T.C.

ALTINBAS UNIVERSITY

Institute of Graduate Studies

Civil Engineering

SEISMIC ASSESSMENT OF PUBLIC HOSPITALS IN PALESTINE

Ahmad ABURAYYAN

Master of Science

Supervisor

Prof. Dr. Zeki HASGUR

Istanbul, 2021

SEISMIC ASSESSMENT OF PUBLIC HOSPITALS IN PALESTINE

by

Ahmad Ghandi Ahmad Aburayyan

Civil Engineering

Submitted to the Graduate School of Science and Engineering

in partial fulfillment of the requirements for the degree of

Master of Science

ALTINBAŞ UNIVERSITY

2021

The thesis titled "SEISMIC ASSESSMENT OF PUBLIC HOSPITALS IN PALESTINE" prepared and presented by Ahmad ABURAYYAN was accepted as a Master of Science Thesis in Civil Engineering.

____

Prof. Dr. Zeki HASGUR

Supervisor

Thesis Defense Jury Members:

Prof. Dr. Zeki HASGUR	School of Engineering and Natural Sciences, Altinbas University	___________________
Prof. Dr. Tuncer CELIK	School of Engineering and Natural Sciences, Altinbas University	___________________
Asst. Prof. Dr. Erdem DAMCI	School of Engineering and Natural Sciences, Istanbul University	___________________

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Approval Date of Institute of Graduate Studies:
____/____/____

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Ahmad Ghandi Ahmad Aburayyan

Signature

DEDICATION

I dedicate my thesis work to my family and many friends. A special feeling of gratitude to my loving parents, whose words of encouragement and push for tenacity ring in my ears. My sisters and brothers have never left my side and are very special. I also dedicate this dissertation to my many friends who have supported me throughout the process.

I will always appreciate everything they did, especially Dr. Belal Mahmoud Almassri and my professors for helping me.

I dedicate this work to my best friend, Rashed Salem

Dr. Mahmoud Arman, Oday Shaheen and Dr. Maan Ahmad

I dedicate this work To My Home, Society; Holy Land of Palestine.

To everyone working in this field

To all of them

ACKNOWLEDGEMENTS

First of all, praise is to Allah for helping me in making this research possible.

Then, I would like to express great thanks and sincere gratitude to my supervisors:

Prof. Dr. Zeki HASGUR

For their guidance, suggestions and assistance during the preparation of this thesis.

Special mention goes to my father and My mother

My brother, sisters,

My Friends and colleagues.

ABSTRACT

Seismic Assessment of Public hospitals in Palestine

Aburayyan, Ahmad

M.Sc. Civil Engineering Department, Altınbaş University

Supervisor Prof. Dr. Zeki HASGÜR

Co- Supervisor Dr. Belal Mahmoud ALMASSRI

Date: July/2021

Pages:149.

Due to their diverse complexities, hospitals have specific regulatory construction requirements to mitigate physical and functional vulnerabilities that can arise during seismic events. Special considerations in seismic risk mitigation are needed to create a safe environment for patients and staff during disasters as well as play a crucial role in saving lives and reducing the suffering after disasters. The importance of safe and reliable hospital structures makes it abundantly clear that the engineers' primary duty to the public is ensuring that hospitals can withstand disasters related to seismic events. Our main concern is seismic parameters that take into account site conditions. In this research, we present a topographic influence and the effect of its modifications on seismic behavior, as well as differences in the design of public-hospital facilities in Palestine based on the time period of the basic mode of construction according to codes of practice. It has been shown that seismic behavior varies from one region to another and from one structure to another according to many variables including the type of soil to be built on, and the seismic behavior variables across regions and structures , different terrains directly affect seismic design elements, system considerations, and structural analysis methodologies. This study chose different types of public hospitals in Palestine and collected various statistical information about the nature of their practice in the previous years. The data were used to build three distinct hospital models based on three different topographies: Qalqilya, Ramallah, and Jericho.

To complete the analysis and provide the required suggestions, the time frame for the basic model of the structure should be established as a minimum to reach the safety of public hospitals in multiple building systems of the same structure. The local code-based assessment was carried out using a variety of tools, including the ETABS program, which employed the capacity spectrum method. Pushover analysis is a form of non-linear static analysis that is used in the construction industry. Results have been reached that put the structural designer on the right track for safe structural design within the engineering specifications and conditions

Keywords: Seismic Assessment, Hospitals, Pushover Analysis.

TABLE OF CONTENTS

Pages

ABSTRACT……………………………………..…….…………...………………...vii

LIST OF TABELS…………..……………………………….............................….xiii

LIST OF FIGURES………………………………...……...……..…………….….xv

LIST OF ABBREVIATIONS …………..……………………................................xviii

LIST OF SYMBOLS ……………………...…………….….……………….......xx

1. INTRODUCTION 1

1.1 RESEARCH PROBLEM……………………………………………………3

1.2 OBJECTIVES AND METHODOLOGY………….…………………………3

2. LITERATURE REVIEW 6

2.1 SEISMIC ASSESSMENT 7

2.2 SEISMIC PERFORMANCE EVALUATION 8

2.3 SEISMIC RELIABILITY ANALYSIS 10

2.4 SEISMIC RISK ASSESSMENT.. 12

2.4.1 Hazard Assessment…..............................…………………………..…12

2.5 STRUCTURAL PERFORMANCE ASSESSMENT 13

2.5.1 Analysis Methods 13

2.5.1.1 Linear Methods ….……….…………….…..……………………13

2.5.1.2 Nonlinear Static Analysis ……….………………………...….......14

2.5.1.3 Nonlinear Dynamic Analysis ………..………………………………15

2.6 SEISMIC PERFORMANCE CLASSIFICATION ……………………........16

2.7 RETROFITTING (REHABILITATION STRATEGIES) …………………..…17

2.7.1 Retrofit Strategies …………………………...……...……………....…17

2.7.2 Rehabilitation Options ……………………...…………..………………20

2.8 EXISTING PUBLIC BUILDINGS ……………………..……………………...…20

2.8.1 Existing Buildings ……………….…………...…………………………20

2.8.2 Hospital Buildings ……………………………….……………………..…21

2.9 INTERNATIONAL BUILDING CODE ……………………….…...............21
2.9.1 International Existing Building Code………………………...............21
2.9.2 Seismic Design By 2003 International Building Code…………………...22
2.10 SEISMIC DESIGN CONCEPTS ………….………….........................22
2.10.1 Behavior Of Existing Buildings To Seismic Loading ………………....24
2.10.2 Seismic Ground Motion ……………..……….……..........................24
2.10.3 Soil Impact And Seismic Soil Structure Interaction 25
3. GENERRL STATISTICS AND MODELING………………………………27
3.1 DATA COLLECTING AND PARAMETERS CONSTRUCTION…........…27
3.1.1 Structural system type …………………………..……..………………27
3.1.2 Number of floors...………..…………………….…………………………27
3.1.3 Presence of Soft Story and Short Columns………..….……………………28
3.1.4 Building Age ……………..……..……………………………………….29
3.2 DIMENSIONS AND LAYOUT ……………………….....……..…………..29
3.3 STRUCTURAL SYSTEMS ……………………………………….……….34
3.3.1 Structural Walls and MRF Systems ………………...………………...34
3.3.2 Solid and Ribbed Slabs …………...…………………….....................36
3.4 OTHER FINDINGS …….………………………...…..…………………37
3.5 SUMMARY OF ALL STATISTICS ……………………….....................38
3.6 MODELING ………………………………………………………...………38
3.6.1 Pushover Analysis …………………………………........................38
3.6.2 Beam Column Joint …..……………………………...…………………45
3.6.3 Structural Irregularities ……………………..……..……………………45
3.6.4 Real (Practice) Model ……………………………….........................47
3.6.5 Sensitivity Study and Meshing Size……………………………………...48
3.6.6 Material Properties…………………………….……..……………………48
3.6.7 Loads on Hospitals Buildings………………………..………………..49

3.7 PROPOSED MODELS……………………………………………………………51

3.7.1 Geometry and Restraints …………………………………………………51

4. TOPOGRAPHIC COMPARISONS AND CALCULATIONS ……….........64

4.1 FORCE DEFORMATION SHAPE ……………………………………………64

4.2 SEISMIC HAZARD MAPS ………………………………………….............66

4.3 SITE CLASSIFICATION AND SITE COEFFICIENTS ………………….68

4.4 PROPOSED SOIL AND SITE CLASSIFICATION……………………….71

4.4.1 Ramallah Region Hospital (Moderate Conditions) ……………………74

4.4.2 Qalqilya Region Hospital (Mild Conditions) ……………………………80

4.4.3 Jericho Region Hospital (sever Conditions) ……….............................86

4.5 SUMMARY OF ALL TYPICAL HOSPITALS …………………………93

4.6 DESIGN RESPONSE SPECTRA……………………………………….....94

4.6.1 Response Spectra from Ground Motion Recording ……………………..94

5. MAIN REQUIREMENTS OF MRF …………………………………….....96

5.1 ORDINARY MOMENT RESISTING FRAMES (OMRF) ……………………97

5.2 INTERMEDIATE MOMENT RESISTING FRAMES (IMRF) ……..............97

5.3 SPECIAL MOMENT RESISTING FRAMES (SMRF) ……………………....98

5.4 ADDITIONAL POINTS TO CONSIDER………………………….........101

5.4.1 Non-Structural Components Effects ………………………………101

5.4.2 Acceleration and Displacement effect on Time Period……………….103

5.4.3 Resonance ………………………………………………………………104

5.4.4 Ribbed Slabs issue ………………………………..........................…105

5.4.5 Topography amplification on earthquakes ……….........................106

6. CONCLUSIONS……………………………………………………….....107

6.1 DISCUSSION ……………………………………………………………107

6.2 FUTURE SCOPE OF STUDY …………………………………………....108

REFERENCES ……………………………………………………………109

APPENDIX A ………………….………………...……..**116**

APPENDIX B ………………….………………...……..**121**

APPENDIX C ………………….…………..……………..…………………….......**132**

APPENDIX D ………………….…………..……………...……............................**134**

LIST OF TABLES

Pages

Table 2.1: Analysis Procedures for Seismic Design and Assessment …………….....13

Table2.2:SeismicPerformance……………………………………………………..17

Table 3.1: Statistics Summary ..…………………………………………………...38

Table 3.2: Practice Model Elements Dimensions …………………………..…….47

Table 3.3 The Automatic Meshing ………………………………...……….…….48

Table 3.4: Properties of Model (A) …………………………..…………………..51

Table 3.5: Properties of Model (B) ………….……………………………………...56

Table 3.6: Properties of Model (C) ………………………………........................59

Table 4.1: TABLE 1613.3.3(1) Value of Site Coefficient F_a 69

Table 4.2: TABLE 1613.3.3(2) Value Of Site Coefficient F_v ………………………69

Table 4.3: Table 20.3-1 Site Classification …………………..……………………...70

Table 4.4: Table 1.5-1 Risk Category Of Building And Other Structure,………,…..76

Table 4.5:Table11.61 Seismic Design Category Based…………...…………………77

Table4.6:Table11.62 Seismic Design Category Based……………...…………….……77

Table 4.7: Table 1.5-2 Importance Factors by Risk Category ………………………78

Table 4.8: Table 11.6 Seismic Design Category ……….………….....................…79

Table 4.9: Table 12.6-1 Permitted Analytical Procedure ……...…………………....79

Table 4.10: Summary of All Types Hospitals in Palestine ……..…..........................93

Table A.1: Table 24.3.2 from ACI38-14 for $Smax$ ………..……………………...119

Table B.1: Columns Dimensions Limitation in SMRF ………...…..………………121

Table B.2: Transverse Reinforcement Spacing in SMRF …..……...………………124

Table B.3:ACI31814/Table18.7.5.4Transverse Reinforcement…………….…...........125

Table B.4: Common Use For Hoops In Beams of IMRF …………………………..128

Table B.5: Common Use For Hoops In Columns Of IMRF ……………..…..….....130

Table C.1: Minimum Dimension………………………...….....………………..132

Table D.1:Code Provisions……………………………………………..………....134

LIST OF FIGURES

Pages

Figure 2.1: Capacities For The Type-2B (5-Story)Structure…………………………..…..9

Figure 2.2: Capacities For The Type-2A (5-Story) Structure……………….……………10

Figure2.3: There Are Many Classes……………………………………………...…..11

Figure 2.4: The Basic Conversion ………………….………………………………15

Figure 2.5: Simplified Design ……………….…………………………..……..…16

Figure 2.6: Bracing Configuration: Plan (a) And 3D (b)…….……........................…19

Figure 2.7: Seismic Design Spectrum of the 2003 IBC …….…..……………………22

Figure 3.1: This Figure From ETABS To Enter the Information …………………......28

Figure 3.2: Statistics for Stories Properties of Hospitals in Palestine 30

Figure 3.3: Statistics for Columns Widths of Hospitals in Palestine..........................30

Figure 3.4: Statistics for Columns Lengths of Hospitals in Palestine ……………….31

Figure 3.5: Statistics for Beams …………………………..………………………31

Figure 3.6: Statistics for Beams Cross Section ……………………………………...32

Figure 3.7: Stress and Strain Diagram for The Reinforced ……………….…....…...32

Figure 3.8: Statistics for Beams Steel Ratios of Hospitals in Palestine……………...33

Figure 3.9: Statistics for Columns Steel Ratios of Hospitals in Palestine…………....33

Figure 3.10: Statistics for Structural Systems of Hospitals in Palestine……………..35

Figure 3.11: Statistics for Types of Slabs Used in Hospitals in Palestine………...…36

Figure 3.12: Force-displacement curve defined …………………..………………...39

Figure 3.13: Plastic hinge formation ……………………………………………...40

Figure 3.14: Define load cases in ETABS ……………….……………...……..…41

Figure 3.15: Load case data in ETABS ………………….……........................41

Figure 3.16: Select object type in ETABS …………….……................................42

Figure 3.17: Frame assignment - hinge in ETABS …...…………..………….…...42

Figure 3.18: Auto hinge assignment data (columns) in ETABS ………………...…43

Figure 3.19: Auto hinge assignment data (beams) in ETABS……………………....43

Figure 3.20: Shell assignment – hinges in ETABS ……………………………………44

Figure 3.21: Frame assignment – hinge overwrites in ETABS…..…........................44

Figure 3.22: Typical Model of hospital ………………………………………...47

Figure 3.23: Material Properties data from ETABS ………………......................…49

Figure 3.24: Seismic Parameters of practice Typical Model ……………...……...50

Figure 3.25: structure looks when it has been analyzed and exposed to loads ……...50

Figure 3.26: Typical Model of hospital (A)…………..…………………………...52

Figure 3.27: Plan for Model (A) ……..…...………………………………………53

Figure 3.28: Base shear force curve for Model (A)...……………………………….53

Figure 3.29: columns curve for Model (A)…………..……………………………54

Figure 3.30: Beams curve for Model (A)………………..………………………54

Figure 3.31: Pushover curve for Model (A) ……………………......................….55

Figure 3.32: Wall curve for Model (A)…………………………………………55

Figure 3.33: Typical Model of hospital (B)…………………………………...……56

Figure 3.34: Plan for Model (B) ……………………………………………………..57

Figure 3.35: Base shear force curve for Model (B) …………………………………57

Figure 3.36: columns curve for Model (B)…………………………………………...58

Figure 3.37: Beams curve for Model (B)………………………...…………..….58

Figure 3.38: Pushover Curve for Model (B)……………………….……...………59

Figure 3.39: Typical Model of hospital (C)………………………………….....…..59

Figure 3.40: Plan for Model (C) …………………………………………………..…60

Figure 3.41: Base shear force curve for Model (C)………………………….………61

Figure 3.42: columns curve for Model (C)…………………………………………...61

Figure 3.43: Beams curve for Model (C)……………………………...………..62

Figure 3.44: Pushover curve for Model (C)………………………………….....…..62

Figure 3.45: Walls curve for Model (C)…………………………………...……63

Figure 4.1: In-Elastic Force Deformation Shape ……………………….……………65
Figure 4.2: UBC97 (Z) Seismic Zone Factors for Palestine ……..……..……….….67
Figure 4.3: Seismic Hazard Map for Building Codes for Palestine ....………………68
Figure 4.4: Long Period Spectral Acceleration For Palestine ……….......................72
Figure 4.5: Short Period Spectral Acceleration For Palestine ……...………………..73
Figure 4.6: Design Response Spectra …………………………...…......................76
Figure 4.7: Design Response Spectra ……………………...…………....................82
Figure 4.8: Design Response Spectra ………………...89
Figure 4.9: Design Response Spectra for Samples of Hospitals in Palestine...………95
Figure 5.1: Minimum Reinforcement Details for Columns...………….………………99
Figure 5.2: Estimated Height Limits for Different Structural System ……………..100
Figure 5.3: Overlapped of Non-Structural Components………………………...…103
Figure 5.4: Schematic Diagram Joining Acceleration………...…………………..…104
Figure A.1: ACI318-14 Beams Parameters …………………………………...….116
Figure A.2: ACI318-14/Fig R18.6.4 Examples of Overlapping Hoop……………..120
Figure B.1: ACI318-14/Fig R18.7.5.2 Examples of Transverse Reinforcement..….123
Figure B.2: Continuous Bars in Ordinary MRF ………….……………………...126
Figure B.3: Hoops at Ends of Beams in IMRF …..………………........................127
Figure B.4: Hoops at Ends of Beams in IMRF …………………......................…..129
Figure B.5: Maximum Transverse Reinforcement In IMRF …………………….....129
Figure B.6: Hoops at Columns in IMRF ……………………...…………………131
Figure C.1: ACI318-14/Fig R18.8.4 Effective Joint Area ……………………….....133
Figure C.3: Types Of Joints In A Moment Resisting Frame ………………....…...134

LIST OF ABBREVIATIONS

RC	:	Reinforced Concrete
SW	:	Shear Wall
DLs	:	Dead Loads
Els	:	Earthquake Loads
UBC	:	Uniform Building Code
MRFs	:	Moment-Resisting Frames
AAL	:	Average Annual Loss
SDL	:	Superimposed Dead Load
FBD	:	Force-Based Design
PGA	:	Peak Ground Acceleration
GLD	:	Gravity Load Designed
SDC	:	Seismic Design Categories
NSC	:	Non-Structural Components
ACI	:	American Concrete Institute
NSPs	:	National Society for Professional Surveyors
SRSS	:	Square Root of the Sum of the Squares
MDOF	:	Multiple Degree Of Freedom
GFRP	:	Glass Fiber Reinforced Plastics
HDRB	:	High Damping Rubber Bearing
ICBO	:	International Conference of Building Officials
BOCA	:	Building Officials and Code Administrators international
IEBC	:	International Existing Building Code
SMRF	:	Special Moment Resisting Frames
IMRF	:	Intermediate Moment Resisting Frames
OMRF	:	Ordinary Moment Resisting Frames

NEHRP	:	National Earthquake Hazards Reduction Program
OSHPD	:	Office of Statewide Health Planning and Development
SHARE	:	Seismic Hazard Harmonization in Europe
SBCCI	:	Southern Building Code Congress International
ASCE / SEI	:	American Society of Civil Engineers- Structural Engineering Institute

LIST OF SYMBOLS

Pages

B : Plan dimension of the building in the direction of period calculation

fi : Lateral Force at level i of the floor

G : Shear modulus

g : Gravity acceleration

H : Height of the building

h : height of the floor

I : Moment of inertia

Ic : Column moment of inertia

j : Polar moment of inertia of the plan

K : Stiffness of the structure

m : Mass of the structure

n : Number of floors

P : Lateral load

T : Period of the structure

W : Unit floor weight

Wi : Weight at level i of the floor

δi : Elastic deflection due to lateral force at level i of the floor

υ : Poisson's ratio

Δ : Total Lateral deflection

D : Dead load

E : Earthquake load

L : Live load

ϕ : Structural resistance factor

Ω_O : Overstrength factor for the lateral-force-resisting system.

g : Acceleration caused by gravity

C_d : Deflection amplification factor

F_a : Short-period site coefficient (at 0.2-s period)

F_v : Long-period site coefficient (at 1.0-s period)

f_y : Specified yield strength of reinforcement [psi (MPa)]

I_e : The Importance Factor

I_p : The component importance factor

KL/r : The lateral slenderness ratio

L : Overall length of the building (ft or m)

PGA: Peak ground acceleration shown

R : Response modification coefficient

S1 : 5% damped, spectral response acceleration parameter at a period of 1 s.

SD1 : 5% damped, spectral response acceleration parameter at a period of 1s.

SDS : 5% damped, spectral response acceleration parameter at short periods.

SM1 : 5% damped, spectral response acceleration parameter at a period of 1s.

SMS : 5% damped, spectral response acceleration parameter at short periods.

T : The fundamental period of the building

V : Total design lateral force or shear at the base

W : Effective seismic weight of the building

γ : Average unit weight of soil, in lb=ft3 (N=m3)

Δ : Design story drift

θ : Stability coefficient for P-delta effects

1. INTRODUCTION

As one of the most important challenges in seismic engineering, the assessment of existing buildings using seismic technology is required. It is known as an extreme act, the product is defined by the occurrence of an unpredictable event throughout its service life, making it necessarily probabilistic in structure design and evaluation.

It is generally acknowledged that predicting ground motion characteristics in a particular location is impossible. In recent years, great effort has been made to reduce the unavoidable uncertainty that comes with seismic events by conducting a systematic mapping of all of the places that are vulnerable to seismic activity to determine the level of seismic hazard, and then utilizing the most accurate seismic prediction.

Once a thorough understanding of the site's seismic risk has been established, the seismic evaluation of structures using nonlinear dynamic analysis necessitates the selection or construction of a suitable collection of ground motions based on the existing data. .[1]-[3]

The hospital system is considered a vital facility in hazardous situations, and it plays an important role in disaster relief efforts, particularly following an earthquake. For example, the earthquake that struck San Fernando in 1971 caused serious damage to hospital units. Going to follow the Northridge earthquake in California in 1994, the state government hired an earthquake study to examine the hospital and nursing home damage in about one-third of hospitals and nearly half of nursing homes.

A total of 20 of the facilities found to be in violation included nursing homes and 8 of the hospitals. Damaged hospital systems have already been seen after major earthquakes in Kobe (1995) and Tohoku (2011). Almost 20% of hospitals throughout the earthquake-affected region were either partially or fully nonfunctional following the disaster. In the 2013 Lushan earthquake, 9 of the 22 district medical facilities sustained severe to major damage. In the event if three entire hospitals had been entirely evacuated, all three of the hospitals would have lost their full functioning. Of the remaining hospitals, one had more than half of its capabilities erased, while another lost half of its usefulness.

As a result of the 2016 Central Italy earthquake, numerous medium and small hospitals were damaged to the point that they no longer functioned, and had to transfer patients to other hospitals. Following the devastating 2017 earthquake in Mexico, the major hospital had to be emptied. Because of the hospitals' reduced efficiency, a thorough research was undertaken to increase the hospitals' seismic efficiency, and as a result, the hospital's emergency rescue was disrupted during the earthquakes[4]. To assess seismic activity and structural system performance, there are three main approaches.

a) Laboratory Testing:

Large-scale physical objects such as full subsystems, elements, or (if the equipment is huge sufficient) entire systems are subjected to static, quasi-static, or dynamic load testing. While using shake tables or energy consumption and mechanically controlled load techniques is preferred, systems must be tested in emulated controlled settings when it is not used. Since the early 1950s, the amount and efficiency of laboratory research has increased, with engineering institutions playing a significant role in the United States. Laboratory testing has also helped us better comprehend complicated soil characteristics and the phenomena of soil-structure interaction

b) Numerical Analysis:

Structures are analyzed for specified loads calculated either by code requirements or projected ground motions unique to a given location using special purpose public-domain or private software.

c) Earth's Natural Laboratory:

The third basic strategy is to use the Earth's natural laboratory to test the behavior and efficiency of structural systems, by observing and analyzing the performance (and probably the damage to structures) after earthquakes. Important improvements in improved designs can be made by evaluating why existing designs are poorly resilient to earthquakes, and new designs are fully tested in laboratories. A nature lab would be a location with an increased risk of earthquakes that offers a diversity of system design structures for study; heavy ground movements as well as moderate-level movements will be observed frequently in optimal test areas.

In order to monitor their responses during future earthquakes the modern equipment of selection structures is key to the approach of the natural laboratory. Therefore, it is crucial to develop and build integrated instruments to completely assess the connection of ground motion which originates to start, there are two points of origin: where the creative source begins, and where it is moved by the soil from one point to another. When you think about it, the road that combined geologists and architects go down is clear. linked networks are created that are designed to detect seismic source, ground motion transfer and structure reaction processes. The pushover study is performed out by putting the structure into the estimated base shear for limiting displacement, then pushing the structure to a complete collapse status and obtaining a pushover curve using ETABS

1.1 RESEARCH PROBLEM

The Palestinian society of structural designers, either in the public or private sector lacks the seismic guidelines related to public and important buildings, these building-like hospitals and schools require special consideration in structural design especially regarding the seismic notes.

1.2 OBJECTIVES AND METHODOLOGY

Research objectives are listed below:

a) To study the actual design of an existing hospital.

b) General Statistics study will be done for all hospitals as built drawings.

c) Investigation study showing the real structural properties of existing hospitals with respect to IBC Code requirements.

d) Evaluate structures and all components of hospitals (nonstructural elements …) and develop buildings through using additional requirements and special needs based on international codes.

e) Generate special rules for hospital design under its local practice in implementations as per region limitations and other special needs that shall govern the design.

f) Achieving the best limitations for hospitals to be rigid structures in terms of structure's Time Period that insure rigidity of structure.

g) Implement of a ETABS numerical simulation framework for investigation the behavior of seismically detailed of different hospitals in Palestine (From North to South based on topography and soil type).

The objective of this research is to find out the extent to which reinforcing concrete structures in such a population are at risk. It serves as a guidance for how hospital buildings should be used, but may also be adapted to other contexts (e.g., schools).

All mentioned will be shown in this thesis as following:

a) *Chapter 2: Literature review* presents several previous studies which were implemented on similar topics, procedures used and all details of special conditions analyzed based on several scientific Methodologies.

b) *Chapter 3* of this research shows engineering statistics for several hospitals in Palestine showing all needed data that support and provide research with a clear view of existing status of hospital structures.

The data will focus on the following:

a) Structural Systems.
b) Columns dimensions.
c) Beams Dimensions.
d) Slab's systems.
e) No. of Floors.

The gained data helped in improving hospitals buildings by its all components so this makes a good guidance in the first modeling of analysis being the first step of improvement.

A model of Typical hospital case is presented (represents all hospitals in Palestine by its own architectural shape) with the case modeled by ETABS software in all its dimensions and applying the loads as per codes and its own functions and accordingly the results displayed as per codes requirements.

c) *Chapter four:* displays a comparative study of three chosen hospitals selected in three different topography regions in Palestine (Jericho, Ramallah, Qalqilya) .These three hospital buildings were analyzed seismically and compared with seismic Code requirements displaying all findings and missing criteria.

d) *Chapter Five:* shows the main Code requirements of Moment resisting frames systems by code of practice and all conditions that shall be available in every design procedure for all ordinary, intermediate, and Special Moment Resisting Frames (MRF) Systems.

2. LITERATURE REVIEW

In recent decades, substantial consideration has been devoted to researching the likelihood of earthquake activity, how earthquakes can affect an area when they strike, and how current structures can be shielded from them. High regional seismicity and the vast number of precarious systems that exist have enabled the government and policies to pay more attention to the topic of urban risk management, especially of critical/strategic buildings and infrastructure.

In the case of seismic incidents, hospitals are central in mitigating population inconvenience; although, their seismic protection is usually poor. Indeed, hospitals are complex systems that contain vast amounts of seismically unstable equipment, much more fragile than the buildings they contain. A major point is the retrofitting of existing hospitals. There are various structural styles, including masonry and Reinforced Concrete (R.C) structures, constructed according to the requirements of the construction age.

In researching the structural protection of large stocks, seismic risk evaluation and damage estimation are critical first steps. Risk is scientifically characterized as the mixture of earthquake risks, seismic susceptibility and exposure, which is the relationship between magnitude of failure and frequency, in other words. Incomplete knowledge on architectural and technical features is a daunting aspect of risk reduction for current systems. In such cases, accounting for risks is certainly necessary and requires, in addition to information on the impact of the earthquake on new structures, information relating to the structure of the building, geometry, structural and non-structural scale and details.[5]

A frequently accepted and successful solution to risk mitigation is the Increasing capacity by making upgrades to structures that may not be strong enough to handle the demands from increased seismic activity. These days, several retrofit techniques are possible, traditional and/or advanced materials are used, and practitioners have many choices. In support to such parameters, i.e., technological and/or financial, each one has separate performances from which each alternative can be evaluated. It may not be easy to choose the most effective retrofit strategy for a specific system since, in many applications, there is no option that explicitly appears, among others, as the right one according to all the parameters considered. Both hospital lifelines should also be closely considered, as both the inside and outside lifeline networks assess their reliability. Therefore, from a larger viewpoint, the seismic vulnerability of a lifeline device at a medical center should be measured.

2.1 SEISMIC ASSESSMENT

These advances in seismic architecture have pointed to the understanding that, contrary to the seismic specifications of current regulations, certain reinforced concrete buildings built before the mid-1970s could be inadequate. As a result, the inspection and retrofitting of buildings in order to enhance their seismic efficiency has been significantly stressed in recent years.

The need to evaluate and, if possible, retrofit old reinforced concrete building systems has been highlighted by the destruction caused by major earthquakes. The most current example was the Hyogo-ken Nanbu quake on 17 January 1995 in Kobe, Japan, which seriously destroyed several structures. For buildings constructed before the new Japanese seismic code came into operation in 1981, seismic activity to reinforced concrete buildings was even more severe. Just minor damage was sustained in most buildings constructed after 1981.

In general, the structural weaknesses of many existing reinforced concrete systems intended for early codes in New Zealand and other countries are not just due to insufficient strength. Longitudinal strengthening of many existing structures, for example, results in a lateral load strength that equals or exceeds that needed for ductile structures by current requirements. The weak structural response during extreme earthquakes is generally attributed to the lack of an approach to capability architecture to ensure the formation of an effective deformation system and/or poor reinforcement detailing, which means that the structure's usable ductility could be insufficient to survive the earthquake without failure.

Seismic inspection of buildings and retrofitting where appropriate to enhance seismic efficiency has increased activity in many countries. In general, the retrofit decision was taken by contrasting the as-built structure specifics with the specifications of existing seismic codes. In these retrofit programs, the focus has been on getting buildings up to near current code standards while offering extra strength and/or ductility. The reality, however, of the testing and study of existing structures and of the reported earthquake destruction, is that not all structures built before the current generation of codes are inadequately susceptible to extreme earthquakes, even though, according to current norms, reinforcement details are substandard in some regions. However, as existing seismic codes propose seismic modeling in terms of seismic design forces and the related need for ductility, most designers would, at least for the time being, wish to use a force-based method for seismic evaluation.[6]

In their attention on what is essential: power or resilience, researchers and engineers remain divided. The response is definitely a mixture of the two, which relies primarily on the essence of the particular structure. [7]

2.2 SEISMIC PERFORMANCE EVALUATION

The pushover approach, which is governed by mass and the first mode form loudness, was used in a deflection manner for each stage level under consideration. In seismic design of multi-story structures, provision must be made considering "P-delta" impacts, which result in "P-delta" impact magnitudes (or "P-delta" impact strengths). These would be the additional toppling moment that the building receives because of the horizontal displacements, Δ, that are generated by lateral earthquake inertia forces, "P" sustained by the building. Those other impacts have already been taken into consideration, which results in a larger displacement, as well as extending the duration that the component and building are functional for Effective stress ratio vs quake mass construction and companies move drift are plotted on a vertically (X-axis) and horizontally (Y-axis) axis, respectively. When Figure 2-1 is drawn, you can see the capacity curves for the designs used as templates. as previously said, transverse reinforcement with varied strength of concrete and spacing is referenced throughout the paragraph on material characteristics. Concrete intensity (MPa) and reinforcement ratio spacing (mm) are referred to by the figure symbol.[8]

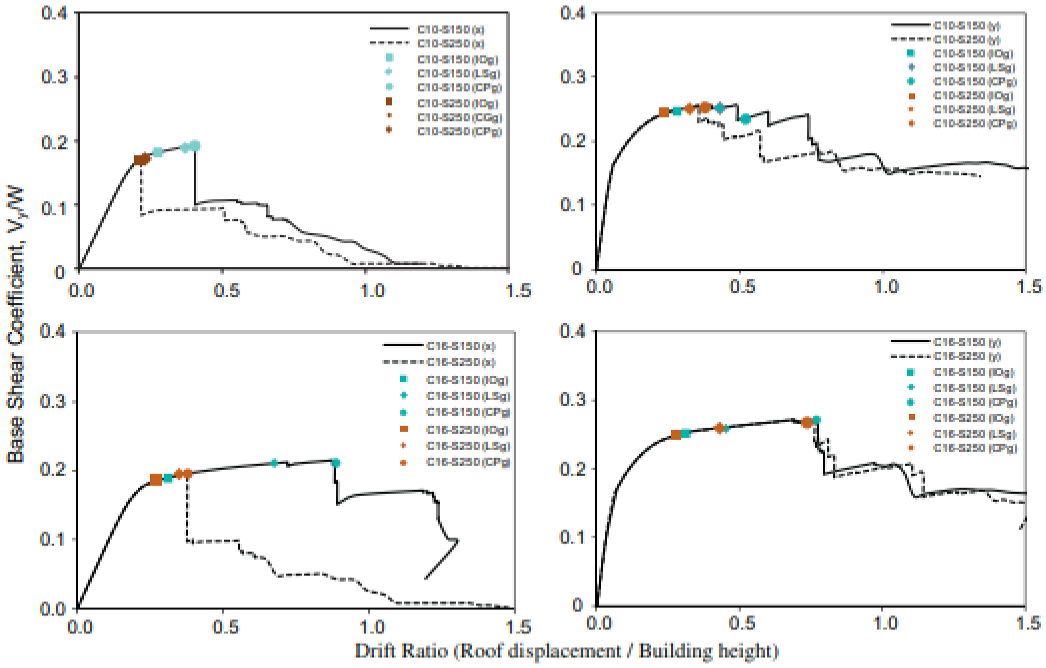


Figure 2.1: Capacities for various concrete strengths and reinforcing spacings for the Type-2B (5-story) structure: [8]

Graphic depiction 2-1 Type-2 demonstrates the building's maximum capacity. Structural elements in the region of longitudinal shear only exists in type-2 (y). One and a half percent of the total area of the bottom level is taken up by the base shear region. The reinforcement ratio separation has an impact on the transverse deformation capacity, as can be observed in Fig. Although the columns have a very restricted reinforcement ratio spacing, the shear weakness is consistent with a considerable limitation on reinforcement ratio deformation capacity. Fragile behavior occurs when longitudinal reinforcement does not ensure flexible bending response and sufficient typical failure resistance. This roof motion has a drift of around 0.25 percent.[8]

For the two examples with concrete compressive strength of 13 and 16 MPa, the influence of concrete compressive strength is confined to weak concrete (10 MPa) when using 150 mm space, whereas this rate is about % for the other two situations using 150 mm space. Column layout might be an explanation for the lower longitudinal displacement capacity, too. The strong longitudinal axis of the strong number of columns results in a strong transverse beam with a weak column action. While the deflection strength, which has a high number of structural members, is regardless of reinforcement ratio space for the radial direction, it does not have a uniform deflection power for the longitudinal direction. The access larger movement capacity ranges between 0.50% and 0.85% based on the concrete breadth. In comparison, the impact of strength of concrete on lateral load capacity is minimal. Structural Elements regulates the behavior. When the building has beyond its capability for lateral loading, it is severely lacking in the capacity to bear vertical weight.[8]

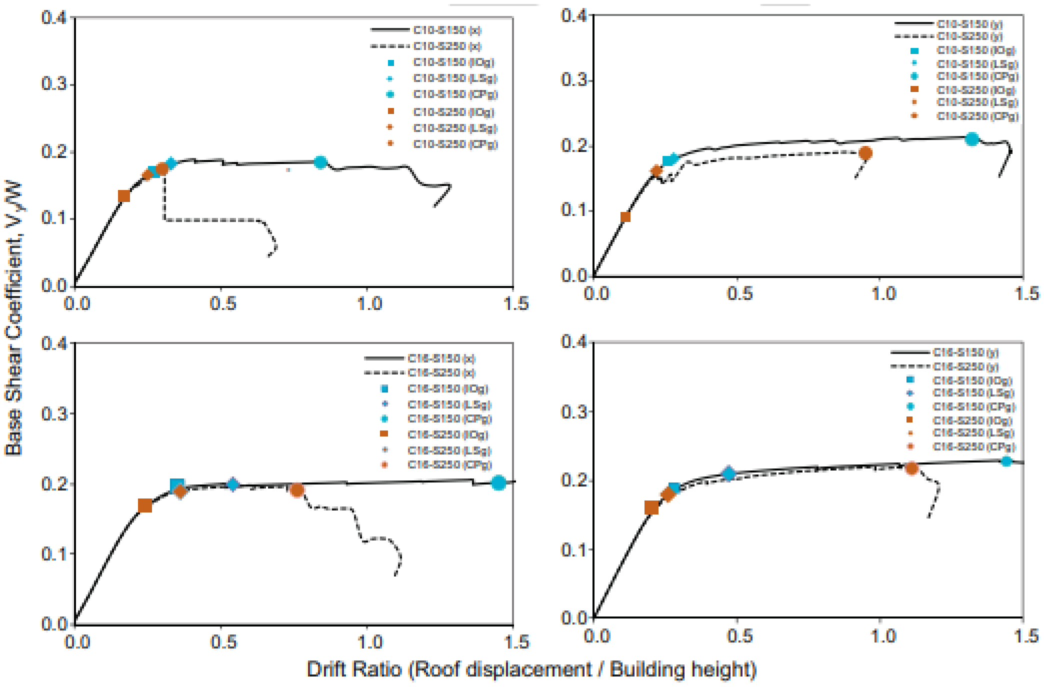


Figure 2.2: Capacities for various concrete strengths and reinforcing spacings for the Type-2A (5-story) structure: [8]

This image illustrates how transversal reinforcing spacs has an impact on deflection capacity. When the columns are stressed beyond their deformation potential, the shear force they produce might cause them to collapse. Because shear failure has not been sufficiently deterred by transverse reinforcement, as well as flexible flexural response, brittle behavior occurs. Lower deflection results in substantial horizontal brittle failure in relation to 16 MPa cement strengths for the 150 mm space scenario, with a smaller influence of strength of concrete on the overall structure of capital.[8]

2.3 SEISMIC RELIABILITY ANALYSIS

The previous discussion shows that it is possible to do quake risk assessments on a number of different levels of sophistication. In Figure 2-3, uncertainties that may be included in the study as seen. Both uncertainties are virtually difficult to recognize in one study. In addition, the approximate likelihood.[9]

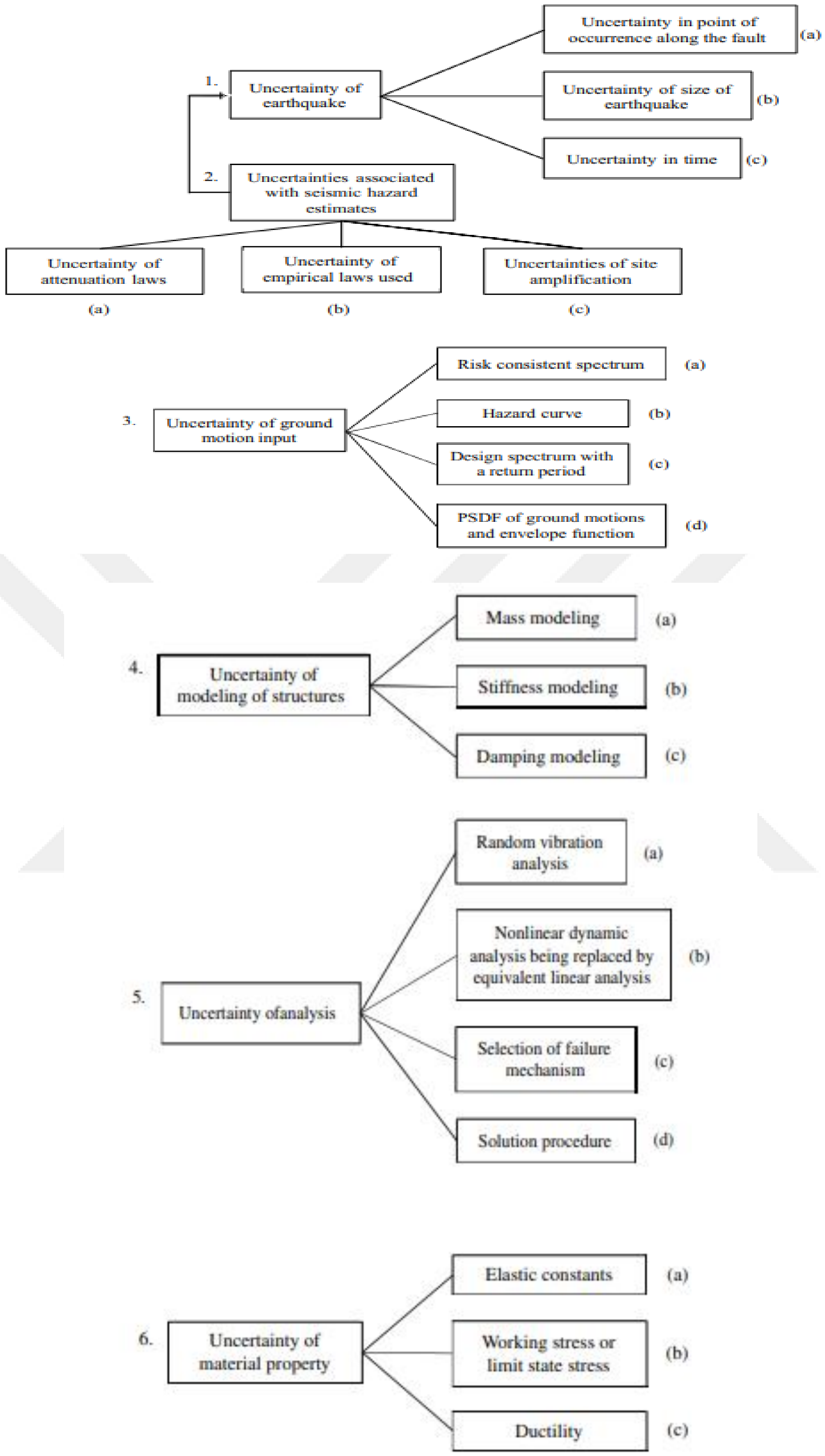


Figure 2.3: There are many classes of uncertainty in the assessment of seismic risk. [9]

Due to the approximations involved with each procedure, any loss obtained by any reliability measurement technique will not be correct. Thus, the seismic strength measured is at best a reasonable approximation of the real durability of systems against the event of collapse. In spite of this, various researchers have suggested several simplistic seismic reliability assessments. Any (but not all) of the uncertainty are useful in generating an estimate of the quake reliability of buildings when taken into account at the same time. Several of them will be discussed in this section. These are some examples: (I) Only ground data ambiguity is taken into account in the reliability study of buildings. (ii) Structures' reliability is assessed based on the earthquake hazard characteristics of the location. (iii) determination of a specific seismic activity time history used in determining the dependability of buildings. (iv) a random land movement building's initial passing reliability study (v) The damage probability matrices were used to conduct reliability test. (vi) probability risk assessment of a compressed design.[9]

2.4 SEISMIC RISK ASSESSMENT

The major parts of the building exposure database are used to perform risk analysis. The research includes the design of a probabilistic seismic risk assessment. The findings are grouped by hospital institutions, for the whole portfolio and for the regional regions. The findings of the probability seismic hazard assessment usually presented in divided by the average annual loss (AAL).[10]

Small earthquakes occur more commonly than major earthquakes, as has already been discussed. They can produce peak ground accelerations of comparable magnitudes, although considerably smaller, these earthquakes are substantially greater. Accordingly, the measurement of seismic activity at the assessment of a probability of earthquake motion developing is included on the site as well of a given intensity as a result of the cumulative effects of regular moderate earthquakes occurring near the site and of occasional larger earthquakes occurring at larger distances. [11]

2.4.1 Hazard Assessment

One of the most critical aspects of risk management is seismic hazards. Seismic strength and its related dispersion are needed for various return periods. The research should include the essential genetic seismic origins within the research region and its description of each source's magnitude of recurrence.[10]

2.5 STRUCTURAL PERFORMANCE ASSESSMENT

2.5.1 Analysis Methods

The next stage is analysis, which requires an acceptable representation of the proposed and its seismic loads. Essentially, the choice of linear elastic or nonlinear characterization of the object's force activity as well as the existence or absence of mathematical nonlinearities, dictates the appropriate technique of research. Dynamically developed load models as well as dynamic character of seismic loading enable dynamic analysis methodologies instead of static methods. When used broadly, there are four different analytical service modes, each of which has different solution techniques that must meet certain requirements (Table 2.1 according (ASCE 7-16).[12]

Table 2.1: Quake Design and Assessment Analysis Methodologies

Category	Structural Model	Seismic Actions	Analysis Method
Linear-static	Linear	equivalent load pattern	linear-static, force-control
Linear-dynamic (A)	Linear	ground motion spectrum	modal response spectrum
Linear-dynamic (B)	Linear	ground motion record	linear time-history
Nonlinear static	Nonlinear	equivalent load pattern	nonlinear-static, displacement-control
Nonlinear dynamic	Nonlinear	ground motion record	nonlinear time-history

2.5.1.1 Linear Methods

While the latest research standards may appear to have obsolete basic linear elastic methods, the current practice standard for seismic design is overwhelmingly majority-bound. Despite numerous efforts in the nonlinear static and dynamic design process, the vast majority of hands-on earthquake engineers continue to perform their work in an elastic manner using the following: (a) equivalent linear static analysis, using an elastic structural model for a fixed lateral load pattern and (b) The modal response spectrum is analyzed, and the clear understanding of MDOF is estimated by combining modality response (usually through SRSS).[12]

During the latter instance, designed spectral or actually ground motion data may well be employed in determining the quake load. elastic and inelastic earthquake forces separation using a R or behavior (q) factor which is meant to reflect conductivity and elevated frequency of a producing structure results in equal consideration of inelastic response.[12]

2.5.1.2 Nonlinear Static Analysis

The framework is exposed to such behavior step-by-step in nonlinear static analysis before its critical and post-critical situations. Because structural reactions aren't affected by false time, no physical attributes are mirrored by the false time. If nonlinear mechanical behavior is expected, nonlinear formulas should be used. A computational context can help solving nonlinear problems be understood and resolved. While nonlinear formulae are often linear using a step-wise approach and solved iteratively,

they are iteratively linear to begin with. A number of iterative procedures, such as the Picard repetition, the Newton-Raphson repetition, and the Risk approaches, are commonly used.

This type of study is typically utilized to carry out quasi-static studies on brick and stone structures and also known as a pushover. Pushover analysis is a simple and methodical procedure that establishes the earthquakes of a structure of masonry when the structure is subjected to increased lateral loads that are assumed to be constant.

Nonlinear static analysis can be an important modeling method to explore facets of the analysis model and the nonlinear solution that are difficult to do by nonlinear dynamic analysis, particularly where the nonlinear static technique is not ideal for a full performance assessment. Nonlinear static analysis, for example, can be useful for (1) testing and debugging the nonlinear analysis model, (2) increasing understanding of the yielding processes and demands for deformation, and (3) exploring alternate configuration parameters and how responses can be influenced by changes in the component properties.[13]

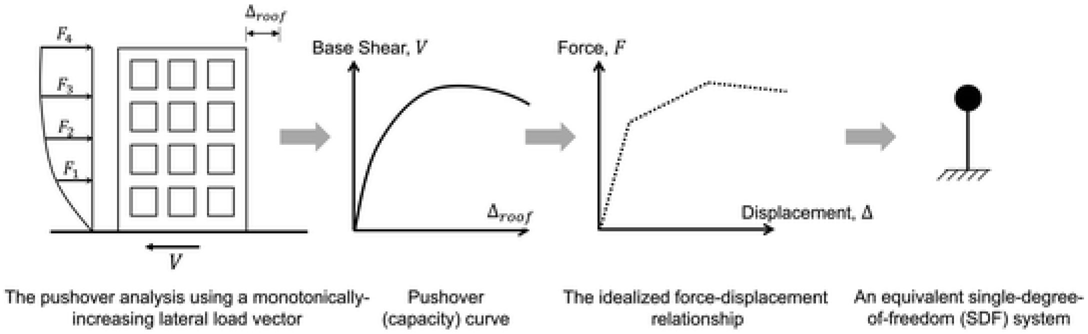


Figure 2.4: The basic conversion of a detailed structural model in to an equivalent SDF system. [14]

2.5.1.3 Nonlinear Dynamic Analysis

The nonlinear dynamic method, unlike the nonlinear static procedure, allows a more precise measurement of the structural reaction to heavy ground shaking when properly applied. Throughout the nonlinear dynamic approach, the inelastic component action is simulated while cyclical seismic ground motions occur. It is only necessary to apply viscous damping to the damping in the linear range and other non-modeled energy dissipation. Input earthquake ground movements are measured for the dynamic response, resulting in response background data on the related demand parameters. Due to the inherent heterogeneity of earthquake ground motions, it is important to quantify statistically robust demand parameter values for a given ground motion strength or earthquake scenario with complex simulations for multiple ground motions.[13]

Since nonlinear dynamic analysis requires less assumptions than the static nonlinear procedure, less constraints than the static nonlinear procedure is imposed. The precision of the findings, however, depends on the specifics of the model of study and how faithfully it captures the important behavioral consequences. Acceptance requirements usually limit the overall structural component deformation to values where the nonlinear dynamic analysis models are stable and where degradation is regulated.[13]

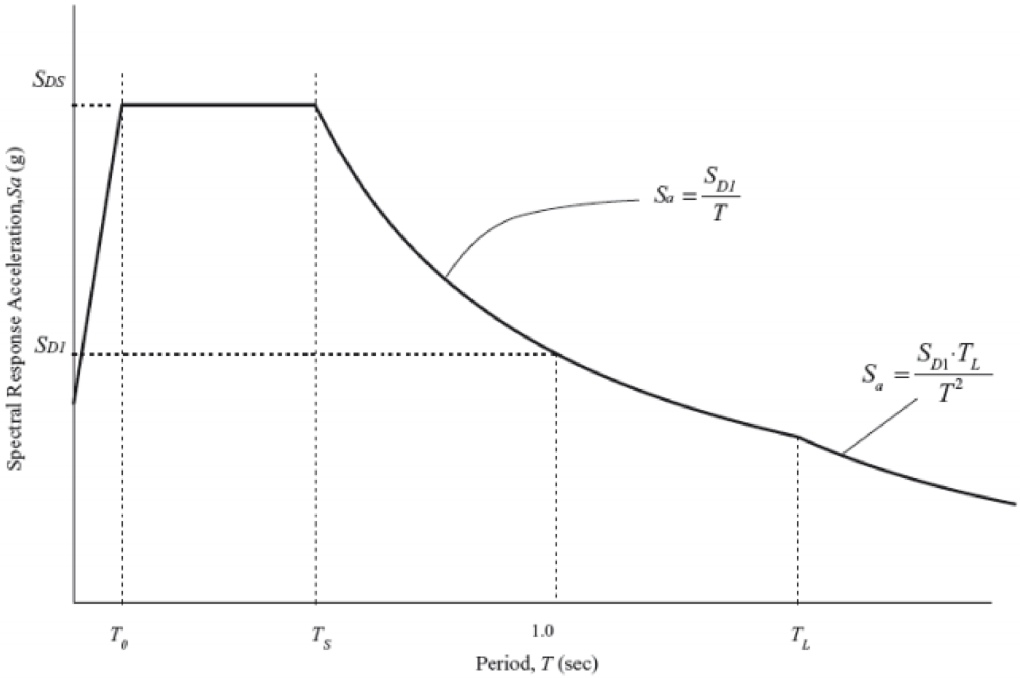


Figure 2.5: Simplified Design Response Spectrum According to ASCE 7-10

2.6 SEISMIC PERFORMANCE CLASSIFICATION

In seismic retrofitting decision making methods, as has previously been underlined, a thorough and precise building performance categorization is important. The categorization of performance is recommended to integrate an acceptable performance measure based on the probabilistic risk assessments carried out in the preceding section.

The seismic risk group is determined with regard to the median (IR,50) and 5th percentile (IR,05) values of the seismic risk Index. In specifically, five categories (A-E) with an increasing susceptibility are distinguished in Table 2-2. The seismic performance classification described in Table 2.2 may be a valuable guide for stakeholders to focus on seismic retrofit strategies, enabling them to measure and communicate threats, encouraging them to prioritize measures for the most fragile structures.[15]

Table 2.2: Seismic Performance Classification Definition of categories[15]

				IR,50		
		------------------	------------------	------------------	------------------	------------------
		<0.2	**0.2-0.3**	**0.3-0.5**	**0.5-0.7**	**>0.7**
	<0.3	E	E	-	-	-
	0.3-0.6	E	D	D	-	-
IR,50	0.6-0.8	E	D	C	C	-
	0.8-1.0	D	C	C	B	B
	>1.0	D	C	B	B	A

2.7 RETROFITTING (REHABILITATION STRATEGIES)

Two alternate techniques can be used to retrofit current structures, taking into account all facets of seismic design: demand and power. The first is focused on traditional techniques of retrofitting which involves introducing to the structure additional structural components (shear wall and braces), extending current members (column and beam jacketing, thickening existing walls), reinforcing with steel plate or layer of carbon fiber. [16]

A common concern in both the field of science and actual practice is the seismic retrofit of existing buildings. Indeed, in the case of seismic incidents, safety tests carried out under current codes can be unsatisfactory in old structures, usually attributable to many inherent deficiencies connected to obsolete provisions.[17]

2.7.1 Retrofit Strategies:

It is helpful to consider how the architectural specifications existed at the time the building was built compare with new provisions in the early stages of determining the need for retrofit of an existing building [18]. In the last few decades, the interest of designers in very seismic regions such as Italy has primarily centered on seismic impacts, with the goal of ensuring sufficient structural efficiency in order to safeguard human life. In reality, with minimal seismic provision, most modern buildings have been planned and constructed in reference to old building codes. In order to increase the structural ability of buildings in the face of seismic events, stabilization measures are also required.

Retrofit actions are typically based on four primary strategies:

a) An improvement in structural strength and rigidity.

b) An increase in the potential for global energy dissipation.

c) An increase in both structural strength and capacity for deformation.

d) A decline in seismic demand.[19]

This article examines four retrofitting solutions for the MCDM, three of which give earthquake behavior, while the last is focused at reducing seismic response. To give options A1, A2, A3, and A4, the layout strategy will be displayed.

Especially, A1 limits the movement of columns and joints by incorporating GFRP and adds to the ability to change shape a construction; A2 is a great increase in structural strength (and stiffness) by stacking multiple braces; and A3 is a partial but simultaneous boost to structural strength and durability provided by concrete encapsulation around then choose columns. It was finally known to as the alternate A4's base insulating. [20]

a) option A1 is to confine in glass fiber reinforced plastic.

   This approach is aimed at enhancing the building's global deformation capacity. overall, to get this purpose, you can use composite in 2 ways. the initial is to create a correct strength structure by shifting the possible plastic hinges; the next is to increase the malleability of the plastic hinges without altering the position of the hinges. Here, the latter method is followed. To add a containment action to the columns, Polymers reinforced with fibers that are externally connected were utilized to increase the ultimate pressure of the concrete and hence the ultimate curvature of the plastic hinges. [20]

b) option A2: Some frames are braced with steel.

   This treatment tries to increase the building's global strength while compromising its global ductile, with a particular emphasis on plan stiffness. Concentric braces of diagonal X have been considered (Fig. 2-4).

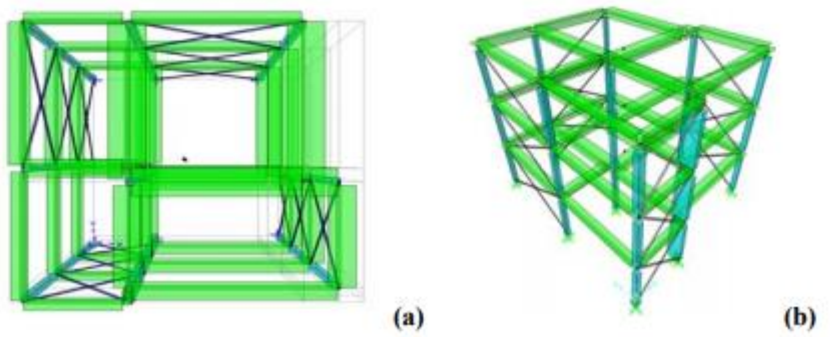


Figure 2.6: Bracing Configuration: Plan (a) and 3D (b) [20]

Concerning the design of the intervention, several aspects were taken into consideration: It is critical to secure two simultaneous frames, in both the X and Y directions, to provide sufficient plant stiffness and, therefore, attempt to decouple the vibrational modes. to ensure that the main seismic motion happens during a large earthquake, it is easier to use a 'echelon formation' and arrange bays across the plant so that the alternate nodal action occurs.

c) option A3: for concrete jacketing of columns.

This intervention aids in the concentration of strength in the design, hence reducing the nonlinear response to disruptive torsional effects; it entails the jacketing of selected columns with concrete, resulting in an increase in the structure's global tensile strength.

d) Option A4 for base isolation.

To have a damping that is efficient, using a high damping rubber bearing (HDRB) system in conjunction with a constructed isolation system is commonly seen. In order to reduce the earthquake burden on the superstructure, the project will prolong the vibration cycle of the building.[20]

2.7.2 Rehabilitation Options

a) LOCAL INTERVENTION METHODS

Defective components that cannot exceed their limit state in response to the building's movement are able to increase their deformation ability by using a local adjustment of components that are separated structurally and non-structurally.

Structural vulnerabilities and a group of participants with structural vulnerabilities must be addressed using locally applied intervention approaches in order to establish an appropriate seismically constructed system.

a) Injection of cracks.
b) Shotcrete (Gunite).
c) Steel plate
d) Steel jacketing

b) GLOBAL INTERVENTION TECHNIQUES

Global intervention strategies are considered in the case of systems with high flexibility or where there is no uninterrupted transverse load path accessible. The most famous global retrofit schemes are presented below.

a) RC jacketing
b) Addition of walls
c) Steel bracing
d) Base isolation[21]

2.8 EXISTING PUBLIC BUILDINGS

2.8.1 Existing Buildings

The seismic assessment of existing buildings, as considered by the prefabricated industrial structures, is highly influenced by the degree of expertise obtained by the engineer during the initial stages of the evaluation process.

Identification of the structural structure, data collection on the properties of the structural components in terms of strength and stiffness, and the properties of the materials adopted should be the important aspects to be investigated. The relation information between structural elements plays an important role for precast structures, as they influence the structure's global behavior during a seismic event.[22]

2.8.2 Hospital Buildings

In order to protect the lives of patients and health staff, as well as to provide emergency care and medical services to the growing number of patients who are transported to health facilities in the first hours after major seismic incidents, hospitals and healthcare centers must be completely operational following earthquakes.

Earthquakes are a threat to buildings and individuals. the total number of deaths and injuries that result from earthquakes is directly linked to the vulnerability of buildings and the willingness to provide first aid. The analysis of existing building earthquake hazard and strategic structures in particular, such as health centers and hospital buildings, is therefore of utmost importance.[23]

2.9 INTERNATIONAL BUILDING CODE

Selecting a building code text is the first step taken in this review. The Standardized Building Code of the International Conference of Building Officials (ICBO), the Southern Building Code of the Southern Building Code Congress International, Inc. (SBCCI), and the National Building Code of Building Officials and Code Managers International, Inc. were the three building codes in common use previously in the United States (BOCA).

All three model codes have recently been reformatted into a single national model code, the International Building Code, established by the International Code Council, an organization consisting of members from the three ICBO, SBCCI and BOCA model code organizations (IBC, 2000). The International Building Code (IBC) was then chosen as the building code document for the implementation of the prototype framework for automated code checking. The legislative specifications in the building code are used as the primary source of information to create an automated code testing system to construct its knowledge base.[24]

2.9.1 International Existing Building Code

The International Current Building Code is one of the codes in the International Code Council family of codes (IEBC). The existing building code addresses existing buildings and controls construction and renovations (as opposed to new buildings before their occupancy). It may also resolve cases of occupancy change and is also related to buildings of cultural or historical significance, where it is not easy to apply building and fire codes. [25]

2.9.2 Seismic Design by 2003 International Building Code

The IBC 2003 is structured in such a way that the code contains all the provisions for seismic architecture, within the code itself, for structures that can be built in accordance with Section 1617.5 of the Simplified Analysis Procedure. In the case of more complicated structures that do not qualify for the Simplified Analysis Procedure, they can be designed in accordance with Sections 1613 to 1623 of the IBC 2003.

In addition to items relevant to the Simplified Analysis Process, the decision was made to retain only the text, tables, and figures necessary for the determination of the Seismic Design Category in the seismic sections of the code.[26]

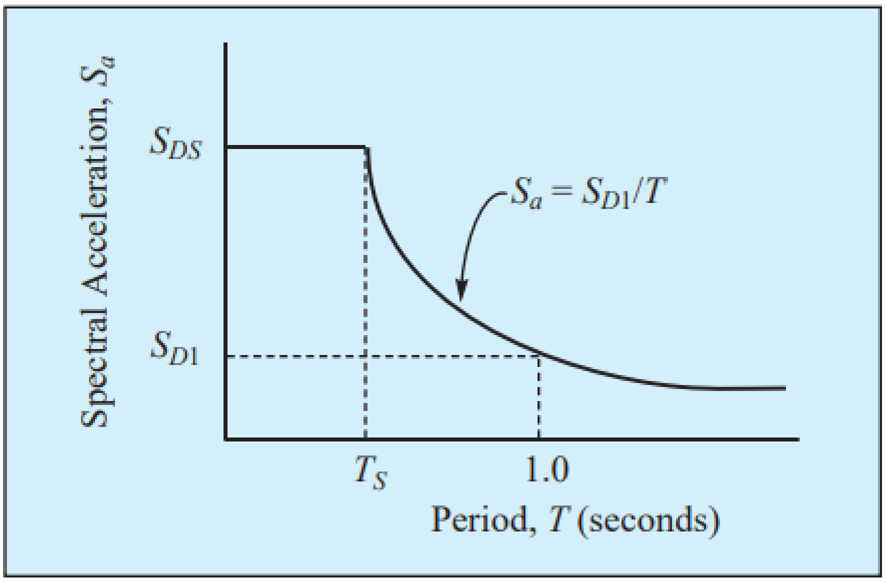


Figure 2.7: Seismic Design Spectrum of the 2003 IBC

2.10 SEISMIC DESIGN CONCEPTS

In the seismic design of structures, it was customary to use linear elastic structural analysis before the mid-1970s to determine the bending moments, axial forces and shear forces due to the gravity loading and seismic forces of the design and to design the members to be at least strong enough to withstand these acts.[27]

Buildings' seismic design has seen radical changes over the last few decades from the usage of a fraction of a structure's earthquake weight as such designed axial forces to a results methodology. Conventionally, specification codes are based on requirements and elastic analysis for force-based design (FBD). The inelastic power consumption that occurs in reactions is reflected by a reduction factor that takes into account the structure's ductility and strength. Many of the new construction methods and the highly advanced appear to overlap when it comes to quake design requirements. most modern codes utilize design factors that include designed base shear, ductility capacity, ductility demand, and drift. In the seismic design of structures, a clear vision of seismic performance is critical. Decision on "how do you want the system to work in an earthquake?" and "what are the best alternative routes?" and "how much danger can you accept?"[28]. The two most significant components of concern to a design team are the design of earthquake loads and providing enough ductility. Many engineering design problems may be tested for dead loads, live loads, and wind loads to varying degrees of precision. While it is clear that these abilities are earthquake-based, the situation with regard to them is rather different. During a seismic, structures are stressed because the ground beneath it moves. Deflections, velocity, and accelerations are defined by uneven motion with a basic quantity, which may be motion in a certain direction, speed, or length.

The inertial forces associated with the building's density, stiffness, and energy-absorbing qualities are referred to as seismic load (e.g., damping and ductility). This building codes static horizontal load technique is known as simply "static axial load," which relies on mass, gross size, building type, and seismic activity. Static design loads are being used to determine the strength of the structure needed to resist dynamic stresses induced by seismic. Uncertainties arise from a variety of variables when the proper earthquake design loads are calculated by the conventional static approach; the most important of these are as follows:

(a) A quantitative assessment of whether a planned structure may be subjected to crucial seismic motions (i.e., magnitude, frequency, and length characteristics) has yet to be completed.

(b) Elastic research doesn't take into consideration the variations in structural materials properties when the project is being developed in a seismic. This poses difficulties in evaluating the dynamic response-affecting structural parameter values (e.g., stiffness and damping) as well as the dynamic soil or support medium characteristics

(c) The relationship between soil and structure and geological conditions has a profound impact on structural results. There is currently no clear-cut methodology for implementing these effects correctly.

Despite these uncertainties, beyond the elastic-code-stipulated tension, the structure should work satisfactorily. In the inelastic range, ductility, the foremost important property, thus becomes a requirement for a structure's earthquake-resistant nature. In general, it is agreed that by following the codes, adequate ductility will be achieved. For standard systems, however, design codes are prepared. Structures needing high ductility, including a light, flexible model linked to a huge building, may require careful thought.

2.10.1 Behavior of Existing Buildings to Seismic Loading

According to internal forces, earth equilibrium is often disrupted and as a result of such disruption, shakes or jerks occur in the surface of the earth, which is known as an earthquake. Earthquake creates low-high waves that vibrate the structure base in different ways and directions, so the structure gains lateral force.

A structure's quake-induced vibrations are referred to as a vibrating hazard. A structure is not affected by the motion of the ground by collision or by external pressure, but instead, it is affected by the structure's own vibration.

A rise in mass will distort the earthquake's appearance in two undesirable ways. Increasing weight and inducing buckling of columns and walls when the mass bends down or is pushed by the lateral forces from the plumb are both associated with the first consequence. The magnitude of the earthquake's impact was lessened so because earthquake's origin was located thousands of miles away.[29]

2.10.2 Seismic Ground Motion

Earthquake ground movements documented in accelerator stations are of crucial importance from an engineering point of view for the production of dynamic structure time-history analysis. The amplitude, the frequency content, and the length of the motion are the most important features to characterize an earthquake ground motion.[30]

These stochastic processes that are not stationary are the ground movement of the seismic and the reaction that follows. It can be said that seismic ground motion has an origin and an end, and although it is often assumed to be static for much of its length, it cannot be fully motionless for practical reasons. In addition, the structural response is non-stationary apart from source acceleration that is static.

Three stages of non-stationary motion display

a) The motion's average strength is maintained.

b) From weak to powerful, the motion accelerates swiftly.

c) The motion is caused by the decrease

In addition, the design of actual ground motions with mid rankings of every ground motion is typically tackled based on the possible level of harm of ground motions, so that seismic analysis of structures situated in low and medium seismic zones is suitable. The seismic environment of the structures will determine if the structure is placed in an area of low or moderate risk.[31]

Earthquake-induced motions excite a radically different human reaction, even though they are more aggressive than those induced by wind: Firstly, because earthquakes happen much less often, secondly, since an earthquake duration of movement is on the order of seconds.[32]

2.10.3 Soil Impact and Seismic Soil Structure Interaction

Earthquakes with over and underground structures requires consideration of the soil-structure connection, when the foundation medium is not very solid. The structure interacts with the underlying soil during earthquake excitation, imposing soil deformation. In exchange, Additionally, the deformations in the soil or the building will influence the motion of the supporting or the contact area of a soil and the building in comparison to the available ground motion. This adjustment is extremely limited for very stiff soil and can be ignored. Therefore, for over-ground structures built on firm soil, consideration of base fixity remains a legitimate assumption. Similarly, the impact on long buried structures of soil-structure contact. Inside firm soil, such as pipelines, is insignificant as it takes the same profile during the earthquake motion as that of the soil.

In order to better comprehend the soil-structure system works, it is essential to understand how seismic waves propagate thru the soil medium. The alteration of the foundation motion affects the properties of the input ground motion as it travels thru the soil, causing the dynamic features of the building to rely on the initial ground motion. This explains why comprehending ground motion alterations is dependent on a comprehending of acoustic waves thru the soil medium.

Understanding of the soil medium's seismic behavior is also required in order to compute soil resistance functions and to determine limits for a semi-infinite soil medium. Hence, because of this, the following chapter discusses wave propagating via soil but instead, the seismic soil-structure interactions difficulties are described.

In an earthquake, by the passage of a complex wave function that propagates from the causative fault through or along the surfaces of the soil and the bed work strata, strains are formed in the soil. As a result of wave reflection and propagation into and out of the system, stresses grow at the soil-structure boundaries. In the vicinity of structures situated basically on top of soil or rock surfaces, intense ground-shaking consists of a body (shear and compression) superposition and surface waves that have propagated a large distance across an inhomogeneous nonlinear medium and that impact the structure from a wide range of directions.[33]

3 .GENERAL INVESTIGATION AND MODELING

As mentioned in previous chapters; this is an investigation of public hospitals in Palestine, and so it is important to have a general view that helps us in studying hospital's properties and its components from structural point view.

3.1 DATA COLLECTING AND PARAMETERS CONSTRUCTION

Seismic characteristics that have previously been found to have an impact on structure performance after seismic activity were acquired using inventories forms throughout the field investigation. In addition to information on inventory categories, many pictures have been collected of each building for monitoring and verification of data. Typical construction site, construction age and apparent construction quality are all typical criteria for a structural structure, usually we reach these criteria depending on the concrete and building materials used.

3.1.1 Structural System Type

In terms of structural systems, structures are categorized as: reinforced concrete, masonry, and other reflective steel, old constructions that cannot be used, and unknown buildings. A group is established according to the number of floors each kind has.

3.1.2 Number of Floors

To estimate their power, this parameter is used to represent the estimated construction time and modal properties of buildings. During the inventory, the exact number of floors for each building is obtained. However, in reports and assessments according to construction form and number of floors, buildings are divided into subgroups as follows. The reinforced concrete structures are classified into three parts: RC 1-2 (floors 1 and 2), RC 3-5 (floors 3, 4 and 5) and RC 6+. (6 and higher Floors).

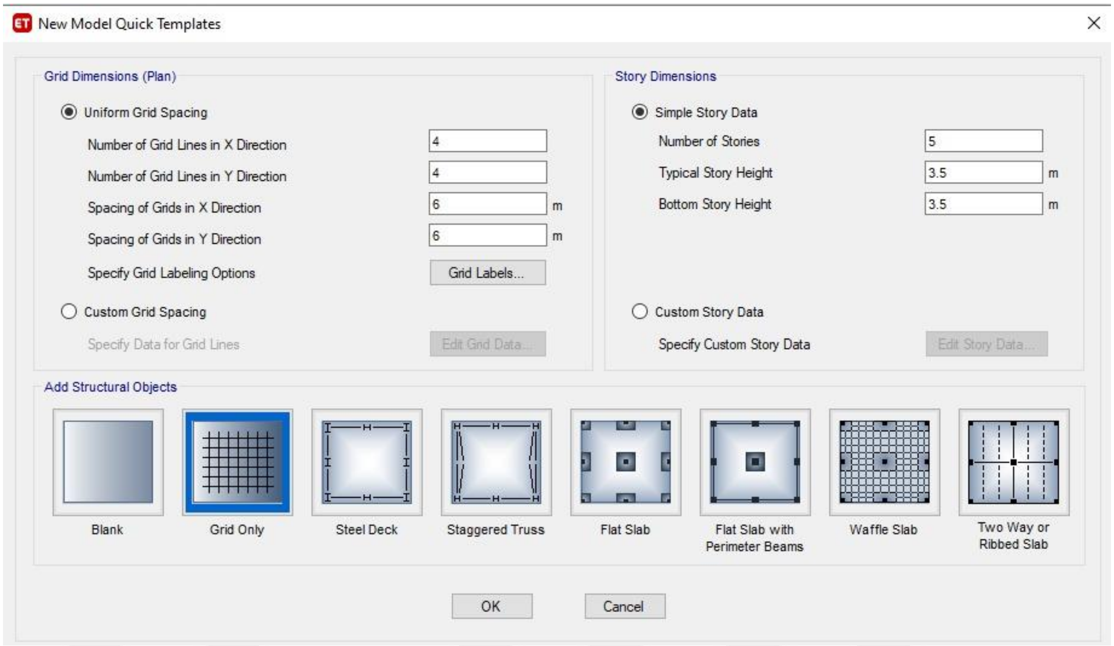


Figure 3.1: This Figure from ETABS to Enter The Information about the building in terms of dimensions and number of floors.

3.1.3 Presence of Soft Story and Short Columns

In a building, soft floors occur where a floor has less rigidity and strength relative to the other stories. This condition typically happens on the ground floor of buildings situated on the side of the entrance street. As an industrial space, the ground floors are used, while the upper stories are used for residential purposes. In such situations, relative to the ground floor, the upper floors have more stiffness and gain from the extra stiffness and strength offered by many partition walls. The influence of these negative characteristics is characterized as a soft floor. Many structures were found to collapse due to a pancaked soft floor in past earthquakes across the globe.

Semi-buried basement band windows, mid-floor stairway beams, and semi-infilled frames allow small columns to be built in reinforced concrete buildings. Small columns typically suffer heavy damage during powerful earthquakes due to their shorter spans.

3.1.4 Building Age

This parameter is one of the main components of practice for assessing the building code and estimating the nature of the materials used in building construction. [34] In many regions and situations in Palestine, the definition would rely on the measurements and structural assumptions and actions of all designs that reflect local experience.

Such as the following:

a) Columns dimensions.

b) Beams dimensions.

c) Floor areas.

d) No. of floors.

e) Structural system.

f) Slabs system.

g) Rooms dimensions.

3.2 DIMENSIONS AND LAYOUT

The number of operating hospitals in Palestine reached 84 in 2019, of which 52 are based in the West Bank, including East Jerusalem, accounting for 61.9% of the total number of operating hospitals in Palestine.

The survey on design drawings for the number of hospital buildings in Palestine showed that most hospitals consist of 5 floors on average, with a height of about 3.5 meters for each floor as shown in the following figure:

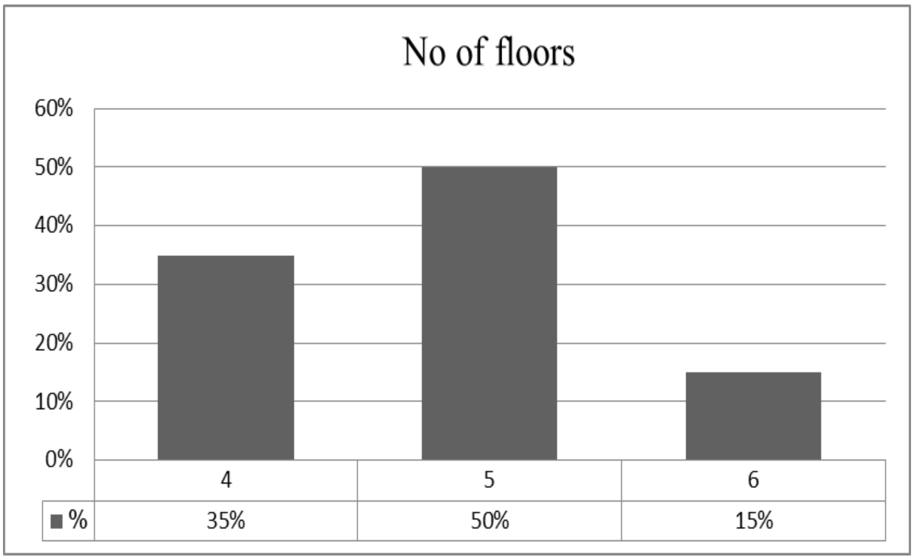


Figure 3.2: Statistics for Stories Properties of Hospitals in Palestine

It is also found that the majority of the main columns are around 30 cm in width and at the same layout, as well as the lengths of columns found in most hospitals to be 60 cm, as shown in the following figure:

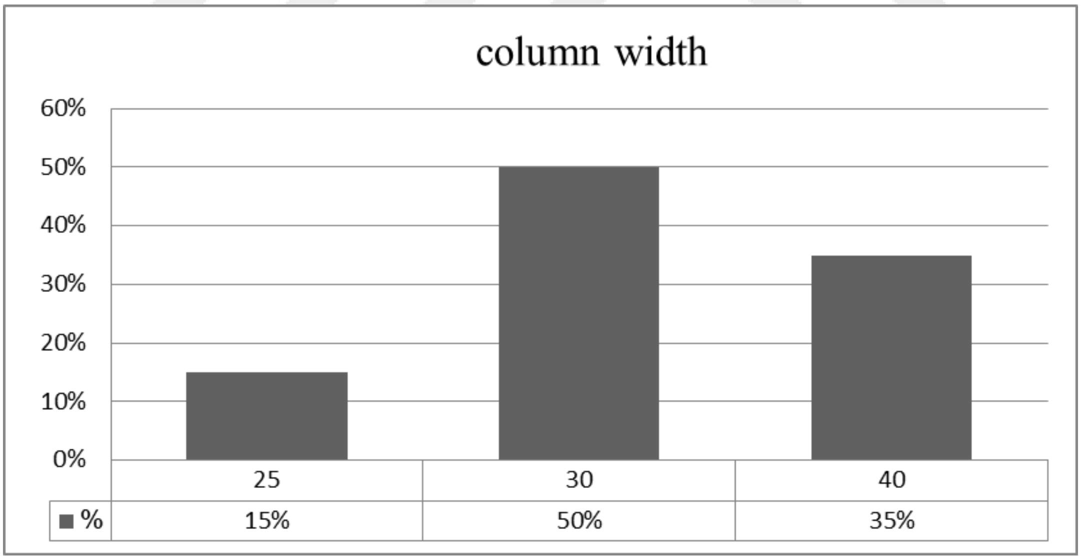


Figure 3.3: Statistics for Columns Widths of Hospitals in Palestine

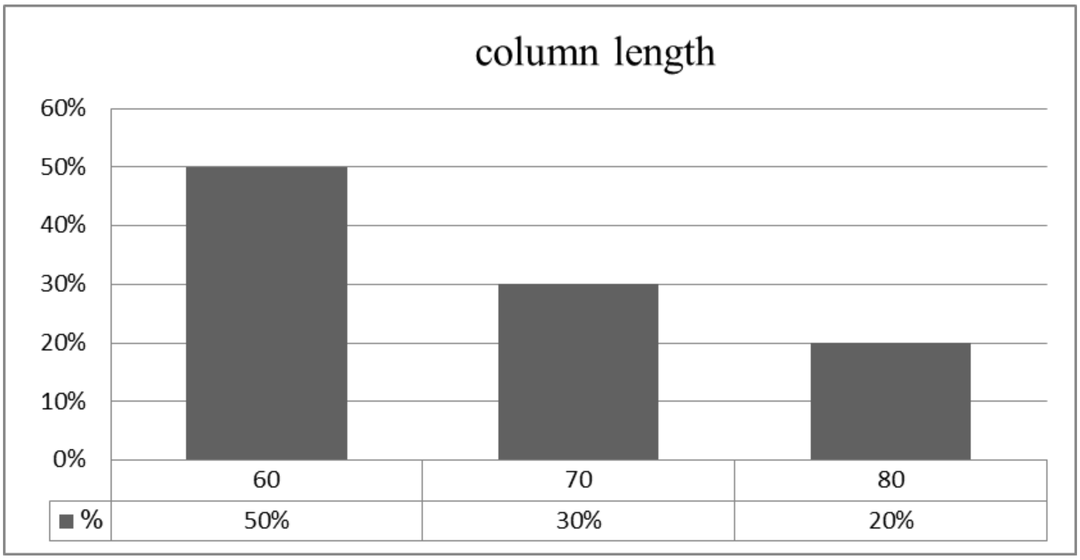


Figure 3.4: Statistics for Columns Lengths of Hospitals in Palestine

Also; regarding the properties of slabs in public hospital systems of the main beam; The dimensions of beam widths are usually 30 cm and the beam depths are 60 cm; this is as follows:

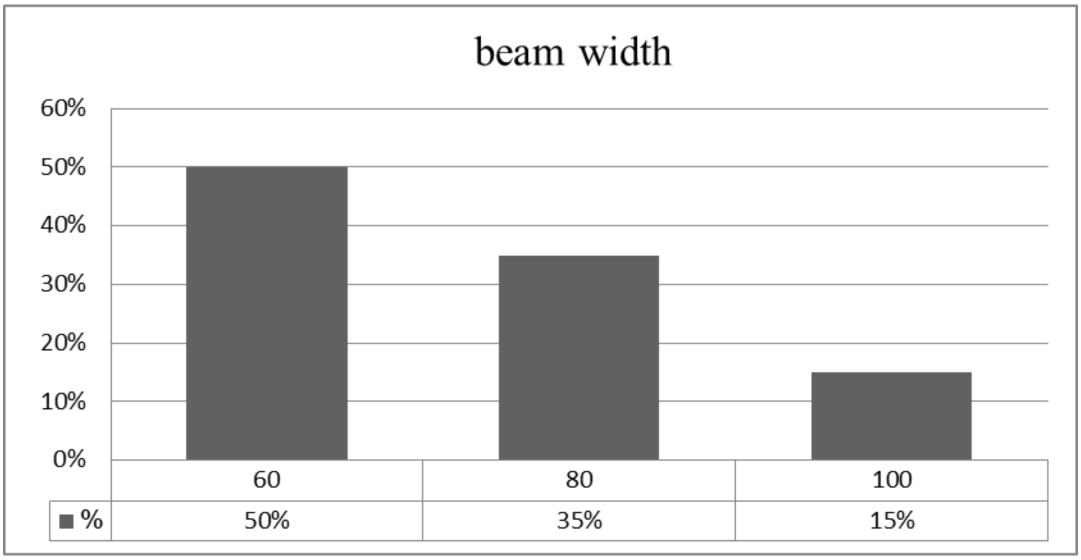


Figure 3.5: Statistics for Beams Cross Section widths of Hospitals in Palestine

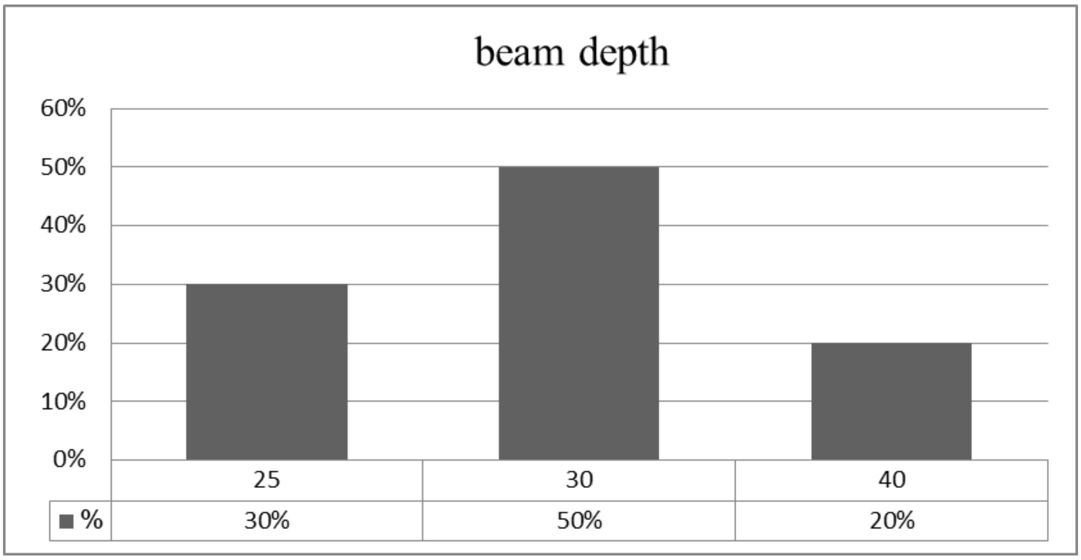


Figure 3.6: Statistics for Beams Cross Section Depth of Hospitals in Palestine

The ratio of steel balance is known as the ratio of steel to the beam area where steel yields occur simultaneously with the crushing of concrete. The equilibrium steel ratio as seen in the following figure is known to explain the stress and stain diagram:

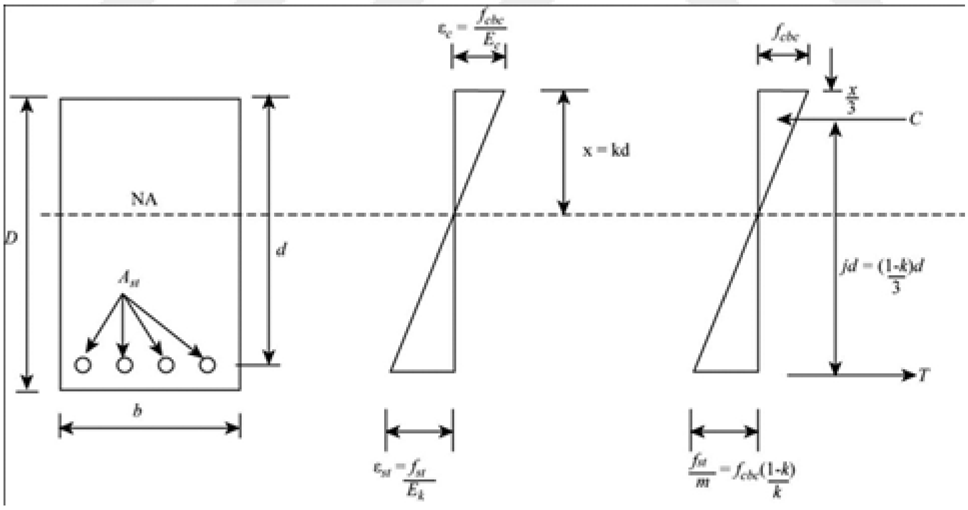


Figure 3.7: Stress and Strain Diagram for the Reinforced Concrete Beam with four bar numbers

The figure shows a stress and strain diagram for the reinforced concrete beam with four bar numbers. Here, b refers to distance, d refers to productive depth and D refers to total depth. The analytical formula used (in percentage) to measure the balanced steel ratio is given as below.

$$\rho_{bal} = \frac{A_{st}}{bd} \times 100 \text{----------------------- (1)}$$

Here, the term *ρbal* is the balanced steel ratio and *Ast* is steel reinforcement area.

And for steel ratios of slab beams built on longitudinal reinforcements with respect to beam cross-section areas; on average, the majority of steel ratios are around (= 0.0055 to 0.0085); as follows:

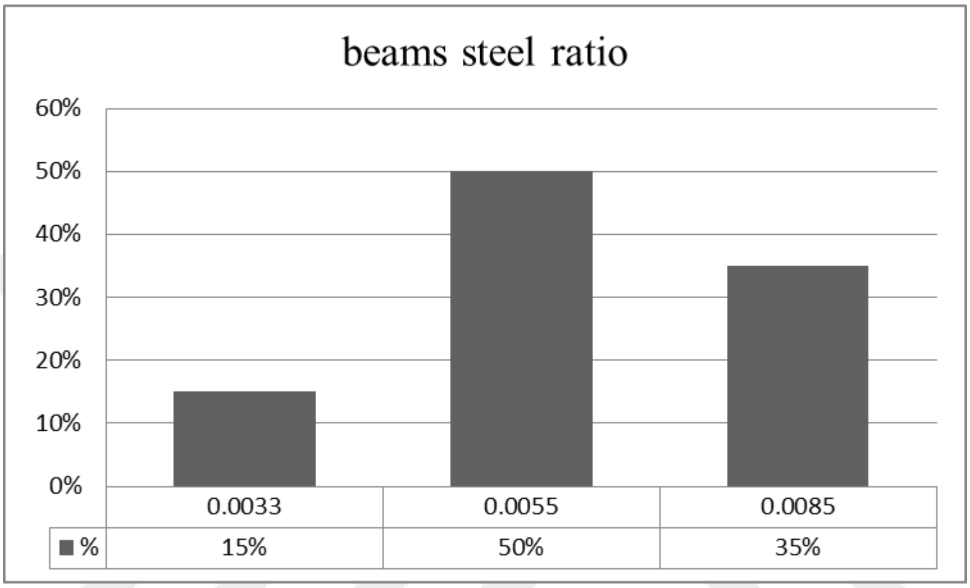


Figure 3.8: Statistics for Beams Steel Ratios of Hospitals in Palestine

Although the steel ratios of columns are measured on the basis of longitudinal reinforcements with respect to cross-sectional areas of columns, the majority of steel ratios are roughly (= 0.013) on average, as seen in the next figure, this value is similar to the minimum steel ratio of 0.01 and far from the actual permitted steel column ratio of 0.07.

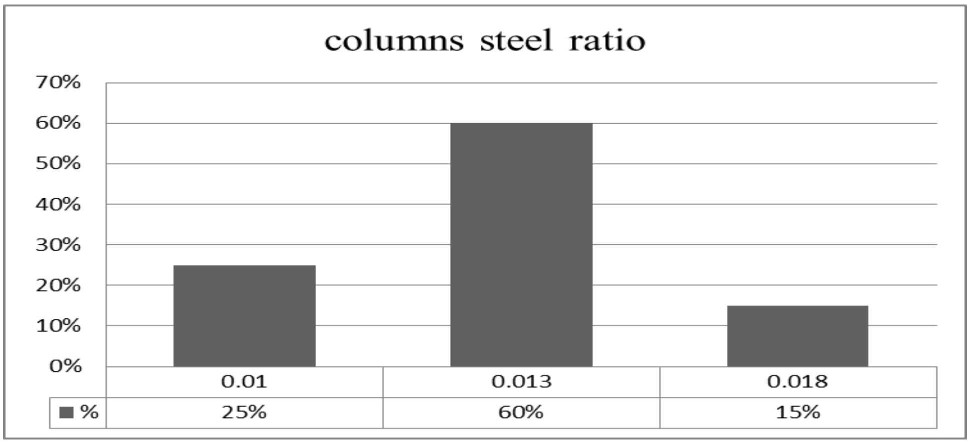


Figure 3.9: Statistics for Columns Steel Ratios of Hospitals in Palestine

3.3 STRUCTURAL SYSTEMS

Structure refers to a structure created by the interconnection of structural members designed to sustain or pass forces and to resist the loads applied to it safely or to avoid the collapse of buildings. By using a framed arrangement known as structural members, a structure supports the house. For the construction of a building, there are two essential steps:[35]

(a) Structural Analysis

(b) Structural Design

The prediction of the performance of a given system under specified loads and other external factors, such as support motions and shifts in temperature, is structural analysis. In Structural Design, the necessary structural members for the given impact load obtained from the structure analysis are chosen or developed. In specific, RC structures are recommended and chosen for reinforced steel and member sizes.

A structure's displacements or drift are functions of many variables, such as stiffness or strength and the structural system's tendency to deform (ductility). Other variables impact the structural deformations, such as the applied stress, both shear or flexure, containment and shear span. The influence of the axial load is an important element in the action of columns and walls. The increase in the axial load increases the member's shear resistance. Furthermore, it was experimentally observed that the increase in axial load decreases the lateral drift.[36]

3.3.1 Structural Walls and MRF Systems

The formulas suggested for vibration intervals can be added for higher to lower standard bare MRF structures and constructions with infills that have plan contrast ratio from 1:1 and 1:7. Buildings built to resist both gravitational and windy loads of the RC MRF are more rigid than those built just to resist gravity loads, resulting in fewer occurrences of vibrating[37].

Although the performance goals and the explanation of the damage associated with them can remain unchanged, it is clear that multiple sets of drift definitions are needed to set limits for different structural structures and components, such as:

a) Reinforced concrete moment resisting frame (MRF)

   a) Well-built ductile frames in line with applicable codes. In the code provisions, the defined drift limits may be used.

   b) Existing non-ductile detailing frame designed for earlier codes in determining the lateral load carrying ability of existing structures, the defined drift limits may be used.

   c) The moment of masonry infill's resisting base.

b) Structural walls :

   a) Aspect ratio (height/length) of flexural structural walls > 1.5

   b) Squat walls with an aspect ratio primarily shear conduct < 1.5. [38]

With respect to conventional quake structural structures, it is widely believed that Moment-Resisting Frames (MRFs) get the greatest energy dissipation capacity because of their ability to dissipate earthquake energy input at the beam ends wherein cyclical flex yield develops. However, due to their decreased lateral stiffness, MRFs contribute to major difficulties in meeting restricted state specifications for serviceability. If the number of stories increases, this downside becomes more and more significant.[39]

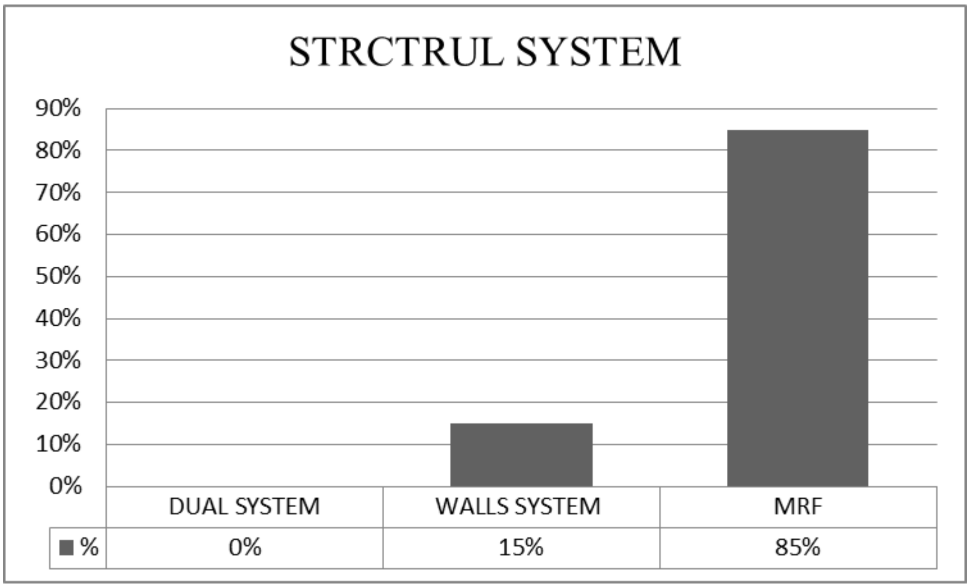
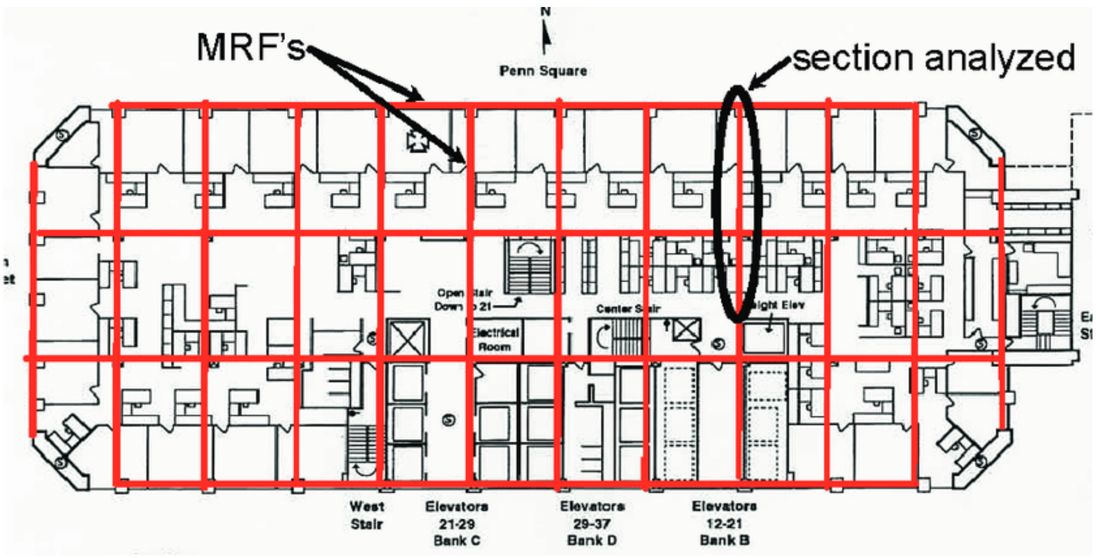


Figure 3.10: Statistics for Structural Systems of Hospitals in Palestine



3.3.2 Solid and Ribbed Slabs

The structural slab systems that will be discussed in the following chapters are either solid slabs or ribbed slabs; but the one-way ribbed slab systems are the most functional in Palestine. The following figure indicates the scanning outcomes:

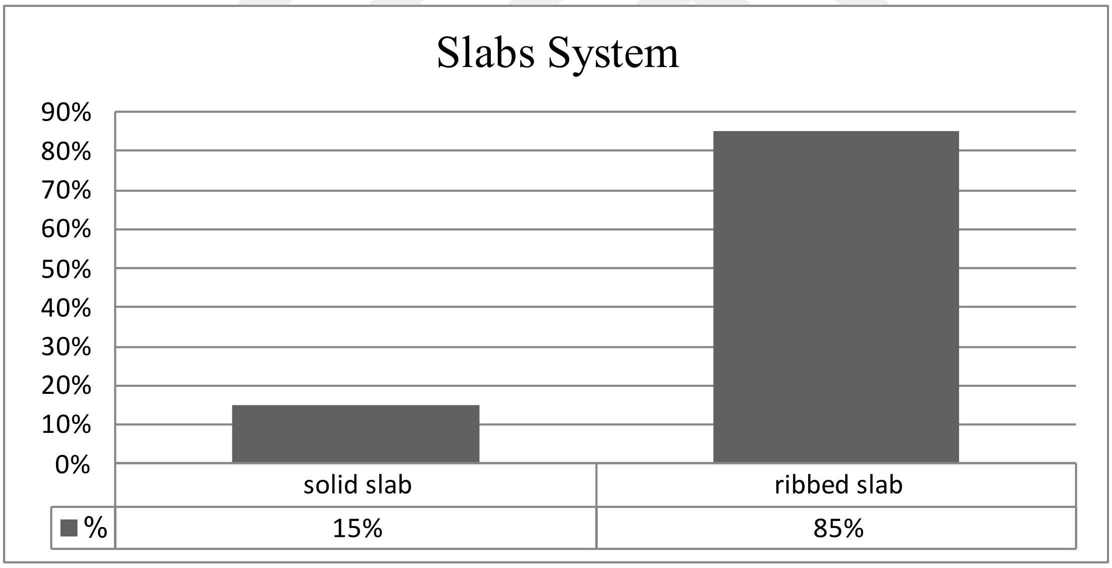


Figure 3.11: Statistics for Types of Slabs Used in Hospitals in Palestine

Solid slab, one-way slab, and two-way slab are two distinct groups. The one-way slab is the simplest kind of a solid slab. Because of its poor performance and weight, it is only considered economical for short stretches (up to 4.6m). [40]

A two-way slab is utilized for high loading and vast stretches on the opposite side. The two-way reinforcement slab is built to allow the slab to do two things: it must behave in two ways, and both things must be reciprocal. In order to decide the load sharing percentage between reinforcement packages, the long-to-short-side ratio of the floor panel is taken into consideration. It is possible to identify two main varieties of the Two-way slab, however:

a) A two-way slab protected from the outside, with either an edge support or a monolith beam as the bearing wall

b) The two-way slab with free end, also known as the free-end slab.[35]

A Ribbed Slab provides substantial additional strength in one direction, while a waffle slab provides additional strength in both directions. And this is only feasible in the monolithically cast concrete that is the beams' two-way grid. The ribbed slab span limits are considerably longer compared to the solid slab. For this type of slab, thus, longer span and light to moderate living loads (generally less than 3 KN/m^2) are used.[41]

3.4 OTHER FINDINGS

As a result of scanning methods, it has been found that some observations may be beneficial for our study and have a closed view of all hospital building components:

a) Most partition walls, with some lintels and jambs of doors and windows, are masonry block walls without reinforcements. It is also found that these partition walls are not isolated from the structure scheme, but are connected to the cold joint frames.
b) Forms of joints (Special MRF): All joint specifics have been found to conform with the joint specifications specified in the Special Moment Resistance Frames Framework (MRF).
c) For all columns and beams and all other elements, the concrete strength of the Moment Resisting Frames is 23.5 MPa strength (300 kg/cm^2 compression strength-for cylinder).
d) No Circular Columns are used in all systems and some square columns are used; in its cross sections, the remaining columns are rectangular.
e) Any hospitals are linked to retaining walls; this is part of the MRF retaining wall.

f) The highest benefit of hospital buildings is 30 percent in height with respect to their length.

g) In the majority of frameworks, expansion joints and other joints are located.

h) The staircases are built of reinforced concrete walls.

i) Any of the hospitals are built of stone walls without attachments to the frames or concrete walls.

3.5 SUMMARY OF ALL INVESTIGATION

The investigation done and the collected results can be presented as a description of previous parts in the following table 3.1 that will be included in the review chapters in this study:

Table 3.1: Statistics Summary for Structural Properties of hospitals in Palestine

Number of Stories	**5 floors**
Story Height	3.5 m
Columns dimensions	60 x 30 cm
Slab Beams Dimensions	60 x 25 cm
Columns Steel Ratios	0.013
Beams Steel Ratios	0.0055
Structural Systems	MRF
Slabs Systems	Ribbed
Secondary Walls	Masonry type

3.6 MODELING

This section will analyze and display models of actual hospitals using traditional models created by dimensional statistics in Palestinian hospitals representing their real activity as existing practice in Palestine, using technique in this thesis to reach important results while maintaining a stepping procedure.

As a development case of the practice model, another model will be developed that represents the minimum specifications of Moment Resisting Frames in Palestine.

3.6.1 Pushover Analysis

The Pushover analysis is a sort of non-linear static analysis used to determine the deformation capacity of a structure and its associated characteristics. It is typical for the performance of structural assessments using the Capacity spectrum method, displacement Coefficient method or the N2 technique to be used by simple non-linear static analytical methods or by pushover analysis. Contrary to that, time-history analysis that is nonlinear in nature, often referred to as non-linear dynamic analysis, can obtain structural response to a single ground motion or a collection of ground motions. Several techniques, for example incremental dynamic analysis, multiple stripe analysis, cloud analysis, etc, have been employed for nonlinear dynamic analysis, The capacity spectrum method was used here in this study. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance-based design. The ATC-40 (1996) documents have developed modeling procedures, acceptance criteria and analysis procedures for pushover analysis. These documents define force-deformation criteria for hinges used in pushover analysis. As shown in Figure 3.12, five points labeled A, B, C, D, and E are used to define the force-displacement behavior of the hinge and three points labeled IO, LS, and CP are used to define the acceptance criteria for the hinge. The IO, the LS and the CP stand for Immediate Occupancy, Life safety and Collapse Prevention, respectively. These are informational measures that are reported in the analysis results and used for performance-based design. Hinges are of various types – namely, flexural hinges, shear hinges and axial hinges. The first two are inserted into the ends of beams and columns. Since the presence of masonry infills have significant influence on the seismic behavior of the structure, modelling them using equivalent diagonal struts is common in PA, unlike in the conventional analysis, where its inclusion is a rarity. The axial hinges are inserted at either ends of the diagonal struts thus modelled, to simulate cracking of infills during analysis.

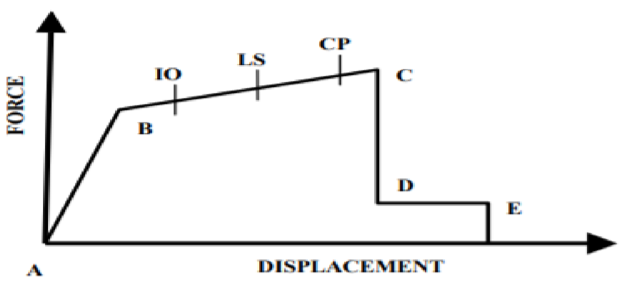


Figure 3.12: Force-Displacement Curve Defined for the Plastic Hinge in the pushover analysis

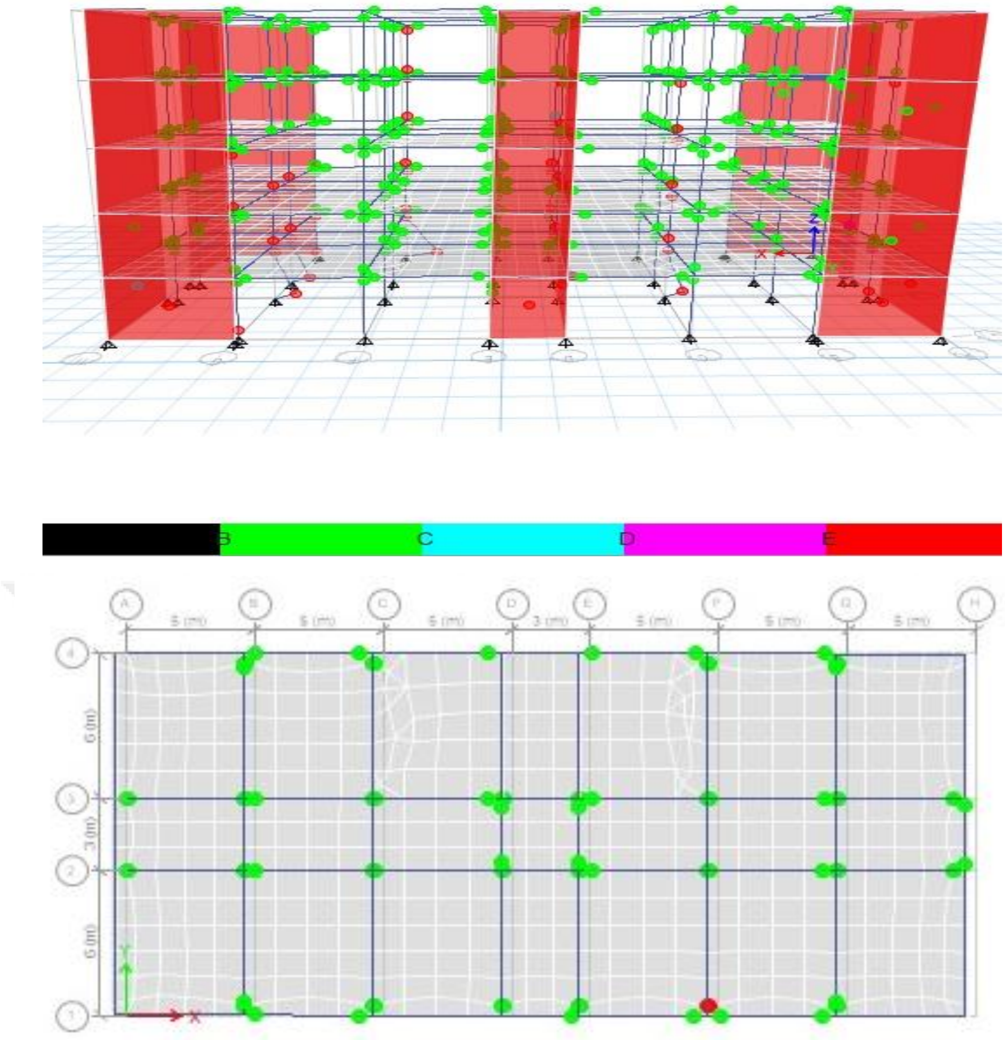


Figure 3.13: Plastic Hinge Formation

● Elastic range ● Beginning of yielding, ● Ultimate strength ● Residual strength ●

Maximum residual strength deformation

Analysis steps in using ETABS program:

a) Define load case pushover whit load case type (Nonlinear static)

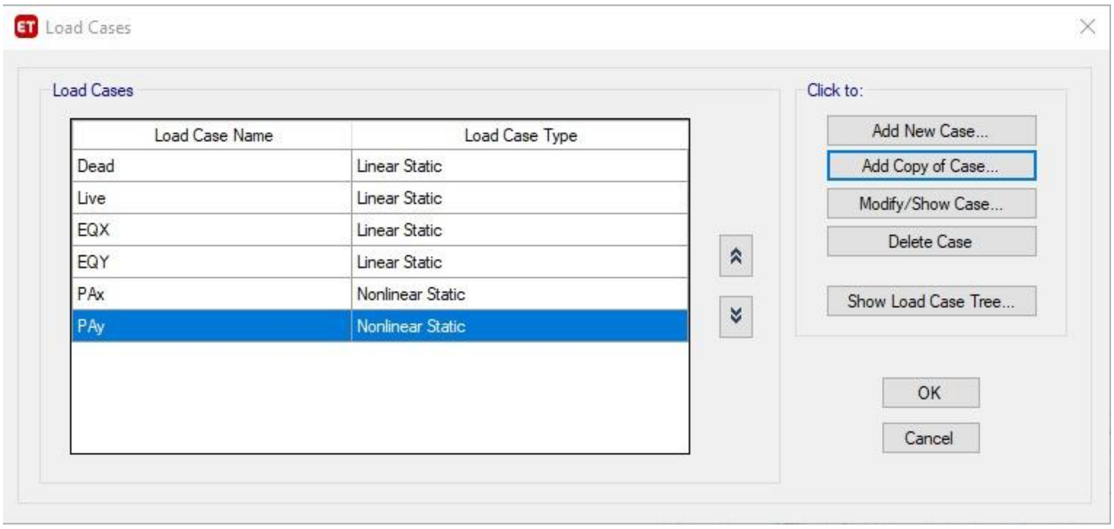


Figure 3.14: Define Load Cases in ETABS

b) Enter pushover analysis data

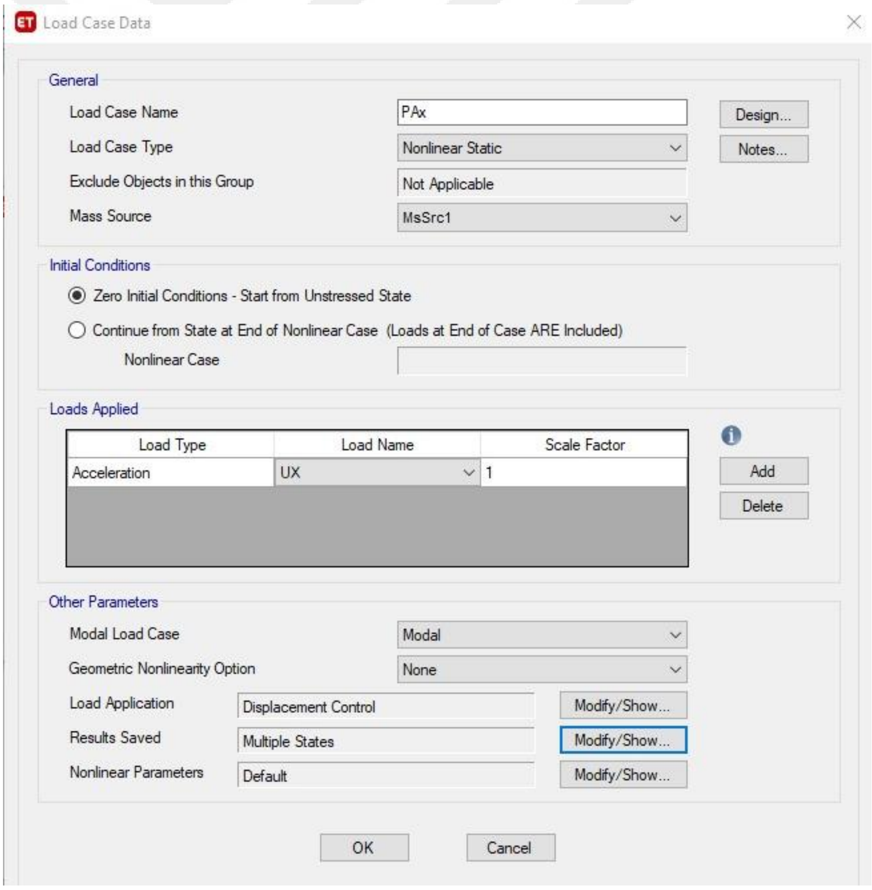


Figure 3.15: Load Case Data in ETABS

c) Select the elements of structural (beams, columns and walls)

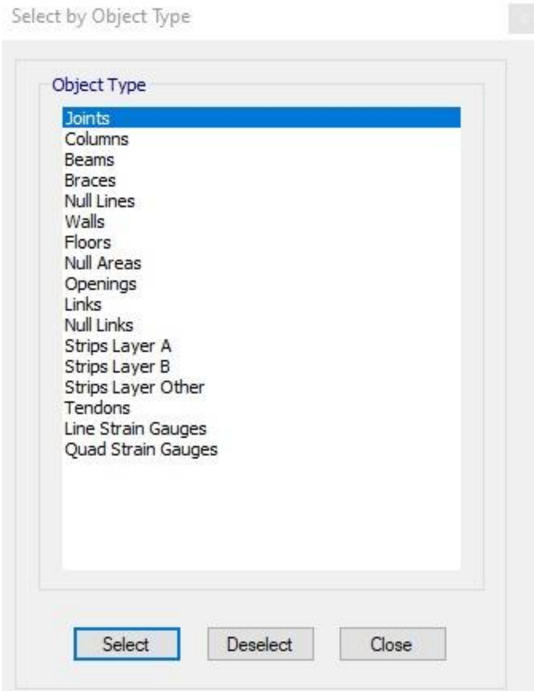


Figure 3.16: Select Object Type in ETABS

d) Frame hinge assignment data

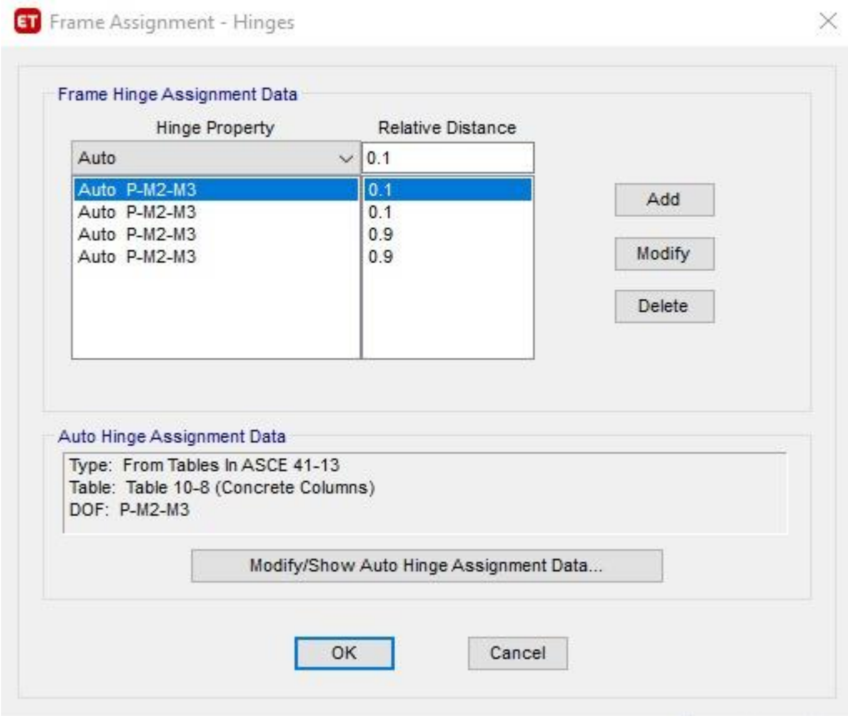


Figure 3.17: Frame Assignment - hinge in ETABS

e) After selecting the columns, we select a hinge table and auto hinge type then chosen the load case

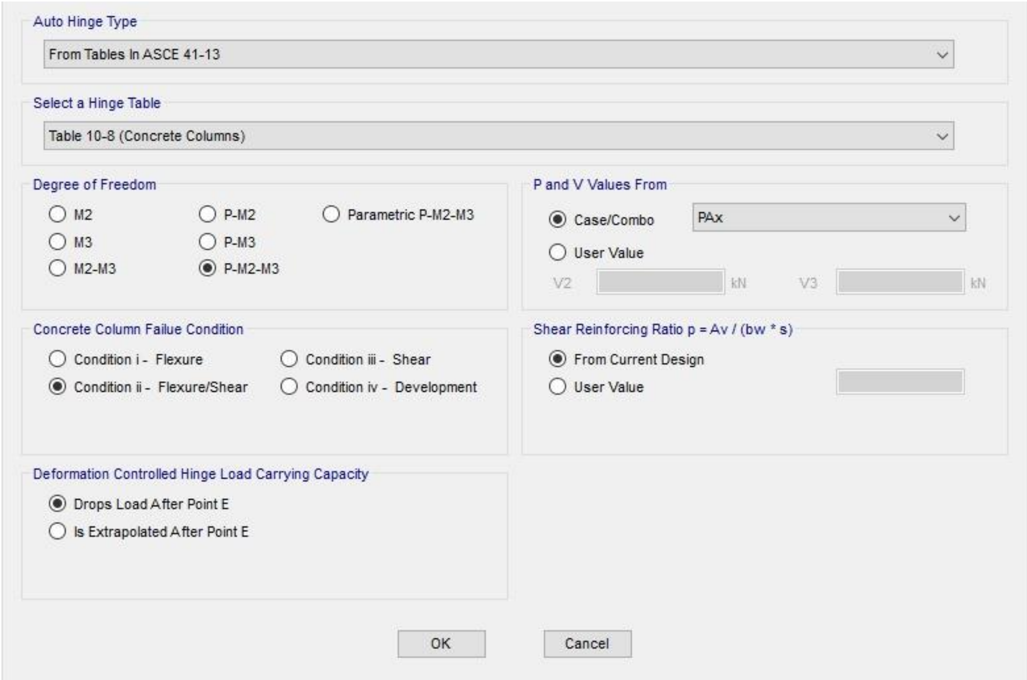


Figure 3.18: Auto Hinge Assignment Data (columns) in ETABS

f) After selecting the beams, we select a hinge table and auto hinge type then chosen the load case

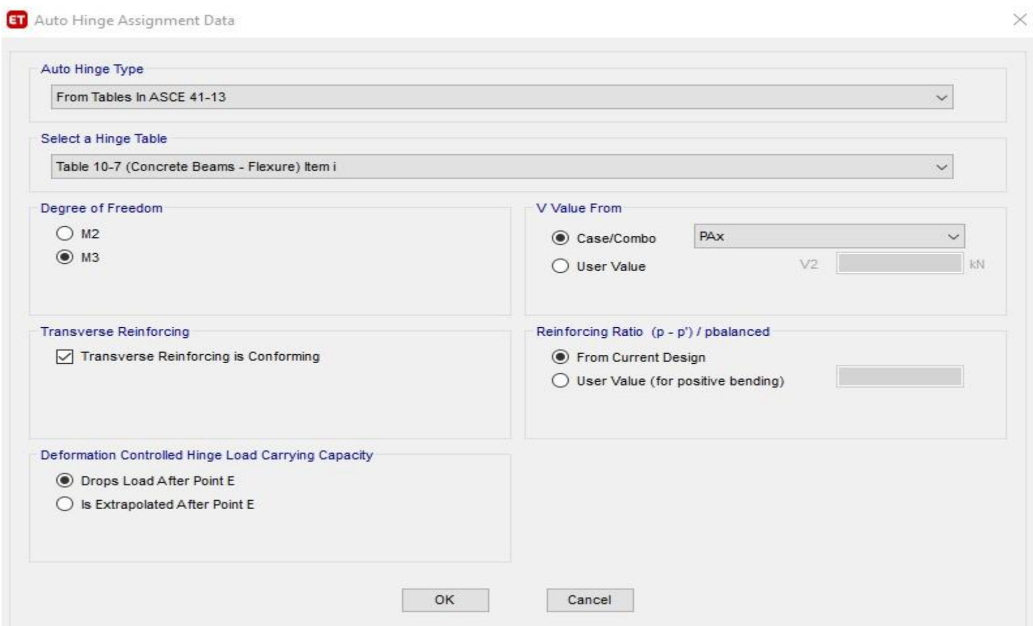


Figure 3.19: Auto Hinge Assignment Data (beams) in ETABS

g) After selecting the walls, we shell hinge assignment data

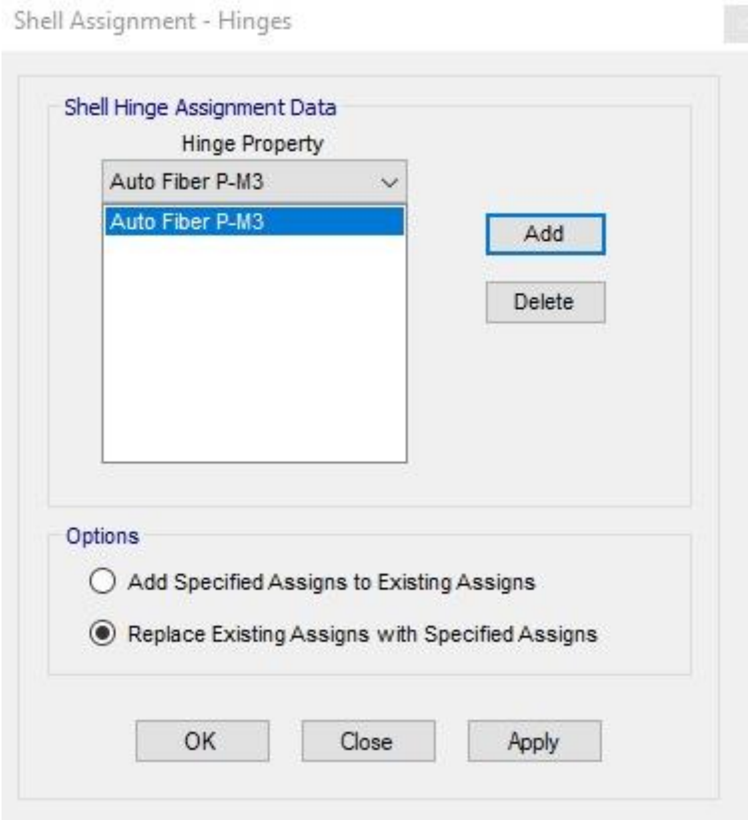


Figure 3.20: Shell Assignment – Hinges in ETABS

h) We selecting columns and beams, and then choose hinge overwrites

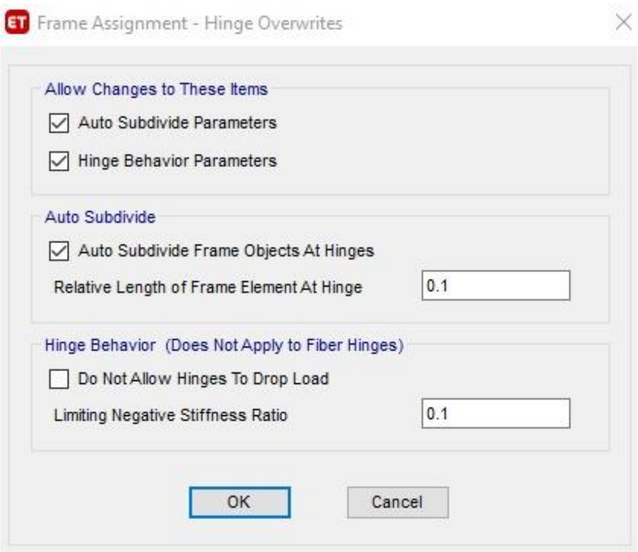


Figure 3.21: Frame Assignment – Hinge Overwrites in ETABS

After applying these steps, we run the analysis of the model until we get the results.

3.6.2 Beam Column Joint

The type of joint used Beam-column joint, Beam-column joint must transfer the forces, such as moment, shear and torsion, transferred by the beam to the column so that the structure can maintain its integrity to carry loads for which it is designed. Another function of the beam-column joint is to help the structure to dissipate seismic forces so that it can behave in a ductile manner.

THINGS TO CONSIDER FOR BEAM COLUMN JOINT

a) Longitudinal bars of beams, or slab, must be able to develop their yield stress, so that the beam/slab can transfer moment to joint. It means that longitudinal bars must have adequate development length for hooked bars. This implies that the size of the column must be such that bars can develop their tensile forces. If bars can transfer moment, they can also transfer shear as far as monolithic construction is concerned.
b) The shear strength of the joint must enable the transfer of moment and shear through it.
c) The joint should be Constructible: Congestion of reinforcement is the main concern.

3.6.3 Structural Irregularities

When a building is subjected to seismic excitation, horizontal inertia forces are generated in the building. The resultant of these forces is assumed to act through the center of mass (C.M) of the structure. The vertical members in the structure resist these forces and the total resultant of these systems of forces act through a point called as center of stiffness (C.S). When the center of mass and center of stiffness does not coincide, eccentricities are developed in the buildings which further generate torsion. When the buildings are subjected to lateral loads, then phenomenon of torsional coupling occurs due to interaction between lateral loads and resistant forces. Torsional coupling generates greater damage in the buildings. Eccentricity may occur due to presence of structural irregularities. The presence of structural irregularity changes the seismic response and the change in the seismic response depends upon the type of structural irregularities. As mentioned previously structural irregularities may be classified into horizontal and vertical irregularities.

Irregularities defined in code address:

a) Mass and/or stiffness irregularities by requiring a dynamic analysis for “taller” buildings in “higher” seismic zones (short period buildings tend to be first mode dominated – static method not bad).
b) Offsets etc.… treated by requiring a dynamic analysis for “taller” buildings and prescribing some system limitations.
c) Post disaster buildings, limit irregularities (basically in “higher” zones, only mass irregularities and nonorthogonal system allowed).

Types Of Structural Irregularities:

a) Vertical stiffness irregularity:
   lateral stiffness of the SFRS in a storey: < 70% of that in any adjacent storey, or < 80% of the average stiffness of the 3 storeys above or below.
b) Weight (mass) irregularity:
   weight of a storey > 150% of weight of an adjacent storey. (a roof lighter than a floor below is excluded).
c) Vertical geometric irregularity:
   horizontal dimension of the SFRS in a story > 130% of that in any adjacent story. (One-story penthouse excluded).
d) In-plane discontinuity:
   in-plane offset of an element of the SFRS, or reduction in lateral stiffness of an element in the story below.
e) Out-of-plane offsets:
   discontinuity of lateral force path e.g., out-of-plane offsets of the elements of the SFRS.
f) Discontinuity in capacity (weak story):
   story shear strength less than that in the story above. (Story shear strength = total of all elements of the SFRS in the direction considered)
g) Torsional sensitivity:
   if the ratio B > 1.7.
   $B = \delta max / \delta avg$

δ : calculated for static loads applied at $\pm$ 0.10 D_n

h) Non-orthogonal systems:

SFRS not oriented along a set of orthogonal axes.

3.6.4 Real (Practice) Model

Geometry, according to previous chapters, especially statistics and hospital form, the dimensions of structural elements given in Table 3.2 will be used in modeling:

Table 3.2: Practice Model Elements Dimensions

Element	Dimensions (cm)			
	Width	Depth	Thick	Height
Columns	80	80	…	350
Beams	80	40	…	…
Solid Slab	…	…	20	…

The average number of stories for all hospitals is four to five stories; hence, we can use 5 stories in the typical model as most often used, as well as rooms in this model as follows:

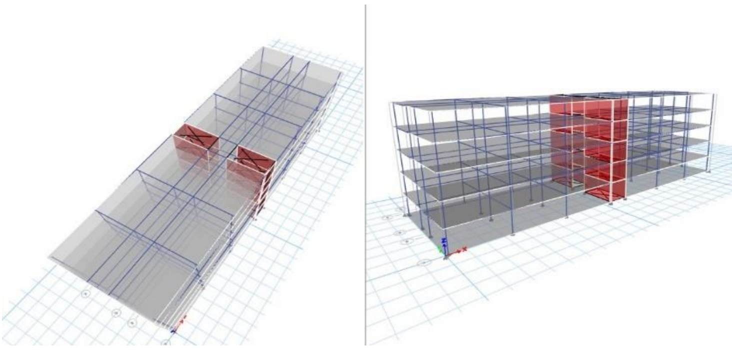


Figure 3.22: Typical Model of Hospital

3.6.5 Sensitivity Study and Meshing Size

Table 3.3: The Automatic Meshing Size Gives a Reliable result compared with different specific sizes

Mesh Size	Maximum displacement (mm) x-axis	Maximum displacement (mm) y-axis
Automatic	56	50
100 mm	110	105
80 mm	87	80
50 mm	66	63

From the table 3.3 shown, it is clear that the automatic meshing size gives a reliable result compared with different specific sizes, it also worth mentioning that the 50 mm meshing size showed a divergence problem and takes much time during the analysis, as the Automatic meshing size and other sizes 100 and 80 gave similar results, the automatic was considered as well as this gave us a hint about the validity of the models results for this pushover analysis study.

3.6.6 Material Properties

Reinforced concrete is a building material that is widely used in the construction of hospitals and most other buildings in Palestine. The assumed unit weight of this reinforced concrete is $(\gamma_c)\ of\ 25kN/m^3$.

Concrete compressive strength $\grave{f}c = 24\ MPa$, modulus of elasticity $E_C = 23270\ MPa$, and steel yielding strength of $fy = 420MPa$ were used for all structural elements composing all versions.

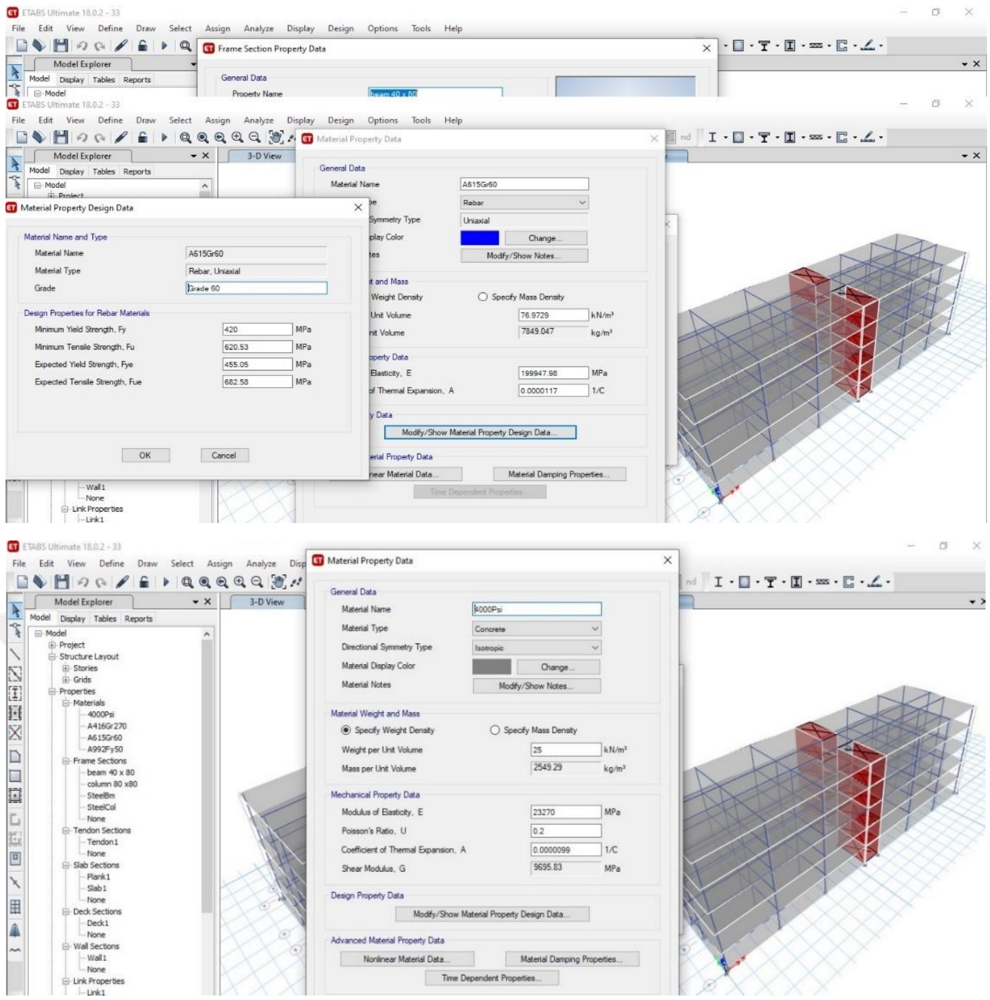


Figure 3.23: Material Properties Data From ETABS.

3.6.7 Loads on Hospitals Buildings

Dead and live loads, as well as seismic loads, are applied to the system in the ETABS model.

The dead load is considered to be the mass of the structure and any dead load that may be added (SDL) to be approximately 4.0 kN/m^2.

According to the requirements and limitations defined by the Palestinian Education Ministry, live loads are set at $5kN/m^2$.

Seismic loads induce multidirectional vibrations of buildings that sit on the ground. In addition, all seismic factor calculations are included in the model and are discussed in Chapter Four.

The picture is below showing the behavior and important parameters of the seismic component used in the model.

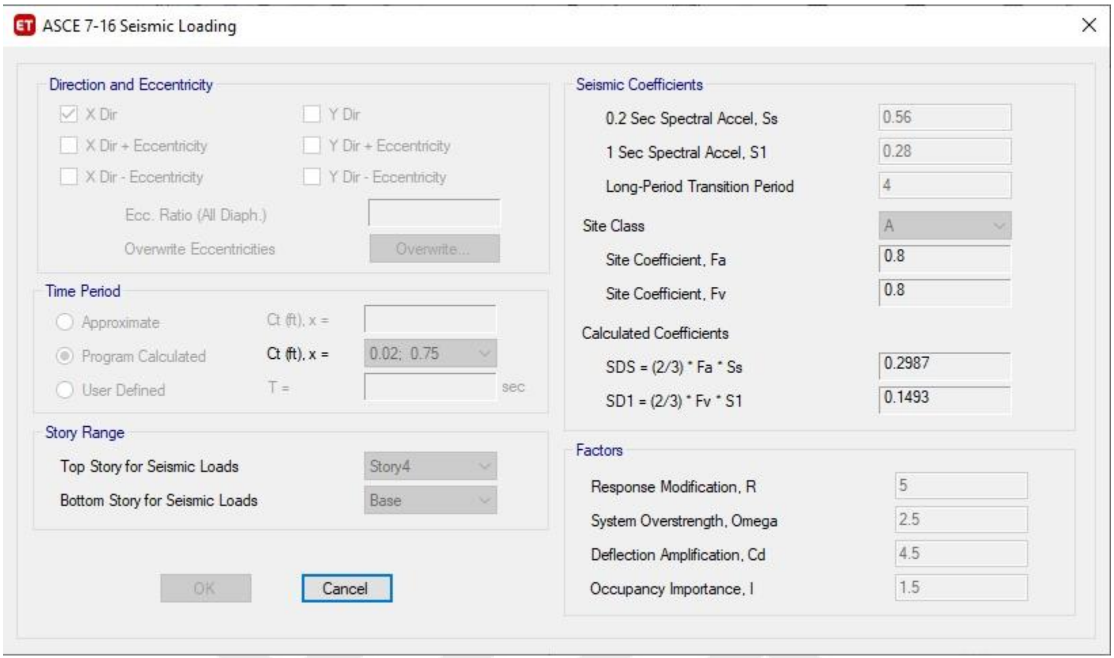


Figure 3.24: Seismic Parameters of Practice Typical Model

After defining all of the loads imposed on the buildings, I selected (pushover x and pushover y) from among the loads to conduct a building analysis, with the results appearing as shown in the diagram below.

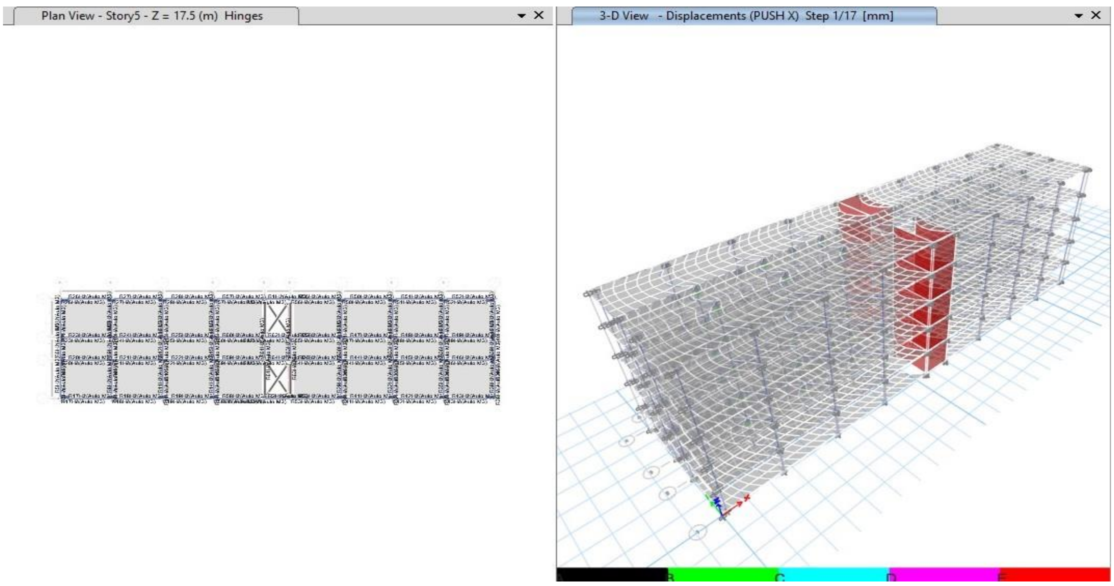


Figure 3.25: Structure Looks when it Has Been Analyzed and Exposed to Loads

3.7 PROPOSED MODELS

Some versions of the following hospital building models have been proposed in accordance with special requirements, so that each model varies from the others in terms of the measurements of the structural components that comprise the structures, with a variation in the dimensions of the rooms to align the dimensions with the design objectives.

The construction of these models, according to the ACI 318, considers the design according to the basic rules for design, so that these designs will be a reference for the designer engineers in the region to learn from them and benefit from the final results.

The aim is to requirements for new solutions and suggestions in the private aspect of hospital design in particular, which can then be applied to public buildings in general.

3.7.1 Geometry and Restraints

With the same geometry as previously stated, the following structural Elements accomplished and meet the specifications of the code of practice.

There may be differences in the structural elements that comprise the models; for example, some models may have reinforced walls whose placement varies from one model to the other, as well as the form of slabs used in the construction.

Table 3.4: Properties of Model (A)

Model	Element	Type	Dimensions (cm)			
			Width	Depth	Thick	Height
(A)	Columns	Rectangular	80	80	…	350
	Beams	Rectangular	40	80	…	…
	Slab	Solid	…	…	20	…
	Shear wall		…	…	25	350

According to these data in the table in terms of the structural parts have dimensions, we designed this building and these are the results resulting from the design by using the ETABS program.

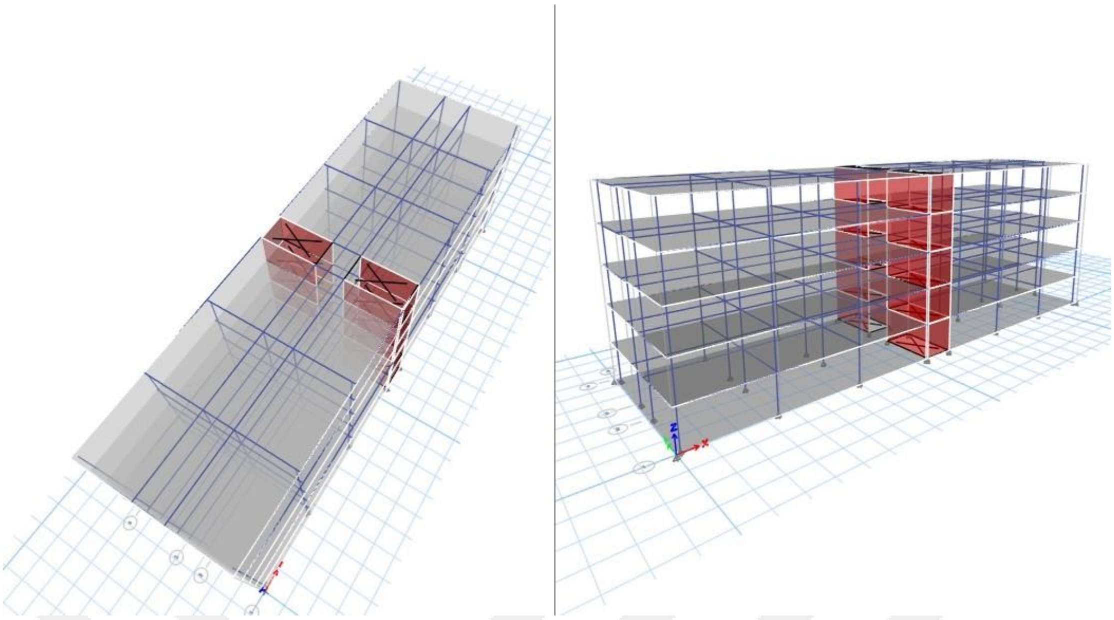
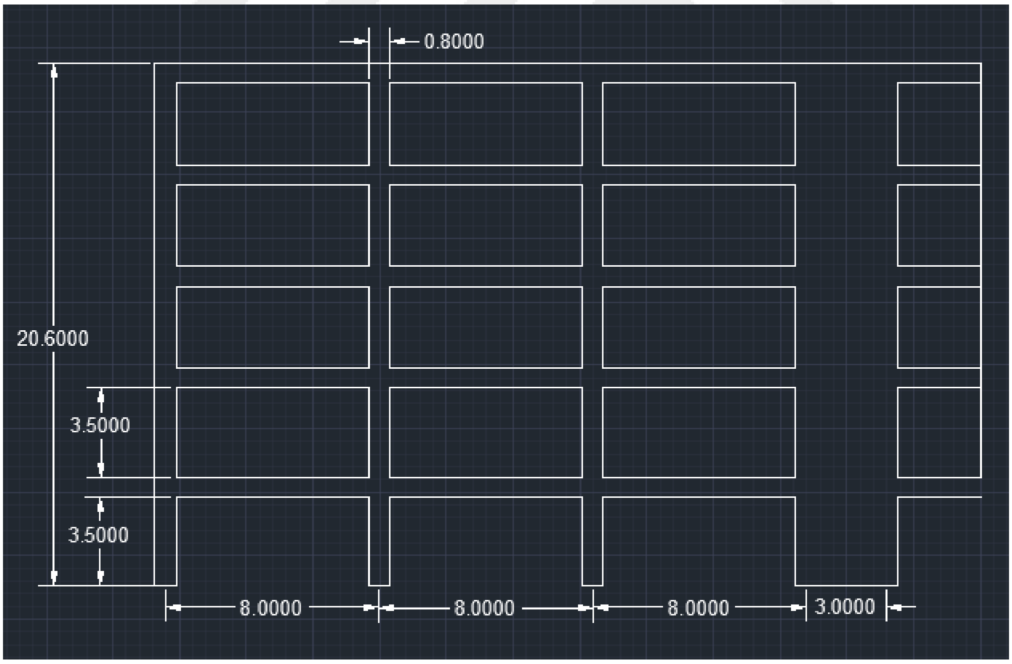


Figure 3.26: Typical Model of Hospital (A)



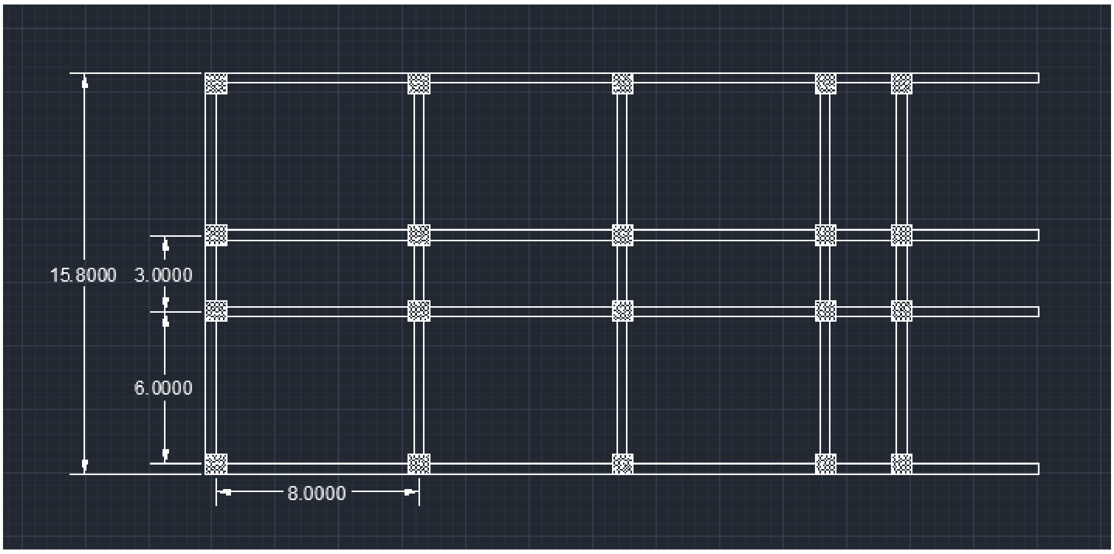


Figure 3.27: Plan for Model (A)

Shear force

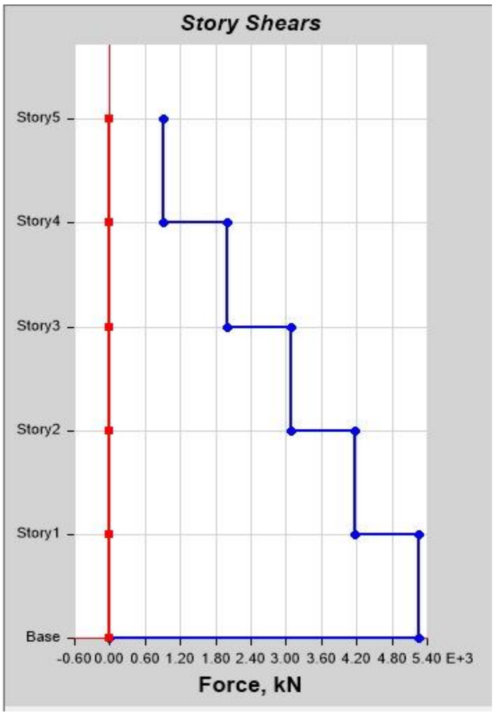


Figure 3.28: Base Shear Force Curve for Model (A)

For columns

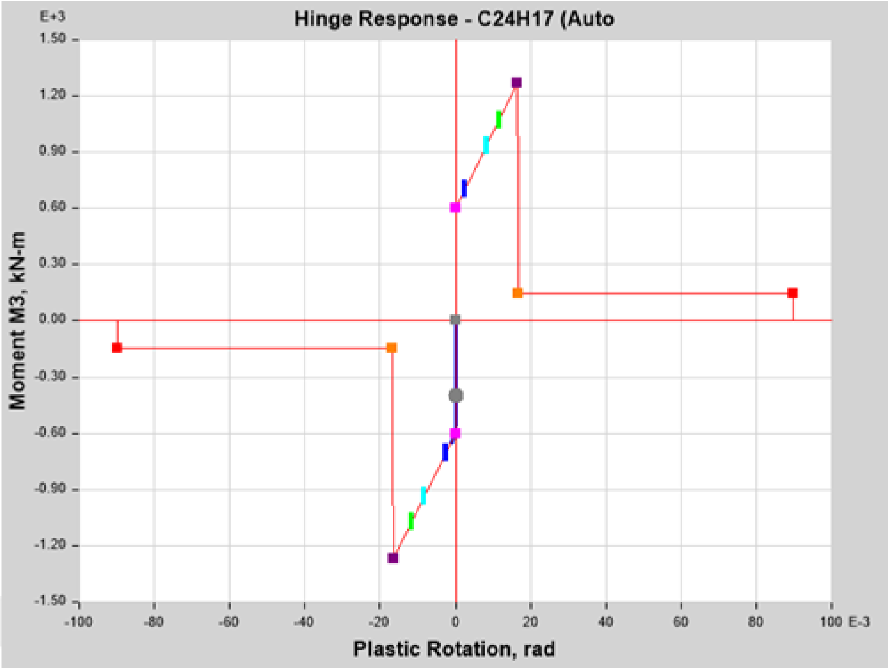


Figure 3.29: Columns Curve for Model (A)

The maximum positive moment 1.22 kN.m, negative -1.22 KN.M

For beams

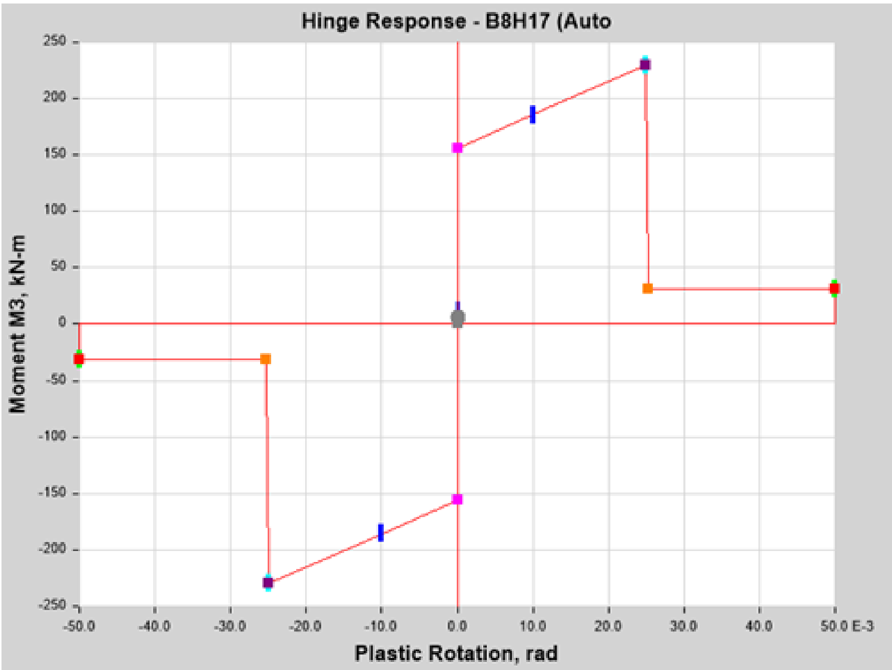


Figure 3.30: Beams Curve for Model (A)

The maximum positive moment 235 KN.M, negative -235 KN.M

Pushover Curve:

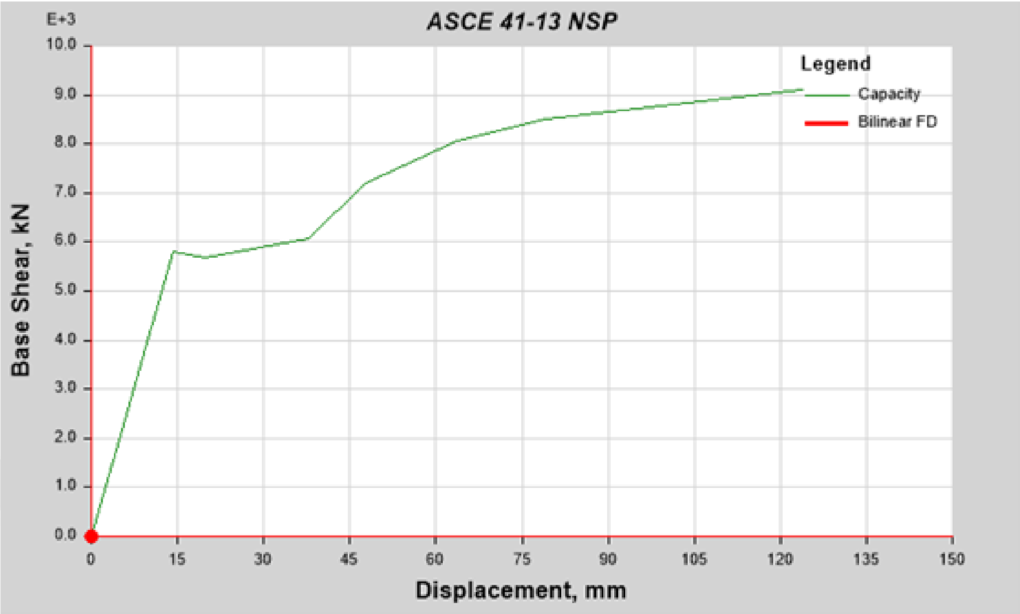


Figure 3.31: Pushover Curve for Model (A)

Base shear of 5.85 kN and displacement of 14.9 mm gives the performance point, whereas base shear of 9.2 kN with displacement of 124 mm yields the maximum point. Wall

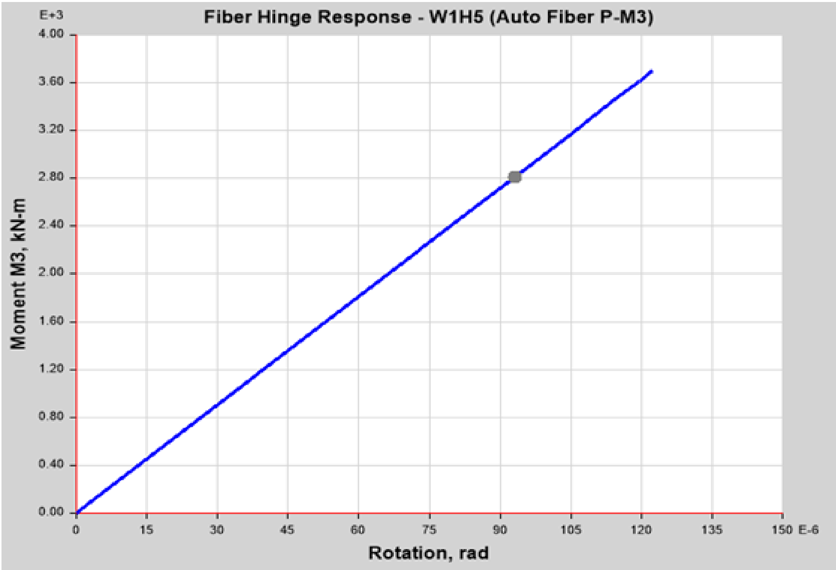


Figure 3.32: Wall Curve for Model (A)

Table 3.5: Properties of Model (B)

Model	Element	Type	Dimensions (cm)			
			Width	Depth	Thick	Height
(B)	Columns	Rectangular	30	60	...	300
	Columns	Rectangular	40	80	...	300
	Beams	Rectangular	30	60	...	...
	Beams	Rectangular	30	80	...	...
	Slab	Solid	...	...	32	...
	Shear wall		...	...	25	300

According to these data in the table in terms of the structural parts have dimensions, we designed this building and these are the results resulting from the design by using the ETABS program.

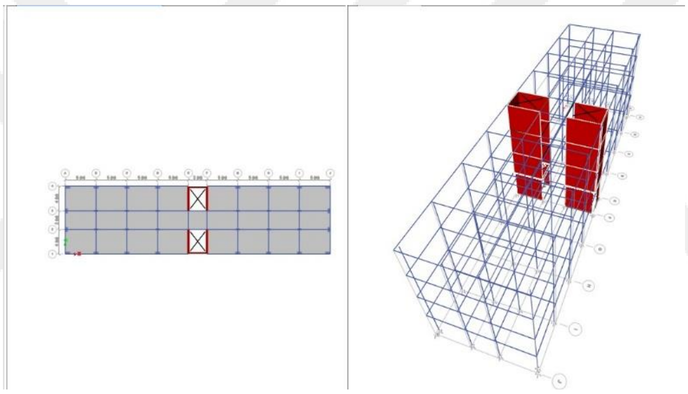
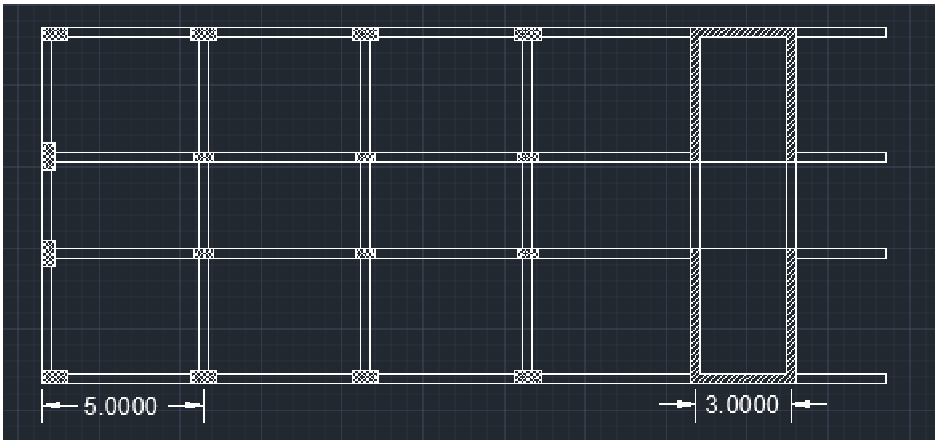


Figure 3.33: Typical Model of Hospital (B)



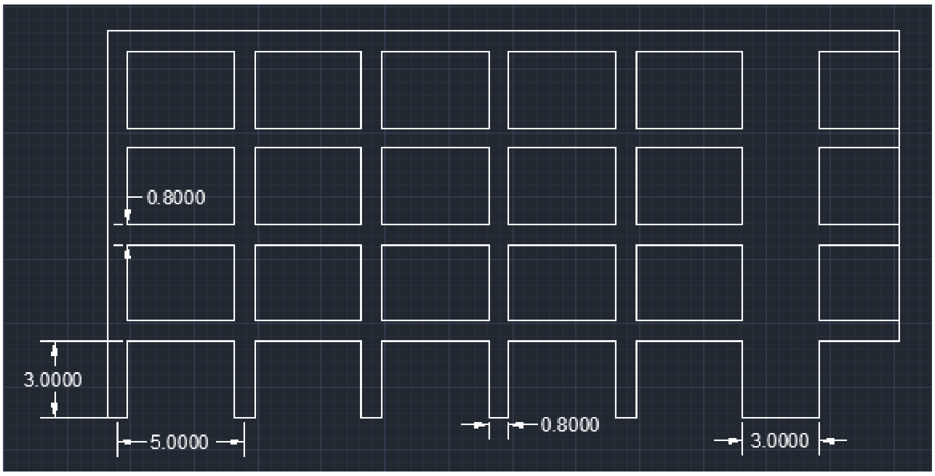


Figure 3.34: Plan for Model (B)

Force shear

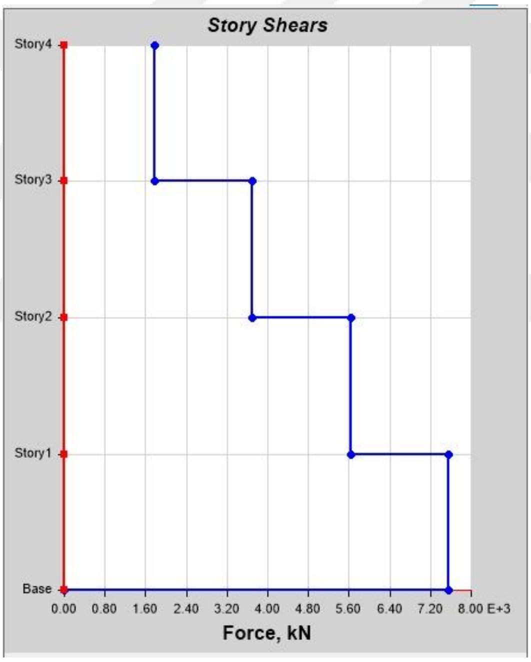


Figure 3.35: Base Shear force Curve for Model (B)

For column

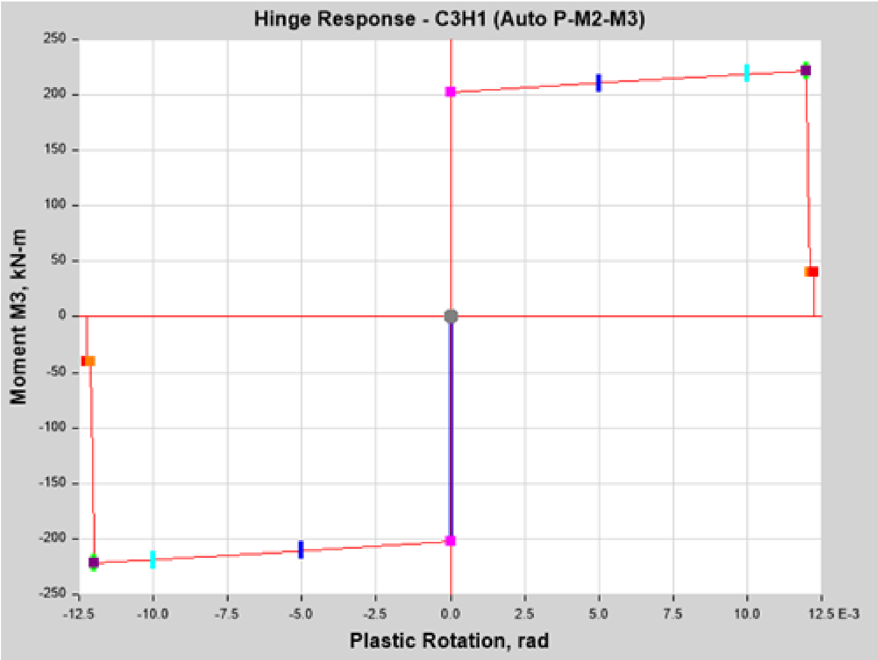


Figure 3.36: Columns Curve for Model (B)

The maximum positive moment 220 KN.M, negative -220 KN.M

For beam

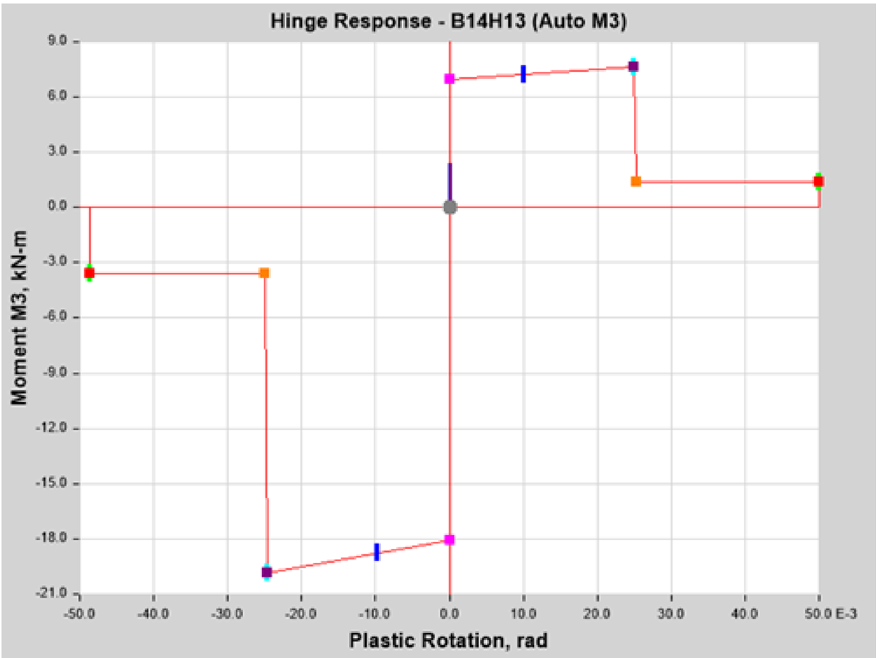


Figure 3.37: Beams Curve for Model (B)

The maximum positive moment 7.2 KN.M, negative -20 KN.M

Pushover Curve

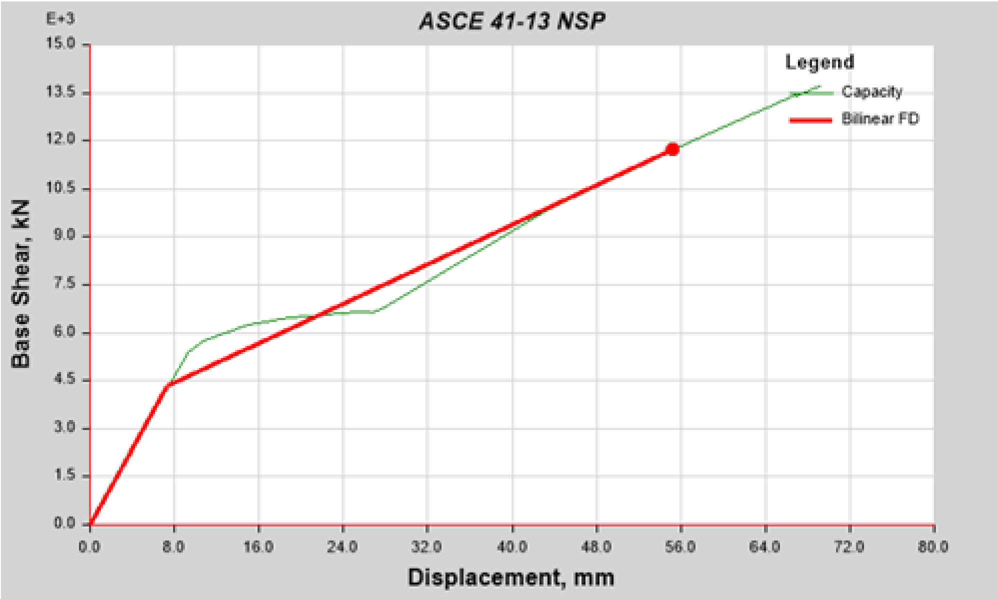


Figure 3.38: Pushover Curve for Model (B)

At a shear level of 4.4 KN and displacement of 7.9 mm, the performance point is reached at a base shear level of 4.4 KN. This base shear level is 4.4 KN, and the highest performance point is gained at a shear level of 13.55 KN and displacement of 68 mm.

Table 3.6: Properties of Model (C)

Model	Element	Type	Dimensions (cm)			
			Width	Depth	Thick	Height
(C)	Columns	Rectangular	80	100	…	350
	Beams	Rectangular	80	40	…	…
	Slab	Ribbed	…	…	32	…
	Shear wall		…	…	25	350

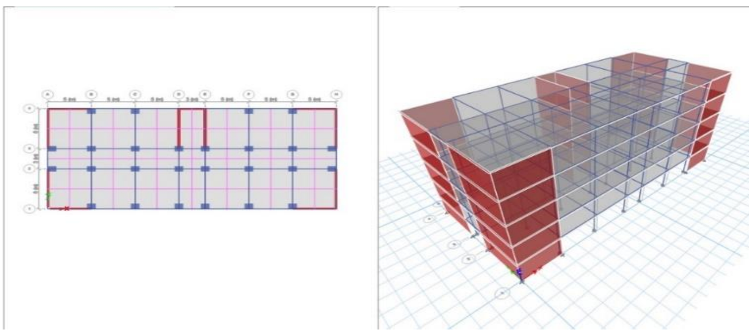
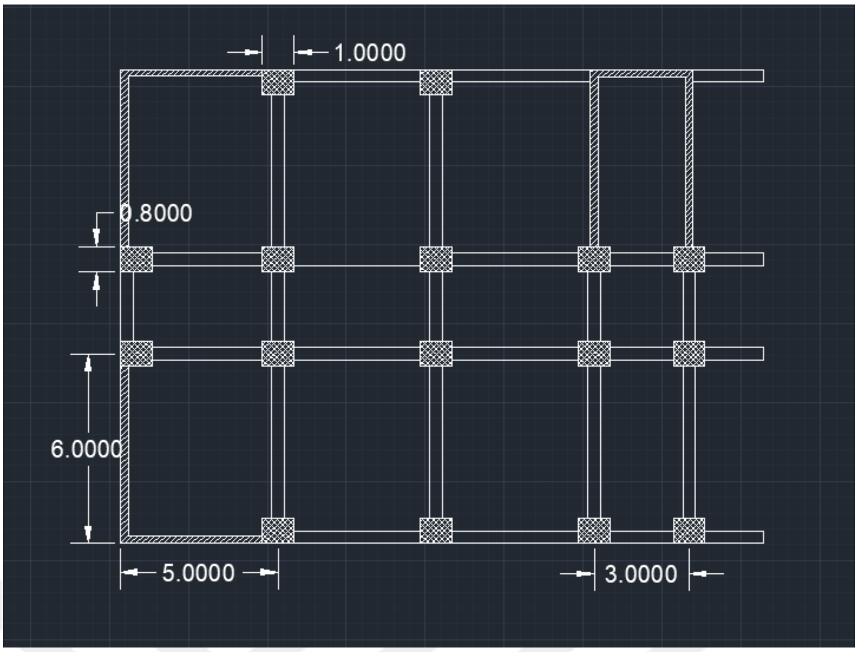


Figure 3.39: Typical Model of Hospital (C)



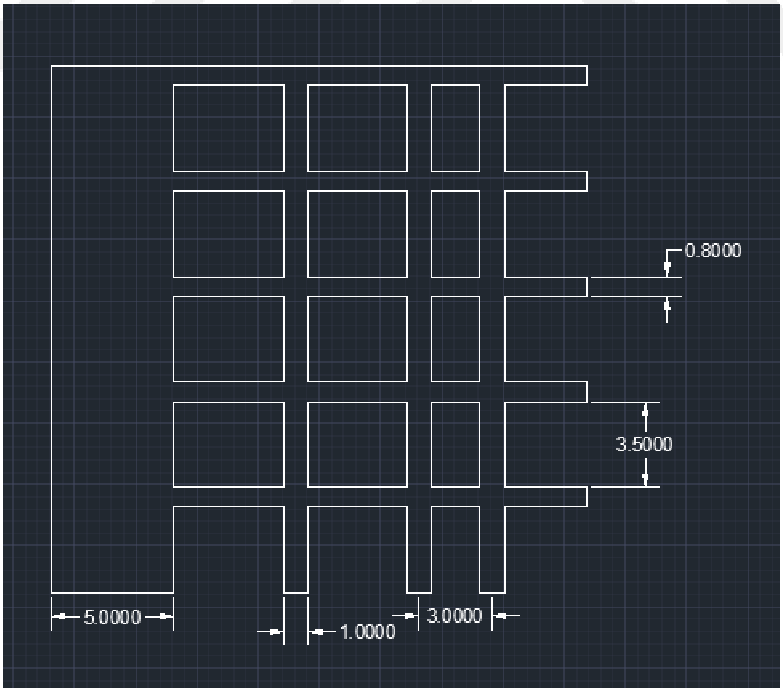


Figure 3.40: Plan for Model (C)

Shear force

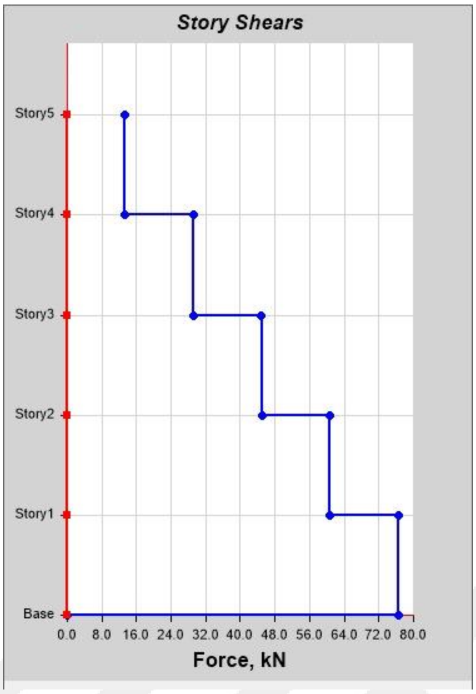


Figure 3.41: Base Shear Force Curve for Model (C)

For columns

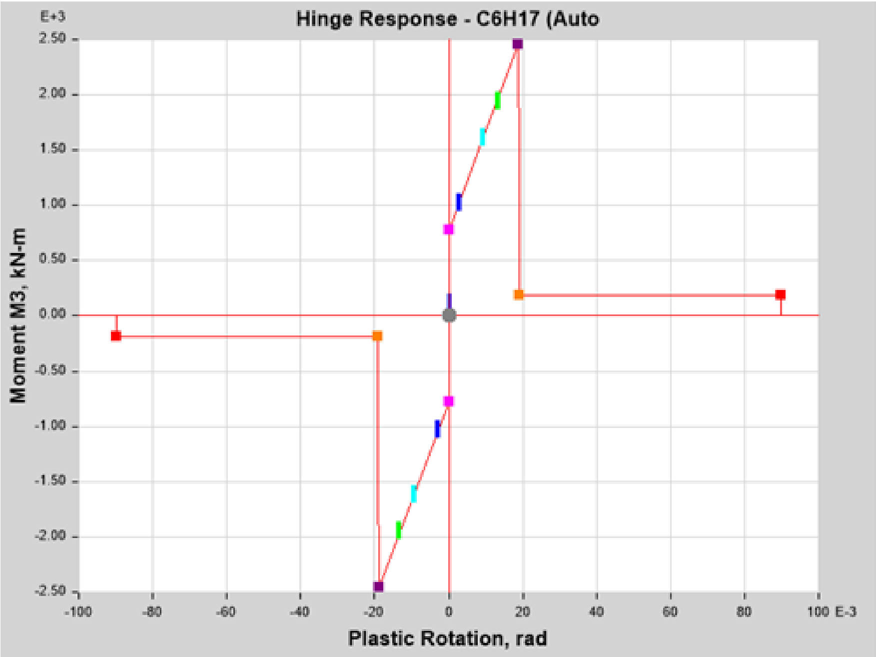


Figure 3.42: Columns Curve for Model (C)

(The maximum moment 2.50 KN.M)

For beams:

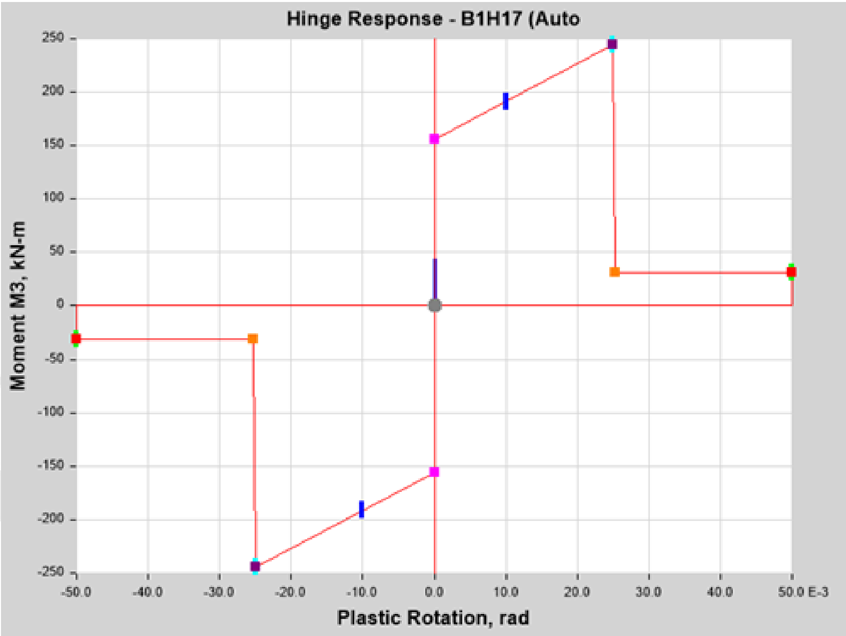


Figure 3.43: Beams Curve for Model (C)

The maximum positive moment 250 KN.M, negative -250 KN. M

Pushover Curve

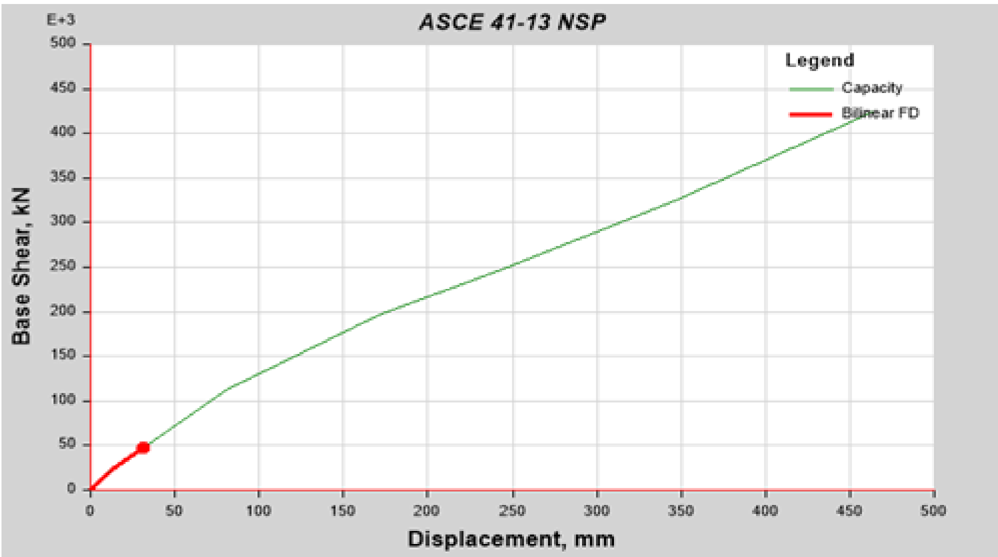


Figure 3.44: Pushover Curve for Model (C)

At 25 kN base shear, the performance point is achieved. In terms of displacement, the maximum point is achieved at 420 kN base shear, and a displacement of 467 mm

For walls

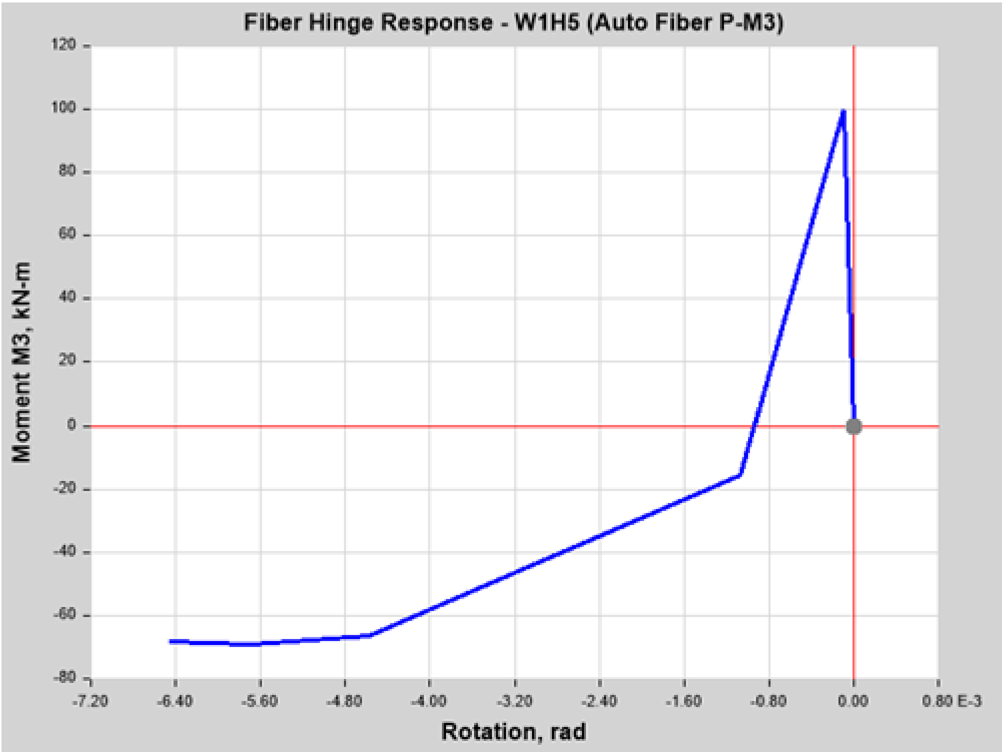


Figure 3.45: Walls Curve for Model (C)

- the highest value of the torque in the positive region is 100 KN and then shifted in the torque value to reach -68 KN in the negative region.

4. TOPOGRAPHIC COMPARISONS AND CALCULATIONS

The topography influences natural variability in hydrological conditions. superficial topography and groundwater movement both affect the distribution of soil wetness. [42] The spatial soil wetness trends have been explained by using topographic parameters. The spatial distribution of groundwater levels has an impact on soil processes, which in turn has an impact on soil properties.[43]

Other things, including changes in the geological and geophysics qualities of nearby soils and rocks, can dramatically affect the intensity of nearby seismic events. As a consequence, it's important that these possible differences are taken into account in seismic hazard evaluations like the one being carried out Earthquake Risk Harmony in Europe, a project funded by that of the European Seventh Framework Program (FP7) (SHARE). Although a local influence can be explained by the shear wave velocity of the top layers at the spot, it is typically taken into consideration when calculating the shear wave velocity of the entire system[44]. Three hospitals were chosen from the three different topographic regions of Ramallah, Qalqilya and Jericho and based on all data obtained from the drawings of public hospitals in Palestine that were scanned and viewed in the previous chapter. Close to the fault, Jericho represents severe soil versus seismicity behavior. Ramallah represents moderate conditions in the middle of the country. When you think of places distant from of the fault, Qalqilya, in southern, comes to mind.

ASCE07-10 and IBC-2015 Codes are applied to make comparisons and analysis of the three hospitals.

4.1 FORCE DEFORMATION SHAPE

Knowledge and practice reveal that structure could be designed to meet only a portion of the estimated elastic earthquake-resistant forces even while upholding the safety-related requirements (SEAOC Seismology Committee 2008).

In accordance with this engineering concept, it is to be expected that inelastic action and building damage will occur. This R influence can be seen when calculating the decrease in quake force designed. The R factor's intention is to simplify the building conceptual design so that most structural systems only require a linear elasticity statically assessment (equivalent lateral load approach).

We believe that a force-controlled component will sustain a far larger rate of earthquake forces than projected. in terms of real seismic design forces to provide ductility when regulating member deformation required to deform inelastically. Additionally, the code calculates real quake forces within those force-controlled parts using an amplifier termed Ωo, which yields a ratio that simply computes the quake forces within those parts. Also, referred to as the "Building Over-strength Ratio for Drifting or Checking" in ASCE07, this concept is applied in a similarly manner A Deflection Amplification Ratio is developed for estimating projected peak permanent deformation, as designed quake forces cause the anticipated peak deformed. Cd in ASCE07 is referred to that as the Cd ratio. The following figure depicts a typical response envelope linking force to deformation, which can be measured by research or a pushover examination

.

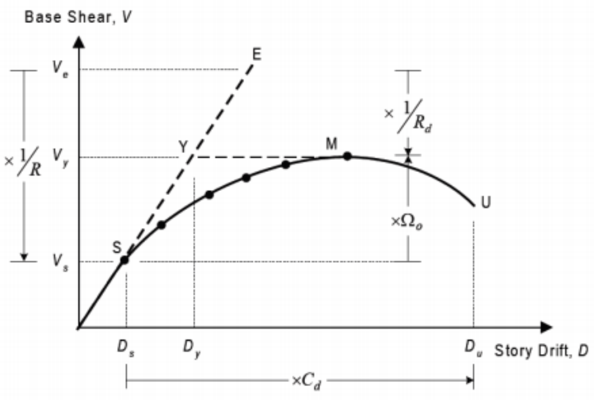


Figure 4.1: In-Elastic Force Deformation Shape

The system first has an elastic response, after which, the horizontal forces increase, resulting in an inelastic reaction. Throughout the structure, a sequence of plastic hinges forms, leading to a yielding mechanism called V_y at the point of pressure. The concept approach is based on a simplified technique. A builder first measures the elastic design base shear, V_e, based on the structure's fundamental linear elastic duration (see point E in the previous figure). The R factor is then added to V_e, leading to an overall design force of earthquake significance (point S in the previous diagram) Vs, above which elastic analysis is no longer accurate.

When doing capacity design, it is useful to know the force created at the planning level of quake force (Vs). We use an overstrength ratio (Ωo) to amplify this force. Displacement readings multiplied by a deformation amplification ratio (Cd) are used to determine the drifting at (Vs). Any obvious improvement is meant to create a rational partnership. Between the inelastic response reduction and the response spectrum demand a provided structural system's capacity.

4.2 SEISMIC HAZARD MAPS

Formula (1) may be used to compute the cumulative distributed form of ground motion considering into consideration various sorts of emitters that surround a study region, their location, and focal thickness The score of the PGA would be computed for a specified risk level. We split the sample area into a grid of 0.25x 0.25-degree area using a computer software (HAZ81).

$$P[A \leq a, t] = 1 - exp\left\{-\sum_{k=1}^{NA} \gamma_k \left(\frac{a}{b_1}\right)^{\delta_k} t\theta_k \int_{R_{1k}}^{l_{2k}} [[R_{hi}^2 + b_4]^{\rho_i}]^{\rho_k} R dR\right\} \text{-------------(1)}$$

The PGA values for each location or node of the grid adopted, extracted for a given property of exceedance. Seismic danger maps can be generated using the results of a grid of different locations in an area, which display the contours of equivalent average PGA. That, beyond a reasonable doubt, will not be surpassed for a duration of (T) years.

The frequency of exceedance or the mean return period time may also be used to calculate the seismic motion level. The following equation can be used to measure the return time.

$$T_R = -\frac{T_L}{\ln(1-P_R)} \text{------------------(2)}$$

where:

T_R = Ground movement returning period, in years.

T_L = The structure life expectancy in years.

P_R = The possibility of surpassing the predicted limit.

Seismic hazard maps are classified into two groups. Big hazard is high-threat infrastructures, including electricity plants and bridges. The maps of return periods can be used to depict this kind (475 and 1000 years). The next cause is a low degree of hazard, which correlates to assets that are less significant. The maps of return periods reflect this kind (50, and 100, and 225 years).

In 2007, a seismic hazard map, was issued in Palestine. Although the attenuation relationship used is not even-date, the peak ground acceleration (PGA) of general rocks was measured and 10% surpassed at 50 years. The Jordanian Seismic Survey Code, which roughly matches the US Unified Building Code, was recently introduced in Palestine. Prior to this, no seismic loads were contemplated during the building's construction. The required (Z) seismic zone factors have been published (Fig.4-2) based on the above danger map; Z represents the PGA value for rock site conditions (Vs30 = 760 to 1500 m / s).

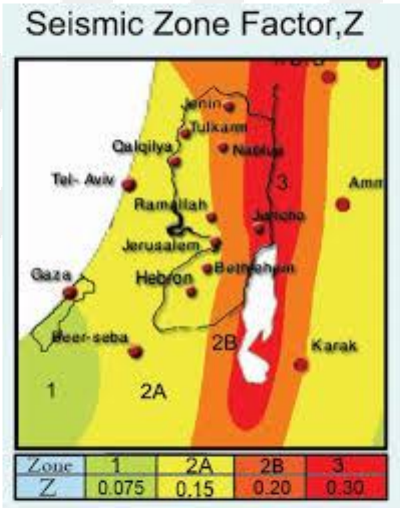


Figure 4.2: UBC97 (Z) Seismic Zone Factors for Palestine, Based on the 2007 risk map for the Levant region.

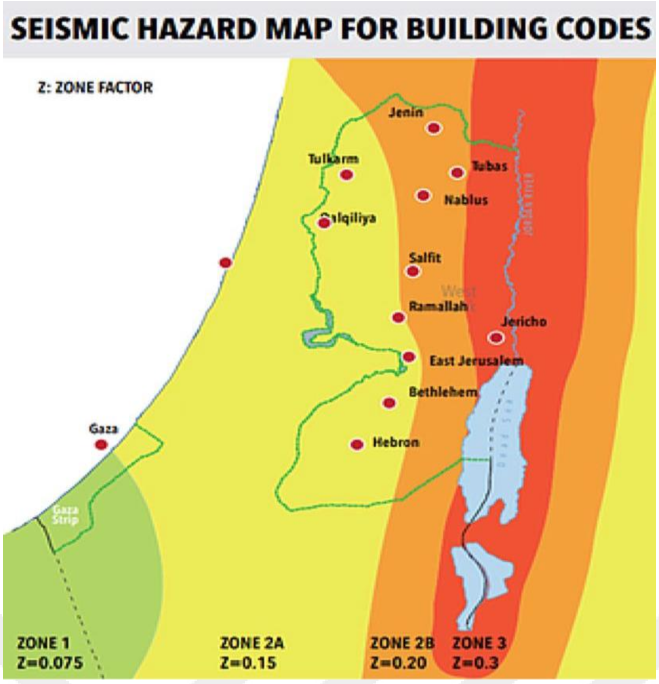


Figure 4.3: Seismic Hazar's Map For Building Codes For Palestine

4.3 SITE CLASSIFICATION AND SITE COEFFICIENTS AND RISK-TARGETED

Site Mapped Acceleration parameters S_s & S_1:

$$Ss = 2.5 * Z$$

$$S1 = 1.25 * Z$$

Develop max regarded seismic spectrum reaction accelerations to be in conformity with IBC 1613.3.3, optimized for site class impact (SMS) based on several parameters, like the length of the recurrence interval.

$$S_{MS} = F_a S_s$$

$$S_{M1} = F_V S_1$$

where:

a) F_a = IBC Table 1613.3.3 specified as a site coefficient (1).

b) F_v = IBC Table 1613.3.3 specified as a site coefficient (2).

Table 4.1: TABLE 1613.3.3(1) Value Of Site Coefficient F_a

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a [a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIOD				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

Table 4.2: Table 1613.3.3(2)Value Of Site Coefficient F_v

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_V [a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

Using spectral acceleration charts, determine the defined maximum spectral reaction acceleration for the considered MCE earthquakes, S_s for a short duration (0.2 s) and S_1 for a long period (1.0 s). If S1 is less than or equal to 0.04, and Ss will be less than or equals to 0.15, then the building is allocated to quake design class A.

Based on soil qualities, divide the field into multiple sections. Section 20 of the ASCE 7 mandates that the project be assigned a "A, B, C, D, E, or F" designation. The location category D should be used if the soil properties are not known to a sufficient depth to classify the location into one of the following classes: Class E or Class F.

Table 4.3: Table 20.3-1 Site Classification

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index $PI > 20$, —Moisture content $w \geq 40\%$, —Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft^2 = 0.0479 kN/m^2.

The calculations comply with IBC 1613.3.3 and arrive at the maximum considered seismic spectral reactive accelerations for various site impacts (short and long durations and S_{M1}), which have been tuned for site conditions. [45]

$$S_{MS} = F_a \, S_s$$

$$S_{M1} = F_V S_1$$

where:

a) Fa = IBC Table 1613.3.3 specified as a site coefficient (1).

b) Fv = IBC Table 1613.3.3 specified as a site coefficient (2).

c) S_{MS} = The greatest short-term accelerations which are used to calculate the spectrum response acceleration are found in section 1613. 3.3.

d) S_{M1} = For long period events, the maximum regarded spectral response acceleration accelerations that were determined in Section 1613. 3.3.

As described in IBC 1613.3.4, the damped designs of 5% SDS and SD1 with respect to spectral acceleration were measured according to these standards.

$$S_{DS} = \left(\frac{2}{3}\right) S_{MS}$$

$$S_{D1} = \left(\frac{2}{3}\right) S_{M1}$$

4.4 PROPOSED SOIL AND SITE CLASSIFICATION

Simple qualitative requirements for site classification, as proposed by some seismic codes, such as the Greco-Greek seismic code (EAK), For site effect assessments do not use soil class that conforms to current needs or conforming soil class for assessments of soil impact.

The existence of quantitative classification criteria that describe the characteristics of soil materials and site conditions makes selecting a suitable category more difficult. [46]

The following are the basic criteria that were used in the site classification scheme:

a) A selection of terms that include hard to soft clays, thick to lose sands, and other soil-rock form classifications are utilized. Average shear wave velocity also ranges between these proposed values.
b) before it reaches bedrock, the total shear wave speed is at its peak.
c) For clayey soils, the plasticity index and the undrained shear strength are used.
d) this duration is used as an approximate measure of soil layers' rigidity and thicknesses.

The architecture spectra have undergone one of the most significant improvements in earthquake codes. The Turkish Earthquake Code's architecture spectra, as well as those of three other well-known codes (UBC, Eurocode 8, and IBC), are compared in this paper.[47]

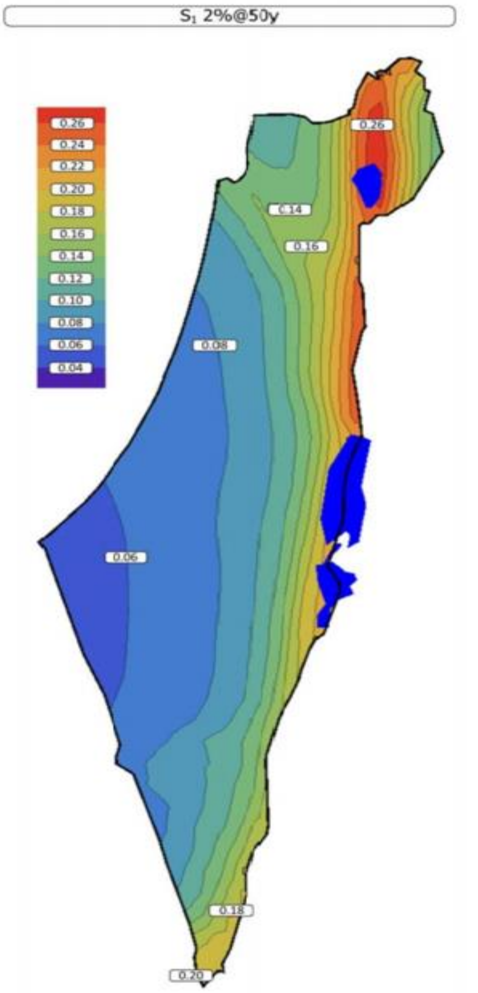


Figure 4.4: Long Period Spectral Acceleration for Palestine [45]

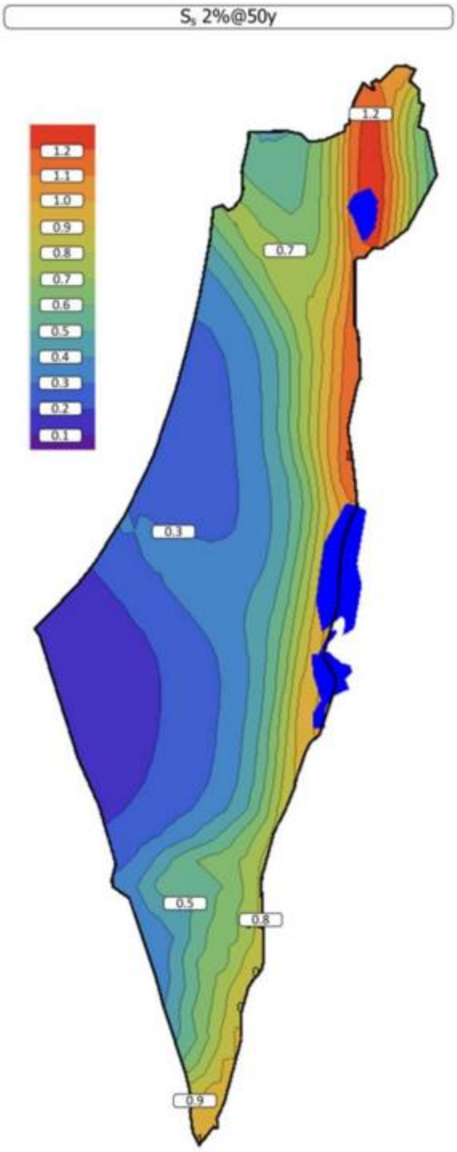
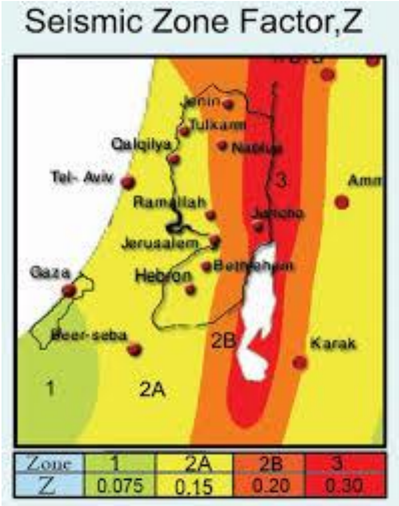


Figure 4.5: Short Period Spectral Acceleration for Palestine

4.4.1 Ramallah Region Hospital (Moderate Conditions)

This section will be evaluated based on its topography, with the following results based on ASCE07-10 and IBC-2015 codes of practice.



a) Site Classification:

In this section, the seismicity factor of zones in the Palestine Seismic Zone Map is calculated as follows:

Site Mapped Acceleration parameters S_s & S_1:

$$Ss = 2.5 * Z$$

$$S1 = 1.25 * Z$$

Using the Palestine Seismic Map, Z equals 0.18 (2A) in Ramallah.

$$Ss = 2.5 * 0.18 = 0.45 rad/sec$$

$$S1 = 1.25 * 0.18 = 0.23\ rad/sec$$

In addition, table 20.3-1 can be used to calculate soil classification

(ASCE7-16)

The bearing capacity of the soil in the hospital area is 2.3 kg/cm^2 (based on drawings), and the site class will be Kind (C) as follows:

Bearing Capacity = 230 KN/m^2

$$\overline{\mathrm{Su}} = \frac{230}{2*0.0479} = 2400 > 2000 \rightarrow tybe\ c$$

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index $PI > 20$, —Moisture content $w \geq 40\%$, —Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft² = 0.0479 kN/m².

b) Site Coefficients and Risk-Targeted:

In compliance with IBC 1613.3.3, calculate the max magnitude of seismic spectrum reaction accelerations that have been deemed for various impacts (optimal for different site classes), during both short and long period.

$$S_{MS} = F_a\, S_s$$

$$S_{M1} = F_V S_1$$

By interpolation, Ss=0.45rad/sec and Site Class type C we get: $F_a = 1.2$

By interpolation, S1=0.27 rad/sec and Site Class type C we get: $F_v = 1.53$

Accordingly, we Found:

$S_{MS} = 1.2 * 0.45 = 0.54\ rad/sec$

$S_{M1} = 1.53 * 0.23 = 0.35\ rad/sec$

c) design characteristics for spectral acceleration:

Design seismic spectrum response acceleration factor should be derived by formulas 11.4-3 and 11.4-4, both at a short time, SDS, and at a 1-second period, SD1.

And referring to section 11.4.5 in ASCE7-16: (For 10% probability)

$S_{DS} = S_{MS} = 0.54$ rad/sec

$S_{D1} = S_{S1} = 0.35$ rad/sec

Then we calculate: $T_0 = 0.2\ (S_{D1} / S_{DS})$

$$T_0 = 0.2\ (0.35 / 0.54) = 0.128 \text{ sec}$$

$$TS = (SD1 / SDS)$$

$$TS = (0.35 / 0.54) = 0.638 \text{ sec}$$

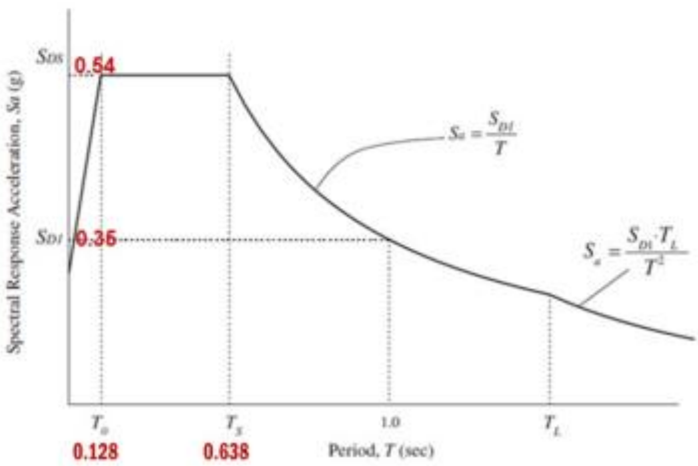


Figure 4.6: Design Response Spectra

d) Risk Category:

Referring to ASCE07-10 Table 1.5-1 the hazard category is (III):

Table 4.4: Table 1.5-1 Risk Category of Building and Other Structure for Flood, Wind, Snow, Earthquake and Ice Lodes

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures designated as essential facilities	IV

Then; Referring to tables (11.6-1 and 11.6-2) in ASCE7-16 and using SDS= 0.54 rad/sec ; SD1= 0.35 rad /sec, and in condition of that schools lay on (III) so it is found that category of risk shall be (D):

Table 4.5: TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

Table 4.6: Table 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

e) Seismic importance factor

Refer to table 1.5-2 in ASCE07-10, it is found that quake importance factor will be 1.25:

Table 4.7: Table 1.5-2Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

f) Quake Design Category.

Referring to the next table (table 11.6 in ASCE 7-10), it is found that in a structure with quake force resisting systems, these systems are referred to as "Special Moment resistant Frames."

Table 4.8: Table 11.6 Seismic Design Category

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category: B	C	D^d	E^d	F^e
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35[h]	NP[h]	NP[h]
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP[i]	NP[i]	NP[i]
5. Special reinforced concrete moment frames[s]	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP

g) Permitted analytical Procedure:

Referring to table 12.6-1 in ASCE 7-10, it is found that all procedures of analysis are permitted except "Equivalent Lateral Force" Procedure, see table:

Table 4.9: Table 12.6-1 Permitted Analytical Procedure

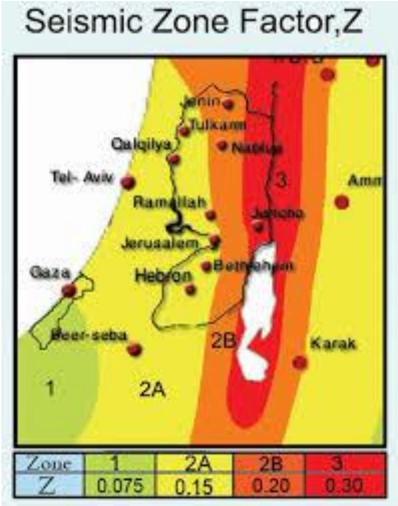
Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Procedure, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9.1, or Linear Response History Analysis, Section 12.9.2[a]	Nonlinear Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding two stories above the base	P	P	P
	Structures of light-frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft (48.8 m) in structural height	P	P	P
	Structures exceeding 160 ft (48.8 m) in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft (48.8 m) in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

4.4.2 Qalqilya Region Hospital (Mild Conditions)

Qalqilyah will be evaluated based on its topography, with the following conclusions based on ASCE07-10 and IBC-2015 codes of practice:



a) Site Classification:

In this section, the seismicity factor of areas near Qalqilya is calculated using the Palestine Seismic Zone Map as follows:
Site Mapped Acceleration parameters Ss & S1:

$$Ss = 2.5 * Z$$

$$S1 = 1.25 * Z$$

Using the Palestine Seismic Map, Z equals 0.18 (2A) in Qalqilya

$$Ss = 2.5 * 0.15 = 0.375 rad/sec$$

$$S1 = 1.25 * 0.15 = 0.188 rad/sec$$

In addition, table 20.3-1 can be used to calculate soil classification (ASCE7-16)

The bearing capacity of the soil in the hospital area is 4 kg/cm^2 (based on drawings), and the site class will be Kind (B) as follows:

Bearing Capacity = 400 kN/m^2

$$\overline{\mathrm{Su}} = \frac{400}{2*0.0479} = 4175 > 2000 \rightarrow tybe\ B$$

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index $PI > 20$, —Moisture content $w \geq 40\%$, —Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft² = 0.0479 kN/m².

b) Site Coefficients and Risk-Targeted:

For brief periods (S_{MS}), use ASCE 7-10 Eqs. 11.4-1 and 11.4-2, which are calibrated for Site Classes results. For 1 second (S_{M1}), use the spectral reaction acceleration factor from the documentation, ASCE 7-10.

$$S_{MS} = F_a * S_s$$

$$S_{M1} = F_v * S_1$$

The following table was used to calculate Fa and Fv equals 1:

Table 11.4-1 Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_S.

Table 11.4-2 Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_1.

$$S_{MS} = F_a * S_s$$

$$S_{MS} = 1 * 0.375 = 0.375 \, rad/sec$$

$$S_{M1} = F_v * S_1$$

$$S_{M1} = 1 * 0.188 = 0.188 rad/ \, sec$$

c) Design Spectral Acceleration Parameters:

The design seismic spectral response velocity factor, at S_{DS} and at S_{D1}, can be calculated using Formulae 11.4-3 and 11.4-4.

And referring to section 11.4.5 in ASCE7-16:

For 10% probability:

$$S_{DS} = S_{MS} = 0.375 \, rad/sec$$

$$S_{D1} = S_{M1} = 0.188 \, rad/sec$$

Then we calculate:

$$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} = 0.10 \, sec$$

$$T_S = \frac{S_{D1}}{S_{DS}} = 0.50 \, sec$$

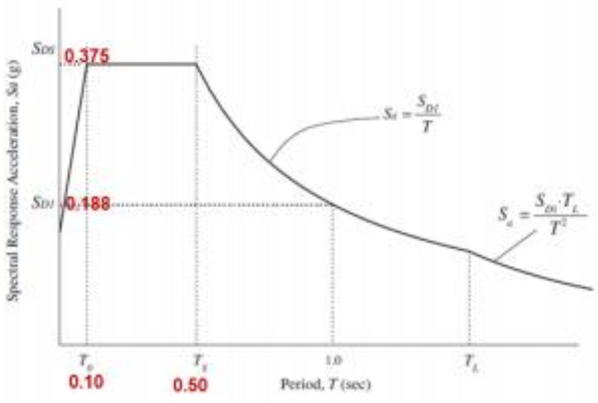


Figure 4.7: Design Response Spectra

d) Risk Category:

Referring to ASCE07-10 Table 1.5-1 the hazard category is (III):

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures designated as essential facilities	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

Then, using ASCE7-16 tables (11.6-1 and 11.6-2) and SDS= 0.375 and SD1= 0.188 in the state that hospitals lay on (III), it is determined that the danger group that shall be (B):

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D
$0.50 \leq S_{DS}$	D	D

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{D1}$	D	D

e) Seismic importance factor:

According to ASCE07-10 table 1.5-2, the seismic value metric will be 1.25, as follows:

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

f) Seismic Design Category:

According to the following table (table in ASCE 7-10), the quake force resisting system must be as follows: Intermediate Moment Resisting Frames:

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category: B	C	D^d	E^d	F^e
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35[h]	NP[h]	NP[h]
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP[i]	NP[i]	NP[i]
5. Special reinforced concrete moment frames[n]	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP

g) Permitted analytical Procedure:

According to table 12.6-1 in ASCE 7-10, all research procedures are allowed, with the exception of the "Equivalent Lateral Force" Technique see table:

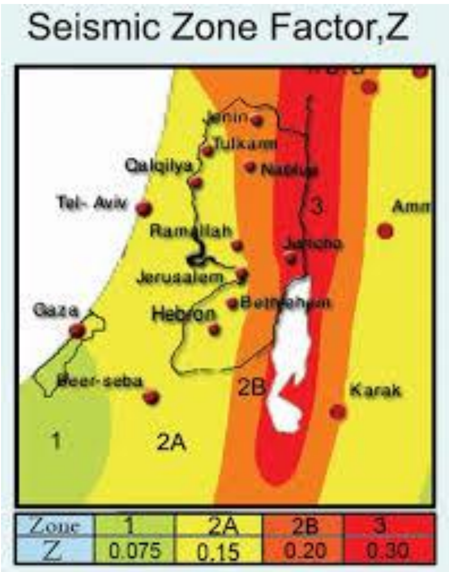
Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Procedure, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9.1, or Linear Response History Analysis, Section 12.9.2[a]	Nonlinear Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding two stories above the base	P	P	P
	Structures of light-frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft (48.8 m) in structural height	P	P	P
	Structures exceeding 160 ft (48.8 m) in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft (48.8 m) in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

4.4.3 Jericho Region Hospital (Sever Conditions)

Jericho will be evaluated based on its topography, with the following results based on ASCE07-10 and IBC-2015 codes of practice:



a) Site Classification:

In this section, the seismicity factor of zones as Jericho is located is calculated using the Palestine Seismic Zone Map as follows:

Site Mapped Acceleration parameters S_S & S_1:

$$S_S = 2.5 * Z$$

$$S_1 = 1.25 * Z$$

Using the Palestine Seismic Map, Z in Jericho equals 0.30. (3)

$$S_S = 2.5 * 0.3 = 0.75 \; rad/sec$$

$$S_1 = 1.25 * 0.3 = 0.375 \; rad/sec$$

In addition, table 20.3-1 (ASCE7-16) can be used to calculate soil classification: The bearing capacity of the soil in the hospital district is 0.95 kg/cm2 (based on drawings), and the site class will be Kind (D) as follows:

Bearing Capacity = 95 KN/ m^2

$$\bar{S}_U = \frac{95}{2 * 0.0479} = 1000 < 2000 \;\rightarrow Type\ D$$

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index $PI > 20$, —Moisture content $w \geq 40\%$, —Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft² = 0.0479 kN/m².

b) Site Coefficients and Risk-Targeted:

For brief periods (S_{MS}), use ASCE 7-10 Eqs. 11.4-1 and 11.4-2, which are calibrated for Site Classes results. For 1 second (S_{M1}), use the spectral reaction acceleration factor from the documentation, ASCE 7-10.

$$S_{MS} = F_a * S_s$$

$$S_{M1} = F_v * S_1$$

The following tables were used to calculate F_a and F_v:

Table 11.4-1 Site Coefficient, F_a

	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period				
Site Class	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_S.

Table 11.4-2 Site Coefficient, F_v

	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period				
Site Class	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_1.

It is found by matching and interpolation that:

$$F_a = 1.2$$

$$F_v = 1.65$$

Accordingly; we Found:

$$S_{MS} = 1.2 * 0.75 = 0.9\, rad/sec$$

$$S_{M1} = 1.65 * 0.375 = 0.62\, rad/sec$$

c) Design Spectral Acceleration Parameters:

The design seismic spectral response velocity factor, at S_{DS} and at S_{D1}, can be calculated using Formulae 11.4-3 and 11.4-4.

In relation, ASCE7-16 section 11.4.5 states:

For 10% probability:

$$S_{DS} = S_{MS} = 0.9\, rad/sec$$

$$S_{D1} = S_{M1} = 0.62\, rad/sec$$

Then we calculate:

$$T_0 = 0.2\frac{0.62}{0.9} = 0.138\,sec$$

$$T_S = \frac{0.62}{0.9} = 0.688\,sec$$

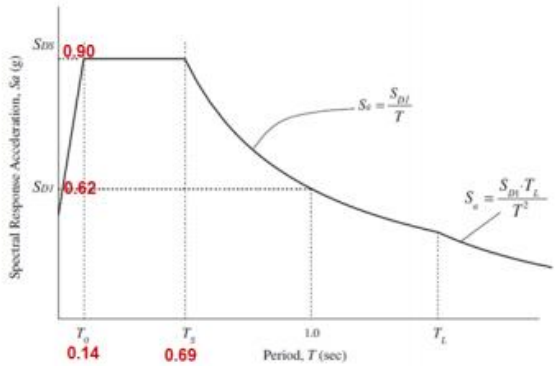


Figure 4.8: Design Response Spectra

d) Risk Category:

Referring to ASCE07-10 Table 1.5-1 the risk category is (III)

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures designated as essential facilities	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

Then, using ASCE7-16 tables (11.6-1 and 11.6-2) and SDS= 0.375 and SD1= 0.619 in the situation that hospitals lay on (III), it is determined that the risk category that shall be (D):

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D
$0.50 \leq S_{DS}$	D	D

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{D1}$	D	D

e) Seismic importance factor;

According to ASCE07-10 table 1.5-2, the seismic value metric will be 1.25, as follows:

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

f) Seismic Design Category:

According to the following table (Table in ASCE 7-10), seismic force resisting systems are classified as "Special Moment Resisting Frames":

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category B	C	D[d]	E[d]	F[e]
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35[h]	NP[h]	NP[h]
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP[i]	NP[i]	NP[i]
5. Special reinforced concrete moment frames[k]	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP

g) Permitted analytical Procedure:

According to table 12.6-1 in ASCE 7-10, all research procedures are allowed, with the exception of the "Equivalent Lateral Force" Technique (see table):

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Procedure, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9.1, or Linear Response History Analysis, Section 12.9.2[a]	Nonlinear Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding two stories above the base	P	P	P
	Structures of light-frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft (48.8 m) in structural height	P	P	P
	Structures exceeding 160 ft (48.8 m) in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft (48.8 m) in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

4.5 SUMMARY OF ALL TYPICAL HOSPITALS

Similar to all the calculations listed in the preceding pages, the table is a list of the standard hospitals showing all the details of the Code of Practice in Palestine and all its categories.

Table 4.10: Summary of All Types Hospitals in Palestine

Region	Qalqilya	Ramallah	Jericho
Soil Bearing Capacity:	4	2.3	0.95
Height from Sea level:	+180	+305	-245
Type of Rock:		Marlstone	
Z	0.15	0.18	0.3
S_s (rad/sec)	0.375	0.45	0.75
S_1 (rad/sec)	0.188	0.225	0.375
Site Class	B	C	D
F_a	1	1.2	1.2
F_V	1	1.53	1.65
S_{MS} $(\boldsymbol{rad/sec})$	0.375	0.54	0.9
$S_{M1}$$(\boldsymbol{rad/sec})$	0.188	0.35	0.62
S_{DS} (rad/sec)	0.375	0.54	0.9
S_{D1}(rad/sec)	0.188	0.35	0.62
T_0 (sec)	0.1	0.128	0.138
T_S (sec)	0.5	0.638	0.688
Importance factor	1.25	1.25	1.25
Risk Category	III	III	III
Response Modification Coefficient (R)	5	8	8
Overstrength Factor Ω_o	3	3	3
Deflection Amplification Factor C_d	4.5	5.5	5.5
ASCE 7-10 table 12.6-1	ELF (NP)	ELF (NP)	ELF (NP)
ASCE 7-10 table 12.2-1	Intermediate MRF	Special MRF	Special MRF

4.6 DESIGN RESPONSE SPECTRA

In quakes design, the response spectrum is a key component. Buildings are directly fed into the dynamic assessment while the connection between both the horizontal designing force factor and periods is formed based on the method. Response spectra can be particularly useful for designers who need to assess how much ground motion an earthquake-damaged structure can handle. Time-history records of ground motion by devices deployed at multiple forums that are activated when detecting the preliminary ground motion of a seismic event are normally available for analysis. When it comes to earthquake ground reaction features, there are always “site impacts” in any earthquake code The simple technique to compensate for site-specific impacts is by employing elasticity response spectra that are tailored to the characteristics of each soil category and earthquake intensity.

The recommended mean reaction acceleration spectra were by each soil group, standardized to the specified response spectra for stone circumstances in Euro code 8. Based on our study and data analysis, we think the site amplifying factors and normalized response spectra give a clearer picture of real site characteristics than do factors like shake strength that account more complex effects on soil non-linearity. Therefore, additional inquiry is necessary to expand a well soil characteristics and powerful records, and the quantity of the soil types, at various soil location circumstances

4.6.1 Response Spectra from Ground Motion Recording

Smooth spectrums may be designed easily, but in a huge range of forms and sizes, the actual world reaction spectra come. PGA and intensity statistics therefore do not provide the complete picture of a seismic event. [48]

The Following chart Summarize the response spectral for the three typical hospitals:

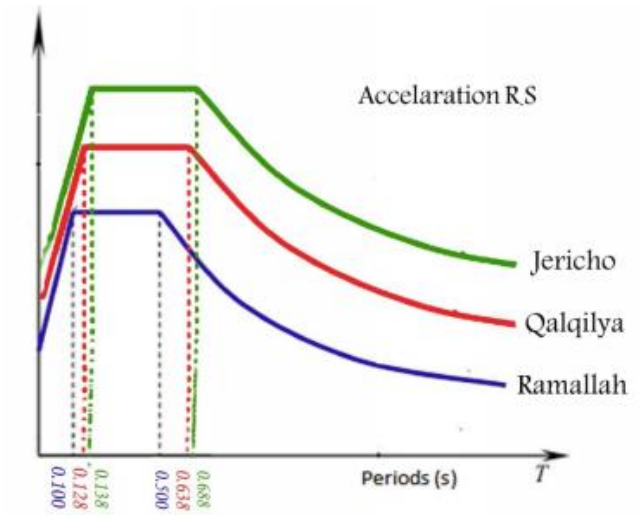


Figure 4.9: Design Response Spectra for Samples of hospitals in Palestine

5. MAIN REQUIREMENTS OF MRF

There are three subcategories of Moment Resisting Frames (MRF): “ductile”, “nominally ductile”, and Gravity Load Designed. MRF structures, which are made up of rigidly linked beams and columns, should be able to withstand extreme earthquakes due to the development of bending moment plastic hinges in the necessary locations of structural elements (beam ends and foundation columns). This is consistent with seismic design theory, which suggests that during seismic activity, structure elements are strained beyond their elastic capacity, resulting in inelastic behavior, which enables the energy caused by earthquake shaking to be dissipated only in tensile elements [49]. Reinforced concrete moment resisting frames (MRF) are employed to resist earthquake loads. In order to withstand seismic forces, various codes describe different types of reinforced concrete moment frames as lateral force resisting structures. ACI 318-05, for example, describes three types of MRF: special MRF (SMRF), intermediate MRF (IMRF), and ordinary MRF (ORF) (OMRF) [50]. As a result, moment frames have frequently been used earthquake resistant system applications cos of their remarkable deformation and energy dissipation characteristics. The timeline is composed of columns and beams that are rigidly connected to one another. In order for the elements of a moment frame to both support gravitational and laterally forces they should be willing to endure those forces. Longitudinal forces were distributed depending on the flex stiffness of every element. Reinforced concrete moment-resisting frame designs have joint areas where beams and columns connect to allow adjoining components to meet their optimum capacity. [51]

MOMENT RESISTING FRAMES OF BUILDING STRUCTURES PROBLEM AREAS

a) Due to inadequate longitudinal reinforcement, members, usually columns, have insufficient flexural strength.
b) Poor anchorage specifics result in insufficient longitudinal reinforcement anchorage.
c) Insufficient transverse reinforcement reduces the ductility and shear strength of prospective plastic hinge areas of beams and columns.
d) Poor anchorage specifics result in insufficient transverse reinforcement anchorage.

e) Due to poor transverse reinforcement, the shear strength of beam-column joints is insufficient.
f) Footings and/or piles and their connections are not strong enough.
g) The presence of non-structural elements (generally curtain walls, infill walls, and partitions) can adversely change the structural response of the frame, causing it to behave in an unexpected way.[52]

In compliance with all previous hospital inspections in Palestine, it is essential to study the essential characteristics of moment resistance frames systems (MRF) as illustrated in the ACI-318-14 code of conduct.

As a result, the parts that follow will go into the basic criteria for hospital systems that should be included during the planning phase of assessing current structures in these buildings.

5.1 ORDINARY MOMENT RESISTING FRAMES (OMRF)

For typical moment-resisting frames, it is common to adopt the strong-column-weak beam idea so that plastic hinging develops in the beams. Since the structure is able to dissipate large power whilst maintaining stability throughout the inelastic region, the frame can become highly effective in energy dissipation. In the scope of such a definition, consistency can be described as the structure's ability to sustain a constant elastic resistance throughout the inelastic limit of reaction.[53] OMRF are considered to have the same structural specifications as all other seismically engineered systems, with the following small differences.

5.2 INTERMEDIATE MOMENT RESISTING FRAMES (MRF)

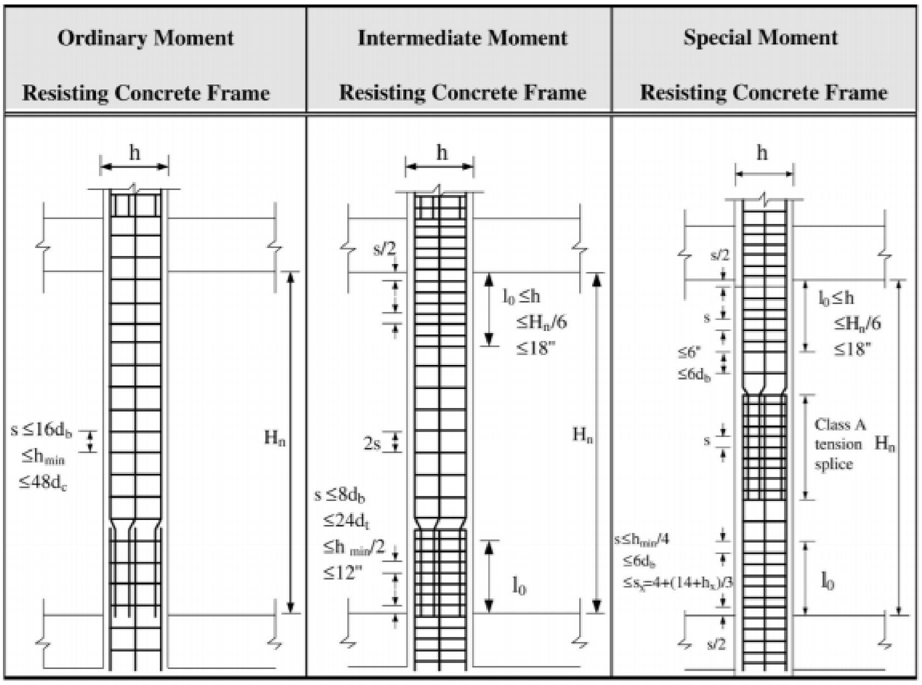
The following are the criteria that should be considered for Intermediate Moment Resisting Frames, which include all ordinary moment resisting frame conditions.

5.3 SPECIAL MOMENT RESISTING FRAMES (SMRF)

This is the structure design that is most popular in contemporary structures: the reinforced concrete, moment-resisting frame (also known as the SMRF). the new architectural regulations for quakes (ACI 318; NZS 3101, 2006; EC 8, 2004) utilize designs to safeguard inhabitants and restrain harm in seismic activity of design intensity and anticipate avoidance of collapse in extreme tremors. The SMRF is defined in ECP201 as a frame with adequate ductility.[54]

The construction protocol for beam-column joints, in general, consists of the following steps:

a) Determine the preliminary size of members based on the longitudinal bars' anchorage requirements.
b) Ensure that the columns have sufficient flexural strength to achieve the desired beam yielding mechanism.
c) Calculate the joint's design shear force by evaluating the neighboring beams' flexural overstrength and the resulting internal forces in the columns that keep the joint in balance.
d) Calculate the effective joint shear region based on the measurements of the adjacent members.
e) Proceed with further inspection to see if the required shear stress is not exceeded. Shear stress is determined by concrete's compressive or diagonal tensile strength, which corresponds to the greatest amount of force which the concrete can tolerate. If you're not satisfied, alter the measurements of the appropriate member.
f) As both confining and shear reinforcement, have transverse reinforcements.
g) Ensure that the reinforcement passing across or ending in the joint has enough anchorage.[55]



d_b, d_t : diameter of longitudinal and transverse bars, s: spacing of lateral bars, h_{min} : minimum dimension of column, s_x :longitudinal spacing of transverse bars within l_o

Figure 5.1: Minimum Reinforcement Details for Columns[56]

The following are the differences between OMRCF and IMRCF and SMRCF:

a) There is no need to meet the strong column–weak beam specifications, which may result to story failure mechanisms during a major earthquake.
b) During a major earthquake, column splices may be positioned just above slabs which are probable to be plastic hinge places.
c) OMRCF's column ties and beam stirrups have significant spacing limits.
d) Internal beam–column joints involve no transverse shear reinforcement, whereas exterior joints need only minimal reinforcement.
e) A beam may be reinforced with discontinuous flexural reinforcement.[57], [58]

During the designed seismic it is envisaged that the SMFs will be able to endure a large inelastic displacement The number of inelastic displacements (deformations that take a constant amount of strain) for IMFs is much lower than for SMFs. While OMFs are supposed to be able to bear relatively minimal displacement inside its elements and connections, OMFs would be less ductile than IMFs, which allows for longer working life spans.

SDCs (ASCE 7 designations) are derived from ASCE 7, and earthquake detailing is divided into three categories: ordinary, intermediate, and special by ACI 318. These classifications have a higher degree of system toughness.

A structure's Earthquake Design Class should rise from A to F to ensure an appropriate level of earthquake performance. This requires a more stringent seismic design and a more ductile system. Wind load would usually control the design of the LRFS for buildings in SDC A and B.

Seismic loads are likely to control design forces in buildings in SDC C, so it is required to perform quake detail. For most systems in this SDC, LRFSs are not height limited, but ASCE drift limits must be met.

Seismic loads almost always control design forces in SDC D, E, and F buildings, requiring further seismic detailing. Maximum height limits in LRFSs are often imposed based on assumed structural performance level behavior. The diagram below (Ali and Moon 2007). Even so, the Asce 7 table below outlines the parameters for choosing a structural system for a particular structure.

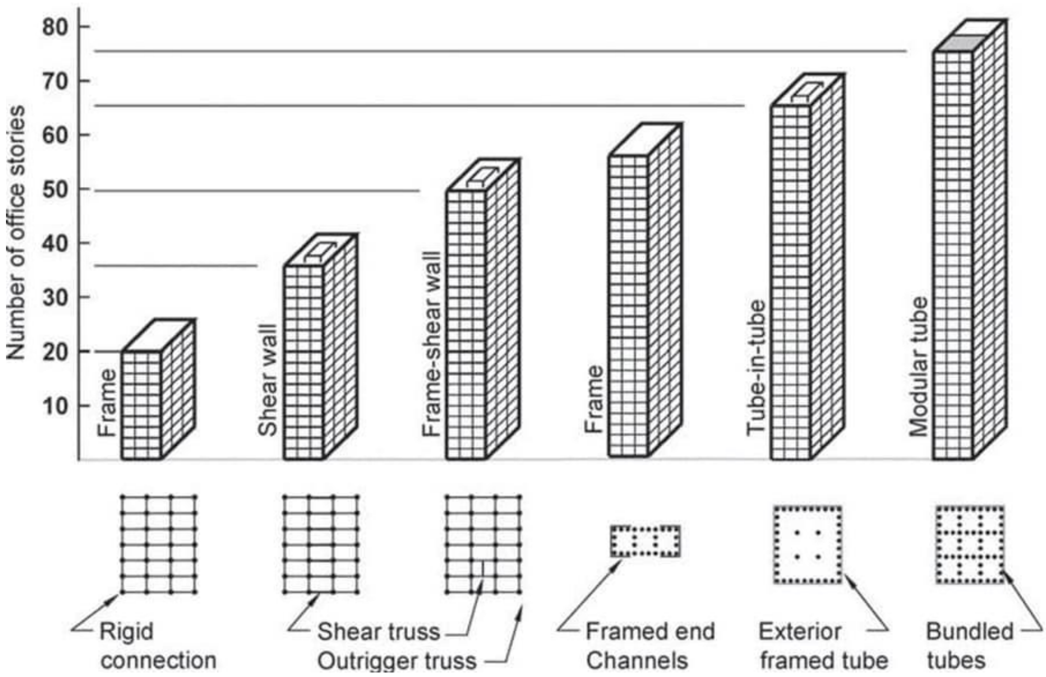


Figure 5.2: Estimated Height Limits for Different Structural System

Type of LFRS	SDC				
	A and B	C	D	E	F
Moment-resisting frames (only):					
Ordinary moment frame (OMF)	NL	NP	NP	NP	NP
Intermediate moment frame (IMF)	NL	NL	NP	NP	NP
Special Moment frame (SMF)	NL	NL	NL	NL	NL
Structural walls (only):					
Building frame systems (structural walls are the primary LFRS and frames are the primary GLRS):					
Ordinary structural wall (OSW)	NL	NL	NP	NP	NP
Special structural wall (SSW)*	NL	NL	160 ft	160 ft	100 ft
Bearing wall systems (structural walls are the primary lateral- and gravity-load-resisting system):					
OSW	NL	NL	NP	NP	NP
SSW*	NL	NL	160 ft	160 ft	100 ft
Dual systems (structural walls are the primary LRFS, and the moment-resisting frames carry at least 25% of the lateral load):					
OSW with OMF	NL	NP	NP	NP	NP
OSW with IMF	NL	NL	NP	NP	NP
OSW with SMF	NL	NL	NP	NP	NP
SSW with OMF	NP	NP	NP	NP	NP
SSW with IMF	NL	NL	160 ft	100 ft	100 ft
SSW with SMF	NL	NL	NL	NL	NL

5.4 ADDITIONAL POINTS TO CONSIDER

During the construction process for hospitals, it is important to weigh the following factors for the optimal safety of the building that can be used as a shelter in critical situations:

5.4.1 Non-Structural Components Effects

Structure and non-structural elements both make up a system (NSC). Because of these main advances in computing scientific knowledge and the use of modern analytical models, it is still common for structural designer to disregard the actions and hardening effects of partitions, exterior, parapet, and stair wells that are directly linked to the building framework

The structural resisting mechanism does not normally include the non-structural building components. As a result, they are not granted enough consideration in structural design procedures. Non-structural components, on the other hand, draw seismic forces and are therefore vulnerable to disruption and loss during earthquakes, resulting in psychological and economic effects in some critical and lifeline systems.[59]

According to the code (IBC-2012 / ASCE 7-10), there are provisions for around five different forms of construction:

a) Buildings,
b) Nonstructural components,
c) Non-building structures,
d) Seismically isolated structures,
e) Damping buildings.

Each of the above groups has its own set of seismic design requirements. There are two subcategories within the nonstructural component's category:

a) Architectural components.
b) Couplings that are mechanical and electrical.

Both equipment and distribution systems fell under the mechanical and electrical component category (R. E. Bachman et all, 2015).

The contribution of two types of non-structural elements, internal full-height partitions and external cladding panels, was investigated in detail.

Another thing to keep in mind is that structures can be categorized as either using a structure system comparable to that of major structures or utilizing a system that is distinctly different from that of major structures

Figure 5-3 (modified from Bachman and Dowty, 2008) depicts certain elements that can be classified as nonstructural components or nonbuilding structures.[60]

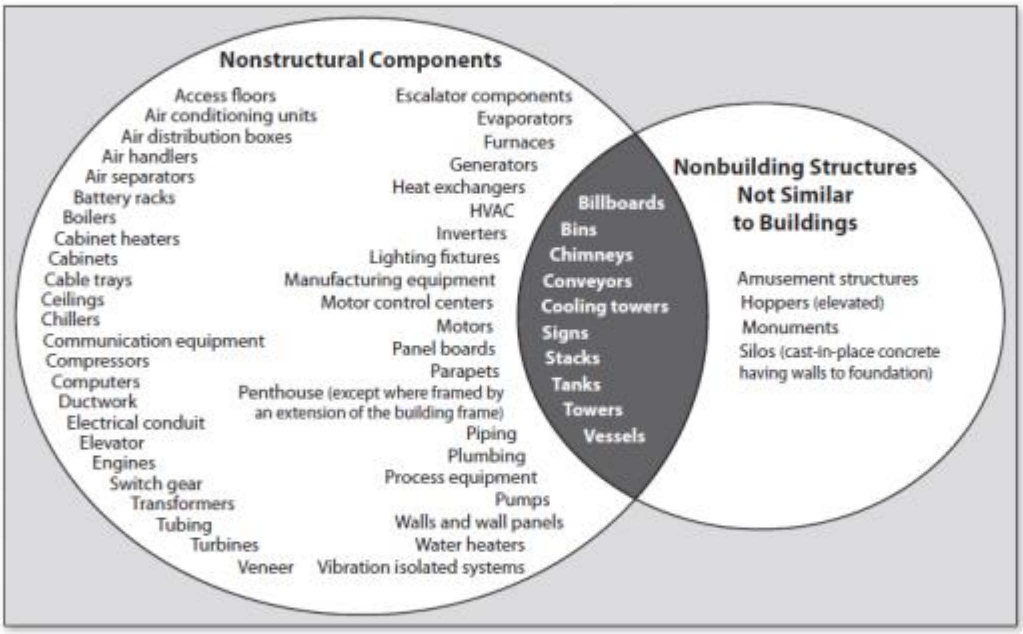


Figure 5.3: Overlapped Of Non-Structural Components And Non-Building Structure[60]

The previous figure indicates that hospitals have many non-structural components; this means that these non-structural components must be secure and have no danger status or effects on hospitals; thus, it is important that these non-structural components do not swing or break down and inflict harm due to the structure's high displacement.

5.4.2 Acceleration and Displacement Effect on Time Period

The adoption of displacement-based design necessitates the development of accurate relative displacement (SD) response spectra for a variety of damping levels and as many response periods as possible. Converting absolute acceleration response spectra (SA) from current seismic design codes would be the easiest way to obtain such spectra.

The next figure shows acceleration response spectra and displacement response spectra, with the aim of demonstrating the direct influence of long or short periods on structure displacement and therefore rigidity. Large displacements are achieved as the time period is increased, indicating that the structure is no longer rigidity

We get simple advantages as we reduce the time period; the first advantage is that we reduce the displacement of the structure.

The influence of these additions is minimally disruptive to the overall movement of the building, although they can lessen the amount of swing.

The next advantage of reducing the structure's time period is that it becomes more rigid, thus reinforcing our hospitals that would be built as shelters. However, it is critical to emphasize that as the time period decreases, care must be taken to stay away from the resonance field, which must be avoided and calculated.[61]

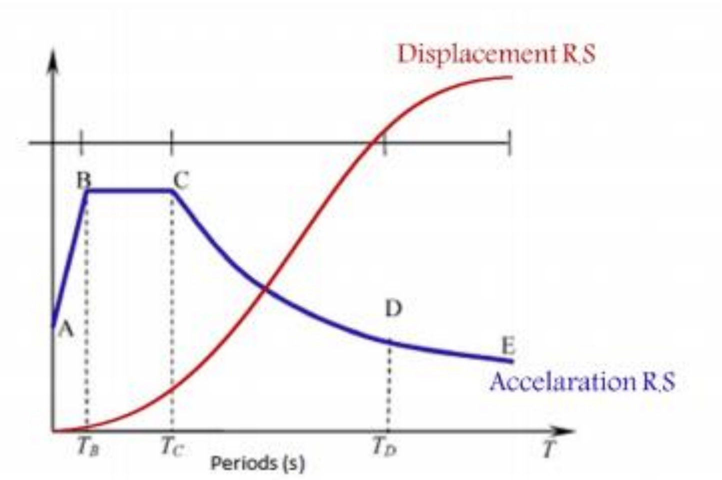


Figure 5.4: Schematic Diagram Joining Acceleration And Displacement Response Spectra Vs Time Period.

In addition, as seen in regions C to E of the previous figure, increasing forces results in high displacement, while decreasing forces results in lower displacement in region AB.

5.4.3 Resonance

All buildings have many natural periods, or resonances, which are (Time Period); described as the amount of seconds it takes for the building to naturally vibrate back and forth. A resonant frequency exists in the ground as well. If the ground motion period matches the normal resonance of a structure, it will experience the most oscillations and suffer the most damage.

A system's normal frequency wn is determined by two main factors: stiffness and mass.$w_n = sqrt(k/m)$ if the normal frequency is w_n.

Where (k) represents stiffness and (m) represents mass. As a result, we must change either k or m, or both, in order to change the normal frequency.

Most anticipated vibrating frequency are typically raised to a frequency that is greater than their normal frequency If the normal frequency is greater or less than any projected vibrating frequency therefore the resonant is unlikely to be stimulated. To avoid unwanted resonant, any structure redesigns are founded on this concept

The following principles should be used in practice to change a normal frequency and reduce a system's vibration response:

a) Increasing the normal frequency by adding stiffness.
b) Increasing mass throughout this way causes the frequency to decrease.
c) Reduced dampening raises the maximum amplitude but also increases the amplitude range.
d) Decreasing forcing amplitudes lessens the magnitude of reaction at resonant.
e) When reducing damping, the reaction range is constricted while the maximum response is amplified.

It is necessary to adequately describe the systems before doing any structural redesign procedures. This would be an excellent concept if it's possible to add heft to the structural framework and then adjust the typical frequency lower to a frequency that is less strenuous.

5.4.4 Ribbed Slabs Issue

It is shown that hollow block of ribbed slabs structures is commonly used in Palestine.

Solid slab elements are used to model the continuous top part of the ribbed slab, while an equivalent beam element with uniaxial properties is used to model the lower (rib) part. The orthotropic nature of ribbed slabs, as well as their membrane behavior, was logically considered. This method may also be used to model deep-deck slabs.

This method relies on hollow blocks with a strength of around 35kg/cm2 that are self-suspended by skin friction with slab ribs without the use of any fixation equipment between the reinforced concrete ribs and the hollow blocks.

This issue draws attention to the importance of important buildings such as hospitals not using dangerous materials that might fall, such as the blocks in ribbed slab systems.

5.4.5 Topography Amplification on Earthquakes

While topographic effects on ground motion have been documented for decades, for this reason, contemporary ground motion prediction equations ignore them (GMPEs). The diffraction and reflection of incident seismic waves are affected by topography, which amplifies or de-amplifies the seismic response. Barani et al. recently established a method for incorporating topography effects into probabilistic seismic hazard analysis (PSHA) (2012). Furthermore, topographical effects were taken into account in the Southern California Earthquake Center's simulation-based seismic hazard tests [62]. After analyzing data gathered after seismic activity, it was discovered that the data acquired at various locations varied and was different. A shift in ground motion is made up of several effects, such as travel waves absorption, loss of coherence, as well as the regional site impact. One of the most significant influencing factors of a location is the topography of the site. The findings indicate that topographical amplifying has a significant effect on earth shaking caused by seismic incidents. The magnitude and regularity of ground motion are heavily influenced by the existence of a high topography elevation (hills, ridges, canyons, or slopes). Comment seismic analysis found that structures at the uppermost points of hills, ridges, and canyons are damaged more severely following a seismic event than those situated at the bottom. Many noteworthy occurrences were recorded during the San Fernando seismic in 1971, the Celebi seismic in 1987, the Chile seismic in 1985, and the Whittier Narrows seismic in 1987. (Armenia Earthquake, 1998). this recent seismic in Boumerdes, Algeria (which occurred in 2003) gave new proof of the catastrophic structural damages that takes place when buildings are erected on hilltops (Corso city).

6. CONCLUSIONS

Based on all previous discussions and analysis, as well as all criteria learned from practice, it is critical to summarize the findings in order to shape the first step in developing our hospitals, which require a high level of care for all of the reasons stated, particularly protecting our future and keeping up with the advancement of society components. This section offers significant findings that should be considered throughout hospital design procedures in Palestine, and it is hoped that it will serve as a first step toward the development of a code of practice for hospital design. The non-linear static analysis of the models evaluated under the ETABS program and the comparison of response variables values with allowable values under international codes conclude:

a) Pushover analysis can be a useful method for the performance analysis of earthquake-prone structures.
b) Seismic demand, power and plastic connections provide insight into the real behavior of a structure.
c) Pushover analysis is a valuable method to evaluate inelastic strength and deformation needs and to detect weaknesses in design. The pushover analysis is a reasonably easy approach of investigating the structure's nonlinear behavior.
d) The desired displacement is estimated based on the low level of seismicity of Thane and the structural characteristics and utilizing ASCE 41-06.
e) The pushover study indicates that the structure is stable, and hence that the building requires minor modifications when loading it to the estimated base shear and limiting the displacement of the control node.

6.1 DISCUSSION

a) The pushover analysis was an appropriate method for examining the structure's non-linear behavior and for evaluating the demands for inelastic strength and deformation and for finding weakness in design.
b) Method of displacement target displacement is computed using displacement coefficient.
c) No failure occurred in the structure examined at the target displacement limit.

6.2 FUTURE SCOPE OF STUDY

a) The addition of shear failure limitations in the performance measures can lead to a better knowledge of the behavior of the building.

b) Non-linear time history analysis may be used to examine the structure's capacity more specifically and to comprehend a more realistic scenario of demand for more parts of the structures.

REFERENCES

[1] D. C. Rai, V. Singhal, B. Raj S, and S. L. Sagar, “Reconnaissance of the effects of the M7.8 Gorkha (Nepal) earthquake of April 25, 2015,” *Geomatics, Natural Hazards and Risk*, vol. 7, no. 1. Taylor and Francis Ltd., pp. 1–17, Jan. 02, 2016. doi: 10.1080/19475705.2015.1084955.

[2] M. Tanganelli, S. Viti, V. Mariani, and M. Pianigiani, “Seismic assessment of existing RC buildings under alternative ground motion ensembles compatible to EC8 and NTC 2008,” *Bulletin of Earthquake Engineering*, vol. 15, no. 4, pp. 1375–1396, Apr. 2017, doi: 10.1007/s10518-016-0028-z.

[3] M. Pianigiani and V. Mariani, “Sensitivity study on the discretionary numerical model assumptions in the seismic assessment of existing buildings,” *Soil Dynamics and Earthquake Engineering*, vol. 98, pp. 155–165, Jul. 2017, doi: 10.1016/j.soildyn.2017.04.001.

[4] Q. Shang, T. Wang, and J. Li, "A Quantitative Framework to Evaluate the Seismic Resilience of Hospital Systems," *Journal of Earthquake Engineering*, pp. 1-25, 2020 , doi: 10.1080/13632469.2020.1802371

[5] N. Caterino, B. M. Azmoodeh, and G. Manfredi, "Seismic Risk Mitigation for a Portfolio of Reinforced Concrete Frame Buildings through Optimal Allocation of a Limited Budget," *Advances in Civil Engineering*,vol.2018, pp. 1-18, 2018., doi: 10.1155/2018/8184756.

[6] R. PARK, "A static force-based procedure for the seismic assessment of existing reinforced concrete moment resisting frames," *Bulletin of the New Zealand Society for Earthquake Engineering*, (30)3, 1997.

[7] M. J. DEJONG, "Seismic assessment strategies for masonry structures.,"Ph.D. Building Technology, Massachusetts Institute of Technology, 2009

[8] H. Bilgin, “Fragility-based assessment of public buildings in Turkey,” *Engineering Structures*, vol. 56, pp. 1283–1294, Nov. 2013, doi: 10.1016/j.engstruct.2013.07.002.

[9] T. K. Datta, Seismic analysis of structures, delhi,india : John Wiley & Sons, 2010.

[10] R. L. Y. a. A. B. Rincón, "Seismic risk assessment of public schools and prioritization strategy for risk mitigation," Proceeding of 16th World Conference on Earthquake, Santiago Chile, Jan, 2017.

[11] T. a. M. N. P. Paulay, Seismic design of reinforced concrete and masonry buildings, USA, 1992, pp. 135-146.

[12] M. V. D. K. M. G. L. N. &. P. Fragiadakis, "Seismic assessment of structures and lifelines," *Journal of Sound and Vibration,* vol. 334, pp. 29-56, 2015.

[13] G. G. DEIERLEIN and A. M. REINHORN, "Nonlinear structural analysis for seismic design.," *NEHRP seismic design technical brief,* vol. 4, pp. 1-36, 2010.

[14] F. A. Najam, "Nonlinear Static Analysis Procedures for Seismic Performance Evaluation of Existing Buildings – Evolution and Issues," in *Sustainable Civil Infrastructures*, 2018, pp. 180–198. doi: 10.1007/978-3-319-61914-9_15.

[15] P. F. L. a. P. F. Croce, "Probabilistic seismic assessment of existing masonry buildings," *Buildings,* vol. 9(12), pp. 1-20, 2019. doi: 10.3390/buildings9120237.

[16] M. a. A. M. Ferraioli, "Base isolation for seismic retrofitting of a multiple building structure: design, construction, and assessment," *Mathematical Problems in Engineering,* vol. 2017 , pp. 1-25, 2017, doi:10.1155/2017/4645834.

[17] U. C. V. S. A. A. D. M. G. &. E. Vitiello, "BIM-based approach for the cost-optimization of seismic retrofit strategies on existing buildings," *Automation in Construction,* vol. 98, pp. 90-101, 2019, doi: 10.1016/j.autcon.2018.10.023.

[18] R. a. G. M. Fenwick, "Comparison of New Zealand Standards used for seismic design of concrete buildings," *Bulletin of the New Zealand Society for Earthquake Engineering* , vol. 42(3), pp. 187-203, 2009.

[19] U. S. A. A. D. D. L. M. &. P. A. Vitiello, "Life-cycle assessment of seismic retrofit strategies applied to existing building structures," *Sustainability,* vol. 8(12), pp. 1-18, 2016, doi: 10.3390/su8121275.

[20] N. Caterino, I. Iervolino, G. Manfredi, and E. Cosenza, “Multi-criteria decision making for seismic retrofitting of RC structures,” *Journal of Earthquake Engineering*, vol. 12, no. 4, pp. 555–583, May 2008, doi: 10.1080/13632460701572872.

[21] G. E. Thermou and A. S. Elnashai, “Seismic retrofit schemes for RC structures and local-global consequences,” *Progress in Structural Engineering and Materials*, vol. 8, no. 1, pp. 1–15, 2006. doi: 10.1002/pse.208.

[22] A. Belleri, M. Torquati, P. Riva, and R. Nascimbene, “Vulnerability assessment and retrofit solutions of precast industrial structures,” *Earthquake and Structures*, vol. 8, no. 3, pp. 801–820, 2015, doi: 10.12989/eas.2015.8.3.801.

[23] D. Perrone, M. A. Aiello, M. Pecce, and F. Rossi, “Rapid visual screening for seismic evaluation of RC hospital buildings,” *Structures*, vol. 3, pp. 57–70, Aug. 2015, doi: 10.1016/j.istruc.2015.03.002.

[24] T.-H. Nguyen, "Integrating building code compliance checking into a 3D CAD system," Computing in Civil Engineering, montreal,canda, 2005.

[25] M. M. Hirschler, "Façade requirements in the 2021 edition of the US International Building Code," *Fire and Materials,* vol. 45(5), pp. 586-597, 2020, doi: 10.1002/fam.2803.

[26] S. K. a. S. D. Ghosh, "The Seismic Design Provisions of the 2003 Edition of the International Building Code," *PCI journal,* vol. 48(3), pp. 92-95, 2003.

[27] V. N. Khose, Y. Singh, and D. H. Lang, “A comparative study of design base shear for RC buildings in selected seismic design codes,” *Earthquake Spectra*, vol. 28, no. 3, pp. 1047–1070, 2012, doi: 10.1193/1.4000057.

[28] V. N. Khose, Y. Singh, and D. H. Lang, “A comparative study of design base shear for RC buildings in selected seismic design codes,” *Earthquake Spectra*, vol. 28, no. 3, pp. 1047–1070, 2012, doi: 10.1193/1.4000057.

[29] D. a. S. M. Dohare, "Seismic behavior of soft storey building: a critical review," *International Journal of Engineering Research and General Science,* vol. 2(6), pp. 1-5, 2014.

[30] J. Ruiz-García, “Mainshock-aftershock ground motion features and their influence in building’s seismic response,” *Journal of Earthquake Engineering*, vol. 16, no. 5, pp. 719–737, Jul. 2012, doi: 10.1080/13632469.2012.663154.

[31] A. Nour, A. Slimani, N. Laouami, and H. Afra, “Finite element model for the probabilistic seismic response of heterogeneous soil profile,” *Soil Dynamics and Earthquake Engineering*, vol. 23, no. 5, pp. 331–348, 2003, doi: 10.1016/S0267-7261(03)00036-8.

[32] C. H. Zhai and L. L. Xie, “A new approach of selecting real input ground motions for seismic design: The most unfavourable real seismic design ground motions,” *Earthquake Engineering and Structural Dynamics*, vol. 36, no. 8, pp. 1009–1027, Jul. 2007, doi: 10.1002/eqe.669.

[33] J. H. Wood, "Earthquake-induced soil pressures on structures," Ph.D. California Institute of Technology, California, 1973

[34] M. Inel, S. M. Senel, S. Toprak, and Y. Manav, “Seismic risk assessment of buildings in urban areas: A case study for Denizli, Turkey,” *Natural Hazards*, vol. 46, no. 3, pp. 265–285, Sep. 2008, doi: 10.1007/s11069-007-9187-1.

[35] D. a. A. A. Ajema, "Cost Comparison between Frames with Solid Slab and Ribbed Slab using HCB under Seismic Loading," *International Research Journal of Engineering and Technology*, vol. 9001, pp. 109-116, 2017

[36] A. Ghobarah, "On drift limits associated with different damage levels," *International workshop on performance-based seismic design*, vol. 28, pp. 1-13, 2004.

[37] B. Shafei, F. Zareian, and D. G. Lignos, “A simplified method for collapse capacity assessment of moment-resisting frame and shear wall structural systems,” *Engineering Structures*, vol. 33, no. 4, pp. 1107–1116, Apr. 2011, doi: 10.1016/j.engstruct.2010.12.028.

[38] A. Al-Balhawi and B. Zhang, “Investigations of elastic vibration periods of reinforced concrete moment-resisting frame systems with various infill walls,” *Engineering Structures*, vol. 151, pp. 173–187, Nov. 2017, doi: 10.1016/j.engstruct.2017.08.016.

[39] E. Nastri, R. Montuori, and V. Piluso, “Seismic design of MRF-EBF dual systems with vertical links: EC8 vs plastic design,” *Journal of Earthquake Engineering*, vol. 19, no. 3, pp. 480–504, Apr. 2015, doi: 10.1080/13632469.2014.978917.

[40] A. M. Ibrahim, M. A. Ismael, and H. A. A. Hussein, “Effect of Construction Type on Structural Behaviour of R.C Bubbled One-Way Slab,” *Diyala Diyala Journal of Engineering Sciences*, vol. 12, no. 01, pp. 73–79, 2019, doi: 10.26367/DJES/VOL.12/NO.1/9.

[41] M. Al-Rubaye, "Flexural behaviour of concrete slabs reinforced with GFRP bars and hollow composite reinforcing systems." M.S. thesis, Dept. Civil. Eng, University of Southern Queensland, Toowoomba, Australia, 2018.

[42] A. R. J. E. N. A. F. A. &. M. M. Jarvis, "Practical use of SRTM data in the tropics: Comparisons with digital elevation models generated cartographic data," International Centre for Tropical Agriculture, Cali, Colombia, 2004.

[43] R. Z. U. &. S. J. Sørensen, "On the calculation of the topographic wetness index: evaluation of different methods based on field observations," *Hydrology and Earth System Sciences Discussions,* vol. 2(4), pp. 1807-1834, 2005.

[44] A. J. D. a. F. C. Lemoine, "Testing the applicability of correlations between topographic slope and VS 30 for Europe," *Bulletin of the Seismological Society of America,* vol. 102(6), pp. 2585-2599, 2012

[45] A. I. T. H. F. &. A. K. A. Sale, "Structural Design of Al-Quds Open University in Salfi," Nablus, 2015.

[46] A. Doğangü and R. Livaoğlu, “A comparative study of the design spectra defined by Eurocode 8, UBC, IBC and Turkish Earthquake Code on R/C sample buildings,” *Journal of Seismology*, vol. 10, no. 3, pp. 335–351, 2006, doi: 10.1007/s10950-006-9020-4.

[47] Pitilakis, K., Gazepis, C., & Anastasiadis, A, “Design response spectra and soil classification for seismic code provisions.” In: *Proceedings of the 13th world conference on earthquake engineering, Vancouver, BC, Canada*. 2004.

[48] S. A. Freeman, "Response spectra as a useful design and analysis tool for practicing structural engineers.," *ISET Journal of Earthquake Technology,* vol. 44(475), pp. 25-37, 2007.

[49] B. F. A. V. G. &. M. F. M. Faggiano, "Numerical study on moment resisting frames under monotonic and cyclic loads," *Key Engineering Materials.*, vol. 763, pp. 625-632, 2018.

[50] H. Daneshvar, "An Investigation Into Ductility Definitions For Reinforced Concrete Members And Frames," Dept. of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada , 2010.

[51] S. R. a. A. M. P. Uma, "Seismic behavior of beam column joints in reinforced concrete moment resisting frames," Department of Civil Engineering, Indian Institute of Technology Madras, Chennai, India, 1996.

[52] R. Park, "A static force-based procedure for the seismic assessment of existing reinforced concrete moment resisting frames.," *Bulletin of the New Zealand Society for Earthquake Engineering*, vol. 30(3), pp. 213-226, 1997.

[53] S. S. a. H. R. R. Mahini, "A new method for improving ductility in existing RC ordinary moment resisting frames using FRPs.," *BUILDING AND HOUSING,* Vols. VOL. 8, NO. 6, pp. 581-595, 2007.

[54] M. Rashid and N. Ahmad, "Economic losses due to earthquake – induced structural damages in RC SMRF structures," *Cogent Engineering*, vol. 4, no. 1, pp. 1-15, 2017, doi: 10.1080/23311916.2017.1296529.

[55] S. R. a. S. K. J. Uma, "Seismic design of beam-column joints in RC moment resisting frames-Review of codes," *Structural Engineering and mechanics*, vol. 23, no. 5, pp. 579-597, 2006.

[56] S. W. Han and N. Y. Jee, "Seismic behaviors of columns in ordinary and intermediate moment resisting concrete frames," *Engineering Structures*, vol. 27, no. 6, pp. 951–962, May 2005, doi: 10.1016/j.engstruct.2005.01.012.

[57] S. W. Han, O. S. Kwon, and L. H. Lee, "Evaluation of the seismic performance of a three-story ordinary moment-resisting concrete frame," *Earthquake Engineering and Structural Dynamics*, vol. 33, no. 6, pp. 669–685, 2004, doi: 10.1002/eqe.367.

[58] J. P. J. D. H. a. C. D. L. Moehle, "Seismic design of reinforced concrete special moment frames," U.S. Department of CommerceBuilding and Fire Research LaboratoryNational Institute of Standards and TechnologyGaithersburg, MD 20899-8600, USA, 2008.

[59] R. K. L. C. A. M. S. M. N. &. L. N. T. K. Su, "Influence of non-structural components on lateral stiffness of tall buildings," *The Structural Design of Tall and Special Buildings*, vol. 14, no. 2, pp. 143-164, 2005.

[60] J. A. &. B. R. E. Gatscher, "Elements of 2012 IBC/ASCE 7-10 nonstructural seismic provisions: Bridging the implementation gap," The 15th World Conference on Earthquake Engineering., Lisbon, 2012.

[61] J. J. A. S. E. a. A. G. W. Bommer, "Compatible acceleration and displacement spectra for seismic design codes," Proceedings of the 12th World Conference on Earthquake Engineering, london, 2000.

[62] S. Barani, D. Spallarossa, P. Bazzurro, and F. Pelli, “The multiple facets of probabilistic seismic hazard analysis: A review of probabilistic approaches to the assessment of the different hazards caused by earthquakes,” in *Bollettino di Geofisica Teorica ed Applicata*, 2014, vol. 55, no. 1, pp. 17–40. doi: 10.4430/bgta0098.

[63] A. Bounif *et al.*, “The 21 May 2003 Zemmouri (Algeria) earthquake Mw 6.8: Relocation and aftershock sequence analysis,” *Geophysical Research Letters*, vol. 31, no. 19, Oct. 2004, doi: 10.1029/2004GL020586.

APPENDIX A

BEAMS OF SPECIAL MOMENT FRAMES

A.1 DIMENSIONAL LIMITS

According to ACI318-14 item 18.6.2.1, the dimensions of beams in special moment resisting frames must meet all of the following requirements:

a) The clear beam span l_n must be at least $4d$.
b) The width of the beams, $b_w$which are equal to the lesser of 0.3h and 250 mm, cannot be less than that value.
c) While each beam must have a clearance on either side equal to half of the larger of c1 and c2, its projection beyond that distance should not be greater than the lesser of those two values, as shown in the following diagram.:

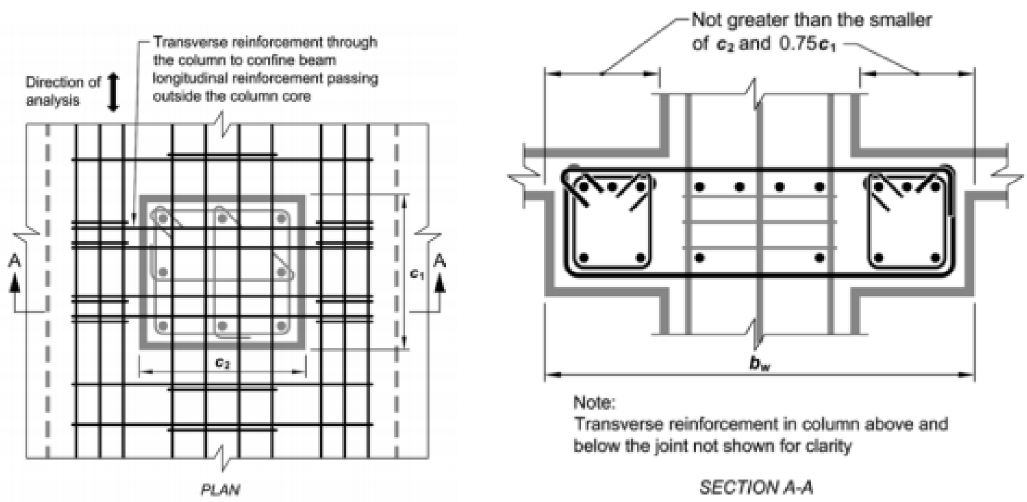


Figure A.1: ACI318-14 Beams Parameters

Commentary:

Regarding the dimensions of the beams, it is stated as follows:

a) It is preferred to use clear spans of not less than $220cm$ when using beams with a height of not less than $60cm$ $(d = 55cm)$.
b) In practice, beams with a height of $60 - 100\ cm$ are recommended, so bw should be in the range of $25 - 30cm$.

c) In the case of the column conditions discussed later, it is suggested to use beam projections of no more than $300mm$ when using columns with short dimensions of no less than $300mm$.

A.2 LONGITUDINAL OF REINFORCEMENT

Bar beams there are at least two consecutive bars on the upper and lower sides of the beam. The sum of top-bottom reinforcement needed by 9.6.1.2 (minimum reinforcement requirements) at each segment may not be less than the requirements listed in 9.6.1.2 (minimum reinforcement requirements), The steel reinforcement grade must not above 0.025 for this structure.

Commentary:

a) As beams of around (30x60xm) dimensions are widely used, the amount of reinforcement does not increase 45cm2 in any case (no more than 0.025).

b) Pursuant to ACI 318-14, paragraph 18.6.3.2, at the joints surface, positively moment strength shall be equal to or greater than negatively moment strength. This law states that both the positively and negatively moment strength at every position throughout the beam has to be at minimum one-fourth of the complete moment strength that exists at each joint.

Commentary:

a) This item will be covered by using the smallest amount of steel in both the positive and negative moments areas.

They may be used if a hoop bar or circular reinforcement has been placed throughout the lap's lengths, as stated in item 18.6.3.3.

The transverse reinforcing enclosing the lap-spliced bars must be spaced no more than $d/4$ and 100 mm apart.

Some other situations where lapping splices were not authorized to include:

a) above and around the joint

b) From within twice the depth of beam of the joint surface.

c) Alongside crucial sections, approximately twice the beam depth away. where lateral displacements outside the elastic range of behavior are likely to cause flexural yielding.

Commentary:

a) Because of the use of a (d) of around $55cm$, the spacing of the transverse reinforcement will still be governed by spacing no more than 10cm in operation as seen in public hospitals.
b) Splicing is not permitted in the region of positive moments in joints.
c) Splicing is not permitted in beams with a length of less than $250cm$.
d) Hoops with a spacing of $100\ mm$ must be used for all splicing. Splicing shall only be used in the center of long beams of more than $250\ cm$ in length.

A.3 TRANSVERSE REINFORCEMENT

Hoops must be given according to the below beam areas, according to ACI318-14 item 18.6.4.1:

a) Over double the beam depths (from the surface of the support column towards to the middle span distance each extremity of the beam is connected to another member over a duration approximately twice the beam depth.
b) Laterally displacements exceeding the elastic limit of action at spans equivalent as double beam depth are likely to happen when beams are subjected to flex yield.

Commentary:

a) Combining the two cases (a+b), hoops must cover the whole portion with small beams less than 250 cm in length.

Transversely supported flexural supporting bars must be spaced no more than 350 mm apart.

Table 24.3.2 - Maximum spacing of bonded reinforcement in non-prestressed and Class C prestressed one - way slabs and beams

Table 24.3.2 from ACI38-14 for $Smax$ - Maximum spacing of bonded reinforcement in non-prestressed and Class C prestressed one - way slabs and beam

Table A.1: Table 24.3.2 From ACI38-14 for $Smax$

Reinforcement type	Maximum spacing s	
Deformed bars or wires	Lesser of	$380\left(\frac{280}{f_s}\right)-2.5c_c$
		$300\left(\frac{280}{f_s}\right)$
Bonded prestressed Reinforcement	Lesser of	$\left(\frac{2}{3}\right)\left[380\left(\frac{280}{\Delta f_{ps}}\right)-2.5c_c\right]$
		$\left(\frac{2}{3}\right)\left[300\left(\frac{280}{\Delta f_{ps}}\right)\right]$
Combined deformed bars or wires and bonded prestressed reinforcement	Lesser of	$\left(\frac{5}{6}\right)\left[380\left(\frac{280}{\Delta f_{ps}}\right)-2.5c_s\right]$
		$\left(\frac{5}{6}\right)\left[300\left(\frac{280}{\Delta f_{ps}}\right)\right]$

Commentary: So, in hospitals based on common practice (using Cc = 25 mm, F_y = 420 Mpa).

$$S_{max} = lesser\ of \left\{\begin{matrix} 380\left[\frac{280}{\left(\frac{2}{3}F_y\right)}\right]-2.5C_C = 380\left[\frac{280}{\left(\frac{2}{3}*420\right)}\right]-2.5*25 = 317.5mm \\ \left[300\left(\frac{280}{\frac{2}{3}F_y}\right)\right] = \left[300\left(\frac{280}{\frac{2}{3}*420}\right)\right] = 380mm \end{matrix}\right\} \quad \ldots..(1)$$

So, it is recommended using maximum value of spacing (s) no greater than 30 cm in any case.

According to item 18.6.4.6, stirrups containing seismic hooks shall be spread out at a maximum value from $d/2$ along the beam's length where hoops are not needed.

The following figures are taken from the ACI-318 code that displays the above item:

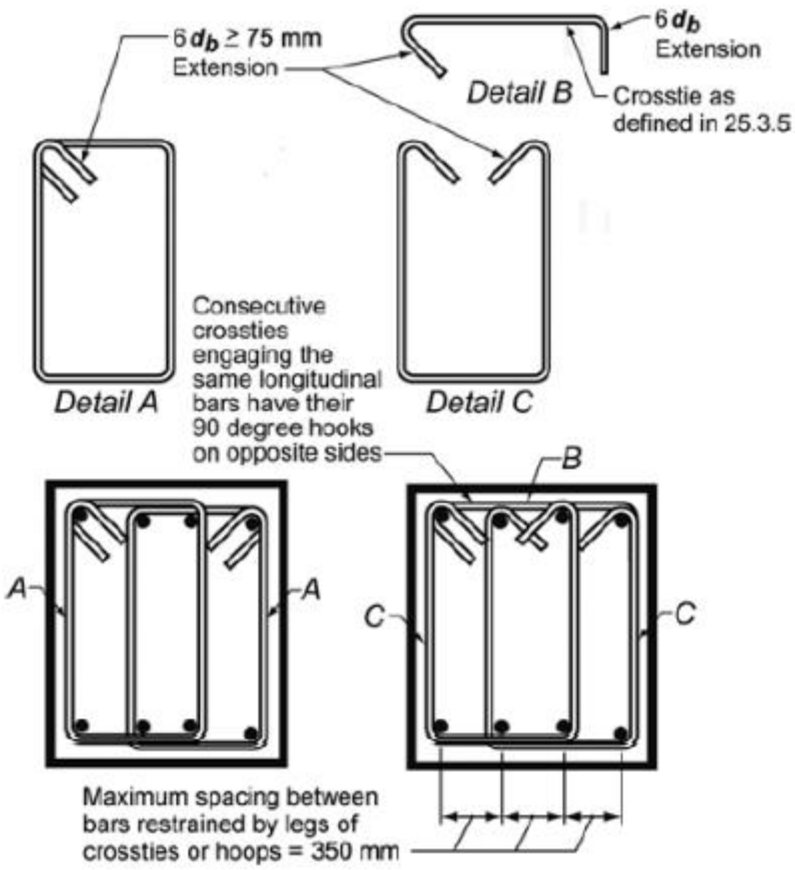


Figure A.2: ACI318-14/Fig R18.6.4 Examples of Overlapping Hoops

Commentary:

This item is stressed on the basis of the local practice criteria to include alternatives to those areas that have difficulty repairing stirrups according to requirements.

APPENDIX B

COLUMNS OF SPECIAL MOMENT FRAMES

B.1 DIMENSIONAL LIMITS

Columns must satisfy (a) and (b) in accordance with item 18.7.2.1.

a) It is necessary that the smallest cross-sectional dimensions would have been at minimum 300 mm along a horizontal line drawn from around geometrical center.
b) The ratios of the smallest horizontal length to the cross-sectional length should be more exceed 0.4.

Commentary:

a) Since it is not permitted to use columns with short dimensions of less than 300mm, it is recommended that the dimensions of the columns, beams, and joints be consistent with all specifications so that the installation can be completed effectively.
b) The best measurements to use in circular or square sections; otherwise, the dimensions drawbacks shall be as shown in the table below:

Table B.1: Columns Dimensions Limitation in SMRF

	Dimension of short direction of column	Maximum allowable dimension of long direction of column
1	300 mm	750 mm
2	350 mm	875 mm
3	400 mm	1000 mm
4	450 mm	1125 mm

B.2 TRANSVERSE REINFORCEMENT

For segments of the column where deformations are anticipated to happen because of elasticity behavior beyond the elasticity spectrum, longitudinal reinforcement is required, as stated in item 18.7.5.1.

Also; Length l_o at minimum the largest of a following must be provided:

a) When column flex yielding is expected, use columns with a joints surface depth that is deeper than usual.
b) 450 mm
c) Only one sixteenth of the clear span of the column was visible.

Commentary:

Because $30x60\ cm$ columns are often used, and the total height of hospital columns is approximately $240\ cm$, lo must be no less than $60cm$ from the joint face.

According to item 18.7.5.2, transverse reinforcement must meet the following requirements:

a) At least singular or overlapped circles, circular rings, or rectangular shapes hoops even without crossties should be used to provide longitudinal reinforcing.
b) rectangular of hoops and cross ties must bend to engage longitudinal strengthening bars on the outside.
c) According to the limits specified in 25.7.2.2, it is permissible to use bars of the equal or lower bar dimension as the hoop, as seen below.
    a) The tie bar diameter must be (a) and (b):

    (a)- No. 10 containing bars lower than No. 32 transverse are excluded.

    (b)- The specifications for no. 13 allow the use of no. 36 or bigger longitudinally bars or bundle longitudinally bars

    b) 25.7.2.2.1. Also known as reinforcing bars support, distorted bars and weir as a substitute to warped bars required as much space as warped bars and weir (per 25.7.2.1). The standards for table 20.2.2.4a apply.

End to end around the longitudinal reinforcement and along the circumference of the cross section, successive crossties must be alternated.

d) Rectangular hoops or crossties should offer horizontal reinforcement to longitudinally reinforcing as per the requirements found in 25.7.2.2 and 25.7.2.3.

e) It is ensured that horizontal bars with lateral protection by a cross-tied or hoop-legged crosspiece do not have a gap of more than 350 mm along the circle of the column.

f) The value of (hx) in columns of rectangular hoop must not reach 200 mm when P<0.3Agfc' or fc' > 70 MPa. Able to factor load configurations in which P u is the max should provide for the highest possible value of Pu.

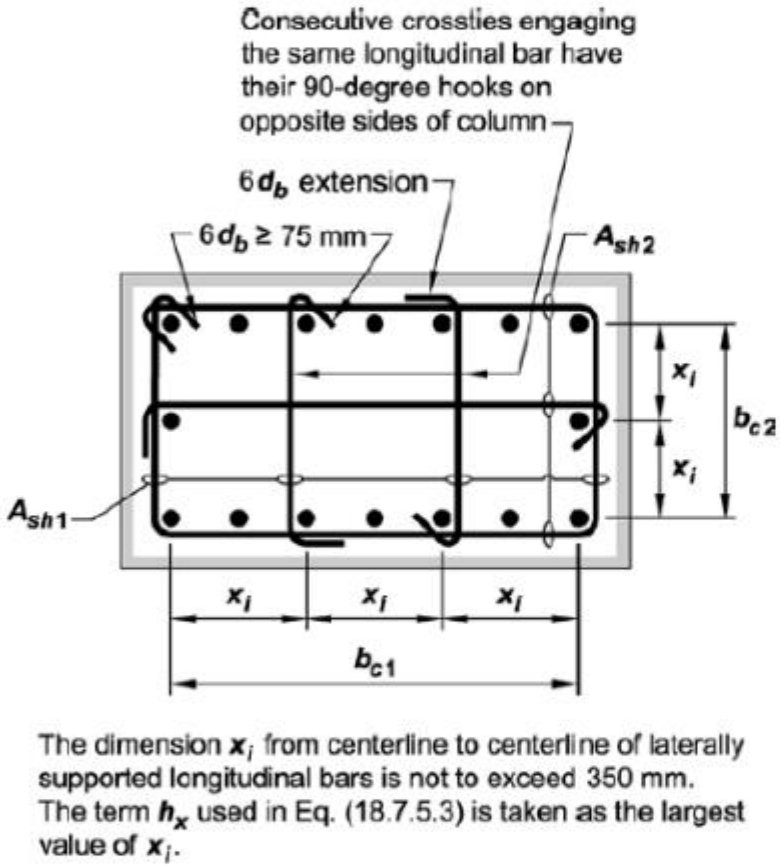


Figure B.1: ACI318-14/Fig R18.7.5.2 Examples Of Transverse Reinforcement In Columns

Commentary:

This item is shown to illustrate the necessary technique for transverse column reinforcement that is simple to enforce.

According to item 18.7.5.3, transverse reinforcement spacing must be no more than the smallest of the ability to follow:

a) The dimensions of one-fourth of the lowest column is used.
b) Six times the length of the lowest bar along the length of the bar.
c) S_o, as calculated by

$$S_o = 100 + (350 - h_x/3) \qquad (18.7.5.3)$$

As long as the length from Equation (18.7.5.3) is between 150 mm and 100 mm, the value of the quantity must not above 150 mm and it should not be taken any lower than 100 mm.

Commentary:

As a result of the above circumstances, the following table summarizes the real-life situation of hospitals as practice parameters:

Table B.2: Transverse Reinforcement Spacing in SMRF

	For Spacing so	Subs.	Result
a	(Minimum Column Dimension)/4	300/4	75 mm
b	6 x (smallest bar diameter)	6x14	84 mm
c	$S_o = 100 + \left(\frac{350 - h_x}{3}\right)$ Where; hx: the largest value of hi shown in figure 5-8.	$S_o = 100 + \left(\frac{350 - 200}{3}\right)$	150 mm

As seen in the last table, a transverse reinforcement spacing of no more than 7.5cm is suggested.

The longitudinal reinforcing quantity shall be based on Table 18.7.5.4, which is given in section 18.7.5.4.

Eqs. (18.7.5.4a) and (18.7.5.4b) are also used to measure the concrete strength factor kf and containment effectiveness factor kn (18.7.5.4b).

$$k_f = \frac{\grave{f}_c}{175} + 0.6 \geq 1 \qquad (18.7.5.4a)$$

$$k_n = \frac{n_l}{n_l - 2} \qquad (18.7.5.4b)$$

A column that has the same shape as a circle, has a number of horizontal bars or bar multipacks that encircle the circumference of the column center, and has horizontally supported rectangular hoops on the outside and a number of hoops' corners or earthquake hooks on the inside is referred to as a wheel column.

Table B.3: ACI318-14/Table 18.7.5.4 Transverse Reinforcement for Columns of SMRF

Transvers reinforcement	Conditions	Applicable expressions	
for rectilinear A_{sh}/sb_c hoop	and $P_u \leq 0.3A_g\grave{f}_c$ $\grave{f}_c$ ≤70 MPa	Greater of (a) and (b)	(a) $0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{\grave{f}_c}{f_{yt}}$ (b) $0.09\frac{\grave{f}_c}{f_{yt}}$ (c) $0.2k_f k_n \frac{P_u}{f_{yt}A_{ch}}$
	or $P_u > 0.3A_g\grave{f}_c$ 70 MPa $\grave{f}_c \leq$	Greater of (a), (b) and (C)	
for spiral or circular ρ_S hoop	and$P_u > 0.3A_g\grave{f}_c$ 70 MPa$\grave{f}_c \leq$	Greater of (d) and (e)	(d) $0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{\grave{f}_c}{f_{yt}}$ (e) $0.12\frac{\grave{f}_c}{f_{yt}}$ (f) $0.35k_f \frac{P_u}{f_{yt}A_{ch}}$
	or $P_u > 0.3A_g\grave{f}_c$ 70 MPa$\grave{f}_c \leq$	Greater of (d), (e) and (f)	

Commentary:

We're in the first row of the table, and we're in the first row of the condition:

(a) $0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{\grave{f}_c}{f_{yt}} \rightarrow 0.3\left(\frac{300*750}{250*700} - 1\right)\frac{30}{420} = 0.0061$

(b) $0.09\frac{\grave{f}_c}{f_{yt}} \rightarrow 0.09\frac{30}{420} = 0.0064$

So, let's use (b) 0.0064 to get the following:

$$\frac{A_{sh}}{sb_c} = 0.0064 \rightarrow s = \frac{3*78.5}{0.0064*250} = 147\ mm\ (\ short\ direction\ )$$

$$\frac{A_{sh}}{sb_c} = 0.0064 \rightarrow s = \frac{3*78.5}{0.0064*700} = 105\ mm\ (long\ direction\ )$$

Another longitudinal reinforcing with a cover of not more than 100 mm and a space of not upwards of 300 mm should be built if the concrete covering beyond the restricting longitudinal reinforcing demanded by (18.7.5.1), (18.7.5.5) and (18.7.5.6) reaches 100 mm.

Beams in moment resisting frames (MRF) structures must have As stated in Article 18.3.2 of ACI 318-14, two continuous bars are placed on both upper and lower surfaces. In addition, in any section of beam, the minimum two continuous bottom bars shall have section areas (as minimum) Bottom bars are permitted to extend by one-fourth of the span length.
bars must also be anchored to develop F_Y as tension at all faces of beams at supports, in addition to what has already been mentioned. The case in question is depicted in the diagram below.

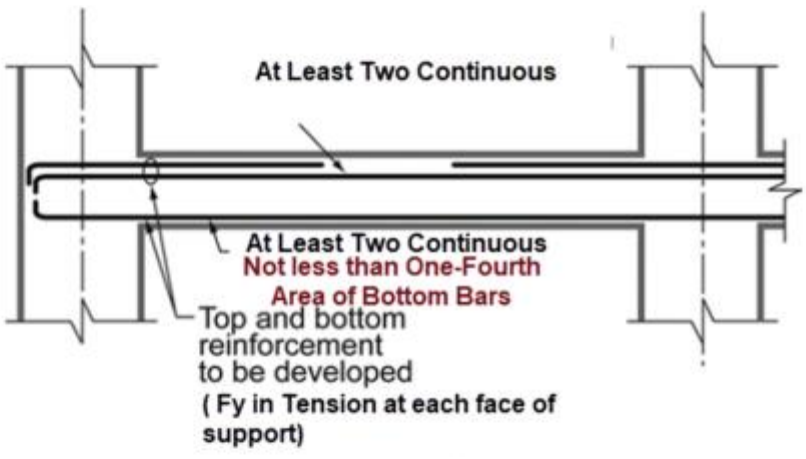


Figure B.2: Continuous Bars in Ordinary MRF

Conditions for Beams:

All beams should have two continuous bars on each upper and lower surfaces as specified in ACI 318-14 section 18.4.2.1. In addition, according to standard MRF, continuous bottom bars in all beams must have a minimum area of One-fourth of the max lower bar surface is utilized for this span. The same as the illustration shown previously

At same conditions of that in Ordinary MRF. and, according to

Article 18.4.2.2 states that at least one-third of the negatively moment strength should be applied at the joint's surface to make the positives moment strength equal to or greater than zero.

Provided that at any position throughout the span length neither the negatively nor the positivity moment strength is at or below one-fifth of the whole moment strength.

According to ACI318-14 item 18.4.2.4, Rings are needed at either ends of the beam to keep the hoop 2 h (as defined from the supportive member's face to a mid-span) from unwinding.

It's important to note that the first hoop can't be more than 50mm from the supporting member's face.

This is shown in the figure below:

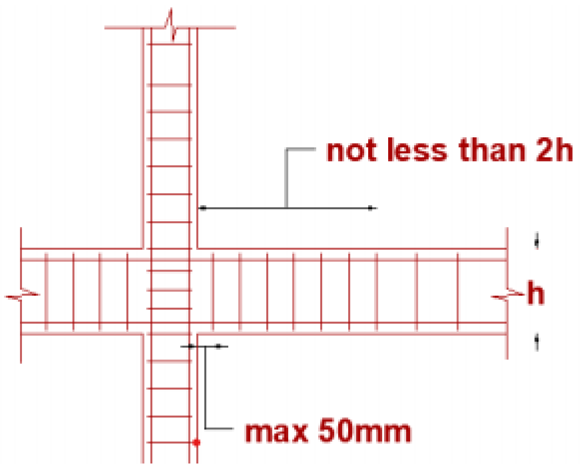


Figure B.3: Hoops at Ends of Beams in IMRF

Also, the distance between hoops must not be greater than the lowest of the three:

a) The lowest longitudinally bar, with an eightfold diameter, is used.
b) The thickness of such hoop bar is 24 times larger than the hoop itself.
c) 300 mm.
d) d/4.

Commentary based on Common Use:

In practice, the following cases through tables are commonly used in Palestine:

Table B.4: Common Use for Hoops in Beams of IMRF

Case 1	Subs	Result
$using\ d = 570mm\ (\ h = 60cm)$	570/4	142.5 mm
Smallest longitudinal bars = 12mm	8 x 12	96 mm
Diameter of hoop bars = 10mm	24 x 10	240 mm
300 mm	300	300 mm
Case 2	Subs	Result
$using\ d = 570mm\ (\ h = 60cm)$	570/4	142.5 mm
Smallest longitudinal bars = 14mm	8 x 14	112 mm
Diameter of hoop bars = 10mm	24 x 10	240 mm
300 mm	300	300 mm

The comparison between the two cases of Hoops in Intermediate MRF Beams is seen in the diagram below.

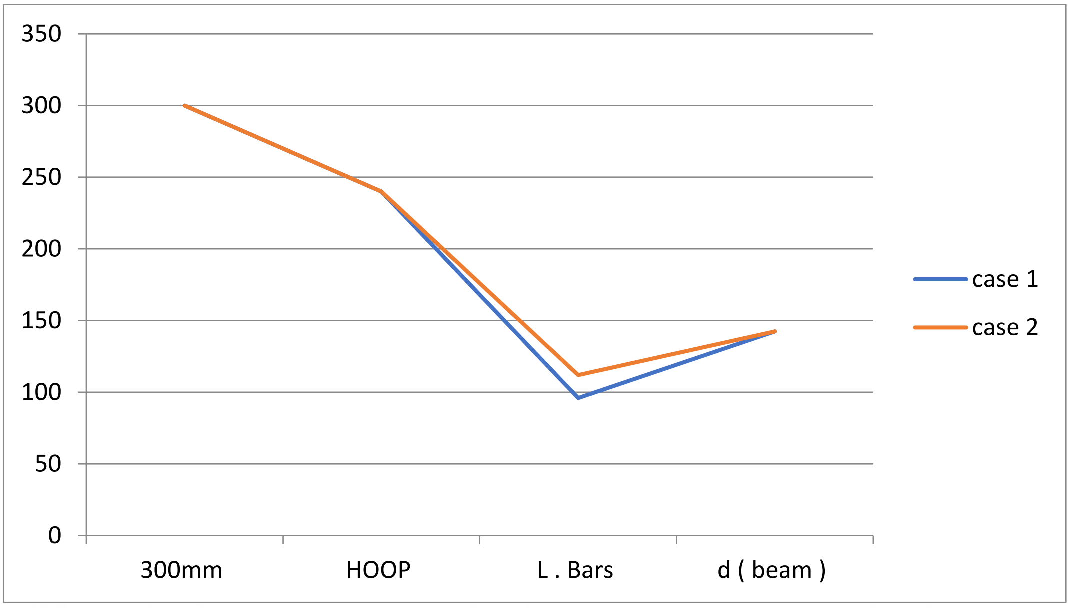


Figure B.4: Hoops at Ends of Beams in IMRF

Commentary:

a) The smallest longitudinal bar diameter in beams that is condition (b) controlling the spacing of hoops at both ends of the beam

b) So, whatever the case might be, using hoop stirrups at both ends of beams for more than 10cm is banned.

Following ACI318-14 item 18.4.2.5, the maximum transverse reinforcement spacing within the length of the beam should be (d/2) as shown in the following diagram:

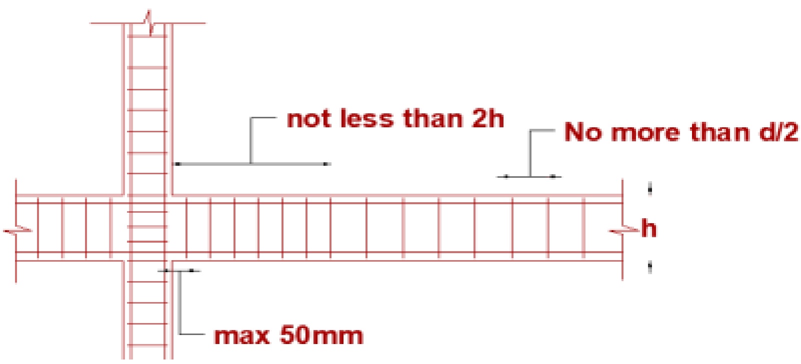


Figure B.5: Maximum Transverse Reinforcement In IMRF

Commentary:

As previously stated, hoops must be spaced no more than 2h from both ends, and in the case of a beam that is less than or equal to 250 cm in height, hoops must be spaced not more than 10cm.

Conditions for Columns:

As stated in item 18.4.3.3, hoops should be spaced a distance of approximately L_o apart, and also found on either ends sides of a column.

The least of the following shall be used for spacing S_O:

a) An eight-fold increase in the lowest longitudinally bar's diameter
b) Lowest cross-sectional diameter of column; one-half of it.
c) Hoop diameter is 24 times that of the hoop bar.
d) 300 mm.

The length L_O must be at least as long as the longest of the following:

a) A clear span of a column is approximately one-sixth of the entire column.
b) The column has the greatest cross-sectional diameter.
c) 450 mm

In practice, the following cases through tables are commonly used in Palestine:

Table B.5: Common Use For Hoops In Columns of IMRF

For Spacing $\boldsymbol{S_O}$	Subs	Result
smallest longitudinal Bars = 14 mm	8x14	112mm
Small cross section of Column stirrups = 10mm	24x10	240
small cross section of Column = 250mm	250/2	125mm
300 mm	300	300 mm

For Length $\boldsymbol{L_O}$	Subs	Result
Clear span on Columns = 3300 mm	3300/6	550 mm
maximum cross section of Column. 600-800 mm	600-800mm	600-800mm
450 mm	450mm	450 mm

The following diagram depicts schematic drawings of the findings discussed in the previous table:

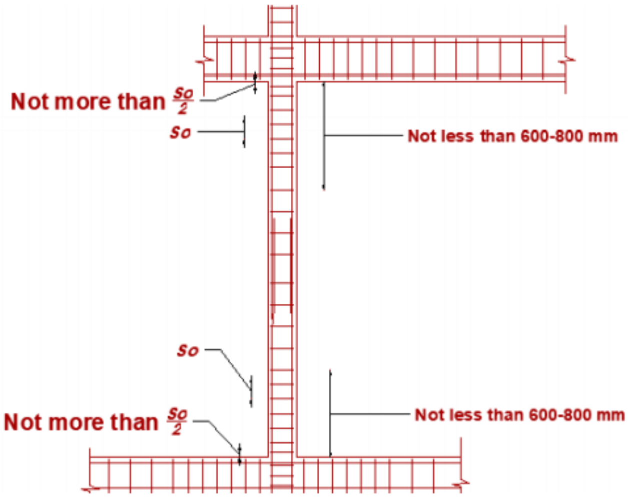


Figure B.8: Hoops at Columns in IMRF

Commentary:

According to Table 5-2, the preferred spacing of hoops in columns should not exceed 10 cm.

Because of the length of the expanded spacing of hoops along the column, it has been determined that it is preferable, extending all 10 cm spacing hoops at least 100 cm from each end of the column Moreover;

In the case of 60cm drop beams, the remaining length of the column is 240cm; thus, it is preferable for all hospitals columns to have no hoop spacing greater than 10cm.

APPENDIX C

JOINTS OF SPECIAL MOMENT FRAMES

Beam-column joints are important in a reinforcing concrete moment resistant frame, where effective transfer of load between both the components is needed (beams and columns).

The design search for joints is not important in standard design procedure for gravity loads, so it is not necessary.

However, in many earthquakes, reinforced concrete frames have failed due to shear in the joints, resulting in the building collapsing.

Where longitudinal beam reinforcement extends, as per item 18.8.2.3 For standard weight concrete, this requirement stipulates that the column diameter perpendicular to the beam reinforcements must be at least Twenty times the dimension of the biggest longitudinally beam bar. This is only required for lightweight concrete however.

Commentary:

The following table C.1 indicates the minimum dimension of Column width based on experience:

Table C.1: Minimum Dimension Of Column Width Based On Experience

	Min. Largest Bar Diameter in Beam	Minimum columns width parallel to the beam main reinforcement
1	16 mm	320 mm
2	18 mm	320 mm
3	20 mm	320 mm

As a result, columns with a diameter of less than 300mm are not advised.

As stated in item 18.8.2.4, To prevent shear from occurring in the beam framed of a joint, the depth of the beam framed joint must not even be smaller than one-half of the depth of the beam framed.

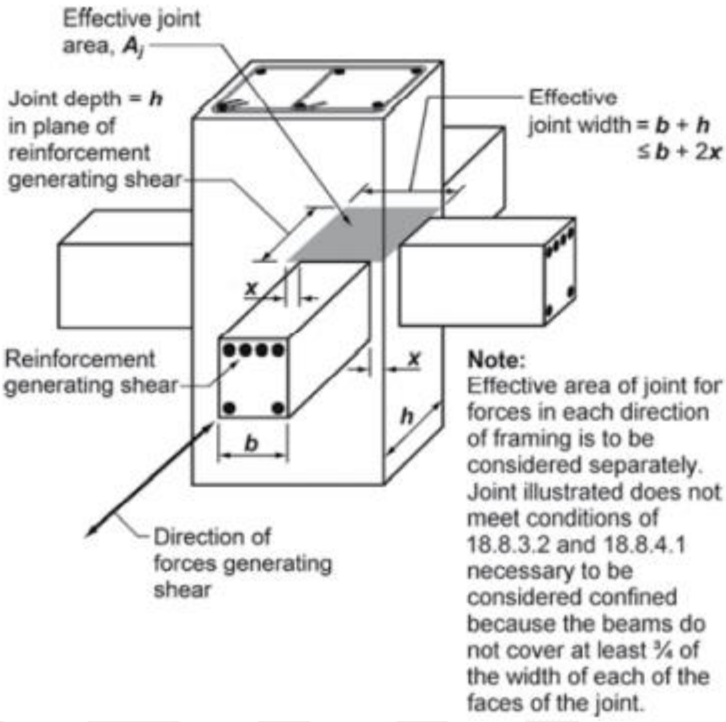


Figure C.1: ACI318-14/Fig R18.8.4 Effective Joint Are

Transverse reinforcement:

The reinforcement needs of 18.7.5.4 may be reduced by half if beams frame through several sides of the joints and if the beam diameters are each three-quarters the column width, and the space requirement of 18.7.5.3 may be increased by 150 mm.

Shear strength:

According to item 18.8.4.2; Assuming the beam dimension is at least three-quarters including its actual joint width, Table 18.8.4.1 shows that a joint face is expected to be restricted by a beam.

Commentary:

Each beam that is shorter than three-quarters of the effective joint diameter should be avoided.

Joints in reinforced concrete moment resisting frames:

As seen in Fig. 1, beam column joints are listed as internal, outer, or corner joints depending on their geometrical arrangement. In terms of code recommendations, there are fundamental variations in the mechanics of beam longitudinal bar anchorages and the shear conditions in two types of joints:

internal joints and exterior joints. An internal beam-column joint has two beams on one side of the column in the loading plane, while an external beam-column joint has a beam ending on one face of the column.

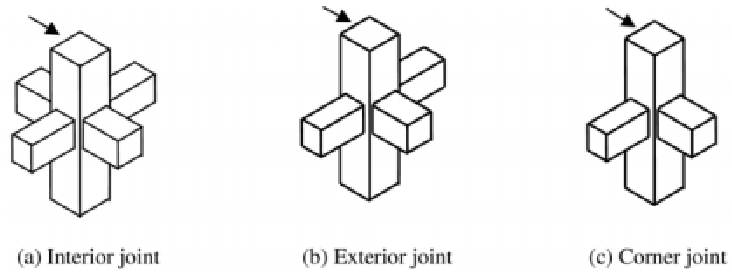


Figure C.2: Types Of Joints In A Moment Resisting Frame

APPENDIX D

CONNECTIONS BETWEEN BEAMS AND COLUMNS.

The connections should be able to transport the beam's moment and shear forces to the column. Additionally, because of materials overstrength and strain stiffening effects, the moment and shear forces could be larger than the designed values that are found via data assessment employing code-specified loads.

Member sizes:

Anchorage criteria play a major role in assessing the sizes of members in seismic environments requiring reversed cyclic packing. In addition, the need for sufficient flexural strength in columns to ensure beam yield affects member sizes.

Table D.1: Code Provisions That Influence The Size Of The Members

Parameters	ACI 318M-02
Development length for interior joints	$\frac{d_b}{h_c} \geq \frac{1}{20}$
Development length for exterior joints	$L_{dh} = \frac{f_y d_b}{5.4\sqrt{\acute{f_c}}}$
Flexural strength of columns	$\sum M_{n,c} \geq 1.2 \sum M_{n,b}$