STATISTICAL STUDY OF THE SEISMIC VULNERABILITY OF EXISTING RESIDENTIAL BUILDINGS IN AMMAN

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COMMITTEE DECISION

This Thesis (Statistical Study of the Seismic Vulnerability of Existing Residential Buildings in Amman) was successfully defended and approved on 22 July, 2007.

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DEDICATION

To

My Father, my Mother, my Brothers and Sisters

My Husband and my Daughter

Botheina Khatib

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The guidance, support, encouragement and assistance of Prof. Hassan Saffarini in conducting this research are genuinely appreciated.

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NOTATION

A_{tot}	total floor area
EI_{eff}	effective section stiffness
EI_p	section stiffness of the pier
EI_{sp}	section stiffness of the spandrel
H_{tot}	total building height
I	second moment of area
M_{cr}	bending moment at cracking
M_u	bending moment at ultimate
M_y	bending moment at yield
R	strength reduction factor
T_1	fundamental period
S_a	spectral acceleration
S_d	spectral displacement
V_b	base shear of the building
V_{be}	equivalent elastic base shear of the building
V_{bm}	shear capacity of the building
V_{bcr}	base shear at cracking
V_c	shear carried by the concrete
V_{cr}	shear force at the onset of cracking of a wall
V_m	shear capacity of the wall

V_N	shear strength enhancement resulting from axial compression
V_s	shear carried by transverse reinforcement
V_{shear}	shear strength of concrete wall
$V(x)$	shear force at x due to the real forces
a	acceleration
	distance between node of assemblage and point of contraflexure.
b_{eff}	effective width
d_f	floor thickness
d_u	depth of underbeam
f_1	fundamental frequency
f'_c	cylinder compressive strength of concrete
f_{ct}	tensile strength of concrete
f_y	yield strength of reinforcement
f_{yh}	yield strength of transverse reinforcement
h	height of a wall element
h_0	height of zero moment
h_E	equivalent height
h_{eff}	effective height
h_i	height of the i-th story from the base
h_p	height of the pier

h_{st}	story height
k	stiffness of the building
k_{eff}	effective stiffness of the wall
h_p	height of the pier
k_s	total shear stiffness of shear beam
$k_{s,tot}$	shear stiffness of one storey
l	length
l_o	length of spandrel
l_p	length of plastic hinge
l_w	wall length
l_x	length of the building in x-direction
l_y	length of the building in y-direction
m_E	equivalent mass
m_i	concentrated storey mass
t	thickness of wall, wall element, pier
x_{cr}	depth of neutral axes at cracking
x_u	depth of neutral axes at ultimate
x_y	depth of neutral axes at yield
Γ	modal participation factor
Δ	horizontal top displacement
Δ_{be}	equivalent elastic top displacement of the building
Δ_{bu}	ultimate top displacement of the building

Δ_{by}	yield top displacement of the building
Δ_{cr}	top displacement at the onset of cracking
Δ_D	displacement demand
Δ_e	equivalent elastic displacement
Δ_u	ultimate top displacement of the wall
Δ_y	yield top displacement of the wall
α_1, α_2	assemblage factors
β	coefficient to calculate the displacement ductility of a wall
ε_c	concrete compressive strain at the extreme compressive fiber
ε_{ct}	concrete tensile strain at the extreme tensile fiber at the onset of cracking
ε_{cu}	ultimate compressive strain of concrete
ε_s	strain in the extreme tensile reinforcement
ε_y	yield strain of reinforcement
ϕ_{cr}	equivalent curvature at cracking on the bilinear moment-curvature relationship
ϕ_i	first mode displacement at the i-th story
ϕ_p	plastic curvature
ϕ_u	ultimate curvature
ϕ'_y	first yield curvature
ϕ_y	yield curvature

μ_D	ductility demand
μ_ϕ	curvature ductility of a wall section
μ_Δ	displacement ductility of the building
μ_W	displacement ductility of the wall
θ_p	plastic rotation
ρ_c	density of concrete
ζ	coefficient
ω	circular frequency
ω_d, ω_m	dimensionless parameters to take into account the effect of frame action

STATISTICAL STUDY OF THE SEISMIC VULNERABILITY OF EXISTING RESIDENTIAL BUILDINGS IN AMMAN

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ABSTRACT

The assessment of the seismic vulnerability of existing buildings is significant to recognize the characteristics of buildings that cause more susceptibility to the effects of earthquakes, which is the first step in mitigating the detrimental results of seismic actions.

Since no major damaging earthquakes have occurred in Amman in the recent decades, vulnerability functions from observed damaged prototypes are not possible to apply. A simple but detailed method based on nonlinear static procedures developed by Kerstin Lang was applied to six building prototypes; the results were vulnerability functions expressing the expected damage of those models as a function of the seismic input.

Results concluded from the vulnerability method were then compared with the output of the software ETABS nonlinear after running one of the models utilizing a static nonlinear Pushover Analysis, and it was found that the selected method is consistent with more refined models such as the pushover analysis.

This study relied on a database of statistics including a population of 110 existing residential buildings in Amman. Statistics have captured many important features of residential buildings, and results were used to relate the outcome of the vulnerability analysis with the current condition of residential buildings, by estimating the cost of repair needed for different building heights in case an earthquake happened.

Several important factors affecting the building's response against earthquakes were studied; number of floors, availability of reinforcements in shear walls, type of soil, and the seismic demand.

By studying the effect of each one of those factors, it was found that buildings with no reinforcements in their shear walls are more vulnerable to seismic action (when considering soil type D, and the double seismic demand specified by the code) than those having reinforced shear walls under the same circumstances. Also buildings with four floors and more are considered not to be vulnerable and showing a good response to seismic action assuming that they have well designed columns and reinforced shear walls with no construction defects and irregularities.

Also it was found in this study that the approximated cost of repair of the whole residential building stock in Amman will be 9 JD/m^2 if the code earthquake takes place, but if double the code earthquake takes place the cost of repair will increase to 30 JD/m^2.

CHAPTER 1 Introduction

1.1 Background

The vulnerability of a building subjected to an earthquake is dependent on seismic deficiency of that building relative to a required performance objective. The seismic deficiency is defined as a condition that will prevent a building from meeting the required performance objective.

Depending on the vulnerability assessment, a building can be condemned and demolished, rehabilitated to increase its capacity, or modified so that the seismic demand on the building can be reduced. Thus, various methods were developed for the seismic vulnerability assessment of existing buildings against future earthquakes.

Any ground movement produced by earthquakes is expected to have an impact on engineering structures. The need arises to undertake measures to protect structures based on assessments of the risk. Earthquake risk may be defined as the probability of the loss of property or loss of function of structures, life, utilities, and so on. The factors entering into the assessment or qualitative estimation of earthquake risk are, the *earthquake hazard* (the probability of occurrence of ground motion due to an earthquake), the *value* of the elements exposed to the hazard (property and lives), and the *vulnerability* of these elements to damage or destruction by ground motions associated with the hazard.

In their simplest functional form the factors entering into the assessment of risk may be expressed by the following relation:

Risk= earthquake hazard * vulnerability * value

This relation may be applied to estimate the risk in financial or economic terms, to buildings and their components. Seismic vulnerability functions express the relationship between damage or loss to a structure or facility and earthquake effects (i.e., intensity).

Earthquake damage to the built environment is caused by a number of factors in addition to ground shaking, e.g. landslides. In this study the attention of vulnerability is restricted to that relating directly to the principal cause of damage, namely ground shaking. All other effects, such as subsidence, landslides, liquefaction and earthquake-induced fires, are supplementary phenomena.

Earthquakes cause damage to buildings, bridges, and other facilities by imposing excessive deformations and resulting stresses in their constitutive elements. Repair of this damage can be costly, and the time to restore the damage can cause substantial additional losses due to business interruption, lost revenues, and other disruptions to the function of the facility, (Dorwick, 2003).

These damages and economic losses are often significant enough to imperil the survival of businesses and threaten the lives of occupants. As a result, the need arises to estimate, or model, future earthquake losses, for planning and risk management purposes.

Early efforts to create structural vulnerability functions were driven in part by a desire to document and understand the consequences of major earthquakes. Business interests also played a role — owners of buildings and other properties had a need to quantify their potential losses — and property insurers wished to understand the risks they were incurring through the sale of earthquake and other insurance.

Property insurers, in particular, faced two challenges when providing earthquake coverage:

• Solvency: Insurers need to assure that a catastrophic event simultaneously affecting many insured properties will not bankrupt the company.

• Profitability: Insurers need adequate premium income to cover future claims and expenses, while yielding a profit and remaining competitive.

Recent statistics provided an adequate basis for predicting future losses for most risks (e.g., fire, auto, life). Earthquakes, however, differ in that they are high consequence–low probability (HCLP) events. That is, decades may pass between events that cause significant loss, by which time construction has changed so significantly that past events provide little insight into future losses.

The insurance industry is not the only user needing to estimate future earthquake losses. Other examples include:

1. A government agency with an emergency-response mission must assure that adequate resources are available when an earthquake strikes a densely populated area.
2. A commercial lender must control losses associated with borrowers defaulting on their mortgage after an earthquake wipes out their equity. The probability of default is related to the degree of damage, which is estimated using seismic vulnerability functions.
3. A manufacturer may wish to mitigate the chance that an earthquake near a critical factory could interrupt production.
4. Building-code authorities wish to know whether the cost of a new code provision is justified by reductions in future damage.

In each case, a purely statistical approach based on past experience is inadequate for reliable estimation of potential future losses.

The tool to overcome this deficiency is probabilistic risk analysis (PRA). A seismic PRA characterizes the probability of occurrence of future earthquakes (the seismic hazard), and the damage or loss conditioned on the effects of the earthquake at the site of each asset (the seismic vulnerability). The general case of a seismic PRA seeks to establish the relationship between loss and probability (or frequency) of exceeding that loss in a particular future time period

1.2 Research objective and significance

The general idea of this research is to investigate the seismic vulnerability of existing residential buildings, while considering several types of those buildings particularly in the City of Amman. The objective is to identify the buildings that are highly vulnerable to the earthquake effects based on certain attributes, and to estimate the cost of repair of buildings after seismic events.

Residential buildings are the target structures of this study given that they represent a percentage of almost 77%[1] of the total building stock in Jordan. At the present time the kingdom is witnessing a huge growth in the housing sector, which means that a careful attention must be given to judge the performance of this sector, in order to protect lives and properties in case of any seismic event.

As the title of the thesis indicated, the seismic vulnerability issue was taken from a statistical perspective; i.e. the study relied on a database of statistics about residential buildings in the city of Amman. These information were essential for the selection of the representative examples and the estimation of the economical losses due to expected earthquakes.

[1] Administration of General Statistics, Building Census 2004.

This vulnerability study gained its impact from the comprehensive and realistic samples of buildings that are a good representation of the majority of population of residential buildings in Amman. Those samples were selected according to many aspects: number of floors, area of floor, soil type, and structural configuration.

Advantage is made of existing vulnerability methods, in order to make an assessment of the selected representative structures. On the other hand a 3-D model using software ETABS 9 Nonlinear was constructed along with static nonlinear pushover analysis to define the performance of those structures under earthquake loading and to verify the results of the vulnerability methods.

1.3 Limitations

This work focused on generating simple but realistic models for residential buildings, and applying a well documented seismic vulnerability method for them. Some simplifications were adopted in the layout of the sample buildings to facilitate the speed of calculations. For example, simple horizontal and vertical configurations were chosen for the models and typical sections for walls and columns were utilized.

The study focused on residential buildings in Amman where the exterior facades are cladded with stone. In spite of the high stiffness of those exterior stone walls, they are assumed to have no beneficial effect on the lateral resistance of the structure. This assumption was based on the strength degradation behavior of stone walls during seismic excitation.

During this study, a statistical effort was made by the author to collect information about existing residential buildings in Amman since there is no ready detailed information about buildings found at the public administrations.

1.4 Research overview

The second chapter gives an overview of the available vulnerability assessment methods classified into four categories; vulnerability methods based on statistics of observations made after earthquake occurrences, vulnerability methods based on simple analytical models, vulnerability methods based on score assignments, and vulnerability methods based on detailed procedures.

The third chapter presents the selected vulnerability method which was applied, starting with the definition of the vulnerability function, and then presenting the concept of the capacity curve of buildings, going through the seismic demand and finally constructing the vulnerability function.

In the fourth chapter, a brief discussion is presented concerning the seismicity of the region as a whole, then looking closely at Jordan and especially the city of Amman.

The fifth chapter presents the source and the method of collecting the statistical information used in this study, as well as the results of this statistics.

In the sixth chapter, the selected building types are presented and then the vulnerability method was applied to them with detailed results.

The seventh chapter introduces the utilization of software ETABS 9 Nonlinear to perform a static nonlinear pushover analysis for one of the prototypes, in order to assess the performance of the structures under earthquake loading, and comparing the results with the vulnerability study.

The eighth chapter summarizes the results of the evaluation of the building prototypes, and relates the vulnerability results with the statistics.

CHAPTER 2 Review of Literature

2.1 Introduction

In this chapter available vulnerability assessment methods are presented to provide an overview of the state-of-the-art.

Those methods are being classified according to the type of information available and the needed computational effort, ranging from vulnerability methods that depend on an earthquake damage database (section 2.2), via methods based on simple models and score assignments (section 2.3 and 2.4), to more detailed procedures (section 2.5).

2.2 Vulnerability methods based on statistics of past earthquakes

Recently, great interest has been placed on the research of methods that estimate the possible risk of earthquake and the vulnerability of the built environment. Among those studies (Otani, 2000) who introduced a seismic vulnerability assessment method that is used by the Japanese Ministry of Construction.

After the 1995 Nanbu earthquake the Japanese congress realized the urgent importance of improving seismic resistance of existing buildings, and therefore legislated the need to apply seismic vulnerability assessment for all buildings and required that the owner should make effort to strengthen the structure if needed.

The method is based on collecting statistics about heavily damaged areas, and the damage level of buildings in that area was assessed by structural engineers according to the following classification;

a) Operational damage

b) Heavy damage

c) Collapse

Several aspects were taken into consideration for the seismic vulnerability assessment, such as:

- Material properties on site
- Structural configuration
- Site conditions
- Soil- structure interaction
- Quality of workmanship
- Importance of the building
- Age of the structure
- Safety of nonstructural elements.

Also the following characteristics were very important to examine through the investigation at the building site as they have a direct impact on the potential structural deterioration of the building:

- Existing cracks
- Observed deflection under gravity conditions
- Uneven settlement caused by foundation deformation
- Rust on reinforcement.

Otani used Newmark's design criteria to determine a minimum base shear coefficient C_y required for an elastic-plastic Single Degree of Freedom, SDOF, system having ductility μ to resist a ground motion whose intensity produces an elastic response base shear coefficient C_e.

$$C_y = \frac{C_e}{\sqrt{2\mu - 1}} \qquad \text{for short period systems}$$

$$C_y = \frac{C_e}{\mu} \qquad \text{for long period systems}$$

The maximum elastic response base shear coefficient was used as an index to represent the intensity of ground motion.

C_e (elastic response base shear coefficient), represented as

$$C_e = I_s . Z . R_t(T) = E_o$$

Where, I_s is the structural seismic capacity index, Z is the seismic zone factor, $R_t(T)$ is the vibration characteristic factor, and E_o is the structural index. This structural seismic capacity index I_s is important to represent the level of seismic safety margin of a structure. The same calculations were extended to include SDOF structures having two types of structural members, for Multiple Degree of Freedom, MDOF, structures and finally for irregular configuration at a story of the structure.

Statistical procedures have been utilized by (Yakut et al., 2006) to investigate thoroughly the performance of low-to mid-rise reinforced concrete buildings during the major earthquakes in Turkey. A damage database of about 500 representative buildings experiencing the 1999 Duzce Earthquake have been used, and discriminate functions expressing damage score in terms of six damage inducing parameters have been developed. Some modifications were then introduced to this procedure to permit for its use in other regions, such as taking into account different soil conditions, site-to-source distance, and the magnitude of the earthquake.

2.3 Vulnerability methods based on simple models

Calvi (1999) presented a method based on the estimation of the displacement and energy dissipation capacity of the structure by defining the elastic displacement response spectra as a function of assumed return period, soil condition and geographical location. He then defined a set of performance levels to express the possible building response, and constructed simplified structural models for different building classes as a function of the available data.

Finally, calculations were made to determine the minimum and maximum displacement capacity, the minimum and maximum period of vibration, and the displacement demand reduction factor. Advantage was made of previous calculations to estimate the probability that a building attains ultimate limit state.

2.4 Vulnerability method based on score assignments

The main reference to this vulnerability assessment method is *FEMA 310, Seismic Evaluation Handbook.*

This handbook is based on the *NEHRP Handbook for Seismic Evaluation of Existing buildings.* This handbook was written to:

- Reflect advancement in technology,
- Incorporate design professional experience,
- Incorporate lessons learned during recent earthquakes,
- Provide evaluation techniques for varying levels of building performance.

FEMA 310 provided a process for seismic evaluation of existing buildings. A major portion was dedicated to instruct the evaluating design professionals on how to determine whether a building is adequately designed and constructed to resist seismic forces. All aspects of building performance were considered and defined in terms of structural, nonstructural and foundation/geologic hazard issues.

2.5 Vulnerability methods based on detailed procedures

A further effort has been made by Masi (2003) who evaluated the seismic vulnerability of existing reinforced concrete frames designed only for vertical loads. This framed structural type was widely common before 1970 in Italy. The first step was to determine the most important structural characteristics of such buildings by referring to the technical documentation of real buildings found in the archives of public administrations, and to the codes and handbooks adopted at that period. Then, typical samples of RC frames were selected to be designed only to vertical loads according to codes of that period. Seismic response was determined using nonlinear dynamic analysis, while the seismic resistance was evaluated using fragility curves. As a final point, the vulnerability class was defined for each type according to the European Microseismic Scale 1998 (EMS).

CHAPTER 3 Selected Vulnerability Method

3.1 Introduction

For the intention of evaluating the seismic vulnerability of existing residential buildings in Amman, the method presented by Kerstin Lang (2002) was used.

This method was presented in a doctoral thesis developed within the framework of the research project "Earthquake scenarios for Switzerland" submitted to the Swiss Federal Institute of Technology – Zurich. And also was published in well documented international journals.

3.2 Advantages of the selected method

As stated in chapter two, four types of vulnerability methods are available. The first type (which depends on statistical information from previous earthquakes) is quite suitable for high seismicity regions (prone areas) where the seismic records of past earthquakes are plentiful and statistical information about the consequences of such earthquakes on buildings and humans are well documented.

In the case of Jordan, however records and observations of previous earthquake damage to contemporary buildings do not exist; the first method cannot thus be applied.

The method based on score assignments is time consuming and requires well experienced professionals to apply over the range of building performance in order to determine the structural deficiencies of the building stock.

Detailed procedures that use the linear analysis (static or dynamic) are really simple but they neglect the important effect of the nonlinear behavior of buildings under seismic action.

Other detailed procedures that relied on nonlinear dynamic approaches require very sophisticated computational effort and can be applied on limited samples.

Therefore it was decided to use the method of Lang (2002) which is based on nonlinear static analysis using simple models. Thus it can be applied to a larger number of buildings and can provide meaningful results.

This method (Lang, 2002) was also used by another researcher (Tahrawi, 2005) from the University of Jordan, and it was applied on two types of buildings; residential buildings (two models) and school buildings (three models). The effect of the material strength and the soil bearing capacity on the seismic vulnerability was studied for those models. The same method was selected for this study also, but it was applied to a larger number of prototypes (that were able to represent and comprehend most of the residential building stock in Amman) and the effect of several parameters was studied (the effect of number of floors, soil type, seismic demand, and the existence of reinforcement in shear walls). In addition, the results of the vulnerability method were verified by comparing them to the results of a pushover analysis, and were also related to statistical information as discussed in the following chapters.

3.3 The concept of vulnerability functions

Vulnerability function is a relationship that defines the expected damage of a building or a class of buildings as a function of the ground motion. The two main components of a vulnerability analysis are the capacity of the building and the seismic demand. In order to estimate the damage, the ability of the building to resist constraints (capacity of the building) must be compared with the constraints on the structure due to the earthquake ground motion (seismic demand).

In earthquake engineering the capacity of a building to resist seismic action is presented by a capacity curve which is defined as the base shear acting on the building as a function of the horizontal displacement at the top of the building, also often referred to as a *pushover curve*. The shear capacity of the building refers to the maximum base shear the building can sustain and the displacement capacity refers to the ultimate displacement at the top of the building.

In general, it is possible to express the capacity of any structure (building) or structural element (wall, wall element) to resist seismic action by the shear force acting on it as a function of the horizontal displacement at the top (capacity curve). Likewise, the shear capacity of any structure or structural element refers to the maximum shear force it can sustain, and the displacement capacity refers to its ultimate horizontal displacement, (Lang, 2002).

To define the seismic demand of a certain area the seismicity nature of the ground should be taken into consideration to express the earthquake behavior. Each country had confirmed a building code that defines the expected earthquake in terms of spectral acceleration S_a, or spectral displacement, S_d.

3.4 Moment-curvature relationship of reinforced concrete sections

For any concrete section with the distribution of reinforcement and the acting normal force are given, the moment-curvature relationship can be established with a bilinear approximation (Figure 3.1).

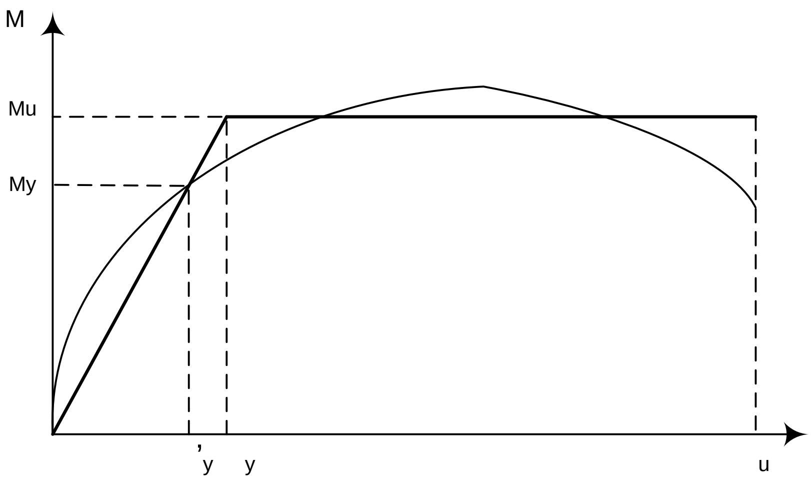

Figure 3.1: Bilinear moment curvature relationship

This bilinear approximation is determined by two points (ϕ'_y, M_y) which corresponds to the point of first yield, and (ϕ_u, M_u) which corresponds to the point of ultimate.

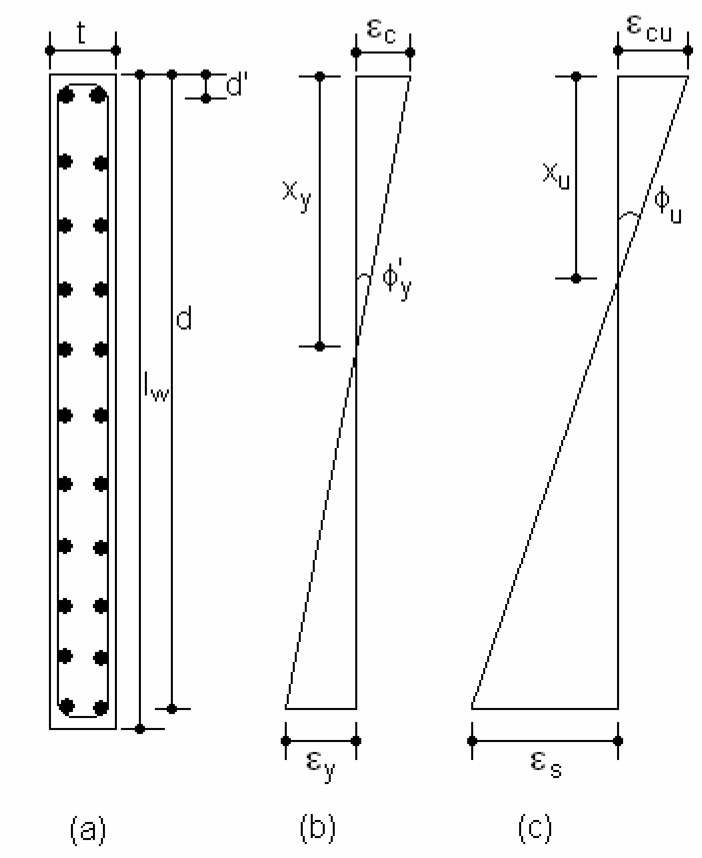

Figure 3.2**:** (a) wall section, (b) strain at first yield and (c) strain at ultimate

From figure 3.2, the first-yield curvature ϕ'_y is given by

$$\phi'_y = \frac{\varepsilon_y}{d - x_y} \qquad \text{…… (3.1)}$$

Where; ε_y is the yield strain of the reinforcement, $\varepsilon_y = \frac{f_y}{E_s}$

d is the depth of the extreme tensile reinforcement.

x_y is the depth of the neutral axis at first yield.

The ultimate curvature ϕ_u is given by:

$$\phi_u = \frac{\varepsilon_{cu}}{x_u} \qquad \text{...... (3.2)}$$

Where; ε_{cu} is the ultimate compressive strain of concrete

x_u is the depth of the neutral axis at ultimate.

The nominal yield curvature of the bilinear approximation can be extrapolated as:

$$\phi_y = \phi'_y \frac{M_u}{M_y} \qquad \text{...... (3.3)}$$

The curvature ductility of the section is defined as:

$$\mu_\phi = \frac{\phi_u}{\phi_y} \qquad \text{...... (3.4)}$$

3.5 Typical types of reinforced concrete buildings in Jordan

The vast majority of residential buildings in Jordan are reinforced concrete buildings that use a frame system consisting of beams and columns in combination with concrete walls either reinforced or plain. Load bearing walls system is hardly used nowadays in residential building; however it was the common practice in the past.

Common structural systems can be categorized into the following types:

a) Structural wall system with negligible frame action

b) Structural wall system with separate frame action

c) Structural wall system with frame action due to coupling of walls

The first type is the structural wall system with negligible frame action (see Figure 3.3 (a)), this type comprises slender reinforced concrete walls that act to transmit the horizontal forces to the ground, with slender columns that carry only gravity loads and reinforced concrete slabs. The frame action can be considered negligible, and the building can be assumed to be a system of interacting cantilevers with a moment distribution over the height of the building due to horizontally acting equivalent earthquake forces.

If the columns are less slender and/or the floors have drop beams a moderate frame action develops which should be taken into account. However, the frame action derives largely from the gravity load columns and not only from the walls, hence "separate". This type is shown in Figure 3.3 (b) with the moment distribution over the height of the building due to horizontally acting equivalent earthquake forces are also shown.

In most existing residential buildings, the structural walls are grouped around staircases and lift shafts, the rest of the building is supported by columns that contribute largely in the frame action of the building.

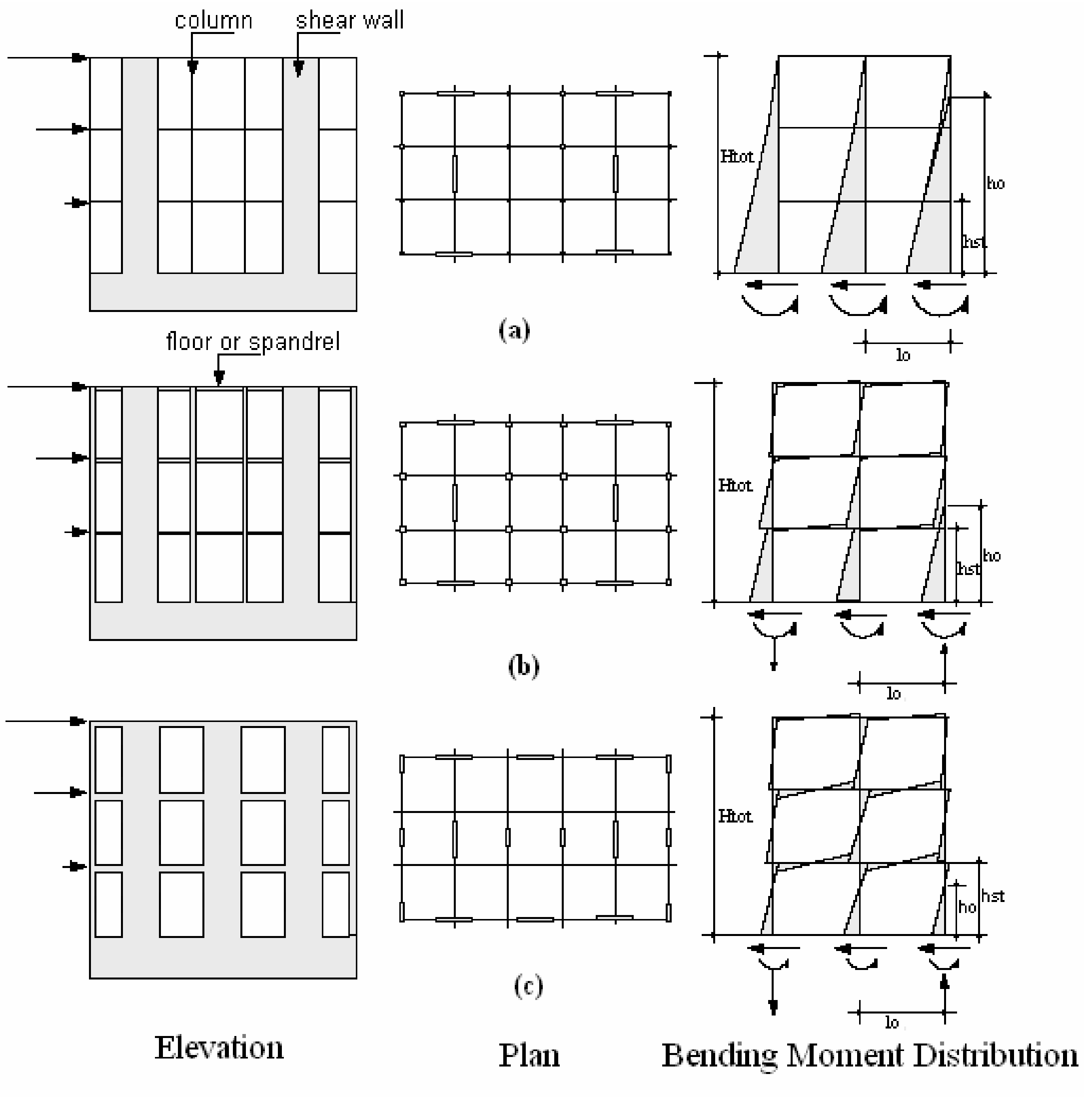

Figure 3.3: Types of structural wall systems with moment distribution for shear walls
(a) Structural wall system with negligible frame action
(b) Structural wall system with separate frame action
(c) Structural wall system with frame action due to coupling of walls

The third type of structural wall systems with frame action due to coupling of walls is shown in Figure 3.3 (c). It consists only of walls which carry both, horizontal and vertical forces and no gravity load columns exist. The frame action derives completely from the coupling of the walls by floors and spandrels. The moment distribution over the height of the building due to horizontally acting earthquake forces is also shown in the same figure. This type of structural wall system with pure reinforced concrete walls is rarely or almost not used in residential buildings in Jordan, but a mixed system of reinforced walls with plain concrete or masonry walls could be found in old houses that consist usually of one or two stories.

3.6 Capacity curve of reinforced concrete buildings

The capacity curve of a building is a plot of the base shear as a function of the top displacement and can be obtained by superposition of the capacity curves of the walls and columns of the building.

The bilinear capacity curves of walls or columns are defined by the three subsequent parameters:

- The shear capacity of the wall V_m
- The nominal yield displacement at the top of the wall Δ_y
- The nominal ultimate displacement at the top of the wall Δ_u

The shear capacity of reinforced concrete walls derives primarily from its flexural strength. Hence, it can be deduced from the moment-curvature relationship as a function of the force distribution and the frame action. The next sections elucidate the construction of the capacity curve of a reinforced concrete building taking into account each one of the three types of structural systems: wall systems with negligible frame action, wall systems with separate frame action and wall systems with frame action due to coupling of the walls.

3.6.1 Structural wall system with negligible frame action

This type of buildings has no coupling between walls and the building can be seen as a system of interacting cantilever walls.

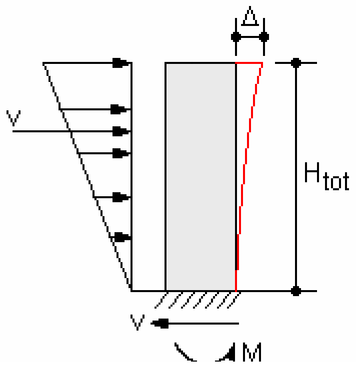

Figure 3.4**:** Cantilever wall with triangularly distributed horizontal forces

From figure 3.4, the top displacement of a cantilever wall due to triangularly distributed forces is:

$$\Delta = \frac{11}{60} \cdot \frac{V.H_{tot}^3}{EI} \qquad \text{...... (3.5)}$$

The height of the resultant force is $\frac{2}{3}H_{tot}$. And then the maximum shear force the wall can sustain is:

$$V_m = \frac{M_u}{\left(\frac{2}{3}.H_{tot}\right)} \qquad \text{...... (3.6)}$$

Knowing that $\phi = \frac{M}{EI}$ (for elastic systems only) and substituting Equation (3.6) into Equation (3.5) gives:

$$\Delta_y = \frac{11}{40}.\phi_y.H_{tot}^2 \qquad \text{...... (3.7)}$$

The above equation considers a triangular force distribution applied at the height of the building. For a more general form, the top displacement at first yield can be given as:

$$\Delta_y = \chi.\phi_y.H_{tot}^2 \qquad \text{......(3.8)}$$

Where χ a coefficient that varies from 0.17 for a single force applied at the top of the building to 0.275 for a triangular distribution.

The ultimate displacement at the top of the wall is calculated using the following relationship:

$$\Delta_u = \mu_w.\Delta_y \qquad \text{...... (3.9)}$$

Where μ_w is the displacement ductility of the wall and can be calculated using the curvature ductility (see equation (3.4)):

$$\mu_w = 1 + \frac{1}{\chi.H_{tot}^2}.(\mu_\phi - 1).l_p.\left(H_{tot} - \frac{l_p}{2}\right) \qquad \text{......(3.10)}$$

l_p is the length of the plastic hinge. This value has a crucial influence on the displacement ductility of the wall. In the literature, three different definitions for the length of the plastic hinge can be summarized:

1. The length over which the detailing of the transverse reinforcement is applied according to capacity design principles.
2. The length over which the longitudinal reinforcement has yielded.
3. The length when multiplied by the plastic curvature $\phi_p = \phi_u - \phi_y$ results in the correct plastic rotation θ_p , i.e. the plastic rotation that is used to predict the top displacement of the wall.

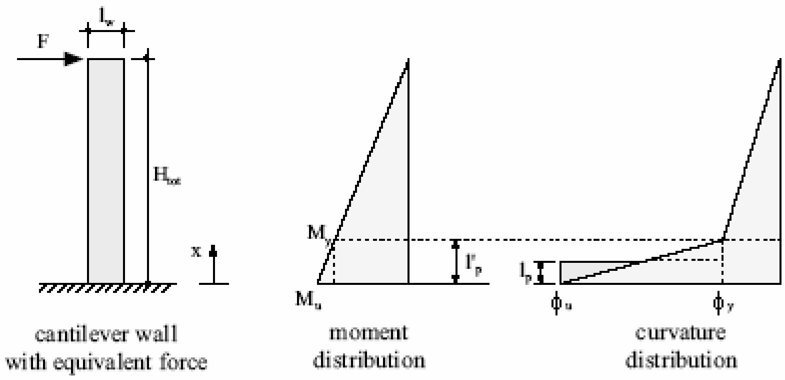

Figure 3.5: Cantilever wall with equivalent horizontal force and the corresponding moment and curvature distribution at ultimate

The first and second definitions refer to a region, only the third definition refers to the length of the plastic hinge, therefore, the third definition was used in the calculations and is expressed as:

$$l_p = 2.H_{tot}.\cos\left(\frac{\varphi}{3} + \frac{4}{3}.\pi\right), \text{ with } \cos\varphi = \frac{M_y}{M_u} - 1 \qquad \text{......(3.11)}$$

Equation (3.11) is very convenient since it expresses the length of the plastic hinge in terms of M_y and M_u that are determined from the moment curvature relationship (see figure 3.5).

After evaluating the length of the plastic hinge the three parameters defining the bilinear capacity curve of a cantilever wall V_m, Δ_y and Δ_u are determined. The capacity curve of the building in one direction can then be obtained by combining the capacity curves of all the walls acting in this direction.

3.6.2 Structural wall system with separate frame action

In this type of structural wall systems the frame action can no longer be neglected since it is derived largely from the gravity load columns and not only from the walls.

Accordingly, the frame action is considered in a further step after the capacity curve of the system of cantilever walls is constructed as discussed in the previous section.

The contribution of the frame action can essentially be described by a shear beam with shear stiffness. In order to assess the shear stiffness of the shear beam the shear stiffnesses of the assemblages consisting of horizontal elements such as floors and spandrels and vertical elements such as walls and columns have to be estimated.

Figure 3.6 shows four cases of standardized assemblages for the estimation of the frame action after Dazio (2000).

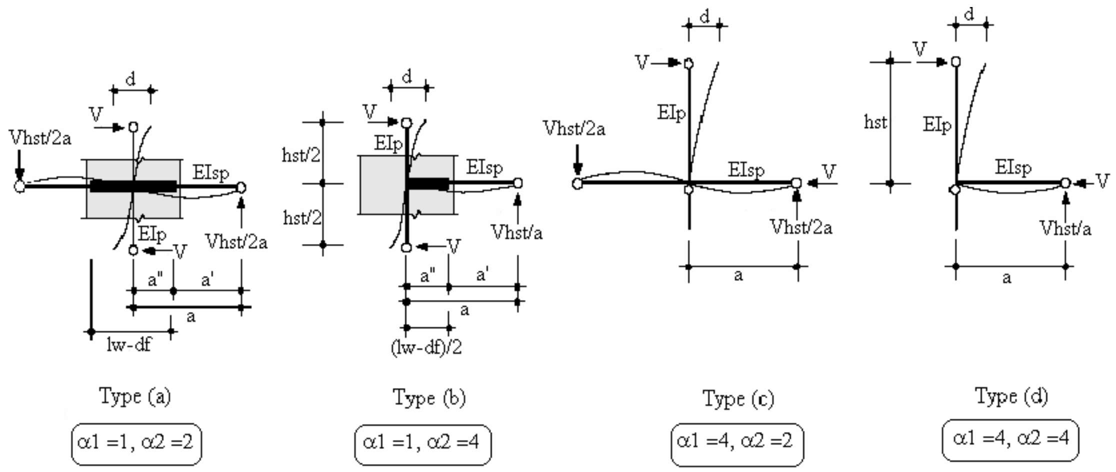

Figure 3.6: Standardized assemblages for the estimation of the frame action (Dazio, 2000).

The height of the assemblages referred to the storey height h_{st}. The distance *a* is the distance between the node of the assemblage and the point of contraflexure in the floor and has to be estimated. In the case of structural walls the floor is assumed to be rigid over the distance a" and only flexible over the distance a'.

The following equation was proposed for the estimation of a':

$$a' = a\frac{l_w - d_f}{2} \qquad \text{......(3.12)}$$

Where l_w is the length of the wall and d_f is the thickness of the slab.

Taking advantage of the virtual work principle, the shear stiffness of an assemblage can be expressed as:

$$k_s = \frac{12.EI_p}{\alpha_1.h_{st}^2 + \alpha_2.h_{st}.a'.(a'/a)^2.(EI_p/EI_{sp})} \qquad \text{......(3.13)}$$

Where EI_p is the section stiffness of the vertical element of the assemblage such as a wall or a column, and EI_{sp} is the section stiffness of the horizontal element such as the slab or a spandrel. α_1 and α_2 are two coefficients that depend on the boundary conditions of the assemblage and to be taken from Figure 3.6.

The total shear stiffness of the shear beam is equal to the sum of the shear stiffnesses of the assemblages:

$$k_{s,tot} = \sum k_s \qquad \text{……(3.14)}$$

The base shear of the whole structural system at which yielding occurs, $V_{by,sys}$, can be determined from the shear capacity of the system of cantilever walls $V_{bm,w}$ as follows:

$$V_{by,sys} = \frac{V_{bm,w}}{\omega_m} \qquad \text{……(3.15)}$$

Where ω_m is a dimensionless parameter that can be approximated for a triangular force distribution as:

$$\omega_m = \frac{1}{\sqrt{\zeta + 1}} \qquad \text{……(3.16)}$$

Where

$$\zeta = \frac{k_{s,tot}.H_{tot}^2}{2.EI_{w,tot}} \qquad \text{……(3.17)}$$

$EI_{w,tot}$ is the sum of the section stiffnesses of the walls.

The yield displacement of the whole system, $\Delta_{by,sys}$ can be determined from the yield displacement of the system of cantilever walls $\Delta_{by,w}$:

$$\Delta_{by,sys} = \frac{\omega_d}{\omega_m} . \Delta_{by,w} \qquad ……(3.18)$$

Where ω_d is a second dimensionless parameter that can be approximated for a triangular force distribution as:

$$\omega_d = \frac{1.32}{1.32 + \zeta} \qquad ……(3.19)$$

3.6.3 Structural wall system with frame action due to coupling of walls

For this type, horizontal and vertical forces are carried entirely by the structural walls, no gravity load columns exist. Here, the frame action is due to the coupling of the walls by floors and spandrels. The coupling effect can be expressed by a single parameter, the height of zero moment h_o (see figure 3.3).

h_o is determined as a function of the ratio of the flexural stiffness of the slab or spandrel to the flexural stiffness of the pier $(EI_{sp}/l_o)/(EI_p/l_o)$.

Knowing h_o and M_u the shear capacity of the wall can be evaluated form:

$$V_m = \frac{M_u}{h_o} \qquad ……(3.20)$$

And using the following equation to find the yield displacement at the top of the wall:

$$\Delta_y = V_m.H_{tot}.\left(\frac{h_p.(3h_o - h_p)}{6.EI_{eff}}\right) \qquad \text{......(3.21)}$$

The effective section stiffness of the cracked section can be obtained from the bilinear moment curvature relationship (see figure 3.1):

$$EI_{eff} = \frac{M_y}{\phi_y'} \qquad \text{......(3.22)}$$

The ultimate displacement at the top of the wall is a function of the curvature ductility and the appropriate mechanism. Depending on the flexural strength ratio, hinges may form first in the spandrels leading to a Spandrel Sidesway Mechanism (SSM) or in the piers, leading to a Pier Sidesway Mechanism (PSM) as shown in Figure 3.7, (Park, 1997).

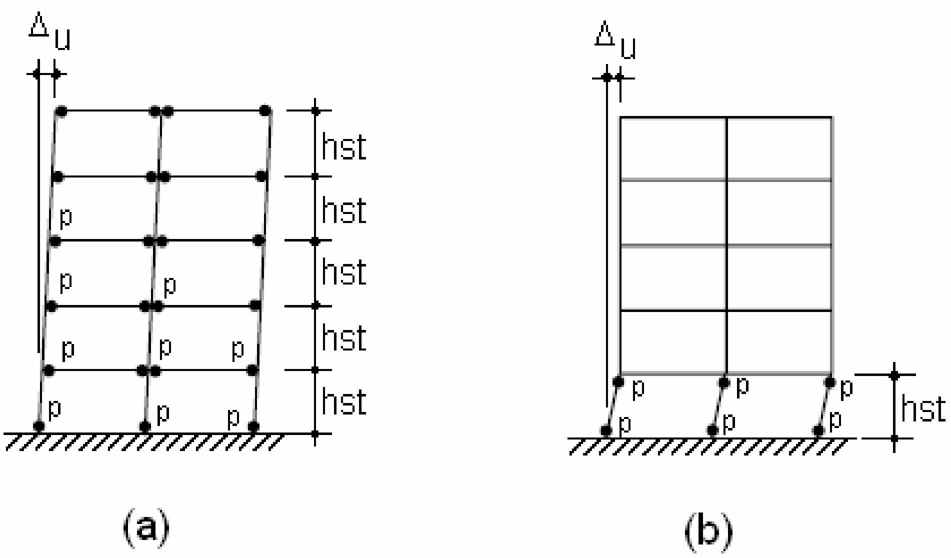

Figure 3.7: Ultimate top displacement for: (a) Spandrel sidesway mechanism (b) Pier sidesway mechanism

For SSM:

The ultimate top displacement for the SSM (Figure 3.7(a))

$$\Delta_u = \Delta_y + \left(H_{tot} - \frac{l_p}{2}\right).\theta_p = \Delta_y + \left(n.h_{st} - \frac{l_p}{2}\right).(\phi_u - \phi_y).l_p \qquad \text{......(3.23)}$$

Then the displacement ductility can be determined using the next equation:

$$\mu_w = 1 + (\mu_\phi - 1).\frac{6.h_o.l_p}{H_{tot}.h_p.(3h_o - h_p)}.\left(n.h_{st} - \frac{l_p}{2}\right) \qquad \text{......(3.24)}$$

For PSM:

The ultimate top displacement for the PSM (Figure 3.7(b))

$$\Delta_u = \Delta_y + \left(h_{st} - \frac{l_p}{2}\right).\theta_p = \Delta_y + \left(h_{st} - \frac{l_p}{2}\right).(\phi_u - \phi_y).l_p \qquad \text{......(3.25)}$$

Then the displacement ductility can be determined using the following equation:

$$\mu_w = 1 + (\mu_\phi - 1).\frac{6.h_o.l_p}{H_{tot}.h_p.(3h_o - h_p)}.\left(h_{st} - \frac{l_p}{2}\right) \qquad \text{......(3.26)}$$

Accordingly the three parameters defining the bilinear capacity curve of a wall with coupling effects V_m, Δ_y and Δ_u are determined. The capacity curve of the building in one direction can then be obtained by combining the capacity curves of all the walls acting in this direction.

3.6.4 Shear strength

It is significant to check shear strength for reinforced concrete elements since they cannot fail in flexure only, but also in shear.

Failure of reinforced concrete, RC, elements in a shear mode causes a sudden failure without warnings, where the ductility of the element decreases significantly.

The shear strength of a wall V_{shear} is considered as the sum of the shear carried by concrete V_c, the shear carried by transverse reinforcement V_s, and the strength enhancement resulting from the axial compression V_n as stated below after Lang (2002):

$$V_{shear} = V_c + V_s + V_N \qquad \text{……(3.26)}$$

1. Contribution of concrete is expressed as a function of the flexural ductility such that:

$$V_c = t \,.\, z \,.\, k \,.\, \sqrt{f_c'} \qquad \text{……(3.27)}$$

Where; k is a factor that decreases from 0.29 for $\mu_\phi < 2$ to 0.1 for $\mu_\phi > 4$, t is the thickness of the wall, and z is the effective depth of the wall section normally taken as z = 0.8 l_W.

2. Contribution of transverse reinforcement is based on a 30^o truss mechanism, therefore:

$$V_s = A_{sh}.f_{yh}.\frac{z'}{s_h}.\cot 30^{\circ} \qquad \text{......(3.28)}$$

Where; z' is the distance between centers of the peripheral transverse reinforcement, A_{sh} is the area of a set of transverse reinforcement, f_{yh} is the yield strength, and s_h is the spacing of the transverse reinforcement.

3. Finally, the contribution of the axial load acting on a wall is considered as an enhancement of the shear strength, thus:

$$V_N = N.\frac{l_w - N/(t.f_c^{'})}{2h_o} \qquad \text{......(3.29)}$$

Where N is the axial load, l_w is the length of the wall, and h_o is the height of zero moment as discussed before.

3.7 Identification of damage grades according to EMS

Different damage grades of reinforced concrete building are identified by the European Macroseismic Scale (EMS 98).

EMS defines the points on the capacity curve at which the building enters the next damage grade. Appendix A.1 gives the classification of damage to reinforced concrete buildings at each grade. Subsequently, each damage grade is defined according to the EMS and how this definition can be interpreted to a point on the capacity curve.

Damage Grade 1:

EMS definition $\Rightarrow$ *Negligible to slight damage (no structural damage, slight nonstructural damage)* Fine cracks in plaster over frame members or in walls at the base. Fine cracks in partitions and infills.

Interpretation $\Rightarrow$ the point of the onset of cracking, i.e. the point when the tensile stress at the extreme tensile fiber of the wall section reaches the tensile strength of concrete.

The curvature at the onset of cracking is then given as:

$$\phi_{cr} = \frac{M_{cr}}{M_u} . \phi_y \qquad \text{……(3.30)}$$

The shear force and the top displacement at the onset of cracking for cantilever walls are given in equations (3.31) and (3.32):

$$V_{cr} = \frac{M_{cr}}{\left(\frac{2}{3} . H_{tot}\right)} \qquad \text{……(3.31)}$$

$$\Delta_{cr} = \frac{11}{40} . \phi_{cr} . H_{tot}^2 \qquad \text{……(3.32)}$$

Similarly for coupled walls:

$$V_{cr} = \frac{M_{cr}}{h_o} \qquad \text{……(3.33)}$$

$$\Delta_{cr} = V_{cr} . H_{tot} . \left(\frac{h_p . (3h_o - h_p)}{6.EI_{eff}} \right) \qquad \text{……(3.34)}$$

The wall or column that cracks first; the one having the smallest value for Δ_{cr} determines damage grade 1. Thus, the couple (Δ_{cr}, V_{cr}) for that wall determines the point on the capacity curve of the building at which the building enters damage grade 1. Before this point the building is considered to be undamaged.

Damage Grade 2:

EMS definition $\Rightarrow$ *Moderate damage (slight structural damage, moderate non-structural damage)*. Cracks in columns and beams of frames and in structural walls. Cracks in partition and infill walls, fall of brittle cladding and plaster. Falling of mortar from joints of wall panels.

Interpretation $\Rightarrow$ the point at which the first wall yields and the stiffness of the building starts to reduce. The corresponding displacement is the smallest yield displacement of all the walls of a building $\Delta_{y,min}$.

The couple $(\Delta_{y,min}, V_b(\Delta_{y,min}))$ determines the point on the capacity curve of the building at which the building enters damage grade 2. Before this point all walls behave linearly and elastically and the stiffness of the building is equal to k.

Damage Grade 3:

EMS definition $\Rightarrow$ *Substantial to heavy damage (moderate structural damage, heavy non-structural damage).* Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of concrete cover, buckling of reinforced rods. Large cracks in partitions and infill walls. Failure of individual infill panels.

Interpretation $\Rightarrow$ the point at which the stiffness of the building tends to zero, which usually corresponds to the point at which the last wall yields. The corresponding displacement is the maximum yield displacement of all the walls of a building $\Delta_{y,max}$. The couple $(\Delta_{y,max}, V_b(\Delta_{y,max}))$ determines the point on the capacity curve of the building at which the building enters damage grade 3.

Damage Grade 4:

EMS definition $\Rightarrow$ *very heavy damage (heavy structural damage, very heavy non-structural damage).* Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns. Collapse of a few columns or of a single upper floor.

Interpretation $\Rightarrow$ the point at which the first wall enters a rocking mode (still did not collapse). Hence, when the smallest ultimate displacement of all the walls of a building $\Delta_{u,min}$ is reached, the building is considered to be very heavily damaged.

The couple $(\Delta_{u,min}, V_b(\Delta_{u,min}))$ determines the point on the capacity curve of the building at which the building enters damage grade 4. Beyond this point the base shear of the building starts to reduce.

Damage Grade 5:

EMS definition $\Rightarrow$ *Destruction (very heavy structural damage).*Collapse of ground floor or parts of the building.

Interpretation $\Rightarrow$ buildings are assumed to be destroyed if the base shear reduces below a certain limit which is considered to be 2/3 of its maximum value.

3.8 Seismic Demand

The seismic demand is determined using a response spectrum. The design response spectrum is an elastic response spectrum for 5 percent equivalent viscous damping used to represent the dynamic effects of the Design Basis Ground Motion for the design of structures, See Figure 3.8.

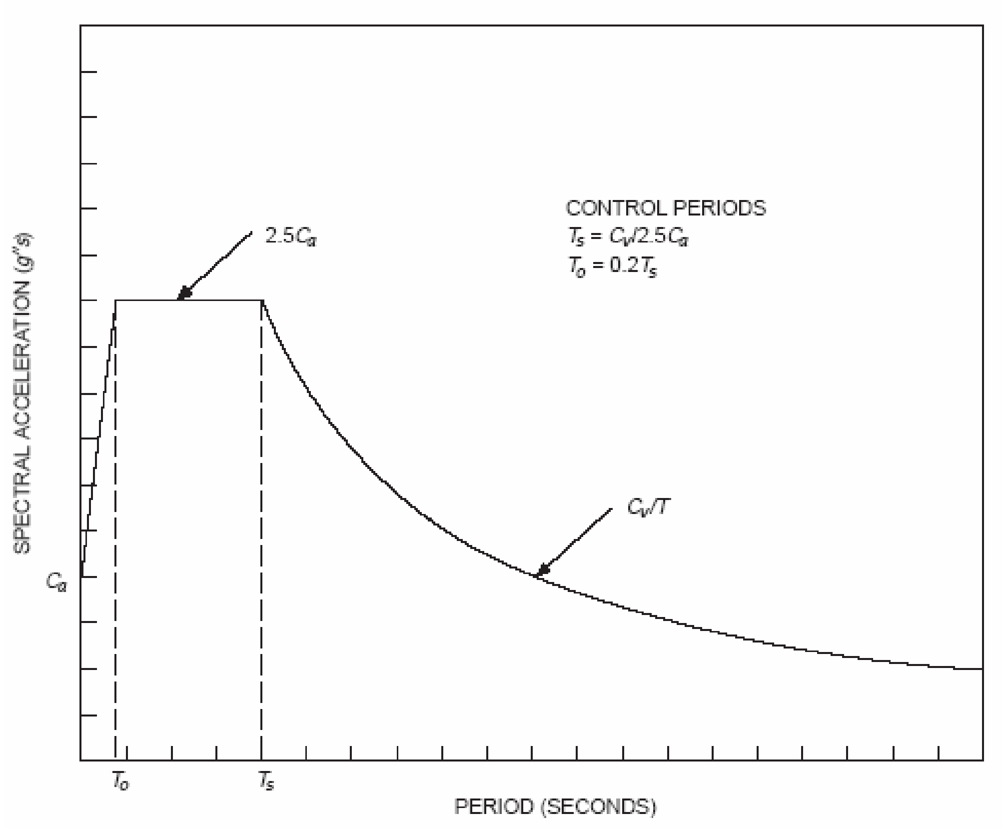

Figure 3.8: Design Response Spectra, UBC 1997

This response spectrum may either be a site-specific spectrum based on geologic, tectonic, seismological and soil characteristics associated with a specific site or may be a spectrum constructed in accordance with the spectral shape (UBC, 1997).

The following steps were used to determine the seismic demand on a building:

1. First, the mass matrix [M] of the building is determined by assuming that masses are concentrated at the floor levels and the masses of the walls and columns are divided between the two levels above and below.

2. The stiffness matrix [K] of the building is formulated by knowing the stiffnesses of all walls and columns at each floor level.

3. Performing the eigenvalue solution via the frequency equation :

$$\left|[K]-\omega^2[M]\right|=0 \qquad \text{......(3.34)}$$

And solving for ω^2 (the eigenvalues), the first mode is related to the smallest value of those eigenvalues.

Then finding the mode shape $\{\Phi\}$ (eigenvector) associated with the first mode of vibration.

4. Modal analysis:

To perform this analysis the earthquake excitation vector $\{R\}$ is required. $\{R\}$ is defined as the vector of rigid body displacements resulting from a unit support displacement in the direction of ground motion.

The modal participation factor for the first mode can be found:

$$\Gamma=\frac{\{\Phi\}^T.[M].\{R\}}{\{\Phi\}^T.[M].\{\Phi\}} \qquad \text{......(3.35)}$$

Subsequently, the period of the structure for the first mode is determined:

$$T_1=\frac{2\pi}{\omega} \qquad \text{......(3.36)}$$

The spectral acceleration value can be read directly from the response spectrum curve; hence the spectral displacement is given as:

$$S_d=\frac{S_a}{\omega^2} \qquad \text{......(3.37)}$$

Thus, the required elastic top displacement of the building is evaluated by:

$$\Delta_{be} = \Gamma . S_d(T_1) \qquad \text{......(3.38)}$$

And the required elastic base shear of the building is

$$V_{be} = K . \Delta_{be} \qquad \text{......(3.39)}$$

3.9 Vulnerability function

As discussed previously in section 3.3, the vulnerability function is constructed by plotting the spectral displacement S_d versus the top displacement of the building Δ for each damage grade.

The use of these damage grades allows a "visual" interpretation of the damage and a physical condition of the building, which is very useful to judge the performance of the structure under the expected earthquake.

CHAPTER 4 Seismicity of Amman City

4.1 Introduction

Tectonic earthquakes result from motion between a number of large plates comprising the Earth's crust or lithosphere (about 15 large plates, in total, see Figure 4.1). These plates are driven by the convective motion of the material in the Earth's mantle, which in turn is driven by heat generated at the Earth's core.

Relative plate motion at the fault interface is constrained by friction and/or asperities (areas of interlocking due to protrusions in the fault surfaces). However, the strain energy which accumulates in the plates eventually overcomes any resistance and causes slip between the two sides of the fault. This sudden slip releases large amounts of energy, which is the earthquake.

Siesmicity of a region determines the extent to which earthquake loadings may control the design of any structure for that location, and the principal indicator of the degree of seismicity is the historical record of the earthquakes that have occurred in the region.

Jordan occupies a major portion of the Arabian plate's northwestern side and is bordered on the west by the African plate boundary, namely the Jordan-Dead sea transform fault system (JDS) which extends from the Gulf of Aqaba, in the northern part of the Red Sea to south Turkey, see

Figure 4.2. The length of the JDS is about 1100 km, and is considered a good example of recent active continental transforms of the world.

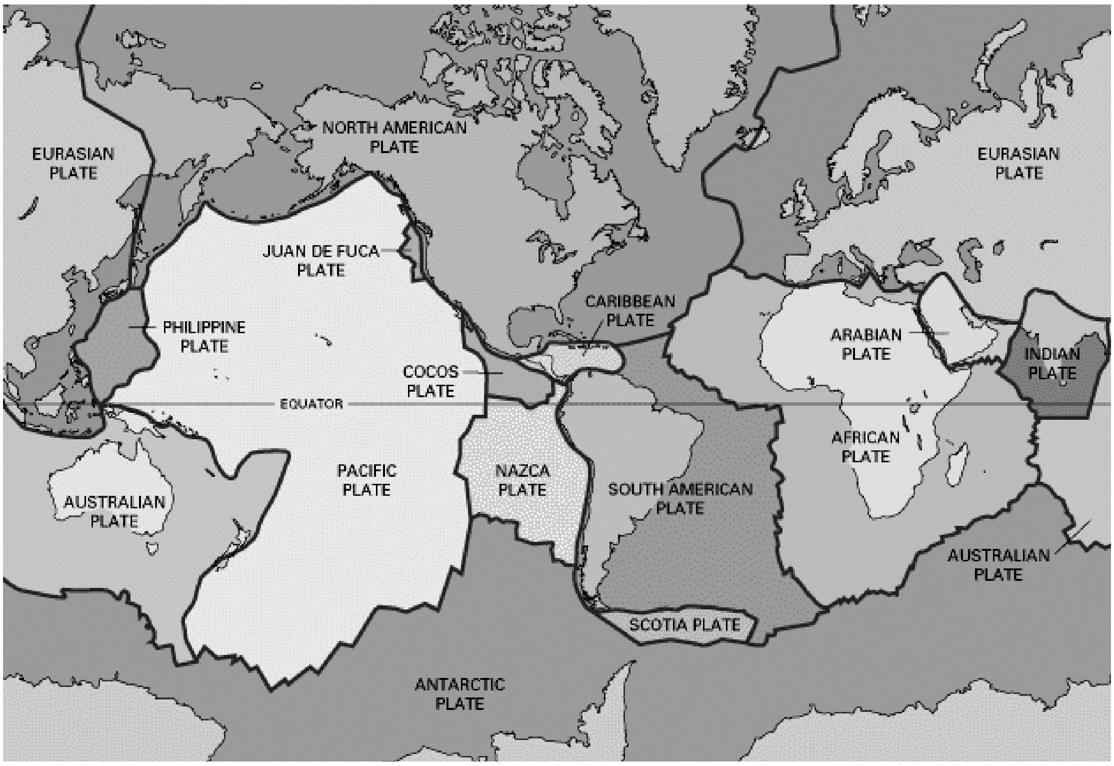

Figure 4.1: Global tectonic plate boundaries (Chen and Scawthorn, 2003)

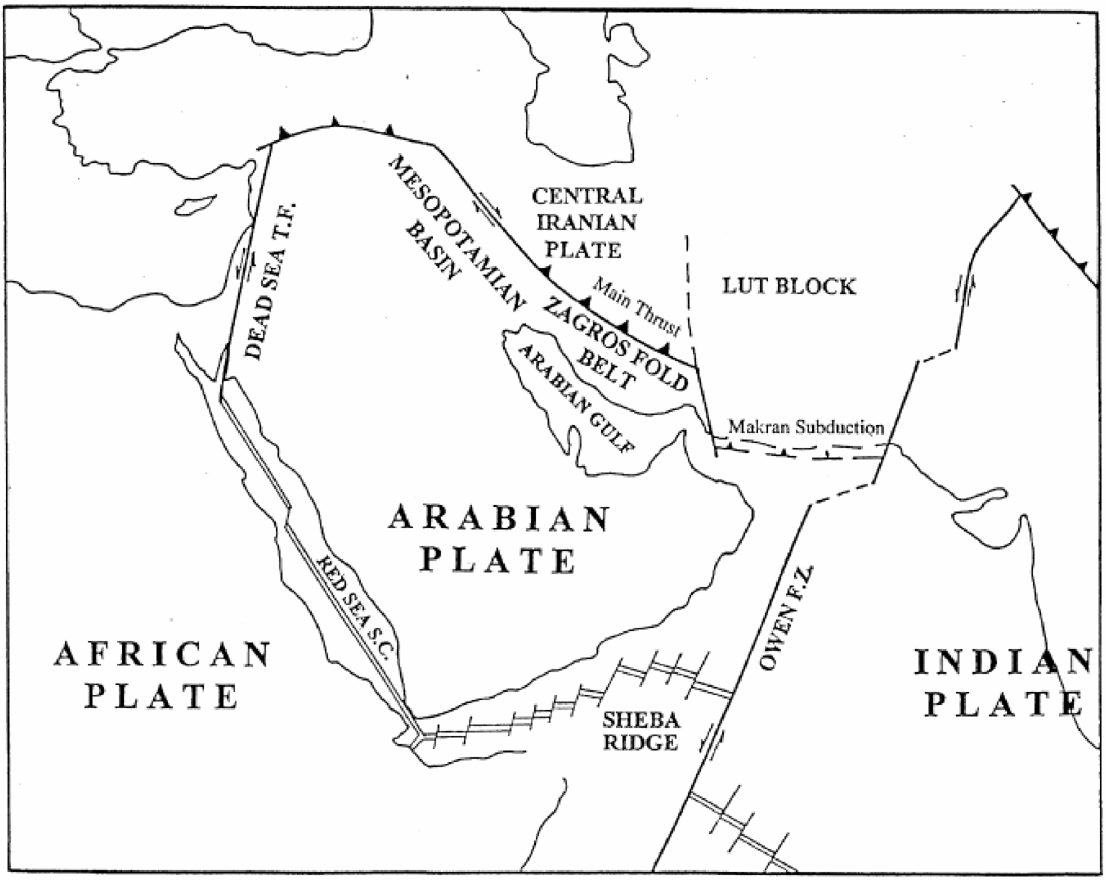

Figure 4.2: Distribution of the major tectonic plates in the Middle Eastern region (Bou-Rabee and VanMarcke, 2001).

4.2 Historical perspective

Throughout the last four thousand years, the Middle East has suffered from many disasters because of earthquakes. The ancient cities of Jordan such as Amman, Irbid, Jerash, Aqaba, El-Salt and Al-Karak (see Figure 4.3) have suffered from many earthquakes causalities, the effect of which can still be observed in the old ruins of these cities.

The seismic activity in the last 80 years is observed to be low. However, the earthquake of 1927 had a magnitude of 6.25 which caused the death of 342 people, and the Gulf of Aqaba earthquake of 1995 with a magnitude of 6.5, strongly indicated that destructive earthquakes are likely to affect Jordan in the future.

Figure 4.3: Jordan map and locations of major cities (www.ammancity.gov.jo)

In recent years, the volume of constructions in Jordan has increased significantly especially in the Greater Amman area, which represent about 40% of the total population of Jordan. This fact makes the evaluation of the seismic hazard and the earthquake resistant design for structures in this region a necessity.

4.3 Tectonic setting of Jordan

Geological structures that branch of the transform are known to be associated with additional hazard. Statistical analysis of historical and instrumental data reveals that seismicity of the area is divided into two successive time periods: the first is active with a maximum probable earthquake magnitude of 6-7 on Richter scale which is expected to occur every 40-80 years. This active period lasts for an average of 160 years and is followed by a less active period with lower magnitude earthquakes that are not expected to exceed 6 on Richter scale, and lasts for an average period of 220-230 years (Jimenez et.al., 2006).

Local faults are distributed in Jordan, such as Amman-Hallabat structure that is composed of folds and faults. This fault extends from the Jordan Valley to the northeast passing through Amman and ending at the eastern part of Zarqa city, and is located on the middle extension of the Syrian Arc.

Al-Karak – Al-Fayha fault is another local main structure that is branching from the Lisan-Dead Sea Peninsula and is directed toward the southeast passing through the Saudis boundaries. Al-Sarhan depression is another major tectonical feature in Jordan. It is composed of normal faults directed northwest-southeast and covered by basalts.

4.4 Tectonic setting of Amman

Amman is situated on the Amman –Hallabat system which extends from Siyagha on the north east corner of the Dead Sea all the way through the eastern part of Zarqa city. Main faults in Amman are:

1. Al-Quweismeh fault.
2. Umm Al-Heran fault
3. Al-Mugabalen fault.
4. Umm Al-Deba.
5. Wadi Saqra.
6. Al-Hussein sports city.

4.5 Response spectrum of Amman

Response spectrum is the most useful measure of earthquakes for engineers. Response spectrum is a chart that plots the response of a single degree of freedom oscillator to a specific earthquake. By varying the frequency or the period and the damping ratio of the system, the maximum structural response quantities can be evaluated in terms of maximum displacement, maximum velocity, and maximum acceleration of the system, (Armouti, 2004).

The first edition of the "Jordanian Code for Seismic Resistant Structures" divided Jordan into four zoning regions according to the expected seismic hazard and intensity. The zoning is based on 10% probability that the assigned ground acceleration of each zone will be exceeded in 50 years. The four zones of the Jordanian Code are illustrated in Figure 4.4. Each zone is defined by a Z-factor as shown in Table 4.1.

Table 4.1: Seismic zones and factors

Zone	1	2A	2B	3
Z	0.075	0.15	0.20	0.30

For each seismic zone the Jordanian code assigns two coefficients; C_a for acceleration and C_v for velocity, see Table 4.2 and Table 4.3. Both coefficients are specified according to the soil profile of the site as clarified in Table 4.4.

Table 4.2: Seismic Coefficient C_a

Soil profile	Seismic zone factor, Z			
	0.075	0.15	0.20	0.30
S_A	0.06	0.12	0.16	0.24
S_B	0.08	0.15	0.20	0.30
S_C	0.09	0.18	0.24	0.33
S_D	0.12	0.22	0.28	0.36
S_E	0.19	0.30	0.34	0.36
S_F	Site specific investigation is required			

Table 4.3: Seismic Coefficient C_v

Soil profile	Seismic zone factor, Z			
	0.075	0.15	0.20	0.30
S_A	0.06	0.12	0.16	0.24
S_B	0.08	0.15	0.20	0.30
S_C	0.13	0.25	0.32	0.45
S_D	0.18	0.32	0.40	0.54
S_E	0.26	0.50	0.64	0.84
S_F	Site specific investigation is required			

Table 4.4: Soil profile types

Soil profile	Soil profile generic description	Average soil properties for top 30m of soil		
		Shear wave velocity(m/s)	Standard penetration test	Undrained shear strength (kPa)
S_A	Hard rock	>1500	—	—
S_B	Rock	760 to 1500		
S_C	Very dense soil and soft rock	360 to 760	>50	>100
S_D	Stiff soil profile	180 to 360	15 to 50	50 to 100
S_E	Soft soil profile	<180	<15	<50
S_F	Site specific investigation is required			

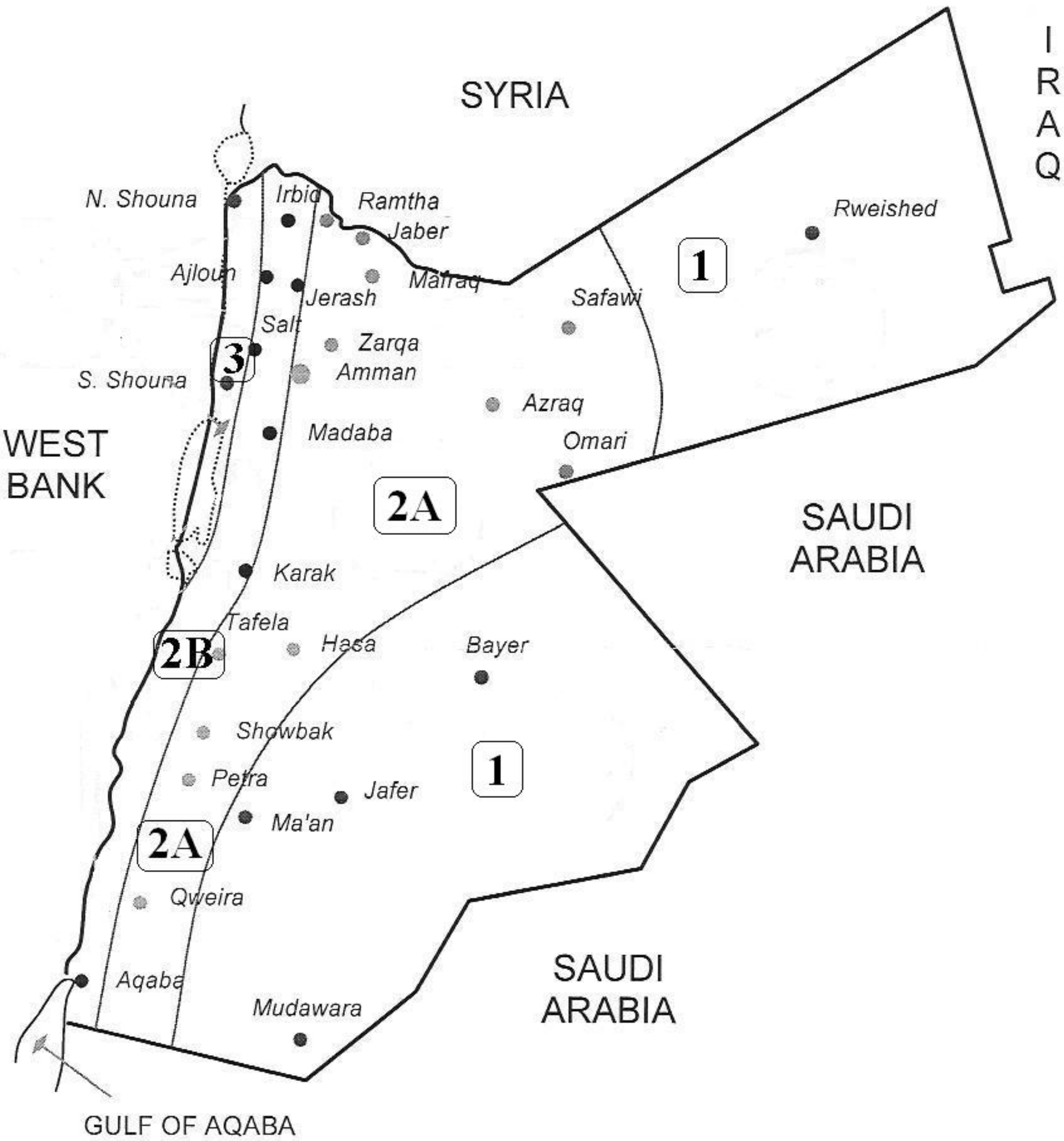

Figure 4.4: Seismic zoning map of Jordan, A.2

The design response spectrum for each zone is given by the code for each soil profile as a function of C_a and C_v. Amman city is located in zone 2A as shown in the map, therefore the design response spectrum for this specific zone was plotted for soil types S_B, S_C, and S_D since those soil types represent almost 100% of the soil characteristic in Amman.

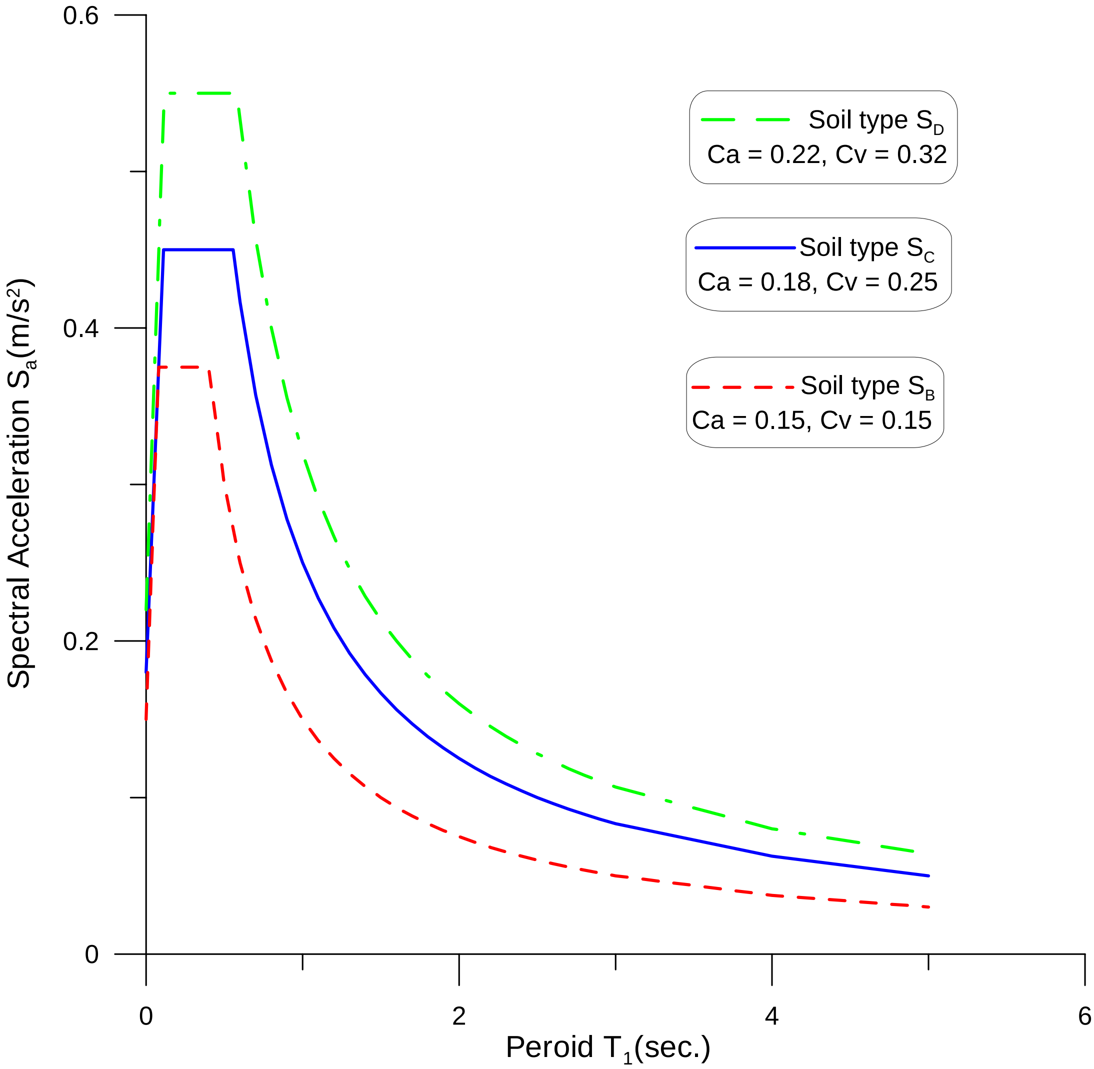

Figure 4.5: Elastic Design Response Spectrum, Zone 2A
(According to the Jordanian Code for Earthquake Resistant Structures, 2005)

CHAPTER 5 Statistical Information of Residential Buildings in Amman

5.1 Introduction

There is no doubt that the assessment of an existing RC building is much more difficult than the design of a new one, because it requires work on structures of which only limited knowledge can be obtained. There have been difficulties in determining sufficiently accurate knowledge of some structural data (e.g. material strength, soil characteristic, lateral load resisting system and the availability of reinforced shear walls). Therefore, database was accumulated by the author from technical documentation of existing residential buildings available in the public domain.

The most valuable data was obtained through reference to statistical information collected from documents available at the Jordan Engineer's Association and the Greater Municipality of Amman (Umm Al-Summaq branch).

Although none of the buildings in the sample was investigated in the field, however they were investigated through their official documents that were submitted to the local authorities for the request of a construction permit. Those documents were very useful since they included all the detailed engineering drawings for the proposed building i.e. architectural plans and elevations, structural plans and reinforcement details.

5.2 Survey Data

A population of 110 residential building in Amman was studied during this survey, and detailed information about each sample was accumulated.

Table 5.1a through Table 5.6b presents the collected information of the whole random samples.

The following definitions are necessary to clarify the tables:

a. *RC core*: reinforced concrete service core, usually used as the elevator shaft.
b. *URC core*: unreinforced concrete (plain concrete) service core, usually used for as elevator shaft.
c. *RC stair*: reinforced concrete walls supporting the stairs
d. *URC stair*: unreinforced concrete (plain concrete) walls supporting the stairs.
e. *RC walls*: reinforced concrete walls supporting the structure other than those used for the elevator and the stairs.
f. *Number of floors*: is the total number of floors including above ground floors and basement floors.
g. *Stone*: exterior wall material for most of the buildings consists of 100mm block, 100mm plain concrete, and 50mm stone cladding.
h. *Class A*: is the official classification of the building and has a front setback =5m, side setbacks =5m and, rear setback =7m

i. *Class B*: is the official classification of the building and has a front setback =4m, side setbacks =4m and, rear setback =6m

j. *Class C*: is the official classification of the building and has a front setback =4m, side setbacks =3m and rear setback =4m

k. *Class D*: is the official classification of the building and has a front setback =3m, side setbacks =2.5m and, rear setback =2.5m

l. *Sym.*: the typical floor plan of the building is symmetric about one axis.

m. *Asym.*: the typical floor plan of the building is asymmetric about both axes.

n. *Reg.*: the floor configuration is regular, i.e. the plan takes almost a square or rectangular layout and has no significant physical discontinuities or irregular features such as skewness, re-entrant corners, trapezoidal shape, or wings.

o. *Irreg.*: the floor configuration is irregular, i.e. the plan has a more complicated shape, such as having setbacks or skewness. Noticing that the irregularities were only in the horizontal direction, no vertical irregularities were detected such as soft story or mass irregularity, i.e. all buildings were considered to have vertical continuity and uniformity.

p. *2apart./floor = 8apart.*: each floor is divided into two apartments having a total of eight apartments in the buildings.

q. *Seismic details - No*: the drawings of this building did not include any extra details for seismic purposes and of course no seismic calculations.

r. *Seismic details - Yes*: the drawings of this building included seismic details.

s. *Seismic details - Yes w/calc.*: the drawings of this building included not only seismic details but also a set of structural seismic calculations.

t. *Source and date - JEA Nov, 25 2006*: the statistical information about this sample was gathered from the Jordan Engineer's Association at the date of 25 Nov 2006.

u. *Source and date - GMA Nov, 25 2006*: the statistical information about this sample was gathered from the Greater Municipality of Amman at the date of 25 Nov 2006.

v. The empty cells indicated that the required information was not available in this specific document such as a soil test report.

Table 5.1a: Statistical Information for Samples 1 to 20, Part 1

Sample No.	Class	Location	Year of Construction Permit	Area of Land (m^2)	Number of Floors	Area of Typical Floor (m^2)	Soil Bearing Capacity (kN/m^2)	Foundation Type
1	C	Wadi seer	Nov, 2006	525	2	247	250	single & strip
2	A	Tla'a ali	Nov, 2006	883	5	330	330	single & strip
3	D	Salt	Nov, 2006	3304	1	300	150	strip & combined
4	C	Wadi seer	Nov, 2006	757	5	390	220	single
5	B	Tla'a ali	Nov, 2006	791	3	307	/	single
6	B	Um uthaina	Nov, 2006	750	5	340	/	single
7	B	Wadi seer	Nov, 2006	900	5	160	/	single
8	A	Marj alhamam	Nov, 2006	1113	5	380	160	single & strip
9	C	Abu qafoor	Nov, 2006	1000	5	223	260	single
10	A	Sweileh	July, 2006	1064	4	414	180	single
11	C	Basman	Nov, 2006	520	5	250	230	single
12	D	Mugabaleen	Nov, 2006	333	5	164	/	/
13	C	Mugabaleen	Nov, 2006	/	2	120	220	single
14	C	Bader	Nov, 2006	719	3	170	/	/
15	B	Jubeiha	Nov, 2006	931	5	400	/	/
16	C	Abdoun	Nov, 2006	760	3	334	177	single & strip
17	A	Marj alhamam	Nov, 2006	933	3	320	200	single
18	C	Khelda	Oct,2006	634	5	322	376	single
19	B	Sweileh	Nov, 2006	562	3	200	270	strip & combined
20	A	Khelda	Nov, 2006	896	2	230	/	single & strip

Table 5.1b: Statistical Information for Samples 1 to 20, Part 2

Sample No.	Slab	Exterior Wall Material	Shear Walls	Beams and Columns	Seismic Details	Configuration	Description	Source and Date
1	310mm 1-way ribs	stone	rc core & stair	yes	yes	sym., reg.	1apart./floor	JEA Nov,25 2006
2	310mm 2-way ribs	stone	core & stair	yes	no	irreg.	1apart./floor	JEA Nov,25 2006
3	310mm 1-way ribs	stone	walls	yes	no	irreg.	villa	JEA Nov,25 2006
4	250mm 1-way ribs	stone	rc walls	yes	yes w/calc.	sym., reg.	2apart./floor=8apart.	JEA Nov,25 2006
5	350mm 1-way ribs	stone	stair	yes	no	sym., reg.	2apart./floor	JEA Nov,25 2006
6	310mm 1-way ribs	stone	core	yes	no	irreg.	3apart./floor=15apart.	JEA Nov,25 2006
7	310mm 1-way ribs	stone	/	yes	no	reg.	3apart./floor	JEA Nov,25 2006
8	250mm 1-way ribs	stone	core & stair	yes	no	irreg.	3apart./floor=12apart.	JEA Nov,25 2006
9	310mm 1-way ribs	stone	stair	yes	no	reg.	1apart./floor	JEA Nov,25 2006
10	310mm 1-way ribs	stone	/	yes	no	irreg.	3apart./floor	JEA Nov,25 2006
11	310mm 1-way ribs	stone	stair & wall	yes	yes	reg.	2apart./floor	JEA Nov,25 2006
12	250mm 1-way ribs	stone	/	yes	no	reg.	2apart./floor	JEA Nov,25 2006
13	250mm 1-way ribs	stone	/	yes	no	reg.	villa	JEA Nov,25 2006
14	250mm 1-way ribs	stone	/	yes	no	reg.	1apart./floor	JEA Nov,25 2006
15	310mm 1-way ribs	stone	core	yes	no	irreg.	2apart./floor	JEA Nov,25 2006
16	250mm 1-way ribs	stone	core & stair	yes	no	reg.	villa	JEA Nov,25 2006
17	310mm 1-way ribs	stone	stair	yes	yes	sym.	2apart./floor	JEA Nov,25 2006
18	250mm 1-way ribs	stone	core, wall & stair	yes	yes w/calc.	sym.	2apart./floor	JEA Nov,24 2006
19	310mm 1-way ribs	stone	walls	yes	no	irreg.	1apart./floor	JEA Nov,24 2006
20	310mm 1-way ribs	stone	core	yes	no	reg.	villa	JEA Nov,24 2006

Table 5.2a: Statistical Information for Samples 21 to 40, Part 1

Sample No.	Class	Location	Year of Construction Permit	Area of Land (m^2)	Number of Floors	Area of Typical Floor (m^2)	Soil Bearing Capacity (kN/m^2)	Foundation Type
21	B	Tareq	Nov, 2006	1013	5	250	/	single
22	C	Basman	Nov, 2006	1277	5	250	233	single
23	C	Yadodeh	Nov, 2006	319	2	150	221	single & strip
24	D	Qwesmeh	Nov, 2006	3040	1	114	250	single
25	D	Swemeh	Nov, 2006	718	5	360	100	single
26	C	Bader	Nov, 2006	500	4	219	250	single & strip
27	A	Marj alhamam	Nov, 2006	1015	2	300	/	single & strip
28	D	Abu nseir	Nov, 2006	312	4	180	200	single
29	A	Shafa badran	Nov, 2006	1000	2	250	/	single
30	C	Naser	Nov, 2006	516	2	155	/	single
31	D	Tareq	Nov, 2006	511	1	100	/	single
32	B	Shafa badran	Nov, 2006	1034	3	509	/	single
33	C	Shafa badran	Nov, 2006	642	2	300	220	single
34	D	Ras elein	Nov, 2006	291	5	138	/	single
35	B	Tareq	Nov, 2006	929	6	385	/	single
36	A	Khelda	Nov, 2006	902	5	350	/	single
37	A	Dabooq	Nov, 2006	5370	3	160	200	single
38	B	Um alsummaq	Dec,2004	762	5	320	260	single & strip
39	A	Khelda	Mar,2006	1049	5	380	/	single & strip
40	A	Dabooq	2004	/	3	340	/	single & strip

Table 5.2b: Statistical Information for Samples 21 to 40, Part 2

Sample No.	Slab	Exterior Wall Material	Shear Walls	Beams and Columns	Seismic Details	Configuration	Description	Source and Date
21	250mm 1-way ribs	stone	core	yes	no	irreg.	1apart./floor	JEA Nov,24 2006
22	310mm 1-way ribs	stone	walls	yes	no	sym., reg.	2apart./floor=9apart.	JEA Nov,24 2006
23	250mm 1-way ribs	stone	walls	yes	no	sym.	villa	JEA Nov,24 2006
24	250mm 1-way ribs	stone	/	yes	yes	sym.	1apart./floor	JEA Nov,24 2006
25	250mm 1-way ribs	stone	core & walls	yes	yes	reg.	2apart./floor	JEA Nov,24 2006
26	250mm 1-way ribs	stone	core	yes	yes	irreg.	1apart./floor	JEA Nov,24 2006
27	250mm 1-way ribs	stone	/	yes	yes	reg.	1apart./floor	JEA Nov,24 2006
28	250mm 1-way ribs	stone	core & walls	yes	yes	sym., reg.	2apart./floor=6apart.	JEA Nov,24 2006
29	250mm 1-way ribs	stone	/	yes	no	reg.	1apart./floor	JEA Nov,24 2006
30	250mm 1-way ribs	plaster	stair	yes	no	irreg.	1apart./floor	JEA Nov,24 2006
31	250mm 1-way ribs	plaster	/	yes	no	reg.	1apart./floor	JEA Nov,24 2006
32	310mm 1-way ribs	stone	core & walls	yes	no	sym.	2apart./floor	JEA Nov,24 2006
33	250mm 1-way ribs	stone	core & stair	yes	yes	sym., reg.	2apart./floor	JEA Nov,24 2006
34	250mm 1-way ribs	stone	stair	yes	no	reg.	1apart./floor	JEA Nov,24 2006
35	250mm 1-way ribs	stone	stair	yes	no	sym.	2apart./floor	JEA Nov,24 2006
36	350mm 2-way ribs	stone	core & stair	yes	no	reg.	1apart./floor	JEA Nov,24 2006
37	310mm 1-way ribs	stone	stair	yes	no	reg.	villa	JEA Nov,24 2006
38	310mm 1-way ribs	stone	rc core	yes	no	sym., reg.	2apart./floor=8apart.	GMA Nov,28 2006
39	310mm 1-way ribs	stone	rc core & urc stair	yes	no	sym.	2apart./floor	GMA Nov,28 2006
40	310mm 1-way ribs	stone	/	yes	no	irreg.	villa	GMA Nov,28 2006

Table 5.3a: Statistical Information for Samples 41 to 60, Part 1

Sample No.	Class	Location	Year of Construction Permit	Area of Land (m^2)	Number of Floors	Area of Typical Floor (m^2)	Soil Bearing Capacity (kN/m^2)	Foundation Type
41	B	Tla'a ali	2005	836	6	350	180	single & strip
42	A	Tla'a ali	2005	1315	6	500	240	single & strip
43	A	Khelda	2004	985	4	230	290	single & strip
44	B	Tla'a ali	2004	750	3	250	270	single & strip
45	C	Tla'a ali	2005	487	3	224	/	single & strip
46	B	Khelda	2004	1138	6	473	260	single & strip
47	A	Khelda	2004	1028	5	370	300	single & strip
48	A	Tla'a ali	2004	1188	7	425	280	single & strip
49	B	Jubeiha	2005	934	6	388	180	single & strip
50	A	Rabeieh	2004	1016	5	365	/	single & strip
51	A	Tla'a ali	2005	1103	5	400	400	single & strip
52	B	Tla'a ali	2004	685	6	305	/	single & strip
53	A	Khelda	2005	909	5	340	400	single & strip
54	A	Tla'a ali	2005	1006	5	400	250	single & strip
55	A	Tla'a ali	2004	998	5	360	210	single & strip
56	A	Um alsummaq	2005	692	5	250	250	single & strip
57	A	Khelda	2005	951	3	300	240	single & strip
58	B	Tla'a ali	1994	736	6	305	240	single & strip
59	A	Tla'a ali	2006	1000	6	370	280	single & strip
60	A	Khelda	2004	529	2	249	/	single & strip

Table 5.3b: Statistical Information for Samples 41 to 60, Part 2

Sample No.	Slab	Exterior Wall Material	Shear Walls	Beams and Columns	Seismic Details	Configuration	Description	Source and Date
41	310mm 1-way ribs	stone	rc core & urc stair	yes	no	sym., reg.	2apart./floor	GMA Nov,28 2006
42	310mm 1-way ribs	stone	rc core	yes	no	irreg.	2apart./floor	GMA Nov,28 2006
43	310mm 1-way ribs	stone	/	yes	no	reg.	villa	GMA Nov,28 2006
44	250mm 1-way ribs	stone	rc core & rc stair	yes	no	reg.	villa	GMA Nov,28 2006
45	310mm 1-way ribs	stone	stair	yes	no	sym., reg.	2apart./floor	GMA Nov,28 2006
46	310mm 1-way ribs	stone	core & stair	yes	no	irreg.	3apart./floor	GMA Nov,28 2006
47	310mm 1-way ribs	stone	core & stair	yes	no	sym., reg.	2apart./floor	GMA Nov,28 2006
48	400mm 1-way ribs	stone	core & stair	yes	no	reg.	2apart./floor	GMA Nov,28 2006
49	310mm 1-way ribs	stone	core & stair	yes	no	sym., reg.	2apart./floor	GMA Nov,28 2006
50	310mm 1-way ribs	stone	core & wall	yes	no	reg.	1apart./floor	GMA Nov,28 2006
51	310mm 1-way ribs	stone	core	yes	no	sym., reg.	2apart./floor	GMA Nov,28 2006
52	250mm 1-way ribs	stone	/	yes	no	sym.	2apart./floor	GMA Nov,28 2006
53	300mm 1-way ribs	stone	core & stair	yes	no	sym.	2apart./floor	GMA Nov,28 2006
54	350mm 1-way ribs	stone	core & stair	yes	no	sym.	2apart./floor	GMA Nov,28 2006
55	310mm 1-way ribs	stone	core & stair	yes	no	sym., irreg.	2apart./floor	GMA Nov,28 2006
56	310mm 1-way ribs	stone	core & stair	yes	no	sym., reg.	2apart./floor	GMA Nov,28 2006
57	250mm 1-way ribs	stone	wall	yes	no	sym.	villa	GMA Nov,28 2006
58	320mm 1-way ribs	stone	core	yes	no	sym., reg.	2apart./floor=10apart.	GMA Nov,30 2006
59	310mm 1-way ribs	stone	core	yes	no	sym., reg.	2apart./floor=10apart.	GMA Nov,30 2006
60	250mm 1-way ribs	stone	/	yes	no	irreg.	villa	GMA Nov,30 2006

Table 5.4a: Statistical Information for Samples 61 to 80, Part 1

Sample No.	Class	Location	Year of Construction Permit	Area of Land (m^2)	Number of Floors	Area of Typical Floor (m^2)	Soil Bearing Capacity (kN/m^2)	Foundation Type
61	A	Tla'a ali	2005	1151	5	410	210	single & strip
62	A	Tla'a ali	2004	1249	8	440	220	single & strip
63	A	Tla'a ali	2005	1157	6	416	230	single & strip
64	B	Tla'a ali	2005	768	5	322	300	single & strip
65	A	Tla'a ali	2005	1341	7	482	300	single & strip
66	A	Tla'a ali	2004	1089	5	390	250	single & strip
67	A	Khelda	2005	1382	6	500	200	single & strip
68	A	Tla'a ali	2005	1062	6	380	230	single & strip
69	A	Tla'a ali	2005	1133	5	440	200	single & strip
70	B	Khelda	2005	1100	6	460	260	single & strip
71	A	Tla'a ali	2005	914	8	400	230	single & strip
72	A	Khelda	2004	1037	6	370	250	single & strip
73	C	Khelda	2005	572	5	270	300	single & strip
74	A	Tla'a ali	2005	969	5	350	/	single & strip
75	B	Tla'a ali	2005	1030	5	430	220	single & strip
76	C	Tla'a ali	2004	958	7	480	220	single & strip
77	B	Tla'a ali	2005	1052	5	440	250	single & strip
78	C	Naser	2006	502	1	239	220	single & strip
79	B	Arjan	2006	954	5	430	200	single & strip
80	C	Qwesmeh	2006	579	4	265	/	single & strip

Table 5.4b: Statistical Information for Samples 61 to 80, Part 2

Sample No.	Slab	Exterior Wall Material	Shear Walls	Beams and Columns	Seismic Details	Configuration	Description	Source and Date
61	310mm 1-way ribs	stone	core	yes	no	irreg.	2apart./floor=8apart.	GMA Nov,30 2006
62	310mm 1-way ribs	stone	core	yes	no	sym.	2apart./floor=12apart.	GMA Nov,30 2006
63	250mm 1-way ribs	stone	core & stair	yes	no	sym., irreg.	2apart./floor=10apart.	GMA Nov,30 2006
64	300mm 1-way ribs	stone	core & stair	yes	no	irreg.	2apart./floor=8apart.	GMA Nov,30 2006
65	310mm 1-way ribs	stone	core	yes	no	irreg.	2apart./floor=12apart.	GMA Nov,30 2006
66	310mm 1-way ribs	stone	core	yes	no	sym., reg.	2apart./floor=8apart.	GMA Nov,30 2006
67	300mm 1-way ribs	stone	rc core, wall &stair	yes	no	sym., reg.	3apart./floor=12apart.	GMA Nov,30 2006
68	250mm 1-way ribs	stone	core & stair	yes	no	sym., reg.	2apart./floor=9apart.	GMA Nov,30 2006
69	250mm 1-way ribs	stone	core & wall	yes	no	sym., reg.	2apart./floor=8apart.	GMA Nov,30 2006
70	300mm 1-way ribs	stone	core & stair	yes	no	sym., reg.	2apart./floor=10apart.	GMA Nov,30 2006
71	310mm 1-way ribs	stone	rc core	yes	no	sym., reg.	2apart./floor=14apart.	GMA Nov,30 2006
72	310mm 1-way ribs	stone	core	yes	no	sym., reg.	2apart./floor=9apart.	GMA Nov,30 2006
73	250mm 1-way ribs	stone	rc core	yes	no	reg.	3apart./floor=12apart.	GMA Nov,30 2006
74	250mm 1-way ribs	stone	rc core & rc stair	yes	no	sym., reg.	2apart./floor=9apart.	GMA Nov,30 2006
75	250mm 1-way ribs	stone	rc wall	yes	no	sym., reg.	2apart./floor=8apart.	GMA Nov,30 2006
76	250mm 1-way ribs	stone	urc core	yes	no	sym., reg.	2apart./floor=12apart.	GMA Nov,30 2006
77	300mm 1-way ribs	stone	rc core & rc stair	yes	no	sym., reg.	2apart./floor=8apart.	GMA Nov,30 2006
78	250mm 1-way ribs	plaster	/	yes	no	sym., reg.	2apart./floor	JEA Dec,2 2006
79	250mm 1-way ribs	stone	rc walls	yes	yes w/calc.	sym., reg.	4apart./floor=16apart.	JEA Dec,2 2006
80	250mm 1-way ribs	stone	rc walls	yes	no	unsym., reg.	1apart./floor=3apart.	JEA Dec,2 2006

Table 5.5a: Statistical Information for Samples 81 to 100, Part 1

Sample No.	Class	Location	Year of Construction Permit	Area of Land (m^2)	Number of Floors	Area of Typical Floor (m^2)	Soil Bearing Capacity (kN/m^2)	Foundation Type
81	C	khrebet elsouq	2006	420	4	206	/	single & strip
82	D	Jabal elnazeef	2006	407	4	210	/	single & strip
83	A	Tla'a ali	2006	412	8	150	190	single & strip
84	D	Jabal elnozha	2006	625	4	185	/	single & strip
85	D	Hai nazzal	2006	300	2	145	/	single & strip
86	B	Tabarbor	2006	1016	7	460	260	single & strip
87	B	Tabarbor	2006	608	5	255	180	single & strip
88	B	Naour	2006	600	3	260	230	single & strip
89	B	Jubeiha	2006	755	5	270	/	single & strip
90	A	Deir ghbar	2006	1070	7	400	350	single & strip
91	C	Qwesmeh	2006	440	3	193	/	single & strip
92	C	Rusaifeh	2006	1000	2	140	/	single & strip
93	C	Khrebet elsouq	2006	600	1	250	/	single & strip
94	B	MMarj alhamam	2006	750	4	250	/	single & strip
95	D	Tabarbor	2006	300	2	180	/	single & strip
96	C	Naser	2006	514	7	230	250	single & strip
97	C	Abu nseir	2006	315	3	180	/	single & strip
98	C	Jeeza	2006	500	3	170	180	single & strip
99	B	Tareq	2006	1160	3	340	/	single & strip
100	C	Abu alanda	2006	153	1	90	150	single & strip

Table 5.5b: Statistical Information for Sample 81 to 100, Part 2

Sample No.	Slab	Exterior Wall Material	Shear Walls	Beams and Columns	Seismic Details	Configuration	Description	Source and Date
81	310mm 1-way ribs	stone	rc walls	yes	no	unsym., reg.	1apart./floor=3apart.	JEA Dec,2 2006
82	250mm 1-way ribs	plaster	/	yes	no	unsym., reg.	1apart./floor	JEA Dec,2 2006
83	250mm 1-way ribs	stone	rc stair	yes	yes	unsym., irreg.	1apart./floor=8apart.	JEA Dec,2 2006
84	250mm 1-way ribs	plaster	urc stair	yes	no	unsym., irreg.	1apart./floor=3apart.	JEA Dec,2 2006
85	250mm 1-way ribs	stone	/	yes	no	reg.	1apart./floor	JEA Dec,2 2006
86	310mm 1-way ribs	stone	rc core, wall &stair	yes	yes w/calc.	reg.	3apart./floor=18apart.	JEA Dec,2 2006
87	310mm 1-way ribs	stone	stair	yes	yes	sym., reg.	2apart./floor=8apart.	JEA Dec,2 2006
88	310mm 1-way ribs	stone	wall	yes	yes	unsym., reg.	villa	JEA Dec,2 2006
89	310mm 1-way ribs	stone	/	yes	yes	sym., reg.	2apart./floor=8apart.	JEA Dec,2 2006
90	310mm 1-way ribs	stone	rc core, wall &stair	yes	yes w/calc.	sym., reg.	2apart./floor=9apart.	JEA Dec,2 2006
91	310mm 1-way ribs	stone	walls	yes	no	unsym., reg.	2apart./floor=3apart.	JEA Dec,2 2006
92	250mm 1-way ribs	plaster	/	yes	no	reg.	villa	JEA Dec,2 2006
93	310mm 1-way ribs	stone	rc stair	yes	yes	reg.	villa	JEA Dec,2 2006
94	250mm 1-way ribs	stone	urc stair	yes	yes	irreg.	2apart./floor=7apart.	JEA Dec,2 2006
95	250mm 1-way ribs	plaster	urc stair	yes	no	unsym., reg.	1apart./floor=2apart.	JEA Dec,2 2006
96	310mm 1-way ribs	stone	urc stair	yes	yes	unsym., reg.	2apart./floor	JEA Dec,2 2006
97	250mm 1-way ribs	plaster	urc stair	yes	yes	sym., reg.	2apart./floor	JEA Dec,2 2006
98	250mm 1-way ribs	plaster	urc stair	yes	no	sym., reg.	1apart./floor=3apart.	JEA Dec,2 2006
99	310mm 1-way ribs	stone	urc stair	yes	no	sym., reg.	2apart./floor=6apart.	JEA Dec,2 2006
100	250mm 1-way ribs	plaster	urc stair	yes	no	sym., reg.	1apart./floor	JEA Dec,2 2006

Table 5.6a: Statistical Information for Sample 101 to 110, Part 1

Sample No.	Class	Location	Year of Construction Permit	Area of Land (m^2)	Number of Floors	Area of Typical Floor (m^2)	Soil Bearing Capacity (kN/m^2)	Foundation Type
101	B	Mugabaleen	June, 2007	909	5	410	200	single & strip
102	B	Wadi elseir	June, 2007	850	2	360	250	single & strip
103	C	Qwesmeh	June, 2007	490	3	260	250	single & strip
104	C	Khrebet elsouq	June, 2007	572	2	280	150	single & strip
105	C	Ras elein	June, 2007	601	5	287	280	single & strip
106	C	Khrebet elsouq	June, 2007	161	1	147	/	single & strip
107	C	Qwesmeh	June, 2007	780	1	215	/	single & strip
108	B	Jubeiha	June, 2007	701	3	314	200	single & strip
109	B	Wadi elseir	June, 2007	760	5	330	230	single & strip
110	A	Mugabaleen	May, 2007	1580	3	300	290	single & strip

Table 5.6b: Statistical Information for Sample 101 to 110, Part 2

Sample No.	Slab	Exterior Wall Material	Shear Walls	Beams and Columns	Seismic Details	Configuration	Description	Source and Date
101	300mm 1-way ribs	stone	rc core, wall & stair	yes	yes w/calc.	sym., reg.	2apart./floor=8apart.	JEA June,2 2007
102	300mm 1-way ribs	stone	rc core & stair	yes	yes	unsym., irreg.	villa	JEA June,2 2007
103	300mm 1-way ribs	stone	rc stair	yes	no	unsym., irreg.	2apart./floor=3apart.	JEA June,2 2007
104	310mm 1-way ribs	stone	rc stair	yes	yes	sym., reg.	villa	JEA June,2 2007
105	250mm 1-way ribs	stone	rc stair	yes	yes	sym., reg.	2apart./floor=8apart.	JEA June,2 2007
106	250mm 1-way ribs	plaster	rc stair	yes	no	sym., reg.	1apart./floor	JEA June,2 2007
107	250mm 1-way ribs	plaster	rc stair	yes	no	sym., reg.	1apart./floor	JEA June,2 2007
108	320mm 1-way ribs	stone	rc stair	yes	no	unsym., irreg.	1apart./floor=3apart.	JEA June,2 2007
109	310mm 1-way ribs	stone	rc core & stair	yes	yes	sym., reg.	2apart./floor=8apart.	JEA June,2 2007
110	250mm 1-way ribs	stone	rc core & stair	yes	yes	unsym., irreg.	1apart./floor=2apart.	JEA June,2 2007

5.3 Statistical Charts

Figure 5.1 through Figure 5.7 summarized the results of the statistics as follows:

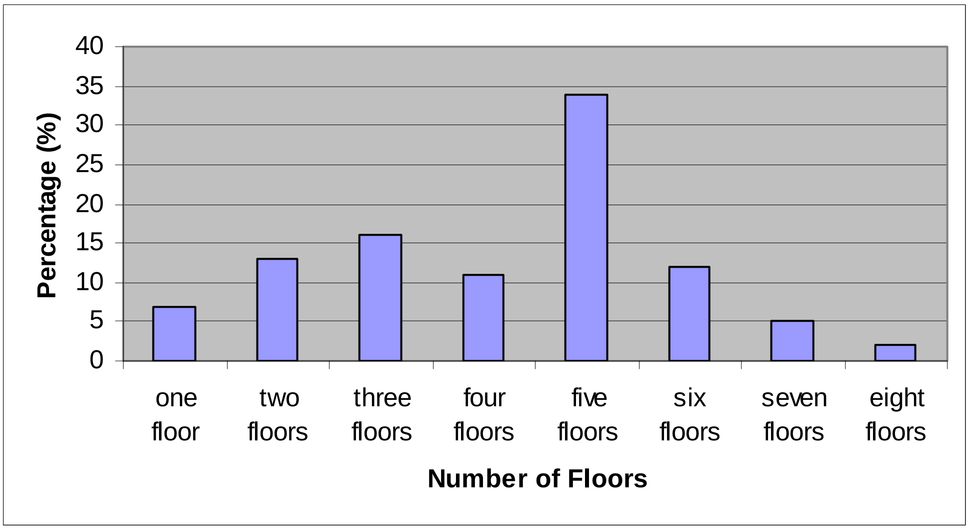

Figure 5.1: Number of Floors (including basements)

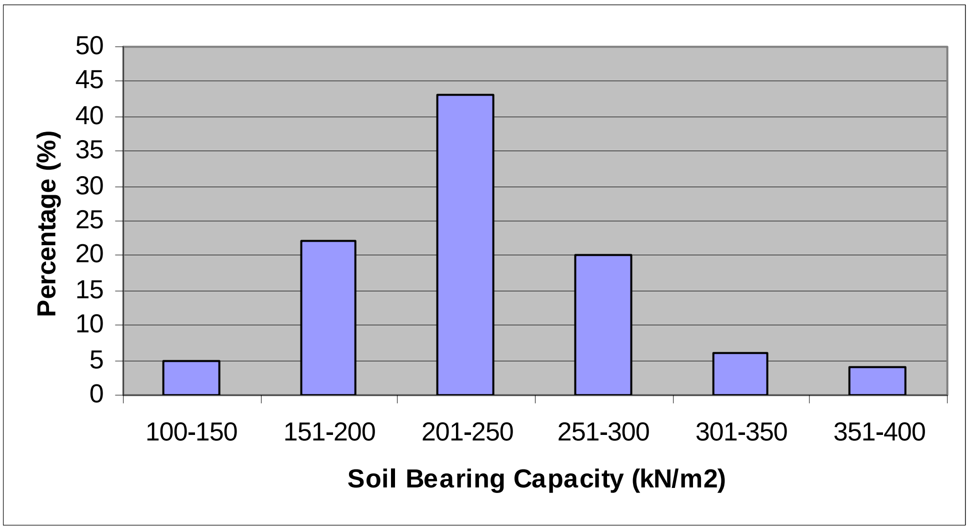

Figure 5.2: Soil Bearing Capacity

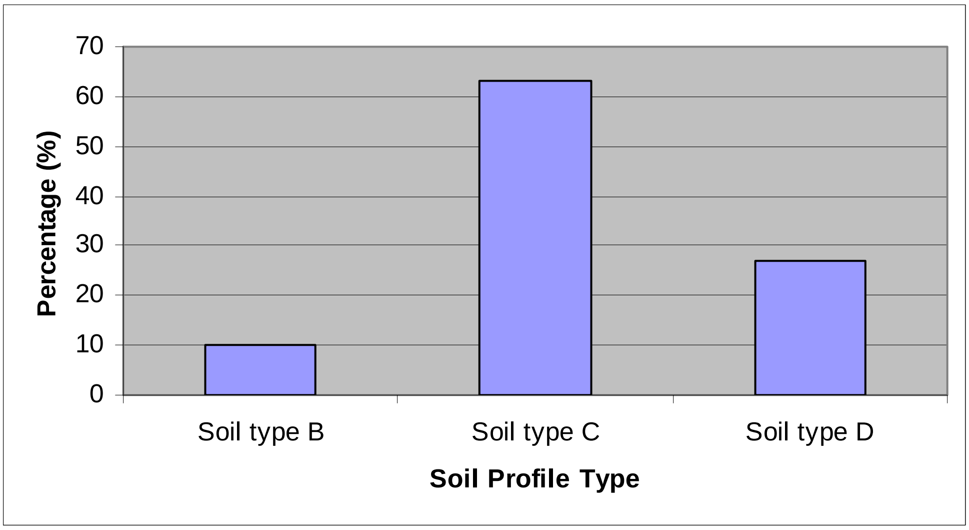

Figure 5.3: Soil Profile Types

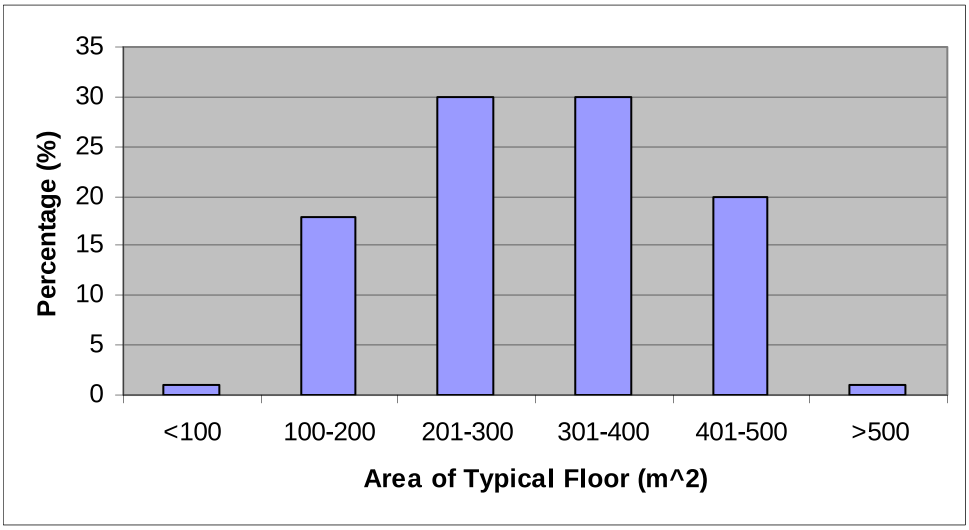

Figure 5.4: Area of Typical Floor

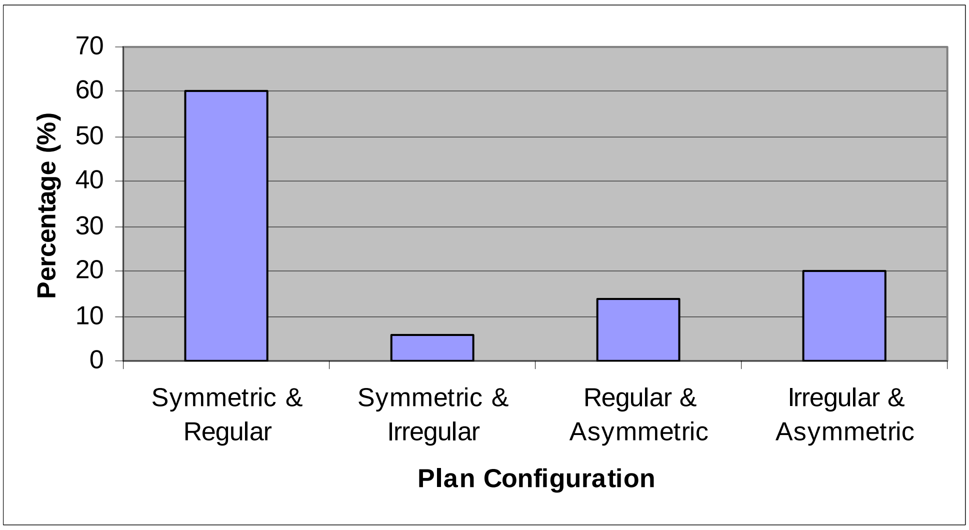

Figure 5.5: Categories of Plan Configuration

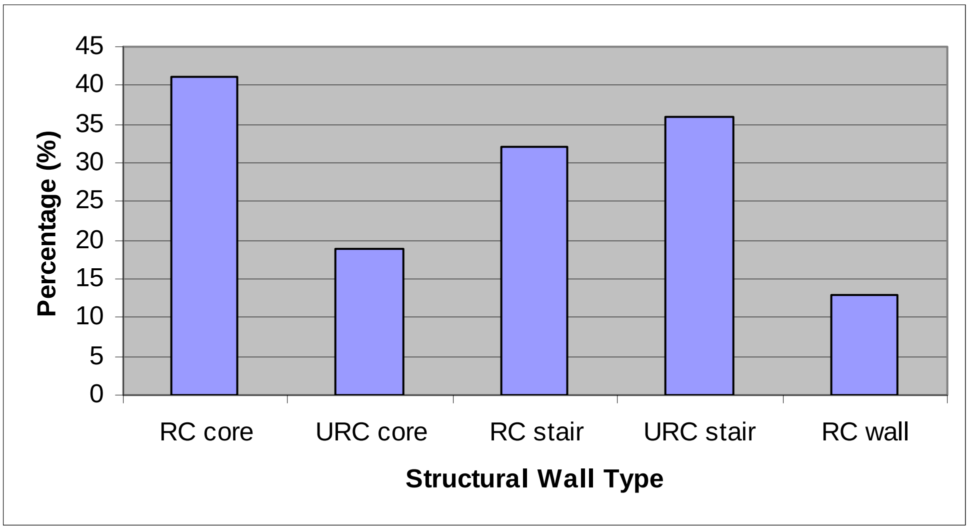

Figure 5.6: Types of Structural Walls

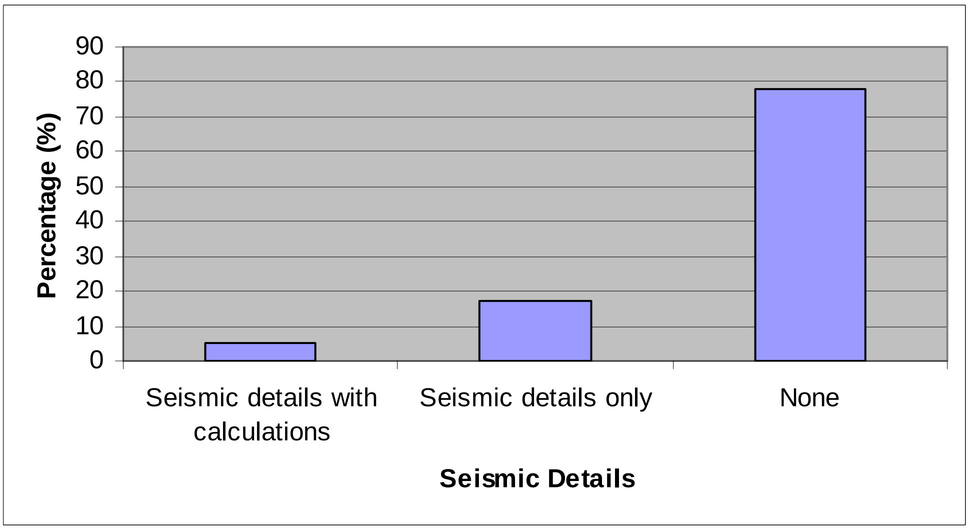

Figure 5.7: Seismic Details Before Seismic Code Enforcement

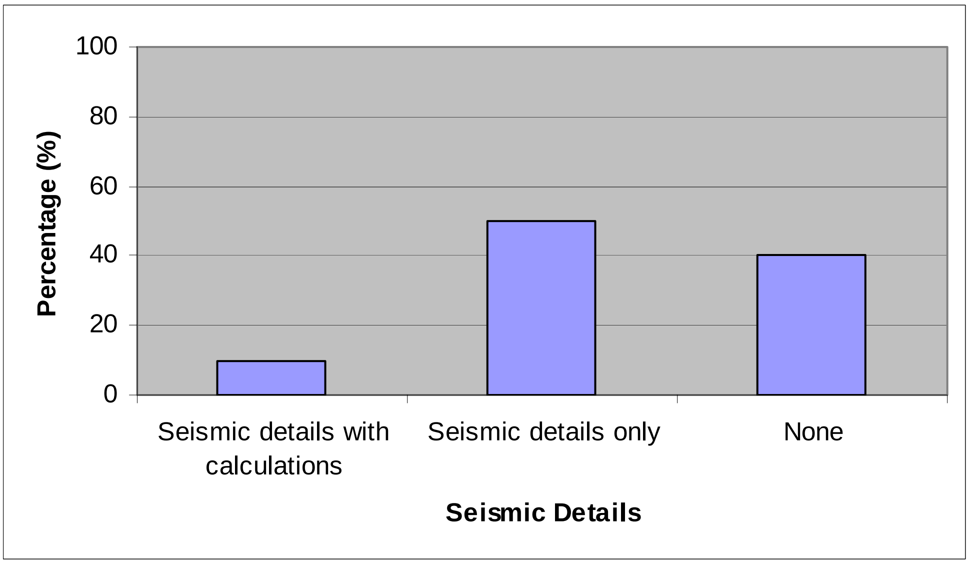

Figure 5.8: Seismic Details After Seismic Code Enforcement

5.4 Discussion of survey data

The data was gathered through two stages; the first stage was between Nov, 2006 and Dec, 2006 and the second stage during June, 2007.

During the first stage, one hundred samples were recorded, and that was prior to putting the new Jordanian Code for Earthquake-Resistant Buildings into practice. The availability of seismic calculations and details was detected for this stage (see Figure 5.7).

The second stage included ten samples. The target of this second collection was to detect the effect of applying the new seismic code, Figure 5.8). It was evident that the percentage of buildings designed without seismic details has decreased significantly from 78% before the code to 40% after the code. Likewise, the percentage of documents with seismic details and calculations has doubled (5% before the code to 10% after code), which is a good sign for a major improvement in the performance of residential buildings in Amman.

Concerning the material strength, it was noticed that 100% of the samples had a concrete compressive strength f_{cu} = 25 MPa, and the yield strength of steel f_y = 420 MPa. Hence, the material strength was not an issue in this study.

As seen in Figure 5.2 the soil bearing capacity of the samples ranged between 100 to 400 kN/m^2, referring to Bowels (1997) the bearing capacity value was related to the soil profile type based on the UBC code (which has the same soil type classification as the Jordanian code, see Table 4.4), and the following results were deduced and arranged in Table 5.13:

Table 5.13: Soil types with corresponding bearing capacities

Soil profile type	Bearing capacity, q (kN/m^2)
S_A	$q > 400$
S_B	$400 \geq q > 300$
S_C	$300 \geq q > 200$
S_D	$200 \geq q > 100$
S_E	$q \leq 100$

Thus, Figure 5.3 shows the percentage of each soil type. Soil types S_A and S_E were not found in the statistics. And in view of the fact that 90% of the population has a soil type S_C or S_D, those two types of soils were considered in the study.

Figure 5.5 exhibits the percentage of each category of plan configurations, and it can be realized that 60% of the population have a symmetric and regular plan. Therefore, analyzed models were assumed to be symmetric and regular through this study. Besides, the used vulnerability method (Kerstin Lang's model) does not clearly monitor the effect of horizontal and vertical irregularities on the seismic behavior of the structure.

Figures 5.1 and 5.4 display the number of floors and the area of typical floor, respectively. The percentages of those two features were helpful in selecting the appropriate characteristics of the studied models, in order to select the models that are able to comprehend the largest portion of buildings in Amman.

Figure 5.6 illustrates the variety of shear wall types available in residential buildings in Amman. It can be noticed that 41% of the buildings have a reinforced concrete core (lift shaft), while 19% have unreinforced cores. Moreover, 32% of the buildings have reinforced concrete walls supporting the stairs, whereas 36% of the walls supporting stairs are unreinforced.

Generally speaking;

- 60% of the total buildings contain an elevator, and all of those buildings have a concrete wall surrounding the elevator (either reinforced or unreinforced).
- 100% of total buildings have stairs, but almost 70% of them have a concrete wall supporting the stairs (either reinforced or unreinforced), while the rest are supporting the stairs by beams and columns.

- 13% of the total buildings contain reinforced shear walls other than those around stairs and elevators.
- 90% of the reinforced shear walls (core, stair or wall) are reinforced with minimum reinforcement; typically ϕ10mm@200mm spacing each way, each face. Only 10% are actually designed.

CHAPTER 6 Vulnerability Study Applied to Residential Buildings in Amman

6.1 Introduction

The focus of this study is buildings used as housing. For this reason typical designs were selected to reflect the actual composition of those buildings, and depending on the depiction of the statistics to capture the appropriate characteristics of those prototypes.

In the next section a brief historical review for the history of buildings in Amman is introduced. Then Section 6.3 describes each one of the chosen prototypes, in order to apply the vulnerability method to them with detailed procedure of calculations in Section 6.4.

Section 6.5 demonstrates the vulnerability results, and displays all related curves for each one of the prototypes. Finally, Section 6.6 analyzes the vulnerability results for each model.

6.2 Historical background of residential buildings in Amman

In ancient times, the architectural scene of Jordan was rich with sites such as Petra, Jerash, Umayyad desert palaces, and crusader castles.

In the early sixteenth century, during the Ottoman era, the area was divided into administrative provinces including Bilad al-sham and Hijaz. Jordan was a part of Bilad al-sham with Syria, Lebanon, and Palestine. At that time, Jordan the least urbanized part of Bilad al-sham, was dominated by Bedouin tribes.

The modern period begins with the founding of the Emirate of Transjordan in 1921. During this period Architects worked on documenting and renovating many of the residential, cultural and heritage sites. Residential houses built in the newly-chosen capital of Amman (from about 1920 to 1950) were architecturally significant; they reflected the events of that era and the major improvements of citizens living conditions.

The newly established government of Transjordan ensured security provided services and encouraged the development of settled life. This has made of Transjordan state a magnet that attracted immigrants from surrounding areas, which caused the population of Amman to increase to about 70,000 inhabitants.

New developments created the need for new buildings and new building materials such as steel beams which made the construction of roofs much easier. Previously, roofs were constructed from stone vaulting or wooden beams spanning to a maximum of three meters, and then covered with mud.

During the 1920s, important public buildings appeared in Amman such as Husayni Mosque, Philadelphia Hotel, and Raghadan Palace. As well, important residential buildings appeared in Amman between 1920 and 1950, those were mainly to house government administrators. They were built on separate plots of land and surrounded by walled gardens from all sides. The structures usually consist of one or two stories, usually simple in their massing, planning arrangements, and architectural details. They usually have flat roofs and in plan they follow a tripartite arrangement.

The earlier examples of these houses had load bearing walls made of rubble stone held together by concrete. Consequently these walls were relatively thick. Steel I-beams which were closely spaced at intervals of about one meter were use to support the roofs which they often consisted of a reinforcement mesh. In some earlier cases, the spans between the steel beams were bridged by barrel vaults constructed of concrete. Roughly-textured stone blocks (locally known as tubzeh blocks) provided the major exterior surface material for the houses.

Initially, reinforced concrete was used solely for roof slabs, and was used instead of, or in association with, steel I-beams. By the 1940s, reinforced concrete was being used not only for roofs, but also for columns supporting them, and such columns gradually replaced the traditional thick load-bearing walls.

The development of buildings continued through the decades where the major building material became reinforced concrete for roofs and columns, but stone remained the dominant exterior surface of the majority of houses of Jordan.

The period between 1950 and 1970 has witnessed the Arab-Israeli struggle, which caused hundreds of thousands of Palestinian refugees to move to Jordan. So Jordan had to cope with those tragic consequences and managed to develop the housing sector by building more residences to shelter the increased population. Also, during this period the apartment buildings appeared in Amman and introduced a new dimension to the city of Amman.

Amman nowadays is built on seven hills each of which defines a neighborhood. The layout is described as eight circles that form the "spine of the city," with the downtown area as the first circle and from there extending to the west. Currently, the land area of the City is about 700 km^2 and has a population of 1,800,000, (Turab, 1997).

6.3 Description of models

This section describes in details each one of the studied models, and that includes the general characteristics of the model and the variable parameters. Next, the applied loads, the layout of the plan and the properties of the sections are explained.

*6.3.1 **Model #1 (F4RC)**: four floors - reinforced concrete shear walls*

A. General Characteristics:

- Number of stories: four stories with total height = 13.6m
- Net area of typical floor = 506 m^2
- Shear walls: Reinforced concrete shear walls with 1ϕ10mm at 200mm spacing each way, each face.
- Slab: One way ribbed slab with thickness= 300mm

B. Variable Parameters:

- Soil Type: the model was studied for two soil types; soil type C and soil type D.
- Seismic Demand: the model was studied for the expected earthquake (according to the seismic zone of Amman), and then double the expected earthquake; achieved by doubling the calculated spectral displacement.

C. Applied Loads:

- Dead load: the dead load of the slab was calculated as shown in Table 6.1;

Table 6.1: Dead load calculation for **Model # 1 (F4RC)**

Material	**Load (kN/m^2)**
24cm hollow block	1.73
RC ribs	3.14
Cement mortar	0.66
Tiles	0.75
Compacted sand	0.90
Plastering	0.44
Partitions	2.38
SUM	**10 kN/m^2**

- Live load: a live load of **2kN/m^2** was applied on the slab according to the Jordanian code for residential buildings

D. Plan Layout:

Figure 6.1 displays the plan layout for the first model, specified on it the section names and span dimensions (in meter).

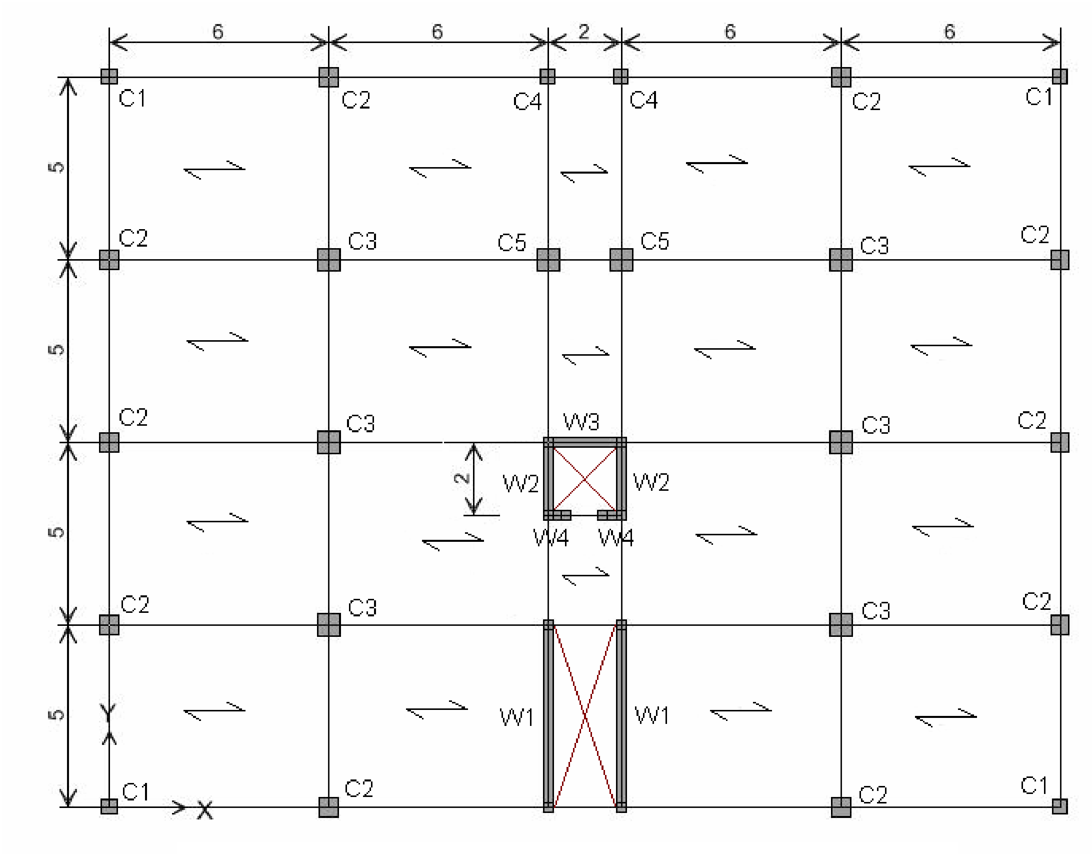

Figure 6.1: Plan layout for **Model # 1 (F4RC)**

E. Section Properties:

Table 6.2 gives the dimensions and reinforcement for each section;

Table 6.2: Section properties for **Model # 1 (F4RC)**

Section name	Dimension (m)	Reinforcement
C1	0.4 x 0.4	8 ϕ 14mm
C2	0.5 x 0.5	12 ϕ 14mm
C3	0.6 x 0.6	16 ϕ 16mm
C4	0.4 x 0.4	8 ϕ 14mm
C5	0.6 x 0.6	16 ϕ 16mm
W1	5.0 x 0.25	1 ϕ 10mm @200mm
W2	2.0 x 0.25	1 ϕ 10mm @200mm
W3	2.0 x 0.25	1 ϕ 10mm @200mm
W4	0.5 x 0.25	1 ϕ 10mm @200mm

*6.3.2 **Model #2 (F4URC)**: four floors - unreinforced concrete shear walls*

A. General Characteristics:

- Number of stories: four stories with total height = 13.6m
- Net area of typical floor = 506 m^2
- Shear walls: Unreinforced concrete shear walls (plain concrete).
- Slab: One way ribbed slab with thickness= 300mm

B. Variable Parameters:

- Soil Type: the model was studied for two soil types; soil type C and soil type D.
- Seismic Demand: the model was studied for the expected earthquake (according to the seismic zone of Amman), and then double the expected earthquake; achieved by doubling the calculated spectral displacement.
- **C. Applied Loads:**

The same as Model # 1 (dead load = 10kN/m^2, live load = 2kN/m^2).

D. Plan Layout:

Figure 6.2 displays the plan layout for Model #2 (F4URC), specified on it the section names and span dimensions (in meter).

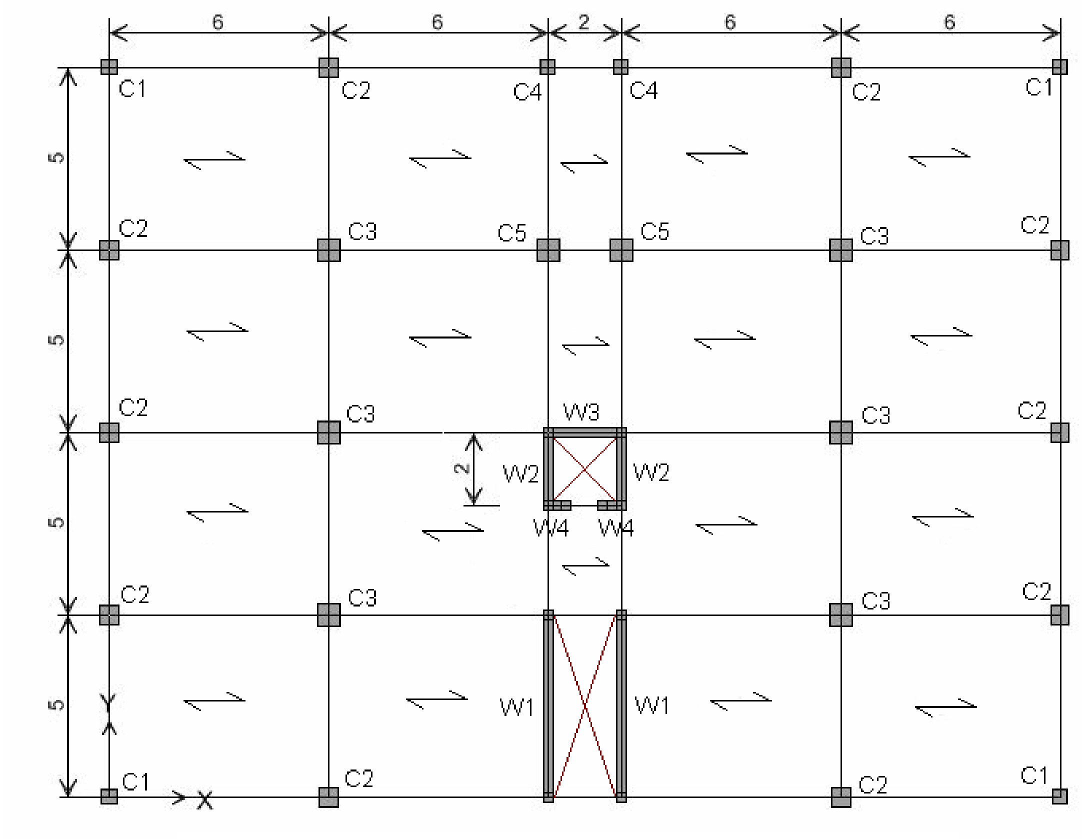

Figure 6.2: Plan layout for **Model # 2 (F4URC)**

E. Section Properties:

Table 6.3 gives the dimensions and reinforcement for each section;

Table 6.3: Section properties for **Model # 2 (F4URC)**

Section name	Dimension (m)	Reinforcement
C1	0.4 x 0.4	8 ϕ 14mm
C2	0.5 x 0.5	12 ϕ 14mm
C3	0.6 x 0.6	16 ϕ 16mm
C4	0.4 x 0.4	8 ϕ 14mm
C5	0.6 x 0.6	16 ϕ 16mm
W1	5.0 x 0.25	/
W2	2.0 x 0.25	/
W3	2.0 x 0.25	/
W4	0.5 x 0.25	/

6.3.3 *Model #3 (F5RC): five floors - reinforced concrete shear walls*

A. General Characteristics:

- Number of stories: five stories with total height = 17m
- Net area of typical floor = 506 m^2
- Shear walls: Reinforced concrete shear walls.
- Slab: One way ribbed slab with thickness= 300mm

B. Variable Parameters:

- Soil Type: the model was studied for two soil types; soil type C and soil type D.
- Seismic Demand: the model was studied for the expected earthquake (according to the seismic zone of Amman), and then double the expected earthquake; achieved by doubling the calculated spectral displacement.

C. Applied Loads:

The same as Model # 1 (dead load = 10kN/m^2, live load = 2kN/m^2).

D. Plan Layout:

Figure 6.3 displays the plan layout for Model # 3 (F5RC), specified on it the section names and span dimensions (in meter).

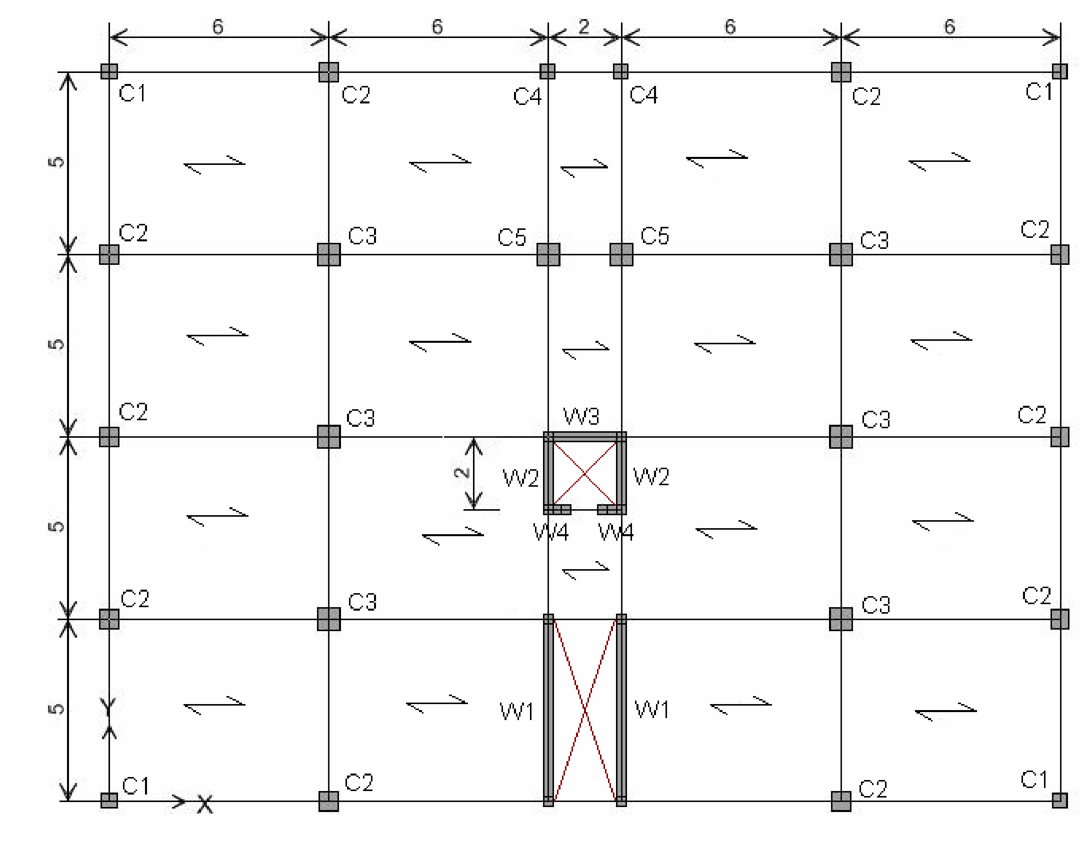

Figure 6.3: Plan layout for **Model # 3 (F5RC)**

E. Section Properties:

Table 6.4 gives the dimensions and reinforcement for each section;

Table 6.4: Section properties for **Model # 3 (F5RC)**

Section name	Dimension (m)	Reinforcement
C1	0.4 x 0.4	8 ϕ 14mm
C2	0.6 x 0.6	16 ϕ 16mm
C3	0.7 x 0.7	16 ϕ 16mm
C4	0.5 x 0.5	12 ϕ 14mm
C5	0.6 x 0.6	16 ϕ 16mm
W1	5.0 x 0.25	1 ϕ 10mm @200mm
W2	2.0 x 0.25	1 ϕ 10mm @200mm
W3	2.0 x 0.25	1 ϕ 10mm @200mm
W4	0.5 x 0.25	1 ϕ 10mm @200mm

6.3.4 *Model #4 (F3URC): three floors - unreinforced concrete shear walls*

A. General Characteristics:

- Number of stories: three stories with total height = 10.2 m
- Net area of typical floor = 376 m^2
- Shear walls: Unreinforced concrete shear walls (plain concrete).
- Slab: One way ribbed slab with thickness= 300mm

B. Variable Parameters:

- Soil Type: the model was studied for two soil types; soil type C and soil type D.
- Seismic Demand: the model was studied for the expected earthquake (according to the seismic zone of Amman), and then double the expected earthquake; achieved by doubling the calculated spectral displacement.

C. Applied Loads:

The same as Model # 1 (dead load = 10kN/m^2, live load = 2kN/m^2).

D. Plan Layout:

Figure 6.4 displays the plan layout for Model # 4 (F3URC), specified on it the section names and span dimensions (in meter).

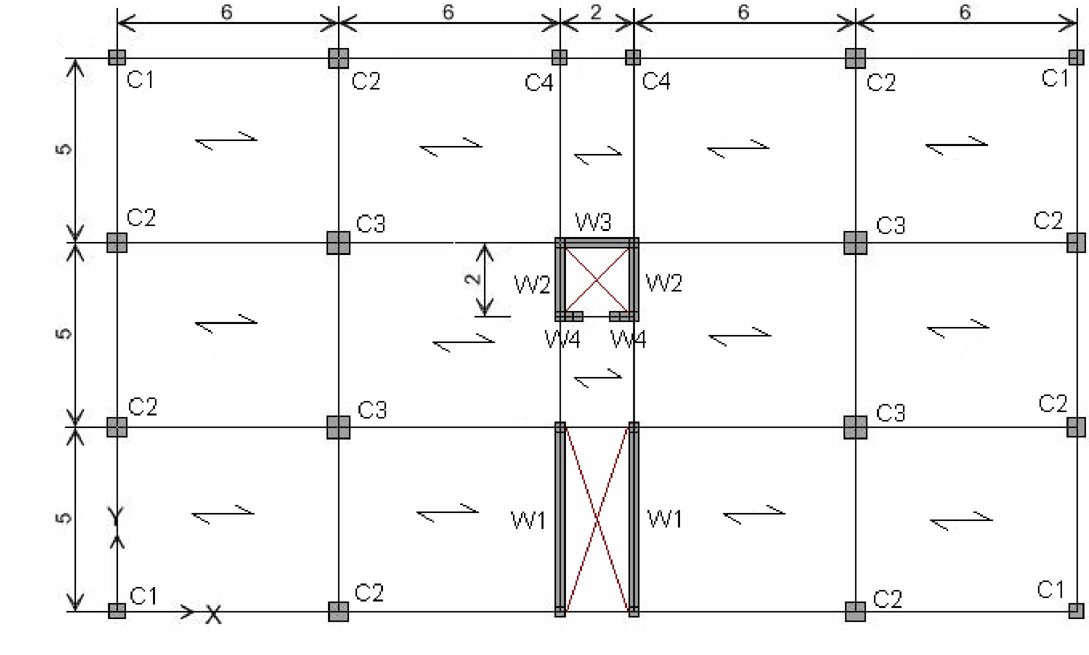

Figure 6.4: Plan layout for **Model # 4 (F3URC)**

E. Section Properties:

Table 6.5 gives the dimensions and reinforcement for each section;

Table 6.5: Section properties for **Model # 4 (F3URC)**

Section name	Dimension (m)	Reinforcement
C1	0.4 x 0.4	8 ϕ 14mm
C2	0.5 x 0.5	12 ϕ 14mm
C3	0.6 x 0.6	16 ϕ 16mm
C4	0.4 x 0.4	8 ϕ 14mm
W1	5.0 x 0.25	/
W2	2.0 x 0.25	/
W3	2.0 x 0.25	/
W4	0.5 x 0.25	/

6.3.5 ***Model #5 (F7RC)****: seven floors – reinforced concrete shear walls*

A. General Characteristics:

- Number of stories: seven stories with total height = 23.8 m
- Net area of typical floor = 506 m^2
- Shear walls: Reinforced concrete shear walls.
- Slab: One way ribbed slab with thickness= 300mm

B. Variable Parameters:

- Soil Type: the model was studied for two soil types; soil type C and soil type D.
- Seismic Demand: the model was studied for the expected earthquake (according to the seismic zone of Amman), and then double the expected earthquake; achieved by doubling the calculated spectral displacement.

C. Applied Loads:

The same as Model # 1 (dead load = 10kN/m^2, live load = 2kN/m^2).

D. Plan Layout:

Figure 6.5 displays the plan layout for Model # 5 (F7RC), specified on it the section names and span dimensions (in meter).

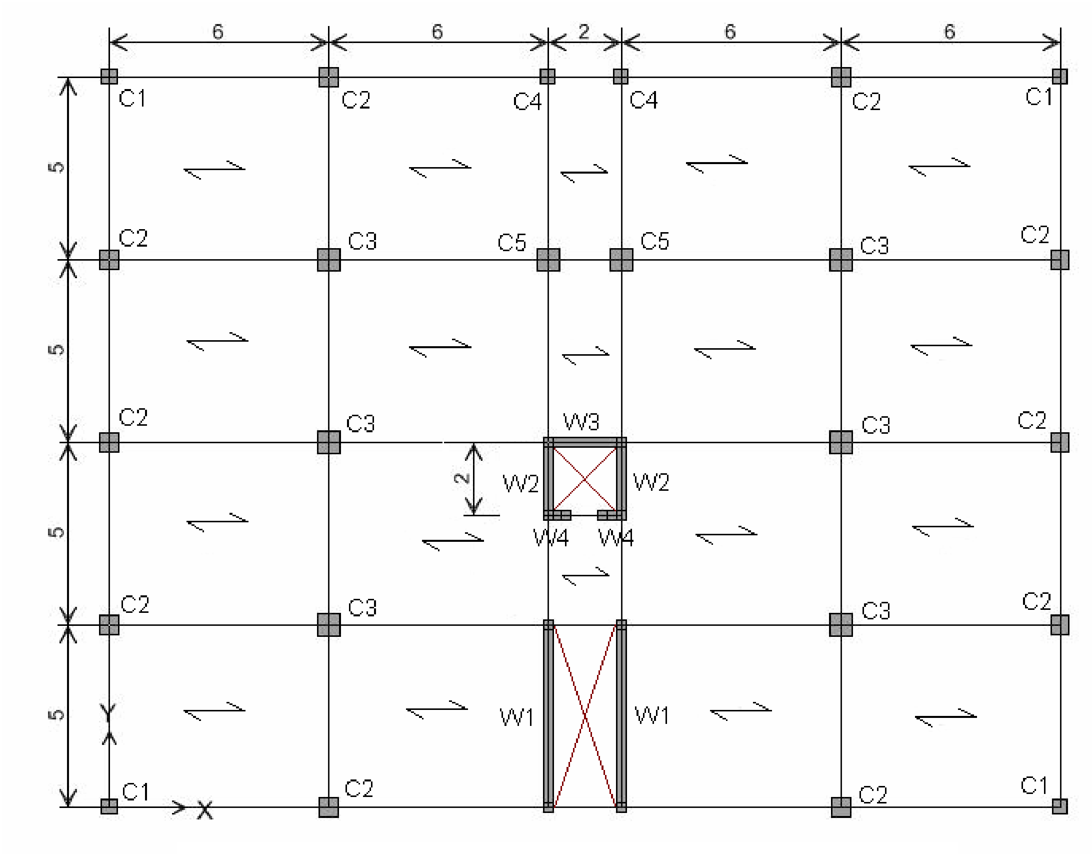

Figure 6.5: Plan layout for **Model # 5 (F7RC)**

E. Section Properties:

Table 6.6 gives the dimensions and reinforcement for each section;

Table 6.6: Section properties for **Model # 5 (F7RC)**

Section name	Dimension (m)	Reinforcement
C1	0.5 x 0.5	12 ϕ 14mm
C2	0.6 x 0.6	16 ϕ 18mm
C3	0.7 x 0.8	24 ϕ 18mm
C4	0.6 x 0.6	16 ϕ 16mm
C5	0.7 x 0.7	16 ϕ 16mm
W1	5.0 x 0.25	1 ϕ 10mm @200mm
W2	2.0 x 0.25	1 ϕ 10mm @200mm
W3	2.0 x 0.25	1 ϕ 10mm @200mm
W4	0.5 x 0.25	1 ϕ 10mm @200mm

6.3.6 *Model #6 (F2URC): two floors – unreinforced concrete shear walls*

A. General Characteristics:

- Number of stories: Two stories with total height = 6.8m
- Net area of typical floor = 210 m^2
- Shear walls: Unreinforced concrete shear walls (plain concrete).
- Slab: One way ribbed slab with thickness = 240mm

B. Variable Parameters:

- Soil Type: the model was studied for two soil types; soil type C and soil type D.
- Seismic Demand: the model was studied for the expected earthquake (according to the seismic zone of Amman), and then double the expected earthquake; achieved by doubling the calculated spectral displacement.

C. Applied Loads:

- Dead load: the dead load of the slab was calculated as shown in Table 6.7;

Table 6.7: Dead load calculation for **Model # 6 (F2URC)**

Material	Load (kN/m^2)
18cm hollow block	1.44
RC ribs	2.53
Cement mortar	0.44
Tiles	0.75
Compacted sand	0.82
Plastering	0.44
Partitions	2.38
SUM	**8.8 kN/m^2**

- Live load: a live load of **2kN/m^2** was applied on the slab according to the Jordanian code for residential buildings

D. Plan Layout:

Figure 6.6 displays the plan layout for model six, specified on it the section names and span dimensions (in meter).

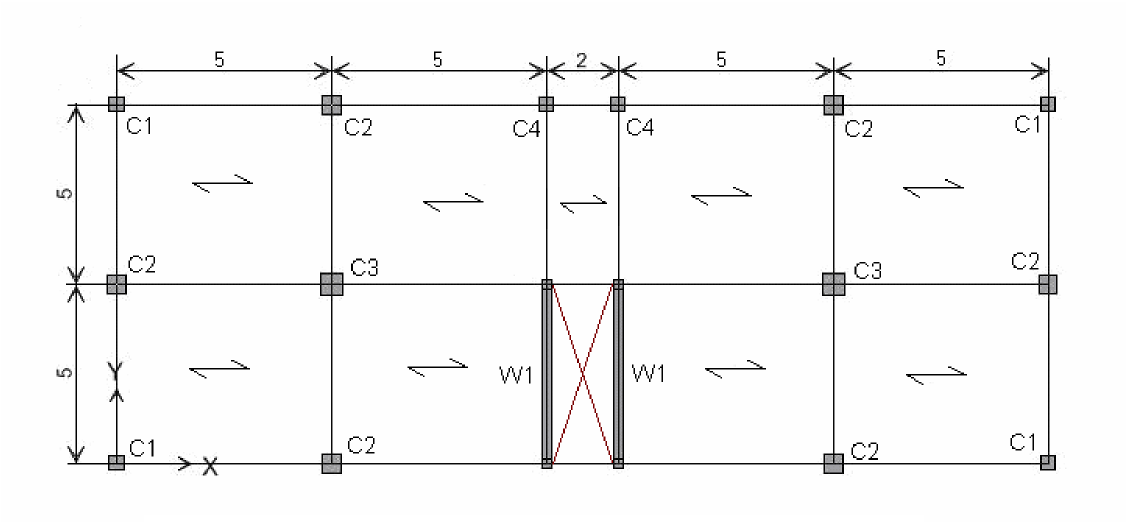

Figure 6.6: Plan layout for **Model # 6 (F2URC)**

E. Section Properties:

Table 6.8 gives the dimensions and reinforcement for each section;

Table 6.8: Section properties for **Model # 6 (F2URC)**

Section name	Dimension (m)	Reinforcement
C1	0.4 x 0.4	8 ϕ 14mm
C2	0.5 x 0.5	12 ϕ 14mm
C3	0.6 x 0.6	16 ϕ 16mm
C4	0.4 x 0.4	8 ϕ 14mm
W1	5.0 x 0.25	/

6.4 Step by Step Procedure

The following steps portray the procedure used in order to perform the vulnerability study and to get the results as a capacity curve of the building under consideration and its vulnerability function:

Step 1) The model general characteristics and variable parameters were determined and the plan layout was set.

Step 2) Structural walls and columns were identified along with the dead load and live load acting on the slabs.

Step 3) Microsoft Excel program was utilized to calculate the load carried by each wall and column at each floor level, and the concentrated masses for each floor.

Step 4) A subroutine generated by Microsoft Visual Basic 6.0 was used to calculate, through an iterative process, the depth of neutral axis and the moment for each section; at the onset of cracking, at first yield, and at ultimate state.

Step 5) Results from step 4 were entered again into Excel to calculate the significant parameters for the capacity curve such as; ϕ_y, ϕ_u, μ_ϕ, l_p, μ_w, ϕ_{cr}, V_m, Δ_y, Δ_u, Δ_{cr}, and K_{eff}.

Step 6) Results from step 5 were used to plot the capacity curve of the walls and columns, and then to combine them to get the capacity curve of the building to define damage grades on it.

Step 7) Mathematica 5.0 software was used to perform the modal analysis of the model by entering the mass matrix and the stiffness matrix in order to solve the eigenvalue λ and the eigenvector Φ (mode shape) for the first mode of vibration. Then the participation factor Γ was determined.

Step 8) According to the design response spectrum of the Jordanian code for different types of soil, the spectral displacement and the displacement at the top of the building for each damage grade were obtained.

Step 9) Spectral displacement vs. displacement at the top of the building were plotted to obtain the vulnerability function of the model, and to study the effect of different soil types and earthquake magnitudes.

Step 10) The response of the model while varying parameters was studied and the performance was analyzed.

6.5 Vulnerability Results

Table 6.9: Results for **Model #1 (F4RC)**

	W1	W2	C1	C2	C3	C 4	C 5
x_y (mm)	1126	489	121	161	215	131	190
M_y(kN.m)	6072	1161	140	287	611	164	476
ϕ_y (1/m)	0.0007	0.0016	0.0087	0.0101	0.0056	0.0087	0.0057
x_u (mm)	674	309	97	148	200	121	164
M_u(kN.m)	7684	1405	155	307	616	173	520
ϕ_u (1/m)	0.014	0.031	0.099	0.065	0.048	0.080	0.059
μ_ϕ	21.8	19.0	11.4	6.4	8.6	9.1	10.4
x_{cr} (mm)	2930	1269	293	381	482	309	449
M_{cr}(kN.m)	2929	587	60	135	297	73	218
V_m (kN)	848	155	17	34	68	19	57
Δ_y (m)	0.033	0.083	0.443	0.512	0.285	0.442	0.287
Δ_u (m)	0.203	0.389	0.944	0.726	0.306	0.660	0.559
Δ_{cr}(m)	0.013	0.035	0.274	0.226	0.137	0.186	0.120
K_{eff}(kN/m)	25527	1865	33	66	238	43	199

Table 6.10: Results for **Model #2 (F4URC)**

	W1	W2	C1	C2	C3	C 4	C 5
x_y (mm)	1003	451	121	161	215	131	190
M_y(kN.m)	4083	851	140	287	611	164	476
ϕ_y (1/m)	0.0005	0.0014	0.0087	0.0101	0.0056	0.0087	0.0057
x_u (mm)	418	222	97	148	200	121	164
M_u(kN.m)	4380	907	155	307	616	173	520
ϕ_u (1/m)	0.023	0.043	0.099	0.065	0.048	0.080	0.059
μ_ϕ	42.9	30.9	11.4	6.4	8.6	9.1	10.4
x_{cr} (mm)	2900	1259	293	381	482	309	449
M_{cr}(kN.m)	2881	581	60	135	297	73	218
V_m (kN)	483	100	17	34	68	19	57
Δ_y (m)	0.027	0.071	0.443	0.512	0.285	0.442	0.287
Δ_u (m)	0.120	0.229	0.944	0.726	0.306	0.660	0.559
Δ_{cr}(m)	0.018	0.046	0.274	0.226	0.137	0.186	0.120
K_{eff}(kN/m)	17726	1404	33	66	238	43	199

Table 6.11: Results for **Model #3 (F5RC)**

	W1	**W2**	**C1**	**C2**	**C3**	**C 4**	**C 5**
x_y (mm)	1213	529	130	188	246	153	207
M_y(kN.m)	6993	1351	161	470	891	260	568
ϕ_y (1/m)	0.0007	0.0014	0.0087	0.0119	0.0061	0.0100	0.0056
x_u (mm)	767	309	117	162	232	132	193
M_u(kN.m)	8595	1405	170	515	896	287	585
ϕ_u (1/m)	0.013	0.031	0.082	0.059	0.041	0.073	0.050
μ_ϕ	19.3	21.5	9.4	5.0	6.8	7.3	8.9
x_{cr} (mm)	3125	1269	307	447	557	369	473
M_{cr}(kN.m)	3511	587	71	214	453	120	270
V_m (kN)	758	124	19	45	99	25	65
Δ_y (m)	0.052	0.115	0.442	0.945	0.311	0.796	0.285
Δ_u (m)	0.258	0.223	0.943	1.260	0.334	1.109	0.553
Δ_{cr}(m)	0.021	0.048	0.184	0.392	0.157	0.333	0.131
K_{eff}(kN/m)	14715	1081	34	48	318	32	227

Table 6.12: Results for **Model #4 (F3URC)**

	W1	**W2**	**C1**	**C2**	**C3**	**C 4**
x_y (mm)	877	3956	111	148	198	120
M_y(kN.m)	3085	647	120	243	519	138
ϕ_y (1/m)	0.0005	0.0014	0.0087	0.0100	0.0056	0.0087
x_u (mm)	310	165	77	121	177	95
M_u(kN.m)	3311	693	136	273	551	153
ϕ_u (1/m)	0.031	0.058	0.125	0.079	0.054	0.101
μ_ϕ	59.7	42.8	14.4	8.0	9.6	11.7
x_{cr} (mm)	2644	1157	275	361	461	292
M_{cr}(kN.m)	2252	457	48	110	242	58
V_m (kN)	487	102	15	40	61	22
Δ_y (m)	0.015	0.039	0.440	0.285	0.286	0.249
Δ_u (m)	0.086	0.169	0.938	0.443	0.307	0.412
Δ_{cr}(m)	0.010	0.026	0.154	0.115	0.126	0.095
K_{eff}(kN/m)	32761	2620	34	141	212	90

Table 6.13: Results for **Model #5 (F7RC)**

	W1	W2	C1	C2	C3	C 4	C 5
x_y (mm)	1365	597	156.7	214	344	187	243
M_y(kN.m)	8785	1720	273	633	1386	463	868
ϕ_y (1/m)	0.0006	0.0016	0.0101	0.0130	0.0088	0.0119	0.0061
x_u (mm)	956	463	140	208	331	160	225
M_u(kN.m)	10300	1914	297	647	1391	510	878
ϕ_u (1/m)	0.010	0.021	0.069	0.046	0.029	0.060	0.043
μ_ϕ	15.6	12.8	6.8	3.6	3.3	5.1	7.0
x_{cr} (mm)	3411	1455	375	545	661	445	553
M_{cr}(kN.m)	4603	919	127	346	709	210	440
V_m (kN)	649	121	33	41	154	32	97
Δ_y (m)	0.100	0.253	0.511	2.021	0.446	1.846	0.310
Δ_u (m)	0.355	0.612	1.089	2.505	0.479	2.366	0.602
Δ_{cr}(m)	0.045	0.121	0.219	1.081	0.227	0.760	0.155
K_{eff}(kN/m)	6464	478	64	20	344	17	312

Table 6.14: Results for **Model #6 (F2URC)**

	W1	C1	C2	C3	C 4
x_y (mm)	695	81	110	146	88
M_y(kN.m)	1906	52	118	239	61
ϕ_y (1/m)	0.0005	0.0079	0.0086	0.0053	0.0078
x_u (mm)	186	56	75	118	70
M_u(kN.m)	2036	61	134	270	68
ϕ_u (1/m)	0.052	0.173	0.129	0.081	0.137
μ_ϕ	104.2	21.9	14.9	15.5	17.5
x_{cr} (mm)	2223	200	273	358	215
M_{cr}(kN.m)	1467	18	46	108	23
V_m (kN)	449	7	30	30	15
Δ_y (m)	0.006	0.401	0.110	0.267	0.100
Δ_u (m)	0.056	0.855	0.201	0.287	0.197
Δ_{cr}(m)	0.005	0.119	0.038	0.107	0.034
K_{eff}(kN/m)	71369	17	269	111	151

Table 6.15: Building output for **Model # 1, # 2, # 3, # 4, # 5 and # 6**

	Model # 1 (F4RC)	Model # 2 (F4URC)	Model # 3 (F5RC)	Model # 4 (F3URC)	Model # 5 (F7RC)	Model # 6 (F2URC)
$V_{by,sys}$ (kN)	2260	1314	2104	1246	2081	911
$\Delta_{by,sys}$ (m)	0.0342	0.0285	0.0505	0.0161	0.0922	0.0062
K (kN/m)	66028	46086	41690	77198	22580	145932
Γ	1.247	1.244	1.254	1.225	1.263	1.178
T_1 (sec.)	2.20	2.01	2.61	1.04	4.87	0.38

Figures 6.7 to 6.18 show the capacity curves and vulnerability functions for the indicated model.

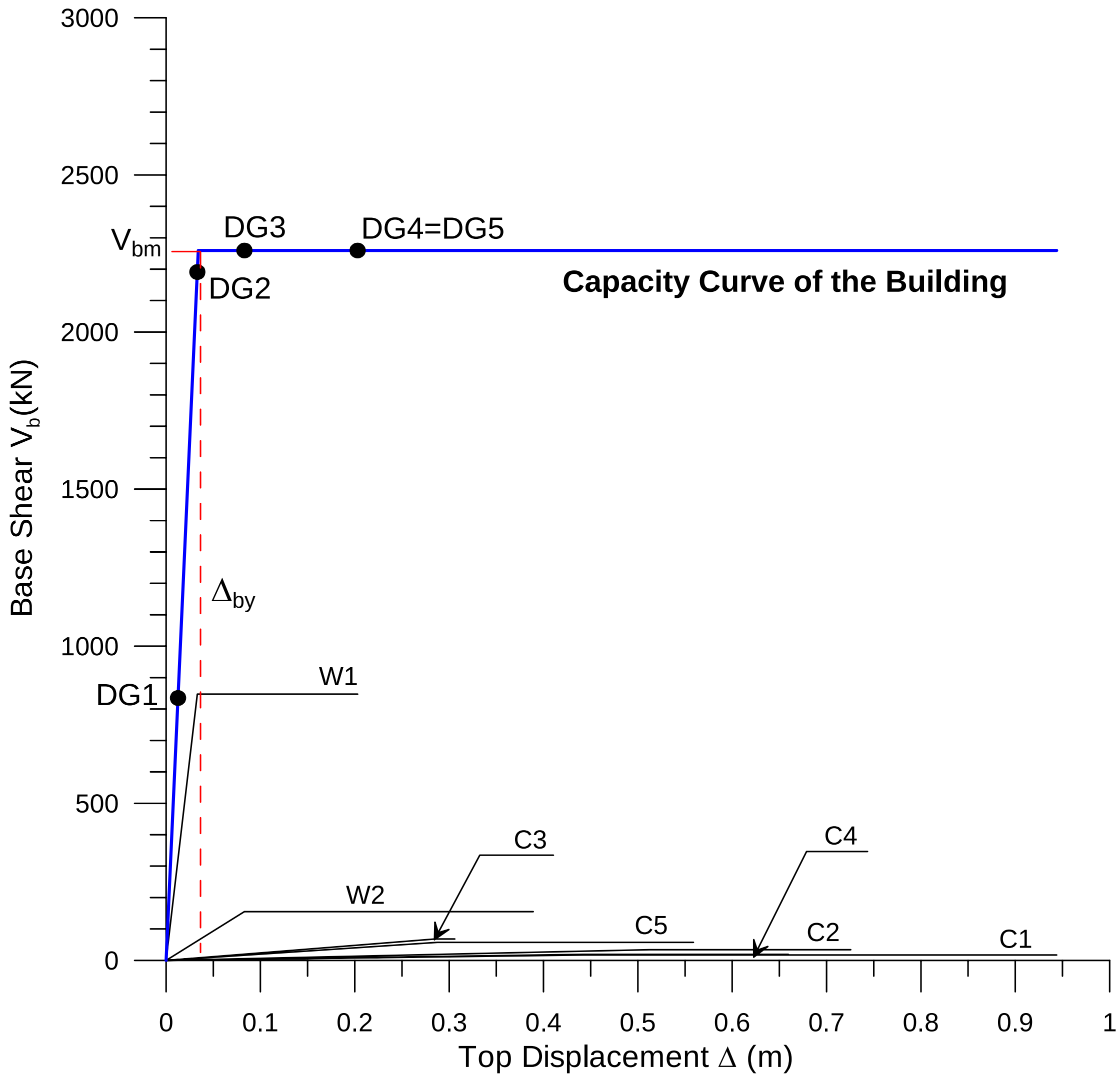

Figure 6.7: Capacity Curve for **Model # 1 (F4RC)**

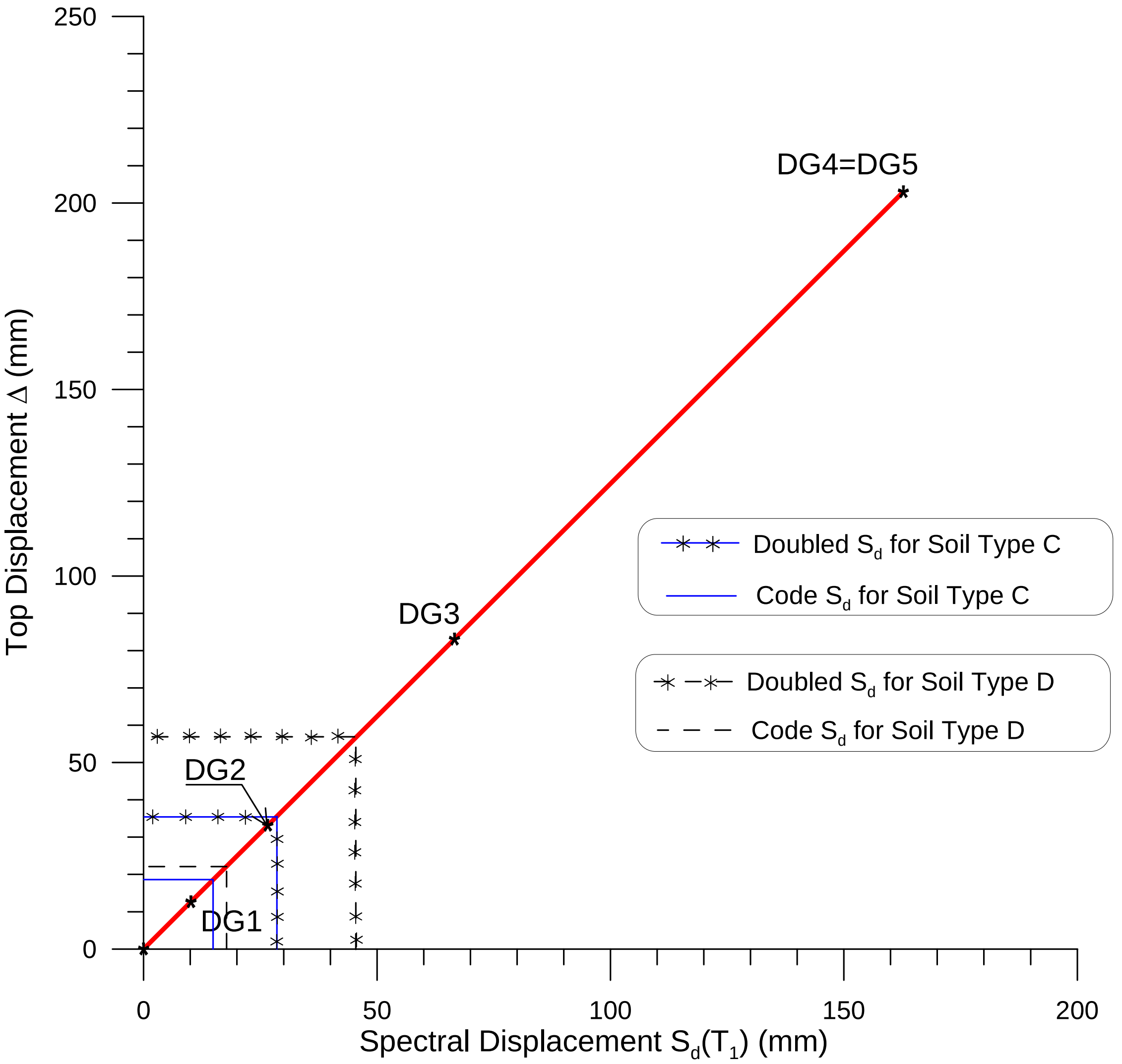

Figure 6.8: Vulnerability Function for **Model # 1 (F4RC)**
For soil type C & D with code and doubled spectral displacement

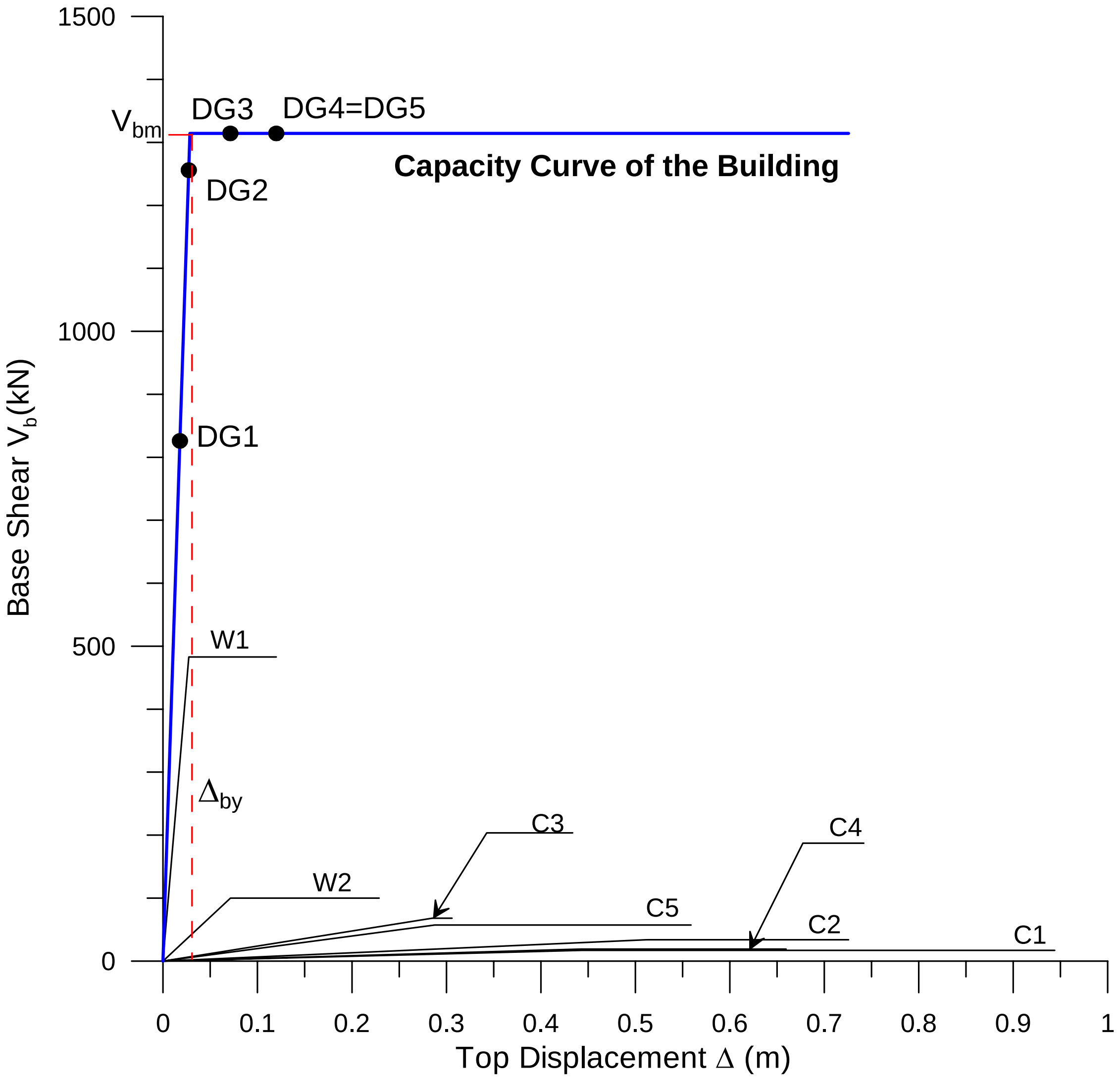

Figure 6.9: Capacity Curve for **Model # 2 (F4URC)**

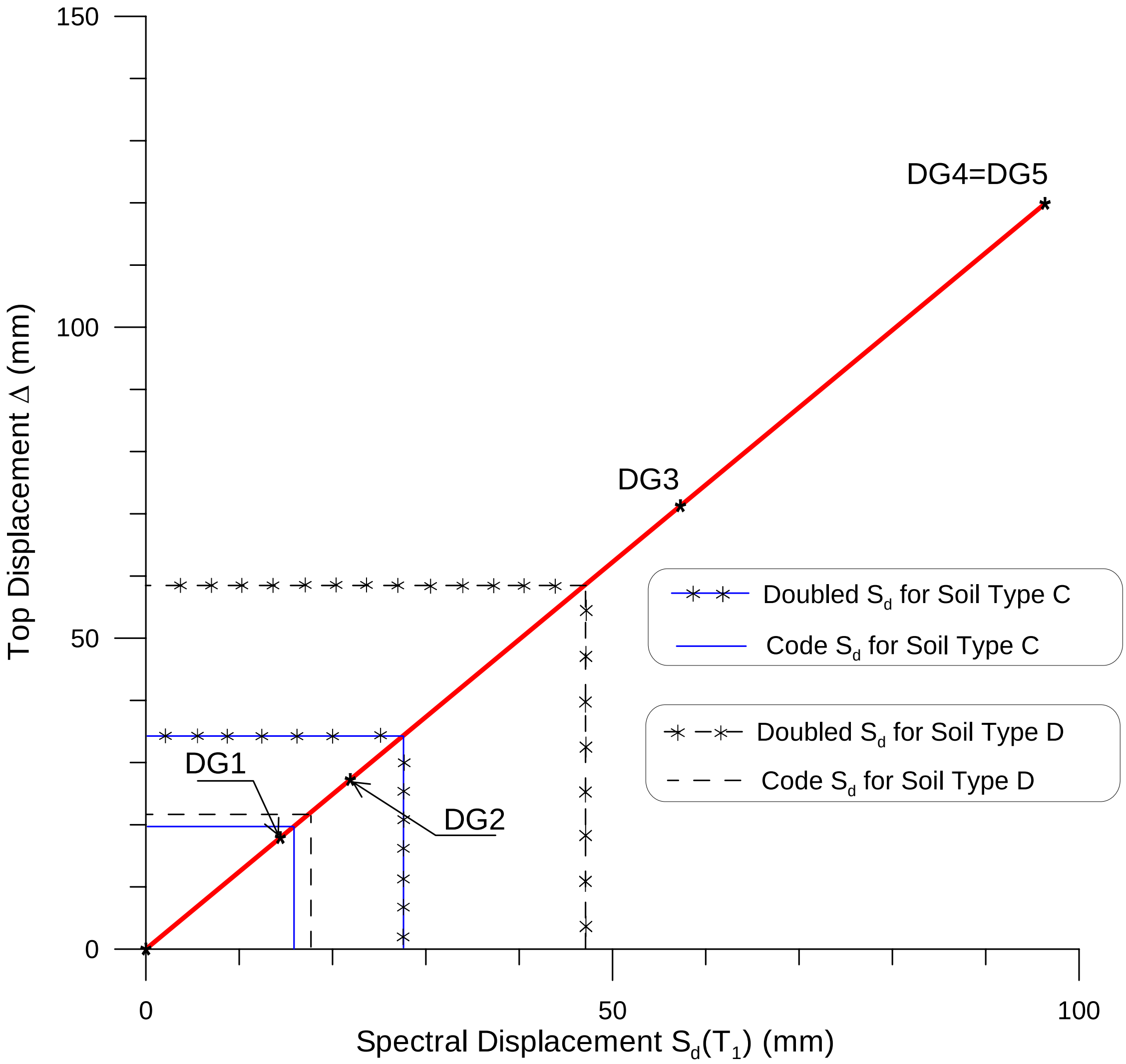

Figure 6.10: Vulnerability Function for **Model # 2 (F4URC)**
For soil type C & D with code and doubled spectral displacement

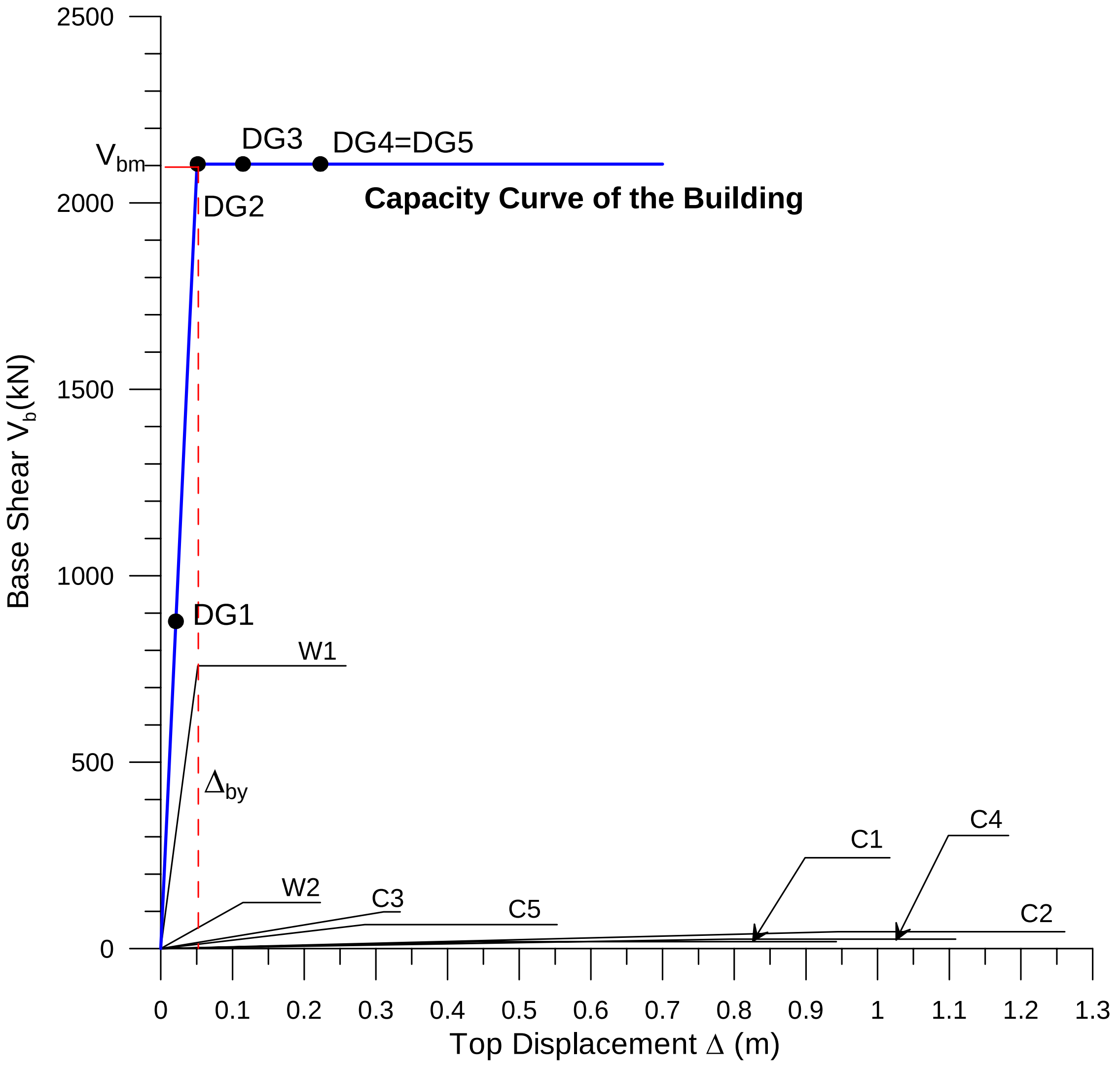

Figure 6.11: Capacity Curve for **Model # 3 (F5RC)**

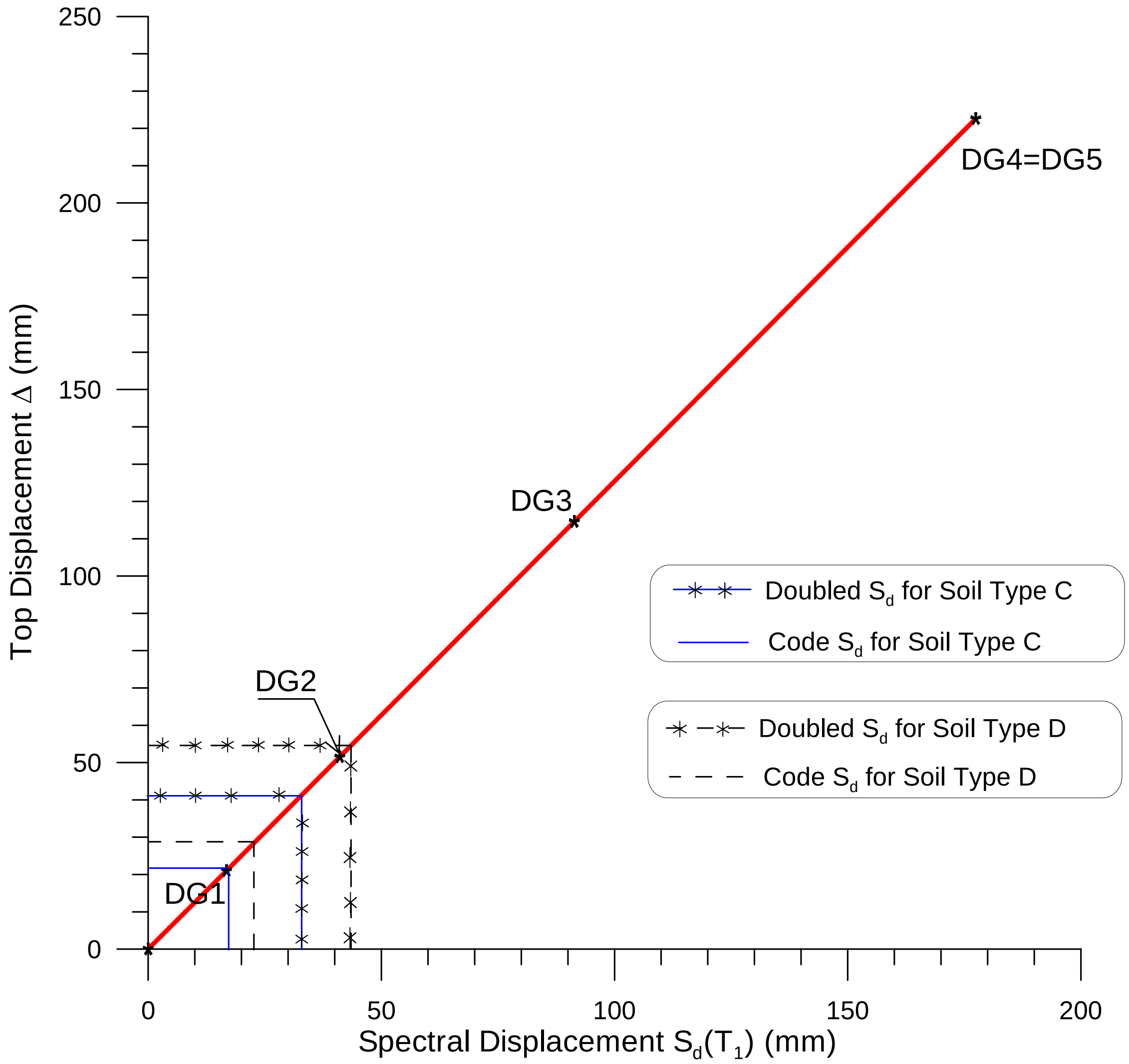

Figure 6.12: Vulnerability Function for **Model # 3 (F5RC)**
For soil type C & D with code and doubled spectral displacement

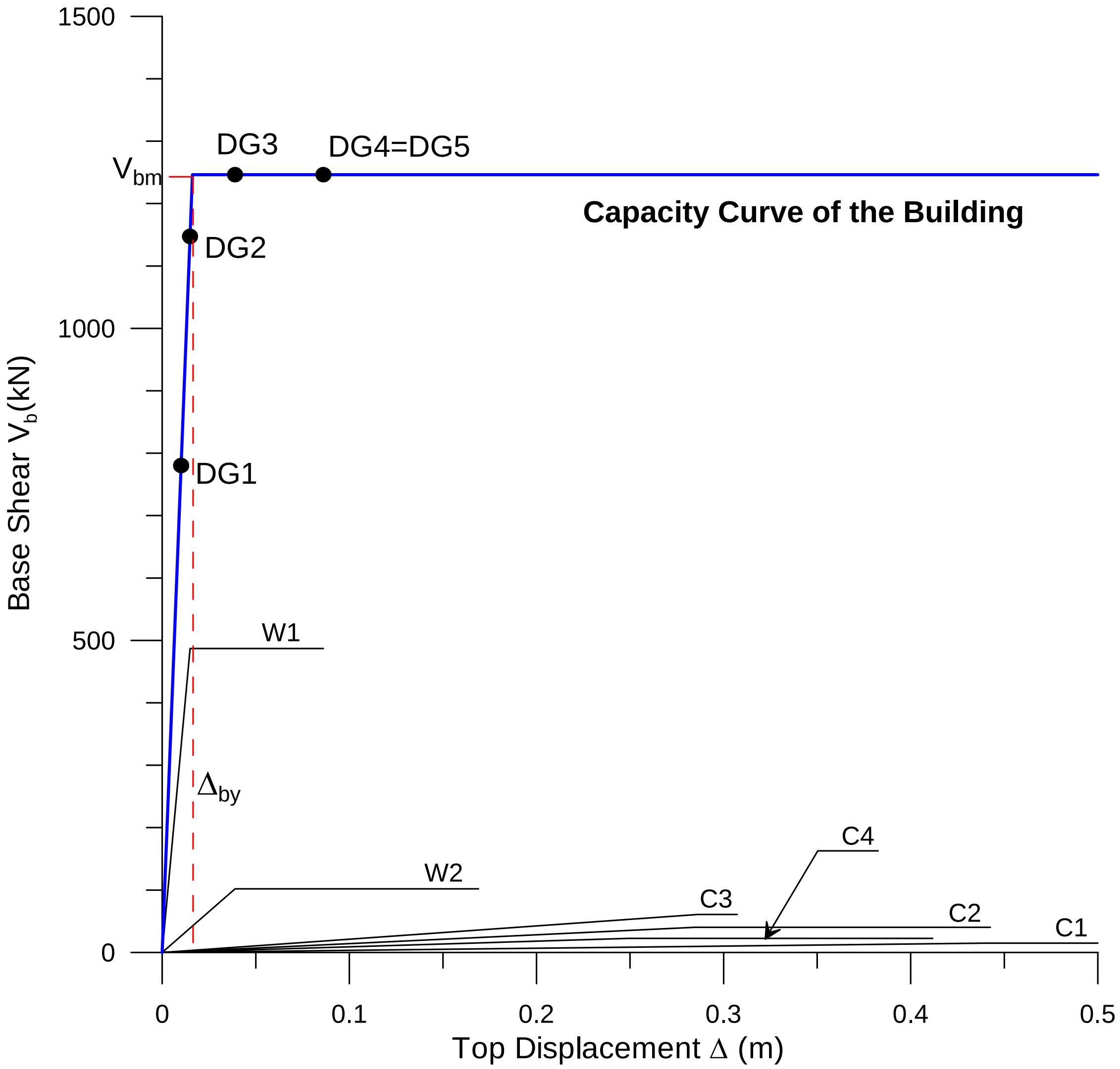

Figure 6.13: Capacity Curve for **Model # 4 (F3URC)**

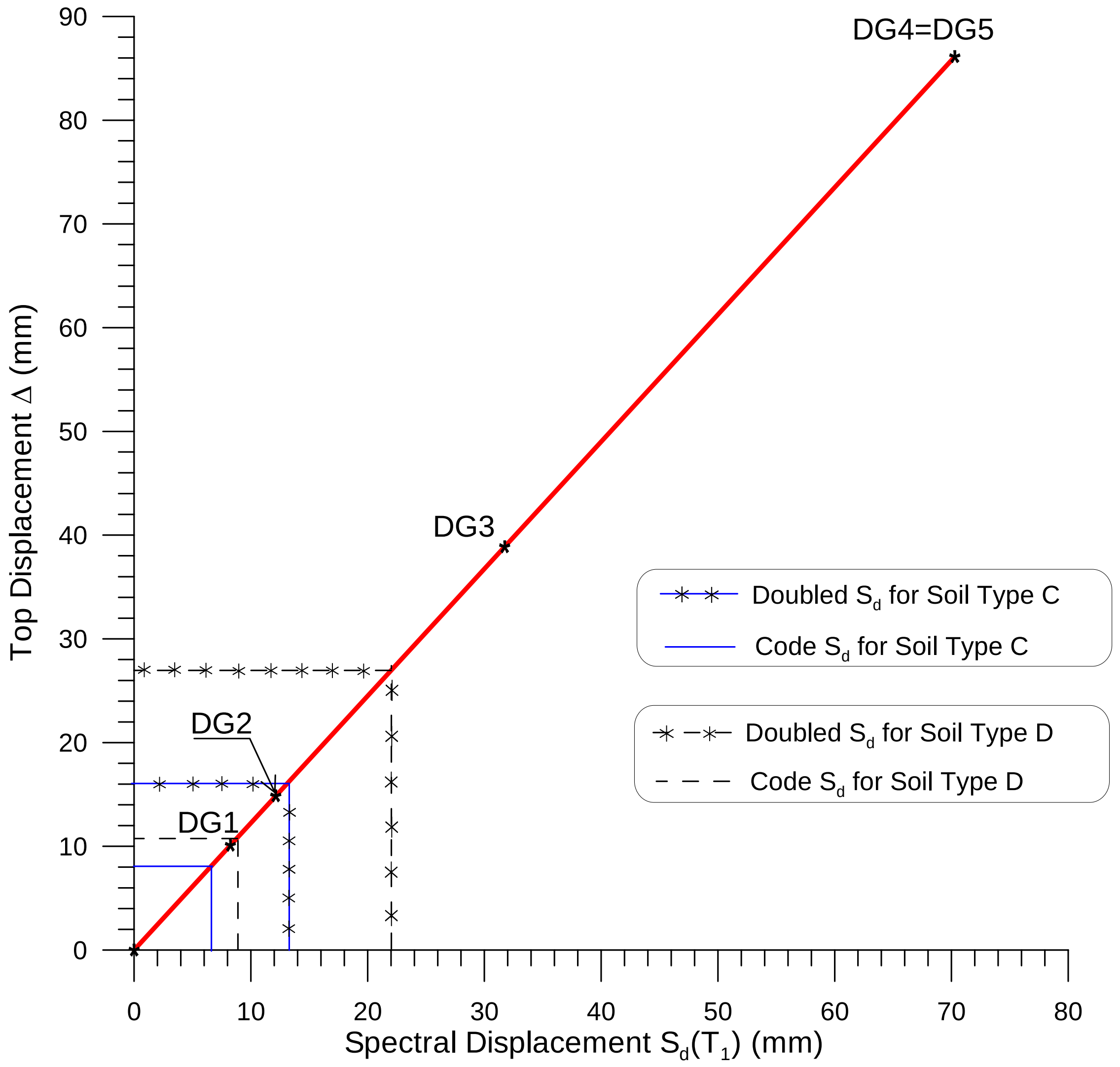

Figure 6.14: Vulnerability Function for **Model # 4 (F3URC)**
For soil type C & D with code and doubled spectral displacement

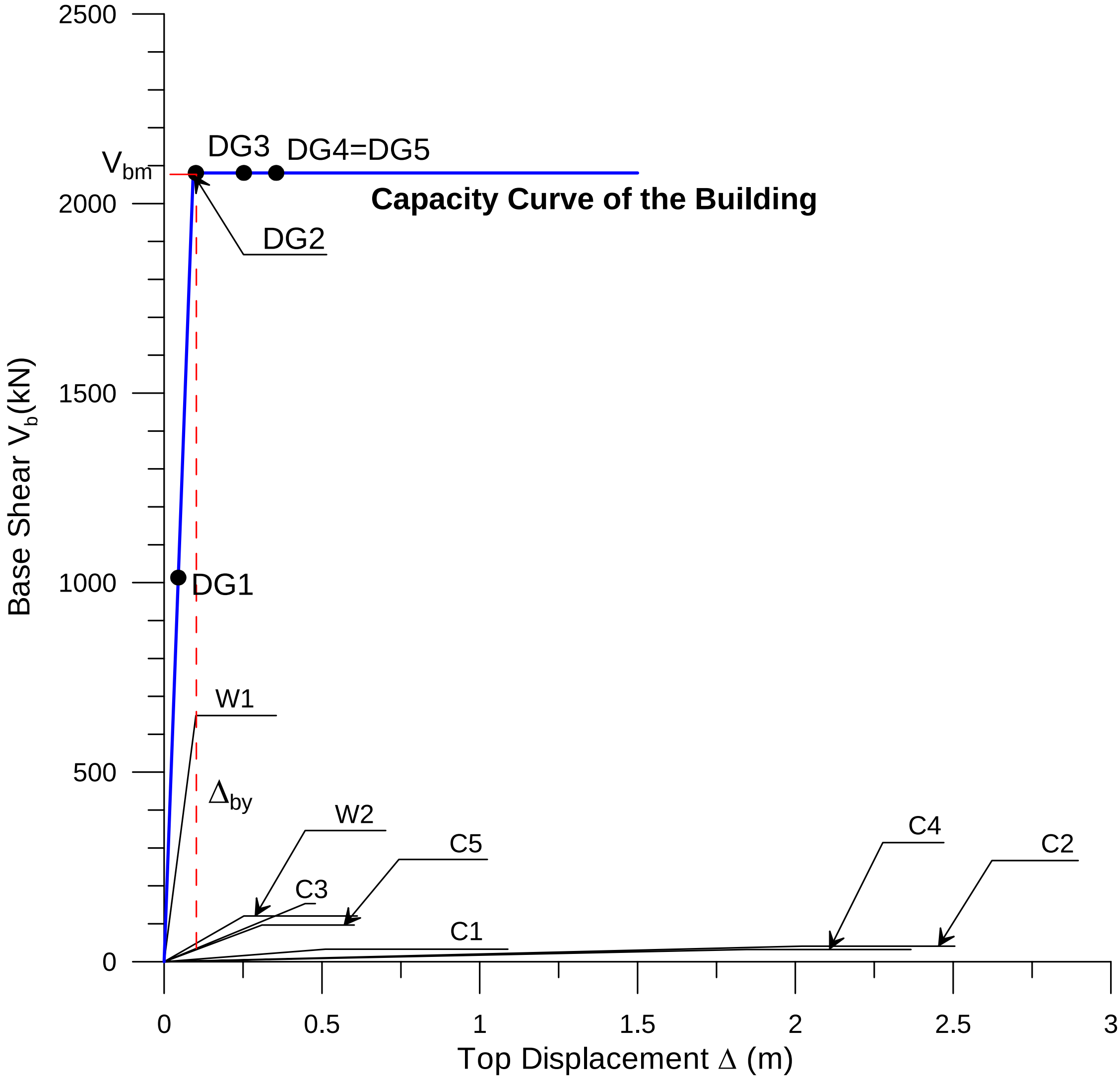

Figure 6.15: Capacity Curve for **Model # 5 (F7RC)**

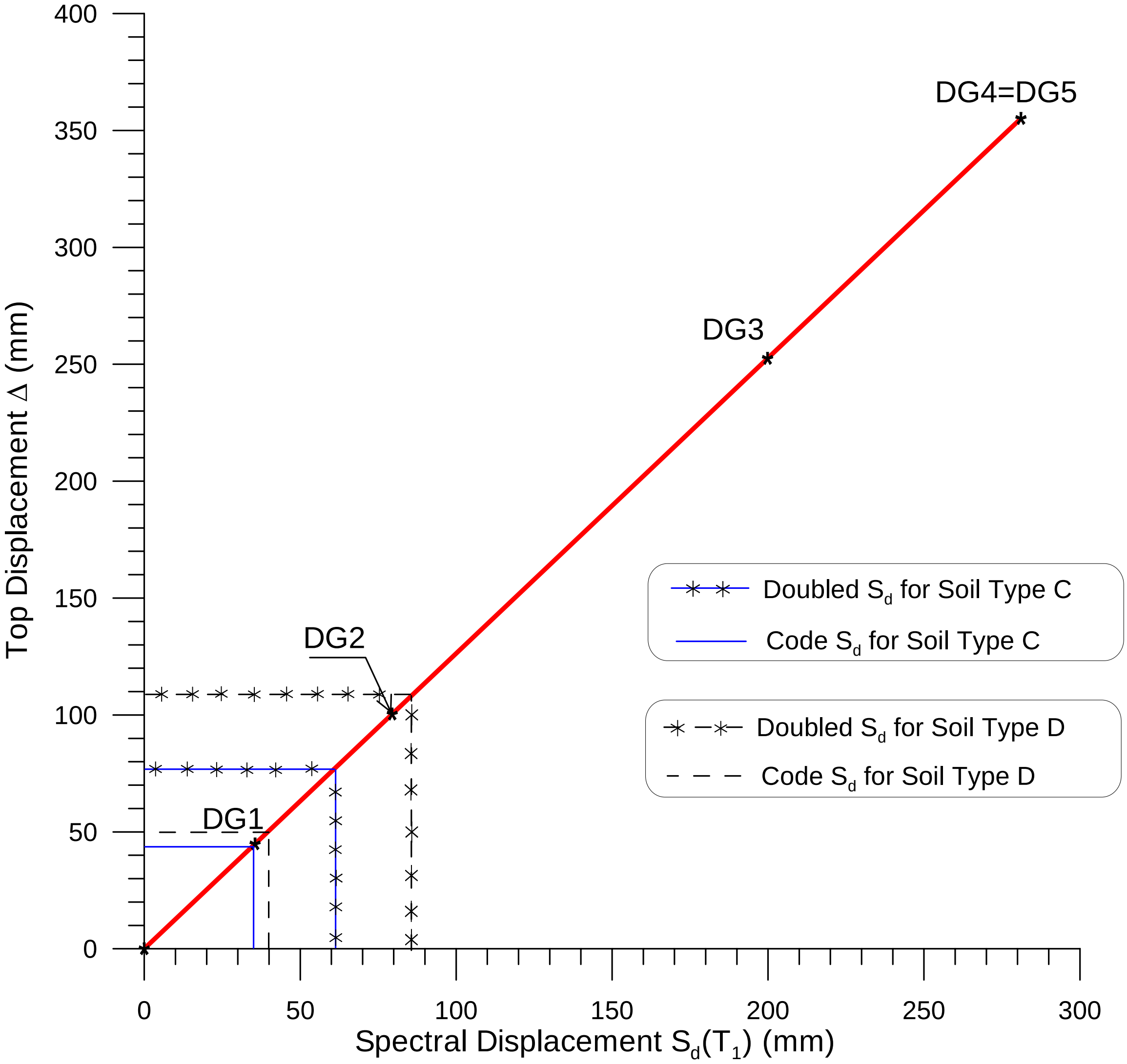

Figure 6.16: Vulnerability Function for **Model # 5 (F7RC)**
For soil type C & D with code and doubled spectral displacement

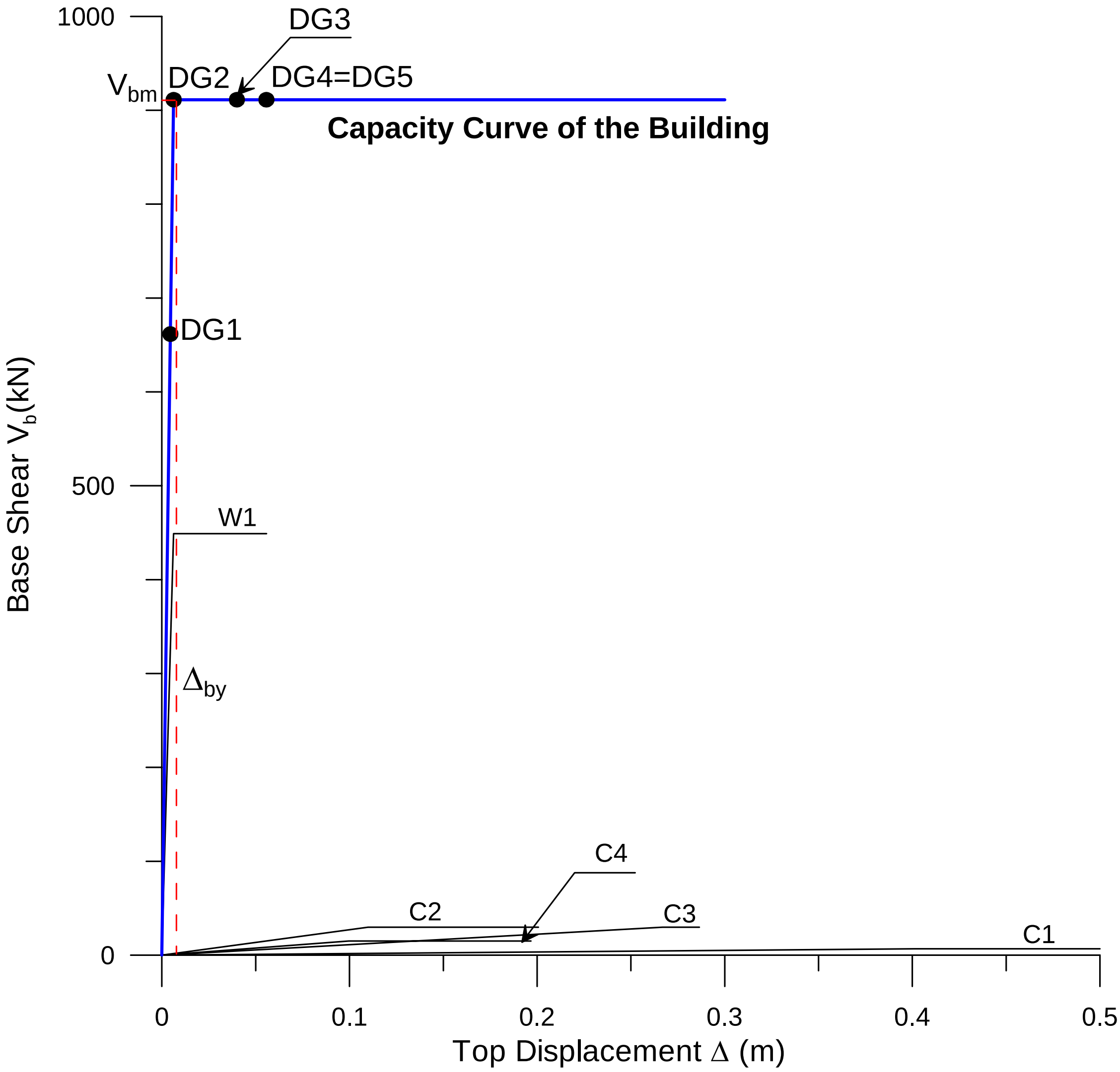

Figure 6.17: Capacity Curve for **Model # 6 (F2URC)**

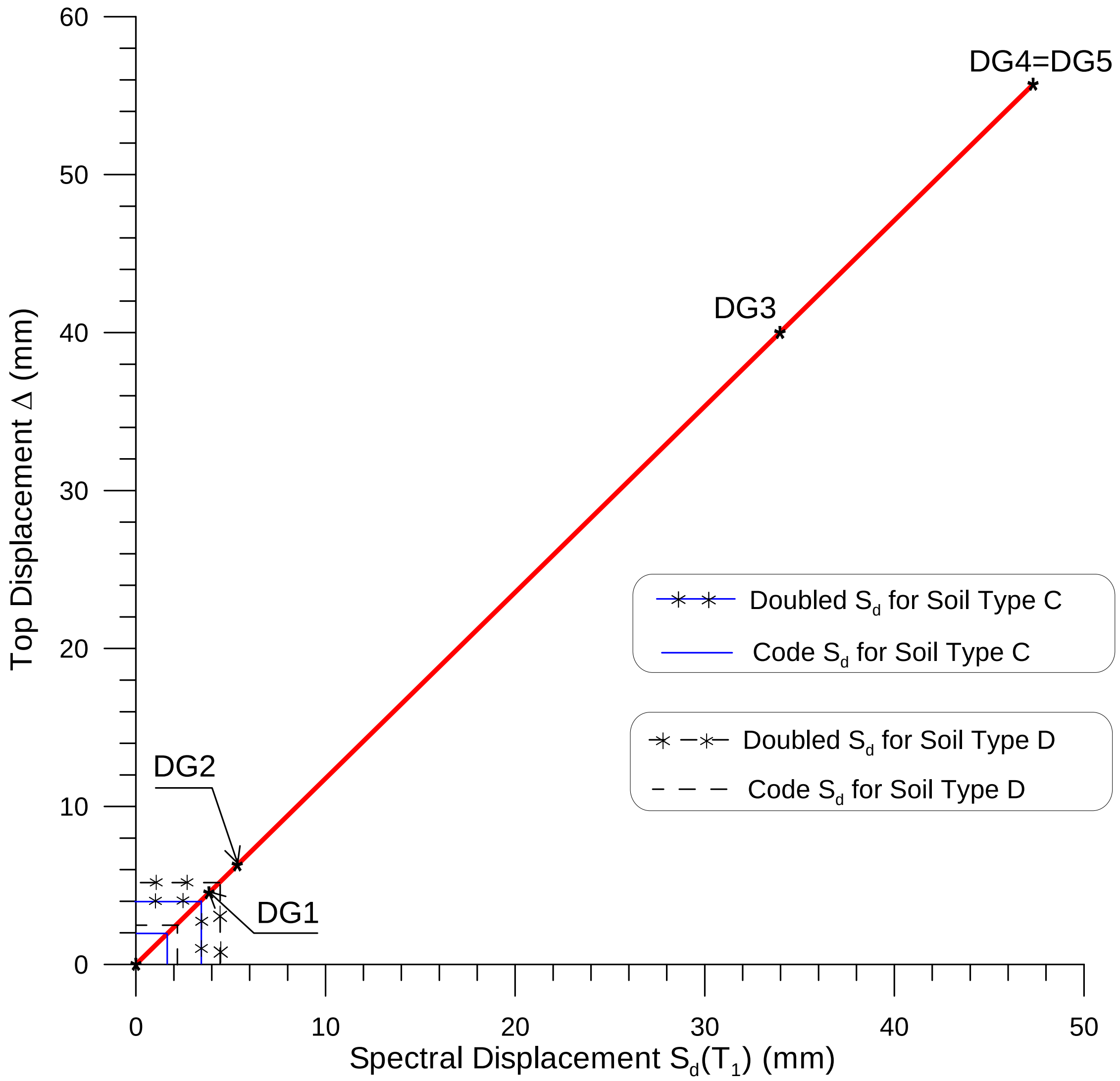

Figure 6.18: Vulnerability Function for **Model # 6 (F2URC)**
For soil type C & D with code and doubled spectral displacement

6.6 Analysis of the Vulnerability Results

It is appropriate to describe the damage grades in a more specific form (Table 6.16), so that it is clearer to describe the physical response of the model under variable parameters.

Table 6.16: Specific definition of damage grades

Damage grade	description	Physical meaning
DG1/A	The model is within the first half of the range of DG1, i.e. nearer to DG1	Fine cracks in plaster
DG1/B	The model is within the second half of the range of DG1, i.e. nearer to DG2	Fine cracks in partitions and infill walls
DG2/A	The model is within the first half of the range of DG2, i.e. nearer to DG2	Cracks in partitions and infill walls; fall of brittle cladding and plaster.
DG2/B	The model is within the second half of the range of DG2, i.e. nearer to DG3	Cracks in columns and beams of frames and in structural walls.
DG3/A	The model is within the first half of the range of DG3, i.e. nearer to DG3	Large cracks in partitions and infill walls, failure of individual infill panels.
DG3/B	The model is within the second half of the range of DG3, i.e. nearer to DG4	Cracks in columns and beam column joints and at joints of coupled walls. Spalling of concrete cover.

<u>Model # 1 (F4RC):</u>

Referring to the statistics (see chapter 5) this model represents the following percentages of the overall residential buildings stock in Amman;

Number of floors (four floors): 11%

Area of floor (500m^2): 20%

Soil type C: 63%

Soil type D: 27%

Structural wall type (RC core & RC stair): 13%

According to Table 6.16, the response of model #1 (F4RC) can be described as:

- For soil type C and the code seismic demand; the building experienced DG1/A
- For soil type C and the doubled code seismic demand; the building experienced DG2/A
- For soil type D and the code seismic demand the building experienced DG1/A
- For soil type D and the doubled code seismic demand the building experienced DG2/A

Model # 2 (F4URC):

Referring to the statistics this model represents the following percentages of the overall residential buildings stock in Amman;

Number of floors (four floors): 11%

Area of floor (500m^2): 20%

Soil type C: 63%

Soil type D: 27%

Structural wall type (URC core & URC stair): 7%

According to Table 6.16, the response of model #2 (F4URC) can be described as:

- For soil type C and the code seismic demand; the building experience DG1/A
- For soil type C and the doubled code seismic demand; the building experience DG2/A
- For soil type D and the code seismic demand the building experience DG1/A
- For soil type D and the doubled code seismic demand the building experience DG2/B

<u>Model # 3 (F5RC):</u>

Referring to the statistics this model represents the following percentages of the overall residential buildings stock in Amman;

Number of floors (five floors): 34%

Area of floor (500m^2): 20%

Soil type C: 63%

Soil type D: 27%

Structural wall type (RC core & RC stair): 13%

According to Table 6.16, the response of model #3 (F5RC) can be described as:

- For soil type C and the code seismic demand; the building experience DG1

- For soil type C and the doubled code seismic demand; the building experience DG1/B
- For soil type D and the code seismic demand the building experience DG1/A
- For soil type D and the doubled code seismic demand the building experience DG2/A

Model # 4 (F3URC):

Referring to the statistics this model represents the following percentages of the overall residential buildings stock in Amman;

Number of floors (three floors): 16%

Area of floor (376m^2): 30%

Soil type C: 63%

Soil type D: 27%

Structural wall type (URC core & URC stair): 7%

According to Table 6.16, the response of model #4 (F3URC) can be described as:

- For soil type C and the code seismic demand; the building experience DG1
- For soil type C and the doubled code seismic demand; the building experience DG2/A
- For soil type D and the code seismic demand the building experience DG1/A

- For soil type D and the doubled code seismic demand the building experience DG2/B

<u>Model # 5 (F7RC):</u>

Referring to the statistics this model represents the following percentages of the overall residential buildings stock in Amman;

Number of floors (seven floors): 5%

Area of floor (500m^2): 20%

Soil type C: 63%

Soil type D: 27%

Structural wall type (RC core & RC stair): 13%

According to Table 6.16, the physical response of model #5 (F7RC) can be described as:

- For soil type C and the code seismic demand; the building experience DG1
- For soil type C and the doubled code seismic demand; the building experience DG1/B
- For soil type D and the code seismic demand the building experience DG1/A
- For soil type D and the doubled code seismic demand the building experience DG2/A

Model # 6 (F2URC):

Referring to the statistics this model represents the following percentages of the overall residential buildings stock in Amman;

Number of floors (two floors): 13%

Area of floor (210 m^2): 30%

Soil type C: 63%

Soil type D: 27%

Structural wall type (URC stair): 36%

According to Table 6.16, the physical response of model #6 (F2URC) can be described as:

- For soil type C and the code seismic demand; the building experience DG1
- For soil type C and the doubled code seismic demand; the building experience DG1
- For soil type D and the code seismic demand the building experience DG1
- For soil type D and the doubled code seismic demand the building experience DG1/A

CHAPTER 7 Pushover Analysis

7.1 Introduction

Pushover analysis is a nonlinear static procedure that implements simplified nonlinear techniques to estimate seismic structural deformation and forces. It can be used to estimate the dynamic demands imposed in structures by earthquake ground motion. A static lateral load, which is roughly distributed where actual seismic equivalent effects occur, is applied to the structure. The structure is then displaced (pushed over) incrementally to the level of deformation expected during the earthquake (target displacement). Base shear and corresponding displacements at each displacement stage are then used to build the pushover curve. The nonlinear load-deformation characteristics of individual components and elements of the structure must be considered in the model to account for the possibility of exceeding elastic limits

Pushover analysis may be used for any structure, and is strongly recommended for the analysis of irregular buildings. It should not be used for structures in which higher modes are significant.

7.2 Procedure to perform pushover analysis

For the purpose of comparison, **Model #2 (F4URC)** was chosen to perform the pushover analysis.

This section presents the steps used to perform the pushover analysis of a three-dimensional building model using ETABS Nonlinear version 9.0.4 program.

1. A 3-D model was created for the building as would be the case for a standard linear static analysis. The building's grid and story data were set as a first step. The frame section properties for beams, columns, slabs and shear walls are then defined. Table 7.1 shows a sample of such section properties:

Table 7.1: Section properties

Section name	Section definition (m)
C1	0.4 x 0.4
C2	0.5 x 0.5
C3	0.6 x 0.6
C4	0.4 x 0.4
C5	0.6 x 0.6
B1	0.6 x 0.3
B2	0.4 x 0.3
CW1	5.0 x 0.25
CW2	2.0 x 0.25
CW3	0.5 x 0.25
SLAB1	Solid slab, 0.2 thickness, 1-way load distribution

2. The geometry of the building was constructed by creating columns and drawing beams, slabs and walls as shown in Figures 7.1 & 7.2:

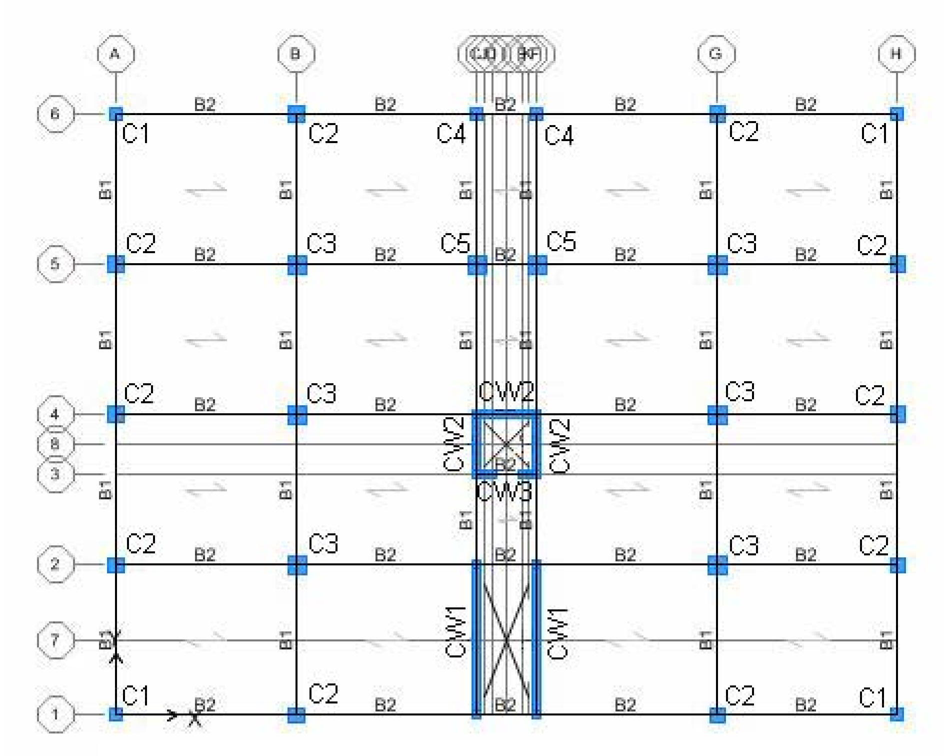

Figure 7.1: Plan for pushover model

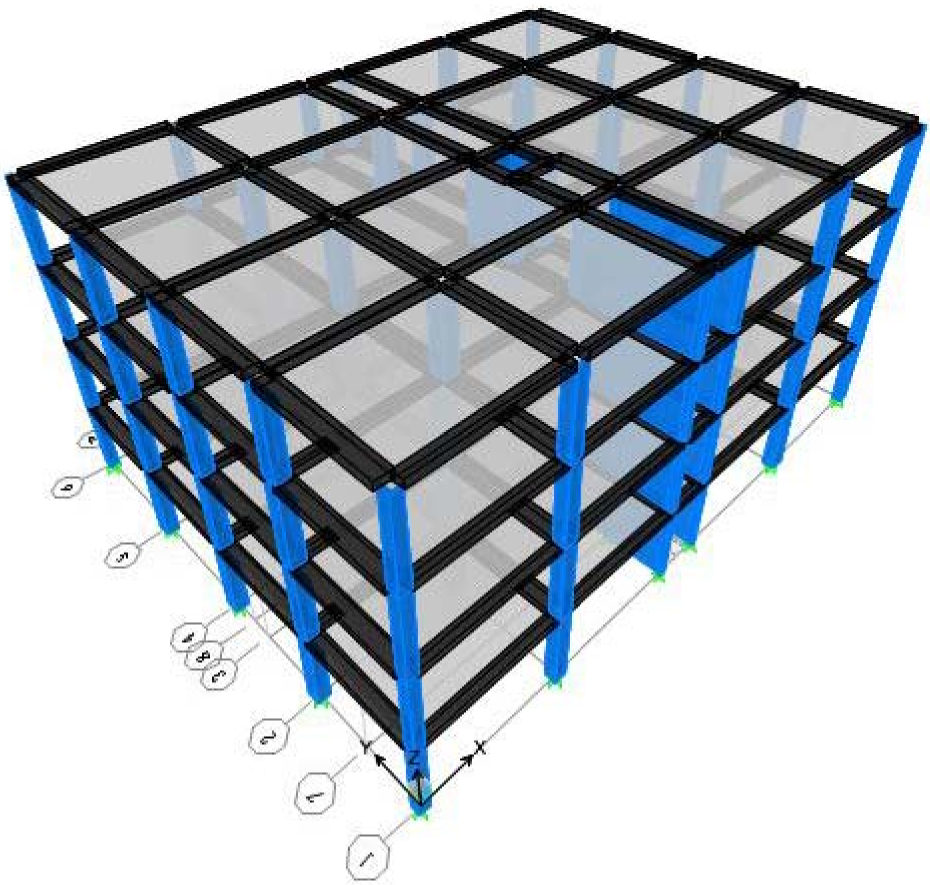

Figure 7.2: Three dimensional pushover model

3. Static load cases were defined including: dead load, live load, seismic load in x-direction, and seismic load in y-direction, as shown in Table 7.2:

Table 7.2: Static load cases for pushover analysis

Static load case	Load magnitude	Description
Dead load	$5kN/m^2$	Include weight of cement mortar, tiles, compacted sand, plastering and partitions. (excluding the own weight of the 20cm solid slab which is added by the program and is equal to $5kN/m^2$)
Live Load	$2kN/m^2$	For residential buildings
Seismic X	According to UBC 97	Soil type C, seismic zone factor 0.15, C_a =0.18, C_v =0.25, R =5.5, and I =1
Seismic Y	According to UBC 97	Soil type C, seismic zone factor 0.15, C_a =0.18, C_v =0.25, R =5.5, and I =1

4. Dead load and live load were assigned as an area load to the slabs.
5. Frame Nonlinear Hinge Properties were defined using M3 hinge for beams, and P-M2-M3 hinge for columns.
6. Frame Nonlinear Hinges were assigned to beams and columns at the start and the end of each member.
7. The basic linear analysis was performed.
8. Concrete design was carried out so that the entered reinforcing steel was checked by the program, and taken into consideration in the next analysis.
9. Static nonlinear/ pushover cases were defined.
10. Static nonlinear analysis was performed.

11. Static pushover curve for the building was displayed as the base shear versus the displacement at the top of the building.

7.3 Results

The result of the pushover analysis is a curve of "Resultant Base Shear vs. Monitored Displacement" as shown in Figure 7.3

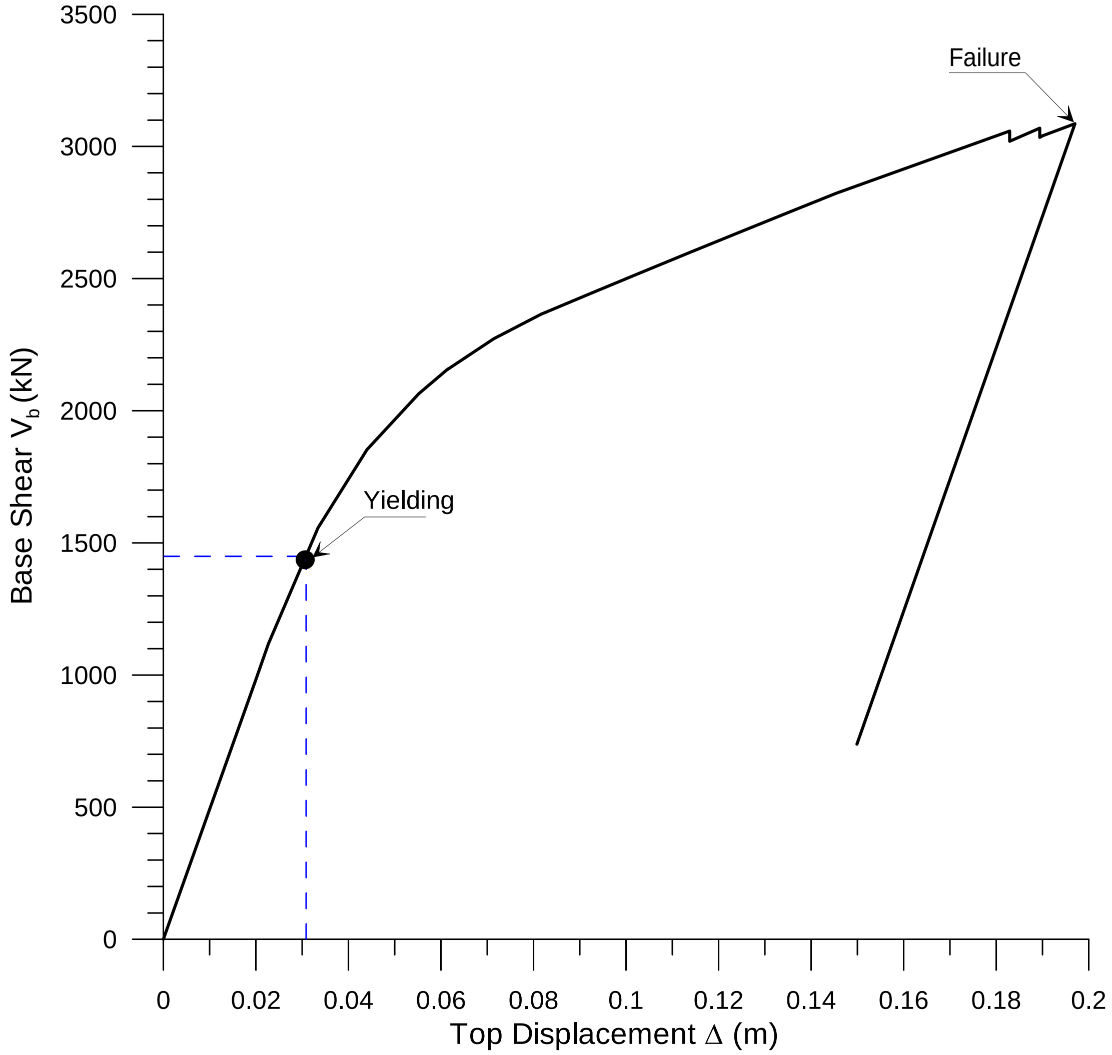

Figure 7.3: Pushover curve for seismic forces in the x-direction

The blob on the curve represents the point at which the building first yields, which is located at the end of the linear part of the curve. This point has coordinates of (Δ , V_b) = (0.031 m , 1460 kN).

7.4 Comparison

The pushover model presented in this chapter is equivalent to **Model #2 (F4URC)**, see Section 6.3.2, Table 6.6 and Table 6.9. The comparison of results between the two methods; Kerstin Lang's vulnerability method and static nonlinear pushover analysis performed by ETABS 9.0.4 can be summarized in Table 7.3:

Table 7.3: Comparison between Kerstin Lang and ETABS

Method	Vulnerability Method	Nonlinear Pushover Analysis
Base shear of the building at first yield (kN)	1314	1460
Top displacement of the building at first yield (m)	0.0285	0.031

- Vulnerability method gave a value for the base shear of the building 10% less than the pushover method.
- Vulnerability method gave a value for the top displacement of the building 8% less than the pushover method.
- Results of the two methods are considered to be sufficiently close to give an indication that the vulnerability method being adopted in this study is consistent with more refined models such as the pushover analysis.

CHAPTER 8 Results and Conclusions

8.1 Introduction

The aim of this study was to assess vulnerability of existing residential buildings in Amman city. The method developed by Kerstin Lang was found to be credible and was adopted during this study, and to support that a pushover model was performed to stand as a tool to verify the results of the vulnerability method.

Models were chosen to reflect the actual condition of buildings in Amman, by studying the effect of many parameters; number of floors, soil type, reinforcement, and the seismic demands.

The survey accomplished during this study has significantly facilitated the process of selecting the prototypes, and was a powerful tool to reflect the actual condition of buildings in Amman. For that reason, it was extremely important to connect between the survey data and the vulnerability results by implementing a cost of repair study as discussed in the following section.

8.2 Statistical Analysis of the Results

Quantifying the economical losses for each type of buildings according to its vulnerability is considered to be an important issue for decision makers to decide appropriate risk management strategies in case an earthquake occurs. An appropriate method to quantify economical losses is to approximately calculate the cost of repair needed for each type of residential buildings according to its vulnerability function for different soil profiles and earthquakes, depending on the statistical data.

8.2.1 Economical losses for each damage grade

Economical losses are expressed in terms of the cost of repair for each damage grade. Table 8.1 shows the general elements of residential buildings in Amman and how much of the value of those elements is being lost at each damage grade, and knowing the average rate of cost for residential buildings, the cost of repair is calculated.

- Rate of cost = 328 JD/m^2
- Structural elements (concrete work) represent 37% of the total cost of the building.
- Nonstructural elements (all elements in Table 8.1 other than the concrete work) represent 63% of the total cost of the building.

Table 8.1: Cost of repair for each damage grade

Elements	% of Cost	Cost (JD/m2)	DG1 % of Loss	DG1 Cost of Repair (JD/m2)	DG2 % of Loss	DG2 Cost of Repair (JD/m2)	DG3 % of Loss	DG3 Cost of Repair (JD/m2)	DG4 % of Loss	DG4 Cost of Repair (JD/m2)
Skeleton (concrete work)	100	121	0	0	10	12.1	35	42.5	100	121
Excavation	4.2	9	0	0	0	0	0	0	100	9
Masonry (partitions)	2.4	5	0	0	20	1.0	70	3.5	100	5.0
Roofing and insulation	3.8	8	0	0	0	0	80	6.3	100	8
Carpentery and joinery (doors)	5.1	11	0	0	0	0	20	2.1	100	11
Metal and aluminum work (windows)	6.3	13	0	0	10	1.3	80	10.4	100	13
Finishes (tiles)	13.2	27	10	2.7	20	5.5	50	13.6	100	27
Stone work	18	37	5	1.9	10	3.7	25	9.3	100	37
Plastering	4.9	10	20	2.1	40	4.1	80	8.1	100	10
Painting and decoration	3.4	7	20	1.4	40	2.8	80	5.6	100	7.0
Mechanical works (piping, heating system)	21.7	45	0	0	5	2.2	20	9.0	100	45
Electrical works	17	35	0	0	0	0	12	4.2	100	35
Sum of Cost of Repair(JD/m2)				**8.1**		**32.7**		**114.6**		**328**
SUM of % of Loss				**2.5 %**		**10 %**		**35 %**		**100 %**

8.2.2 Cost of repair for each type of buildings

Types of buildings in this section are indicated by the number of floors. This study has analyzed five different numbers of floors; two floors, three floors, four floors, five floors, and seven floors. Those models represented the whole residential building stock in Amman.

Table 8.2 shows the percentage of loss of the value of the building and the cost of repair (based on the vulnerability of the building) for soil type C and D due to the earthquake specified by the code.

Table 8.2: Cost of repair for each building type for the code Earthquake

	Model	#6(F2URC)	#4(F3URC)	#1(F4RC)	#3(F5RC)	#5(F7RC)
	No. of Floors	1- 2 Floors	3 Floors	4 Floors	5 Floors	6- 8 Floors
	% from Statistics	20 %	16 %	11 %	34 %	19 %
	Correction Factor	0.4	0.48	0.44	1.7	1.33
	Corrected %	9.2 %	11.0 %	10.1 %	39.1 %	30.6 %
SOIL TYPE C 75%	Damage Grade	0.44	0.8	1.23	0.98	0.87
	% of Loss	1.1 %	2.0 %	4.32 %	2.45 %	2.18 %
	Cost of Repair (JD/m2)	**3.6**	**6.6**	**14.2**	**8.0**	**7.2**
SOIL TYPE D 25%	Damage Grade	0.53	1.06	1.47	1.18	1.09
	% of Loss	1.33 %	2.95 %	6.03 %	3.85 %	3.18 %
	Cost of Repair (JD/m2)	**4.4**	**9.7**	**19.8**	**12.6**	**10.4**

Table 8.3 shows the percentage of loss of the value of the building and the cost of repair (based on the vulnerability of the building) for soil type C and D if double the earthquake specified by the code happened.

Table 8.3: Cost of repair for each building type for double the code Earthquake

	Model	#6(F2URC)	#4(F3URC)	#1(F4RC)	#3(F5RC)	#5(F7RC)
	No. of Floors	**1- 2 Floors**	**3 Floors**	**4 Floors**	**5 Floors**	**6- 8 Floors**
	% from Statistics	20 %	16 %	11 %	34 %	19 %
	Correction Factor	0.4	0.48	0.44	1.7	1.33
	Corrected %	9.2 %	11.0 %	10.1 %	39.1 %	30.6 %
SOIL TYPE C 75%	**Damage Grade**	0.86	2.06	2.04	1.67	1.6
	% of Loss	2.15 %	11.5 %	11 %	7.53 %	7 %
	Cost of Repair (JD/m2)	**7.1**	**37.7**	**36.1**	**24.7**	**23.0**
SOIL TYPE D 25%	**Damage Grade**	1.13	2.49	2.49	2.06	2.05
	% of Loss	3.48 %	22.25 %	22.25 %	11.5 %	11.25 %
	Cost of Repair (JD/m2)	**11.4**	**73.0**	**73.0**	**37.7**	**36.9**

- A correction factor was introduced for the size of the building based only on the number of floors.
- The damage grade for each building type was linearly interpolated using the vulnerability results of the corresponding model.
- Percentage of loss for each building type was also linearly interpolated using the % of loss for each damage grade from Table 8.1.
- Cost of repair for each building type was determined by multiplying the % of loss by the rate of cost.

- Cost of repair of the whole building stock is determined by summing the multiplications of (cost of repair * corrected %) for all building types.
- The calculated cost of repair will be increased 20% of its value to account for irregularities (horizontal and vertical irregularities), major defects in construction, lack of good reinforcement detailing, and poor material quality.

8.3 Conclusions

Findings of this study can be summarized as follows:

8.3.1 General Conclusions

[1] Buildings that have unreinforced shear walls are considered not vulnerable and experience DG1/A for soil type C and D, and for the seismic demand of zone 2A according to the code. However, with the strength of the soil decreased to type D (stiff soil) and doubling the seismic demand the response of the building appeared to change drastically to DG2/B, which means that the structure tends to be vulnerable and may need structural investigation since cracks in columns and beams of frames and in structural walls are going to occur.

[2] Buildings that have reinforced shear walls are considered safe (not vulnerable) and experience DG1/A for soil type C and D, and for the seismic demand of zone 2A according to the code. However, with the strength of the soil decreased to type D (stiff soil) and doubling the seismic demand the response of the building appeared to change moderately to DG2/A, which means that the structure is still safe and not vulnerable since only cracks in partitions and infill walls, and falling of plaster is expected to occur.

[3] Buildings with three floors or less are considered not vulnerable under the seismic demand of the code although the shear walls are not reinforced, but doubling the seismic demand leads them to experience cracks in partitions and plaster (DG1/A).

[4] Buildings with four floors and more are considered not to be vulnerable for the code seismic demand with either soil type C or D, but increasing the seismic demand changes the response to DG2/A. Usually, those buildings are assumed to have reinforced shear walls and well designed structural elements as they take more engineering input than lower buildings. Therefore under increased seismic demands those buildings are considered not to be vulnerable.

[5] Soil investigation is very important before the construction of any structure since it has a major role on the response of structures under earthquakes especially for higher buildings and buildings with plain concrete shear walls.

[6] Amman city is located on the boarder between zone 2A and 2B, and since Amman extends within a wide area and is expanding progressively, it was not an exaggeration to consider the double spectral displacement of the code in this study.

[7] Reinforcing the shear walls have a major role in making the structures less vulnerable even under larger earthquakes, and in reducing the effect of week soil profiles.

[8] For higher buildings it is not favorable to have huge floor masses unless additional core area is introduced, since increasing the stiffness of the structure along with increasing the height means attracting more seismic forces.

8.3.2 Quantified Conclusions

[9] If the earthquake specified by the code occurs, and the soil profile was type C, the cost of repair of the whole residential building stock in Amman will be 8 JD/m^2.

[10] If a code earthquake occurs, and the soil profile was type D, the cost of repair of the whole residential building stock in Amman will be 12 JD/m^2.

[11] If double the code earthquake occurs, and the soil profile was type C, the cost of repair of the whole residential building stock in Amman will be 25 JD/m^2.

[12] If double the code earthquake occurs, and the soil profile was type D, the cost of repair of the whole residential building stock in Amman will be 43 JD/m^2.

[13] In general, if the code earthquake occurs, the cost of repair of the whole residential building stock in Amman will be 9 JD/m^2.

[14] In general, if double the code earthquake occurs, the cost of repair of the whole residential building stock in Amman will be 30 JD/m^2.

[15] Taking irregularities and construction defects into account; the cost of repair becomes 11 JD/m^2 for the code earthquake, and 35 JD/m^2 for the doubled seismic demand.

8.4 Recommendations

[1] Future work to detect the precise effect of horizontal and vertical irregularities on the vulnerability of buildings should be undertaken.

[2] Applying vulnerability methods is required on a wide scale to cover the rest of the country, to identify specifically the seismic condition of other types of buildings such as; commercial buildings, industrial buildings, and rural type of building construction.

[3] Field investigation is a very useful tool in vulnerability studies since it reflects the actual practices at the field of construction, which may be different than permit plans.

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APPENDIX

A.1 Classification of damage to reinforced concrete buildings according to EMS 98

- **Grade 1:** Negligible to slight damage (no structural damage, slight non-structural damage).

  Fine cracks in plaster over frame members or in walls at the base. Fine cracks in partitions and infills.

- **Grade 2:** Moderate damage (slight structural damage, moderate non-structural damage).

  Cracks in columns and beams of frames and in structural walls. Cracks in partition and infill walls, fall of brittle cladding and plaster. Falling of mortar from joints of wall panels.

- **Grade 3:** Substantial to heavy damage (moderate structural damage, heavy non-structural damage).

  Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of concrete cover, buckling of reinforced rods. Large cracks in partitions and infill walls. Failure of individual infill panels.

- **Grade 4:** very heavy damage (heavy structural damage, very heavy non-structural damage).

  Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns. Collapse of a few columns or of a single upper floor.

- **Grade 5:** Destruction (very heavy structural damage)

  Collapse of ground floor or parts of the building (e.g. wings of buildings).

A.2 Seismic zoning map of Jordan

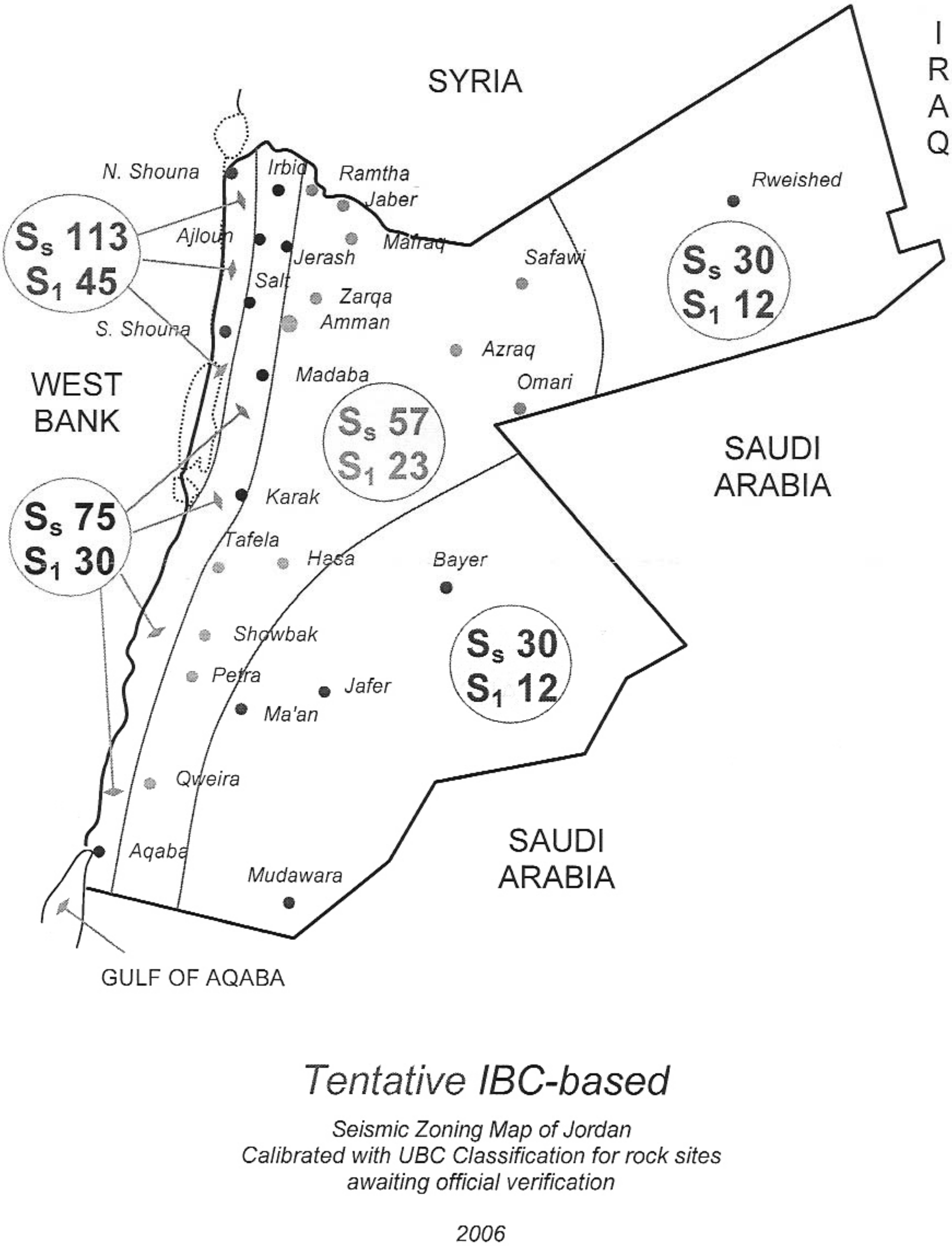

Figure A.1: Seismic zoning map of Jordan,
by Dr. Nazzal Armouti

دراسة إحصائية لقابلية المباني السكنية القائمة للإصابة نتيجة التأثيرات الزلزالية في مدينة عمــان

إعداد

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ملخـــــص

إنَ تقييم قابلية المباني للإصابة بتأثير الزلازل المتوقعة مهم من أجل تحديد خصائص المباني التي تجعلها أكثر عرضة من غيرها للضرر تحت تأثير الزلازل، وذلك يعتبر خطوة أولى في سبيل التخفيف من الأضرار و الخسائر الناتجة عن تأثير الزلازل.

وفي حين أنه لم تحدث أي زلازل شديدة في مدينة عمان خلال العقود الأخيرة، فإن دوال قابلية الإصابة التي تعتمد على سجلات و مشاهدات من الأضرار التي تخلفها الزلازل غير ممكنة التطبيق، حيث أن مثل تلك السجلات غير متوفرة.

لذلك تم إتباع طريقة تحليلية موثقة تعتمد على أساليب ستاتيكية غير خطية، و هي طريقة وضعت من قبل الباحثة (كيرستن لانج) من سويسرا، و قد تم تطبيقها على ستة من النماذج الواقعية المختارة التي تحاكي المباني السكنية القائمة في مدينة عمان. وقد تمثلت النتائج على شكل

دوال قابلية الإصابة التي تعبر عن الضرر المتوقع لتلك النماذج من المباني بدلالة المدخلات الزلزالية الخاصة بمدينة عمان.

بالإضافة لما سبق فقد تم استخدام البرنامج ETABS لإجراء التحليل الإنشائي غير الخطي لنموذج من المباني بواسطة تفعيل (الطريقة الدفعية) (pushover method) التي تعتمد على الدفع المتزايد على المبنى حتى وصوله إلى مرحلة الفشل. وقد تمت مقارنة النتائج المستخلصة من الطريقة التحليلية مع تلك الناتجة عن نموذج الطريقة الدفعية.

وتتميز هذه الدراسة باحتوائها على معلومات إحصائية عن عينة من المباني السكنية القائمة في مدينة عمان تقدر ب 110 عينة تم جمعها من قبل الباحثة، وقد تم الإستناد على هذه الإحصائية في إعطاء توقعات عن الكلفة المادية اللازمة لترميم المباني نتيجة حدوث الزلزال بالإعتماد على إمكانية حدوث الإصابة لدى المبنى السكني قيد الدراسة حسب خصائصه. حيث تم رصد عدد من تلك الخصائص الهامة التي تؤثر على تصرف المبنى تحت تأثير الزلزال مثل: عدد الطوابق، وجود التسليح في جدران القص أو عدم وجوده، قوة تحمل التربة، و مقدار الزلزال المتوقع.

وقد تم الإستنتاج بأن المباني التي لا تحتوي على تسليح في جدران القص هي أكثر قابلية للإصابة من تلك التي تحتوي على تسليح تحت نفس الظروف. وقد وجد أن المباني التي تتألف من أربعة طوابق فأكثر ليست معرضة للإصابة و أنها تبدي تصرفا جيدا للزلزال المتوقع على إفتراض أنها تحتوي على أعمدة و جدران قص مصممة و مسلحة و أنها لا تحتوي على عيوب إنشائية أو شكل غير منتظم.

و تشير الدراسة إلى أن الكلفة الإجمالية اللازمة لترميم جميع المباني السكنية القائمة نتيجة حدوث الزلزال المتوقع في مدينة عمان تقدر بتسعة دنانير للمتر المربع من مساحة المبنى، في حين أن هذه الكلفة ترتفع إلى ثلاثين ديناراً للمتر المربع في حال حدوث زلزال بقوة مضاعفة.