بسم الله الرحمن الرحيم

Sudan University for Science And Technology

College of Graduate Studies

Application of UBC and IBC Seismic Codes For R.C. Buildings Design in Sudan

تطبيق مدونتي الزلازل UBC و IBC

لتصميم المباني الخرسانية المسلحة في السودان

By

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بسم الله الرحمن الرحيم

الآيـة

قال تعالى :

(... ى رَبُّكَ إِلَى النَّحْلِ(أَنِ اتَّخِذِي مِنَ الْجِبَالِ بُيُوتاً وَمِنَ الشَّجَرِ
وَمِمَّا يَعْرِشُونَ)

صدق الله العظيم

سورة النحل [68]

Dedication

I would like to dedicate this research

to any person who helped

me throughout in my life especially

my parents
&
my friends

Acknowledgement

Praise to my God for His blessing and guidance which made this project come out.

I wish to extend my greatest gratitude and gratefulness to my supervisor, Dr. Abdel Rahman Elzubair for his valuable guidance, advice and suggestions throughout this project. His efforts and concern that enabled me to achieve my project.

A lot of thanks to all staff of Civil Engineering Department, Sudan University of Science and Technology and to all my friends who provided me with a high level of information . My thanks are extended also to all my colleagues, postgraduates of Structural Engineering Department for their support and cooperation.

Abstract

This research dealt with the investigation of possibility of the use of the two representative seismic codes, UBC 1997, and IBC 2006, for seismic force analysis of R.C. buildings in Sudan. The theory of the seismic forces analysis was studied and presented.

A R.C. building of eight storeys constructed in two sites, a very low seismic active site and a moderate seismic active site in Sudan which was analyzed by using ETABSv9.7.4 computer program. Utilizing data of the two sites, application of the provisions of each code for each case was conducted. A comparison was made for the provisions of the two codes, and the difficulties of using each code recorded. Then, the results obtained were analyzed, compared and discussed .Based on the comparison recommendations were drawn to which of the codes is a suitable choice for use in Sudan.

The study concluded that the IBC 2006 code is more suitable and is to be adopted for use in Sudan.

ملخص الدراسة

هذا البحث اهتم بالتحقق من امكانية استخدام اشهر مدونتي زلازل وهما UBC 1997 وIBC 2006، لتحليل قوى الزلازل بالنسبة للمباني الخرسانية المسلحة في السودان . قد تم دراسة وعرض النظريات الخاصة بتحليل قوى الزلازل .

تم تحليل مبنى خرسانة مسلحة مكونة من ثمانية طوابق مشيدة في منطقتين في السودان ، احداهما ذات نشاط زلزالي منخفض جداً وأخرى ذات نشاط زلزالي متوسط باستخدام برنامج الكمبيوتر ETABSv9.7.4. باستخدام بيانات المنطقتين تم تطبيق شروط كل مدونة على حدة لكل منطقة واجريت مقارنة لشروط المدونتين و دونت الصعوبات التي تصاحب استخدام اى منهما .

ومن ثم تم تحليل النتائج ومقارنتها ومناقشتها .و بناءاً على المقارنة صيغت بعض التوصيات التي تشير الى ان اي من المدونتين هى الخيار المناسب للاستخدام في السودان .
وأخيراً خلصت الدراسة الى أن المدونة IBC 2006 مناسبة أكثر واوصت باستخدامها في السودان .

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Symbol	Descriptions
Mb	Body-wave magnitude scale.
M_L	Richter magnitude scale or local magnitude scale.
M_s	Surface-wave magnitude scale.
M_w	Moment magnitude.
Ms_t	Storey overturning moment.
N	Number of storeys.
ω^-	Fundamental applied load frequency.
β	Damping ratio.
F	Frequency
T	Fundamental period of vibration
F_i	Lateral force
V_b	Base shear
W_i	Building mass lumped at the i^{th} floor.
ω	Natural frequency.
c	Damping coefficient.
C_c	Critical damping coefficient
S_d	Spectral displacement.
S_v	Spectral velocity.
S_a	Spectral acceleration..
R	Total response.
R⁻	Maximum total response.
r	Individual response of the natural mode
	The modal participation factor
e	Effective modal mass.
U	Relative displacement.
W	Total load

Y	Peak ground acceleration parameters such as Acceleration (a), velocity (v) or displacement (d).
M	Mass matrix
C	Viscous damping matrix
K	Stiffness matrix
U	Displacement vector
Ú	Acceleration
Ü	Velocity.
F(t)	Equivalent lateral forces
Mn	Modal mass
Ψ	Mode- shape factor
D	Dead load
E	Earthquake load.
L	Live load.
P	Incidence factor for live load.
f_i	Distributed lateral force.
F_t	Lateral force at the top of the structure.
F_x, F_i or F_n	Lateral force applied at level x, i, or n
g	Gravity constant.
h_i, h_n or h_x	Height of the level i, n or x above the base
I	Importance factor.
Rω	System factor
Z	Seismic zone factor.
S	Soil factor
T_a	Fundamental period of the structure.
C_a, C_v	Seismic coefficients.
Δst	Storey drift.
F_{px}	Diaphragm force.
M_{st}	Storey overturning moment

V_b	Base shear.
V_x	The storey shear force at any storey x of the building.
W	Total weight of the building.
W_x or W_i	Seismic weight of the x^{th} of i^{th} level.
δ_i	Lateral deflection
θ	Phase angle
S_s, S_1	Ground motion parameters.
Fa, Fv	Seismic Site coefficients
S_{Ds}, S_{D1}	Design ground motion parameters.
S_{Ms}, S_{M1}	Maximum ground motion parameters.

Chapter One

General Introduction

1.1 Introductory Remarks:

Structures in earthquake regions and those subjected to wind loads must be designed to resist the gravitational and lateral forces. These forces will depend on the size and shape of the building, as well as on its geographic location. The maximum probable values of these forces must be established before the design can proceed [1].The probable accuracy of estimating the dead and live loads and the probability of simultaneous occurrence of different combinations of gravity loading, both dead and live, with either wind or earthquake forces as lateral loads is included in limit state design through the use of prescribed factors [1].

In earthquake regions, any inertial loads from the shaking of the ground will be dominant factor in influencing the building's structural form, design and cost. Earthquake as an inertial problem, the building's dynamic response plays a large part in influencing and in estimating the effective loading on the structure .Therefore, in contrast to structural response to essentially static gravity loading or even to wind loads, which can be validly treated as static loads, the dynamic character of the response to earthquake excitation can seldom be ignored [1]. It is more difficult to estimate earthquake loading from the past events like gravity loads. These, in different codes of practice, although rationally based, tend to be empirical in their presentation [1]. Although some parts of Sudan are earthquake regions, there is no seismic code to be used in the analysis, and design of buildings. Therefore, in this research, methods from reasonably representative modern seismic codes are being discussed, and to determine which of them is suitable for Sudan.

1.2 Problem statement:

Some parts of Sudan are earthquake regions and structures built in these regions must be designed for seismic loads .Engineers tend to use some reasonably representative modern codes such as uniform building code UBC 1997 and international building code IBC 2000.Thus, determining which seismic code would be applicable for Sudan seismicity will be of great benefit .This research attempts to apply the two seismic codes for Sudan seismicity requirements. Also, the research aims to recommend the provisions from seismic codes which are preferred for use in Sudan. The possibility of recommending new provisions suitable for Sudan conditions is also investigated.

1.3 Research Objectives:

1.3.1 General objective:

Application of UBC and IBC seismic codes for Sudan seismicity requirements.

1.3.2 Specific objectives:

The specific objectives are to**:**

a. review and study different codes to point out requirements of seismic design.
b. review the specific conditions adopted in different seismic codes and compare them.
c. out line the seismic conditions of Sudan.
d. apply and compare the two seismic codes to show how they cover almost all Sudan conditions and recommend one of them to be used in seismic design.

1.4Research methodology:

The research methodology for this study focused on the following:

1. Introducing the earthquake forces nature, the causes of earthquakes, the behavior of building during earthquakes and the types of analysis of the earthquake forces.
2. The evaluation of the chosen seismic codes UBC 1997and IBC 2006 provisions.
3. The application of provisions of the two codes on R.C. building using Sudan seismic data of earthquakes by applying response spectrum analysis in ETABSv9.7.4 computer program, then comparison of the results.

1.5 Research Outlines:

Chapter one presents introductory remarks , problem statement , the objectives of the research, the research methodology and outlines of thesis.

Chapter two is concerned with literature review that covers the previous studies in the field of the research which includes the nature of earthquakes forces and the main concepts of it.

Chapter three deals with the theory of seismic forces, types of analysis and the provisions of the seismic codes.

Chapter four deals with the application on a R.C .building, results of analysis by using ETABS v9.7.4 computer program with the provisions of the two chosen codes and discussion of the results.

Chapter five presents the conclusions and recommendations.

Chapter two

Literature Review

2.1. Introduction (definition) and nature of earthquakes

2.1.1 Definitions:

An earthquake is a spasm of ground energy in the earth's lithosphere (i.e. the crust plus part of the upper mantle). This energy arises mainly from stress built up during tectonic process, which consist of interaction between the crust and the interior of the earth. In some parts of the world; earthquakes are associated with volcanic activity, collapse of cave roofs or human activity, like mining, applying or removing of large loads (impounding of large amounts of water behind earth dams) and explosions (***Chopra and chakrabarti, 1975***) [2]. Other definitions, an earthquake is a phenomenon during which strong vibrations occur in the ground due to release of enormous energy within a short period of time causing sudden disturbance in the earth's crust or upper mantle. It may, therefore, also be defined as a sudden transient motion or series of motions of the ground originating in a limit region and spreading from there in all directions, great forces acting deep in earth put stress on the rock, which then bends and changes the volume (strain) of the rock .The rock can only deformed so far before it breaks .When the rock breaks , waves of energy are sent through in the earth. These waves of energy, called seismic waves, are that caused the ground to tremble and shake during an earthquake. As earthquake's ground motion pass under each structure, they impart kinetic energy into the structure. Through its design and constructions, the structure must absorb and control this energy as it is dissipated throughout the structure, else it will deform and perhaps collapse [3].

From an engineering point of view, the most important are the earthquakes of tectonic origin, that is, those associated with large-scale strains in the earth's crust. It so because of the frequency of tectonic earthquakes, the energy they release, and the extent of areas they affect [3].

In major earthquakes, a chain reaction would take place along the entire length of slip, but at any given instant .The earthquake origin would lie in a small volume of crust- practically a point-and the origin travel a long the fault. However, some seismologists hold that earthquake originate in phase change in rock, a companied by volume changes in relative small volumes of the crust, i.e. volume changes of rock to reach equilibrium may be due to important changes in litho static compression caused by migration of the material toward or away from the surface of the earth, or they may be due to the application or removable of large loads [3].

An upper limit to the ground acceleration has not been established under the assumption that strong ground motions caused by phase change of rocks .On the other hand the maximum ground velocity that can be transmitted is limited by the breaking strains of rocks and the velocities of shear waves. It may be contended there is no absolute upper limit to the earthquake intensity. It flows that no matter how conservative a design is, there is always a finite probability of structural failure in any finite interval of time, and there is certainty of eventual failure unless the structure in question is purposely demolished .The conclusion is valid even in so called non seismic areas. Seismic regionalization must therefore be taken in sense of greater or less risk, and one acceptable basis for maps of ‘maximum probable intensity” is the mean recurrence times with which they may be associated [3].

When scientists find the causes of the strange animal behavior, they may be able to predict earthquakes within hours. A government agency in China has reported that strange animal behaviors were observed just hours before an earthquake. Cattle, sheep, mules, and horses would not enter corrals. Rats fled their homes. Hibernating snakes left their burrows early. Pigeons flew continuously and did not return to their nests. Rabbits raised their ears, jumped about aimlessly, and bumped into things. Fish jumped above water surfaces. China was not the only country to report such unusual animal behavior. Late on May 6, 1976, an earthquake shook a town in Italy. Before the earthquake, pet birds flapped their wings and shrieked. Mice and rats ran in circles. Dogs barked and howled. Perhaps the animals sensed the coming earthquake? For many years farmers throughout the world have told stories about changes in animal's behavior just before an earthquake. Chinese scientists were among the first to believe these stories might have a scientific basis. They have even proposed that zoo animals might forewarn people of a coming earthquake. Scientists in many countries are interested in finding the causes for the strange behavior. They have suggested that one or more of the following may be possible causes:

a. slight changes in the earth's magnetic field;
b. increased amounts of electricity in the air;
c. very small air pressure changes;
d. changes in noise level
e. gas escaping from the ground [4].

2.1.2 The nature of the earthquake forces:

Earthquake loading consist of the inertial forces of the building mass that result from the shaking of its foundation by a seismic disturbance. Earthquake resistant design concentrates particularly on the translational inertia forces, whose effects on a building are more significant than the vertical or rotational shaking components [1].

Other earthquake forces may exist, such as:

i. Due to land sliding.
ii. Subsidence.
iii. Active faulting below the foundation.
iv. Liquefaction of the local sub grade as a result of vibration.

The four mentioned above forces are called ***local effects***. So many suggest instead the selection of alternative site. [1]. Where earthquakes occur, their intensity is related inversely to their frequency of occurrence. e.g.:

a. Serve earthquake $\longrightarrow$ rare to happen.
b. Moderate earthquake $\longrightarrow$ once happen.
c. Minor earthquake $\longrightarrow$ relatively happen frequently. [1]

In (c) above, there is no any damage. In (b) no structural elements damage but accepting the probability of non-structural elements damage. In (a) accepting probability of structural elements as well as non structural elements damage, but no collapse [1].

Ground motion resulting from earthquakes presents unique challenges to the design of structures. The forces that a structure must resist in an earthquake result directly from the distortions caused by the motion of the ground that supports it are the response – magnitude .The distribution of forces and displacements of a structure resulting from such ground motion is influenced by:

i. The properties of the structure.
ii. The type of foundation [5].
iii. The character of the existing motion.

2.2 The characters and components of Earthquake forces

Earthquake produce large – magnitude forces of short duration that must be resisted by a structure without causing collapse and preferably without significant damage to the structural members, lessons from past earthquakes and researches have provided technical solutions that will minimize Earthquake Forces Characteristics and Components[5].

Loss of life and properties damage associated with earthquakes. For material such as concrete that lack inherent inelastic deformability or ductility, a critical part of the solution is to provide special detailing of the reinforcement to assure a ductile response to lateral forces. Inelastic deformability is the ability of a structure to sustain gravity loads as it deforms laterally beyond the stage where the deformations are recoverable, i.e beyond the stage where no residual deformations remain in a structure once the earthquake motion subsides irrecoverable deformations are associated with damage, while recoverable deformations are associated with no damage [5].

Fig (2.1) illustrates a simplified representation of a building during an earthquake. As the ground in which the building resets is displaced, the base of the building moves with it. However, the inertia of the building mass resists this motion and causes the building to suffer a distortion (greatly exaggerated in the Fig 2.1). This distortion wave travels along the height of structure. The continued shaking of the base causes the building to undergo a complex series of oscillations [5]

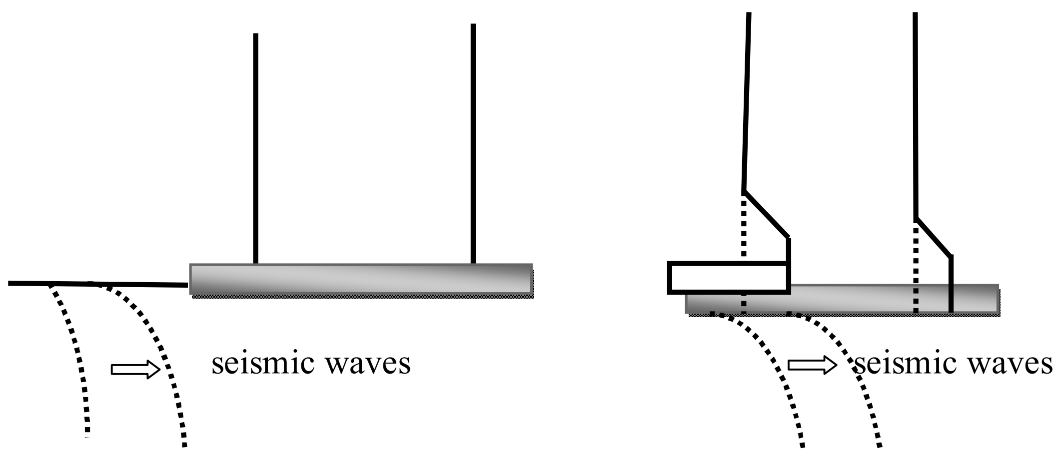


Fig (2.1) Simplified Representation of building Behavior during an Earthquake[5].

It is important to draw a distinction between forces due to wind and those produced by earthquakes. These forces are often thought of as being similar, just because codes specify design wind as well as earthquake forces in the terms of equivalent static forces, although both wind and earthquake forces are dynamic (varying in time). In character, a basic difference exists in the manner in which they are induced in a structure. Whereas wind loads are external loads applied and hence proportional to the exposed surface of the structure, but the earthquake forces are essentially inertia forces. The latter result from the distortion produced by both the earthquake motion and the inertial resistance of the structure.

Their magnitude is a function of the mass of the structure rather than its exposed surface. Also, in contrast to structural response to essentially static gravity loading or even to wind loads, which can be validly treated as static loads, the dynamic character of the response to earthquake excitation is unlikely to be neglected[5].With exception of dead loading, the loads on a building cannot be assessed accurately. While maximum gravity live loads can be anticipated approximately from previous field observations, wind and earthquake loadings are random nature, more difficult to measure from past evens, and more difficult to predict with confidence. The application of probabilistic theory has helped to rationalize, if not in every case to simplify the approaches to estimating wind and earthquake loading [1].

The earthquake ground motion quantity most commonly used in analytical studies is the time-wise variation of the ground acceleration in the immediate vicinity of a structure. At any point, the ground acceleration may be described by horizontal components along two perpendicular directions and a vertical component. In addition, rocking and twisting (rotational) components may be present; however, these are usually negligible. Because building and most other structure are most sensitive to horizontal or lateral distortions, it has been the practice in most instances to consider structural response to the horizontal components of ground motion only. The effect of the vertical components of ground motion generally has not been considered significant enough to merit special attention. In most instances, a further simplification of the actual three- dimensional response of structure is made by assuming horizontal acceleration components to act non-concurrently in the direction of each principle plan axis of a building. It is implicitly assumed that a building designed by this approach will have adequate resistance against the resultant acceleration acting in any direction [5].

The complete system of inertia forces in a structure can be determined only by evaluating the acceleration of every mass particle. The analysis can greatly simplified if the deflections of structure can be defined adequately by a limited number of displacement components or ordinates. The number of displacement components required to specify the position of all significant mass particles in a structure is called the number of degrees of freedom of the structure. In the so- called lumped-mass idealization, the mass of the structure is assumed concentrated at a number of discrete locations. Because the floor and roof elements (diaphragms) in a building are relatively heavy, a large proportion of the building mass is concentrated in these elements. For the structural analysis purpose, the mass of other building components such as walls and columns, and that associated with the superimposed dead loads, are normally assumed concentrated at the floor and roof levels [5].

2.3 Structure behavior or response due to seismic forces:

To know the exact behavior of the structure, better to in light some terms concerned. The term dynamic may be defined simply as time varying. Thus dynamic load is any of which the magnitude, the direction or position varies with time, similarly, the structural response to dynamic load, i.e. the resulting deflection and stresses, is also time-varying or dynamic[6]. Two basically different approaches are available for evaluating structural response to dynamic loads:

a. Deterministic.
b. Non-deterministic.

The choice of method depends upon how the loading is defined, if the time variation of loading is fully known, it will be referred to herein as a prescribed dynamic loading, and the analysis of structural response is defined as a deterministic analysis. On the other hand, if the time variation is not completely known but can be defined in statistical sense, the loading is termed a random dynamic loading and the structural response analysis is determined as a non –deterministic analysis [6].

In general, the structural response to any dynamic loading is expressed basically in terms of the displacements of the structure. Those deterministic analysis to displacement-time history corresponding to the prescribed loading history, other aspect of the deterministic structural response, such as stresses, strains, internal forces, are usually response, such obtained as a secondary phase of analysis, from the previously established displacements patterns [6]. The simplest quantitative statement about an earthquake consist of grading it by means of single number-the intensity or local destructiveness of the earthquake –in some conventional scale. This is too crude a basis for most engineering purposes.

A more adequate description includes the accelerograms or other time histories of three orthogonal translations components of ground motion at a point: the two horizontal and the vertical components. This description is sufficient for the purpose of computing the effects of the earthquake on buildings of small and moderate size [3].

In special instances the space derivatives of ground accelerations become important. This is the case with the rotational components of ground acceleration for slender structures and the soil strains for large civil-engineering works [3].

2.4 The important factors affecting the potential earthquake damage:

a. The natural rocks factor

The natural rock is the best subsoil from the point of view of its earthquake properties. Sandy soil saturated with water and artificially backfilled land are considered to be particularly critical. The widely-feared liquefaction effects (plasticization of the soil) can occur if an earthquake coincides with high ground water levels. The building may subsequently remain at a slant or both the building and the surrounding terrain may subside. Also deep foundation generally displays better seismic resistance than shallow foundations.

b. Floating of foundation factor:

Foundation Floating can prove advantageous on soft ground, since they may be better able to attenuate resonance action. The risk of subsidence is considerably greater with floating foundations than with deep foundations. “Base isolation” is an anti-seismic construction technique that uses the principle of attenuation to reduce vibrations. The building is isolated from the solid subsoil by damping elements arrange on a foundation ring or foundation plate. Another was employed for the court of Appeals in San Francisco. The building was retroactively more or less mounted on ball bearings which are intended to gently damp down the impact of a future earthquake [7].

c. The height of the building factor:

Tall buildings are more susceptible to damage from strong remote earthquakes than from weak earthquakes close at hand. They normally have a lower resonant frequency and a lower attenuation than low building short-wave Oscillation components in earthquakes are rapidly damped, while the long-wave components (frequency F< 1 HZ) can still make themselves felt at a distance of several hundred kilometers, particularly in the form of surface waves [7].

d. The supporting structure factors:

A distinction can generally be made between rigid and elastic supporting systems. Rigid systems, such as solid wall and ceiling elements, are difficult to deform and transmit the seismic loads through their rigidity. Due to the stiffness and lack of ductility in the supporting structure, however, shear cracks can develop in the building. The problem is that more and more energy must be absorbed through the high rigidity and that more and more material is required for this purpose. Elastic supporting structures, such as reinforced concrete or steels frames, are highly deformable and absorb the applied seismic energy in this way. The nodes connecting the horizontal and vertical elements of the supporting structure are highly stressed, however, and peak loads occur here and on the reinforcing elements (bonds) which must be taken in to account when producing these connection. However, integrated non-supporting partition walls may suffer excessive stresses and break out on account of the major deformation of the frame structure [7].

e. The shape of the building factor:

When parts of different height permanently connected to one another as the case in high-rise buildings with atriums, the various elements in the building can be subjected to considerable torsion stresses by the seismic loads [7].

f. Resonance factor:

Resonance effect can also cause buildings to oscillate so strongly that they hammer against one another. Another effect observed in high-rise buildings is the soft-storey effect: due to applies, atriums or glazed shopping passage, some floors usually near the ground floor-are distinctly “softer” than those above them. These “soft” floors then collapse in an earthquake. A further source of loss potential relates to the standards applied. Many countries do not have their own earthquake standards and simply adopt the corresponding regulations from others, such as the Uniform Building codes from U.S.A. This means, however, that common local seismic effects are not covered [7].

2.5 The strength of earthquakes magnitude and intensity:

During earthquakes the release of crust stresses is believed to involve the fracturing of the rock a long a plane which passes through a point of origin (the hypocenter or focus) of the event. Sometimes, especially in larger shallower earthquakes, this rupture plane, called a Fault, breaks through to the ground surface, where it is known as a Fault trace [2].

The cause and nature of earthquakes is the subject of the study of the science of seismology, and future back ground may be obtained from the books by ***Richter*** (1958), ***Bolt*** (1999) and **Lay** and ***Wallacy*** (1995) [2].

Unfortunately, for non-seismologists at least, understanding the general literature related to earthquakes is impeded by the difficulty by finding precise definition of some fundamental seismological terms. For assistance, definition of some basic terms is set out below [2].

The strength of an earthquake is not an official technical term, but is used in the normal language sense of how strong was that earthquake? Earthquake strength is defined in two ways:

- Firstly: the strength of shaking at any given place (called the intensity).
- Secondly: the total strength (or size) of the event itself (called magnitude, seismic moment, or magnitude moment). These entities are described below [2].

2.5.1. Intensity:

a. Is a quantitative measure of the severity of seismic ground motion at specific site. Over the years, various subjective scales of what is often called felt intensity have been devised, notably the European Macroseismic, which are very similar. The most widely used in the English speaking world is the modified Mercall scale (commonly denoted MM), which is in twelve grades denoted by roman numerals I – XII.
b. Quantitative instrumental measures of intensity include engineering parameters such as peak ground acceleration, peak ground velocity, the Housner spectral intensity, and response spectral in general. Because of the high variability of both subjective and instrumental scales, the correlation between these two approaches to describing intensity is inherently weak [2].

2.5.2 Magnitude:

Is a quantitative measure of the size of an earthquake, related indirectly to the energy released, which is independent of the place of observation. It is calculated from amplitude measurements on seismograms, and is on a logarithmic scale expressed in ordinary numbers and decimals. Unfortunately several magnitude scales exist, of which the four most common ones are described here (M_L, M_s, M_b and M_w) [2].

The following are descriptions of the above scales:

The most commonly used magnitude scale is that devised by and named after Richter and denoted M or ML. It is defined as:

$$M_L = \text{Log } A - \text{Log} A_0 \qquad (2.1)$$

Where

A= is the maximum recorded trace amplitude for a given earthquake at a given distance as written by a wood – Anderson instrument.

A_0= is that for a particular earthquake selected as standard.

The Wood-Anderson seismograph ceases to be useful shakes at distances beyond about 1000km, and hence Richter magnitude is now precisely called local magnitude (**M_L**) to listing it from magnitude measure in the same way but from recording on long-period instruments, which are suitable for more distant events. When these latter magnitudes are measured from surface wave impulses they are denoted by M_s Gutenburg proposed what he called unified magnitude, denoted M or Mb, which is dependence on body waves, and is now generally named body wave magnitude (M_b). This magnitude scale is particularly appropriate for events with a focal depth greater than C. 45km. All three scales M_L, M_b and Ms suffer from saturation at higher values [2].

The most reliable and generally preferred magnitude scale is moment magnitude, Mw. This derived from seismic moment, Mo, which measures the size of an earthquake directly from the energy released, ***Wyss*** and ***Brune*** (1968), through the expression

$$M_0 = \mu AD \qquad (2.2)$$

Where;

μ= is the shear modulus of the medium (and is usually taken as $3X10^{10}$ Nm)

A= is the area of dislocation or fault surface.

D= is the average displacement or slip on that surface.

Seismic moment is a modern alternative to magnitude, which avoids the shortcomings of the latter but, is not readily determined. Up to 1985, seismic moment had generally only been used by seismologists. Moment magnitude is a relatively recent magnitude scale from ***Kanamori*** (1977) and ***Haks*** and ***Kanamori*** (1979), which overcomes the above-mentioned saturation problem of other magnitude scales by incorporating seismic moment in to its definition, such that moment magnitude as in Eq.(2.3)[2].

$$M_w, = \tfrac{2}{3}\mathrm{Log}M_0 - 6.03 \quad (M_0 \text{ in Nm}) \qquad (2.3)$$

2.6 Probabilistic Seismic Hazard of Sudan:

Sudan has a long history of earthquakes but they are confined to certain area. Sudan is known to seismologists as an area of low to moderate seismic activity, because it is located within an intera plate with a relatively low level of seismic activity. The buildup of strain necessary to reach the point of fracture is much slower than in seismic belts. This means that earthquakes in such area are infrequent, but strong earthquakes often occur as catastrophic events to the authorities and people. There are many earthquakes occurred in Sudan in the last century such as Suakin (Eastern Sudan) earthquake in may 1938 M5.8, the Northern Kordofan for earthquake in August and November 1993 M 5.4 and M 4.4 respectively, and earthquake that struck the population of Khartoum state in 1993. In 2003 the Sudanese seismological Network (SSN) was installed and started to work around Khartoum area. There were 22 local events located by the SSN during November 2003 and September 2007. Most of them were located in the Eastern and Western parts of Khartoum area. [8].

For all the coming data about the seismic hazard assessment for Sudan and its vicinity was done using the probabilistic hazard maps and seismic hazard curves for 10% probability of exceedance of 50, 100, 200 and 500 years were prepared using the modified EQRISK software (***Sobahi et. al., 1992***) and other reprocessing and post processing programs [8].

2.7 Seismicity of Sudan

2.7.1 Background:

Most parts of Sudan are seismically active with different return periods, and moderately active. Many researchers investigated the seismicity of Sudan, such as:

Qureashi and ***Sadig*** (1967) studied the seismological nature of the subsurface faulting in Jebel Dumbeir area in Sudan and Rajaf in Republic of South Sudan.

Ambraseys and ***Adam*** (1986) presented some information about the seismicity of Sudan .They divided the region according to the degree of activity into Red Sea area and remaining areas of relatively stable zones. They suggested that there must be structure in this intra plate region capable of producing medium magnitude damaging earthquakes [8].

Mula (1983) postulated, from fault plane considerations on inner plate subdividing the Nobian plate into a Sudan plate marked by a north-east trending sinistrial strike-slip fault from the Red Sea through Sabaloka to Kordofan and Durfor.

The Western boundary is suggested to be also a sinistrial strike-slip fault passing through neighboring countries (Jebel Dumbeir in Sudan and through Juba in Republic of South Sudan).

The eastern boundaries are east African rift system having normal motion.

The Seismological Research Unit (SRU)-Sudan-Catalogue of Seismicity of Sudan, 1996 after two years of active data gathering from different sources inside and abroad, was able in 1996 to compile the catalogue of seismicity of Sudan, which contains all the seismic events which occurred in the period 1632 – 1994. Obviously, the new data have provided a totally new insight in the seismicity of Sudan. The SRU divided Sudan initially into two parts according to the dominated seismicity and tectonic features. These parts were central parts and north parts (the third part is been in the Republic of Southern Sudan). The epicenter distribution of earthquakes in Sudan and its vicinity is shown in Fig(2.2) [8].

2.7.2 Seismotectonics of Sudan

There are six major rift systems in Sudan shown in Fig.(2.3.) Some of these rifts are part of major rift that span the whole African continent and beyond. These rifts are the Central African Rifts System (CARS); the East African Rift System (EARS); the White Nile Rift System (WNRS); the Blue Nile Rift System (BNRS) and the Atbra River Rift System (ARRS).The WNRS,BNRS and ARRS are similar in tectonic character to the South Sudan Republic Rift System (SSRRS) in that they follow similar structural trends and terminate in line at their North – West end. The CARS extends for at least 2000km across Africa. The CARS extend in a North-East to South - West direction from the gulf of Guinea in the Atlantic Ocean through Cameroon, Southern Chad and the central Africa Republic into western Sudan, It passes North of Nuba Mountains and North of Kordofan into the Red Sea (***Browne*** and ***Fairhead***, 1985). There are many faults and shear zones which are parts of some of these rifts. Most of the faults of these rifts have experienced some earthquakes of various magnitudes during historical and instrument events Fig. (2.3) [8].

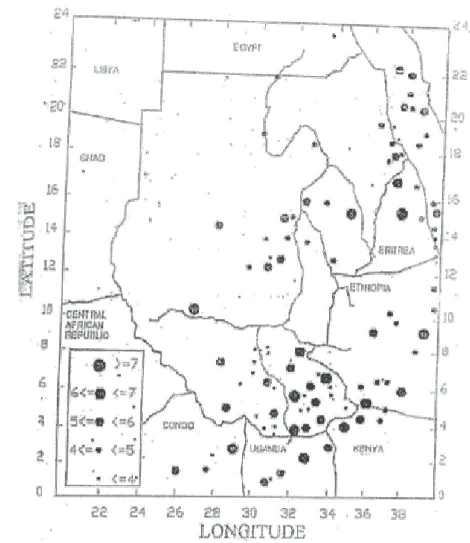


Fig.(2.2): Proposed Regional Seismicity of Sudan and its vicinity [8].

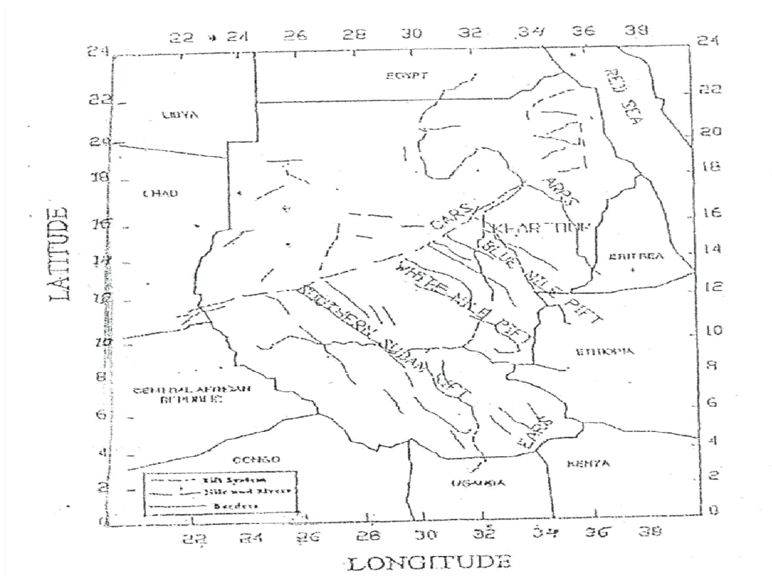


Fig.(2.3):Seismic rift system in Sudan[8].

2.7.3 Earthquake catalogue for Sudan

A previous catalogue covering the period 1632-1994 was used as input data to perform the Probabilistic Seismic Hazard Analysis (PSHA) for Sudan. The catalogue was compiled by Sudanese National Centre for Research – Seismological Research Unit (catalogue of the Seismicity of Sudan, 1996).But recent catalogue is been provided from study carried out by Ali (2009), which has additional contributions that updated the previous one . These contributions contain:

a. Some events were added, extending the catalogue up to 2007.
b. All magnitudes were homogenized to surface wave magnitude (MS) in order to make the catalogue unified and consistent by using equations adopted by the Eastern and Southern African Regional Seismology Working Group (ESARSGW, 1995), [8].

2.7.4 Seismic zoning of Sudan:

The zonation maps are made by studying the seismic hazard distribution for a large region. There are many types of zonations according to types of parameters which were chosen in the study of hazard distribution. These types of zonations are as follows:

a. **Seismicity zonation** , it depends on parameters of seismicity.
b. **Intensity zonation, it** depends on parameters of intensity .There is other intensity zonation depends on intensity parameters plus damage level.
c. **Ground motion zonations:**
    i. Ground motion zonation of indirect attenuation (acceleration) which can be obtained from intensity.
    ii. Ground motion zonation of direct attenuation (acceleration) from strong motion observed data (acceleration contours).
    iii. Ground motion zonation of effective peak acceleration and effective velocity with a set of probabilities of exceedance .This last one is

very good for satisfying different requirements of engineering problems [8].

The above are kinds of worldwide zonations, but in Sudan, Jamal et. al.{(1997)[8]}presented preliminary maps of seismic zoning of Sudan from observed and felt earthquakes in Sudan based on peak ground accelerations and intensities as shown in fig.(2.4)and (2.5). On the other hand , Ali (2009), has added new recent data (1632-2007) in his study then he proposed a seismic zoning map of Sudan as shown in fig.(2.6)[8].

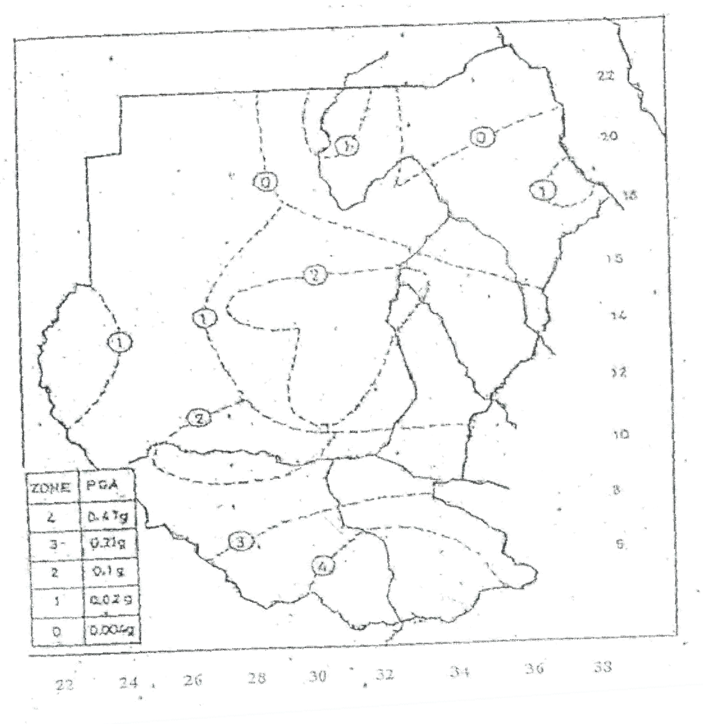


Fig.(2.4):Zoning Map of Sudan, Jamal. 1997 [8].

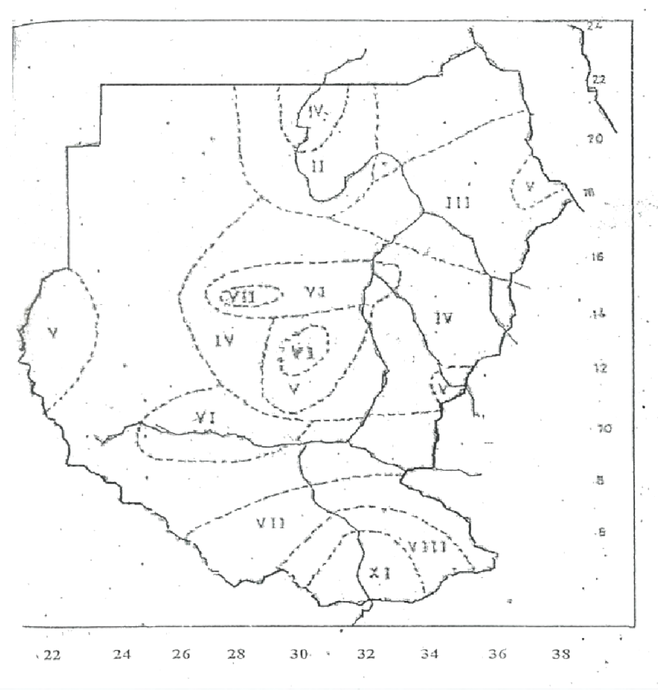


Fig.(2.5):Maximum Earthquake Intensity distribution of Sudan ,Jamal. 1997[8].

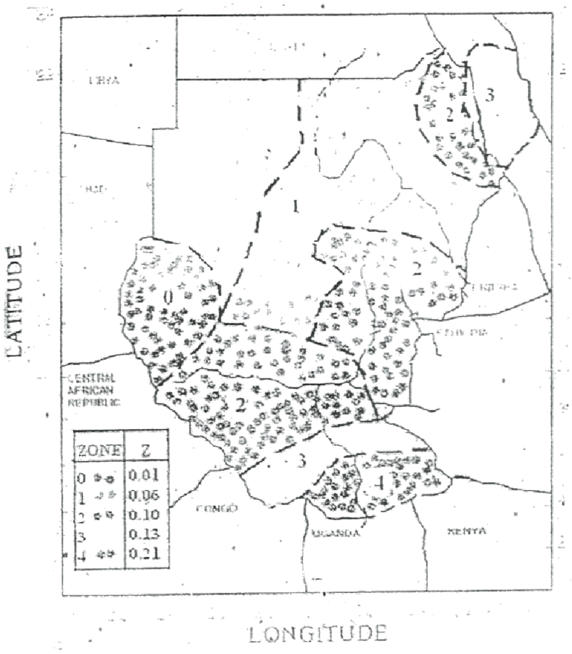


Fig.(2.6):Proposed Seismic Zoning Map of Sudan ,Ali 2009[8].

Chapter Three

Dynamic Response of Buildings under Seismic Forces

3.1 Introduction

In This chapter the types of seismic forces analysis and how we should use a suitable analysis, such as equivalent static lateral analysis or modal dynamic analysis, will be discussed. On the other hand, the evaluation of the more representative seismic codes is carried out. The evaluation involves the factors that are used in these seismic codes, the formulas, the equations and provisions of each code.

3-2 Analysis for seismic forces:

3-2-1 Dynamic Response to earthquake motions:

For structure subjected to ground accelerations $\hat{U}_g$ in some particular direction. The governing equation of motion becomes:

$$M\ddot{u} + C\acute{u} + Ku = - MI\acute{u}_g(t) \qquad (3.1)$$

Where:

M, C, and K are, Mass matrix, the viscous damping matrix, and the stiffness matrix respectively.

$u \equiv$ is the displacement vector

$I \equiv$ is a unit vector

The aim from eqn. (3.1) is to find the displacement response u(t), then the internal forces, and other response quantities of interests can subsequently be obtained [1].

The mass to be used in the analysis of a building be based on dead loads plus the percentage of the live load (not all floors are full at the sometime) [1].

The masses and the stiffness are calculated from dimensions and elastic properties of the structural and nonstructural elements, but the damping

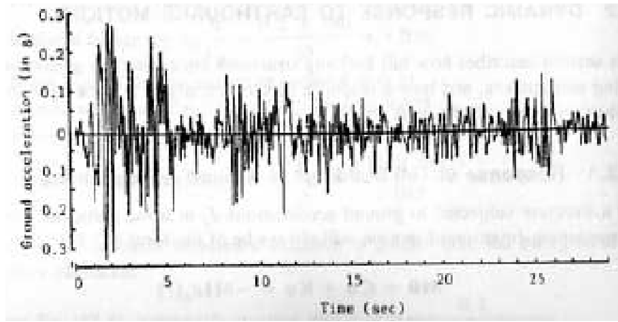


Fig.(3.1):Typical earthquake accelerogram .North-South component of general acceleration recorded at El Centru (approximately 4 miles from fault) [1].

matrix is difficult to be determining like this, so it determined from overall basis in terms of modal damping ratios [1].

3-2-2 Types of dynamic analysis

3-2-2-1 The Time history dynamic analysis:

The equation of time history analysis is:

$$F(t) = Ku(t) \tag{3.2}$$

u≡ the displacement

So to find the value of u, we solve Eq. (3.1) by integrating numerically for input time – history of accelerations $\bar{u}_g(t)$. To do the above, time – history of earthquake ground motions as measured by strong-motion accelerographs are to be prepared for a number of earthquakes like in Fig. (3.1). The accelerograph records the three orthogonal components of ground acceleration, each of which be integrated to yield the corresponding velocity and displacement- time histories. The earthquake accelerogram, or acceleration time- traces can be interpreted directly to obtain peak ground acceleration, duration of strong ground shaking, and frequency content. In more details, the accelerogram can be digitized, so that to define accurately the ground acceleration by numerical ordinates of the accelerogram at time intervals [1].

To obtain complete history response for a building, is not necessary, we need only the maximum response to earthquake to be established [1].

So for the above reason, the other two techniques (method of analysis) the equivalent lateral analysis and the model analysis method are advised to be used, because in these two methods the peak dynamic response can be obtained directly. The last tow procedures are have the same capabilities and subjected to similar limitations [1].

3.2.2.2 Equivalent static lateral force analysis:

In the equivalent static lateral force procedure, the magnitude of the forces according to (UBC) is based on:

i. Fundamental of structure (need preliminary design to obtained)
ii. Seismicity of the area.
iii. Operational importance factor.
iv. Preliminary Design for finding fundamental natural frequency of vibration of the building.
v. The structural system.
vi. Dead weight of the structure

The equivalent static lateral analysis procedure is used for:

i. Preliminary analysis for very high buildings or irregular buildings.
ii. Regular low rise buildings
iii. For very tall buildings or irregular buildings the modal analysis must be carried.
iv. If there is doubt about whether the lateral force procedure is necessary or not a quick calculation based on the equivalent lateral force be employed to know if even modal analysis is advisable or not [1].

a. The steps of equivalent lateral force procedure:

i. Calculate the lateral forces and story shears from the equivalent lateral force procedure.
ii. Approximately make dimensions for the structural members.
iii. Calculate the lateral deflections δ_i of the design structure due to the lateral forces from step i.

iv. Calculate new sets of lateral forces F_i and corresponding story shears from the formula (UBC) $F_i = V_0 \frac{\omega_i \delta_i}{\sum_{i=1}^{n} \omega_i \delta_i}$ (3.3)

Where:

V_0 ≡ the base shear

ω_i ≡ the building mass lumped at the i^{th} floor level

δ_i ≡ the lateral deflection at the i^{th}. floor level.

n ≡ number of storeys in the building.

v.If at any storey the recalculated storey shear from step iv differs from the original value in step i by more than 30%, a modal analysis is necessary, if the difference is less than 30%, the modal analysis is unnecessary, and the structure should be designed using the storey shears from step iv[1].

b.The natural frequencies (ω) and modes of vibration(ψ):

The dynamic response of a structure to any exciting (earthquake, wind, plast and mechanic) is depend on its vibration characteristics, defined by the natural frequencies (ω) and modes of vibration ψ_n , n = 1 to N for storeys of N degree of freedom). In the case of an undamped structure and in the absence of any exciting forces (undamping free vibration) the equation of motion becomes

$$M\ddot{u} + Ku = 0 \quad (3.4)$$

Assuming that the free vibration motion is simple harmonic, the displacement may be expressed in the form

$$U = \psi \sin(\omega_t + \theta) \quad (3.5)$$

Where:

θ ≡ the phase angle.

Substitution of Eq. (3.5) into Eq. (3.4) yields the governing eigenvalue equation (3.6) [1]: $K_\psi = \omega^2 M_\psi$ (3.6)

3.2.2.3 The modal method dynamic analysis and the response spectrum dynamic analysis:

The peak response of earthquake ground motion dynamic force can be get by combination of the modal method of analysis with earthquake design response spectra.

If the natural frequency of structure is calculated and the degree of damping is present, the other corresponding parameters such as the maximum displacement and maximum acceleration can be obtained directly from the response spectrum diagram in fig. (3.3) after using probabilistic analysis to linearize. If the building with unknown frequency oscillation ω, and a estimated damping ratio β, a response spectrum diagram such as shown in fig. (3.4) can be used combined with a modal method of analysis, to determine the peak response of the structure to the design earthquake [1].

To make a model the following steps will be carried out:

a. Earthquake design response spectrum derivation

Always, the irregularity in earthquake a ccelerograms indicates the irregularity of the ground accelerations as a function of time.

Although these provides basic information about the nature of the ground motions,the structural engineer requires a more meaningful characterization for design purposes. This is provided by response spectrum, which can be defined as a graphic representation of the maximum response of a damped single degree of freedom (SDOF) mass spring system with continuously varying natural periods to a given ground excitation.

The SDOF mass – spring system employed is represented in fig. (3.2).

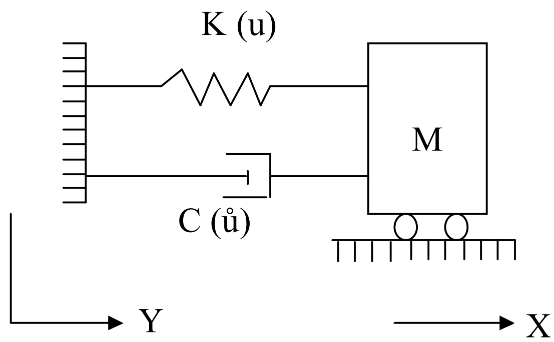


Fig.(3.2) single degree of freedom Damped spring mass oscillation system [1].

The mass M is connected to the support by a spring of stiffness K acting in parallel with a dashpot to simulate the viscous damping in the system, equal toCů· the support is assumed to displace by an amount Y(t) and the mass by x(t), the relative displacement U(t) being equal to (x – y).

The equation of motion is given by

$$M\ddot{u}(t) + C\acute{u}(t) + Ku = - M\ddot{y}(t) = - P(t) \qquad (3.7)$$

Where P is the effective support excitation loading.

The equation (3.7) may be expressed alternatively in the form

$$\ddot{u}(t) + 2\beta\omega\acute{u}(t) + \omega^2 u(t) = -\ddot{y}(t) \qquad (3.8)$$

Which: $\omega = \sqrt{\frac{K}{M}}$, $\omega^2 = \frac{K}{M}$ is the natural frequency of vibration

And $\beta = \frac{C}{2M\omega}$ is the fraction critical damping or damping ratio, where the damping coefficient and the critical damping (2Mω) is the minimum value of C that results in a non vibrating response. $\beta = \frac{C}{2M\omega} = \frac{C}{C_c}$ where

$C_c = 2M\omega$

The solution of Eq. (3.8) at time t is just (3.5)

$$u(t) = -\frac{1}{\omega\sqrt{1-\beta^2}} \int_0^{\tau} \left[\int (t)\, exp[-\omega\beta(t-\tau)]. \sin \omega\sqrt{1-\beta^2}(t-\tau) \right] dt \qquad (3.9)$$

In which τ is a dummy variable of integration.

Thus, before it is possible to determine the relative displacement-time history, it is necessary to know the acceleration- time history of the support, the natural frequency ω of the system and the fraction of critical damping β, which is a measure of the structures energy dissipative qualities.

For any input acceleration $\ddot{Y}$, the solution will yield the maximum absolute value of relative displacement u, termed the spectral displacement S_d, which will be a function of the natural frequency ω (or period) and damping factor .

$$S_v = \omega \, S_d$$

$$S_a = \omega^2 \, S_d \qquad (3.10)$$

Where:

$S_v \equiv$ maximum pseudo relative velocity

$S_a \equiv$ maximum Absolute pseudo acceleration

The pseudo acceleration is identical to the maximum acceleration when there is no damping, which for normal levels of structural damping, is particularly the same as the maximum acceleration [1].

From, Eq. (3.9), (3.10) a complete response spectrum be presented on a tripartite logarithmic plot of the form as show in Fig. (3.3) [1].

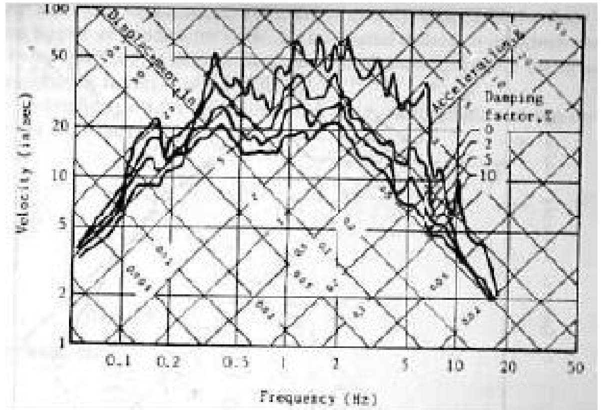


Fig.(3.3) Response spectra. El Centro earthquake .N-S direction (after Newmark and Hall)[1].

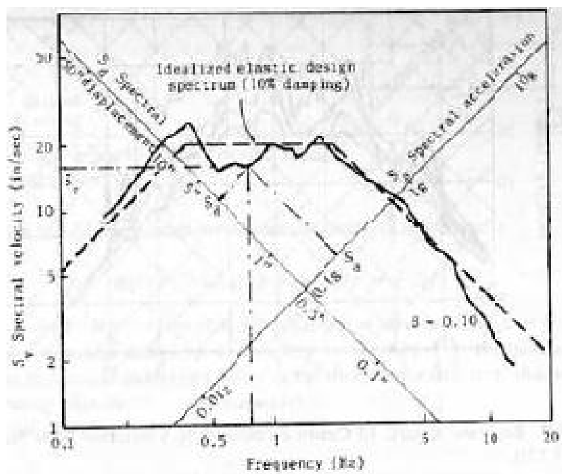


Fig.(3.4):Idealized design response spectrum[1].

b. The modal analysis:

In the modal analysis a lumped mass model of the building with horizontal degrees of freedom at each floor is analyzed to determine the modal shapes and modal frequencies of vibration. The results are then used in conjunction with an earthquake design spectrum, and estimates of the modal damping, to determine the probable maximum response of the structure from the combined effect of its various modes of oscillation .For the building with torque, which causes tensional vibration and of coupling of lateral and tensional mode, the modal method will be used by adding to the structural model a third, rotational degree of freedom at each floor [1].

The modal method is applied in the strictest sense, only to linear elastic systems. Consequently, the results for a building structure's response are at best significant inelastic deformations in only moderate earthquakes. More accurate values of response may be obtained for buildings by the model analysis method, using modified design response spectra for inelastic system [1].

In general, the set of governing dynamic equations of motion [Eq. (3.1)] must be solved simultaneously by a viable computational procedures include Eq. [(3.5), (3.6)] to determine all displacements u that define the motions of the structure.

This approach can be avoided by using the computationally more efficient modal method of analysis. The method which is based on linear elastic structural behavior employs the superposition of a limited number of modal peak responses, as determined from prescribed response spectrum, and with appropriate modal combination rules it will yield results that compare closely with those from a time – history analysis [1].

This method of analysis is based on the fact that for certain forms of damping that are reasonable approximations for many buildings, the equations of motion can be uncoupled so that the response in each natural mode of vibration can be calculated independently of the others. Each mode will respond with its own particular displacement profile, the mode of vibration ψ_n, its own frequency, the natural frequency of vibration ω_n, and with its own modal damping, the damping ratio β_n. [1]

In the structural idealization, the mass is usually lumped at the floor levels. Only one degree of freedom per floor, the horizontal deflection for which the structure is being analyzed, is used, and so that matrices involved are of the same order as the number of storeys N in the building [1].

The uncoupling of the N equations yields the typical equation of motion for the n^{th} natural mode as in Eq. (3.9)

$$\ddot{y}_n + 2\beta_n\omega_n\dot{y}_n + \omega_n^2 Y_n = -\frac{Ln}{M_n}\ddot{u}_g(t) \qquad (3.11)$$

Where: $Ln = \sum_{i=1}^{n} m_j \Psi_j\, n$

and the modal mass $M_n = \sum_{i=1}^{N} m_j \Psi^2 j\mathrm{n}$

The modal participation factor $\gamma_n = \frac{L_n}{M_n}$

Where: m_j is the mass at the j^{th} floor level.

Equation (3.11) is of the same form as that for the single degree of freedom dynamic system [Eq.(3.8)] with natural frequency ω_n and damping ratio β_n, excited by aground excitation $\left(\frac{L_n}{M_n}\right) \ddot{u}_g(t)$.

In Eq.(3.11), the displacement function Y_n is the normal or generalized coordinate, or modal amplitude, for the n^{th} natural mode, used to simplify the equations of motion [Eq.(3.1)][(3.5)]. The geometric coordinate U_n in Eq.(3.1) is equal to the product of the generalized coordinate Y_n and the mode- shape vector ψ_n.

Thus:

$$Y_n(t) = -\frac{L_n}{M_n}\frac{1}{\omega_n\sqrt{1-\beta_n^2}}\int_0^{\tau}\left[\ddot{u}_g(t)exp[-\beta_n\omega_n(t-\tau]sin\omega_n\sqrt{1-\beta_n^2}(t-\tau)\right]dt \tag{3.12}$$

The contribution of the n^{th} mode to the modal displacement $u_j(t)$ at the j^{th} floor is then equal to the product of the amplitude generalized coordinate and the mode shape [1].

$$U_{jn}(t) = Y_n(t)\Psi_{jn} \tag{3.13}$$

To determine the dynamic storey shears and moments, it is convenient to introduce the concept of equivalent lateral forces, defined as the static external process p that would produce the same structural displacements U. Hence, at any time (t), the equivalent forces corresponding to the modal displacements $U_n(t)$ will be, from Eq. ((3.2) and (3.6)

$$P_n(t) = KU_n(t) = K\Psi_n Y_n(t) = \omega_n^2 M\Psi_n Y_n(t) \tag{3.14}$$

The equivalent lateral forces at the j^{th} floors level is then

$$P_{jn}(t) = \omega_{jn}^2 M_j \Psi_{jn} Y_n(t) \tag{3.15}$$

In which Y_n is given by Eq. (3.12) the contributions from each node then summed to give the total equivalent force P at each floor level for example, at level j [1].

$$P_h = \sum_{n=1}^{N} P_{jn}(t) \qquad (3.16)$$

The internal dynamic shears and moments at any level can then be obtained by summing all the storey forces and the moments of these forces above the level concerned [1].

In a similar manner, the displacement at any level may be obtained by combining the response from each mode at that position. The drift U_n at the top of the building is then

$$U_N = \sum_{n=1}^{N} u_{Nn} \qquad (3.17)$$

And the interstorey drift is given by the difference between the total displacement of the floors above and below the level concerned [1].

The great attraction of this method is that an independent analysis can be made of a single – degree – of – freedom system for each natural mode of vibration. The response generally needs to be determined for only the first few modes since the total response to earthquakes is primarily due to the lowest modes of vibration, sufficiently accurate design values of forces and deformations in tall buildings should be achieved by combining no more than about six modes in each component direction. Three would probably suffice for medium- rise buildings. Checks may be carried out since the relative influence of each successive mode on the important design parameters may be examined during the calculations [1].

The earthquake response is obtained by combining the contributions of all the modes of vibration involved, and this can be used to give a complete time-history of the structural actions. However, only the evaluation of the peak response is of importance in design, and this may be derived directly from the design response spectrum [1].

c. Design response spectrum analysis

Since in the modal analysis the response of the structure in each mode of vibration is deriven from a single – degree – of – freedom system, the maximum response in that mode can be obtained directly from the earthquake design response spectrum [1].

The maximum response in the n^{th} mode can be expressed in terms of the ordinates of the displacement S_{dn}, pseudo velocity S_{vn}, and acceleration S_{an}, which correspond to the frequency ω_n and damping ratio β_n. The three quantities are related by, Eq. (3.10) [1].

$$S_{an} = \omega_n S_{vn} = \omega_n^2 S_{dn} \tag{3.18}$$

Expressed in terms of the modal participation factor γ_n, the maximum values of the modal response quantities then become, from Eqs. (3.11) to (3.17):

Maximum modal displacement

$$\bar{Y}_n = \gamma_n S_{dn} \tag{3.19}$$

Maximum displacement at j^{th} floor

$$\bar{u}_{jn} = \gamma_n S_{dn} \psi_{jn} \tag{3.20}$$

Maximum inter storey drift in j^{th} storey

$$\bar{\Delta}_{jn} = \gamma_n S_{dn}(\psi_{jn} - \psi_{j-1}) \tag{3.21}$$

Maximum value of equivalent lateral force at j^{th} floor $\bar{P}_{jn}$

$$\bar{P}_{jn} = \gamma_n S_{an} m_j \psi_{jn} \tag{3.22}$$

In Eq.(3.19) to (3.22), a bar above a particular variable is used to denote the maximum value of the quantity concerned. The maximum values of the internal forces in the building, particularly the storey shear and moments, are then obtained by a static analysis of the structure taking due account of the senses of these equivalent forces. Although they act in the same direction for the lowest natural modes, they may act in opposite directions in the higher modes [1].

The maximum base shear V_{on} and base moment $\bar{M}_{on}$ will then be, using Eq.(3.22)

$$\overline{V}_{on} = \sum_{j=1}^{N} \overline{P}_{jn} = \gamma_n S_{an} \sum_{j=1}^{N} m_j \Psi_{jn} = \gamma_n L_n S_{an} \tag{3.23}$$

$$\overline{M}_{on} = \sum_{j=1}^{N} h_j \overline{P}_{jn} = \gamma_n S_{an} \sum_{j=1}^{N} h_j m_j \Psi_{jn} \tag{3.24}$$

Where h_j is the distance from the j^{th} floor to the base [1].

The maximum modal response can thus be expressed in terms of the displacements or accelerations, evaluated for particular frequency and damping ratio for the mode, from, the design response spectrum [1].

Equations (3.18), (3.20) and (3.22) shows that if the displacement and forces are both expressed in terms of the spectral displacement S_{dn}, the forces are multiplied by the square of the natural frequency(which presented in Eq.3.18). Consequently, the higher modes will be of greater significance in defining the forces in the structure than they are in the deflections, and it will be necessary to include more modal components to evaluate the forces to the same degree of freedom of accuracy as the deflections [1].

The total response R of the building to earthquake motions is the sum of the individual responses r_n of the natural modes. However, the maximum total response R^{-} is not generally equal to the absolute sum of the maximum modal responses r^{-}_{n}, since they will not normally occur simultaneously. Such a sum would, however, give an upper bound to the maximum likely total response [1].

A more realistic design estimate of the maximum response is to combine the modal maximum according to the square root of the sum of the squares (SRSS) method

$$R^{-} = \sqrt{(\sum r_n^{-2})} \tag{3.25}$$

The maximum values of displacements, interstorey drifts, storey shears, and moments may all be evaluated using Eq. (3.25) [1].

This formula will generally give realistic estimates of peak response for structures in which the natural frequencies of vibration are well separated, a property that is usually valid for idealized building structures in which lateral displacements in one plane are considered. If this is not the case, and some natural frequencies are so close that the motions may be coupled together, a more realistic combination, such as the complete quadratic combination method, should be undertaken [1]. The maximum estimated response due to earthquake motion is derived from the following procedure

a. Establish the response spectrum from the ground motions.
b. Calculate the mass and stiffness matrices M and K, and estimate the modal damping ratios β_n.
c. Determine the first few natural frequencies ω_n and the modes of vibration ψ_n.
d. Calculate the maximum response of the structure in each individual mode as follows:
   i. For a natural frequency ω_n and damping ratio β_n, determine the ordinates S_{dn}, S_{an} of the displacement and acceleration response spectra.
   ii. Calculate the floor displacements from Eq. (3.20) and the storey drifts from Eq. (3.21).
   iii. Calculate the internal shears and moments from statics for a cantilevers structure subjected to these lateral forces.
e. Calculate the peak value of the major design actions (displacement, drift, storey shear, and moment) by combining the maximum modal values according to Eq. (3.25).

It is necessary to consider only the modes that contribute most to the response of the structure. Since most of the energy of vibration is contained in the lower modes of vibration, it is generally sufficient to consider no more than six modes in each horizontal direction. A convenient rule is to include a sufficient number of modes r so that an effective modal mass e of at least 90% of the total mass of the building is represented by the modes chosen [1], that is:

$$e = \frac{\sum_{n=1}^{r} \gamma_n {}^{L_n}/_{100}}{\sum_{j=1}^{N} M_j} \nleq 90 \qquad (3.26)$$

f. The lateral –torsional Coupling and Vertical components of Ground Motion:

If the center of mass of building is coincident with its resistance, the spectrum analysis considers only the response of the structure to only two independent orthogonal motions (translation motion).However, if either the eccentricity between the centers of mass and resistance are large, or the natural frequencies of lateral and torsional vibration are close together, in this situation, coupling between translational motions and rotational motions about a vertical axis will occur. For such structures, independent analysis of two orthogonal will not be sufficient, so, it will be analyzed by three degrees of freedom-two translational and one rotation should be included at each floor level .The modal method of analysis then will be applied to the system with three degrees of freedom at each floor level. From above mentioned, the modes which are predominantly torsional can be excited by translational components of ground motion, and that a particular mode can be excited by both horizontal components. The resulting maximum should then be calculated combinedly [1].

In some cases, although horizontal ground motions are normally dominant, it may be not possible to neglect the vertical accelerations, which, may be large compared to the gravitational acceleration in regions of high seismicity. The dynamic vertical response should be estimated by the use of vertical response spectrum to obtain the equivalent vertical forces for which the building must be designed [1].

Although a nonlinear analysis can always be achieved from a step – by-step piecewise -linear elastic analysis, with the calculation divided up in to a series of time steps, and with the current stiffness of the structure appropriate to the stress level concerned being used during each step, such procedure would be too complicated [1].

Although the modal method of analysis, is applied only to elastic systems, a simple procedures that produces results of acceptable accuracy uses a linear analysis in conjunction with an inelastic response spectrum. The approach is reasonably accurate for building structures in which the deflection is limited to ductility factors about 5or 6 [1].

The inelastic design spectrum can be derived from the elastic spectrum and allowable ductility factor μ, defined as the ratio of the ultimate or maximum displacement u_m to the effective limit deflection $\mathbf{u}_y$ that is:

$$\mu = um/uy \qquad (3.27)$$

The elastic design spectrum of Fig.(3.5) be divided into three regions –the displacement region (ω between 0.1 and 0.4 Hz), velocity region (ω between 0.4 and 4 Hz), and the acceleration region(ω between 4 and 7.7 Hz).If the elastic plastic load – displacement relationship is of bilinear form , as shown in Fig.(3.6), the inelastic spectrum can be obtained by multiplying the values in the displacement and velocity regions by a factor $1/\mu$, and the values in the acceleration region by a factor $\frac{1}{(2\mu-1)1/2}$. The resultant inelastic design spectrum is shown by the broken line in Fig. (3.5) for a ductility factor of 2.

With this approach, it is possible to define design spectra to take account of inelastic structural behavior [1].

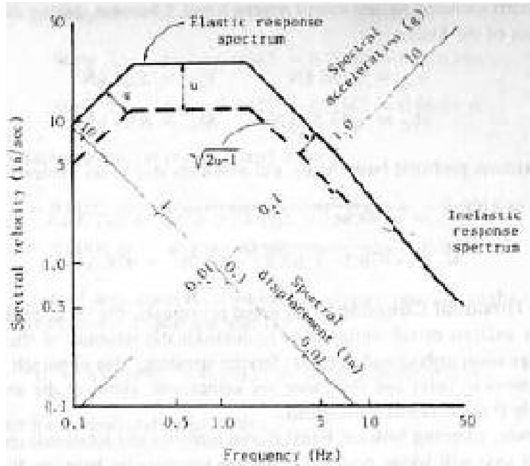


Fig. (3.5): Elastic and inelastic response spectra [1].

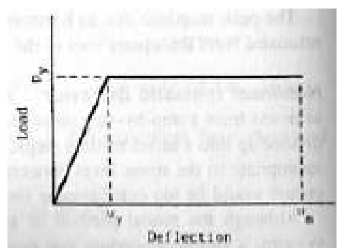


Fig. (3.6): Idealized load –displacement curve [1].

3.2.2.4 Using computer program analysis:

Nowadays, computer programs provide a good opportunity for analysis and design in field of engineering works. From these several programs ,in this research , analysis has been carried out by using ETABS, version V9.7.4 program, which is capable of giving reasonable results for different types of buildings with ability of utilizing different codes of practice among which are ,UBC 1997 and IBC 2006.

The following are some out lines which could be used in the program:

1. Adding all dead loads, live loads and other cyclic loads, like seismic, wind, load, ext., with all details of parameters.
2. Displaying the most quantities of analysis and design.
3. Enables for doing model of a building with walls having openings.
4. Enables for conducting nonlinear analysis.
5. Giving summary of input and output data, for analysis and design.

3.3 Berif description of the two codes[UBC and IBC]

3.3 .1 Seismic Design UBC1997 Code Provisions

3.3.1.1 Background of UBC1997 seismic Code [9[.

The Uniform Building Code is dedicated to the development of better building construction and greater safety to the public by uniformity in building laws. The code is founded on broad principles that make possible the use of new materials and new construction systems.

The Uniform Building Code was first enacted by the International Conference of Building Officials at the Sixth Annual Business Meeting held in Phoenix, Arizona, October 1927 and the last edition was in 1997 which has been substituted by IBC [10]. Revised editions of this code have been published since that time at approximate three-year intervals. New editions incorporate changes approved since the last edition.

The Uniform Building Code is designed to be compatible with related publications to provide a complete set of documents for regulatory use [9].

Most seismic building codes require that structures be designed to resist specific static lateral forces related to the properties of the structure and the seismicity of the region. In the event of server earthquakes, buildings are likely to behave in elastic manner and the ultimate of buildings, while tolerating reasonable amount of structural damage. Almost all developed codes follow one of the following pioneering codes , which are based on similar concept. The pioneering codes are the Uniform Building code (UBC-94), Structural Engineering Association of California (SEAOC-99), and later the International Building Code (IBC-2000)[8].

Below are the seismic provisions , which provides several factors according to UBC code to be used in the analysis and design of buildings subjected to earthquake forces.

3.3.1.2 Calculation of Base Shear (V_b)

The base shear is be determined as a fraction of the effective weight of the structure .This value depends on several factors and coefficients, such as seismic activity of the region (seismic zone factor),important factor of the structure use, structural seismic factor, soil factor in addition to construction material and quality factors. Using the static lateral force procedure and according to UBC-97 the base shear in a building can be calculated using the following procedure:

$$V_b = \frac{Z^* I \ ^* C}{R\omega} W \qquad (3.28)$$

Where:

Z ≡ seismic zone factor;

I ≡ importance factor;

$R\omega$ ≡ numerical seismic coefficient (response modification factor);

W ≡ weight of the building computed from total dead and a percentage of the live load;

C ≡numerical coefficient determined from following formula:

$$C = \frac{1.25\ S}{T^{213}} \qquad (3.29)$$

Where:

S ≡ site coefficient for soil characteristics

T ≡ fundamental period of vibration of the structure

The following parameters are be determined for base shear calculation:

a. Seismic zone factor (z)

The seismic zone factor (z) represents the maximum effective peak ground acceleration for each zone. It is expressed as a fraction of gravitational acceleration, g. According to AbuBakr et.al. [11] Sudan has been delineated into 5 macro-seismic zones generating from seismic zone map based on peak ground acceleration. Table (3.1) shows numbers and zone factors.

Table (3.1): Seismic Zone Factor (Z)(UBC)

Zone Number	Z
1	0.075
2A	0.15
2B	0.2
3	0.3
4	0.4

b. The site soil factor (S)

These are factors that adjust lateral load coefficient to include soil-structure interaction. Soft soils may result in amplification of the ground motion. It usually shakes more severely than rock sites. However, building codes divide potential sites into three categories: those with low, medium and high soil dynamic amplification factors. These factors are judgments based upon a review of existing soils data from borings as in table (3.2).

Table (3.2): Site Soil Factor (S) (UBC)

Soil Type	Description	Factor (S)
1	Rock or stiff soil conditions or stable deposits and, gravel or stiff clays of depth less than 20 meters.	1.0
2	Deep cohesionless of stiff clays exceeding 20 meters.	1.3
3	Soft to medium stiff clays and sands.	1.5

d. The importance factor (1):

These provisions recognize the social importance of certain building in relation to their intended use, and increase the required minimum design base shear by means of importance factor given in table (A1.2) in appendix A1. The highest importance factor, 1.25, is assigned to so-called 'postdisaster' buildings such as hospitals, fire and police stations, and other structures related to public safety that must remain functional during and immediately after earthquakes.

d. Structural system factor (Rω)

A factor that measures the ability of systems to sustain cyclic inelastic deformations and absorb energy without collapse. The smaller the value of this factor, the greater the capacity of the structure to accommodate deformations through yielding and inelastic energy absorption. An extremely brittle structure requires a high value, while a very resilient, ductile system may have a low value. The recommended values of this factor are shown in table (A1.3) in appendixA1.

Rω Ranges: 8.5 > Rω >2.2, Design Structural System factors for Seismic force Resisting systems.

Response factor R= Rω/1.4

e.Seismic coefficients C_a and C_v:

C_a and C_v are parameters depend on acceleration and velocity due to earthquake force respectively.

Table (3.3): Seismic coefficient C_a (UBC)

Soil profile type	Seismic Zone factor Z				
	Z=0.075	Z=0.15	Z=0.2	Z=0.3	Z= 0.4
S_A	0.06	0.12	0.16	0.24	0.32N_a
S_B	0.08	0.15	0.2	0.3	0.4N_a
S_C	0.09	0.18	0.24	0.33	04N_a
S_D	0.12	0.22	0.28	0.36	044N_a
S_E	0.19	0.3	0.34	0.36	0.36N_a
S_F	#				

#: Site specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil profile Type S_F

Minimum Design Base Shear:

For all seismic zones:

$$V_{min.} = 0.11 C_a I W \tag{3.30}$$

Minimum Design Base Shear for zone 4:

$$V_{min.} = 0.8 N_v I W / R \tag{3.31}$$

Table (3.4): Seismic coefficient C_v (UBC)

Soil profile type	Seismic Zone factor Z				
	Z=0.075	Z=0.15	Z=0.2	Z=0.3	Z= 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.2	0.3	$0.4\ N_v$
S_C	0.13	0.25	0.32	0.45	$0.56\ N_v$
S_D	0.18	0.32	0.4	0.54	$0.64\ N_v$
S_E	0.26	0.5	0.64	0.84	$0.96\ N_v$
S_F	#				

#: Site specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil profile Type S_F. Nv≡ the near source factor for velocity.

f. Fundamental period (T):

The fundamental period T of the structure may be calculated using different empirical formulas. For buildings with frame designed to resist lateral forces without shear walls, the fundamental period T in seconds may be determined by:

$$T = 0.1\ N \tag{3.32}$$

Where N is the number of storeys.

or $$T = C_t\,(\mathbf{h}_n)^{3/4} \tag{3.33}$$

$\mathbf{h}_n$ =is the total building height

C_t =0.035 for steel moment –resisting frames

C_t =0.03 for reinforced concrete moment –resisting frames and eccentrically braced frames.

C_t =0.02 for all other buildings

For buildings other than moment resisting space frames, T can be calculated as:

$$T = \frac{0.09h_n}{\sqrt{D}} \qquad (3.35)$$

Where

h_n ≡ height of the structure above the base to the uppermost level n, in meters.

D ≡building length in meters, in the direction of the seismic action.

Another method of determining the fundamental period is by using Rayleigh formula given by the following expression:

$$T = 2\pi\sqrt{\left(\sum_{i=1,}^{N} \omega_i \delta_i^2\right) / \left(g \sum_{i=1}^{N} f_i \delta_i\right)} \qquad (3.36)$$

Where

f_i ≡ distributed lateral force;

δ_i ≡ calculated deflection using applied lateral force.

The T calculated from Eq.(3.35) should not exceed30% for zone 4 and 40%for zone1, 2, 3 than T that been calculated in Eq. (3.33).

g. Load combinations

In designing for seismic forces, the following two combinations can be considered:

$$W = D + L.p + E_m \qquad (3.36)$$

$$W = 0.85D + E_m \qquad (3.37)$$

Where:

D ≡ dead load.

L ≡ live load.

P ≡ incidence factor for live load.

$E_m \equiv$ earthquake load.

h. Seismic weight (W):

The seismic weight W of the structure includes the load of the building, permanent equipment, and a fraction of live load (incidence factor) as indicated in table (3.5).

Table (3 .5): Incidence factor for live load (p) (UBC)

Type of Structure	**Incidence factor (p)**
1. Residential buildings, hotels, offices, hospitals, public buildings, etc.	0.25
2. Storage areas and warehouses	0.50
3. Tanks, reservoirs, silos and the like	1.00

3.3.1.3 Distribution of Lateral Forces:

The base shear (V_b) is distributed along the height of the building using the static lateral procedure, according to the following formula:

$$F_x = \frac{(V_b - F_t) W_x h_x}{\sum_{i=1}^{n} W_i h_i} \tag{3.38}$$

With an additional force F_i at the top of the building, given by

$$F_t = 0 \quad for\ T \leq 0.7\ sec; or \tag{3.39}$$

$$F_t = 0.07TV\ (\leq 0.25V_b) \quad for\ T\ > 0.7\ sec.$$

Where

n ≡ total number of storeys above the base of the building.

F_t ≡portion of the base force V_b at the top of the structure in addition to F_n

h_x ,h_i ≡ height of the level x or i above the base.

W_x, Wi ≡ seismic weight of the x^{th} or i^{th} level.

3.3.1.4 Storey Shear Force:

The storey shear V_x at any storey i of the building is given by the sum of the forces above that storey; that is:

$$V_x = F_t + \sum_{i=x}^{N} F_i \qquad (3.40)$$

Where $F_i \equiv$ lateral force applied at level i.

For specific elements of structure, the minimum design strength $=\Omega F_x$

$\Omega \equiv$ seismic force over strength factor

$F_x \equiv$ design seismic force

3.3.1.5 Storey Overturning Moments:

These provisions require that overturning moments shall be determined at each level of the structure. The overturning moment is determined using the seismic design forcers F_x and F_i[Eq. (3.38) and (3.39)] which act on levels above the level under consideration. Hence, the overturning moment M_x at level x of the building is given by

$$M_x = F_t(h_N - h_x) + \sum_{i=x+1}^{N} F_i(h_i - h_x) \qquad (3.41)$$

Where x = 0, 1, 2, …, N – 1.

3.3.1.6 Lateral displacements (δ_i)

The lateral elastic displacement at the various levels of the building may be determined by elastic analysis of the structure, which is acted upon by the seismic forces F_x and F_i given by [Eq. (3.38) and (3.39)]. Alternatively, the elastic displacement δ_i may be determined as follows:

$$\delta_i = \frac{8T^2F_x}{4\pi^2W_x} \qquad (3.42)$$

In which the additional forces F_i is included in F_x for the top level of the building.

3.3.1.7 Storey Drift Limitation (Δst)

Storey drift is the displacement of one level relative to the level above or below due to the design lateral forces.

$$\Delta_{st} = x_i - x_{i-1} \qquad (3.43)$$

Δ_{max} .= 0.7R Δ_{st}

$\Delta_a = 0.02hsx$ for $T \geq 0.7$ sec.

$\Delta_a = 0.025hsx$ for $T < 0.7$sec.

$\Delta_a \equiv$ allowable storey drift

$\Delta_{st} \equiv$ total storey drift

$h_{sx} \equiv$ storey height below level x.

Where x_i and x_{i-1} are respectively, the lateral displacements for level i and for level i – 1.

$\Delta_{max.} \equiv$ maximum inelastic Response displacement (the analysis used to determine inelastic Response maximum displacement Δ_{max} , shall include p-delta effect.)

The limitations above for drift can be exceeded if the greater drift is within the tolerance of the life safety of the structural elements and nonstructural elements.UBC relates drift limitations to the period T and Rω in Table A1.3.

.

Table (3.6): Allowable Storey Drift UBC code

Structure type(according to Ta in seconds)	Drift limitation
Ta<0.7 sec.	0.025hsx
Ta≥0.07 sec.	0.02hsx

3.3.1.8 Building separations:

All structures should be separated from adjoining structures. Separations should allow for the displacement Δ_{max} . Adjacent buildings on the same property should be separated by at least where

$$\Delta_{max} = [(\Delta_{m1})^2 + (\Delta_{m2})^2]^{1/2} \quad (3.44)$$

Where Δ_{m1} and Δ_{m2} are the displacements of the adjacent buildings.

Set back the buildings from property lines by at least Δ_{max} of that structure

3.3.1.9 Dynamic Analysis for UBC

The analysis should be based on appropriate ground motion representation and should be performed using accepted principles of dynamics. The requirements of dynamic analysis are as follows:

a. The ground motion

The ground motion representation, should at minimum, be one having a 10-percent probability of being exceeded in 50 years and should not be reduced by the factor R and may be one of the following:

i. An elastic design response spectrum constructed in accordance with fig. (3.7), using the values of C_a and C_v consistent with the specific site. The design acceleration ordinates should be multiplied by the acceleration of gravity (9.815 m/s^2).

ii. A site specific elastic design response spectrum based on the geologic, tectonic, and seismologic and soil characteristics associated with the specific site. The spectrum should be developed for damping ratio of 0.05, unless different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site.

iii. Ground motion time histories developed for the specific site should be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, should approximate the site design spectrum conforming to in ii above.

iv. For structures regular or irregular located on Soil profile Type S_f, that has a period greater than 0.7 second. The analysis should include the effective of the soils at site and the following requirements should be applied:

-Firstly, The ground motion representation should be developed in with accordance to items ii and iii above.

-Secondly, Possible amplification of building response due to the effects of soil structure interaction and lengthening of building period caused by inelastic behavior should be considered.

v. The vertical component of ground motion could be defined by scaling corresponding horizontal accelerations by a factor 2/3.Alternative factors may be used when substantiated by site –specific data .Where the near Source factor N_a is greater than 1.0, site –specific vertical response spectra should be used in lieu of the factor of 2/3.

b. Dynamic analysis procedures for UBC

The dynamic analysis procedure includes either Response spectrum analysis or Time –history analysis

i. Response spectrum dynamic analysis:

Is an elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response .Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

ii**. Time history dynamic analysis for UBC:**

An analysis of the dynamic response of a structure at each increment of time when the base is subjected to a specific ground motion time history. Formed with pairs of appropriate horizontal ground motion time-history components that should be selected and scaled from not less than three recorded events .For each pair of horizontal ground motion components ,the square root of the sum of squares (SRSS) of the 5 percent-damped site –specific spectrum of the scaled horizontal components should be constructed .The motion should be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent –damped spectrum of the design –basis earthquake for periods from 0.2T second to 1.5T seconds. The parameter of interest should be calculated for each time–

history analysis. For three pairs, the maximum response of the parameters of interest should be used for design, but for seven or more pairs, the average of the parameter should be used for design.

The mathematical model of physical structure (three-dimensional model having rigid or semi rigid diaphragm) should present the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response.

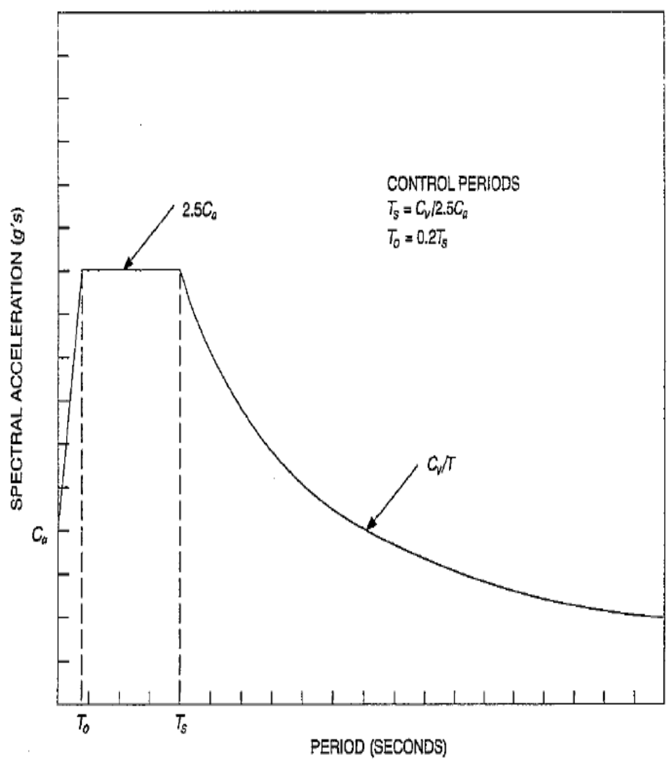


Fig.(3.7) Design response spectra in UBC[9].

3.3.1.10 How to choose the analysis type for UBC:

a. Selection of Simplified design base shear.

$V_b=3C_aW/R$ (3.45)

Vertical distribution force $F_x=3C_aw_i/R$

The above simplified method used for:

i. Buildings of 3storeys (excluding the basement) for one family dwelling

ii. Any other buildings of 2 storeys (excluding basements).

b-The selection and use of equivalent static lateral force procedure:

i. All structures, regular or irregular, in seismic zone 1 and in occupancy categories 4 and 5 in seismic zone 2.

ii. Regular structures under (73.152m) in height with lateral resistance system provided by system listed in table(A1.3) in appendix A1, except in section 3.3.9.1, item a (iv),above applies.

iii. Irregular structures not more than 5 storeys (19.812m) in height

iv. Structure having flexible upper portion supported on a rigid lower portion where both portions of the structure considered separately can be classified as being regular, the average storey stiffness of the lower portion is at least 10 times the average storey stiffness of the upper portion and the period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

c-The dynamic analysis procedure is used in:

i. Structures more than (73.152m) in height, except as permitted by section 3.3.1.9 item b(ii) above.

ii. Structures having a stiffens ,weight or geometric vertical irregularity,or structures having irregular features .

iii. Vertical irregular types or plan irregular types*, except that permitted when has a vertical combination system of resistance.*

iv. Structures over 5 storeys or (19.812m) in height in seismic zones 3 and 4 not having the same structural system throughout their height except that permitted by vertical combination of structural systems(the lesser R-value is used for entire structure).

v. Structures ,regular or irregular ,located on soil profile Type S_f ,that have a period greater than 0.7 second .The analysis should include the effects of the soils at the site when {the ground motion representation should be developed in accordance with items(i),(ii)above}and{the consideration taken for possibility of amplification of building response due to the effects of soil-structure interaction and lengthening of building period caused by inelastic behavior}.

vi. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two –thirds. Alternative factors may be used when substantiated by site specific data. Where the Near Source factor, Na, is greater than 1.0, site – specific vertical response spectra should be used in lieu of the factor of two thirds.

3.3.1.11 Diaphragm forces F_{px}:

$$F_{px}=[F_t+\sum F_i]w_{px}/\sum w_i \quad (3.46)$$

Where:

$0.5C_aI\ w_{px} \leq F_{px} \leq 1.0C_aI\ w_{px}$

w_{px} ≡ the weight of the diaphragm and element tributary there at level x including applicable portions of the loads.

3.3 .2 Seismic IBC 2006 provisions [10]:

3.3.2.1 Back ground:

International Building Code (IBC) is code to be used in U.S.A. and internationally .IBC was prepared combining mainly previous three model codes that had been used in U.S.A, i.e. Uniform Building Code (UBC) which had been used in western part, National Building Code (NBC) in eastern and northern parts and Standard Building Code (SBC) in southern part. The first edition of IBC was published in 2000, and it has been revised ever three years. First editions included detailed requirements, but the 2003 and 2006 editions were revised to include only principal requirements, and refer to ASCE[12] "Seismic Design Requirements for Building Structures" for detailed requirements for seismic design[10].

IBC covers not only structural requirements but also interior finishing, environments, etc. concerning buildings [10].

As to the seismic codes of U.S.A., the first code was prepared by the committee in the Structural Engineers Association of California (SEAOC).

The SEAOC code then transferred to UBC and then to IBC. The provisions by Applied Technology Council (ATC) which is called as ATC 3 and the provisions by the National Earthquake Hazard Reduction Program (NEHRP) also influenced the IBC. The flowchart of Fig.(3.8) schematically shows the relationships of these seismic codes.

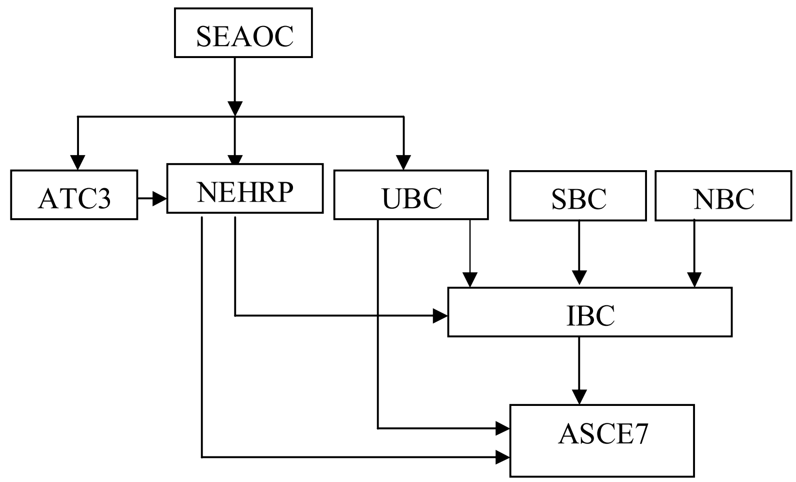


Fig. (3.8) Flowchart of seismic codes in U.S.A.

SEAOC: Structural Engineers Association of California
ATC: Applied Technology Council
NBC: National Building Code
SBC: Standard Building Code
UBC: Uniform Building Code
NEHRP: National Earthquake Hazard Reduction Program
IBC: International Building Code
ASCE: American Society of Civil Engineers

The earthquake regulations of Sections 1613 through1623 of the 2000 IBC (ICC 2000), based on the 1997 NEHRP Provisions (BSSC 1997), are substantially different from the corresponding provisions of the 1997 UBC (ICBO 1997). The differences of relevance to this are discussed below.

3.3.2.2 Calculation of base shear by Equivalent Lateral Force Procedure.

This analysis procedure can be used for all structures assigned to SDC B and C as well as for some types of structure assigned to SDC D, E, and F.

In a given direction, the design base shear V_b is determined from Eq. (3.47):

$$V = C_s W \tag{3.47}$$

$$Cs = \frac{\mathrm{SD1}}{\left(\frac{\mathrm{R}}{\mathrm{IE}}\right)\mathrm{T}} \tag{3.48}$$

So, $V = \frac{\mathrm{SD1}}{\left(\frac{\mathrm{R}}{\mathrm{IE}}\right)\mathrm{T}} W$

Where C_s is the seismic response coefficient and W is the effective weight of the structure, which includes the total dead load and the following, other loads:

- In areas used for storage, a minimum of 25 percent of the reduced floor live load (floor live load in public garages and open parking garages need not be included).
- When an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 psf of floor area, whichever is greater?
- Total operating weight of permanent equipment.
- Twenty percent of flat roof snow load where flat roof snow load exceeds 30 psf. To calculate the base shear ,the following parameters are being determined:

a. Seismic Response Coefficient, *C_s*:

The seismic response coefficient ***Cs*** is determined

From IBC **Eq.** (3.49)**:** $$Cs = \frac{\mathrm{SD1}}{\left(\frac{\mathrm{R}}{\mathrm{IE}}\right)\mathrm{T}} \tag{3.49}$$

Where $S_{DI} \equiv$ design spectral response acceleration at 1 second period

R $\equiv$ response modification factor.

$I_E \equiv$ occupancy importance factor.

T ≡ elastic fundamental period of the structure.

The value of C_s need not **exceed** that from Eq. (3.50):

$$Cs = \frac{SDs}{\left(\frac{R}{IE}\right)} \tag{3.50}$$

Where S_{DS} = design spectral response acceleration at short period (0.2 sec)

Also, C_s must not be less than that determined from Eq. (3.51):

$$Cs = .044SDSIE \tag{3.51}$$

For structures assigned to SDC E or F, and for structures located where $S1 \geq 0.6g$, C_s shall not taken less than that determined from IBC Eq. (3.52):

$$Cs = \frac{0.5S1}{\left(\frac{R}{IE}\right)} \tag{3.52}$$

The forces level close to that defined by Eq. (3.47) through (3.49) is also used as the lower bound for the dynamic lateral force procedure of IBC.

b. The Design spectral response acceleration $SDs, SD1$:

In the IBC2000 the design ground motion parameters which are SDs and S_{D1}, rather than Z. S_{DS} and S_{D1}, are 5%-damped design spectral response accelerations at short periods an1 sec. period respectively. The above parameters are taken from two seismic contour maps quantities (S_s and S_1) from which S_{Ds} and S_{D1} are to be derived. The mapped quantities are the Maximum Considered Earthquake spectral response accelerations S_s (at short periods) and S_l (at 1 sec. period).

c.The Design Response Spectrum

For IBC code the maximum considered earthquake is the 2500-year return period earthquake (2% probability of exceedance in50 years) in most of the USA except that in coastal California, it is the largest (deterministic) earthquake that can be generated by the known seismic sources. The design earthquake of the IBC is two-thirds of the MCE (2/3)).

The redefinition of the design earthquake in the IBC is intended to provide a uniform level of safety across the USA against collapse in the Maximum Considered Earthquake. The mapped MCE spectral Response accelerations S_s and S_1 of the 2000 IBC are also mapped on Type S_B soil or rock soil. In IBC, S_{Ds} and S_{D1} are two-thirds of S_{Ms} and S_{M1} which are the soil-modified (Maximum Considered Earthquake) spectral response accelerations at short period and 1 sec. period, respectively. S_M is obtained by multiplying the mapped MCE spectral response acceleration S_s by F_a, the acceleration-related soil factor. S_{M1} is similarly obtained by multiplying the mapped MCE spectral response acceleration S_1 by F_v, the velocity-related soil factor. F_a and F_v, are correspond to Ca,/Z and Cv/Z of the 1997 UBC, respectively. The 2000 IBC has adopted the soil classification and the associated site coefficients first introduced in the 1994 NEHRP Provisions. If S_{Ds} of the 2000 IBC is equal to $2.5C_a$ and S_{D1} of the 2000 IBC is equal to C_v, for a certain site, then(the soil-modified seismicity for that site will not change from the 1997 UBC to the 2000 IBC) a correlation of ground motion parameters between the two codes(UBC and IBC) is possible. No need for the near-source factors in the 2000 IBC because the artificial truncation of ground motion is not a feature of this code and both S_s and S_1 attain high values in the vicinity of seismic sources that are judged capable of producing large earthquakes.

The design spectrum (mapped acceleration parameters) SDs, S1 shall be determined from the 0.2 and 1-second spectrum accelerations .is depicted in Fig. (3.9).That is based on the assumption that a structure will undergo several cycles of inelastic deformation during major earthquake ground motion; therefore, the force level is related to the type of structural system and the structure's estimated ability to sustain these deformations and dissipate energy without collapse. It is important to note that the lower bound of the force level used before could be used in the dynamic lateral force procedure of IBC.

To find the values of Ss and S1 for a certain site, if not provided, it is permitted to take straight-line interpolation where a site is between contours [5].

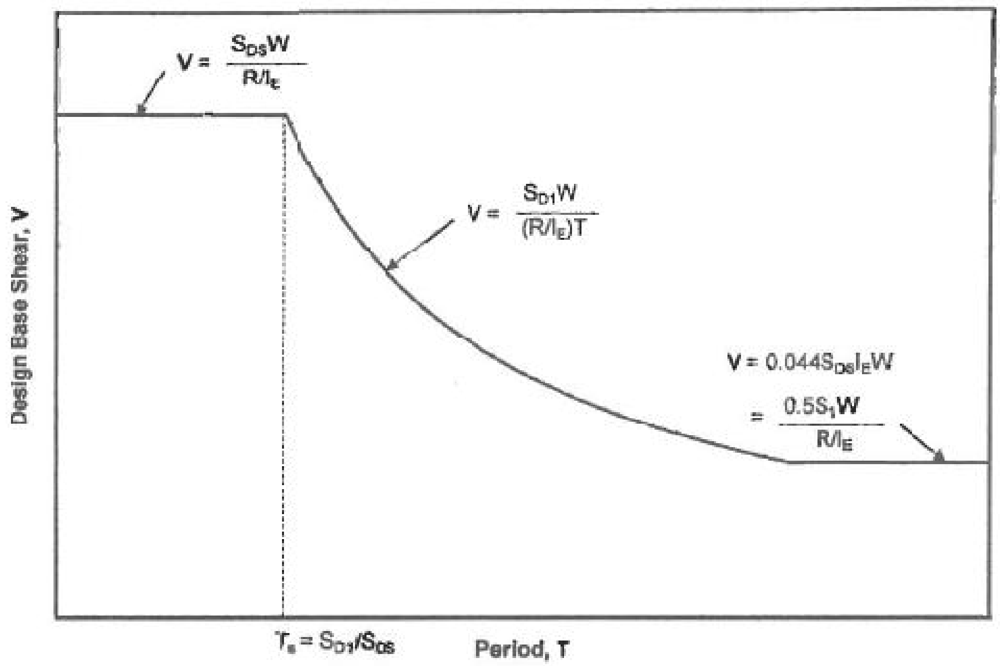


Fig.(3.9): Design Response Spectrums According to the Equivalent Lateral Force Procedure in IBC [5].

The minimum seismic base shear is included in view of the uncertainty and lack of knowledge of actual structural response of long-period buildings subjected to earthquake motions. The lower bound was adopted into the IBC from the 1997 NEHRP Provisions. A form of this lower bound equation originally appeared in the 1997 UBC. Originally applicable to structures assigned to SDC E and F only, the applicability was later expanded to all structures located where $S_1 \geq 0.6g$.

That the IBC design base shears are liable to be up to 29% higher than the values of UBC. For short period structures in IBC, I_{max}=1.5,R_{max}=1.4 ,but in UBC , I_{max}=1.25,R_{max}=1.5.

d. Site classifications:

Based on the site soil properties, the site should be either Site Class A, B, C, D, E or F in accordance with the table below. Site-specific geotechnical investigation will be required on Site Class E. This would currently not be required by the 1997 UBC, see table (A2.1) in appendix A2.

e. Site coefficients F_a,F_v and adjusted maximum considered earthquake spectral response acceleration parameters S_{MS},S_{M1}:

The maximum considered earthquake spectral response acceleration for short periods S_{MS} and at 1-second period S_{M1} adjusted for site class effects shall be determined by

$$S_{MS} = F_a S_s \tag{3.53}$$

$$S_{M1} = F_v S_1 \tag{3.54}$$

Where**:**

F_a = is site coefficient in table (3.7)

F_v= is site coefficient in table (3.8)

Table (3.7): Values of site coefficients F_a*(IBC)

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

a. Use straight line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7-10[12].

Table (3.8): Values of site coefficient F_v*(IBC)

Site class	Mapped Spectral Response Acceleration at short period				
	$S_I \leq 0.1$	$S_I = 0.2$	$S_I = 0.3$	$S_I = 0.4$	$S_I \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.5	1.3
D	2.4	2.0	1.8	1.8	1.5
E	3.5	3.2	2.8	2.8	2.4
F	Note b	Note b	Note b	Note b	Note b

a. Use straight line interpolation for intermediate values of mapped spectral response acceleration at 1 second period, S_I.

b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7[12].

f. The Important factor I:

The IBC has brought the importance factor (the structural use factor) back ,which dropped by ATC 3, the predecessor as it was in UBC beside two other requirements that used to ensure enhanced performance of structures in higher occupancy categories**,** which are**:**

i. Drift limits tighter for structures in higher occupancy categories.

ii. The level of detailing and other restrictions a function of the seismic risk at the site of structure.

These provisions recognize the social importance of certain building in relation to their intended use, and increase the required minimum design base shear by means of importance occupancy category given in table (A2.2) in appendix A2.

Buildings and other structures containing toxic, or highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment that a release of the substances is commensurate with the Risk Category, see tables (A2.3)and (A2.4) in appendix A2.

g. Determination of seismic design category

Where S_S is less than or equal to 0.15and S_1 is less than or equal to 0.04,the structure is permitted to be assigned to seismic design category A. Structures classified as Occupancy Category I, II or III that are located where the mapped spectral response acceleration parameter at I-second period, S_1 is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Structures classified as Occupancy Category IV that are located where the mapped spectral response acceleration parameter at I-second period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a seismic design category based on their occupancy category and the design spectral response acceleration coefficients, S_{DS} and S_{D1}determined in accordance with Section 1613.5.4 of ASCE-7 or the site specific procedures of ASCE- 7. Each building and structure shall be assigned to the more severe seismic

In the Uniform Building Code UBC(ICBO 1997), the Seismic Zone in which a structure is located (The 2000 IBC and the 1997 New Provisions of NEHRP have replaced the Seismic Performance Category (SPC) (used by ATC3 1978 and NEHRP 1994) with a Seismic Design Category (SDC) for the determination of permissible structural systems including the level of detailing required for structural members and joints that are part of the lateral-force-resisting system and for the structural components that are not, limitations on height of structure and structural irregularity, the type of lateral load analysis that must be performed as the basis of design, as well as nonstructural component requirements.. The SPC was a function of occupancy (called Seismic Hazard Exposure Group in the documents being discussed) and of the seismic risk at the site of the structure in the form of the peak velocity related acceleration coefficient, $\mathbf{A_v}$. The SDC is a function of occupancy (called Seismic Use Group in the 2000 IBC and the 1997 NEHRP Provisions) and of soil-modified seismic risk at the site of the structure in the form of the design spectral response acceleration at short periods, S_{Ds}, and the design spectral response acceleration at 1 sec. period, S_{D1}. For a structure, the SDC needs to be determined twice - first as a function of S_{Ds} by table (3.9) and a second time as a function of S_{D1} by table (3.10). The more severe category will govern. When ATC 3 in 1978 made the level of detailing (and other restrictions concerning permissible structural systems, height, irregularity and analysis procedure) a function of occupancy that was a major departure from UBC practice. The departure was continued in all the NEHRP Provisions through the 1994 edition. Now, in the 2000 IBC and the 1997 *NEHRP* Provisions, the level of detailing and the other restrictions have been made a function of the soil characteristics at the site of a structure. This is a further major departure from UBC practice and indeed from current practice across the USA - a move that is likely to have significant impact on the economic and other aspects of earthquake-. Resistant Construction.

Design category in accordance with tabl (3.9) or (3.10), irrespective of the fundamental period of vibration of the structure.

Table (3.9): Seismic design category based on design Short-period (0.2 second) response accelerations S_{Ds} (IBC)

Values of S_{DS}	Occupancy category		
	I or II	III	IV
S_{DS}<0.167g	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

Table (3.10): Seismic design category based on design long period (one second) response accelerations S_{D1} (IBC)

Values of S_{D1}	Occupancy category		
	I or II	III	IV
$S_{DI} < 0.067g$	A	A	A
$0.067g \leq S_{DI} < 0.133g$	B	B	C
$0.133g \leq S_{DI} < 0.20g$	C	C	D
$0.20g \leq S_{DI}$	D	D	D
$0.75 \leq S_{DI}$	E	E	F

From the above two tables, the critical category will be taken.

h. Elastic Fundamental Period, T.

The design base shear is dependent on the elastic fundamental period T for buildings in the intermediate height range. However, T, which is a function of the mass and the stiffness of a structure, cannot be determined until a structure has been designed, since until then, the stiffness and the mass cannot be evaluated. Basically, seismic design cannot be started without a period, and a period cannot be determined until seismic design has been done. This being the situation, building codes includes approximate period formulas, the purpose of which is to get the Design process started. The approximate period formulas deliberately produce estimated periods that are shorter than the "real" periods of actual structures, the idea being that if the initial period estimates is later not refined, the design should still be safe. Since design base shear is inversely proportional to period (Eq. 3.47 and 3.49), a shorter period means a higher base shear used in design. Eq. (3.55) to determine an approximate fundamental period T_a .

Approximate fundamental period Ta:

$$T_a=C_t(h_n)^x \tag{3.55}$$

Where $\mathbf{C_t}$,x are approximate period coefficients or parameters given in table(A2.5) in appendix A2.

hn $\equiv$ height above the base to the highest level of the building.

The fundamental period of the structure, T, in certain direction which is being calculated from the structural responses and deformational characteristics of the resisting elements in a properly substantiated analysis should be, $T \leq T_a C_u$

Where

$C_u \equiv$ coefficient for upper limit on calculated period given in table.

Alternative value for approximate fundamental period Ta for buildings of:

i. 12 storeys or less and the average storey height is at least 3m.

ii. For buildings of entirely resist frame of concrete or entire steel

with above two requirements the value of T, becomes:

$T_a = 0.1N$ (in seconds) (3.56)

N $\equiv$ is the number storeys.

This approximate equation has long been in use for low to moderate-height frames. The base is the level at which the horizontal seismic ground motions are considered to be imparted to a structure. Having started design based on the approximate period and having gone part of the way through, it is then possible to refine the initial period estimate if desired. The code permits this to be done. Period may be estimated by any rational procedure as long as it is in conformance with the principles of mechanics. There is, however, scope for potential abuse here. The rationally computed period of a concrete building is very much dependent upon what stiffness assumption is made in the period computation. Gross section stiffness versus cracked section stiffness makes a big difference; how low the cracked section stiffness is taken to be obviously has a major impact. In order to ensure that an unreasonably low design base shear is not taken by calculating an unduly long period based on unrealistic stiffness assumptions, the code imposes a limit on rationally computed period. Therefore, the rationally computed T may not be taken any longer than a multiplier $\boldsymbol{C_u}$, which is obtained from IBC table (A2.6) in appendix A2, times the approximate period T_a, (this restriction typically does not apply to drift Computations). In the 2000 IBC, C_u depends on the long-period design spectral response acceleration, S_{D1} , and varies from 1.2 for $S_{D1} \geq 0.4$ to 1.7 for $S_{D1} \leq 0.1$. The period $Ts = S_{D1} / S_{DS}$ in Fig.(3.8) is the dividing line between "short-period" and "long-period" response. If the period of a structure T is less than or equal to the transition period, *Ts,* its response is governed by the "flat top" or period-independent part of the spectrum, making it a short-period structure. If, on the other hand, T is larger than or equal to T_s, its response is governed by the period-dependent part of the spectrum; making it long-period structures Equation (3.56) may also be used.

i. Response Modification Factor, R:

The response modification factor R is intended to account for differences in the inelastic deformability or energy dissipation capacity of various structural systems. It reflects the reduction in structural response caused by damping, over strength, and inelasticity. IBC has been suggested that an R-value of 2 is used in design would result in essentially elastic response of a structure to the design earthquake of the IBC. By Contrast, the R-values assigned by the 2000 IBC to structural systems of concrete range from 1.5 to 8. An R-value of 8 (for special reinforced concrete moment frames or dual systems combining special moment frames with special reinforced concrete shear walls) represents one quarter of the strength level that would have been needed for elastic response to the design earthquake of the IBC. An R-value of 1.5 (for a bearing wall system consisting of ordinary plain concrete shear walls) represents elastic response to the design earthquake of the IBC, with a margin of safety built in. The R-values contained in IBC table are largely based on engineering judgment of the performance of various materials and systems in past earthquakes for ATC 3-06 where R was first introduced. There was a certain agreed-upon reference structures which were selected. Two systems having high and low expected levels of performance were chosen to be the special moment frame of steel and a bearing wall system consisting of masonry or concrete shear walls (The R-values for these two systems were chosen considering the seismic design forces assigned to them by older editions of the UBC). No compelling arguments were offered to change the design basis loads for these systems or to change their interrelationship. The expected performances of other systems were then evaluated relative to these reference systems in order to determine the other R-values. Considerations focused on the following issues:

Stability of the total building system.

i. The degree to which the system can be allowed to go beyond the elastic range, its degree of energy dissipation in so doing, and the stability of the vertical load-carrying system during inelastic response due to maximum expected ground motion.
ii. The consequence of failure or partial failure of vertical elements.
iii. Seismic-force-resisting system on the vertical load-carrying capacity.
iv. The inherent redundancy of the system that would allow some progressive inelastic expansion without overall failure. One localized failure of apart must not lead to failure of the system.
v. Where dual systems are employed, important performance characteristics include the ability of the secondary (back-up) system to maintain vertical support when the primary system suffers significant damage at the maximum deformation response .In that situation the back-up system can serve to redistribute lateral loads when the primary system undergoes degradation and should stabilize the building in the event that the primary system is badly damaged.

j. Load combinations:

1-$W=1.2D+F_1L+E_m$ (3.57)

2-$W=0.9D+E_m$ (3.58)

wher:

W= the total Design structural weight.

D=effective dead load

L=Live load

f_l=1 for floors in place of public assembly, for live loads in excess of 100psf (4.79Kn/m2) and for parking garage live load, or f_l=0.5 for other live loads.

E_m= the maximum effect horizontal and vertical forces of seismic as set forth in Section 12.4.3 of ASCE7.

3.3.2.3 Vertical Distribution of Seismic Forces.

Once the design base shear V_b has been determined, the lateral force F_x to be applied at level $\boldsymbol{x}$ of the structure is determined from Eq. (3.59) and (3.60):

$$F_x = C_{vx} V_b \qquad (3.59)$$

$$C_{vx} = w_x h_x^k / \sum_i^n w_i h_i^k \qquad (3.60)$$

So, $F_x = w_x h_x^k V_b / \sum_i^n w_i h_i^k$

Where C_{vx} ≡ vertical distribution factor

k ≡distribution exponent related to the building period

k = I for T ***≤0.5*** sec

= 2 for T ≥ 2.5 sec

= 2, or is to be determined by linear interpolation between 1 and 2 for 0.5 sec < T < 2.5 sec

hi, hx ≡ height from the base to level i and $\boldsymbol{x}$

$w_i, \boldsymbol{w_x}$ ≡ portion of the total load W located or assigned to level ***i*** and ***x***

For structures with T ≤ 0.5 sec, V_b is distributed linearly over the height, varying from zero at the base to a maximum value at the top. For 0.5 sec < T < 2.5 sec, a linear interpolation between a linear and a parabolic distribution is permitted, or a parabolic distribution is also allowed . When T ≥ 2.5 sec, a parabolic distribution is to be used. The larger the value of k, the higher the proportion of V_b distributed to the upper portions of a structure. This produces more overturning moment for the same base Shear, which is characteristic of flexible building response.

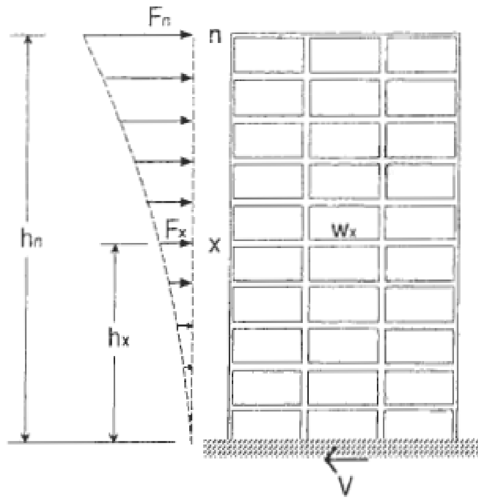


Fig. (3.10): Vertical distribution of Seismic forces[5] .

3.3.2.4 Horizontal Distribution of Seismic Forces.

The seismic design storey shear $\boldsymbol{V_x}$ in any storey $\boldsymbol{x}$ is the sum of the lateral forces acting at the floor or roof level supported by that storey and all the floor levels above, including the roof .

$$V_x = \sum_{i=1}^{n} f_i \tag{3.61}$$

Where:

$f_i = C_{vx} V_b$ [vertical distributed lateral force for one storey].

The distribution of $\boldsymbol{V_x}$**,** to the vertical elements of the lateral-force-resisting system (shear walls and frames) in storey $\boldsymbol{x}$ is determined by the flexibility of the supported diaphragm. A diaphragm is flexible for the purpose of distribution of the storey shear and torsional Moment when the lateral deformation of the diaphragm is more than two times the average storey drift of the associated storey (the storey supporting the diaphragm), determined by comparing the computed maximum in-plane deflection of the diaphragm itself under lateral load with the storey drift of adjoining vertical resisting elements under equivalent tributary lateral load. A diaphragm that is not flexible by the above definition is rigid for the purposes of the code. For flexible diaphragms, the seismic design storey shear V_x is distributed to various vertical elements based on the area of the diaphragm tributary to each line of resistance. The vertical elements of the seismic-force-resisting system may be considered to be in the same line of resistance if the maximum out-of-plane offset between such elements is less than 5 percent of the building dimension perpendicular to the direction of the lateral force .For rigid diaphragms, V_x, is distributed to the various vertical elements of the seismic force-resisting system in the storey under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm . Virtually all computer programs utilized for structural analysis assume diaphragms to be rigid (by default), unless otherwise specified.

3.3.2.5 Torsion, Including Accidental Torsion.

Where diaphragms are rigid, provisions must be made for the increased horizontal forces induced on vertical elements of the lateral force- resisting system resulting from torsion due to eccentricity between the center of application of the lateral forces (center of mass) and the center of rigidity of the seismic force- resisting system (through which the resultant of the resistances to the lateral forces act). Forces are not to be decreased due to torsional effects. The torsional design moment at a given storey must be the moment resulting from eccentricities between applied design lateral forces at levels above that storey and the center of rigidity of the vertical resisting elements in that storey plus an accidental torsion. To compute the accidental torsion, the mass at each level must be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building plan dimension at that level perpendicular to the direction of the force under consideration. Where a torsional irregularity or an extreme torsional irregularity exists, as defined , the effects must be accounted for by increasing the accidental torsion at each level by torsional amplification factor A_x **(1≤ A_x≤ 3.0),** given by IBC Eq. (3.58).

$$A_x = [\delta max/1.2\delta_{avg}]^2 \qquad (3.62)$$

Where:

δmax ≡ the maximum displacement at level x computed assuming Ax=1(in or mm).

δ_{avg} ≡ the average of the displacements at the extreme points of the structure at level x computed assuming Ax=1(in or mm).

This last requirement applies to buildings in Seismic Design Categories C, D, E, and F onIy.

3.3.2.6 Storey Overturning moments:

The structures are to be designed for the effects of overturning caused by the seismic forces determined from section 3.3.2.3 The overturning moment M_x at level $\boldsymbol{x}$ is determined from Eq.(3.63):

$$M_x = \tau\sum F_i\,(h_i-h_x) \qquad (3.63)$$

Where $\boldsymbol{F_i}$ ≡ portion of V induced at level i

h_i, $\boldsymbol{h_x}$ ≡ height from the base to level $\boldsymbol{i}$ and $\boldsymbol{x}$

τ ≡ overturning moment reduction factor

= 1 ***.O*** for the top 10 storeys

= 0.8 for the 20th storey from the top and below

= value between 1.0 and 0.8 determined by a straight-line interpolation

For storeys between the 20th and 10th storeys below the top Note that τ is permitted to be taken as 1.0 for the full height of the structure. The reduction, represented by τ, below overturning moments that are statically consistent with the forces *Fi***,** is justified in terms of higher mode response. A11 the masses at the various floor levels move in the same direction only when response is in the fundamental mode.

3.3.2.7 Interstorey drift and its Limitations:

Drift computation starts with δ**xe** the elastically computed lateral deflection at floor level x under code-prescribed seismic forces (the design base shear V_b**,** distributed along the height of the structure in the manner prescribed by the code). Recognizing that the deflections δ**xe** are much lower than the actual lateral deflections the various floor levels would undergo if the structure were to be subjected to the design earthquake of the IBC (two-thirds of the maximum considered earthquake), the deflections δ**x**e are multiplied by the deflection amplification factor *Cd* producing estimated design earthquake displacements at the various floor levels. At the same time, the δ**x**e values are divided by the occupancy importance factor I_E by which the code prescribed seismic forces (under which the deflections δ**x**, were calculated) were increased for structures belonging to the higher occupancy categories. This is necessary because the limiting values of interstorey drift in the IBC are more stringent for structures in the higher occupancy categories. Without the division by $\boldsymbol{I_E}$ **,** structures with *IE* larger than 1.0 would be doubly penalized. Thus, the deflection , at level **x** is determined from IBC, Eq. (3.64):

$$\delta \mathbf{x} = C_d \, \delta \mathbf{x} e / I_E \qquad (3.64)$$

The design storey drift **Δ** is computed as the difference of the deflections δ , at the centers of mass of the diaphragms at the top and bottom of the storey under consideration (see Fig. 3.10). For structures assigned to Seismic Design Category C or higher, with torsional or extreme torsional irregularity, **Δ** is computed as the largest difference of the deflections along any of the edges of the diaphragms at the top and bottom of the storey under consideration. Deflections δ**x**e must be determined based on the elastic properties of all elements of the lateral-force-resisting system, including the spatial distribution of the mass and the stiffness of the structure. For concrete elements, stiffness properties must include the effects of cracked sections. For the purposes of drift analysis, the upper bound limitation on the computed fundamental period T (T $\leq C_u T_a$) does not apply. The design storey drift **Δ** must be increased by the incremental factor

1.0/(1- ө) when P-delta effects are determined to be significant. When calculating drift, the redundancy coefficient p is to be taken as 1.0

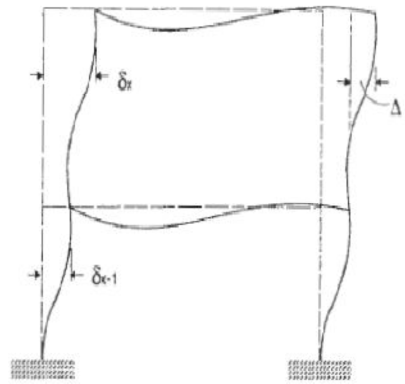


Fig. (3.11): Interstorey Drift **Δ**[5].

Once the design storey drifts are computed, they are to be compared to the allowable storey drift $\boldsymbol{\Delta_a}$ **,** contained in table (3.11). Buildings subjected to earthquakes need drift control to restrict damage to partitions, shaft and stair enclosures, glass and other fragile nonstructural elements, and more important, to minimize differential movement demand on the seismic safety elements. The 2000 IBC limitations on storey drift depend on the following:

The seismic using group (SUG), and generally become more restrictive for the higher use groups, to provide a higher level of performance. The limits also depend on the type of structure. The design storey drifts must not exceed the allowable values.

Table (3.11): Allowable storey drift(IBC)

Structure Type	Risk Category		
	I or II	III	IV
Structures other than masonry shear wall structures, 4 storeys or less, with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the storey drifts.	$0.025h_{sx}$	$0.02h_{sx}$	$0.015h_{sx}$

Masonry cantilever shear wall structures	$0.01h_{sx}$	$0.01h_{sx}$	$0.01h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.02h_{sx}$	$0.015h_{sx}$	$0.01h_{sx}$

a. h_{sx} is the storey height below Level x.

b. For seismic force-resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable storey drift shall comply with the requirements of Section 3.3.2.7.

c. There shall be no drift limit for single-storey structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the storey drifts. The structure separation requirement of Section 3.3.2.8 is not waived.

d. Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

3.3.2.8 Structural Separation

All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact as set forth in this section. Separations shall allow for the maximum inelastic response displacement (δM). δM shall be determined at critical locations with consideration for translational and torsional displacements of the structure including torsional amplifications, where applicable, using the following equation:

$\delta M = C_d \ \delta max./I_e$ (3.65)

Where δmax = maximum elastic displacement at the critical location

Cd ≡ Deflection Amplification factor give n in table with R-values.

Adjacent structures on the same property shall be separated by at least δMT, determined as follows:

$\delta MT = [(\delta M1)^2 + (\delta M2)^2]^{0.5}$ (3.66)

Where:

$\delta M1$ and $\delta M2$ are the maximum inelastic response displacements of the adjacent structures at their adjacent edges.

Where a structure adjoins a property line not common to a public way, the structure shall be set back from the property line by at least the displacement δM of that structure. There is exception for the above, Smaller separations or property line setbacks are permitted where justified by rational analysis based on inelastic response to design ground motions.

3.3.2.9 P-delta Effects:

P-delta effects on storey shears and moments, the resulting member forces and moments, and the storey drifts induced by these effects and evaluation of overall structural stability are not required to be considered where the stability coefficient (ɵ)[ɵ = the ratio of secondary moment to primary moment] as determined by the following equation is equal to or less than 0.10:

$\Theta = Px \Delta Ie / Vx\ hsx\ Cd$ (3.67)

Where:

$Px \equiv$ the total vertical design load at and above level x (kip or kN); where computing Px, no individual load factor need exceed 1.0

$\Delta \equiv$ the design storey drift (in. or mm) as defined in Section 3.3.2.7 occurring simultaneously with Vx.

$Ie \equiv$ the importance factor.

$Vx \equiv$ the seismic shear force acting between Levels x and $x - 1$ (kip or kN).

$hsx \equiv$ the storey height below Level x (in. or mm).

$Cd \equiv$ the deflection amplification factor in table with R-values.

The stability coefficient (ɵ) shall not exceed θ_{max} determined as follows:

$\Theta_{max} = (0.5/\beta\ \boldsymbol{Cd}) \leq 0.25$ (3.68)

If Θ is greater than θ_{max}, the structure is potentially unstable and must be redesigned.

Where **β is** the ratio of shear depend on shear capacity for the storey between levels x and x - 1. If β is not calculated, β is to be taken equal to 1.0. For $0.10 < \theta \leq \theta_{max}$ interstorey drift and element forces must be computed including P-delta effects.

Cd ≡Deflection Amplification factor given in table (A2.7) in appendix A2.

To obtain the storey drift for determining P-delta effects, the design storey drift **Δ is** to be multiplied by [I .0/(I – ɵ)](se e preceding section).

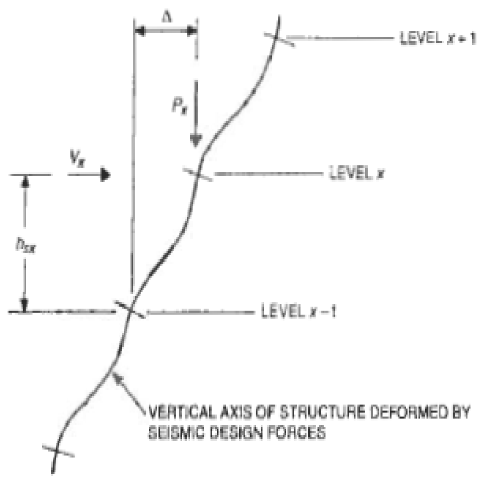


Fig. (3.12): P-delta Effect[5].

3.3.2.10 Simplified Lateral Force Analysis Procedure in IBC

An equivalent lateral force analysis shall consist of the application of equivalent static lateral forces to a linear mathematical model of the structure. The lateral forces applied in each direction shall sum to a total seismic base shear given by Section 3.3.2.10 and shall be distributed vertically in accordance with Section 3.3.2.10. For purposes of analysis, the structure shall be considered fixed at the base.

3.3.2.10.1 Seismic Base Shear

The seismic base shear, V, in a given direction shall be determined in accordance with Eq. (3.69):

$$Vb= FS_{Ds}W/R \tag{3.69}$$

Where:

$S_{DS} = (2/3)\ F_a S_s$

Where F_a is permitted to be taken as 1.0 for rock sites, 1.4 for soil sites, or determined in accordance with Section 3.3.2.2 item d,. For the purpose of this section, sites are permitted to be considered to be rock if there is no more than 10 ft (3 m) of soil between the rock surface and the bottom of spread footing or mat foundation. In calculating S_{DS} and S_s shall be in accordance with Section 3.3.2.2 item b, but need not be taken larger than 1.5.

$F \equiv 1.0$ for buildings that are one storey above grade plane

$F \equiv 1.1$ for buildings that are two storeys above grade plane

$F \equiv 1.2$ for buildings that are three storeys above grade plane

$R \equiv$ the response modification factor from table (A2.7) in appendix A2.

$W \equiv$ effective seismic weight of the structure that includes the dead load, as defined in Section 3.1, above grade plane and other loads above grade plane as listed in the following text:

a. In areas used for storage, a minimum of 25 percent of the floor live load shall be included .There are some exceptions as follows:
    i. Where the inclusion of storage loads adds no more than 5% to the effective seismic weight at that level, it need not be included in the effective seismic weight.
    ii. Floor live load in public garages and open parking structures need not be included.
b. Where provision for partitions is required by Section 3.3.2.2 in the floor load design, the actual partition weight, or a minimum weight of 10 psf (0.48 kN/m2) of floor area, whichever is greater.
c. Total operating weight of permanent equipment.
d. Where the flat roof snow load, *Pf*, exceeds 30 psf (1.44 kN/m2), 20 percent of the uniform design snow load, regardless of actual roof slope.

e. Weight of landscaping and other materials at roof gardens and similar areas.

3.3.2.10.2 Vertical Distribution

The forces at each level shall be calculated using the following equation:

$$F_x=w_xV_b/W \tag{3.70}$$

where w_x = the portion of the effective seismic weight of the structure, W, at level x.

3.3.2.10.3 Horizontal Shear Distribution

The seismic design storey shear in any storey, Vx (kip or kN), shall be determined from the following equation: $V_{xi}= \sum F_i$ (3.71)

where F_i = the portion of the seismic base shear, V (kip or kN) induced at Level i

3.3.2.11 Modal response spectrum analysis for IBC:

An analysis shall be conducted to determine the natural modes of vibration for the structure. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response considered by the model.

The value for each force-related design parameter of interest, including storey drifts, support forces, and individual member forces for each mode of response shall be computed using the properties of each mode and the response spectra divided by the quantity R/I_e. The value for displacement and drift quantities shall be multiplied by the quantity C_d/I_e.

3.3.2.11.1 Combined Response Parameters

The value for each parameter of interest calculated for the various modes shall be combined using the square root of the sum of the squares (SRSS) method, the complete quadratic combination (CQC) method, the complete quadratic combination method as modified by ASCE 4 (CQC-4), or an approved equivalent approach. The CQC or the CQC-4 method shall be used for each of the modal values where closely spaced modes have significant cross correlation of translational and torsional response.

3.3.2.11.2 Horizontal Shear Distribution :

The distribution of horizontal shear shall be in accordance with

$$V_x = \sum_{i=1}^{n} F_i \quad (3.72)$$

3.3.2.12 Diaphragm Design forces:

Diaphragms shall be designed for both the shear and bending stresses resulting from design forces. At diaphragm discontinuities, such as openings and reentrant corners, the design shall assure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

Floor and roof diaphragms shall be designed to resist design seismic forces from the structural analysis, but shall not be less than that determined in accordance with Eq. (3.73) as follows:

$$F_{px} = \sum_{i=1}^{n} F_i \, W_{px} / \sum_{i=1}^{n} W_i \quad (3.73)$$

where

$F_{px} \equiv$ the diaphragm design force

$F_i \equiv$ the design force applied to Level i

$w_i \equiv$ the weight tributary to Level i

$w_{px} \equiv$ the weight tributary to the diaphragm at Level x

The force determined from Eq. (3.73) should be

$$0.2S_{DS}I_e w_{px} \leq F_{px} \leq 0.4S_{DS}I_e w_{px} \quad (3.74)$$

Where the diaphragm is required to transfer design seismic force from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offsets in the placement of the elements or to changes in relative lateral stiffness in the vertical elements, these forces shall be added to those determined from Eq. (3.73). The redundancy factor, ρ, applies to the design of diaphragms in structures assigned to Seismic Design Category D, E, or F. For inertial forces calculated in accordance with Eq. (3.73), the redundancy factor shall equal

1.0. For transfer forces, the redundancy factor, ρ, shall be the same as that used for the structure. The requirements of structures having horizontal or vertical structural irregularities shall also be applied.

Chapter 4

Dynamic Analysis of R.C. Building Using UBC and IBC

4.1Introduction:

Sudan does not have its own seismic code, but there are many studies had been carried out with some provisions for seismic design .Most of these studies were based on UBC code and almost there is no study based on IBC code .The UBC is an old seismic code and was replaced by IBC code as a modern seismic code with some modern ways of collecting seismic data and parameters. In this study, the UBC and IBC codes are used , to study if there are significant differences in their provisions or in their out puts when applied in the same case study .A R.C. building of eight storeys is used in two analysis cases .In case one the site condition is of very low seismic action, which represents most parts of Sudan ,and in case two ,site condition is of moderate seismic action, which represents central Sudan. In both two sites, the soil type is the same but, the seismicity is different. In analysis case one, minimum seismic ground motion parameters from the two codes are used. But in analysis case two ,there is difficulty in defining Sudan seismic parameters for IBC code .To overcome the above difficulty the parameters for cities in the vicinity of Khartoum in the neighbour countries (Cairo , West Tripoli, Addis Ababa and Jeddah) were used to obtain the parameters by linear- interpolation of the altitudes of these cities and their seismic parameters (S_s and S_1) which are provided in tables from study done for selected cities in the world , which are shown in appendix B[13].

4.2 Comparison UBC1997 with IBC 2006 provisions:

The earthquake regulations of Sections 1613 through1623 of the 2000 IBC (ICC 2000), based on the 1997 NEHRP Provisions (BSSC 1997), are substantially different from the corresponding provisions of the 1997 UBC (IBCO 1997). The differences of relevance to this are discussed below.

The biggest change from the 1997 UBC to the 2000 IBC is in the design ground motion parameters which are now S_{Ds} and S_{D1},rather than Z. S_{Ds} and S_{D1}, are 5%-damped design spectral response accelerations at short periods and 1sec. period respectively. The seismic zone map of the UBC has been replaced by contour maps giving two quantities from which S_{Ds} and S_{D1} are to be derived. The mapped quantities are the Maximum Considered Earthquake spectral response accelerations S_s (at short periods) and S_l (at 1 sec. period).

The importance factor of building use that has been used in seismic design by the UBC for a long time is also used by IBC but beside the important factor IBC uses other two provisions, drift limits tighter for structures in higher occupancy categories and it made the level of detailing and other restrictions a function of the seismic risk at the site of a structure with the occupancy of the structure.

Site-specific geotechnical investigation will be required on Site Class E. This would currently not be required by the 1997 UBC.

The ratio of upper-bound seismic design forces (for short-period structures) by the IBC and the UBC. It should be noted that the ratio has three components as below:

1. IF (S_{Ds}/2.5Ca) =1, then
2. Maximum (I_{IBC}/I_{UBC}) = (1.5/1.25) = 1.2
3. Maximum (R_{UBC}/R_{IBC}) = (1.5/1.4) = 1.7

Therefore:

$(I_{IBC}/I_{UBC})\ (S_{Ds}/2.5Ca)*(R_{UBC}/R_{IBC}) = 1.29$

has maximum value of 1.29**,** which mean that the IBC design base shears are liable to be up to 29% higher than the values of UBC.

Table (4.1):comparison of UBC and IBC provisions

UBC 1997	IBC 2006
Based on zone factor z	Based on S_S and S_1 and seismic design category
Soil- modified seismicity	Soil -modified seismicity
$2.5C_a$, C_v	S_{DS},S_{D1}
10% probability of exceedance in 50 years	2% probability of exceedance in 50 years
Return period 475 years	Return period 2500 years
Important factor small relatively	Important factor large relatively plus restriction of detailing as a function of risk category
Near- Source factor	-
-	Tighter drift limits for structures of high occupancy categories
Structure resistance factor are large relatively	Structure resistance factor are small relatively
Z = 0,1 are low seismic	SDC = A , low seismic
Z = 2, moderate seismic	SDC = B , moderate seismic
Z = 3 , high seismic	SDC = C,E and F , high seismic

Fig.(4.1):flow chart for procedures of calculating base shear of UBC.

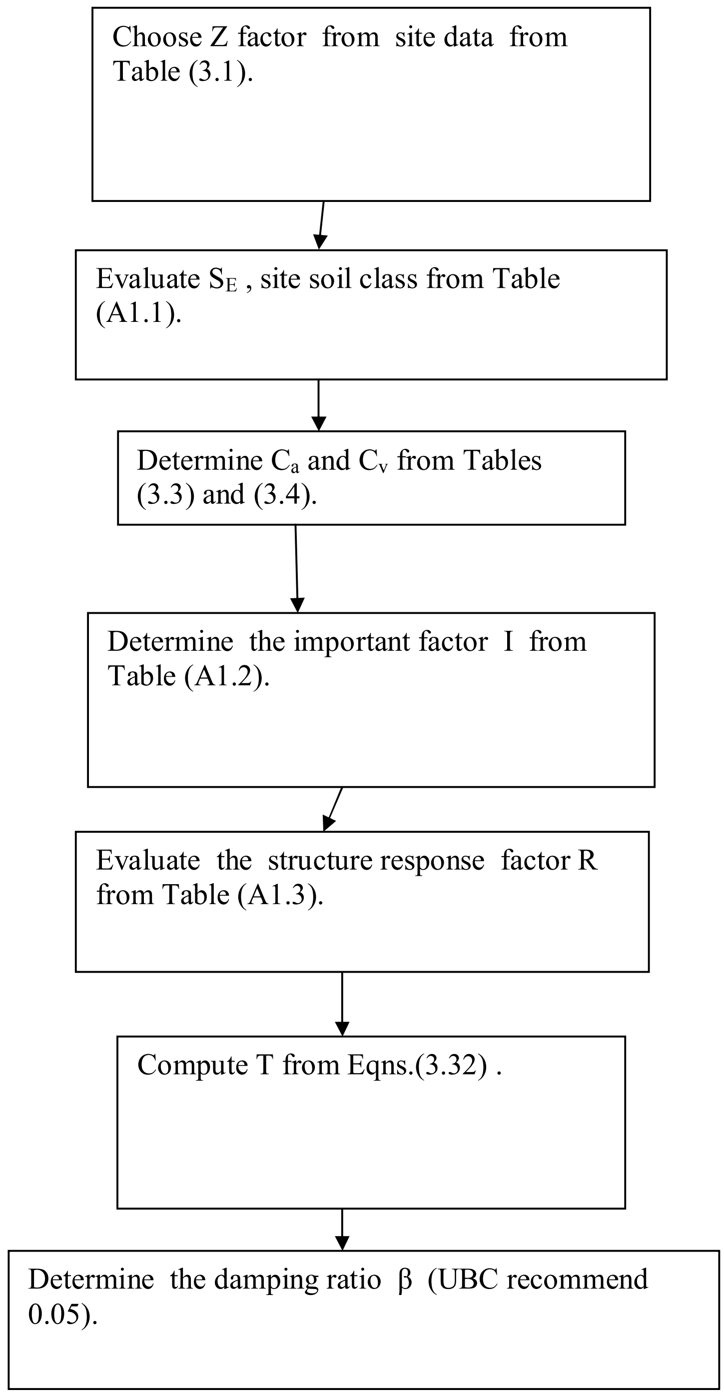
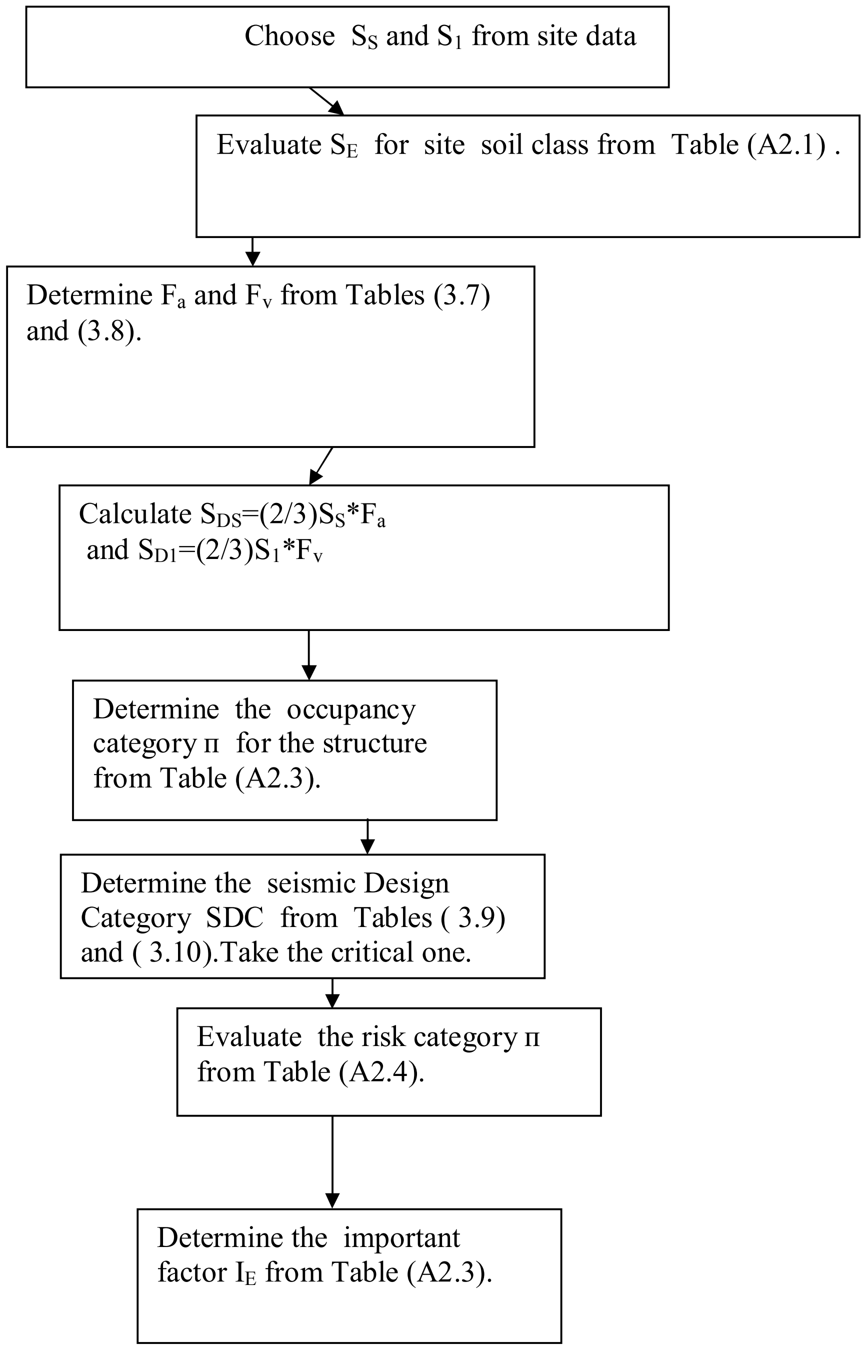


Fig.(4.2):flow chart for procedures of calculating base shear of IBC



Evaluate structu re system response factor R from Table (A2.7)

Compute T from Eqns.(3.55)-(3.56)

Evaluate T_L from site data (mapped in ASCE-7-10).

Determine damping ratio .Take it 0.05 as recommended by IBC.

4.3 Application of Comparative Seismic Dynamic analysis of R.C. building using ETABS v9.7.4 computer program.

4.3.1 The Description of the building for the two cases:

A R.C.commercial building (see Appendix B), its type is building frame system, ordinary braced frames .Special occupancy structure, located on soft clay soil.

The building is eight storeys with total height of 28.5m, the ground height is 4m and all other storeys are 3.5m in height. The sections of the elements are as follows:

i. All columns are 500x500 mm in all storeys.
ii. The slabs in all storeys are 220 mm thick.
iii. The plan is tow bays, 2x5m=10m, in y-axis and six bays, 6x4m=26m, in x-axis.

4.3.2 The loading of the building:

a. Dead loads:

i. Partitions: Brick wall=18x.25=4.5kN/m^2
ii. Finishing=0.1x20=2kN/m^2
iii. Electrical and other suppliers equipments=0.5kN/m^2

The total superimposed dead load =4.5+2+0.5= 7kN/m^2

b. live load :the live load =2.5kN/m^2

c. The earthquake loads:

Use two seismic codes ,UBC and IBC codes .Apply two cases of seismic regions in Sudan ,in case one very low earthquake region and in case two moderate earthquake region (central Sudan).

1. In case one:

The UBC data are as follows:

Zone 0, zone factor z = 0.01, the lowest seismic active zone in UBC, Table (3.1)..

Soil class E, from Table (A1.1)

Important factor I = 1, from Table (A1.2)

System resistance factor R = 5.6, from Table (A1.3)

Seismic coefficient C_a = 0.19 from Table (3.3)

Seismic coefficient C_v = 0.26, from Table (3.4)

Damping ratio = 0.05, recommended in UBC.

The IBC data are as follows:

Seismic design category SDC A, the lowest seismic active category in IBC.

Soil class E, from Table (A2.1)

S_S = 0.15, Recommended for SDC A

S_1 = 0.04, Recommended for SDC A

T_L(long period transitional period) = 4 ,from maps in ASCE7 -10 (approximate) not provided for Sudan cities and vicinity country cities.

Damping ratio = 0.05 recommended in IBC.

2. Case two:

The UBC code data are as follows:

Zone 2A, zone factor z = 0.1, From Table (3.1)

Soil class E from, Table (A1.1)

Important factor I=1, from Table (A1.2)

System resistance factor R=5.6, from Table (A1.3)

Seismic coefficient C_a=0.3, from Table (3.3)

Seismic coefficient C_v=0.5 from Table (3.4)

Damping ratio =0.05, recommended in UBC.

The IBC data are as follows:

Seismic design category C, the critical one from Tables (3.9) and (3.10)

Soil class E, from Table (A2.1)

S_S=0.48 The location of Khartoum (central Sudan) does not has values of Ss and S1,so by interpolation of S_s and S_1 of the vicinity cities of Khartoum in the neighbour countries (Cairo, West Tripoli, Addis Ababa and Jeddah) were used to obtain the parameters by linear- interpolation of the altitudes of these cities and their seismic parameters (S_s and S_1) which are provided in tables from study done for selected cities in the world , which are shown in appendix B[13].

S_1=0.18 obtained the same as S_s above.

(long period transitional period)T_L=8, from maps in ASCE7 -10 (approximate) not provided for Sudan cities and vicinity country cities.

Damping ratio=0.05, recommended in IBC.

For other input data see appendix B.

4.3.3 ETABS Response Spectrum Analysis

The building was modeled using ETABS v9.7.4 computer program which applies ultimate limit state design by default with UBC and IBC codes. The Response Spectrum Analysis was conducted in two analysis cases (see Appendix C).

4.4 Analysis of Results

The deformed shape of the building is shown in Figures (4.5.) which is visually accepted, ensured numerically by, displacements, drifts, forces and shears tables.

1. The output of the analysis indicate that, the lateral loads to diaphragms increase from base to top story uniformly, in two analysis cases for the values of two codes . In two analysis cases the values of UBC code are greater than IBC code as shown in Figure (4.6) and table (4.2).Where, the biggest difference between the values of UBC and IBC codes for the two analysis cases are 42.62% and 37.91% respectively.

2- The Analysis Results show that , the lateral loads to storeys ,are increase from base to top storey . These are uniform for values of UBC code, in both tow analysis cases, but for the values of IBC code, in both two analysis cases, increase uniformly until storey 7, then suddenly decrease in the last storey. In two analysis cases the greater values are in UBC code as shown in Figure (4.7) and table (4.3). Where, the biggest difference between the values of UBC and IBC codes for the two analysis cases are 28.72 and 43.6% respectively.

3- It was noticed that the diaphragm displacements was zero in analysis case one from base to stoery 3 and then increased to 0.01m,from storey 4 to storey 8,so ,exactly, the same behavior in two codes as shown in Figure (4.6) and table (4.3). In two analysis case, the values of two codes are zero displacements from base to storey, then increased to 0.01m, in storey 2 .There are increase and stay constant in the values of both two codes ,until end up to,0.04m , in the last storeys , for values of two codes . So, nearly the same behavior in both two codes, in two analysis cases, (but, there is tendency to be greater in IBC2 comparing to UBC2).

4- The diaphragm drifts **Δ** , increase from base to top storey in all analysis cases, for values of both two codes as shown in Figure (4.7) and table (4.4). The greater values are in IBC, in both two analysis cases. Where, the biggest difference between the values of UBC and IBC codes for the two analysis cases are 27.3% and 17.1% respectively

5- The storey displacements , δ_{st}, are in all analysis cases and for values of both codes, start from zero at base and increase uniformly until reach the maximum in the top storeys. The greater values are in IBC, in both two analysis cases as shown in Figure (4.8) and table(4.5). Where, the biggest difference between the values of UBC and IBC codes for the two analysis cases are 25% and 1465% respectively

6- The maximum storey drifts $\mathbf{\Delta_{st}}$, are in both two analysis cases for the values of two codes, increase from base to top storey uniformly, and the greater values are in IBC code in two analysis cases as shown in Figure (4.9) and table (4.6). Where, the biggest difference between the values of UBC and IBC codes for the two analysis cases are 15.28 and 13.96% respectively. For IBC code, in two analysis cases, the allowable storey drift, Δa= 0.02hst. So, for all storeys other than ground, Δa =0.02x3.5=0.07m. For ground floor, Δa =0.02x4=0.08m.

For UBC code, the approximate fundamental period, Ta, is greater than 0.7second, in two analysis cases, so, the allowable storey drift, $\mathbf{\Delta_a} = 0.02hst$, as in IBC above ,0.07m, and 0.08m ,for other storeys and ground floor, respectively. Notice that, all the storey drifts in the table are less than the allowable storey drift values.

7- The obtained base shears values are greater in IBC code in both two analysis cases .The storey shears are decreases from base to top storeys uniformly in values of two codes, and in two analysis cases.

However ,in the last storeys, it is opposite, so the greater storey shears are in the values of UBC code as shown in Figure (4.10) and table (4.7) .It happened ,because, in UBC code, some tall buildings, do not have uniform triangular distribution of the lateral force through the height of the structure , therefore, when T > 0.7 seconds, there should be concentrated lateral force, ($F_t = 0.07TV \leq 0.25V$), at the top of the last floor,{so, the sum become (F_n+F_t)}, which increases the storey shear force for the last storey, but it does not the case in IBC code . Where, the biggest difference between the values of UBC and IBC codes for the two analysis cases are 15.63% and 14.35% respectively.

8- By noticing Figure (4.11) and table(4.8),give attention that, the storey overturning moments M_{st}, in both two analysis cases and in the values of the two codes, decrease from base to top storey uniformly and the greater values are in IBC code, which are compliance with the behavior of storey shears, in spite of, it is different in storey 8,which, here, in both analysis cases, the values in UBC code are greater than these in IBC code . Where, the biggest difference between the values of UBC and IBC codes for the two analysis cases are 13.87% and 17.61% respectively.

Table (4.2): Lateral loads to diaphragms (kN).

Height (m)	Lateral loads to diaphragms (kN)						
	Storey No.	Case 1			Case 2		
		UBC 1	IBC1	Difference %	UBC 2	IBC2	Difference%
0	Base	4.55(GM)	2.49	42.62	19(GM)	11.53	37.91
4	1	4.55	2.49	18.31	33.68	11.53	20.76
7.5	2	7.8	6.18	6.6	48.36	27.17	6.3
11	3	11.71	11.17	8.68	65.64	46.53	5.86
14.5	4	15 .1	16.59	21.25	80.75	69.99	16.19
18	5	18.86	22.76	23.6	96.73	94.7	22.66
21.5	6	22.22	29.05	28.85	112.27	122.29	27.13
25	7	26.12	36.1	5.5	158.48	151.52	7.2
28.5	8	37.4(G)	35.34	5.5	158.91(G)	146.99	7.2
		Greater in UBC (42.62%)			Greater in UBC (37.91%)		

G ≡ The greater value in the corresponding one case values.

GM ≡ The greater in minimum values, in the corresponding one case values.

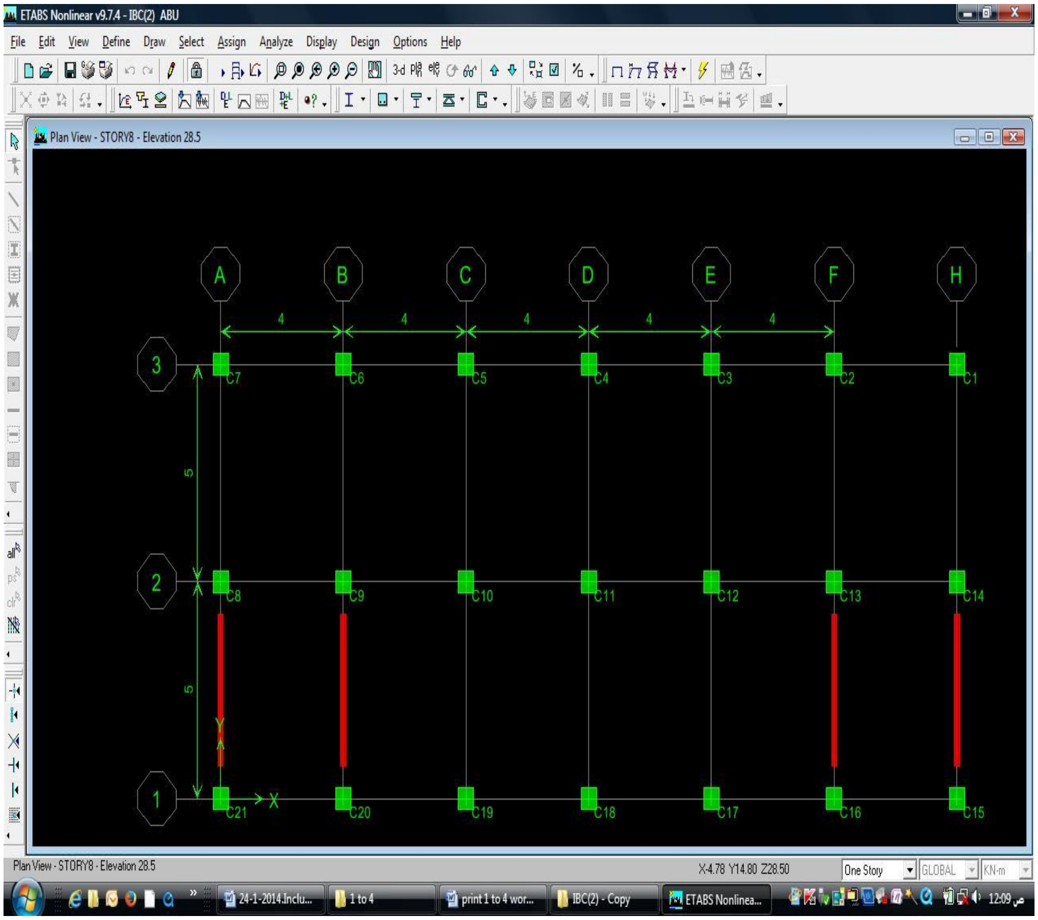


Fig.(4.3): The building plan (for large scale figures, see appendix C).

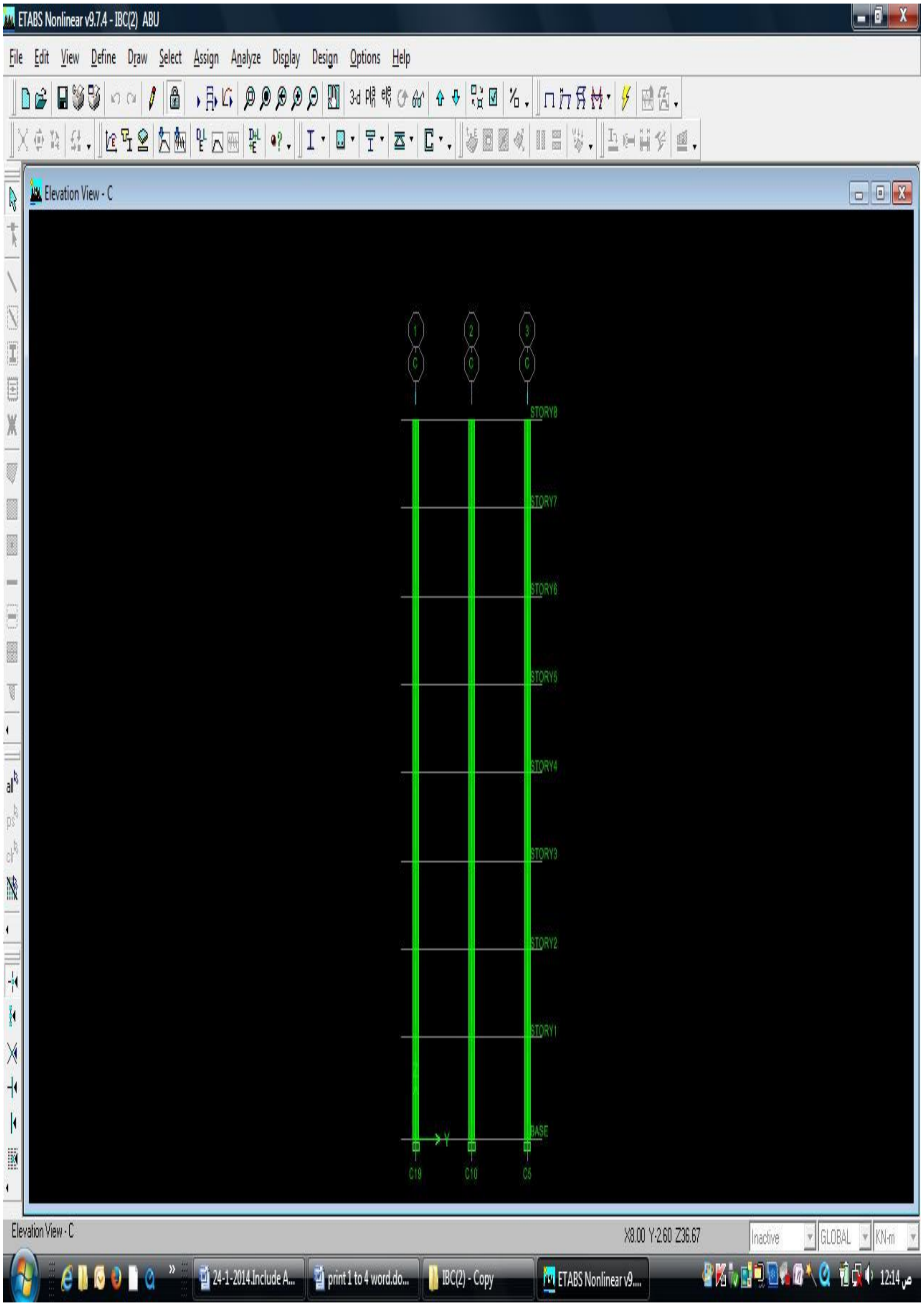


Fig.(4.4): The building West elevation.

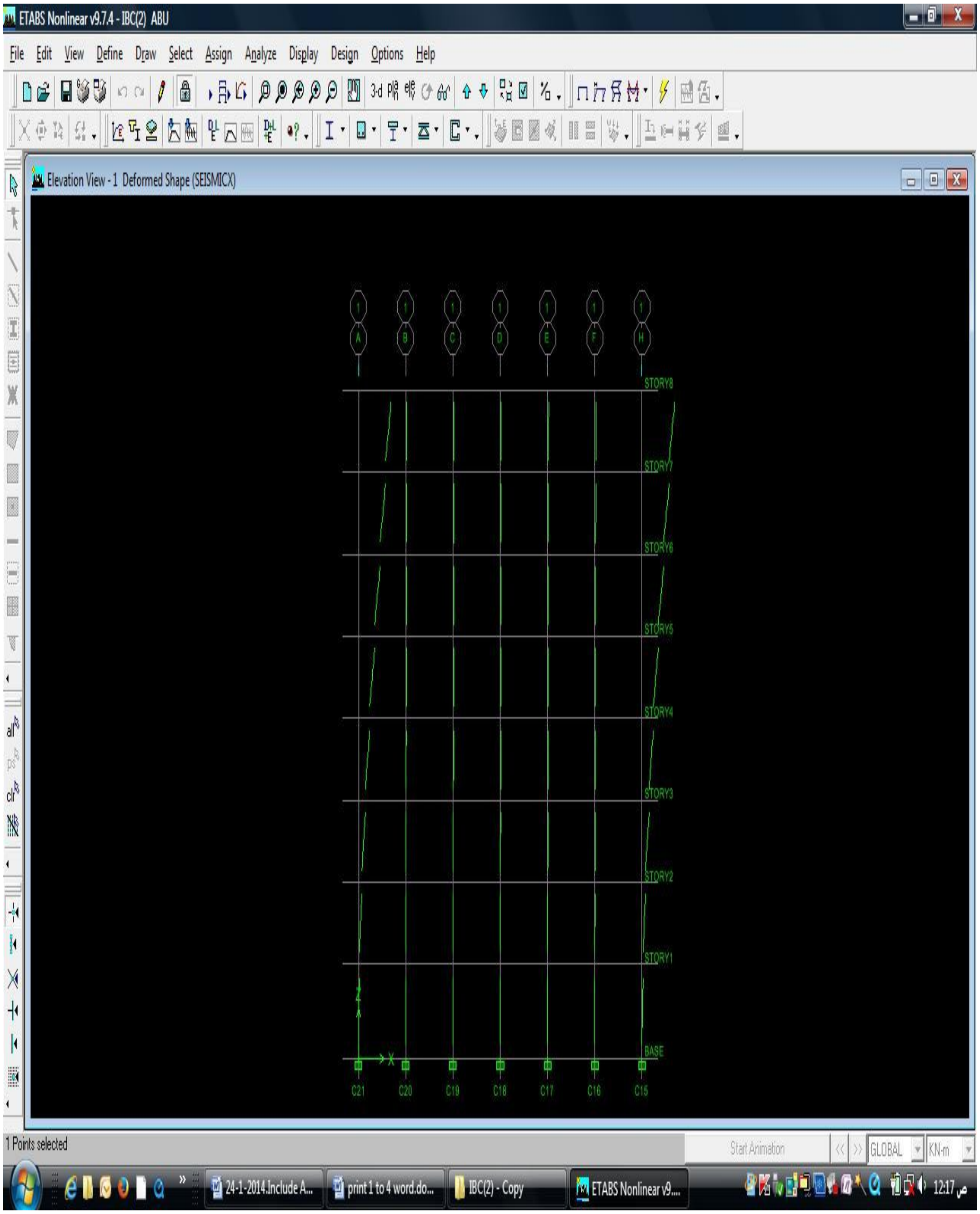
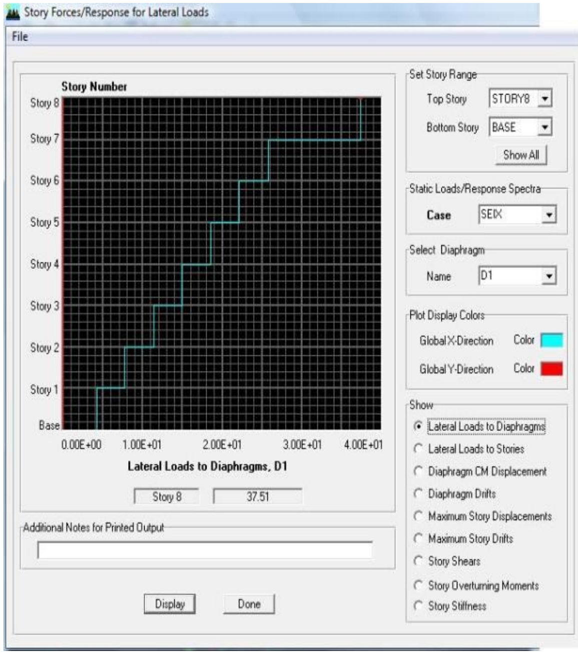
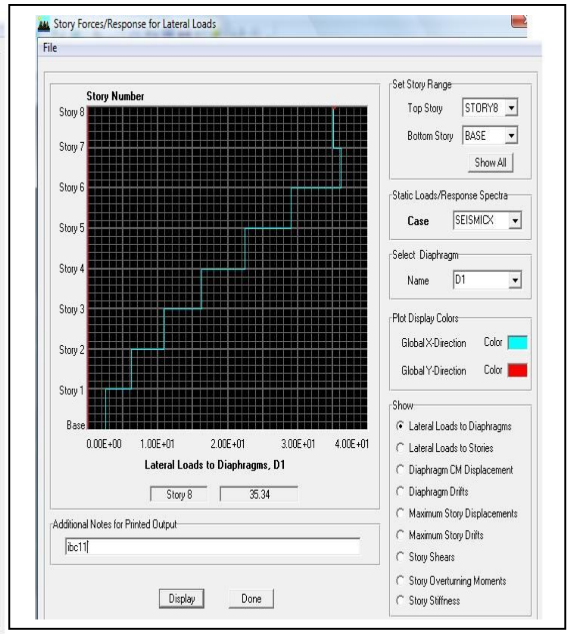


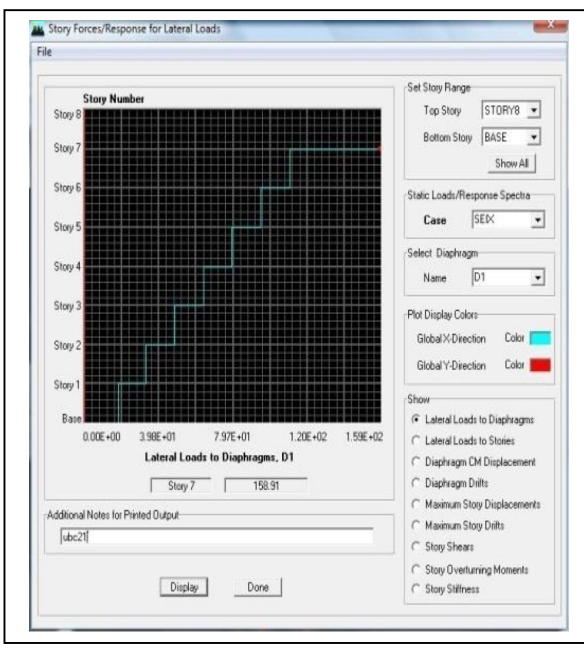
Fig.(4.5) :The building South elevation Deformed shape.

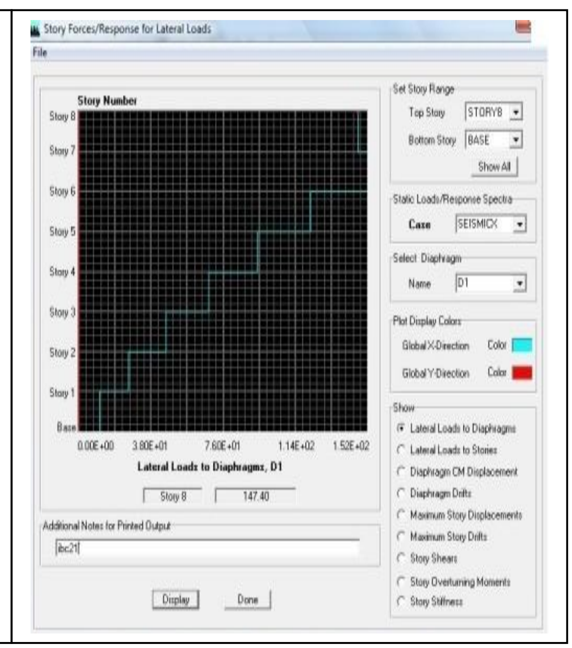


(a) UBC 1



(b) IBC 1





(c)UBC2 (d) IBC 2

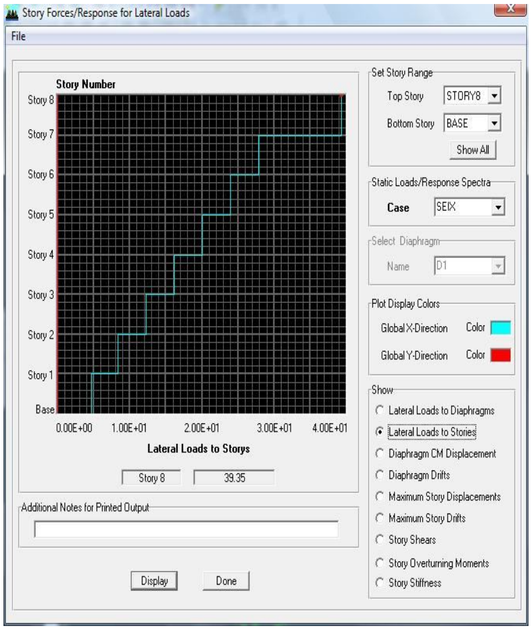
Fig. (4.6): Lateral loads to diaphragms (for large scale, see appendix C).

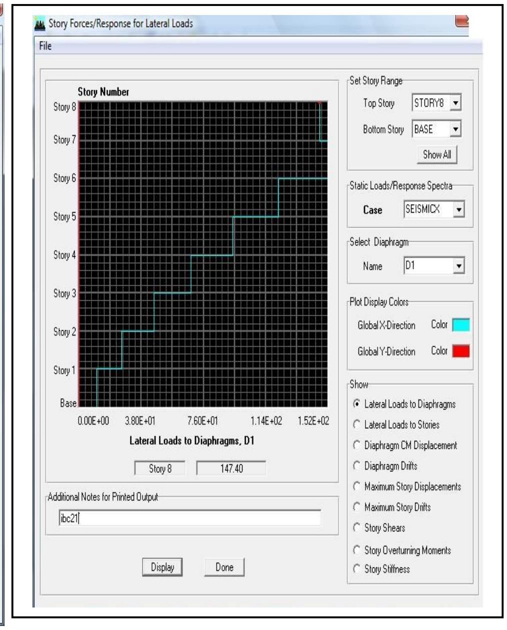
Table (4.3): Lateral loads to storeys (kN).

Height (m)	Lateral loads to storeys (kN)						
	Storey No.	Case 1			Case 2		
		UBC 1	IBC 1	Difference %	UBC 2	IBC 2	Difference %
0	Base	4.88(GM)	2.93	39.95	21.35(GM)	12.93	43.6
4	1	4.88	2.93	18.2	36.8	12.93	22.44
7.5	2	8.46	6.83	0.91	53.15	28.98	6.86
11	3	12.47	12.03	7.9	69.51	50.39	6.09
14.5	4	16.37	17.78	17.79	87.22	74.47	14.95
18	5	20.49	24.5	24.14	103.58	102.56	22
21.5	6	24.17	31.54	28.72	119.02	132.88	27.46
25	7	28.08	39.35	6.07	166.7	164.09	6.64
28.5	8	39.46(G)	36.86	6.07	166.7(G)	156.07	6.64
		The biggest difference (28.72%)			The biggest difference (43.6%)		

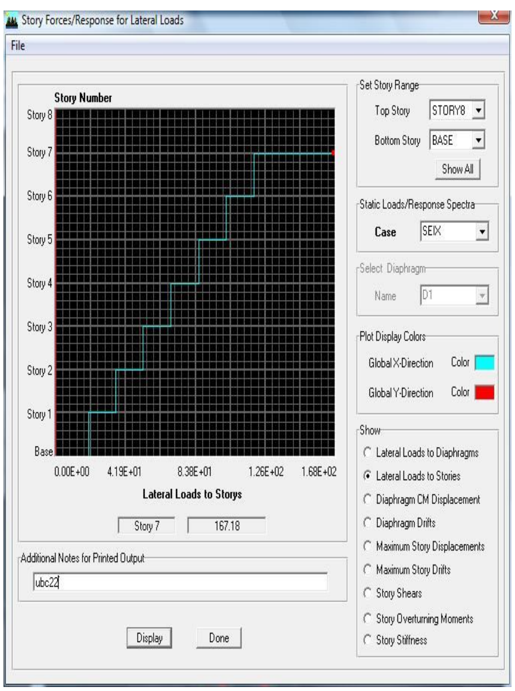
Table (4.4): Diaphragm CM displacements δ(m).

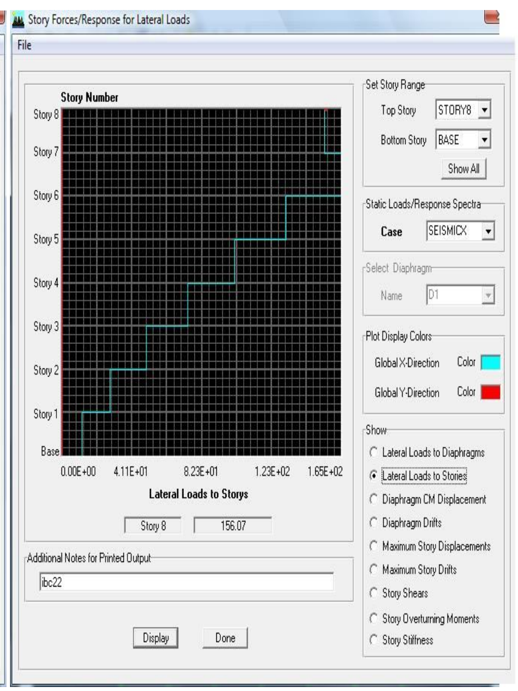
Height (m)	Diaphragm CM Displacementsδ (m)						
	Storey No.	Case 1			Case 2		
		UBC 1	IBC 1	Difference %	UBC 2	IBC 2	Difference %
0	Base	0.00	0.00	0	0.00	0.00	0
4	1	0.00	0.00	0	0.00	0.00	0
7.5	2	0.00	0.00	0	0.01	0.01	0
11	3	0.00	0.00	0	0.02	0.02	0
14.5	4	0.01	0.01	0	0.02	0.03	33.33
18	5	0.01	0.01	0	0.03	0.03	0
21.5	6	0.01	0.01	0	0.03	0.04	25
25	7	0.01	0.01	0	0.04	0.04	0
28.5	8	0.01	0.01	0	0.04	0.04	0
		The same			The same(tend to be great in IBC)		





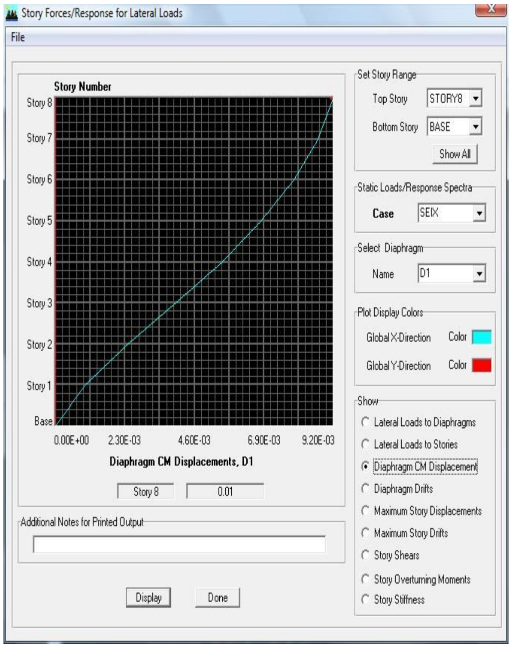
(a) UBC 1 (b) IBC 1



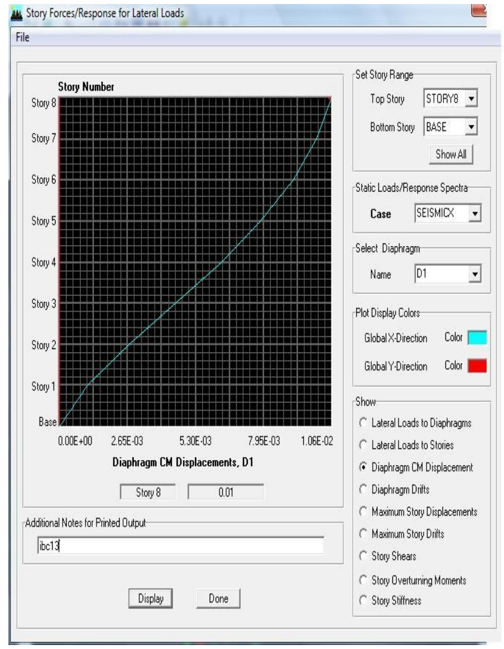


(c)UBC2 (d) IBC 2

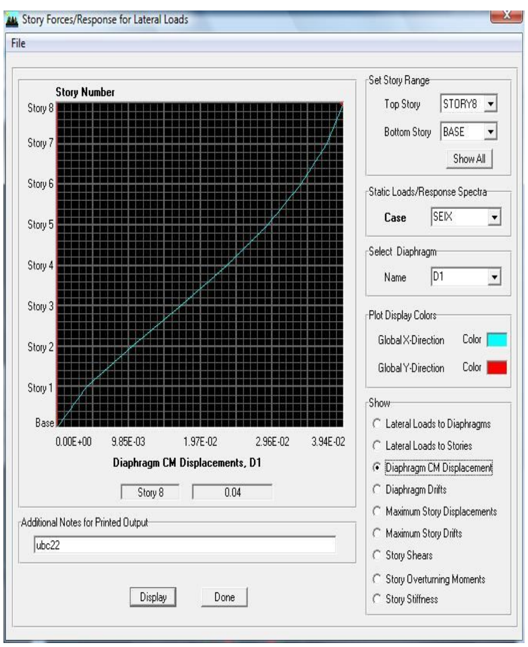
Fig. (4.7): Lateral loads to storeys.



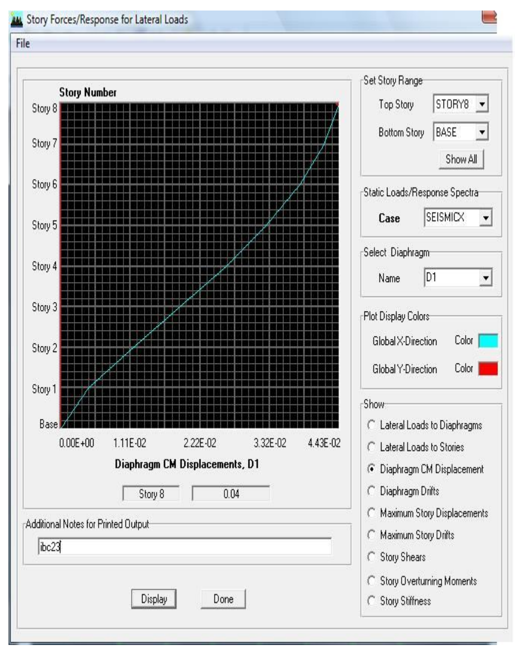
(a) UBC 1



(b) IBC1



(c)UBC2



(d) IBC 2

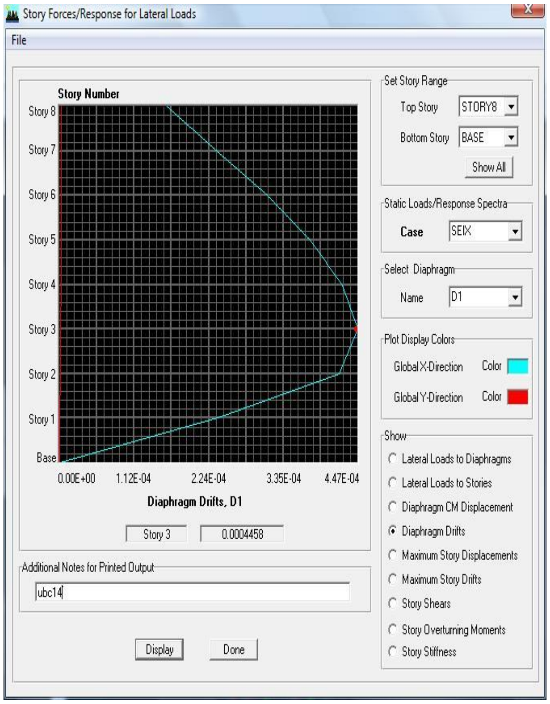
Fig. (4.8): Diaphragm CM displacement.

Table (4.5): Diaphragms drifts Δ (m).

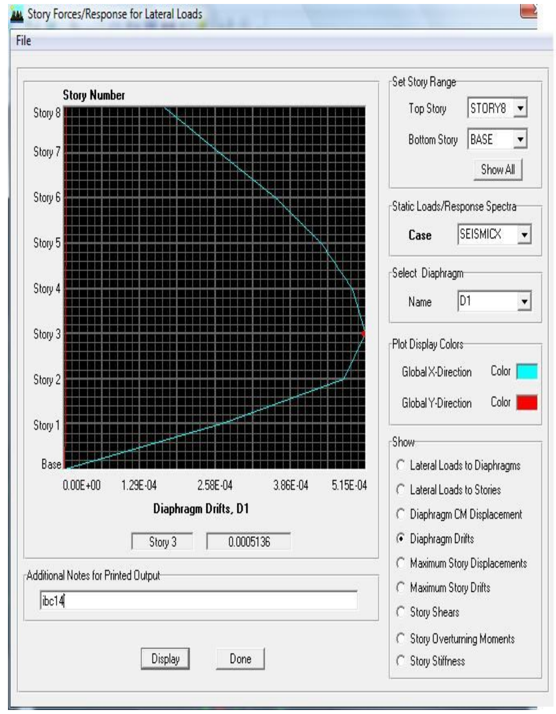
Height (m)	Diaphragm drifts Δ(m)						
	Storey No.	Case 1			Case 2		
		UBC 1	IBC 1	Difference %	UBC 2	IBC 2	Difference %
0	Base	0.0000061	0.0000082(GM)	27.3	0.0000258	0.0000234(GM)	9.3
4	1	0.0002359	0.0002582	8.98	0.0010594	0.001175	9.83
7.5		0.0004191	0.0004787	12.45	0.0017933	0.002005	10.55
11	3	0.0004458	0.0005136(G)	13.2	0.0019018	0.0021512(G)	11.59
14.5	4	0.0004252	0.0004927	13.7	0.001814	0.0020635	12.09
18	5	0.0003731	0.0004382	14.85	0.0015918	0.001835	13.25
21.5	6	0.0003089	0.0003587	13.88	0.001323	0.0015081	17.1
25	7	0.0002350	0.000261	9.96	0.0010026	0.001099	8.77
28	8	0.0001635	0.0001759	7.04	0.0006977	0.0007248	3.73
		The biggest difference (27.3/%)			The biggest difference (17.1%)		

Table (4.6): Maximum storey displacementsδ_{st} (m).

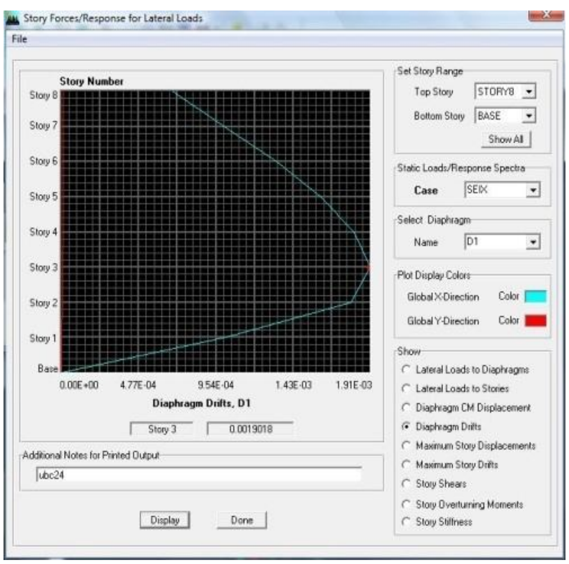
Height (m)	Maximum storey displacements δ_{st} (m)						
	Storey No.	Case 1			Case 2		
		UBC 1	IBC 1	Difference %	UBC 2	IBC 2	Difference %
0	Base	0.00	0.00	0	0.00	0.00	0
4	1	0.01	0.01	0	0.05	0.05	0
7.5	2	0.03	0.04	25	0.14	0.15	6.66
11	3	0.06	0.07	14.28	0.27	0.31	12.9
14.5	4	0.1	0.12	16.66	0.42	0.49	14.28
18	5	0.14	0.17	17.64	0.6	0.7	33.33
21.5	6	0.19	0.22	13.63	0.8	0.93	13.97
25	7	0.24	0.28	14.28	0.99	1.16	14.65
28.5	8	0.28	0.33(G)	15.15	1.19	1.37(G)	13.13
		The biggest difference (25%)			The biggest difference (14.65%)		



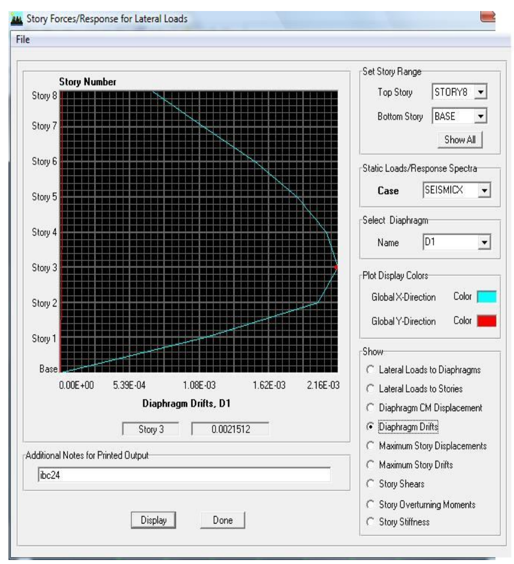
(a) UBC 1



(b) IBC1

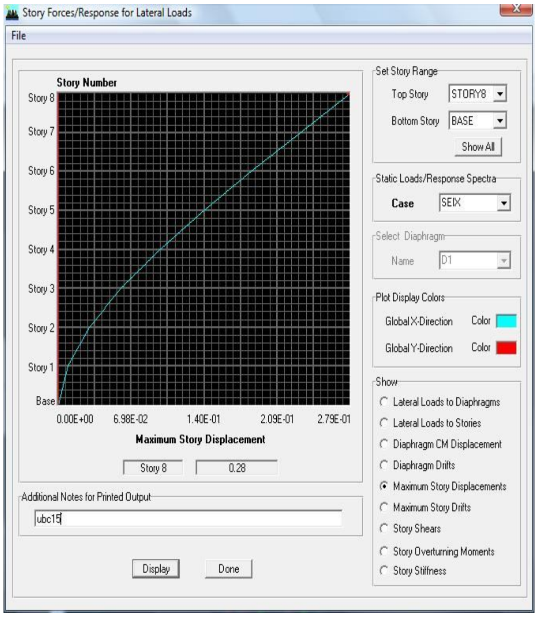


(c) UBC 2

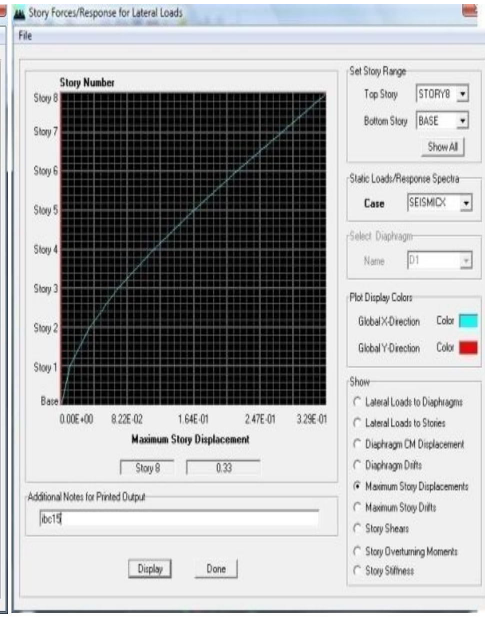


(d) IBC 2

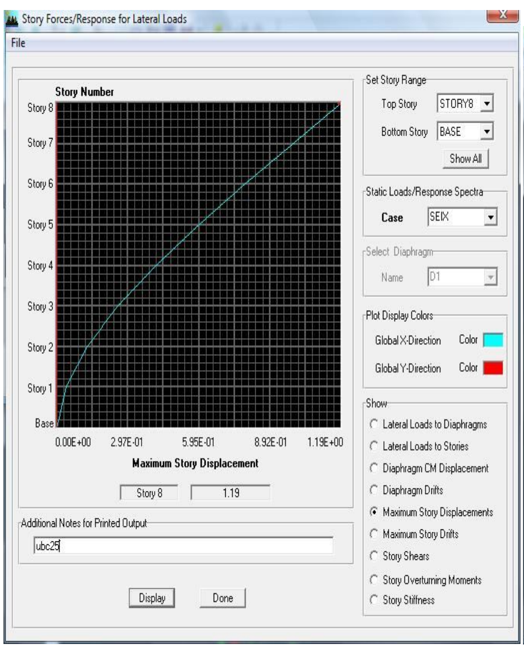
Fig. (4.9): Diaphragm drifts.



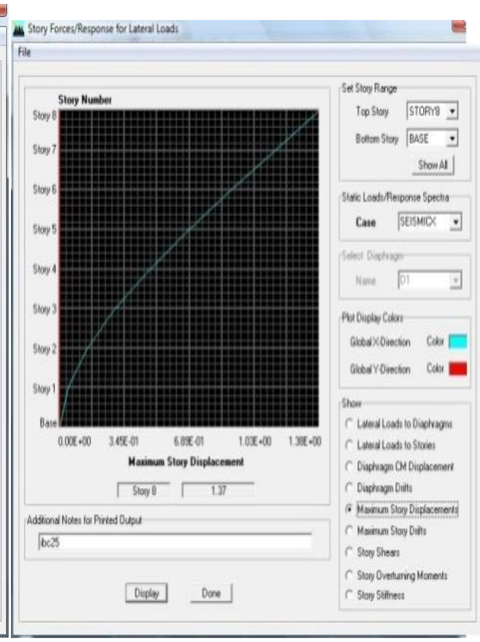
(a)UBC1



(b) IBC1



(c)UBC2



(d) IBC2

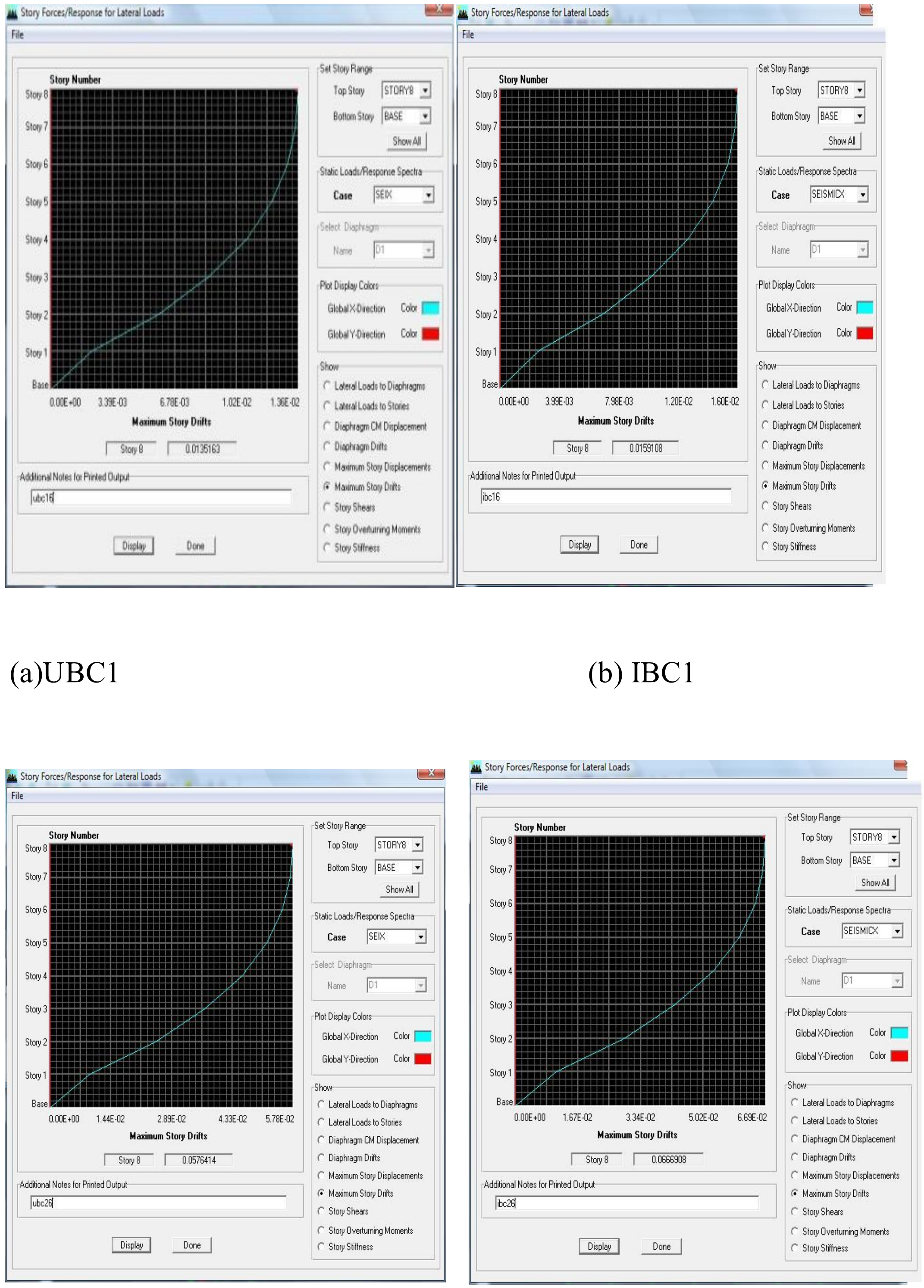
Fig. (4.10): Storey displacements.

Table (4.7): Maximum storey drifts Δ_{st} (m).

Height (m)	Maximum storey drifts Δst(m)						
	Storey No.	Case 1			Case 2		
		UBC 1	IBC 1	Difference %	UBC 2	IBC 2	Difference %
0	Base	0.0000735	0.0001729(GM)	15.03	0.0003133	0.0007249(GM)	13.57
4	1	0.000024241	0.0027239	15.05	0.0100246	0.0114172	9.13
7.5	2	0.0060236	0.0070907	15.04	0.025688	0.0295397	13.03
11	3	0.0087784	0.0102469	11.77	0.0371223	0.0431315	13.93
14.5	4	0.0107616	0.0126248	14.75	0.0460504	0.0529177	13.08
18	5	0.012194	0.0143543	15.04	0.05200025	0.0603479	13.83
21.5	6	0.0130021	0.0153487	15.28	0.0556051	0.0645161	13.81
25	7	0.0134428	0.0158243	15.14	0.0573281	0.06663283	13.96
28.5	8	0.0135165	0.0159108(G)	9.69	0.0576414	0.0666908(G)	13.56
		The biggest difference (15.28%)			The biggest difference (13.96%)		

Table (4.8): Base shears and Storey shears V_x(kN).

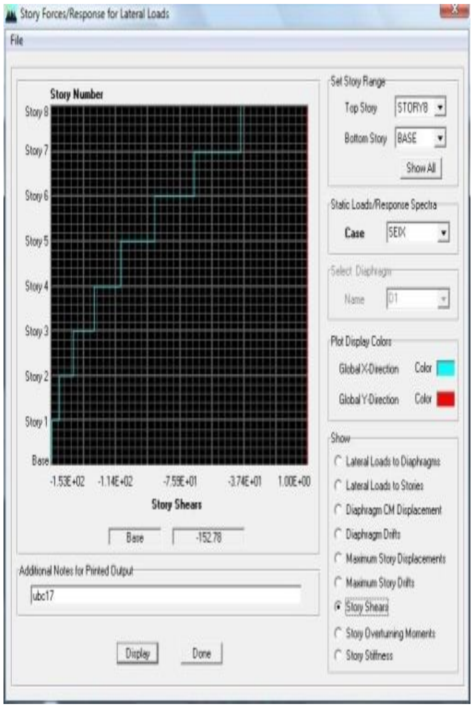
Height (m)	Storey shears V_x(kN)						
	Storey No.	Case 1			Case 2		
		UBC 1	IBC 1	Difference %	UBC 2	IBC 2	Difference %
0	Base Shear	152.78	169.85(G)	10.05	651.54	715.8(G)	8.97
4	1	152.36	169.85	12.03	646.23	711.91	9.22
7.5	2	147.36	167.52	13.39	628.55	698.32	9.99
11	3	139.44	161.00	15.3	589.65	671.12	12.13
14.5	4	126.11	148.9	15.63	534.83	624.5	14.35
18	5	110.69	131.21	15.25	472.93	548.74	13.8
21.5	6	90.68	107.00	11.45	388.05	449.67	13.7
25	7	66.93	74.88	7.1	283.71	315.58	10.09
28.5	8	39.01(GM)	36.24		165.23(GM)	152.46	8.37
		The biggest difference (15.63%)			The biggest difference (14.35%)		



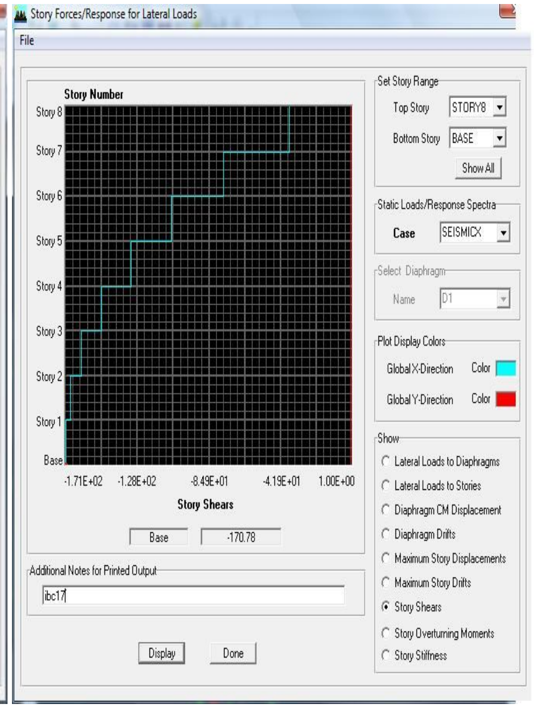
(a)UBC1 (b) IBC1

(c)UBC2 (d) IBC2

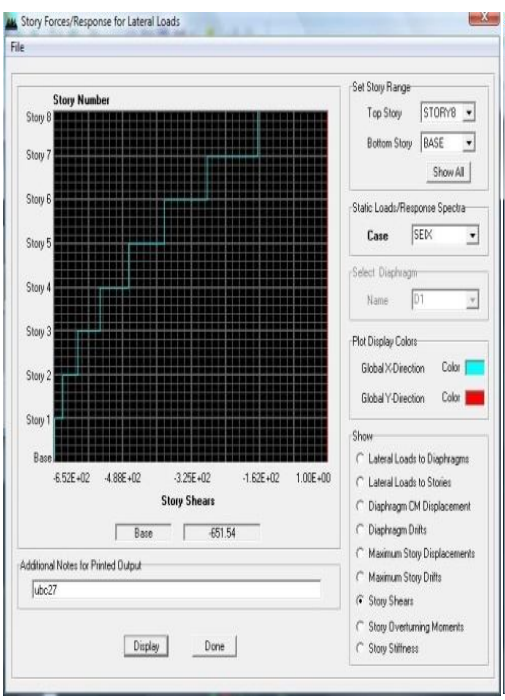
Fig. (4.6): Storey drifts.



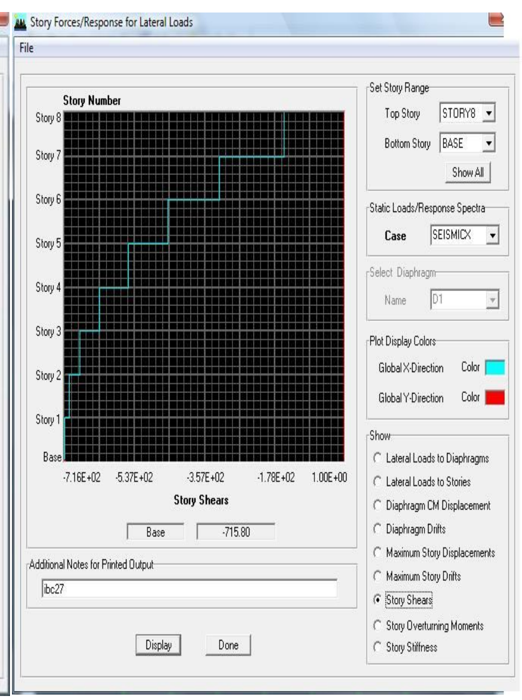
(a)UBC1



(b) IBC1



(c)UBC2

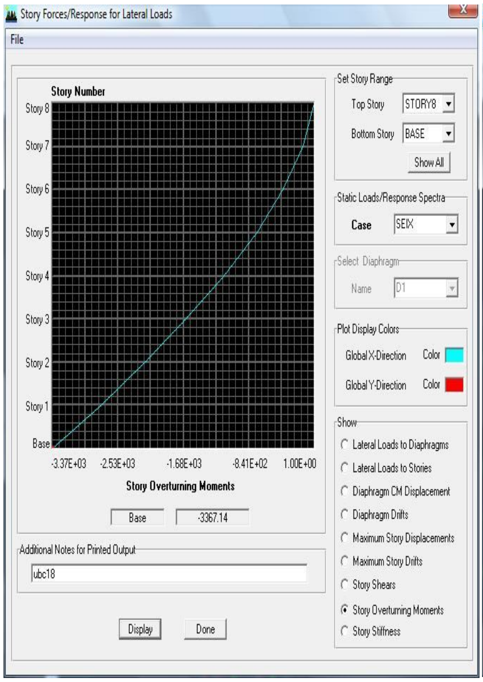


(d) IBC2

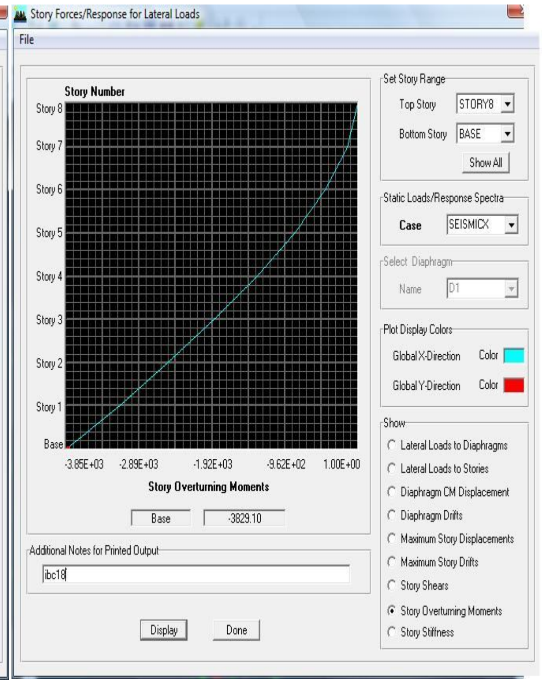
Fig. (4.10): Storey shears.

Table (4.9): Storey overturning moments M_{st} (kN.m).

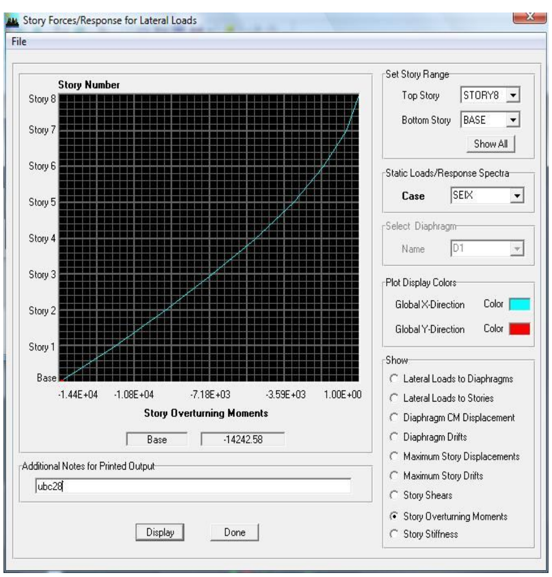
Height (m)	Storey overturning moments M_{st}(kN.m)						
	Storey No.	Case 1			Case 2		
		UBC 1	IBC 1	Difference %	UBC 2	IBC 2	Difference %
0	Base	3339.75	3839.54(G)	13.01	14203.66	16093.17(G)	11.74
4	1	2709.94	3119.44	13.12	1135.15	13119.25	11.9
7.5	2	2144.02	2472.39	13.28	9144.47	10407.73	12.13
11	3	1623.74	1877.53	13.51	7004.04	7914.88	11.5
14.5	4	1139.97	1319.7	13.61	4941.44	5378.3	8.12
18	5	701.84	802.59	12.55	3073.4	3410.27	9.87
21.5	6	373.24	416.45	10.37	1633.51	1748.37	6.56
25	7	126.79	124.24	2.01	582.75	480.08	17.61
28.5	8	8.13	9.44(GM)	13.87	37.92	42.73(GM)	11.25
		The biggest difference (13.87%)			The biggest difference (17.61%)		



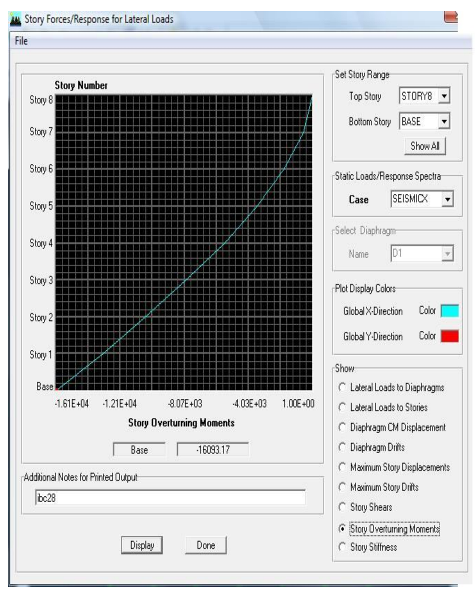
(a)UBC1



(b) IBC1



(c)UBC2



(d) IBC2

Fig. (4.11): Storey over turning moments.

Chapter Five

Conclusions and Recommendations

5.1Conclusions:

From this research and the results obtained it can be concluded that:

1. The provisions of IBC are more accurate for the site local conditions since:

   i. the ground motion parameters of IBC are more representative of the values of accelerations of site than it for zone factors of UBC.

   ii. to enhance the performance of the structure ,IBC provides beside the important factor(structure using factor) other two provisions, firstly, the tighter drift limitations for structure of higher occupancy and secondly, making the detailing level a function of (seismic risk at the site of a structure and the occupancy of structure).

   iii. for the ground motion parameters, the duration of returning period and probability of exceedance are more accurate for IBC.

   iv. R-values have more details in IBC than it in UBC, which with important factors(structure using factor), give forces that are larger for IBC than UBC for the same condition.

2. From the results of the analysis, in contrast to other parameters, the lateral loads to diaphragm (158.91 kN, UBC, 146.99 kN, IBC) and lateral loads to storeys (166.7kN,UBC,156.07kN,IBC) are larger in UBC than in IBC. Inspite of this, the deformation results of the diaphragm, the diaphragm CM (center of mass) displacements, are (the same in both two codes) and diaphragm drifts (0.0005136m, UBC, 0.0021512m, IBC) are not larger in UBC.

3. The values obtained for base shear (651.54kN, UBC,715.8kN, IBC), storey shears, overall drifts (0.05764m, UBC, 0.06669m, IBC), story drifts (0.05764m, UBC, 0.06669m, IBC), storey displacements (1.19m, UBC, 1.37m, IBC) and overturning moments (14203.66 kN.m, UBC,

16093.17kN.m) are all larger in IBC code than corresponding values in UBC code.

4. The building under consideration is safe because all the values of the analysis results obtained by either UBC or IBC are within the allowable values.
5. It is noticed that the top storey shear obtained by UBC code is greater than the corresponding value obtained by IBC code but IBC code offset this variation by value of greater overturning moment at the top storey.
6. All the corresponding values of the analysis results obtained by the two codes have very small variations or are the same except the base shears, storey shears and overturning moments. For case one (very low seismic sites in Sudan) the results in general are the same for the two codes.
7. For more safe R.C. Building the international building code IBC is better to be used in Sudan. However, the data for using IBC code is very rare. On the other hand, almost all studies carried out in Sudan are based on uniform building code UBC.

5.2 Recommendations:

5.2.1-From the results obtained in this research it is recommended to:

1. use IBC code in the analysis of R.C. building in Sudan.
2. use other lateral load resisting system ,e.g. beam frame system or core system to obtain larger factors of safety.
3. update Sudan seismic data continuously and compare IBC code with ASCE-7, 2010 code (more advanced seismic code) to find out if there are any differences between these other two codes.

5.2.2 For further studies in the field of structures subjected to earthquakes it is recommended to:

1. use other types of analysis, like time- history analysis, are to be used to evaluate these seismic codes.
2. use different types of structures resisting systems for different structures and different materials to compare the results of the above seismic codes.
3. base studies on IBC new versions and ASCE-7, 2010 rather than UBC1997, because the parameters used by first two codes are provided by advanced instruments and their provisions are have additional restrictions.

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Appendix A1: UBC Tables

Table (A1.1):Soil classification(UBC)

Soil profile type	Soil profile Name/General Description	Average Soil profile for top 100 feet(30 430)of soil profile		
		Shear wave Velocity, feet/second(m/s)	Standard Penetration Test for Cohesionless Soil layers	Undrained Shear strength
S_A	Hard Rock	> 5000 (1500)	_	-
S_B	Hard	2500 to 5000 (760 to 1500)		
S_C	Very Dense Soil and Soft Rock	1200 to 2500 (360 to 760)	> 50	> 2000 (100)
S_D	Stiff Soil profile	600 to 1200 (180 to 360)	15 to 50	1000 to 2000 (50 to 100)
S_E	Soft Soil profile	< 600 (180)	< 15	< 1000 (50)
S_F	Soil Requiring Site –specific Evaluation. See section 1629.3.1			

Table (A1.2): Importance Factor (1)(UBC)

Occupancy Category	Occupancy or functions of Structure	Seismic Important Factor (I)
1.Essential facilities	1.Group 1, Division 1Occupancies having surgery and emergency treatment areas Fire and police stations 2.Garages shelters for emergency vehicles and emergency aircraft structures and emergency _preparedness centers	1.25

	3.Aviation control towers 4.Structures and equipment in government communication centers other facilities required for emergency response 5.Standby power _generating equipment for category 1 facilities 6.Tanks or other structures containing housing or supporting water or other fire suppression material or equipment required for the protection of Category 1,2or 3structures	
Hazard facilities	1.Group H,Divisions 1,2,6 and 7 occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing supporting or containing quantities of toxic or explosive substances that, if contained with abuiling,would cause that building to be classified as a Group H,Division 1.2or 7 occupancy	1.25
Special occupancy structures	Group A, Division 1,2and 21Occupancies Buildings housing Group E, Divisions 1and 3Occupancies with a capacity greater than 300 students Buildings housing Group B occupancies used for college or adult education with a capacity greater than 500 students Group 1,Divisions 1and2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group1,Division 3 Occupancies All structures with an occupancy greater than 5000 persons Structures and equipment in power generating stations, and other public utility facilities not included in Category1or Category2 above, and required for continued operation	1.0
Standard	All structures housing occupancies or having functions	1.0

occupancy structures	not listed in category 1,2or3and Group U Occupancy towers	
Miscellaneous structures	Group U Occupancies except for towers	1.0

Table :(A1.3) Structure system factor R(UBC)

Basic structural system	Lateral force resisting system Description	Rω	Ω_o	Height Limit for seismic zones 3 and 4(feet) X304.8for mm
1.Bearing wall system	1.Light-frames with shear panels a. Wood structural panel walls for structures three stories or less b. All other light -framed walls 2.Shear walls a. Concrete b. Masonry 3. Light steel framed bearing walls with tension –only bracing 4.Braced frames where bracing carries gravity load a. Steel b. Concrete c. Heavy Timber	5.5 4.5 4.5 4.5 2.8 4.4 2.8 2.8	2.8 2.8 2.8 2.8 2.2 2.2 2.2 2.2	65(19812mm) 65(19812mm) 160(48768mm) 160(48768mm) 65(19812mm) 160(48768mm) – 65(19812mm)
2.Building frame system	1.Steel eccentrically braced frame EBF 2.Light –framed walls with shear panels a. Wood structural panel walls for structures three stories or less	7 6.5	2.8 2.8	240(73152mm) 65(19812mm)

	b. All other light –frame walls 3.Shear walls a. Concrete b. Masonry 4.Ordinary braced frames a. Steel b .Concrete c. Heavy Masonry 5.Special concentrically braced frames a. Steel	5 5.5 5.5 5.6	2.8 2.8 2.8 2.2	65(19812mm) 240(73152mm) 160(19812mm) 160(19812mm)
3.Moment –resisting frame system	1.Special moment –resisting frame(SMRF) a. Steel b. Concrete 2.Masonry moment resisting –wall frame(MMRF) 3.Concrete intermediate moment resisting frame(IMRF) 4.Ordinarymoment–resisting frame(OMRF) a. Steel b. Concrete 5.Special truss moment frames of steel(STMF)	 8.5 8.5 6.5 5.5 4.5 3.5 6.5	 2.8 2.8 2.8 2.8 2.8 2.8 2.8	 NL NL 160(19812mm) – 160(19812mm) – 240(73152mm)
4.Daul systems	1.Shear walls a. Concrete with SMRF b. Concrete with steel OMRF c. Concrete with concrete IMRF d .Masonry with steel SMRF e .Masonry with steel OMRF F. Masonry with IMRF	 8.5 4.2 6.5 5.5 4.2 4.2	 2.8 2.8 2.8 2.8 2.8 2.8	 NL 160(48768mm) 160(48768mm) 160(48768mm) 160(48768mm) _

	g. MASONRY WITH Masonry MMRWF 2.Steel EBF a. With Steel SMRF b. With Steel OMRF 3.Ordinary braced frames a. Steel with steel SMRF b. Steel OMRF c. Concrete with concrete SMRF d. Concrete with concrete IMRF 4.Special concentrically braced frames a. Steel with Steel SMRF b. Steel with Steel OMRF	6 8.5 4.2 6.5 4.2 6.5 4.2 7.5 4.2	2.8 2.8 2.8 2.8 2.8 2.8 2.8 2.8 2.8	160(48768mm) NL 160(48768mm) NL 160(48768mm) _ _ NL 160(48768mm)
5.Cantilevered column building systems	1.Cantilever column elements	2.2	2	35(10668mm)
6.Shear wall-frame interaction systems	1.Concrete	5.5	2.8	160(48768mm)
7.Undefined systems	Sections1629.6.7and 1629.9.2	_	_	_

NL-No limit

Appendix A2:IBC Tables

Table (A2.1) Site class definitions (IBC)

Site class	Soil profile name	Average properties in top 100 feet(30.48m)		
A	Hard rock	V, > 5,000	N/A	N/A
B	Rock	2,500 < v, ≤ 5,000	N/A	N/A
C	Very dense soil and soft rock	1,200 < v, ≤ 2,500	N > 50	S, ≥ 2<000
D	Stiff soil profile	600 ≤ v, ≤ 1,200	15 ≤ N ≤ 50	1,000 ≤ s, ≤ 2,000
E	Soft soil profile	V, < 600	N < 15	S, < 1,000
E	-	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index pI > 20, 2. Moisture content w ≥ 40%, and 3. Undrained shear strength s, < 500 paf		
F	-	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays (H > 10 feet of peat and/or highly organic clay were H = thickness of soil) 3. Very high plasticity clays (H > 25 feet with plasticity index PI > 75) 4. Very thick soft/medium stiff clays (H > 120 feet)		

For SI: I foot = 304.8 mm, I square foot = 0.0929 m^2 , 1 pound per square foot = 0.0479 kPa. N/A = Not applicable

Table (A2.2) Occupancy of buildings and other structures(IBC)

Occupancy Category	Nature of occupancy
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Occupancy Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Covered structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structure with elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structure with occupant load greater than 500 for colleges or adult education facilities. • Health care facilities with an occupant load of 50 or more resident, but not having surgery or emergency treatment facilities. • Jails and detention facilities. • Any other occupancy with an occupant load greater than 5,000. • Power-generating stations, water treatment for potable

	water, waste, water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.
IV	Building and other structures designed as essential facilities, including but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities. • Fire, rescue and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communication, and p\operation centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials as defined Section 307 where the quantity of the material exceeds the maximum allowable quantities o Table 307.1.(2). • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water treatment facilities required to maintain water pressure for fire suppression.

Table(A2.3)Importance factor by risk category(IBC)

Risk category	Seismic Importance factor I
I	1.0
II	1.0
III	1.25
IV	1.5

Table (A2.4):Risk category factor (IBC)

Risk category of buildings and other structures	Risk category
Buildings and other structures that represent a low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I,III and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life. Buildings and other structures ,not included in Risk Category IV ,with potential to cause a substantial economic impact and /or mass disruption of day-to-day civilian life in the event of failure. Buildings and other structures not included in Risk Category IV (including, but not limited to ,facilities manufacture, process ,handle ,store ,use ,or dispose of such substances as hazardous fuels ,hazardous chemicals ,hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and sufficient to pose a threat to	III

the public if released.	
Buildings and other structures designated as essential facilities. Buildings and other structures, the failure of which could pose a substantial hazard to community. Buildings and other structures (including ,but not limited to ,facilities that manufacture ,process ,handle ,store ,use ,or dispose of such substantial as hazardous fuels ,hazardous chemicals ,or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be the dangerous to the public if sufficient to pose a threat to the public if released. Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	IV

Table (A2.5), Values of approximate period parameters(IBC).

Structure type		
Moment -resisting frame system in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces.	C_t	x
Steel moment –resisting frames	0.0724(m)	0.8
Concrete moment-resisting frames	0.0466	0.9
Steel eccentrically braced frames in accordance with Table 12.2.1 lines B1 or D1	0.0731	0.75
Steel buckling –restrained braced frames	0.0731	0.75
All other structural frames	0.0488	0.75

Table (A2.6) Coefficients of upper limit on calculated period(IBC)

Design Spectral Response Acceleration parameter at 1-sec., S_{D1}	Coefficient C_u
≥0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤0.1	1.7

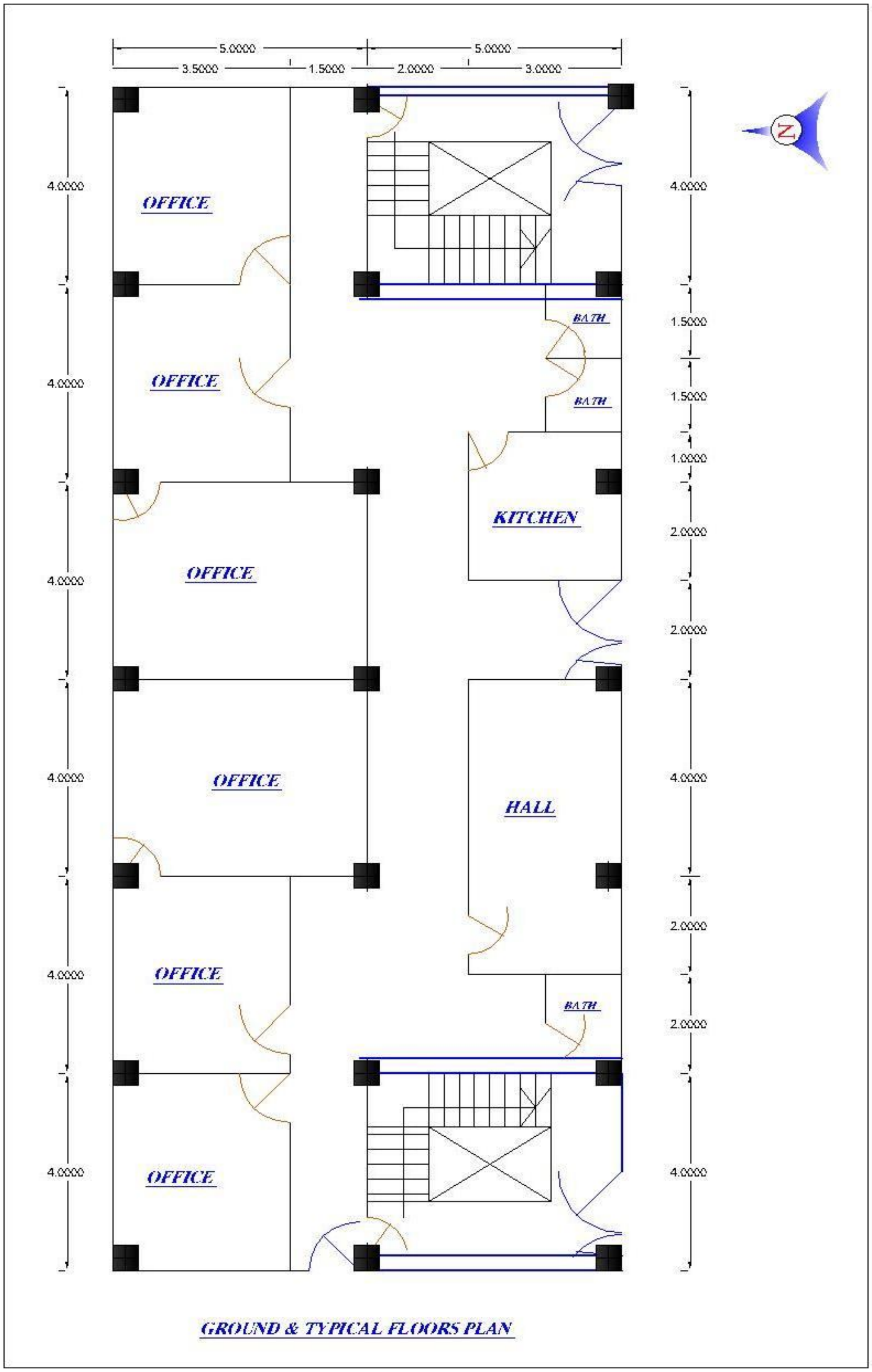
Table (A2.7):Seismic-Force-Resisting Systems of Concrete(IBC)

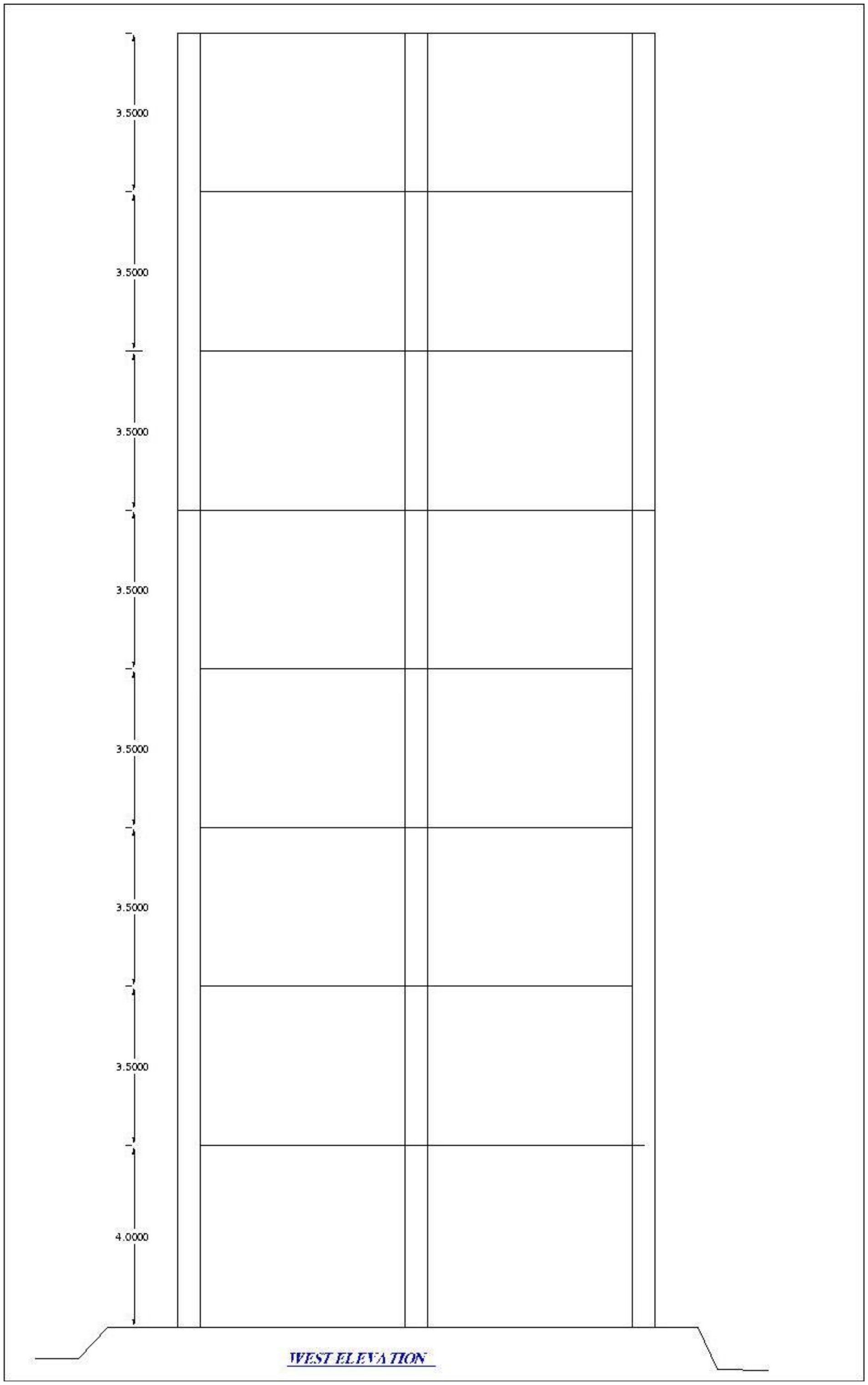
Basic Seismic-Force-Resisting System	Detailing Reference Section	R	O_o	C_d	System Limitations and Building Height Limitations (ft) by SDC*				
					A or B	C	D	E	F
Bearing Wall Systems									
Special reinforced concrete Shear walls	1910.2.4	5½	2½	5	NL	NL	160	160	160
Ordinary reinforced concrete shear walls	1910.2.3	4½	2½	4	NL	NL	NP	NP	NP
Detailing plain concrete shear walls	1910.2.2	2½	2½	2	NL	NP	NP	NP	NP
Ordinary plain concrete shear walls	1910.2.1	1½	2½	1½	NL	NP	NP	NP	NP
Building Frame System									
Special reinforced concrete Shear walls	1910.2.4	6	2½	5	NL	NL	160	160	100
Ordinary reinforced concrete shear walls	1910.2.3	5	2½	4½	NL	NL	NP	NP	NP
Detailing plain concrete shear walls	1910.2.2	3	2½	2½	NL	NP	NP	NP	NP
Ordinary plain concrete shear	1910.2.	2	2½	2	NL	NP	NP	NP	NP

walls	1								
Moment-resisting Frame Systems									
Special reinforced concrete moment frames	ACI 21.1	8	3	5½	NL	NL	NL	NL	NL
Intermediate reinforced concrete moment frames	ACI 21.1	5	3	4½	NL	NL	NP	NP	NP
Ordinary reinforced concrete moment frames	ACI 21.1	3	3	2½	NL	NP	NP	NP	NP
Dual Systems with Special Moment Frames									
Special reinforced concrete Shear walls	1910.2.4	8	2½	6½	NL	NL	NL	NL	NL
Ordinary reinforced concrete shear walls	1910.2.3	7	2½	6	NL	NL	NP	NP	NP
Dual System with Intermediate Moment Frames									
Special reinforced concrete Shear walls	1910.2.4	6	2½	5	NL	NL	160	160	100
Ordinary reinforced concrete shear walls	1910.2.3	5½	2½	4½	NL	NL	NP	NP	NP
Shear wall-frame interactive system with ordinary i\reinforced concrete moments frames and ordinary reinforced concrete shear walls	ACI 21.1 1910.2.3	5½	2½	5	NL	NP	NP	NP	NP
Inverted Pendulum Systems									
Special reinforced concrete moment frames	ACI 21.1	2½	2	1½	NL	NL	NL	NL	NL

*NL = not limited, NP = not permitted

Appendix B: Samples of input and Results




3.5000
3.5000
3.5000
3.5000
3.5000
3.5000
3.5000
4.0000
WEST ELEVATION

Country, City	S_S	S_1	Country, City	S_S	S_1	Country, City	S_S	S_1
AFRICA	------	------	**Kenya**	------	------	**South Africa**	------	------
Algeria	------	------	Nairobi	0.62	0.28	Cape Town	1.24	0.56
Alger	1.24	0.56	**Lesotho**	------	------	Durban	0.62	0.28
Oran	1.24	0.56	Maseru	0.62	0.28	Johannesburg	0.62	0.28
Angola	------	------	**Liberia**	------	------	Natal	0.31	0.14
Luanda	0.06	0.06	Monrovia	0.31	0.14	Pretoria	0.62	0.28
Benin	------	------	**Libya**	------	------	**Swaziland**	------	------
Cotonou	0.06	0.06	Tripoli	0.62	0.28	Mbabane	0.62	0.28
Botswana	------	------	Wheelus AFB	0.62	0.28	**Tanzania**	------	------
Gaborone	0.06	0.06	**Malagasy Republic**	------	------	Dar es Salaam	0.62	0.28
Burundi	------	------	Tananarive	0.06	0.06	Zanzibar	0.62	0.28
Bujumbura	1.24	0.56	**Malawi**	------	------	**Togo**	------	------
Cameroon	------	------	Blantyre	1.24	0.56	Lome	0.31	0.14
Douala	0.06	0.06	Lilongwe	1.24	0.56	**Tunisia**	------	------
Yaounde	0.06	0.06	Zomba	1.24	0.56	Tunis	1.24	0.56
Cape Verde	------	------	**Mali**	------	------	**Uganda**	------	------
Praia	0.06	0.06	Bamako	0.06	0.06	Kampala	0.62	0.28
Central African Republic	------	------	**Mauritania**	------	------	**Upper Volta**	------	------
Bangui	0.06	0.06	Nouakchott	0.06	0.06	Ougadougou	0.06	0.06
Chad	------	------	**Mauritius**	------	------	**Zaire**	------	------
Ndjamena	0.06	0.06	Port Louis	0.06	0.06	Bukavu	1.24	0.56
Congo	------	------	**Morocco**	------	------	Kinshasa	0.06	0.06
Brazzaville	0.06	0.06	Casablanca	0.62	0.28	Lubumbashi	0.62	0.28
Djibouti	------	------	Port Lyautey	0.31	0.14	**Zambia**	------	------
Djibouti	1.24	0.56	Rabat	0.62	0.28	Lusaka	0.62	0.28
Egypt	------	------	Tangier	1.24	0.56	**Zimbabwe**	------	------
Alexandria	0.62	0.28	**Mozambique**	------	------	Harare	1.24	0.56
Cairo	0.62	0.28	Maputo	0.62	0.28	**ASIA**	------	------
Port Said	0.62	0.28	**Niger**	------	------	**Afghanistan**	------	------
Equatorial Guinea	------	------	Niamey	0.06	0.06	Kabul	1.65	0.75
Malabo	0.06	0.06	**Nigeria**	------	------	**Bahrain**	------	------
Ethiopia	------	------	Ibadan	0.06	0.06	Manama	0.06	0.06
Addis Ababa	1.24	0.56	Kaduna	0.06	0.06	**Bangladesh**	------	------
Asmara	1.24	0.56	Lagos	0.06	0.06	Dacca	1.24	0.56
Gabon	------	------	**Republic of Rwanda**	------	------	**Brunei**	------	------
Libreville	0.06	0.06	Kigali	1.24	0.56	Bandar Seri Begawan	0.31	0.14
Gambia	------	------	**Senegal**	------	------	**Burma**	------	------
Banjul	0.06	0.06	Dakar	0.06	0.06	Mandalay	1.24	0.56
Guinea	------	------	**Seychelles**	------	------	Rangoon	1.24	0.56
Bissau	0.31	0.14	Victoria	0.06	0.06	**China**	------	------
Conakry	0.06	0.06	**Sierra Leone**	------	------	Canton	0.62	0.28
Ivory Coast	------	------	Freetown	0.06	0.06	Chengdu	1.24	0.56
Abidjan	0.06	0.06	**Somalia**	------	------	Nanking	0.62	0.28
------------------------------	------	------	Mogadishu	0.06	0.06	Peking	1.65	0.75

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Country, City	S_S	S_1
ASIA	------	------
China	------	------
Shanghai	0.62	0.28
Shengyang	1.65	0.75
Tibwa	1.65	0.75
Tsingtao	1.24	0.56
Wuhan	0.62	0.28
Cyprus	------	------
Nicosia	1.24	0.56
Hong Kong	------	------
Hong Kong	0.62	0.28
India	------	------
Bombay	1.24	0.56
Calcutta	0.62	0.28
Madras	0.31	0.14
New Delhi	1.24	0.56
Indonesia	------	------
Bandung	1.65	0.75
Jakarta	1.65	0.75
Medan	1.24	0.56
Surabaya	1.65	0.75
Iran	------	------
Isfahan	1.24	0.56
Shiraz	1.24	0.56
Tabriz	1.65	0.75
Tehran	1.65	0.75
Iraq	------	------
Baghdad	1.24	0.56
Basra	0.31	0.14
Israel	------	------
Haifa	1.24	0.56
Jerusalem	1.24	0.56
Tel Aviv	1.24	0.56
Japan	------	------
Fukuoka	1.24	0.56
Itazuke AFB	1.24	0.56
Misawa AFB	1.24	0.56
Naha, Okinawa	1.65	0.75
Osaka/Kobe	1.65	0.75
Sapporo	1.24	0.56
Tokyo	1.65	0.75
Wakkanai	1.24	0.56
Yokohama	1.65	0.75
Yokota	1.65	0.75

Country, City	S_S	S_1
Jordan	------	------
Amman	1.24	0.56
Korea	------	------
Kwangju	0.31	0.14
Kimhae	0.31	0.14
Pusan	0.31	0.14
Seoul	0.06	0.06
Kuwait	------	------
Kuwait	0.31	0.14
Laos	------	------
Vientiane	0.31	0.14
Lebanon	------	------
Beirut	1.24	0.56
Malaysia	------	------
Kuala Lumpur	0.31	0.14
Nepal	------	------
Kathmandu	1.65	0.75
Oman	------	------
Muscat	0.62	0.28
Pakistan	------	------
Islamabad	1.68	0.65
Karachi	1.65	0.75
Lahore	0.62	0.28
Peshawar	1.65	0.75
Quatar	------	------
Doha	0.06	0.06
Saudi Arabia	------	------
Al Batin	0.31	0.14
Dhahran	0.31	0.14
Jiddah	0.62	0.28
Khamis Mushayt	0.31	0.14
Riyadh	0.06	0.06
Singapore	------	------
All Locations	0.31	0.14
South Yemen	------	------
Aden City	1.24	0.56
Sri Lanka	------	------
Colombo	0.06	0.06
Syria	------	------
Aleppo	1.24	0.56
Damascus	1.24	0.56
Taiwan	------	------
All Locations	1.65	0.75
------------------	------	------

Country, City	S_S	S_1
Thailand	------	------
Bangkok	0.31	0.14
Chiang Mai	0.62	0.28
Songkhla	0.06	0.06
Udom	0.31	0.14
Turkey	------	------
Adana	0.62	0.28
Ankara	0.62	0.28
Istanbul	1.65	0.75
Izmir	1.65	0.75
Karamursel	1.24	0.56
United Arab Emirates	------	------
Abu Dhabi	0.06	0.06
Dubai	0.06	0.06
Viet Nam	------	------
Ho Chi Min City	0.06	0.06
Yemen Arab Republic	------	------
Sanaa	1.24	0.56
ATLANTIC OCEAN AREA	------	------
Azorea	------	------
All Locations	0.62	0.28
Bermuda	------	------
All Locations	0.31	0.14
CARIBBEAN SEA	------	------
Bahama Islands	------	------
All Locations	0.31	0.14
Cuba	------	------
All Locations	0.62	0.28
Dominican Republic	------	------
Santo Domingo	1.24	0.56
French West Indies	------	------
Martinique	1.24	0.56
Grenada	------	------
Saint Georges	1.24	0.56
Haiti	------	------
Port au Prince	1.24	0.56
Jamaica	------	------
Kingston	1.24	0.56
Leeward Islands	------	------
All Locations	1.24	0.56
Puerto Rico	------	------
All Locations	0.83	0.38
Trinidad & Tobago	------	------
All Locations	1.24	0.56

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١٩ ديسمبر ، ٢٠١٣ ١٦:٠٢ ETABS v9.7.4 File:IBC(2) ABU Units:KN-m
PAGE 1
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STORY7 STORY8 3.500 25.000
STORY6 STORY8 3.500 21.500
STORY5 STORY8 3.500 18.000
STORY4 STORY8 3.500 14.500
STORY3 STORY8 3.500 11.000
STORY2 STORY8 3.500 7.500
STORY1 STORY8 4.000 4.000
BASE None 0.000
١٩ ديسمبر ، ٢٠١٣ ١٦:٠٢ ETABS v9.7.4 File:IBC(2) ABU Units:KN-m
PAGE 2
S T A T I C L O A D C A S E S
STATIC CASE AUTO LAT SELF WT NOTIONAL
NOTIONAL
CASE TYPE LOAD MULTIPLIER FACTOR
DIRECTION
DEAD DEAD N/A 1.0000
LIVE LIVE N/A 0.0000
SEISMICX QUAKE IBC2006 0.0000
SEISMICY QUAKE IBC2006 0.0000
SUPDD SUPER DEAD N/A 0.0000
١٩ ديسمبر ، ٢٠١٣ ١٦:٠٢ ETABS v9.7.4 File:IBC(2) ABU Units:KN-m
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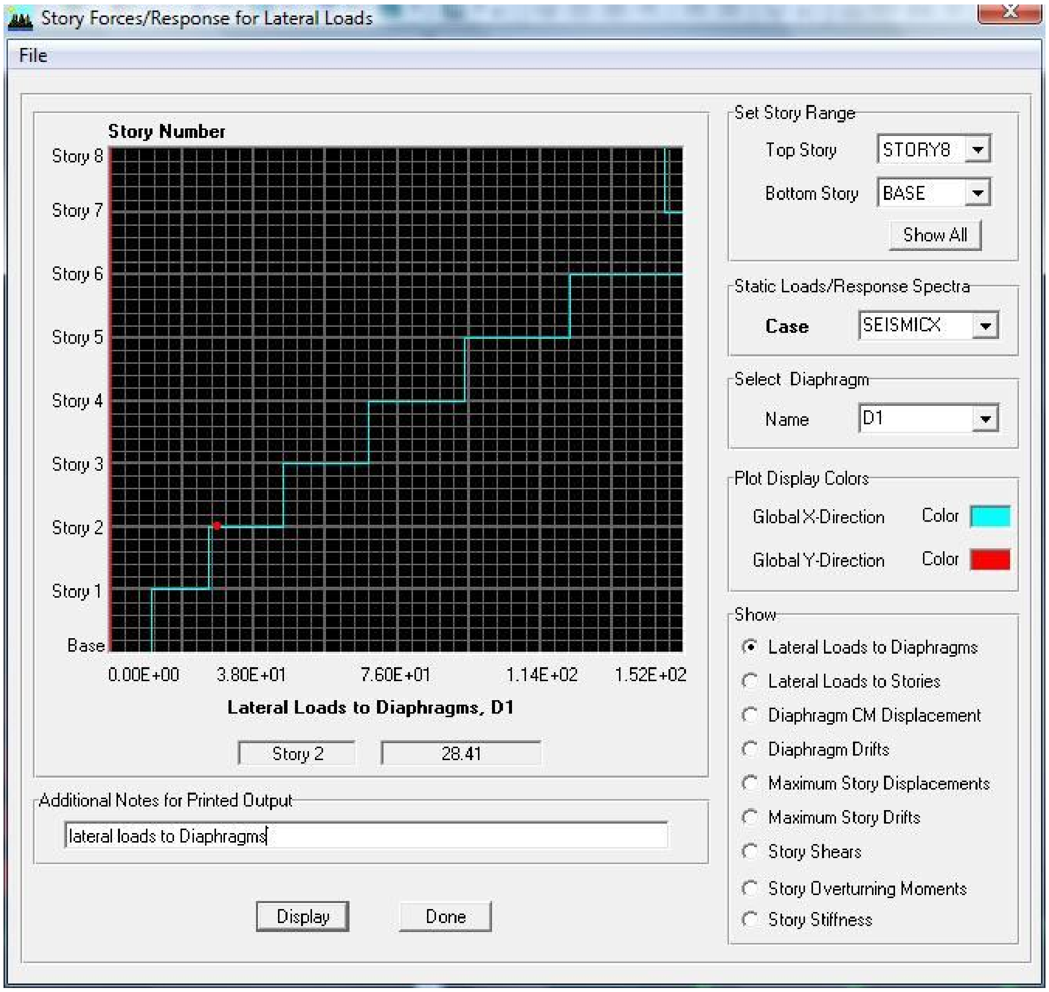
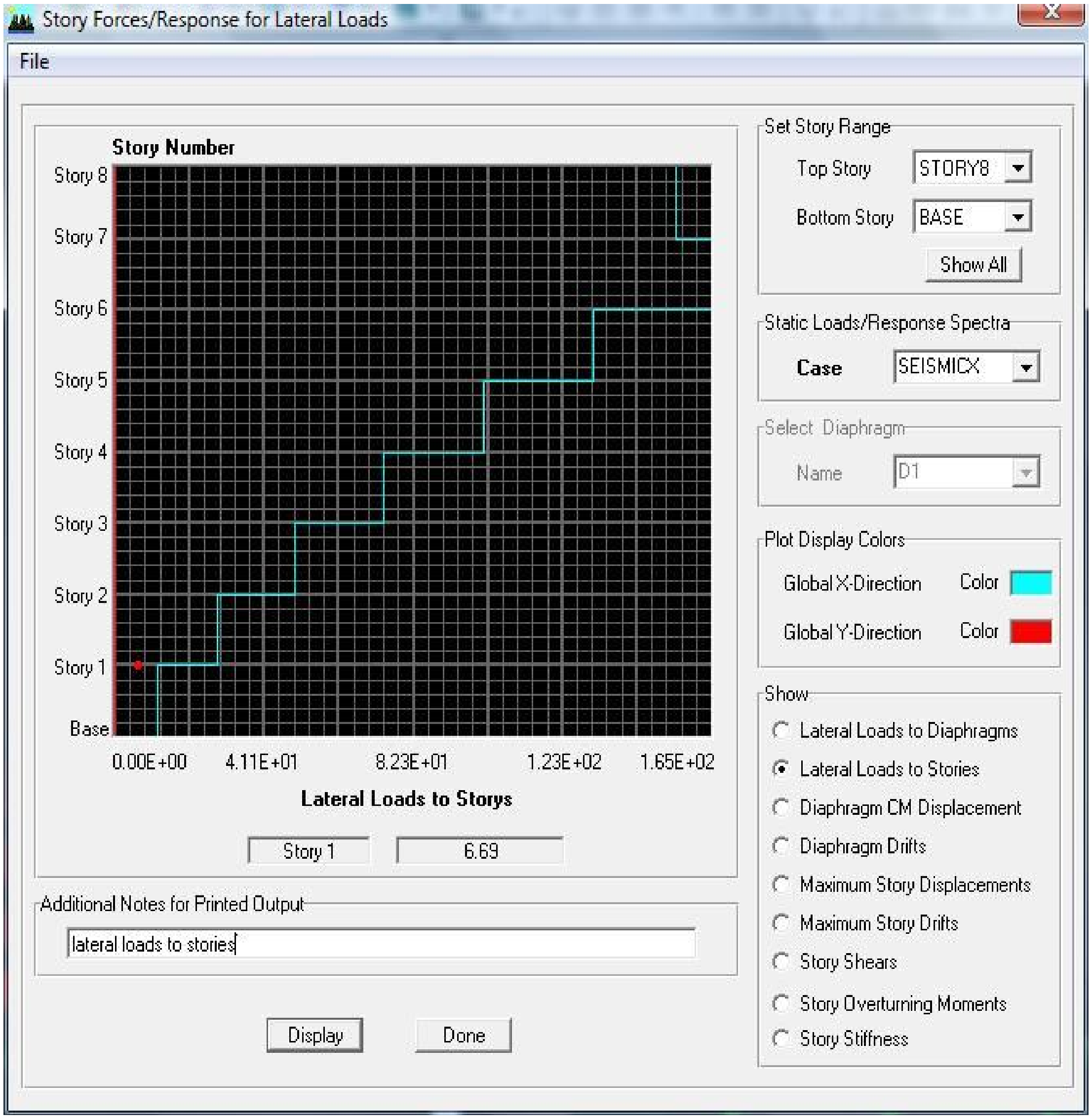
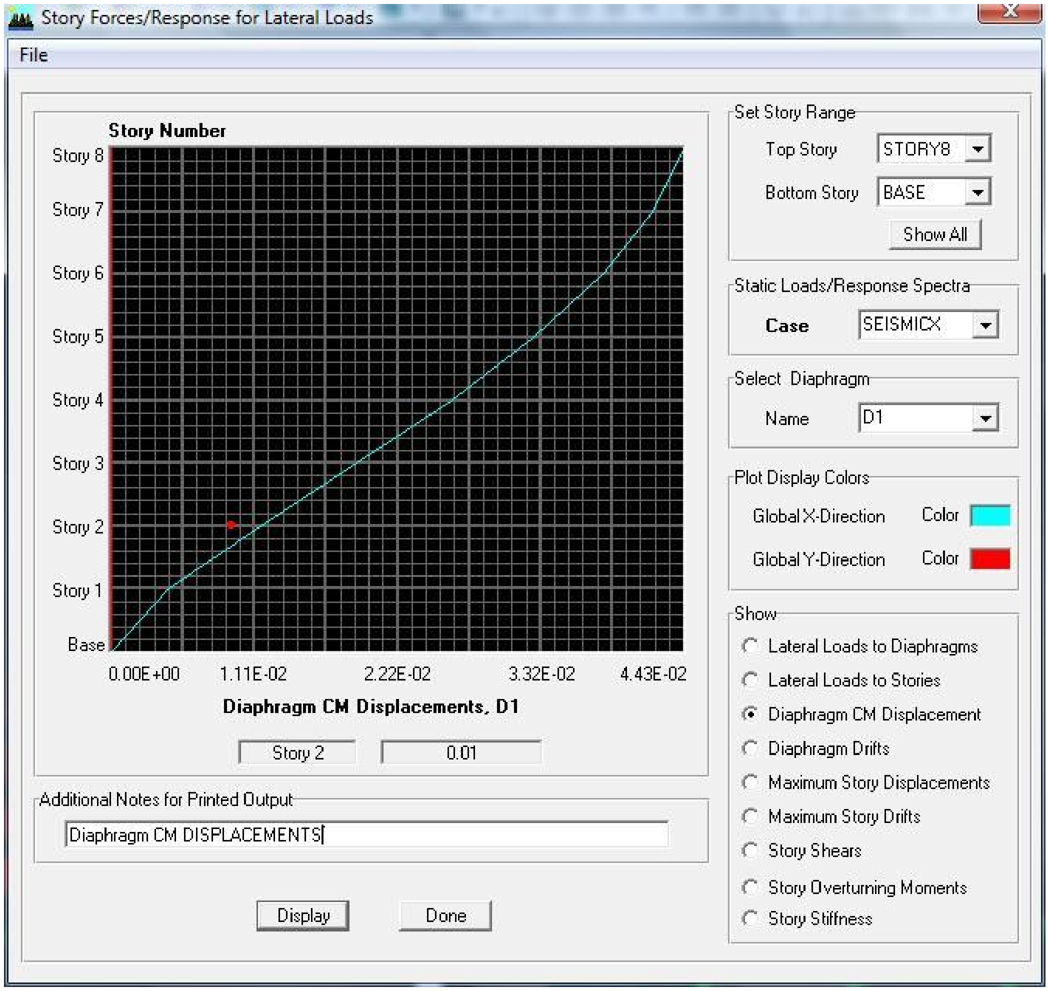
RESP SPEC CASE: SPEC1
BASIC RESPONSE SPECTRUM DATA
MODAL DIRECTION MODAL SPECTRUM TYPICAL
COMBO COMBO DAMPING ANGLE ECCEN
CQC SRSS 0.0500 0.0000 0.0500
RESPONSE SPECTRUM FUNCTION ASSIGNMENT DATA
DIRECTION FUNCTION SCALE FACT
U1 IBC2006 1.0000
U2 ---- N/A
UZ ---- N/A
RESP SPEC CASE: SPEC2
BASIC RESPONSE SPECTRUM DATA
MODAL DIRECTION MODAL SPECTRUM TYPICAL
COMBO COMBO DAMPING ANGLE ECCEN

Appendix C : Samples of large scale figures for Results




Story Forces/Response for Lateral Loads
File
Story Number
Story 8
Story 7
Story 6
Story 5
Story 4
Story 3
Story 2
Story 1
Base
0.00E+00
4.11E+01
8.23E+01
1.23E+02
1.65E+02
Lateral Loads to Storys
Story 1
6.69
Additional Notes for Printed Output
lateral loads to stories
Display
Done
Set Story Range
Top Story
STORY8
Bottom Story
BASE
Show All
Static Loads/Response Spectra
Case
SEISMICX
Select Diaphragm
Name
D1
Plot Display Colors
Global X-Direction
Color
Global Y-Direction
Color
Show
Lateral Loads to Diaphragms
Lateral Loads to Stories
Diaphragm CM Displacement
Diaphragm Drifts
Maximum Story Displacements
Maximum Story Drifts
Story Shears
Story Overturning Moments
Story Stiffness


Story Forces/Response for Lateral Loads
File
Story Number
Story 8
Story 7
Story 6
Story 5
Story 4
Story 3
Story 2
Story 1
Base
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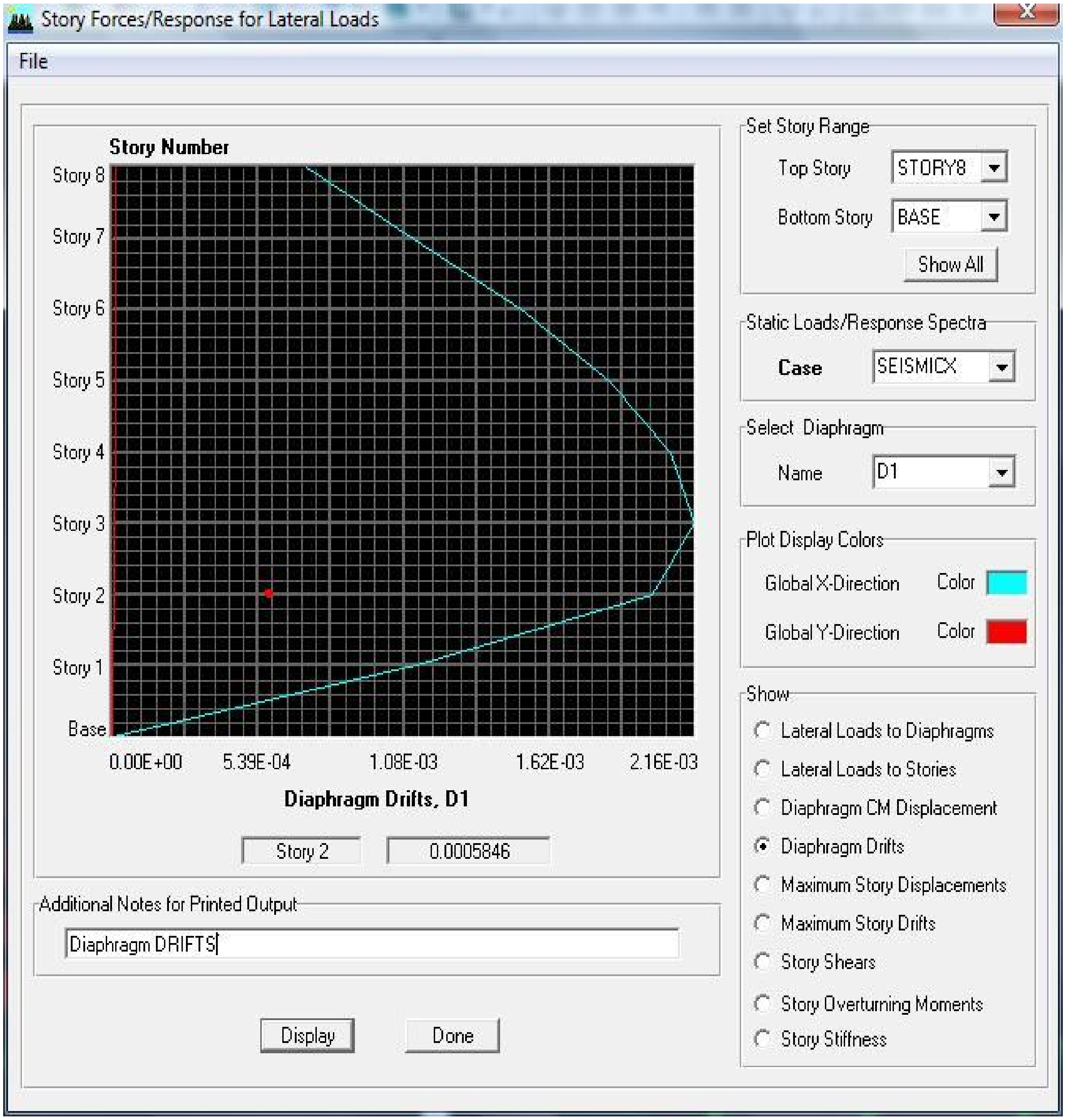
1.11E-02
2.22E-02
3.32E-02
4.43E-02
Diaphragm CM Displacements, D1
Story 2
0.01
Additional Notes for Printed Output
Diaphragm CM DISPLACEMENTS
Display
Done
Set Story Range
Top Story
STORY8
Bottom Story
BASE
Show All
Static Loads/Response Spectra
Case
SEISMICX
Select Diaphragm
Name
D1
Plot Display Colors
Global X-Direction
Color
Global Y-Direction
Color
Show
Lateral Loads to Diaphragms
Lateral Loads to Stories
Diaphragm CM Displacement
Diaphragm Drifts
Maximum Story Displacements
Maximum Story Drifts
Story Shears
Story Overturning Moments
Story Stiffness


Story Forces/Response for Lateral Loads
File
Story Number
Story 8
Story 7
Story 6
Story 5
Story 4
Story 3
Story 2
Story 1
Base
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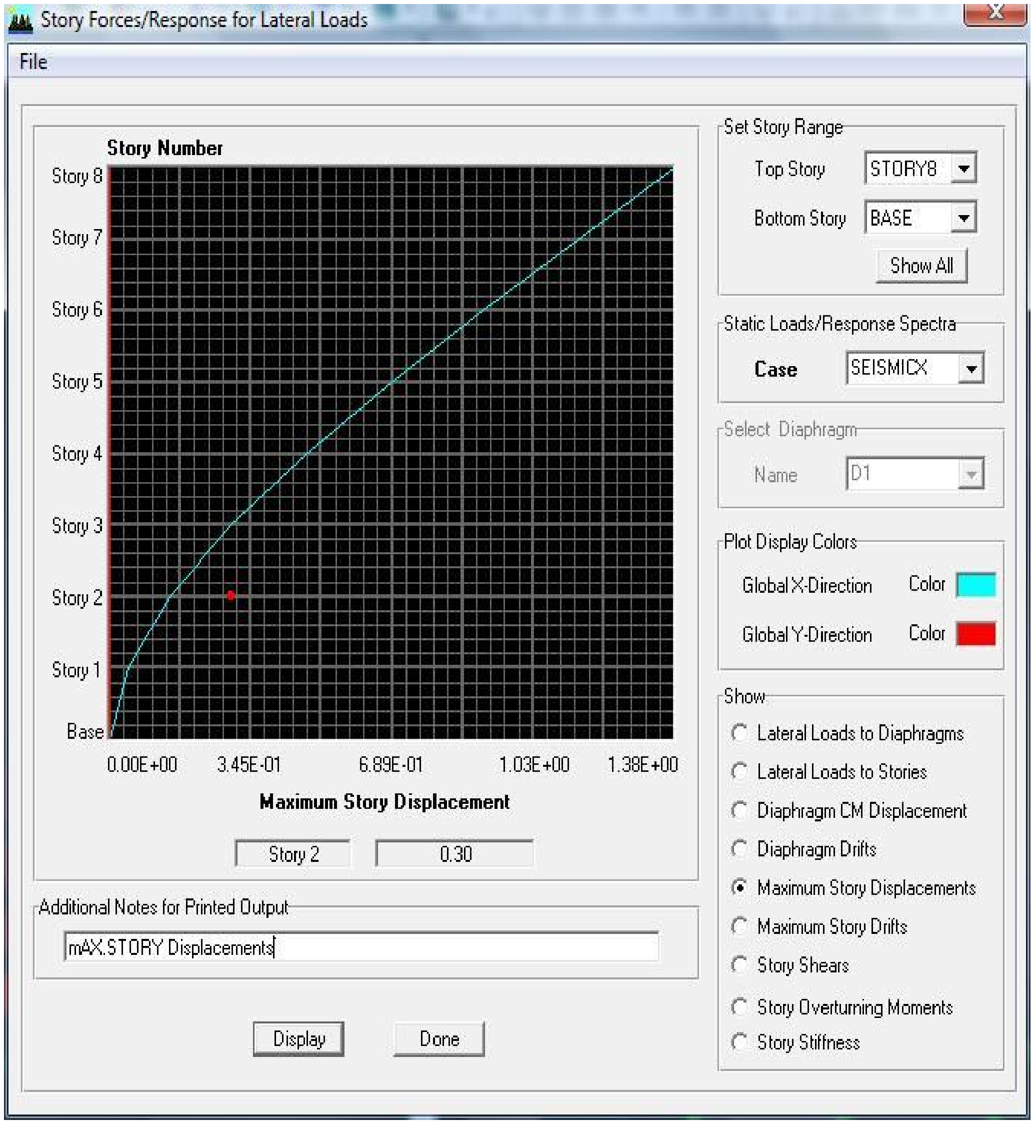
5.39E-04
1.08E-03
1.62E-03
2.16E-03
Diaphragm Drifts, D1
Story 2
0.0005846
Additional Notes for Printed Output
Diaphragm DRIFTS
Display
Done
Set Story Range
Top Story
STORY8
Bottom Story
BASE
Show All
Static Loads/Response Spectra
Case
SEISMICX
Select Diaphragm
Name
D1
Plot Display Colors
Global X-Direction
Color
Global Y-Direction
Color
Show
Lateral Loads to Diaphragms
Lateral Loads to Stories
Diaphragm CM Displacement
Diaphragm Drifts
Maximum Story Displacements
Maximum Story Drifts
Story Shears
Story Overturning Moments
Story Stiffness


Story Forces/Response for Lateral Loads
File
Story Number
Story 8
Story 7
Story 6
Story 5
Story 4
Story 3
Story 2
Story 1
Base
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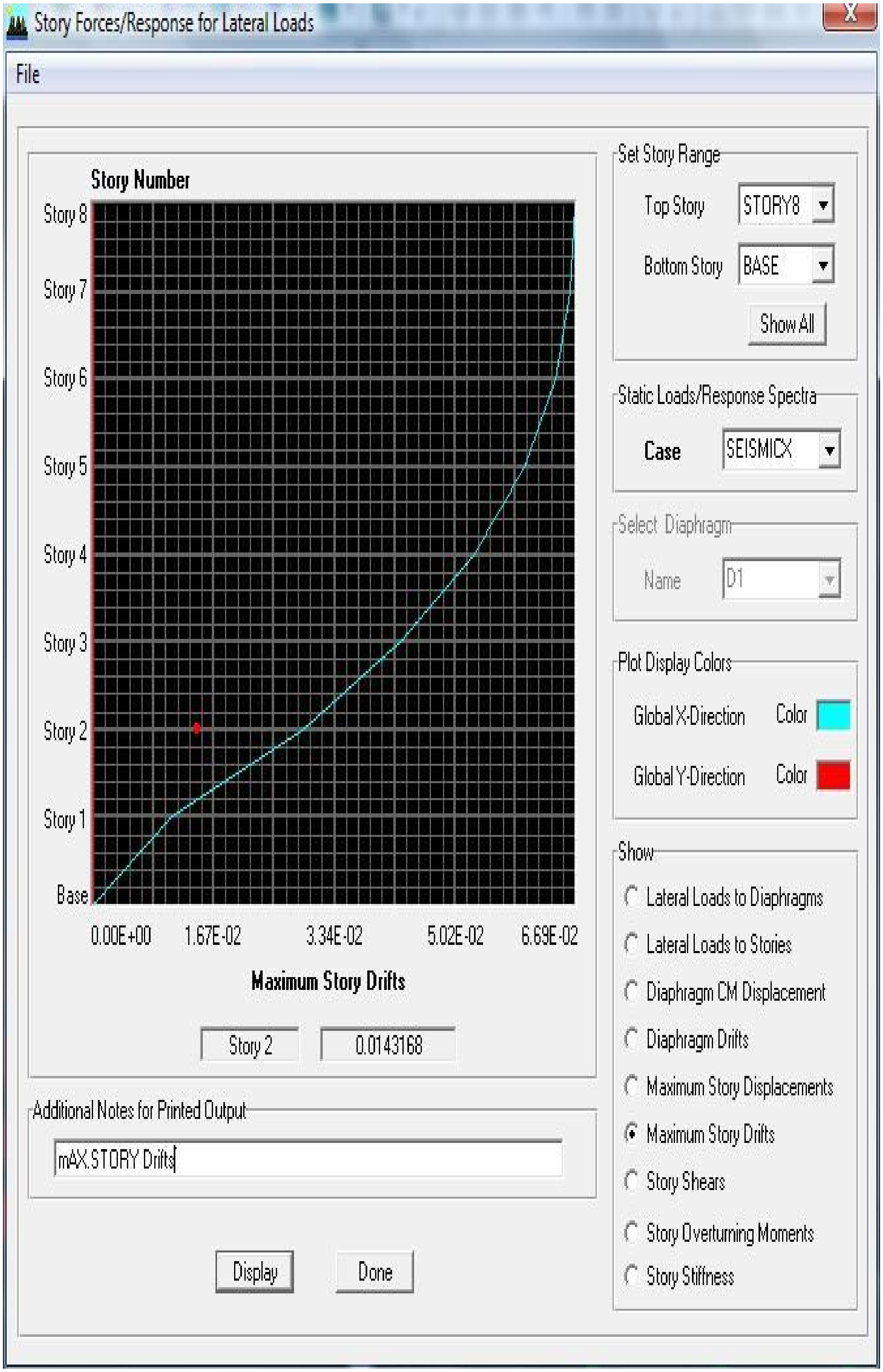
3.45E-01
6.89E-01
1.03E+00
1.38E+00
Maximum Story Displacement
Story 2
0.30
Additional Notes for Printed Output
mAX.STORY Displacements
Display
Done
Set Story Range
Top Story
STORY8
Bottom Story
BASE
Show All
Static Loads/Response Spectra
Case
SEISMICX
Select Diaphragm
Name
D1
Plot Display Colors
Global X-Direction
Color
Global Y-Direction
Color
Show
Lateral Loads to Diaphragms
Lateral Loads to Stories
Diaphragm CM Displacement
Diaphragm Drifts
Maximum Story Displacements
Maximum Story Drifts
Story Shears
Story Overturning Moments
Story Stiffness


Story Forces/Response for Lateral Loads
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Story Number
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Story 7
Story 6
Story 5
Story 4
Story 3
Story 2
Story 1
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1.67E-02
3.34E-02
5.02E-02
6.69E-02
Maximum Story Drifts
Story 2
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Additional Notes for Printed Output
mAX.STORY Drifts
Display
Done
Set Story Range
Top Story
STORY8
Bottom Story
BASE
Show All
Static Loads/Response Spectra
Case
SEISMICX
Select Diaphragm
Name
D1
Plot Display Colors
Global X-Direction
Color
Global Y-Direction
Color
Show
Lateral Loads to Diaphragms
Lateral Loads to Stories
Diaphragm CM Displacement
Diaphragm Drifts
Maximum Story Displacements
Maximum Story Drifts
Story Shears
Story Overturning Moments
Story Stiffness