	295.92	−0.85	−17.854	87.642	26855.96
1	W4	COMB3	Bottom	−1403.03	295.92	−0.85	−17.854	−35.375	69469.02
1	W13	COMB3	Top	−1370.87	318.27	−0.47	23.217	39.984	22171.98
1	W13	COMB3	Bottom	−1403.21	318.27	−0.47	23.217	−28.367	68003.28
1	W13	COMB4	Top	−2076.41	325.51	−0.93	37.279	83.266	20526.98
1	W13	COMB4	Bottom	−2108.75	325.51	−0.93	37.279	−51.112	67400.84
1	W10	COMB5	Top	−1138.15	374.66	0.14	−0.257	−13.098	3598.528
1	W10	COMB5	Bottom	−1164.09	374.66	0.14	−0.257	6.684	57549.06
1	W6	COMB5	Top	−1136.07	374.79	−0.14	0.195	13.081	3603.439
1	W6	COMB5	Bottom	−1162.01	374.79	−0.14	0.195	−6.405	57573.44
1	W10	COMB6	Top	−1743.53	388.67	0.47	−4.667	−44.959	1148.561
1	W10	COMB6	Bottom	−1769.46	388.67	0.47	−4.667	22.726	57116.78
1	W6	COMB6	Top	−1739.23	388.87	−0.47	4.56	44.95	1161.575
1	W6	COMB6	Bottom	−1765.16	388.87	−0.47	4.56	−22.228	57159.42
1	W12	COMB4	Top	−3154.8	469.23	−1.77	−58.189	165.673	74367.51

(continued)

Table 6.1 (continued)

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
1	W12	COMB4	Bottom	−3199.31	469.23	−1.77	−58.189	−88.608	141936.2
1	W9	COMB4	Top	−3453.41	476.19	2.77	100.125	−269.951	78491.33
1	W9	COMB4	Bottom	−3497.93	476.19	2.77	100.125	129.204	147063
1	W12	COMB3	Top	−2180.16	478.06	−0.85	−17.307	78.541	71704.44
1	W12	COMB3	Bottom	−2224.67	478.06	−0.85	−17.307	−43.665	140544.9
1	W9	COMB3	Top	−2460.85	483.84	1.26	31.866	−124.088	76149.69
1	W9	COMB3	Bottom	−2505.37	483.84	1.26	31.866	57.65	145822.8
1	W8	COMB3	Top	−1371.04	498.52	−1.61	93.67	158.008	64737.07
1	W8	COMB3	Bottom	−1415.56	498.52	−1.61	93.67	−74.511	136524.6
1	W5	COMB3	Top	−1483.58	504.78	2.9	−155.355	−275.062	69261.93
1	W5	COMB3	Bottom	−1528.09	504.78	2.9	−155.355	142.023	141950.5
1	W8	COMB4	Top	−2345.9	507.09	−2.53	134.339	244.84	61521.33
1	W8	COMB4	Bottom	−2390.42	507.09	−2.53	134.339	−119.127	134542.7
1	W5	COMB4	Top	−2478.37	512.17	4.41	−223.68	−420.915	66660.65
1	W5	COMB4	Bottom	−2522.89	512.17	4.41	−223.68	213.751	140413.3

6.3.4 Stress Transfer Mechanism in Existing Building with Proposed Shear Wall

Earthquake story forces at each story levels, which is further distributed on all shear walls will act as the interface shear between beams and shear wall and it is required to be transmitted through vertical dowels and the same will be required to transmit between columns and shear walls using horizontal dowels (Fig. 6.10).

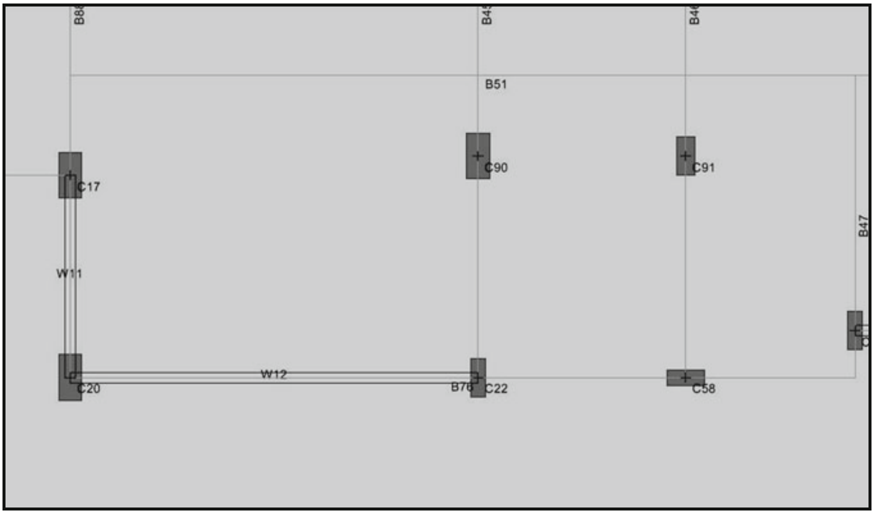


Fig. 6.9 Location of shear wall [W-12]

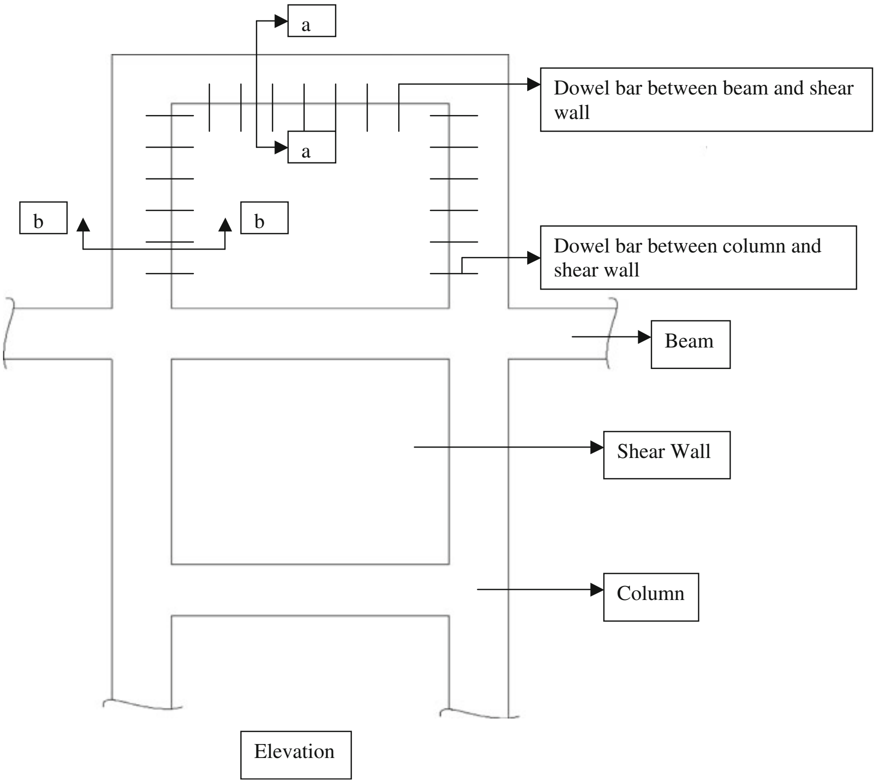


Fig. 6.10 Elevation

6.3.5 Beams and Shear Wall Connection

Shear stress between beam and shear wall is resisted by the dowel bars provided between them. A through hole is drilled into the beam across its width above the existing flexural reinforcement and a straight dowel bar is passed through it. After passing the bar above the flexural r/f of the beam it was bent down and tie with the r/f of the shear wall. Adequate length is provided for the proper bond between wall and beam to transfer the stresses. This is shown in the Fig. 6.11.

6.3.6 Column and Shear Wall Connection

A dowel bar between column and shear wall transfers the shear stress. A hole of sufficient length is drilled in the column to insert and fix from one end of the bar. Epoxy is then injected in the hole which after hardening and gaining strength holds

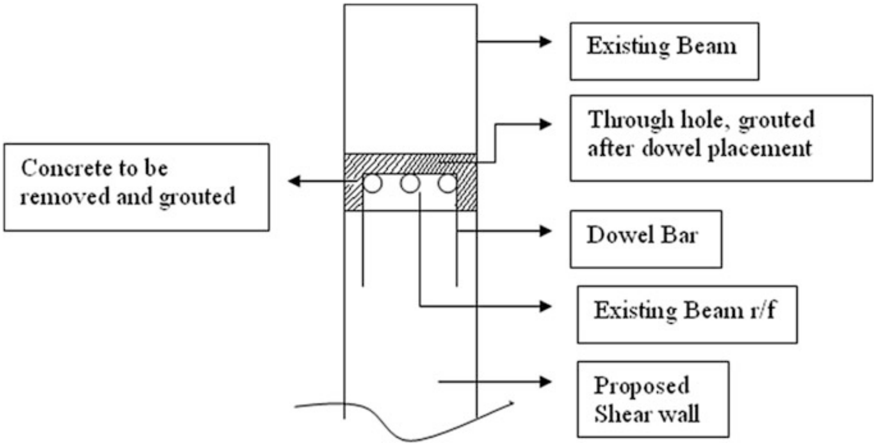


Fig. 6.11 Section a-a

the bar firmly in the column. Epoxy injection requires a high degree of skill for satisfactory execution, including a very keen inspection of the holes to be filled by epoxy and the proper use and application of the related guidelines and specifications.

6.3.7 Detail Procedure for Column and Shear Wall Connection

The steps are as follows:

(a) Selection of proper epoxy
(b) Drilling the whole in columns of required length
(c) Installing entry ports (packers) for epoxy injection
(d) Placing the dowel bars
(e) Mixing the epoxy
(f) Injecting the epoxy
(g) Removing the packers

6.3.7.1 Selection of Epoxy

In selecting an epoxy consideration must be given to environmental conditions. There are two important factors affecting the selection of epoxy: (a) in general epoxy is thermally incompatible with concrete and (b) strength of epoxy decreases sharply with increase in ambient temperature.

6.3.7.2 Drilling the Whole

A whole will be drilled at sufficient locations. The number of holes can be determined from the analysis of the dowel required by dividing the length by the spacing. The spacing must be sufficient in order to prevent the spilling of old concrete (Fig. 6.12).

6.3.7.3 Installing Entry Ports

The epoxy needed to be injected under pressure through the ports, which are also called as packers. There are two types of packers—(a) flat packers and (b) long packers. The flat packers are glued directly. High pressure will not be required, as the drilled length is not more. Therefore in this case we will use flat packers because fitting is suitable when the injection pressure is not too high.

6.3.7.4 Position the Dowel Bars

The dowel bars are inserted in the drilled holes. By dividing the shear force by the number of dowels, the shear force on each dowel is calculated. This shear force is used for the designing of the dowel bars (Fig. 6.13).

6.3.7.5 Mixing the Epoxy

The mixing should be done in strict compliance with the manufacturer's recommendations. Mixing is done either by batch mix or continuous mix. In batch

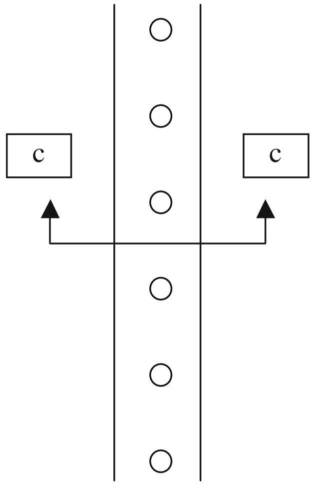


Fig. 6.12 Section b-b

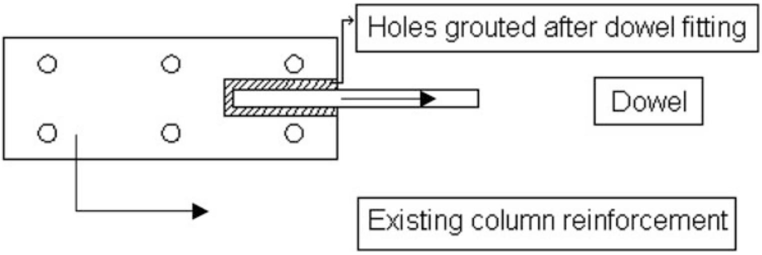


Fig. 6.13 Section c-c

mixing, the pre-weighed epoxy components are mixed together according to manufacturer's instructions usually with the help of a mechanical stirrer. In a continuous mixing system, the two liquid components pass through metering and driving pump prior to passing through an automatic mixing system.

6.3.7.6 Injecting the Epoxy

Normally pressure pumps are used. The mixed resin is poured into the pump and the injection gun is triggered to effect the injection.

For vertical or inclined holes, the injection begins at the lowest elevation and is continued until the epoxy level reaches the entry ports above, which is confirmed when epoxy flows out. The lower injection port is then capped or sealed and the process is repeated at successively higher ports.

6.3.7.7 Remove the Packers

Once the epoxy has cured, the packers are removed. This is a time consuming operation. If the packers are safely removed, they are reused.

6.3.8 Design for Dowel Bars

The expected shear force between existing beams and the new shear wall; can be obtain from above software results. Where as the expected shear force between existing columns and new shear wall will be the axial force in the columns of the structure without shear wall

Area of Dowel bars between beams and columns = design shear force on the wall/∅ fy
$478.058/0.7 * 60$
$11.4\,\text{in}^2$
Can be taken as $12\,\text{in}^2$

This area of steel is calculated without considering the friction between old and new concrete and by considering it we can save the quantity of steel required (Fig. 6.14).

6.4 Strengthening of Beams

After the application of any of above proposal, it is required to strengthen the beams that are found deficient in shear. There are different beams of different stories require shear strengthening. It will be considered the weakest beam in the sense that is required maximum level of strengthening and obtain a solution. The solution will be valid for all other beams.

In general, the shear capacity of an ordinary reinforced concrete beam is the sum of strength of concrete section and steel (shear reinforcement).

$$V = Vc + Vs \tag{6.8}$$

According to ACI (2004) code, a section that has a maximum shear reinforcement, should not have steel shear capacity greater than,

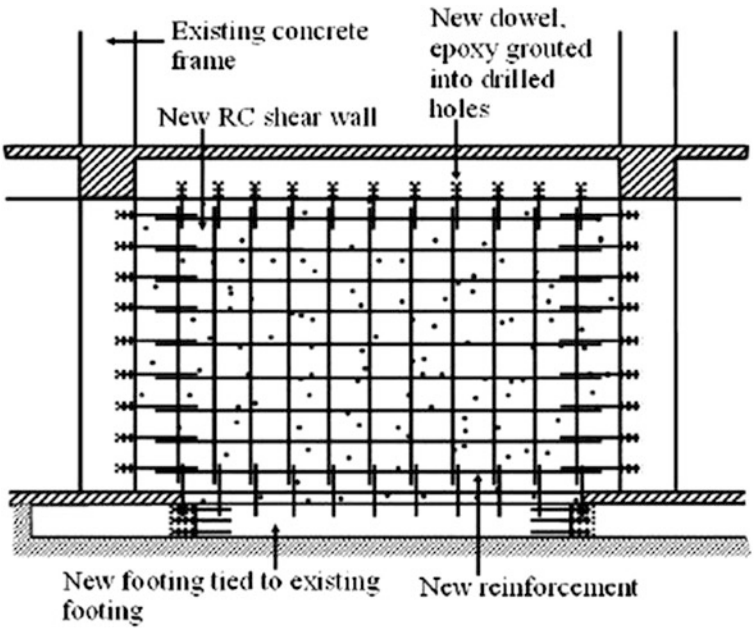


Fig. 6.14 Connection of shear wall with existing beams columns

$$\text{Vs} \leq [8\sqrt{\text{fc}'}\,\text{bw d}] \tag{6.9}$$

And the concrete shear capacity

$$\text{Vc} = 1.9\sqrt{\text{fc}'}\,\text{bw d} \tag{6.10}$$

The above Eq. (6.9) is valid for the beam subjected to normal shear force, having no torsion. But the beam which is under our consideration has both shear and torsion. As shown in the end of previous chapter.

For this case, code has imposed another requirement with considering of both shear and torsion effects

$$\sqrt{}[(\text{Vu}/\text{bbdd})^2 + (\text{Tu Ph}/1.7\text{Aoh}^2)^2] \leq \emptyset(\text{Vc}/\text{bw d}) + 8\sqrt{\text{fc}'} \tag{6.11}$$

6.4.1 *Calculation of the Additional Shear Element Required*

Using the software design outputs, as shown above for the beam under consideration, we get:

$$\begin{aligned} \text{Vu} &= 47.801\,\text{kips} \\ \text{Tu} &= 799.331\,\text{kip - in} \\ \text{Aoh} &= 250.25\,\text{in}^2 \\ \text{Ph} &= 90\,\text{in} \\ \text{Vc} &= 0.101\,\text{ksi} \\ \text{fc}' &= 3.5\,\text{ksi} \end{aligned}$$

For a beam having maximum shear reinforcement should satisfy the equation below

$$\sqrt{}[(\text{Vu/bbdd})^2 + (\text{TuPh/1.7Aoh}^2)^2] = \sqrt{}[(\text{Vc/bwd}) + 8\sqrt{\text{fc}'}]$$

Using the above values we get:

$$\begin{aligned} &\sqrt{}[(47801/10 * 42)^2 + (799.331 * 1000 * 90/1.7 * 250.25^2)^2] \\ &\quad = 0.85[(0.101 * 1000) + 8\sqrt{3500}] \end{aligned}$$

So, the beam has to be strengthened in shear to give an additional shear strength element. Let x would be the additional element to meet our requirement as follows:

$685.24 \leq 488.23 + x$
$197.096 \leq x$

For safe side let us consider x = 0.2 kips which can be achieve by any suitable retrofitting technique. Following are the few of the suitable techniques for shear strengthening

- External FRP Reinforcement
- External shear reinforcement (in the form of plate or bars)
- Concrete Overlays

The use FRP Reinforcement for shear strengthening is discussed here.

6.4.2 External FRP Reinforcement

6.4.2.1 Brief Introduction

The use of externally bonded fiber reinforced polymer (FRP) reinforcement to strengthen RC structures is becoming an increasingly popular retrofitting technique. The light weight and formability of FRP reinforcement make these systems easy to install. Since the materials used in these systems are non-corrosive, non-magnetic, and generally resistant to chemicals, they are an excellent option for external reinforcement. The detailed retrofitting with FRP will be discussed in Chap. 7 of this book. In this section the building under consideration for retrofitting will be discussed relating to the Shear strengthening by FRP.

6.4.2.2 Shear Strength of RC Beams Strengthened with FRP Reinforcement

The nominal shear strength of a RC beam may be computed by the basic design equation presented in ACI 318–95 and given below as Eq. (6.12).

$$Vn = Vc + Vs \tag{6.12}$$

In this equation the nominal shear strength is the sum of the shear strength of the concrete (which for a cracked section is attributable to aggregate interlock, dowel action of the longitudinal reinforcement, and the diagonal tensile strength of the uncracked portion of concrete) and the strength of the steel shear reinforcement. In the case of beams strengthened with externally bonded FRP sheets, the nominal shear strength may be computed by the addition of a third term to account for the contribution of the FRP sheet to the shear strength. This is expressed in Eq. (6.13).

$$Vn = Vc + Vs + Vf \tag{6.13}$$

The design shear strength is obtained by multiplying the nominal shear strength by a strength reduction factor. It is suggested that the reduction factor of = 0.85 given in ACI 318–95 be maintained for the concrete and steel terms. However, the reduction factor for CFRP reinforcement will require an adjustment as discussed later.

6.4.2.3 Design Approach Based on the Effective FRP Stress

The design approach based on fracture of the CFRP sheet is quite similar to the approach used to compute the contribution of steel shear reinforcement. The stress in the sheet at ultimate must be calculated in the vertical direction and multiplied by the area of sheet that crosses a potential shear crack. However, instead of the ultimate condition being governed by a yield point, as with steel, the rupture point of the CFRP sheet must be considered. Triantafillou (1997) noted that CFRP sheets used for shear strengthening rupture at stress levels below their ultimate strength due to stress concentrations in the sheet. If the level of strain at rupture is considered as the effective strain, fe, the contribution of externally bonded FRP sheets to the shear capacity of an RC beam may be computed from Eq. (6.14).

$$\mathrm{vf} = \rho\mathrm{f}\ \mathrm{Ef}\ \varepsilon fe\ \mathrm{bw}\ 0.9\mathrm{d}(1 + \cot\beta)\ \sin\beta \tag{6.14}$$

This equation, as presented by Triantafillou, is in the Euro code format. The shear reinforcement ratio, ρf, is the FRP shear reinforcement and the angle β is the angle between the orientation of the principal fibers in the sheet and the longitudinal axis of the beam. This equation may be rewritten in ACI code format as Eq. (6.15).

$$\mathrm{vf} = \frac{\mathrm{Afffe}\ (\sin\beta + \cos\beta)\mathrm{df}}{\mathrm{Sf}} \tag{6.15}$$

Here the effective strain times the modulus of elasticity is replaced with the effective stress. The area of CFRP shear reinforcement is the total thickness of the sheet (usually 2tf for sheets on both sides of the beam) times the width of the CFRP strip. Note if continuous sheets are used the width of the strip, wf, and the spacing of the strips, sf, should be equal.

ffe = R ffu

6.4.2.4 Design

So, for the above beam V f required = 0.2 kips i.e. equal to 200 lb. (200 *4.4 = 880 N)
A f = 2* width * thickness of CFRP
Thickness of CFRP = 4.5 mm = (4.5/25 in.)
Width = 12″
d = 42″

ffu = 492 psi
R = 0.3
Let the spacing Sf = 12″
β = 90°

So,

$$\text{vf} = \frac{\text{Afffe}\,(\sin\beta + \cos\beta)\text{df}}{\text{Sf}}$$

V f = 2*4.5/25*12*480*0.3*42/12
V f = 2177.28 lbs

So the provision of 4.5 mm thick C.F.R.P strip can make the section adequate enough to resists the above predicted shear. The method can be applied for all beams section, that are inadequate in shear strength.

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Chapter 7
Retrofitting by Use of FRP

Abstract The composite materials made of fibers in a polymeric resin, also known as fiber-reinforced polymers (FRPs), have emerged as an alternative to traditional materials for repair and rehabilitation. For the purposes of this document, an FRP system is defined as the fibers and resins used to create the composite laminate, all applicable resins used to bond it to the concrete substrate, and all applied coatings used to protect the constituent materials. The fibers and resins used in FRP systems are relatively expensive compared to traditional strengthening materials such as concrete and steel, however, it is cost efficient in terms of labor and equipment costs. One great advantage of FRP systems is that it can be used in areas with limited access where traditional techniques would be difficult to implement. FRP sheets can be wrapped around concrete columns to increase the strength. The procedure for applying FRP sheet is described in this chapter. This FRP increases the load capacity of the column and increases the deformation capability.

7.1 Introduction

FRP is a composite material generally consisting of carbon, aramid, or glass fibers in a polymeric matrix (e.g. epoxy resin). Among many options, this reinforcement may be in the form of preformed laminates or flexible sheets. The laminates are stiff plates or shells that come pre-cured and are installed by bonding the plate to the concrete surface with epoxy. The sheets are either dry or pre-impregnated with resin (pre-preg) and cure after installation onto the concrete surface. This installation technique is known as wet lay-up.

The strengthening or retrofitting of existing concrete structures to resist higher design loads, correct strength loss due to deterioration, correct design or construction deficiencies, or increase ductility has traditionally been accomplished using conventional materials and construction techniques. Externally bonded steel plates, steel or concrete jackets, and external post-tensioning are just some of the many traditional techniques available.

R.R. Hussain et al., *Computer Aided Seismic and Fire Retrofitting Analysis of Existing High Rise Reinforced Concrete Buildings*,
Solid Mechanics and Its Applications 222, DOI 10.1007/978-94-017-7297-6_7

Composite materials made of fibers in a polymeric resin, also known as fiber-reinforced polymers (FRPs), have emerged as an alternative to traditional materials for repair and rehabilitation. For the purposes of this document, an FRP system is defined as the fibers and resins used to create the composite laminate, all applicable resins used to bond it to the concrete substrate, and all applied coatings used to protect the constituent materials. Coatings used exclusively for aesthetic reasons are not considered part of an FRP system.

FRP materials are lightweight, noncorrosive, and exhibit high tensile strength. These materials are readily available in several forms, ranging from factory-made laminates to dry fiber sheets that can be wrapped to conform to the geometry of a structure before adding the polymer resin. The relatively thin profiles of cured FRP systems are often desirable in applications where aesthetics or access is a concern.

The growing interest in FRP systems for strengthening and retrofitting can be attributed to many factors. Although the fibers and resins used in FRP systems are relatively expensive compared with traditional strengthening materials such as concrete and steel, labor and equipment costs to install FRP systems are often lower (Nanni and Dolan 1999). FRP systems can also be used in areas with limited access where traditional techniques would be difficult to implement.

7.2 Applications and Use

FRP systems can be used to rehabilitate or restore the strength of a deteriorated structural member, retrofit or strengthen a sound structural member to resist increased loads due to changes in use of the structure, or address design or construction errors. The licensed design professional should determine if an FRP system is a suitable strength-ening technique before selecting the type of FRP system.

To assess the suitability of an FRP system for a particular application, the licensed design professional should perform a condition assessment of the existing structure that includes establishing its existing load-carrying capacity, identifying deficiencies and their causes, and determining the condition of the concrete substrate. The overall evaluation should include a thorough field inspection, a review of existing design or as-built documents, and a structural analysis in accordance accordance with ACI 364. I R (2007). Existing construction documents for the structure should be reviewed, including the design drawings, project specifications, as-built information, field test reports, past repair documentation, and maintenance history documentation. The licensed design professional should conduct a thorough field investigation of the existing structure in accordance with ACI 437R (2003) and applicable ACI documents. As a minimum, the field investigation should determine the following:

- Existing dimensions of the structural members
- Location, size, and cause of cracks and spalls; Location and extent of corrosion of reinforcing steel
- Presence of active corrosion
- Quantity and location of existing reinforcing steel
- In-place compressive strength of concrete
- Soundness of the concrete, especially the concrete cover, in all areas where the FRP system is to be bonded to the concrete.

The tensile strength of the concrete on surfaces where the FRP system may be installed should be determined by conducting a pull-off adhesion test in accordance with ACI 503R (1993). The in-place compressive strength of concrete should be determined using cores in accordance with ACI 318-05 requirements. The load-carrying capacity of the existing structure should be based on the information gathered in the field investigation, the review of design calculations and drawings, and as determined by analytical methods. Load tests or other methods can be incorporated into the overall evaluation process if deemed appropriate.

FRP systems used to increase the strength of an existing member should be designed in accordance with Part 4, which includes a comprehensive discussion of load limitations, rational load paths, effects of temperature and environment on FRP systems, loading considerations, and effects of reinforcing steel corrosion on FRP system integrity.

Fire and life safety-FRP-strengthened structures should comply with all applicable building and fire codes. Smoke generation and flame spread ratings should be satisfied for the assembly according to applicable building codes depending on the classification of the building. Smoke and flame spread ratings should be determined in accordance with ASTM E84. Coatings (Apicella and Imbrogno 1999) and insulation systems (Bisby et al. 2005a; Williams et al. 2006) can be used to limit smoke and flame spread.

Because of the degradation of most FRP materials at high temperature, the strength of externally bonded FRP systems is assumed to be lost completely in a fire, unless it can be demonstrated that the FRP temperature remains below its critical temperature (for example, FRP with a fire-protection system). The physical and mechanical properties of the resin components of FRP systems are influenced by temperature and degrade at temperatures close to and above their glass-transition temperature T_g (Bisby et al. 2005b). The T_g for FRP systems typically ranges from 140 to 180 OF (60–82 °C) for existing, commercially available FRP systems. The T_g for a particular FRP system can be obtained from the system manufacturer (Fig. 7.1).

The following chart illustrates that why FRP materials are the best for repair and retrofitting.

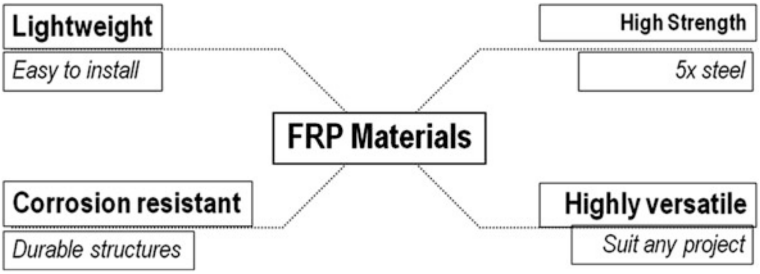


Fig. 7.1 The pictorial representation of the application of the FRP material in repairs and strengthening

7.3 Skematic Representation of the Analysis and Design Using FRP

The following figure illustrates the application of FRP for flexure and shear of the beam and for confinement in columns (Fig. 7.2).

As discussed in the earlier chapter that FRP is the composite of Fibres which provide strength and stiffness like carbon, glass, aramid etc. and matrix which protects and transfers load between fibres like epoxy, polyester, vinyl ester. These two components (fibre and matrix) create a material with attributes superior to either component alone. The typical stress strain graph is soon in Fig. 7.3

If we compare FRP properties with the steel we can find that FRP have linear elastic behavior. It has no yielding. It gives lower strain at failure and it gives higher

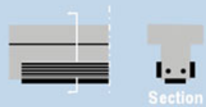
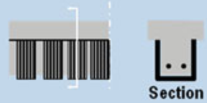
TYPE	APPLICATON	FIBRE DIR	SCHEMATIC
Flexural	Tension and/or side face of beam	Along long. axis of beam	
Shear	Side face of beam (u-wrap)	Perpendicular to long. axis of beam	
Confinement	Around column	Circumferential	

Fig. 7.2 The application of the FRP for various structural elements

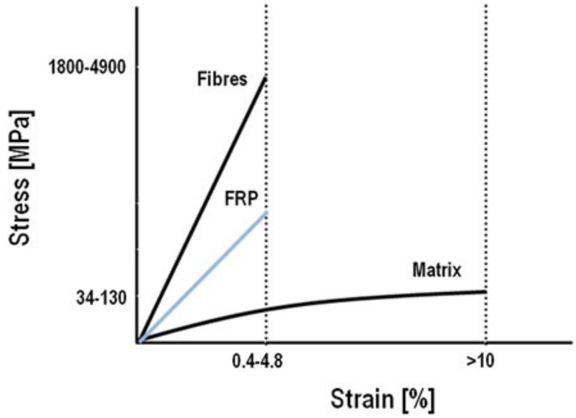


Fig. 7.3 The typical stress-strain behavior of FRP, fibres and matrix

ultimate strength as compared to steel. The comparison of FRP with GFRP and steel is shown below in the Fig. 7.4.

7.3.1 *Flexural Strengthening by FRP Using ISIS Modules*

7.3.1.1 Assumptions

(1) Failure caused by FRP rupture and Crushing of the concrete.
(2) Plane sections remain plane.
(3) Perfect bond between steel/concrete, FRP/concrete.
(4) Adequate anchorage and development length provided for FRPs.
(5) FRPs are linear elastic to failure.
(6) Concrete compressive stress-strain curve is parabolic, no strength in tension.
(7) Initial strains in FRPs can be ignored.

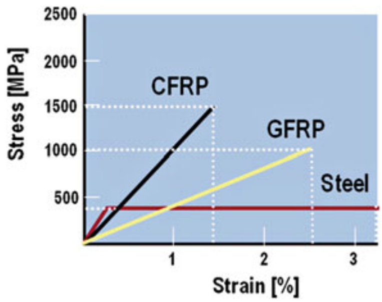


Fig. 7.4 Stress-strain comparison among CFRP, GFRP and steel

7.3.1.2 Resistance Factors

For Steel in buildings the resistant factor is taken as $\varnothing_S = 0.85$
For Concrete in buildings the resistant factor is taken as $\varnothing_c = 0.6$
For FRP buildings the resistant factor is taken as for carbon $\varnothing_{frp} = 0.6$
For FRP buildings the resistant factor is taken as for Glass $\varnothing_{frp} = 0.5$

7.3.1.3 Failure Modes

Four potential failure modes:

Concrete crushing before steel yields
Steel yielding followed by concrete crushing
Steel yielding followed by FRP rupture
Debonding of FRP reinforcement

$\Downarrow$

Debonding is prevented through special end anchorages

Assume failure mode $\Rightarrow$ Perform analysis $\Rightarrow$ Check failure mode

The initial strain is considered zero as per ISIS Canada.

7.3.2 *General Design*

Apply strain compatibility and use these equations to solve for neutral axis depth, c (Fig. 7.5)

$$M_r = T_s + (d - a/2) + T_{frp} + (h - a/2) \tag{7.1}$$

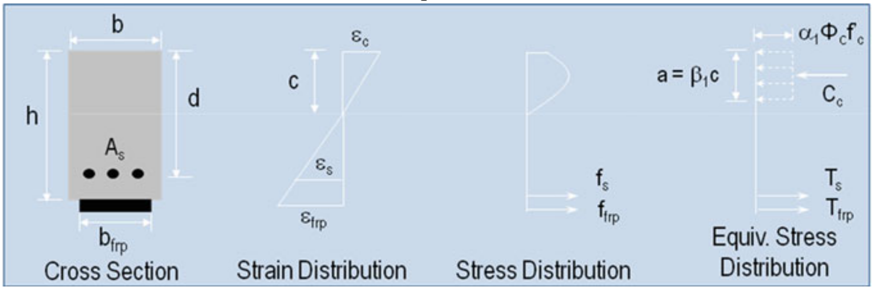
<u>**Step 1: Assume failure mode**</u>
Assume that section fails by concrete crushing after steel yields

$$\varepsilon_c = \varepsilon_{cu} = 0.0035 \tag{7.2}$$

$$\varepsilon_c = \varepsilon_{cu} = (h - c)/c \tag{7.3}$$

$$\varepsilon_c = \varepsilon_{cu} = (d - c)/c \tag{7.4}$$

For at equilibrium



$$\boxed{T_s + T_{frp} = C_c}$$

$$T_s = \phi_s A_s f_s \qquad T_{frp} = \phi_{frp} A_{frp} E_{frp} \varepsilon_{frp} \qquad C_c = \phi_c \alpha_1 f'_c \beta_1 bc$$

Fig. 7.5 General analysis and design procedure for FRP

Step 2: Determine compressive stress block factors

$$a_1 = 0.85 - 0.0015 f'_c > 0.67 \tag{7.5}$$

$$b_1 = 0.97 - 0.0025 f'_c > 0.67 \tag{7.6}$$

Step 3: Determine neutral axis depth, c

$$\emptyset_s A_s f_s + \emptyset_{frp} A_{frp} E_{frp} \varepsilon_{frp} + \emptyset \alpha_c f'_c \beta_1 bc \tag{7.7}$$

Step 4: Check if assumed failure mode is correct

$$\varepsilon_{frp} = \varepsilon_{cu}(h - c)/c > \varepsilon_{frpu} \tag{7.8}$$

If true, go to Step 6 and If false, go to Step 5.

Step 5: Calculate factored moment resistance

$$M_r = \emptyset_s A_s f_s (d - a/2) + \emptyset f_{rp} A_{frp} E_{frp} \varepsilon_{frp} (h - a/2) \tag{7.9}$$

$\varepsilon_s = \varepsilon_{cu}(d - c)/c > \varepsilon_y$ If yes, OK, If no, reduce FRP amount and recalculate.

Step 6: Assume different failure mode

Assume failure occurs by tensile failure of FRP

$$\varepsilon_{frp} = \varepsilon_{frpu} \tag{7.10}$$

$$\text{thus} \quad \varepsilon_c < \varepsilon_{cu}$$

Step 7: Determine depth of neutral axis

$$\emptyset_s A_s f_s + \emptyset_{frp} A_{frp} E_{frp} \varepsilon_{frpu} = \emptyset_c \alpha_1 f' \beta_1 bc \tag{7.11}$$

Step 8: Check if assumed failure mode is correct

$$\varepsilon_c < \varepsilon_{cu} \tag{7.12}$$

$$\varepsilon_{frpu} c/(h - c) < \varepsilon_{cu} \tag{7.13}$$

Step 9: Calculate factored moment resistance

$$M_r = \emptyset_s A_s f_s (d - a/2) + \emptyset_{frp} A_{frp} E_{frp} \varepsilon_{frp} (h - a/2) \tag{7.14}$$

7.3.3 *With Compression Steel*

Similar analysis procedure

Add a compressive stress resultant (Fig. 7.6).

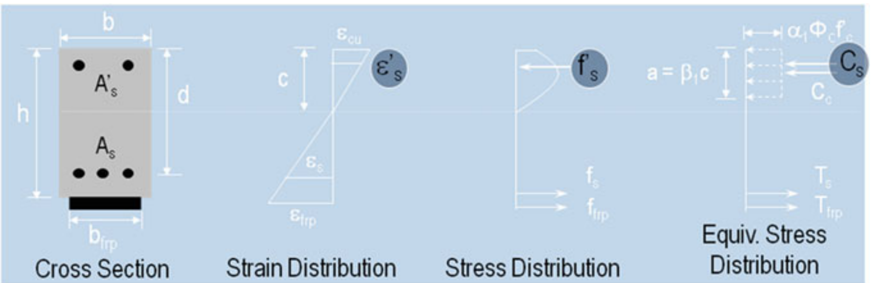


Fig. 7.6 General analysis and design procedure for FRP with compression steel

7.3.4 Tee Beams

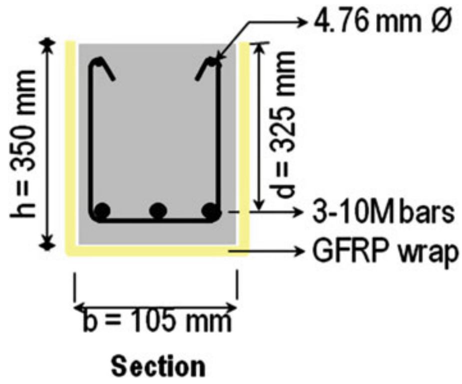
Similar analysis procedure

Neutral axis in flange: treat as rectangular section (Fig. 7.7).
Neutral axis in web: treat as tee section.

Shear Strengthening Examples by FRP

Calculate the shear capacity (V_r) for an FRP-strengthened concrete section

f_y = 400 MPa (stirrup), f_y = 400 MPa (rebar), s_s = 225 mm c/c, E_{frp} = 22.7 GPa,
ε_{frpu} = 2.0 %, s_{frp} = 200 mm, w_{frp} = 100 mm, t_{frp} = 1.3 mm, λ = 1.0, f'_c = 45 MPa



Solution:

Step 1: FRP reinforcement ratio

Concrete

$$V_c = 0.2 f_c \sqrt{f'_c} b_w d$$
$$V_c = 0.2(0.6)\sqrt{45}\,(105)\,(325)$$
$$V_c = 27470 \text{ N} = 27.47 \text{ kN}$$

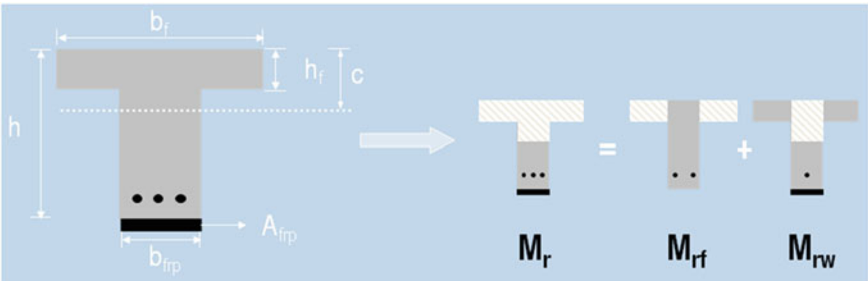


Fig. 7.7 General analysis and design procedure of Tee beams for FRP

Steel

$$
\begin{aligned}
V_s &= \left(f_s f_y A_v d\right)/\ s \\
&= 0.85(400)\,(36)\,(325)/225 \\
V_s &= 17680\ \text{N} = 17.68\ \text{kN} \\
A_{frp} &= 2\ t_{frp} w_{frp} = 2(1.3)\,(100) \\
A_{frp} &= 260\ \text{mm}^2
\end{aligned}
$$

FRP shear reinforcement ratio $\left(\rho_{frp}\right) = \left(2t_{frp}/b_w\right)\left(W_{frp}/S_{frp}\right)$

$$
\begin{aligned}
&= (2(1.3)/105)(100)/200) \\
\rho_{frp} &= 0.0124
\end{aligned}
$$

Step 2: Determine A_{frp}, ρ_{frp}, Le for effective strain calculation

Effective anchorage length $L_e = 25350/\left(t_{frp}E_{frp}\right)^{0.58}$

$$
\begin{aligned}
&= 25350/(1.3\ \text{x}\ 22700)^{0.58} \\
Le &= 64.8\ \text{mm}
\end{aligned}
$$

Step 3: Determine k1, k2 and effective strain, ε_{frpe}

$$
\begin{aligned}
k_1 &= (f'_c/27.65)^{2/3} \\
&= (45/27.65)^{2/3} = 1.38
\end{aligned}
$$

$$
\begin{aligned}
K_2 &= \left(d_{frp} - n_e L_e\right)/d_{frp} \\
&= (325 - 1(64.8))/325 \\
&= 0.80
\end{aligned}
$$

$$
\begin{aligned}
\varepsilon_{frpe} &\le \alpha k_1 k_2 L_e/9525 \\
\varepsilon_{frpe} &= 0.8(1.38)\,(0.80)\,(64.8)/9525 \\
\varepsilon_{frpe} &= 0.0060
\end{aligned}
$$

Step 4: Determine R and effective strain,

$$
\begin{aligned}
R &= \alpha\lambda_1\left\{f'c^{2/3}\Big/\left(\rho_{frp}E_{frp}\right)^{0.58}\right\}^{\lambda 2} \\
R &= 0.8(1.23)\left\{45^{2/3}/(0.0124 \times 22700)\right\}^{0.47} \\
R &= 0.229
\end{aligned}
$$

ε_{frpe} = R $e_{frpu} \leq 0.004 \Longrightarrow$ **[Prevents shear cracks from widening beyond acceptable limits and ensures aggregate interlock.]**

$$\varepsilon_{frpe} = 0.229(0.02) = 0.0046$$

Step 5: Determine governing effective strain

For design purposes, use the smallest limiting value of:

$$\varepsilon_{frpe} = 0.0046$$
$$\varepsilon_{frpe} = 0.0040$$
$$\varepsilon_{frpe} = 0.0060$$

Step 6: Calculate contribution of FRP to shear capacity

$$Vfrp = \emptyset_{frp} A_{frp} E_{frp} \varepsilon_{frpe} d_{frp} (\sin\alpha + \cos\beta)$$
$$= 0.5\,(260)\,(22700)\,(0.004)\,(325)\,(\sin 90 + \cos 90)/200$$
$$V_{frp} = 19200 \text{ N} = 19.2 \text{ kN}$$

Step 7: Compute total shear resistance of beam

$$V_r = V_c + V_s + V_{frp}$$
$$V_r = 27.5 + 17.7 + 19.2$$
$$V_r = 64.4 \text{ kN}$$

Step 8: Check maximum shear strengthening limits

$$V_r \leq V_c + 0.8\lambda\emptyset_c f_c' b_w d$$
$$64400 \leq 27500 + 0.8\,(1)\,(0.6)\,(45)\,(105)\,(325)$$
$$64400 \leq 137400 \quad \text{OK}$$

Step 9: Check maximum band spacing

$$s_{frp} \leq w_{frp} + d/4$$
$$200 \leq 100 + 325/4$$
$$200 \leq 181$$

Not true, therefore use 180 mm spacing

Additional factors to consider which must be considered during design are Deflections, Crack widths, Vibrations, Creep, Fatigue, Ductility.

7.4 Column Strengthening by FRP

FRP sheets can be wrapped around concrete columns to increase strength. The procedure for applying FRP sheet is shown in figure. This FRP increases the load capacity of the column and increase the deformation capability.

Design equations are largely empirical (from tests). For illustration of the column strengthening the ISIS modules which are simple to understand for the new upcoming structural engineers are followed in this book to have better understanding of the column jacketing by FRP (Fig. 7.8).

ISIS equations are applicable for the following cases;

(1) Undamaged concrete column.
(2) Short column subjected to concentric axial load.
(3) Fibres oriented circumferentially.

Problem statement
Determine the FRP wrap details for an RC column as described below

RC column factored axial resistance (pre-strengthening) = 3110 kN
New axial live load requirement P_L = 1550 kN,
New axial dead load requirement P_D = 1200 kN New factored axial load, P_f = 4200 kN
l_u = 3000 mm, D_g = 500 mm, A_g = 196,350 mm^2 A_{st} = 2500 mm^2, f'_c = 30 MPa
f_{frpu} = 1200 MPa, t_{frp} = 0.3 mm, f_{frp} = 0.75, f_y = 400 MPa

Solution:
Step 1: Check if column remains short after strengthening
Only valid for non slender column: $l_u/D_g \le (6.25/(P_f/f'_c\ A_g)^{0.5}$ where
Ag = gross cross-sectional area of column

f′c = concrete strength
Pf = factored axial load (*The axial load capacity is increased by the confining effect of the wrap. Column may become slender. Ensure the column remains short.)*

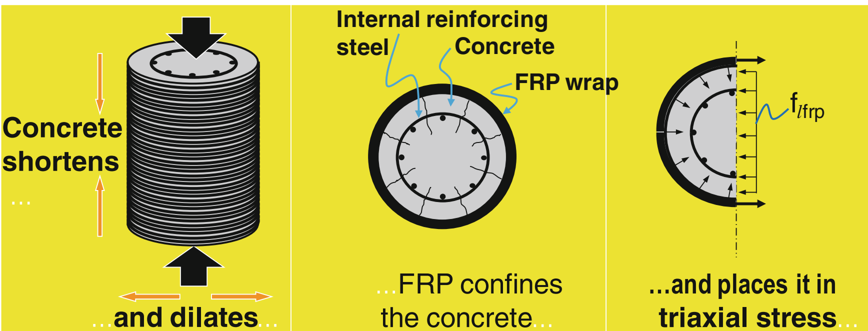


Fig. 7.8 General application of FRP for column jacketing

lu = unsupported length
Dg = column diameter
$300/500 \le 6.25/(4{,}200{,}000/(30 \times 196{,}350)^{0.5})$
$6 \le 7.4$ OK

Step 2: Compute required confined concrete strength, f′cc

$$P_{rmax} = k_e\left[\alpha_1 \emptyset_c f'_{cc}\left(A_g - A_s\right) + \emptyset_s f_y A_s\right]$$
$$f_{cc} = \left\{(P_f/k_e) - f_s f_y A_s\right\}/\{\alpha\emptyset c(Ag\text{- }As)\}$$
$$\alpha_1 = 0.85 - 0.0015 f'_c = 0.85 - 0.0015(30) = 0.81$$
$$f'_{cc} = \{(4200000/0.85) - 0.85(400)(2500)\}/\{0.81(0.6)(196350 - 2500)\}$$
$$f'_{cc} = 43.4 \text{ MPa}$$

Step 3: Compute volumetric strength ratio

$$f'_{cc} = f'_c + k_1 f_{lfrp} = f'_c\left(1 + \alpha_{pc} w_w\right)$$
$$w_w = \left(f'_{cc}/ f'_c - 1\right)/\alpha_{pc}$$
$$= (43.4/30 - 1)/1$$
$$w_w = 0.447$$

Step 4: Compute required confinement pressure, flfrp

$$\rho_{frp} f_{frp} f_{frpu}/\alpha c\ f'_c = 2\ f_{lfrp}/\alpha c\ f'_c$$

$$f_{lfrp} = w_w \emptyset_c f'_c/2$$
$$= 0.447(0.6)(30)/2$$
$$f_{lfrp} = 4.02 \text{ MPa}$$

Check F_{LFRP} Again Confinement Limits:
Minimum, $f_{lfrp} = 4.02 > 4.0$

$$f_{lfrp} = 4.02 < \left\{f'c/2\ \alpha_{pc}\right\}(1/k_e - \phi_c)$$
$$f_{lfrp} = 4.02 < 30/2(1)(1/0.85 - 0.6)$$
$$= 8.56 \text{ OK, limits met}$$

Step 5: Compute required number of FRP layers

$$f_{lfrp} = 2\ Nb \emptyset_{frp} f_{frpu} t_{frp}/Dg$$

$$Nb = f_{frpu} Dg/\left(2\ f_{frp} f_{frpu} t_{frp}\right)$$
$$= 4.02(500)\{2(0.75)(1200)(0.3)\}$$
$$Nb = 3.72\ \textit{Use 4 layers}$$

<u>Step 6: Compute factored axial strength of FRP-wrapped column</u>

$$
\begin{aligned}
f_{lfrp} &= 2\ Nb\emptyset_{frp} f_{frpu} t_{frp/} Dg \\
&= 4.32\ \text{MPa} \\
w_w &= 2\ f_{lfr}/\emptyset c\ f'c \\
f'cc &= f'c\ \left(1+\alpha_{pcw} w\right) = 44.4\ \text{MPa} \\
P_{rmax} &= k_e\left[\alpha_1 \emptyset_c f'_{cc}\left(A_g - A_s\right) + \emptyset_s f_y A_s\right] \\
P_{rmax} &= 4230\ \text{kN}\ > P_f = 4200\ \text{kN}
\end{aligned}
$$

Note: Additional checks should be performed for creep analysis

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Chapter 8
Fire Retrofitting of High Rise Buildings by Manual Approach

Abstract Concrete is generally fire resistant. A fire in a concrete structure rarely results in a serious damage as to require substantial demolition. To work out proper and efficient repair strategy, however, would require a thorough investigation of the effect of fire on the structural properties of the concrete and steel; the significance which any permanent change in material characteristics may have on the future structural performance of the member; the feasibility of repairs to compensate of any unacceptable reduction in structural performance, durability, and so on; and the influence which fire exposure of individual member may have on the performance of the entire structure. These mentioned tasks are dependent on the complete analysis of the fire-damaged building. Without it, no repair works estimation, extent of repair and kind of repair can be carried out for the fire-damaged buildings. Therefore, the impeccable analysis and design is of utmost importance for repair of such buildings after preliminary investigation of the extent of fire damages to the concrete structural members. This forms the basis of this research study, which aims at detailed analysis and design of the actual existing high-rise fire-damaged buildings for fire retrofitting and assessment of fire damages by non-destructive techniques. Fire damages in buildings due to explosion, accidents or by some other reasons cause severe structural damages. The structural integrity of existing buildings is now a burning issue. Analytical, theoretical and design-cum-construction techniques are constantly being reviewed by government agencies and engineering consultants. Therefore, researchers are delving into this matter to find the best retrofitting techniques for fire-damaged buildings. This paper is an outcome of such detailed research studies. It covers the actual case study of existing buildings, review of existing knowledge for fire damages and their mitigation and protective design technologies, and analytical and computational techniques, which have limited research data. In this study, Extended 3D Analysis of Building Systems (ETABS) is used as software for fire retrofitting analysis, and UBC-97 is used as a code for the fire analysis and design. The ETABS building model is verified by manual calculations as well.

R.R. Hussain et al., *Computer Aided Seismic and Fire Retrofitting Analysis of Existing High Rise Reinforced Concrete Buildings*,
Solid Mechanics and Its Applications 222, DOI 10.1007/978-94-017-7297-6_8

8.1 Introduction

The retrofitting of any existing building is very sophisticated and complex analysis and design procedure which requires not only designing skills but also vigilance in field inspection. The structural engineer must know the layout of the building under retrofitting; the drawings of the buildings with all details and also the existing structural elements of the building with their dimensions must be known to him. As computer and presence of time saving structural design softwares have made life easy for the structural engineers, therefore for retrofitting of the existing building; it is first modeled in the software like ETABS (2008) and then from the analysis results, the suitable retrofitting technique can easily be applied. The analysis and inspection procedure for fire retrofitting of the existing buildings is described below.

8.1.1 Assessment of Column Loads

For the assessment of column loads of the existing building, the dimensions of the existing structural elements beams, slabs and columns are measured first, then these elements are modeled by applying dead, live and seismic loads. The authenticity of the model and the columns loads is then verified by general reinforced concrete analysis. The method of finding the loads of the column will be discussed in the later section of this chapter.

8.2 Initial Evaluation Procedure

8.2.1 Condition Survey

Condition Survey is an examination of concrete for the purpose of identifying and defining area of distress. While it is referred in connection with survey of concrete and embedded reinforcement that is showing some degree of distress, its application is recommended for all buildings and structures. The system is designed to be used for recording the history of the project from its inception to completion and subsequent life.

8.2.1.1 Objective

The objective of Condition Survey of a building structure is:

a. To identify
    i. Causes of distress and
    ii. Their sources;

b. To assess

   i. The extent of distress occurred due to corrosion, fire, earthquake or another reasons.
   ii. The residual strength of the structure and
   iii. Its rehabilitee.

c. To priorities the distressed elements according to seriousness for repairs.
d. To select and plan the effective remedy.

8.2.1.2 Stages

Condition Survey of a building/structure is generally undertaken in four different stages to identify the actual problem so as to ensure that a fruitful outcome is achieved with minimum efforts and at the least cost. The four stages of Condition Survey are:

a. Preliminary Inspection,
b. Planning,
c. Visual Inspection,
d. Field and Laboratory testing.

Preliminary Inspection

A program is evolved to obtain as much information as possible about the distressed structure at reasonable cost and in a reasonable time. Accordingly, the information required from the owner/client is listed out. Even though, many construction details and other related information may not be available with the owners/clients, yet as much as information and details as possible are gathered during the Preliminary Inspection. This information includes:

i. Period of construction.
ii. Construction details including architectural and structural drawings.
iii. Exposure conditions of structure.
iv. Details of repairs, if carried out in the past.
v. Cause of distress.
vi. Photographs of distressed portions of structure.

Planning

Planning stage involves preparation of field documents. These field documents are in the form of floor plans, charts and statistical formats to record relevant data, observations, locations, quality, type and extent of damage etc. Field documents are required for study of damage pattern and its extent as well as to work out Bill of Quantities of various repair items based on condition survey. Also the structural members are grouped as per their type and based on similarity of exposure conditions for proper appreciation of the cause of distress.

Visual Inspection

i. Visual examination of a structure is the most effective qualitative method of evaluation of structural soundness and identifying the typical distress symptoms together with the associated problems.
ii. This provides valuable information to an experienced engineer in regard to its workmanship, structural serviceability and material deterioration mechanism.
iii. It is meant to give a quick scan of the structure to assess its state of general Health. It helps to point out those areas which needs detail inspections i-e core test requirements.

Field/Laboratory Testing Stage

i. It may neither be feasible nor is the practice to conduct field/laboratory testing on every structural member in an existing distressed building.
ii. The field/laboratory testing of structural concrete and reinforcement is to be undertaken, basically for validating the findings of visual inspection.
iii. These may be undertaken on selective basis on representative structural members from each of the various groups based on exposure conditions as explained in the preceding sections.
iv. The programmer of such testing has to be chalked out based on the record of visual inspection.

8.3 Pre-fire Analysis of the Building

The yield strength of the steel bars used in the existing building must be known for future analysis and also the compressive strength of the concrete. The LOADS including dead loads and live loads.

The DEAD LOADS which normally for reinforced concrete building are as follows

Finishes = 35 psf
AC/FC = 10 psf
Partition = 50 psf
Slab Girders = 10 psf

Roof has a dead load of:
Finishes = 60 psf
AC/FC = 10 psf

LIVE LOADS

Live load = 50 psf

These above loads are general floor load for special cases these loads can be increased or decreased.

8.4 Evaluation of Mechanical Properties of Concrete After Fire

8.4.1 Introdution

Non-Destructive Evaluation (NDE) for concrete and components are well known and extensively used. While they are very good tools for establishing quality levels in new constructions, applying these techniques to damaged structures requires certain level of experience and understanding of limitations of these methods.

A single technique may not be adequate and a combination of techniques has to be adopted to get a truly representative data on the condition of the building.

8.4.2 Evaluation Techniques

To assess the existing condition of any structure the compressive strength of in situ concrete must be determined. The quality of concrete is mainly judged by its compressive strength, which directly influences the load bearing capacity and the durability of the structure.

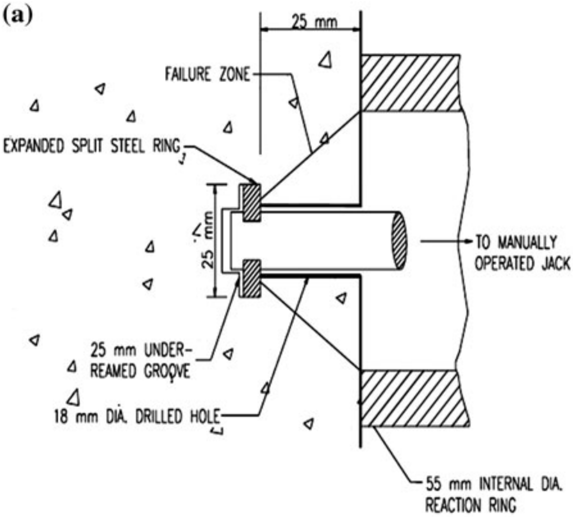
8.4.2.1 The CAPO (Cut and Pull Out) Test

Various international codes of practice have designed this technique as the most reliable and exact method for evaluating the in situ compressive strength of concrete. This is the most recently developed technique and the equipment was planted in April 1996.

In the CAPO-test, the pullout force of a ring, expended in an undercut groove, is a direct measure of the compressive strength, since the concrete between the ring and the counter pressure is being crushed. Testing and Principle of CAPO are shown in Fig. 8.1.

Fig. 8.1 **a** The principle of CAPO test. **b** The performance of CAPO test

(a)



(b)

8.4.2.2 Digi-Schimidth (Digital Rebound Hammer) (ASTM C-805 or BS 1881)

In this test, a testing hammer hits the concrete at a defined energy. Rebound occurs which is dependent on the hardness of concrete and is measured by referring to the conversion tables, the rebound values is used to determine the compressive strength of concrete. Principle of Hammer test is explained through Fig. 8.2.

8.4.2.3 Core Test

Cores extracted from various structural members were first examined in office for assessing depth of discoloration, and then sent to laboratory for verification of

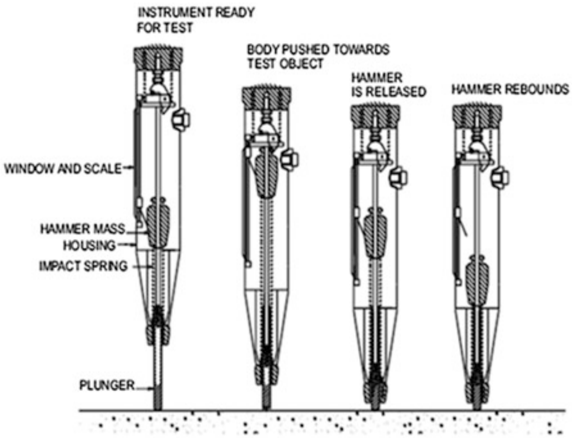


Fig. 8.2 The principle of hammer test

compressive strength of concrete. The field extraction of column core and core sample are shown in Figs. 8.3 and 8.4.

Manual Calculations for Fire Retrofitting of High Rise Buildings

Slab in Shear (Pre-fire)

Assumed Data

Catchment Area = A4 = $22' \times 19' - 3''$
Slab Thickness = 8 in

Fig. 8.3 Performance of core test on columns

Fig. 8.4 Sample for core test

$$\text{Column} = 30'' \times 30''$$
$$\text{Assume} = \#5 - \text{bar}$$

Calculations

$$d = 8 - 0.75 - \tfrac{5}{8} = 6.625 \text{ in}$$
$$b = 12 \text{ in}$$
$$b_o = 4(c + d) = 146.5 \text{ in}$$
$$W_u = \text{load from slab} = 0.314 \text{ ksf}$$

Two Way Shear

$$V_u = W_u\left[l_1 x l_2 - \left(\left(\tfrac{c+d}{12}\right)^2\right)\right]$$
$$V_u = 130 \text{ kips}$$
$$\varnothing V_c = \varnothing 4\sqrt{f_c'}b_o d \qquad \text{(ACI Code Eq. 11.3)}$$
$$\varnothing V_c = 206 \text{ kips}$$
$$\varnothing V_c > V_u$$

Two way shear is satisfied.

One Way Shear

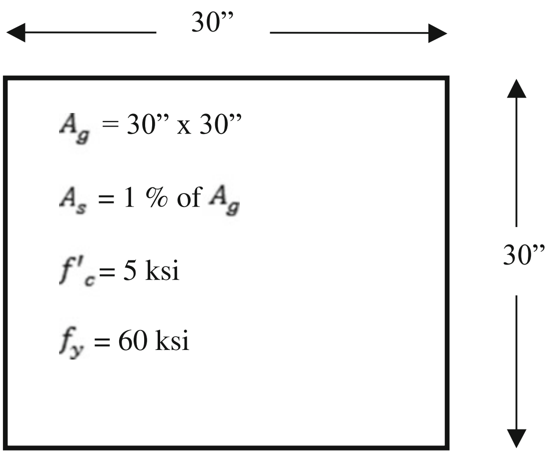
$V_u = W_u \left[\frac{l_1}{2} - \left(\left(\frac{c}{12x2}\right) - \frac{d}{12}\right)\right]$
$V_u = 0.98$ kips
$\varnothing V_c = 2\sqrt{f_c'} b_o d$ (ACI Code Eq. 11.3)
$\varnothing V_c = 8.432$ kips
$\varnothing V_c > V_c$

One way Shear is satisfied.

Adequacy of Pre-fired Column

Assumed Data

Catchment Area of the column is = A = $22' \times 19' - 3'' = 423.5\,\text{ft}^2$
Assumed Column Size = $30'' \times 30''$
W_u = load from slab = 5.547 kips
$\varnothing = 0.85$



Calculations

Load on Column

$P_u = W_u \times$ Area
$P_u = 2439$ kips

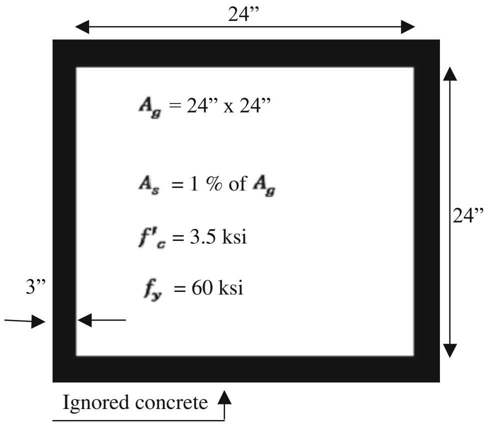
Capacity of Column

$\varnothing P_c = \varnothing 0.8(0.85 f_c'(A_g - A_s) + A_s f_y)$ (ACI Code 10.3.6)
$\varnothing P_c = 3284$ kips
$\varnothing P_c > P_u$

Hence, Column is adequate.

Adequacy of Post Fired Column
Assumed Data

Catchment Area $= A = 22' \times 19' - 3'' = 423.5\,\text{ft}^2$
Column $= 24'' \times 24''$
$W_u =$ load from slab $= 5.502$ kips
$\varnothing = 0.85$



Load on Column

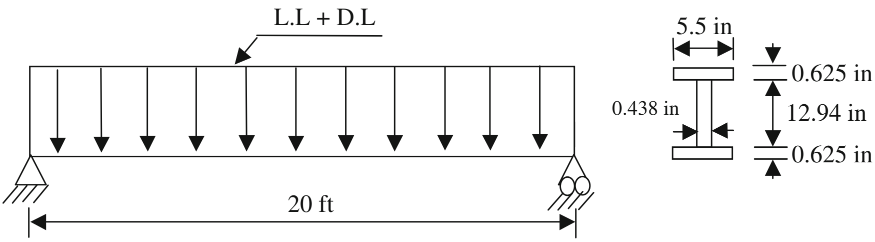
$$P_u = W_u \times Area$$
$$= 5.502 \times 423.5$$
$$P_u = 2330 \text{ kips}$$

Capacity of Column

$\varnothing P_c = \varnothing 0.8(0.85 f'_c(A_g - A_s) + A_s f_y)$ (ACI Code 10.3.6)
$\varnothing P_c = 1612$ kips
$\varnothing P_c < P_u$

Hence, Column is inadequate.

Steel Beam Calculation



The Ultimate load is to be calculated as:

Total Load Wu $= 1.2(20 + 15 + 10 + 15 + 10) + 120 + 1.6 \times 50$
Wu $= 284$ psf
Loads on beam $= 2.84$ kip
$M_u = \frac{Wxl^2}{8}$
$M_u = 142\text{k}-\text{ft}$

Check for Compactness

$\frac{b_f}{2t_f} = 4.4$
$\frac{65}{\sqrt{\text{Fy}}} = 9.2$
$\frac{b_f}{2t_f} < \frac{65}{\sqrt{\text{Fy}}}$

Therefore beam is compacted.
For Nominal Moment

$M_n = M_p = Z_x F_y$
$M_n = M_p = 65.325 \times 50$
$M_p = 272.018\,\text{k}-\text{ft}$
Check $M_p \leq 1.5\, M_y$
$\frac{Z_x}{S_x} = 1.16$
$1.16 < 1.5$ (safe)
$\varnothing M_n = 0.9 \times 272.18$
$\varnothing M_n = 244.9\text{k}-\text{ft} > \text{M}_\text{u}$

Beam satisfied.
Check for LTB

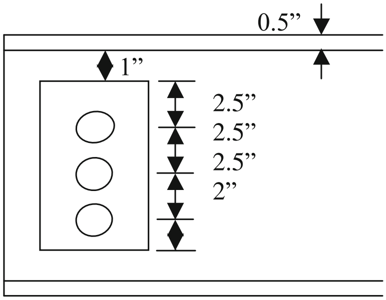
$L_p = \frac{300 r_y}{F_y}$
$C_b = 1.14, r_y = 1.17$ in
$L_p = 50''$ or $4.16'$
$L_p < L_b$; Therefore LTB is present
CHECK $L_p < L_b < L_r$
$L_r = \frac{r_y X_1}{\left(F_{y-} F_r\right)} \sqrt{1 + \sqrt{1 + X_2\left(F_y - F_r\right)^2}}$
$X_1 = \frac{\pi}{S_x}\left(\sqrt{\frac{EGJA}{2}}\right)$
$\text{E} = 29,000\,\text{ksi},\ \text{G} = 11,200, \text{J} = 1.291\,\text{in}^4$
$X_1 =$ 2877 ksi
$X_2 = 4\frac{C_w}{I_y}\left(\frac{S_x}{GJ}\right)^2$

$X_2 = 0.719^{-2}\text{ksi}$
$L_r = \frac{r_y X_1}{(F_{y-}F_r)} \sqrt{1 + \sqrt{1 + X_2 (F_y - F_r)^2}}$
$L_r = 500.34$ in or 41.69 ft
$L_p < L_b < L_r$; Therefore strength is based on Inelastic LTB

Calculating Nominal Capacity

$M_n = C_b \left[M_p - (M_p - M_r) \left(\frac{L_b - L_p}{L_r - L_p} \right) \leq M_p \right]$
$M_n = 268.89$ kip − ft
$M_n < M_p = 272.18\,\text{kip}{-}\text{ft}$

Check for Shear



$V_u \leq \varnothing v \times V_n$
$V_u = 28.4\,\text{kips}$
$V_n = 0.6 F_y A_w$
$V_n = 170.625\,\text{kips}$
$\varnothing V_n = 153.56\,\text{kips}$
$\varnothing V_n > V_u(\text{ok})$

Block Shear

$A_{gv} = (2.5 + 2.5 + 2.5) \times 0.4375$
$A_{gv} = 3.28125\,\text{in}^2$
$A_{ny} = (7.5 - \#\text{ of holes} \times \text{dia of hole})\, t_w$
$A_{ny} = (7.5 \times 2.5 \times 1.125)(0.4375) = 2.507\,\text{in}^2$
$A_{gt} =$ the gross tension area is:
$A_{gt} = 1.75 \times t_w$
$A_{gt} = 0.765625\,\text{in}^2$

The net tension area is

$A_{nt} = (1.75 - \text{dia of hole}/2)\, t_w$

$A_{nt} = 0.51953\,\text{in}^2$
$F_u \times A_{ny} =$ 65 x0.51953 $= 33.769$ kips
$0.6F_u \times A_{ny} = 0.6 \times 65 \times 2.507 = 97.773\,\text{kips}$

Since,

$F_u\,A_{nt} < 0.6\,F_u\,A_{ny}$

Therefore we will use:

$\varnothing R_n = \varnothing(0.6\,F_u\,A_{ny} + F_y \times A_{gt})$
$\varnothing = 0.75$
$\varnothing R_n = 102.04\,\text{kips}$

Check upper limit:

$\varnothing R_n = \varnothing(0.6F_u \times A_{nu} + F_u \times A_{nt})$
$\varnothing R_n = 98.65\,\text{kips}$

Since 98.65 kips < 102.04 kips, the strength is controlled by the upper limit, and

$\varnothing R_n = 98.65\,\text{kips}.$

The maximum factored load reaction based on block shear = 98.65 kips
Deflection

Maximum permissible deflection $= \frac{L}{240}$
$\Delta = 20 \times \frac{12}{240} = 1''$
Total deflection $= \frac{5}{384}\frac{wL^4}{EI}$
$\Delta = 0.8209''$
$\Delta\text{max} > \Delta(\text{safe})$

Adequacy of Column is Shear Before Jacket
The shear force on column from ETABs is calculated to be:

$V_u = 76.27\,\text{kips}$

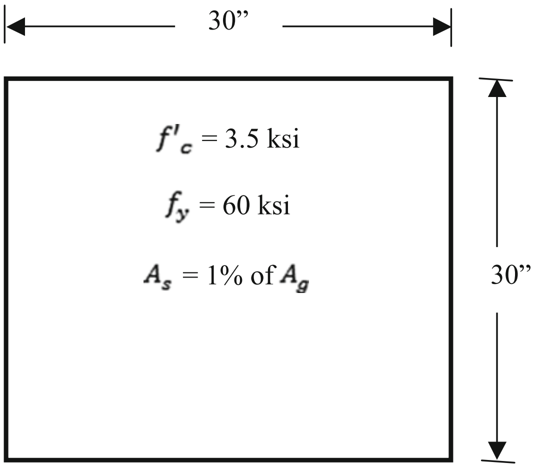
The internal Capacity of the columns is:

$\varnothing V_c = \varnothing\,2\sqrt[2]{f_c' bd}$ (Eq. 11.3 ACI Code)
$\varnothing V_c = 67.08\,\text{kips}$

Since

$\varnothing V_c < V_u$

Therefore, jacketing is required to achieve the compressive strength of concrete on column.



Adequacy of Jacketed Column in Shear
The compressive strength used:

$$f_c' = \frac{[3.5\times 30^2 + 5\times(3\times 36 + 3\times 30)\times 2]}{[30^2 + (3\times 36 + 3\times 30)]}$$
$$f_c' = 4000\,\text{psi}$$

The shear force on column from ETABs is come out as:

$$V_u = 76.27\,\text{kips}$$

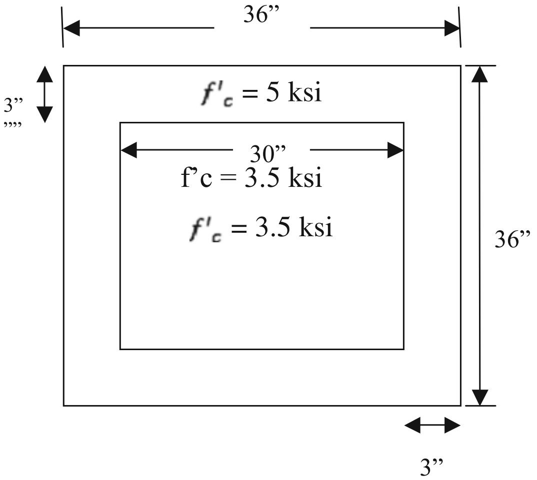
The internal Capacity of the columns is:

$$\varnothing V_c = \varnothing \sqrt[2]{f_c' bd} \quad \text{Eq. 11.3(ACI Code)}$$
$$\varnothing V_c = 119.5\,\text{kips}$$

Since,

$$\varnothing V_c > V_u$$

Therefore, now is adequate.



Adequacy of Revised Column for Axial Loads

Area = A4 = 423.5 ft^2
Revised Column = 36″ × 36″

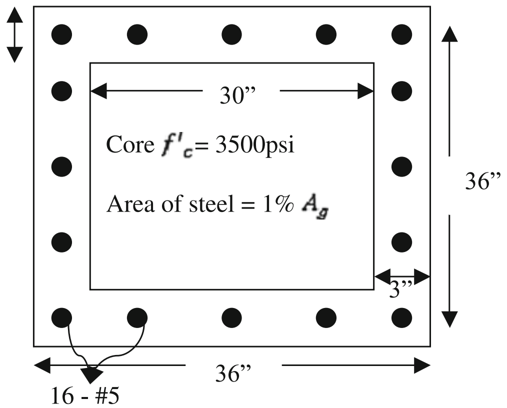
a. Load on Column
By the previous calculation load on column (36″ × 36″) is:

$P_u = 1855$ kips

b. Capacity of Column

Capacity of Internal Core of Column
Let Capacity of internal core of column (ignoring 1.5″ thick layer from all sides) with assuming 1 % of steel and $f'_c = 3.5$ ksi. This capacity should be able to resist ultimate self weight of structure.

$\varnothing P_c = \varnothing 0.8(0.85 f'_c(A_g - A_s) + A_s f_y)$ (ACI Code 10.3.6)
$A_g = (30 - 3) \times (30 - 3)$
$A_g = 27 \times 27$
$A_s = 1\,\%$ of gross area
$f_y = 60{,}000$psi
$\varnothing = 0.65$
$\varnothing P_c = 1504$ kips



Capacity of Jacketing of Column

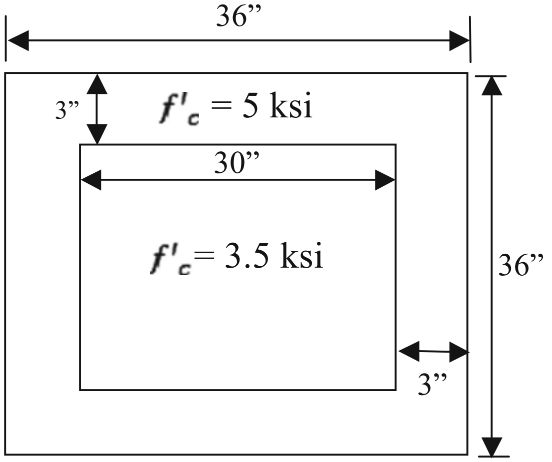
$\varnothing P_c = \varnothing 0.8(0.85 f'_c(A_g - A_s) + A_s f_y)$ (ACI Code 10.3.6)
$A_g = (36 \times 36) - (30 \times 30)$
$A_g = 396\,\text{in}^2$
$A_s = 16\ -\ \#5\,\text{bars}$
$f_y = 60\,\text{ksi}$
$\varnothing\ =\ 0.65$
$f'_c = 3500\,\text{psi}$
$\varnothing$ Pc $= 846\,\text{kips}$
Total Capacity of Column = 1504 + 846
$\varnothing P_c =\ 2350\,\text{kips}$

Since,

$\varnothing P_c > P_u$

Hence, jacketed column is adequate.
Shear Interface
Transformed the internal core area

$n = \frac{E_{CJ}}{E_{CC}}$
E_{CJ} = Modulus of Elasticity of Jacket Concrete
E_{Cc} = Modulus of Elasticity of core Concrete
n = 1.2
$nA_c = 1.2 \times 30'' \times 30''$
$nA_c = 1080\,\text{in}^2$



The internal Shear resistance between core and jacketing is calculated by:

$v = \frac{VQ}{Ib}$

Since,

V = Shear Force = 56.51 kips(from ETABs)
Q = First moment of Inertia = A × y = 36 × 36 × 16.5

$Q = 1782\,in^3$
I = Moment of Inertia $= \frac{bh^3}{12}$
$I = 139968\,in^4$
b = 30 in
then, ν = 0.024 ksi = 24 psi

Reference

ETABS Software (2008) Version 9.2, Computer and structures, Inc. Berkeley, California, USA

Glossary

Axial Load Force directly coincident with the primary axis of a structural member such as a beam.

Adequacy The quality of being able to meet a need satisfactorily.

Base Shear The total shear force acting at the base of a structure.

Block Shear The shear which tears out the segment of a material at the end of the member.

Compressive Strength The measured resistance of a concrete or mortar specimen to axial loading expressed as pounds per square inch of cross-sectional area; the maximum compressive stress which material, such as portland cement, concrete, or grout is capable of sustaining.

Cover The least distance between the surface of the reinforcement and the outer surface of the concrete.

Deflection Displacement or bending of a structural member due to application of external force.

Deformation Any change of form, shape, or dimensions produced in a body by stress or force, with rupture.

Degradation A decline in quality; degeneration.

Design Life Length of time that the building is presumed to remain functional.

Deterioration Decline in the quality of equipment or structures over a period of time due to the chemical or physical action of the environment.

Drift The horizontal displacement or movement of structure when subjected to lateral forces.

Ductility The ability of a material to be plastically deformed by elongation, without fracture.

Durability The characteristics of a structure to resist gradual degradation of its serviceability in a given environment for the design service life.

R.R. Hussain et al., *Computer Aided Seismic and Fire Retrofitting Analysis of Existing High Rise Reinforced Concrete Buildings*,
Solid Mechanics and Its Applications 222, DOI 10.1007/978-94-017-7297-6

Ferrocement It is thin walled reinforcement cementitious construction, using more than one layer of continuous small diameter metallic wire mesh.

Fiber wraps Technique Integrating woven or non-woven high strength fibre with epoxy around structural element to protect its structural capacity.

Flexural Strength That property of a solid which is an indication of its ability to withstand internal tension and compression stresses.

Formwork Temporary structure built to contain concrete while it sets; also called Form.

Girder A beam that supports other beams; a very large beam, especially one that is built up from smaller elements; a timber beam used to support wall beams or joists.

Grouting Process of filling tile joints, masonry block or cells, or any masonry type product with grout.

Gutted Building Utterly exhausted building due to high temperature.

Lateral Load The horizontal component of the load produced by an arch, dome, vault, or rigid frame or a horizontal load applied to a structure or member such as wind or earthquake.

Non Destructive Evaluation The sampling/testing in laboratory/field without affecting, the functional utility of structure.

Passivation Formation of a protective layer of ferric oxide around embedded reinforcement under highly alkaline environment.

Patch Repair The repair to a position of a structural member to restore it to its original state.

Rebound Aggregate and cement or wet shotcrete that bounces away from a surface against which it is being projected.

Rehabilitation The process of repairing or modifying a structure to a desired useful condition.

Rehabilitation The process of repairing or modifying a structure to a desired useful condition.

Reinstate Restore to a previous condition or position.

Repairs Replace or correct damaged or faulty materials or elements of structure.

Residual Strength The load or force that a damaged object or material is still carry without falling.

Retrofitting Upgrading the existing structure to meet the enhanced structural requirements in terms of load carrying capacity of existing structural element or by introducing additional structural members integral to the existing structure.

Service Life The time taken from the completion of a structure till the structure is no longer usable due to deterioration process.

Serviceability The necessary performance requirement of a structure to meet its intended function.

Shear Interface A surface forming a common boundary between two regions or a point where interaction occurs between two systems or processes.

Shear Wall A wall which in its own plane carries shear, resulting from forces such as wind, blast or earthquake.

Shear strength The maximum shear stress which a material can withstand without rupture.

Shuttering The structure of boards that make up a form for pouring concrete in construction.

Spalling The cracking, breaking, or splintering of materials, usually due to heat or loss of strength and integrity of cover of concrete with the interior concrete due to expansive force.

Stiffness The ratio of force applied to a structure (or structural element) to the corresponding displacement.

Strengthening Measures taken for a deteriorated structure or any of its structural member to restore its design load carrying capacity.

Superimposed Load Loads and stresses added to the dead load of the structure; live load.

Superstructure The part of a building above the foundation.

Yield Strength The stress at which a material exhibits a specified deviation from proportionality of stress and strain.

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