Comparison between the American code ASCE7-16 and the Australian code AS1170.4 against the seismic design effects

مقارنة بين الكود الامريكي و الكود الاسترالي في تصميم الزلازل

by

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ABSTRACT

“Engineering is a professional art of applying science to the efficient conversion of natural resources for the benefit of man. Engineering, therefore, requires above all creative imagination to the innovative, useful application for natural phenomenon.” (International Journal of Engineering Research & Technology (IJERT), Vol. 3 Issue 3, March – 2014). This work implemented to compare two codes, American and Australian by designing a sixty -floors building using reinforced concrete structure from the economic point of view. Two critical structural codes adopted, ASCE-7 16 and AS1170. These codes compared in term of strength design necessities of structure elements and the shear design included as well.

During this study, calibration and elaboration of the models adopted along with the criteria consecration revealed. Though the main principles are the same and the seismic zone will moderate, the details were different and made the comparison accurately, the analysis implemented via ETABs software from CSI Company established in the USA.

This work will show the development of the Australian code especially for the earthquake loading using the AS1170.4, 2007. The Australian engineers established new response spectrum design contains a better illustration of the acceleration, displacement and velocity for the type of site soil (soft and rock). The used procedure to establish the spectrum could use in another country which has a low/ moderate seismicity. The AS1170.4 used a tiered approach for the earthquake load starting from simple force to complicated displacement method. One of the significant advantages of this method especially on the low seismic zone is given the engineers to design against the vertical and wind loads then perform the displacement checking for the earthquake impact.

In parallel, the ASCE-7 16 deliver the all the possible design requirements for general structures along with the dead, live, snow, flood, rain, soil, wind load, and atmospheric ice with the proper combinations which is fit for the building code. The standard is a revision of ASCE-7 10 which is a revision of ASCE-7 5 respectively and it's providing a full update and re-arranged of the wind load. Also, it has a new ultimate wind load maps along with the reduced load factors and it has an update on the risk targeted seismic maps. This standard is a comprehensive and important version as a part of the building codes which are used in the United States.

ملخص

"الهندسة هي فن احترافي لتطبيق العلم على التحويل الفعال للموارد الطبيعية لصالح الإنسان. لذا ، تتطلب الهندسة ، قبل كل شيء ، الإبداع المبتكر للتطبيق المفيد المبتكر للظاهرة الطبيعية".

تم تنفيذ هذا العمل لمقارنة الكودين الأمريكي والأسترالي من خلال تصميم مبنى مكون من ستين طابقاً باستخدام بنية خرسانية مسلحة من وجهة النظر الاقتصادية بلاعتماد على اشهرهما لعام 2007 لاستراليا و 2016 لامريكا و مقارنة هذه الرموز في شروط تصميم القوة من عناصر الهيكل وتصميم القص كذلك.

خلال هذه الدراسة ، اعتمدت نفس المعايرات و الاعدادات في نمدجة المبنى و تم اختيار نفس المنطة الزلزالية لاظهار نتايج مختلفة و تفاصيل مختلفة . سيُظهر هذا العمل تطوير الكود الأسترالي خاصةً فيما يتعلق بالزلازل الذي يتم تحميله باستخدام اصدار 2007 حيث أقام المهندسون الأستراليون تصميمًا جديدًا لاستجابة الطيف يحتوي على توضيح أفضل للتسارع والنزوح والسرعة لنوع تربة الموقع (لينة وصخرية). الإجراء المستخدم لإنشاء الطيف في بلد آخر له زلزالية منخفضة / معتدلة.

استخدم الكود الاسترالي أسلوبًا متدرجًا لحمل الزلازل بدءًا من القوة البسيطة إلى طريقة الإزاحة المعقدة. أحد المزايا الهامة لهذه الطريقة خاصة في المنطقة المنخفضة من الزلازل يتم منح المهندسين تصميمها مقابل الأحمال الرأسية والريحية ثم إجراء فحص الإزاحة للتأثير على الزلزال.

وبالتوازي مع ذلك ، يوفر الكود الأميريكي جميع متطلبات التصميم الممكنة للهياكل العامة إلى جانب الاحمال الميت والحيوان والثلج والفيضانات والمطر والتربة وحمل الرياح مع التركيبات المناسبة التي تناسب حالة المبنى . هذه المواصفة القياسية ، هي تعديل ل 2010 و2005 على التوالي وهي توفر تحديثًا كاملًا وإعادة ترتيب حمل الرياح.

أيضا ، لديها خرائط حمولة الرياح النهائية الجديدة جنبا إلى جنب مع انخفاض عوامل الحمولة ولديها تحديثا على الخرائط الزلزالية المستهدفة المخاطر. هذه المواصفة هي نسخة شاملة ومهمة كجزء من رموز البناء المستخدمة في الولايات المتحدة

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T_1 = the fundamental period of the structure in the direction under consideration

T_{1D} = effective period, in seconds, of the fundamental mode of vibration of the structure at the design displacement in the direction under consideration, as prescribed by Section 18.4.2.5 or 18.5.2.5

T_{1M} = effective period, in seconds, of the fundamental mode of vibration of the structure at the maximum displacement in the direction under consideration, as prescribed by Section 18.4.2.5 or 18.5.2.5

T_R = period, in seconds, of the residual mode of vibration of the structure in the direction under consideration, Section 18.5.2.7

V_m = design value of the seismic base shear of the m^{th} mode of vibration of the structure in the direction of interest, Section 18.4.2.2

V_{min} = minimum allowable value of base shear permitted for design of the seismic force-resisting system of the structure in the direction of interest, Section 18.2.2.1

V_R = design value of the seismic base shear of the residual mode of vibration of the structure in a given direction, as determined in Section 18.5.2.6

$\bar{W}_1$ = effective fundamental mode seismic weight determined in accordance with Eq. 18.4-2b for $m = 1$

$\bar{W}_R$ = effective residual mode seismic weight determined in accordance with Eq. 18.5-13

δ_i = elastic deflection of Level i of the structure due to applied lateral force, f_i, Section 18.5.2.3

δ_{i1D} = fundamental mode design deflection of Level i at the center of rigidity of the structure in the direction under consideration, Section 18.5.3.1

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δ_{iRD} = residual mode design deflection of Level i at the center of rigidity of the structure in the direction under consideration, Section 18.5.3.1

δ_{im} = deflection of Level i in the m^{th} mode of vibration at the center of rigidity of the structure in the direction under consideration, Section 18.6.2.3

Δ_{1D} = design story drift due to the fundamental mode of vibration of the structure in the direction of interest, Section 18.5.3.3

Δ_D = total design story drift of the structure in the direction of interest, Section 18.5.3.3

Δ_M = total maximum story drift of the structure in the direction of interest, Section 18.5.3

$C(T)$ = elastic site hazard spectrum for horizontal loading as a function of period (T)

$C(T_1)$ = value of the elastic site hazard spectrum for the fundamental natural period of the structure

$C_d(T)$ = horizontal design response spectrum as a function of period (T)

$C_d(T_1)$ = horizontal design action coefficient (value of the horizontal design response spectrum for the fundamental natural period of the structure)

$C_h(T)$ = spectral shape factor as a function of period (T) (dimensionless coefficient)

$C_h(T_1)$ = value of the spectral shape factor for the fundamental natural period of the structure

$C_v(T_v)$ = elastic site hazard spectrum for vertical loading, which may be taken as half of the elastic site hazard spectrum for horizontal loading ($C(T)$)

$C_{vd}(T)$ = vertical design response spectrum as a function of period (T)

$C_h(0)$ = bracketed value of the spectral shape factor for the period of zero seconds

R_c = component ductility factor

S_p = structural performance factor

T = period of vibration, which varies according to the mode of vibration being considered

T_1 = fundamental natural period of the structure as a whole (translational first mode natural period)

T_v = period of vibration appropriate to vertical mode of vibration of the structure

V = horizontal equivalent static shear force acting at the base (base shear)

V_i = horizontal equivalent static shear force at the ith level

W = sum of the seismic weight of the building ($G + \psi_c Q$) at the level where bracing is to be determined and above this level, in kilonewtons

W_c = seismic weight of the part or component, in kilonewtons

W_i = seismic weight of the structure or component at the ith level, in kilonewtons

W_j = seismic weight of the structure or component at level j, in kilonewtons

W_n = seismic weight of the structure or component at the nth level (upper level), in kilonewtons

W_t = total seismic weight of the building, in kilonewtons

Z = earthquake hazard factor which is equivalent to an acceleration coefficient with an annual probability of exceedance in 1/500, (i.e., a 10% probability of exceedance in 50 years)

μ = structural ductility factor (μ = mu)

θ = stability coefficient

ψ_c = earthquake imposed action combination factor

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CHAPTER ONE

INTRODUCTION

The purpose of this work is to compare the current Australian seismic design provision (AS-1170) with the American code (ASCE-7 16).

Structural engineers are using the design code of each country to obtain the data and design procedure for different structural elements. There is notably differences between these codes which depend on the country circumstances such as weather and earthquake zone; these verities can be found in the recommended load action to evaluate the resistance elements sections and also some codes asking for more durability requirements.

To reduce the consummation of the natural materials along with the best sustainable development, it is our duty as scientific searchers and civil engineers to have the maximum structure design with the minimum using of materials with no mitigation in safety aspect. The construction field is the most vital cause of energy consumption and carbon dioxide emission. The US Green Building Council 2008 mentioned, construction field cause 40% of the global environmental problem. For that, a comparison between the American and the Australian codes (ASCE-710 and AS1170-1) deliver appreciated understandings into the sustainable design of the structures. This work contains a comparing case study focusing on assessing the discrepancies in loads, factors, and resistance values specified in codes. (Mourad, 2015).

A comparison between three codes (American, Britain, and Egypt) implemented by Bakhoum and Shafiek, the study focused on loads and strength resistance of elements sections, another case study implemented by Nandi and Guha how compared the Europe code with Indian code based on materials properties and limit of rebar area of several elements sections also they compare formulas for computing the ultimate capacity. A comparison between Egyptian code and Euro code by designing a four-story reinforced concrete building done by El- Shennawy et al. the comparison applied based on environment and economic impact. There is another comparison adopted based on flexure consideration only between the American and the Egyptian codes by Hawileh et al. The conclusion was, Egyptian code has more safety factors than the American. Nevertheless, the difference is insignificant for live/dead ratio which is more than four. Tabsh published a comparison between American and British codes based on Flexural, axial compressive strength, and shear for structure members along with checking different sections while adopting a different ratio of Dead/ Live. The conclusion was, the American code has a more significant section and more reinforcements than the British code: a comparison between the Egyptian, Euro code 8, and the (uniform building code) UBC implemented by Hassan et al for the seismic load effects which were concentrating on calculating lateral forces, reduction factor, ductility needs, and the applicable design acceleration. Bakhoum has adopted a comparison between the European, American, Egyptian, and Japanese codes for the design of the bridge, and there was a massive difference between the codes especially in the values of the traffic action. However, this substantial different reduced while considering the permanent action values. Also, he compared the used loads in designing the railway bridges such as vertical, dynamic, longitudinal forces, and the fatigue impact. Moreover, Bakhoum compared the requirements of the service limit state for the bridge design using the mentioned codes. This work will focus on the dynamic forces and used design rules for a high-rise building considering seventy floors building with all the necessary elements such as columns, shear walls, and beams using the mentioned codes with the same materials. The difference between the two codes during the design stage highlighted. This study intended to deliver a clear vision for the applicability of mixing codes and a comparison for the factor of safety as well. (Applied Technology Council, 1978).

1.1. *Research Significance*

The seismic analysis and design to stand against earthquake effects can be obtained by using the proper strength, stiffness, and ductility. To do so, there are several calculation methods such RESPONSE SPECTRUM METHOD (smooth methods), TIME HISTORY METHOD which depends on the actual collected data from actual earthquakes, and EQUIVALENT LATERAL FORCES ELF which can calculate the adopted lateral inertial forces extracted from the ground wave motions.

The current codes agreed, the lateral force's actions usually are subjected to the dynamic actions of the elastic structure system. Thus, when the building subjected to an inelastic failure as a result of an extreme earthquake strike which does not comply with the designed code, the building will be not as the ideal structure properties, and that will cause a deformation in the building which causes a potential failure or unforeseeable consequences. Seeking for the best structurer efficiency created many design procedures. Since the implemented techniques contain a massive computational process and limitations requirements with difficulties to calculate the approximate expressions to enhance the structure resistance, the demand to create traditional optimisation design procedures can be obtained by the solution of the elastic system with static loads.

1.2. *Research Objectives*

As known, the concrete is commonly used in all the new structures and infrastructures since it has incredible durability and it can be fit in any size and shape is designed. Also, the low cost can be one of the crucial advantages.

The primary objective of this work makes a seismic design comparison between the Australian and American codes with the current requirements for each code. A high-rise building contains sixty floors will be adapted to face a differ provisions of seismic load using the mentioned codes.

Below, the main aspects will be studied: -

- Examine and compare the seismic design provisions for the American society of civil engineers ASCE7 published in 2016 and the Australian AS-1170 code.
- Specify the seismic loads for the sixty floors building. The used design spectra, response modification factors, and the seismic design parameter are extracted from UAE/Dubai zone and used in the seismic design loads.
- Comparing the seismic design loads which are established from the considered codes.
- Assess the effect of the overstrength factors and the accelerations spectral on the seismic design loads.
- Recommend updates for the current Australian AS-1170 seismic design provision as defensible as the result of this research project.

1.3. Case Study

With the purpose to assess and evaluate the theoretical outcome of the mentioned ASCE and AS method, a case study of 210m vertical height of three symmetric buildings with shared podium implemented to compare the two codes outcome.

The building is a seventy floors tower assumed to be constructed in the UAE region. The typical plan is three L-shapes planted on ten story of podium plan with unparalleled alignment which is noteworthy. Reinforced concrete walls implemented along with exterior I-section columns frames and shear walls for each level. The slabs assumed to be a hollow core slab with thickness 330mm and 10mm as a topping slab (see figure-1). Structure details already found in appendix A.

ETABs software modules have been made implementing all the provisions and the requirements of each code.

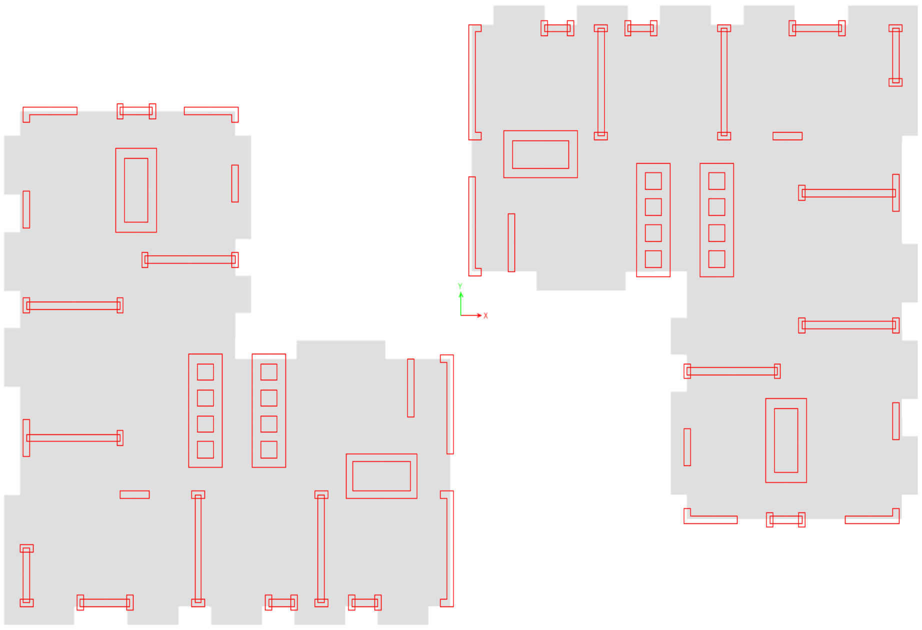


Figure1.1 Typical plan of the tower

Chapter Two

2- LITERATURE REVIEW

The significance of this chapter is to discuss the historical earthquake of each code (Australian and American). Mainly, this chapter will go through the seismic history structural response spectrum explaining all the differences between the two codes and how each code deal with the earthquake resistance and the whole theory of each code which is based on the analysis of real events occurred on the history and how each code started developing his criteria and the relationship between the codes as well.

2.1. Seismic History: -

For the last hundred years, Australia faced an average one earthquake above the five Magnitude every year and six every five years. Most of this event was not notable, and others caused physical damages which the historical records of these events contributed as a source of modelling the seismic hazard map.

While in America in early 1900, the earthquake consideration was minor in building design. Although many earthquakes occurred in the united states such as Cap Ann in Boston 1755, New Madrid 1811,1812, Hayward California 1868, and Charleston 1886. The building design was not changed to resist the earthquake effects at the time.

The Australian earthquake design code established from 1920 to 1930. At the time, the earthquake design limited by taking ten per cent of the building load and apply it horizontally into the building considering it as the lateral force of the structure uniformed applied along the building height. The strong ground motion accelerograph invented at the beginning of the sixties which is recording the ground motion created by the earthquake. Installing these devices in the building in varied floors make it possible to estimate and realize the structural dynamic response during the earthquake time while the main changes in the American code occurred after the San Francisco earthquake1906 which caused a massive fire in the city with massive number of casualties (three thousand deaths) and destroying most of the buildings, this event considered as the deadliest happened in the USA. Steel buildings were the most noteworthy of the perfect act especially which connected by rivets and using the Chicago type.

AS2121, 79 was the first Australian seismic code, the seismic design provisions of the code starting from the explanation of the three-dimensional distribution of seismic hazard, actions demand, and detailing requirements covered in this code. since the zero zones were covering most of the cities, the impact of the code was below the expectation of the engineers.

So, in 1989 an earthquake hit north N.S.W caused eleven lives with huge physical damages, and it specified as a zero-hazard area. The reason for this fatal error was because the engineers depended on seismic modelling activities and the monitoring of seismological up to 1980. The University of Melbourne came up with the response actions of the building during an earthquake seismic effects research. Even though, the Australian code AS1170.4-1993 based on the uniform building code UBC91 form the United States. Standards Australia. (1988). Earthquake loads. AS 1170.4. Homebush, Sydney.

On the other hand, the created the San Francisco Department of the ASCE 1907 assessed the earthquake effects in in-depth detail. It noticed that designing a structure with a suitable system of bracing wind load at 30 lbs. /ft^2 can withstand the earthquake stresses safely which was the first standard of providing an earthquake resistance design in the US. Santa Barbara and Tokyo earthquake 1925, 1923, stimulate main research efforts which directed to create and develop the first seismic actions record equipment in order to assess the earthquake effects on structures and the boards devoted to creating a provisioning code of design earthquake resistance. Later on, the UBC, 1927 published and it was the first time that US code contains an establishment of earthquake effects and resistance design. (International Conference of Building Officials, 1927).

The Australian engineers used the adequate and the actual motion earthquake records as input data which gave more realistic about building behaviour. In recent days, in-elastic combined time history analysis sets which indicated that many structure designs previously couldn't resist the earthquake effects and it might cause severe damage and maybe collapse. However, the periodical inspection for the exits structure shows that although these building does not design to resist the earthquake effects, it does not necessarily occur a collapse or severe damage in case of earthquake occurs. The structural strength can be obtained not only through the building degradation like in-elastic distortion, many structures stand-up against earthquake with economic repairing. Nevertheless, structure subjected to high strength losses often turn out to be unstable and often distorted.

Based on that, it was essential to change the design procedure by confirming the post-elastic strength retention is one of the critical parameters to ensure the building is safe during the earthquake impact. By the time, some post-elastic calculation methods became preferable than others since it is easy to combine the in-elastic expected distortion while the other calculation were highly sensitive to the quick distortion and resulting in a collapse. Curbing these calculation methods by appropriate detailing is possible. Therefore, the influence of successful recent earthquake design is relying on structure elements details in order to identify and comply with the post-elastic requirements and exclude the formation of unwanted response modes. (Stehle, J.S., Goldsworthy, H.M. and Mendis, P., 2001).

The desired calculation methods which are suitable enough to stand against the normal live load with no damage can resist the substantial in-elastic distortion with less noteworthy losses in strength or load capacity. These types of calculations found to be involved in the moment response for the designed element no matter what is the materials of the elements or the connection type.

On the other hand, the undesired post-elastic calculations methods inside the structural elements take a brittle specification along with a shear failure in the steel reinforced concrete column, failure in the steel bonding, and tensile fail for the brittle elements like under steel concrete and timbers. The global response calculations methods contain the progress of the story-soft in the structure especially when in-elastic distortion requirements to be focused which make a high demand for the resist capability for elements inside the zone or other structures which is irregular, and these will not simplify in the modelling design.

After the Newcastle earthquake 1989, an enormous number of engineers from different disciplines along with the Geoscience Australia made research on the earthquake effects along with the University of Melbourne. Another investigation held by the University of Adelaide for the un-reinforced masonry walls, these three sustain works along with many other groups in Australia focused on the Australian seismic conditions which developed significant revisions along the world especially for the low-moderate seismic countries. In1990 the establishment of the Australian Earthquake Engineering Society along with the yearly conference gives acknowledge and research exchanges which create a strong relationship between the earthquake world institutes. The AS1170.4.2007 version depended on the 1993 version for the seismic hazard maps which depended on the local geological building information's and the faults groups which producing the future earthquakes.

In 1939, an earthquake attacked the south of California and the school building distortion was the most critical facet but luckily it happened after the school timing. After a broad investigation, new regulations of the earthquake design especially for schools created which introduced a new engineering philosophy of the building impotence factor. Additionally, California restricted to construct structures without reinforcements which adopted for all the high-risk zone afterwards. Later, significant actions have been taken in California with local engineers to draft a provision region seismic design for LA and S. Francisco 1943, 1948, developing these codes was simplified throughout the El Centro Earthquake, 1940. Also, it was the first time of recording the ground motion by accelerograph. (Mourad, (2015).

The ASCE S. Francisco and the north California engineering association gathered in 1951 and published the 'ASCE Proceedings-Separate No. 66. Separate 66' which was the milestone papers of the earthquake provision design. After that, the Structural Engineers Association of California SEAOC published the Suggested Lateral Force needs or as called the Blue Book from SEAOC which became a milestone paper for updating and growing the provisions of the seismic design in Uniform Building Code UBC which is used mostly in the west of the united states. The blue book updated regularly up the end of 99th which adopted in the UBC as well.

Anchorage and San Fernando Earthquakes 1964, 1971 were essential occasions which show a substantial problem for the undetailed ductility reinforced concrete buildings behaviour. Which occurred a significant distortion in these types of structures and that was not accepted by the engineers. Later, the Applied Technology Council ATC created an innovative provision of earthquake design. 'ATC 3-06, Tentative Provisions for the Development of Seismic Regulations for Buildings, 1978' which is now the fundamental of the new standard.

More than ten thousand people died and huge distortion in buildings because of the Mexico City earthquake 1985, the essential facets was the centre of the earthquake two hundred miles from the city. The reason for that was the geological nature of Mexico City which contains an ancient lake bed of mud and soil clay which created a ground motion stretched more than usual. This type of ground motion caused damages more than the average and tall buildings because these buildings were in resonance with a ground motion which required to add the site factor in the seismic design of the buildings. American Society of Civil Engineers, (1907).

The Northridge earthquake1994, cause 57 deaths and over twenty billion dollars of loses. The considerable repair damage costs highlighted the necessity of involving the total structure performance along with the falling of structures and drive the engineers to move to the Performance-Based design. This earthquake discovered the most critical event to unexpected distortion of the steel moment frame building while it expected that it is the system to resist the seismic effects. Still, several buildings had cracks in the welds area which linked the beams to the column. Directed too many years of studying and improving the steel moments frame performance against the earthquake.

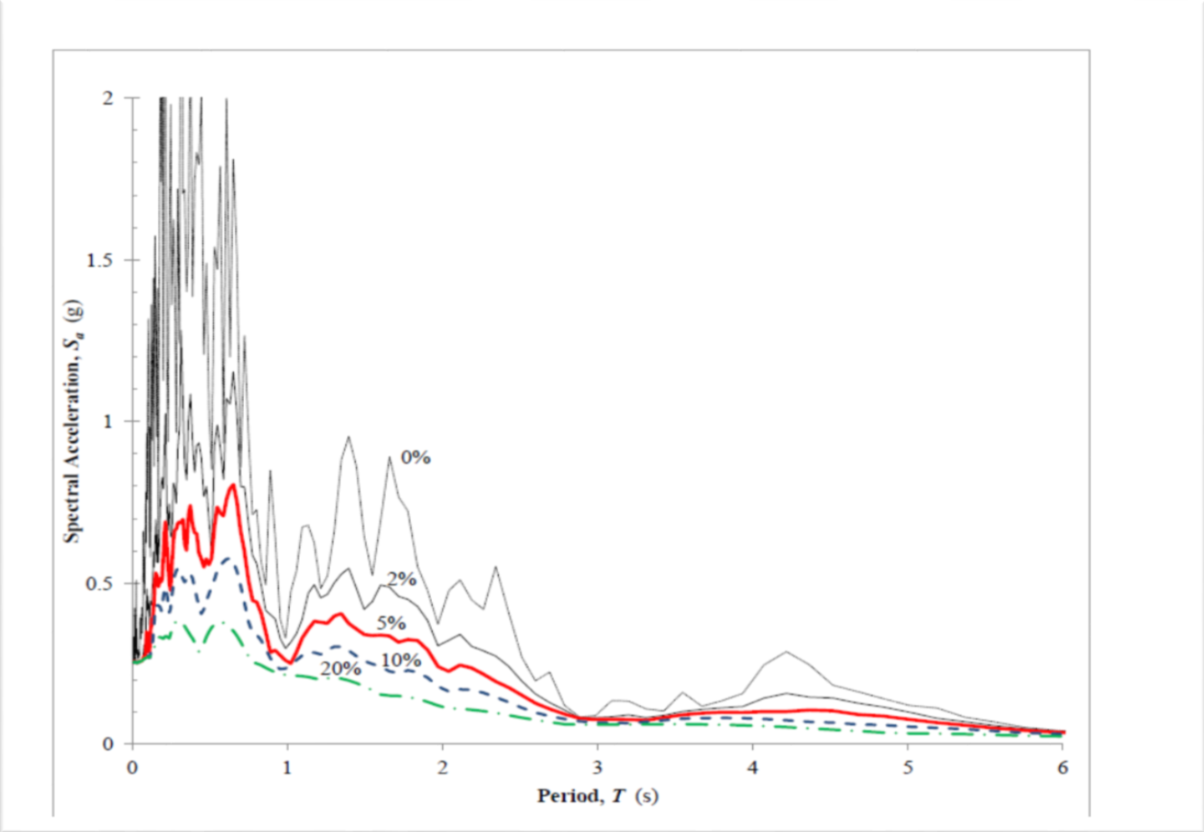


Figure (2.1) response spectrum of ground acceleration for different damping percentage

It assumed that structure has an elastic behaviour. It mentioned previously that the ordinary buildings could strain outside the limit of elastic in reacting to the ground motion earthquake which is different from the other stress actions types. So, strain not allowed to reach the limit of elastic for economic purpose. Figure (1.6) illustrate the maximum response acceleration for 1g (acceleration because of gravity) for building having evenly low damping. For evenly significant seismic and also for building stand against the lateral action contain a strength of lateral static about 20%-40% of the gravity.

The meaning of the dynamic type of ground earthquake shaking is a significant percentage of motion energy possible to waste by the inelastic distortion when the building is ductile, and a few distortions acknowledged. The below figure (1.8) used to show the essential different by wind and seismic loads. Figure (1.9) illustrates the column when W is tremendous and cantilever beams when W is minor. Displacement created after a wind pressure impact the building and while the ground motion impacts the building created a displacement as a result of the mass above and base foundation, and that produced internal forces. Movement (displacement) consider as an independent variable while the forces called the secondary outcome based on that, two plots extracted independent on X-axis and dependent on the Y-axis. So, Figure (1.8, b) illustrate the response to force for wind (weight of gravity) and Figure (1.8, c) illustrate displacement from seismic effects.

Deformation of building beyond the elastic limit called ductility, different materials besides different structure arranging can cause different ductility. Doherty, K., Griffith, M.C., Lam, N.T.K. and Wilson, J.L. (2002).

In general, the response acceleration can be mitigated by a factor equal to the ratio of the ductile for the building who have a long natural period while it is essential for the short natural period exactly like the elastic buildings with an increase in the displacement. For the intermediate natural period building and this is almost all the building using it, the response acceleration is mitigated while the response displacement is like the ductile building like an elastic building have enough toughness to resist the seismic with no yield

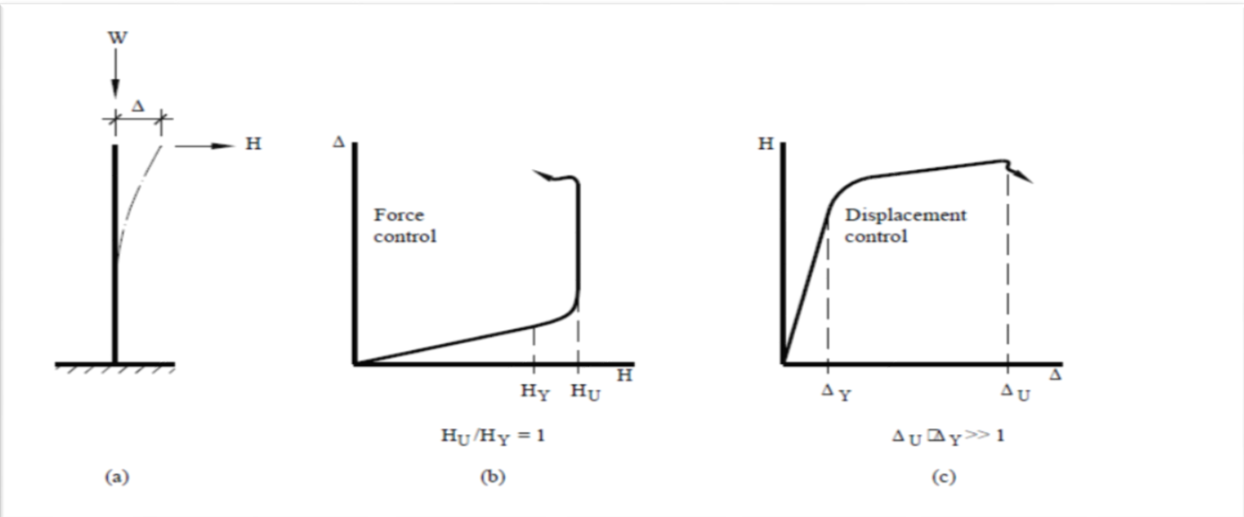


Figure (2.2) Force Measured Strength against Displacement Measured strength

The inelastic response is very complicated. The seismic ground motion contains an essential number of reverses and reappearances of the strains. So, the remarks of the inelastic characteristic of elements, materials, or system stand against growing loads till the failure possible to be misleading. The repeated deformation can lead to failure in stiffness, strength, or together — these systems who confirmed the ability to resist a significant number of cycle distortions approved to use a significant portion of their maximum ductile for the seismic design resistance.

Almost all the buildings designed and analysed for earthquake response via long elastic study with building strength which is bounded by the toughness at its dangerous position. Most building so complicated which make the max. The strength of the building ductility is not accurate by this type of estimation. Below figure (1.9), is figure illustrate the relationship between the actions and displacement for a standard frame? The yield should create four positions before the peak resistance reached — the boundary between first yield and peak strength called over-strength, which is very important in the ground motion resistance. The American code allows doing a redistributing of the internal forces. (Applied Technology Council, 1978).

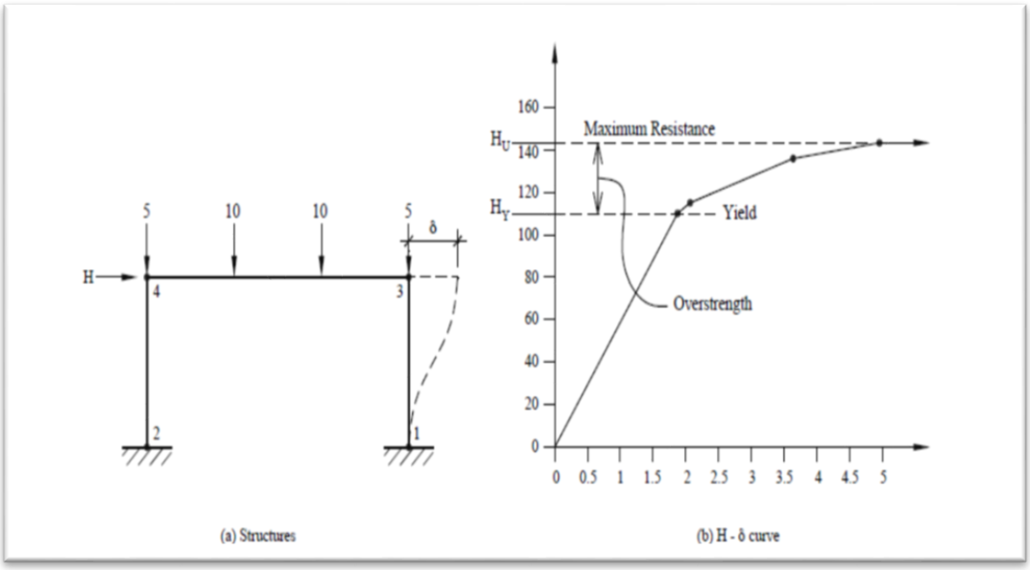


Figure (2.3) relation between the actions and displacement for normal frame

Finally, elements features are very vital to define the structure's seismic response are ductility, damping, over strength, a natural period which is subjected to the stiffness and the building mass, and the resistance constancy when frequent reversals of inelastic distortion occurred. The first fours elements are subjected to a system of the building and not to the shape and size. The coefficients Ω_0, C_d, and R contain the over strength, ductility, and resistance stability which is proposed to be a conventionally low estimation of the decrease of response acceleration in the system ductility from an elastic oscillator with a specific damping degree which is adapted to calculate the strength. The displacement calculations depend on the ground motion factored by R to reduce the real displacement. Ductility C_d proposed to be the necessary amplification to translate elastic response displacement calculate to decrease the ground motion to the real displacement Ω_0 proposed to bring a high calculation of the highest force created in the building.

2.2. Structural Response and Earthquake Phenomena Research: -

The major studies in Australia focused on the post-elastic behaviour and response of the structures which designed for the typical actions (dead, imposed, and wind loads) without taking the seismic earthquake actions into the considerations. Starting from the AS1170.4, 93 inward the researchers directed their attention on evaluating the ductility displacement, over-strength resistance, patterns fail, and recently focused on the capacity displacement for several diverse structure elements and systems by adopting experimental and analysis methods. Recently, improving illustrative nonlinear pushover curvatures became essential for differ structure systems to combine it with the capacity spectrum method (CSM) as suggested in ATC-40, 96. The CSM delivers a quick assessment method to evaluate the structural systems performance against exciting earthquake events. The demonstrative seismic earthquake in Australia can define in response spectrum terms along with real period returns by using the model created by Wilson and Lam, 2003.

The buildings which contain soft-stories identified as weak and the collapse possibility is highly expected with severe damages when it is subjected to an earthquake. Despite this, it is normal to find this type of buildings in the low-moderate seismic zones like Australia. Program research conducted to evaluate the dead, imposed, and lateral actions along with the capacity of the displacement of the reinforced concrete structures which have a soft story. The displacement model calculates the impact of the moment, vertical compression, shear, pier end turning, footing flexibility and the deformation of the plastic pivot. An experiential program is running to assess the correctness and dependability of the current analysis model. Based on these studies, many of these structures which have a soft-story unsuccessful with moment ductility limit (instead of brittle shear fail) through drift story capacity about two per cent only. An evaluation of the capacity displacement and seismic displacement requirements recommended that a lot of soft-story structures on shallow soil and rock locations can endure earthquakes effects with periods return around five-hundred years.

In 2003, Griffith set up a pioneering retrofit system to enhance the capacity drift of the soft-story buildings. The system has implementing FRP or steel plates into the moment area of the pier with bolts. Assessments specified that retrofitted piers create a capacity drift more than 2.5% with analysis models telling that ten per cent of the capacity drift can gain. Standards Australia. (1988). Earthquake loads. AS 1170.4. Homebush, Sydney.

Large experiential and systems analysis study program implemented to assess the seismic behaviour of the reinforced concrete long beams buildings. The studies of the sub-assemblage test showed that this type of buildings which designed for the dead and imposed actions only along with the least detailing needs in Australia had a capacity drift about 2.5% before reducing the strength of the lateral capacity.

A pioneering technique of disconnect nonstop top reinforcement in the group beam in line to the pier proved that the level of damage along with large drifts could reduce.

Another experimental and analytical assessments implemented for concentrically braced steel frames CBF in specific, connections of crosswise braces and pier considered to stand against elastic wind action and without the seismic actions with over-strength and fail mechanism. The research found the connections are vulnerable than members in case of adopting the over-strength factor of the weld connections is about one and a half. This fail occurs throughout the weld low cycle failure cracking which caused mitigation in the displacements capacity of the system. The research was about the retrofitting cost impact which indicated some potential.

Precast concrete low-rise building behaviour investigated by Robinson et al., 1999. This type of structure is widespread use for the flats accommodation building which has a weaker connection than the precast element itself. Based on the study, as much as the connections are strong, the ductile capacity and the drift capacity will have increased from one to three per cent based on the connection length embedment in the floor slab.

Also, the domestic structure performance against the seismic load investigated by Gad et al., 1999, 2000. The investigation mentioned the non-structure plaster-board involved important lateral strength of the entire system. On the other hand, the veneer of the brick is not essential and will collapse after the shake. This investigation recently extended to check the thresholds damage under low level of vibration. (Stehle, J.S., Goldsworthy, H.M. and Mendis, P., 2001).

A creative displacement-based method to evaluate response out-of-plan for masonry buildings recently created by Griffith. Common base-force calculation method shows defensive and unreliable in expecting masonry walls failure. Displacement method uses a tri-linear formula to describe actual non-linear displacement forces attitude also shown in the extensive experimental systematic program.

Generally, there are many analytical studies implemented to investigate the structural system behaviour. These investigations included; reduction ductility factor for designing building against seismic effects, damping equivalent ratio in frame structures which designed as a reinforced concrete structure for combination to structure technique for response seismic displacement calculations, and inelastic torsion response of structures by the displacement-based method.

In the United States, based on scientist's believe, the earthquake happened due to tectonic movements. Surface movements identified as a fault. The sudden motion created an energy strain which causes seismic in the fault zone. The mentioned waves caused ground shaking which is considered in the design. A sudden ground movement found in surface around the fault intersects, during this phenomenon, many effects occurred such as ground deflection, sliding, and liquefaction which encourage the designer to enhance the soil to resist these effects.

A major earthquake in the west of United States occurred in the tectonic border while east and centre of the United States occurred in the different zone which makes it hard for engineers' anticipation and nearly impossible to draw a seismic map. (Applied Technology Council, 1978).

Seismic magnitude reduced significantly from the earthquake centre. Also, the attenuation rate is small in low frequency than high frequency. Seismic hazard estimated based on three limitations; high motion frequency S_S, mid-frequency S_1 (both are based statistic analytical seismic evidence), and last one is low motion frequency with extended period T_L and not related to robust analysis. Basically, there are two sources to determine seismicity for the citrine zone; history record and geologic records, since earthquake does not frequently occur like snow and wind effects, history records are unreliable to use it alone which lead us to use both sources. Geologic records data needs a good understanding, and it is used commonly to enhance the seismic knowledge. In several places, these data developed normalcy and the two data sets used to draw ground shaking seismic provision maps. These maps developed since then and many criteria added to maps. Before 1997 edition, life safety during the earthquake was the foundation of design by taking 10% as a likelihood of occur in 50 years' time while next edition considered avoiding collapse during the maximum considered earthquake MCE by taking 2% as a likelihood of occurring in the 50 years' time. New code design based on 1% as a likelihood of occur in the 50 years' time for conventional structures. Doherty, K., Griffith, M.C., Lam, N.T.K. and Wilson, J.L. (2002).

Chapter three

3. BUILDING MODELLING AND ANALYSIS

To establish a realistic comparison between the ASCE7 and the AS1170, its necessary to analyse and design a building using the mentioned codes and compare the outcomes, this chapter will be designated for this purpose. In both codes, using the static and dynamic analysis is essential as mentioned later. The analysis will also be based on two zones, the first zone will be in USA/Indio city and the second in Australia/ Sydney-Katoomba in the moderate seismic zone with similar circumstances to illustrate a real comparison. Each analysis will contain; calculating the base shear along with the forces story, the regular and irregular torsion, P-delta effects and drift, the orthogonal forces and load combination. In general, a shear-walls system will be used.

3.1. Seismic Design Approach:

The general approach in both codes is designed a structure against an earthquake effects by considering several points; determine the plastic hinges points for the building, implementing mechanism of the structure in each zone, determine the elements capacity inside the plastic hinge position to resist all the possible loads (dynamic and static), determine the region to provide proper details to eliminate the behaviour of post plastic, determine suitable over strength magnitude, and the outcomes must be adjusted to allow deformation to occur in the joint location only.

Different between earthquake loads and others is dynamic, wind load considered as static in most structures. Loads inside building became pressure loads instead of mass acceleration. Nevertheless, during an earthquake; structure above the ground will not be subjected to forces. Strains and stress inside the building will develop from the dynamic response of the base motion. Though most of the earthquake engineers using the Equivalent lateral force ELF which is a static method, knowledge of shaking theories of the building is mandatory. (International Conference of Building Officials, 1927).

<u>3.1.1.</u> ***Site Response Modelling and Micro-zonation's: -***

Both codes have the same categories of the soil classification. When the footing on level 30m and above. In case of lack in this information, proper soil characteristic is allowed to be valued by the recorded design expert to make the soil report rely on recognized geologic settings, and in case there aren't enough details to specify the site class, its recommended to take it D or if the authority/ geotechnical information required E or F. the ASCE7 code prohibited to assign A and B in case the foundation fixed upon soil have only 10.1m to reach the rook level and its vice-versa for the Australian code as shown in chapter 4 and 20 from the ASEC7.16.

The Australian code defines the site classification based on the depth and type of soil, which specify the dynamic stiffness and period of the site. These two factors are the primary key to specify the dynamic response features and the soil damping, the resistance contrast with original rock, grade of nonlinearity as well. Rodsin, K., Lam, N.T.K., Wilson, J.L. and Goldsworthy, H.M. (2003).

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

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

Table (3.1) (AS1170.4) the classification of the soil site for class c

Soil type and description		Property		Maximum depth of soil
		Representative undrained shear strengths (kPa)	Representative SPT *N*-values (Number)	(m)
Cohesive soils	Very soft	<12.5	—	0
	Soft	12.5 – 25	—	20
	Firm	25 – 50	—	25
	Stiff	50 – 100	—	40
	Very stiff or hard	100 – 200	—	60
Cohesionless soils	Very loose	—	<6	0
	Loose dry	—	6 – 10	40
	Medium dense	—	10 – 30	45
	Dense	—	30 – 50	55
	Very dense	—	>50	60
	Gravels	—	>30	100

Table (3.2) The classification of the soil site

Strong Rock	Ae (0.8 Fa) (0.8 Fv)
Rock	Be (1.0 Fa) (1.0 Fv)
Shallow Soil	Ce (1.25 Fa) (1.4 Fv)
Deep or Soft Soil	De (1.25 Fa) (2.3 Fv)
Very Soft Soil	Ee (1.25 Fa) (3.50 Fv)

Table (3.3) Sub-Soil Class values as per the AS1170.4, 2007

The American code is using the below values specified using online or Figures 22-1 and 22-2 from the ASCE7,16, the Risk values Ss & S_1 ‘set the Maximum Considered Earthquake MCE_R motion Ground limit with 0.2 &1.0 seconds and 5% of Critical Damping for soil B class.

Implementing the usual method to get the proper response acceleration limits for the specific site class else than B class, S_1&S_s to be revised in line with sec.11.4.3. The revision implemented by Fa & Fv coefficients to scale these values for the site other than B Class.

The revised MCE_R acceleration response spectra for specific site class S_{M1} & S_{MS} for 0.2 &1.0 sec. To be modified as per tables 11.4-1 & 2 from the ASCE7 (table 2.13&14) and then using the equation below which is from the same section, the MCE_R for the 0.2 and 1.0 sec. S_{MS} and S_{M1} respectively can be found in EQ. (1.17&18).

$$S_{MS} = F_a \, S_S \qquad \text{EQ. (1.17)}$$

$$S_{M1} = F_v \, S_1 \qquad \text{EQ. (1.18)}$$

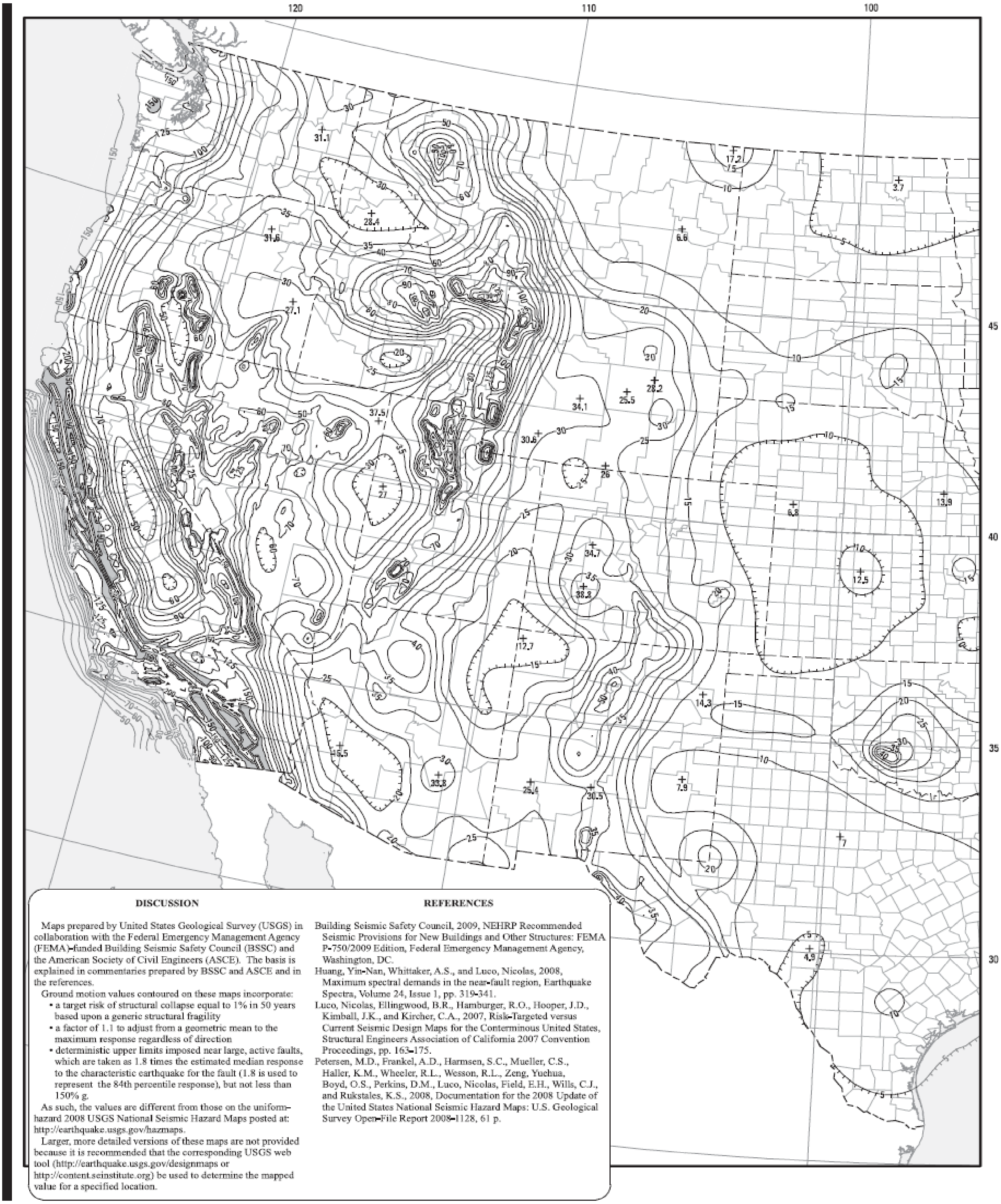


FIGURE 22-1 S_S Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.

Figure (3.1) Ss value

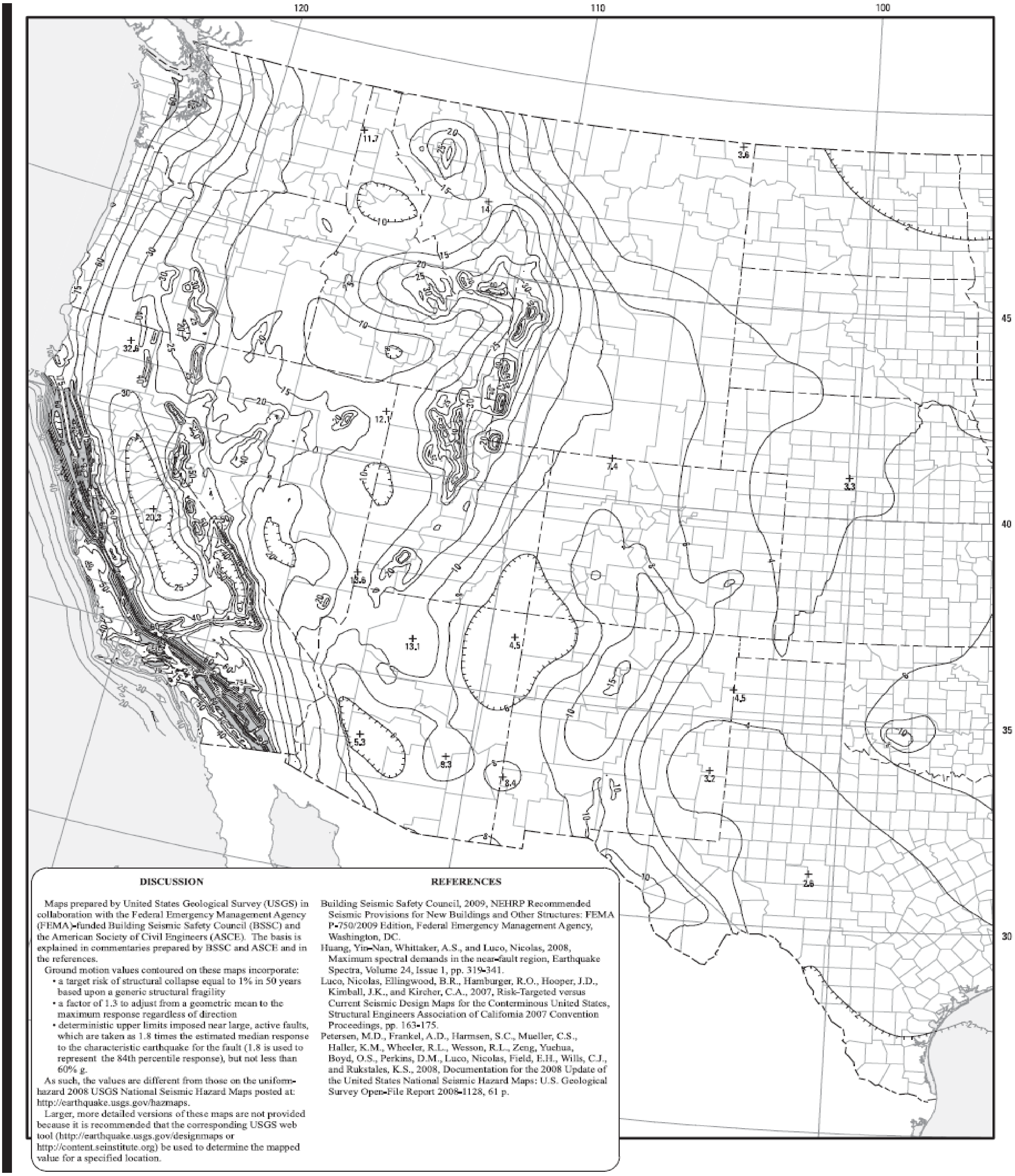


FIGURE 22-2 S_1 Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 1 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.

Figure (3.2) S1 value

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

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

Table (3.4) the Site Coefficient, Fa

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

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

Table (3.5) the Site Coefficient, Fv

3.1.2. Importance level: -

Essentionaly, to identify the importance level, for the Australian code, using the BCA code. It can specify based on the occupancy type and the building use type, as shown in the table (3.6). Generally, there are five classes of buildings in the Australian code, and in case the design building is not covered by these categories, it will be subjected to the engineers' judgment. In case the building has various usages, the designer must use the maximum level of an important factor, and in case there is access to two building, the designer must follow the same concept. As per the BCA, there is four level which is itemised and linked with Probability Factors k_p and the return periods RP as shown:

Table (3.6) importance levels as per the BCA

Level	Type of structure	Return Periods RP(Years)	Probability Factors k_p
IL1	Temporary and very minor structure		
IL2	General structure engaged by public	500	1
IL3	Occupied by a big number of public	1000	1.3
IL4	Crucial structure with a post-disaster function	1500	1.5

Consequences of failure	**Description**	**Importance level**	**Comment**
Low	**The low** consequence of loss of human life, *or* **small or moderate** economic, social or environmental consequences	1	Minor structures (failure not likely to endanger human life)
Ordinary	**medium** consequence for loss of human life, *or* **considerable** economic, social or environmental consequences	2	Normal structures and structures not falling into other levels
High	**The high** consequence of loss of human life, *or* **very great** economic, social or environmental consequences	3	Major structures (affecting crowds)
		4	Post-disaster structures (post-disaster functions or dangerous activities)
Exceptional	nces where reliability must be set on a case by case basis	5	Exceptional structures

Table (3.7) consequences of failure for importance levels

For the American code, huge earthquake usually in the frequent occasion with great motion ground and predictable to occur damages for structures even when it designed for this type of failures. Damages levels are different based on structure type in the same location as hospital and ordinary building. The aim of this segment is to get the best recover capability after an earthquake event, and it is achievable by categorising the structures in:

- Important structures to reply directly after an earthquake event.
- Offer a possible disastrous injury.
- Have a huge quantity of tenants their ability to take care of themselves is below regular.

the building must have a proper low collapse possibility in infrequent occasions called as the maximum considered earthquake MCE_R motion ground; the second is life threat casualties which is majorly from the non-structure damages elements on/in buildings will be improbable in seismic design motion ground (well-defined as 2/3 MCE_R).

ASCE7 reports the goals by assigning each building to one of the risk types mentioned in Chpt.1 along with the importance factor I_e.

It is important to mention the building Risk Category in the ASCE7 when the SDCs or the seismic design categories provide an easy way to design and execute structures with the lowest needs of detailing and cost needs in the level of hazard and failure value. The ASCE7 specified the risk category of each structure as per the table below:

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

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the substances is commensurate with the risk associated with that Risk Category.

Table (3.8) Risk Category of Buildings and Other Structures for Earthquake Loads

The Ie factor used in the ASCE7 in measurable principles for strength and the majority of those factors divided by RP or R in order to decrease the damage to significant buildings also to prevent collapse. These factors correct the response elastic linear to the proper amount in the design stage. The primary correction in many buildings is ductile. Dropping the factor R will raise the yield strength requirements. The critical building must be capable of attending their objective. (See table 3.9)

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

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

Table (3.9) Importance Factors by Risk Category Buildings and Other Structures for Earthquake Loads

3.1.1. Building Systems: -

Generally, three types of lateral resistance system available, walls, frames braced, and frames un-braced or as called moment resistance which adapted to arrange structure classify type in American and Australian codes. Braced frames and walls receive smaller elastic response reduction than braced system due to redundant of frames, and it has the same level of stress in different position while same joints of column-beam are usually capable of keeping steady response over several cycles of inverted inelastic bends.

Usually, detail connection of the braced systems makes ductility development more difficult, and compression buckling for the member is limited to the inelastic response, the failure commonly happened due to local buckling along with general element buckling usually caused a high strain in members who caused a brittle failure and reach the tension yield. Eccentric bracing along with the new detailing and fitting for concentrated braced frame advanced to beat these faults.

Un-gravity loaded shear walls taken reduction than loaded walls because of redundancy, and axial pressure usually reduces the ductility moment in a concrete element. Chile 2010 earthquake gave us a good understanding of shear walls seismic resistance which reflect in new code revision. Reduction in combined system is more significant than other in elastic response due to redundancy. Redundancy is commonly mentioned as a necessary characteristic for seismic resistance, ASCE7 identify it as 1.0 or 1.3 for systems need more careful design.

Redundancy is frequently cited as a desirable attribute for seismic resistance. A quantitative measure of redundancy is included in the *Provisions* in an attempt to prevent the use of substantial reductions from the elastic response in structures that possess very little redundancy — Chandler, A.M., Lam, N.T.K. and Sheikh, M.N. (2002).

3.2. Seismic Analysis Approach: -

3.2.1. Earthquake Base Shear: -

Engineers created advance techniques to calculate the base shear loads. Below, the most common calculation methods used by the earthquake engineers: -

A- <u>Integrate time history method:</u>

This method depends on stepwise outcome using time extracted from freedom millidegree motion formulas which reveal the real response of structure behaviour, and it is an elegant analysis technique, and it is the real meaning of earthquake motion ground selected as participation limit for a specific structure. Records are rare to obtain for certain location and generally, either revised actual free records or motion ground are used. Generally, the structure model must characterise for post-elastic and elastic response appearances of the structure. This details data is hard to find it during the design; this analysis method is commonly bounded to check the right expectations in the design stage of structure than a technique of delivery lateral actions. Lam, N.T.K. and Wilson, J.L. (1999).

B- <u>Multi-modal analysis method:</u>

This method is a dynamic elastic response calculation applied by specifying the response of the building for each mode trailed by the outcome reactions of every significant mode response.

It is essential to take the building mass as an intensified mass at every level which produces an SDOF in every level and case the structure vulnerable against the torsion and lateral response; it is required to double the response modes number. This method contains will specify the shape of mode and period response, specifying the coefficient of the lateral shear of every mode (using the period mode and the spectra design) and base shear result distribution in line with the shape response of each level, then; all the mode response gathered using the complete quadratic combination CQC or square root sum of squares SRSS techniques.

The equivalent static analysis method or any other static techniques sued to specify the forces (loads) and displacement of the whole building however the above methods considered to allow for actual response features of the structure and not presuming the first response mode only. We have to keep in our mind that it still evaluating the building response although it is elastic. Preventing structure from collapse and damage control needs much supposition in order to reach the response of inelastic. Also, the development of the model is often contained a few modifications to the final design.

C- <u>Equivalent Static Analysis method:</u>

Although post-elastic attention involved in this method it is majorly an elastic technique to specify the lateral coefficient forces. Also, it is easier to use than other techniques, along with a rare with the implicit facilitate presumption being maybe more hormonal with other presumption implicit in another place in design steps. The design procedure can be pointed as below:

- Calculate the initial response period mod of the structure from the spectral design.
- Use spectral design to ensure that base shear forces of whole structure coordinated with assumed response post-elastic (ductile).
- Base shear forces to be distributed among the different combined mass level by applying ninety per cent of the forces as a triangle and the rest to be applied at the top level in order to tolerate higher effects mode.

- Determine elements loads and actions by using presumed to spread lateral forces and assess it.
- Specify the total building response, mainly about the inter-story drifts evaluated for elastic response building.

About the American code, the ASCE7 established many measures to define the effects forces from the ground shake. The systematic (analytical) procedures categorised into two effects; non-linear against linear and equivalent lateral force ELF against dynamic. The most commonly used methods are the equivalent static force and the response spectrum methods. Also, there's another calculation method called the dynamic history response, the non-linear method also used but very limited, it is very delicate to assumptions for building behaviour analysis and motion ground hired as input data. The mentioned methods are using the real ground motion as an input data and need to scale to the first response spectrum. The used design spectrum for the first two methods is identical. In order to use the elastic spectrum reduction in these methods, the ASCE7 recommended to divide the elastic spectrum into R coefficient, since the design calculated based on the design spectrum which is 2/3 MCE, the total elastic reduction is between 1.9-12 while the used damping 5%, and some of the factor R achieved by the level of damping. Overstrength and ductility cause a significant reduction as well. ASCE7 identify the total seismic effect as a grouping of horizontal and vertical response motion. Vertical effects calculated as a percentage from the dead load (-/+) then, internal forces along with the gravity loads to be checked with element capacity mitigated vie factor resistance.

To determine the design spectrum using the equivalent static force method, we need to set up the necessary seismic acceleration which is possible by selecting the natural period, soil profile, and R-value. The period calculation extracted from the analysis model which is limited, and the reason for the limitation is not to use the flexible model to have small acceleration and significant period. After calculating the total response acceleration, base shear found thru multiplying it with the entire active mass of the structure, which is the entire permanent load. After specifying the total lateral load, the ELF method determine the distribution of these forces on the building which is developed as an outcome of dynamic examination of uniform structures and is proposed to get a summation of shear loads at each level and that envelop loads will influence building and create overturning moments bigger than various necessary dynamic studies. The dynamic analysis is essential and necessary for all the tall buildings. Unless the building is not indeterminate mass centre, stiffness, strength of structure members, and turning mechanisms in the underlying ground motion, the ELF calculation method is the same. Which called ‘horizontal torsion’ and this can be calculated by adding a rough amount of displacement between the force centre and the calculated mass centre each direction which is called the accidental torsion. After comparing the actual and accidental torsion with the allowable limit, the accidental torsion should be calibrated.

The modal analysis method is parallel the equivalent lateral forces method. The significant difference is the identical deflection shape and the natural period is necessary to be identified for many of the natural vibration modes which can be estimated by building a mathematical model. The calculation needs at least ninety per cent of building mass to participate. Base shear loads extracted from the designed spectrum which is similar to the ELF method and the acceleration with displacement spreading, story shear, a moment of overturning, and drift can be specified for every mode. Total values for subsequent analysis and design determined by taking the square root of the sum of the squares for each mode. This summation gives a statistical estimate of maximum response when the participation of the various modes is random and in case the there are two modes so close in period values, it is necessary to use the more developed method to gather these values, and the total absolute amount of each mode is reasonable.

The minimum based-shear limit extracted from modal analysis method determined from the static method and the estimated periods as well. In many occasions, the limit braked and to solve this issue is required to scale the outcomes. The plane torsion is like the static method since the ELF applied at each level, the moment overturn and the shears at story can be taken from the summation method, and these results are not well-matched because the calculated moment from the collective level forces will not match the summation moment. The advance consideration of these issues will avoid the analysis and check issues. As mentioned earlier, there is three primary type of analysing the structure to resist earthquake for both codes (Australian and American).

The base-shear formula as per the AS1170.4, 2007, complied with the international standards which lead to revising the all the factors along with notation, the Equivalent Lateral Forces (Static Shear) ELF applied at the building base with direction has been considered and estimated from the below following: -

$$V = C_d(T_1)\ W_t \qquad \text{EQ. (1.1)}$$

$$C_d(T_1) = C(T_1)\ (S_p/\mu) \qquad \text{EQ. (1.2)}$$

$$C(T_1) = C_h(T_1)\ kp\ Z \qquad \text{EQ. (1.3)}$$

As result:

$$V=[K_P Z C_h(T) S_P/\mu]\ Wt \qquad \text{EQ. (1.4)}$$

$$\text{Base shear multiplier} = V/W_t = Ch(T_1)\ k_p Z\ (S_p/\mu) \qquad \text{EQ. (1.5)}$$

The AS1170.4, 2007 don’t contain a low limit for the seismic load weight value implemented on the buildings. To compare the base-shear earthquake value, the per cent of the seismic building weight can be estimated to use it in the calculation can be taken from code which is referred to as the multiplier base shear.

The below requirements and parameters are necessary to find the design coefficient of the horizontal action or in another words amount of horizontal design response spectrum at the significant natural period of building C_d (T_1).

$C_d(T) = C(T)\ S_P/\mu$ EQ. (1.6)

Where C(T) is the elastic hazard spectrum value at the site

$C(T) = K_P Z C_h(T)$ EQ. (1.7)

Where T is the variation period of mode vibration of the building

V also can be formalised as:

$$= \frac{W_i h_i^k}{\sum_{j=1}^{n}\left(W_j h_j^k\right)}\left[k_p Z C_h\left(T_1\right)\frac{S_p}{\mu}\right]W_t$$ EQ. (1.8)

In general, the Equivalent lateral forces (F_i) can be distributed vertically using equation (1.9)

$F_i = K_F\ V$ EQ. (1.9)

Where $K_F = W_i h_i^k / \sum(W_j h_j^k)$ EQ. (1.10)

$$T = 1.25 k_t h_n^{0.75}$$ EQ. (1.11)

The seismic elastic shear base is equivalent to the produced elastic seismic coefficient times the effective building weight. Basically, the seismic actions using the dynamic analysis calculation method are obtained for every mode vibration and the outcome of the forces combined by two methods either by the Square Root of the Sum of the Squares (SRSS) or the Complete Quadratic Combination method (CQC), and as per the Australian code section 5.4.2.1of the AS1170.4, the building components along with foundation that involved to resist the horizontal forces earthquake in main two directions of building, the influence of the axes specified individually should be combined as hundred per cent in one direction and thirty per cent in the perpendicular direction. The vertical ground motion influence is calculated by taking the full of the permanent load in combine with seismic actions. Griffith, M.C., Wu, Y. and Oehlers D. (2003).

In case the vertical earthquake load considered, the up and downwards directions should be taken in the considerations and design as equation 1.12

C_{vd} (T) = C_v (T_v) Sp EQ. (1.12)

= 0.5C (T_v)S_p

= 0.5kpZC_h (T_v) Sp

Where C_v (T_v) = elastic site hazard spectrum for vertical loading for the vertical period of vibration.

On the other hand, the American code calculates the base shear considered some essential requirement to design any building or structure to do all the needful assessments for the structure. First, it is essential to calculate the building/ structure weight and the suggested damping to be considered five per cent either it is static nor dynamic. Second, for the vibration building period, it can be obtained from the ASCE7 using formula 12.8-7 as shown below

Ta = C_t h^X where h is the building height, Ct and X can be taken from table 3.10:

Structure Type	C_t	x
Moment-resisting frame systems 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:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731)[a]	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a]Metric equivalents are shown in parentheses.

Table (3.10) Values of Approximate Period Parameters as per the ASCE7

Then, it is necessary to find the upper limit for the calculated period from the formula Cu*Ta and the Cu from table 3.11.

Design Spectral Response Acceleration Parameter at 1 s, 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 (3.11) Coefficient for Upper Limit on Calculated Period as per the ASCE7

The next step is to calculate the seismic response coefficient as mentioned in section 12.8.1.1 and as below:

CS min < CS < CS Max

- CS max = $\frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$ Equation 12.8-2.

The value of C_s computed in accordance with Eq. 12.8-2 need not exceed the following:

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \quad \text{for} \quad T \le T_L \qquad (12.8\text{-}3)$$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \quad \text{for} \quad T > T_L \qquad (12.8\text{-}4)$$

C_s shall not be less than

$$C_s = 0.044 S_{DS} I_e \geq 0.01 \qquad (12.8\text{-}5)$$

In addition, for structures located where S_1 is equal to or greater than $0.6g$, C_s shall not be less than

$$C_s = 0.5 S_1/(R/I_e) \qquad (12.8\text{-}6)$$

where I_e and R are as defined in Section 12.8.1.1 and

S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4 or 11.4.7

T = the fundamental period of the structure(s) determined in Section 12.8.2

T_L = long-period transition period(s) determined in Section 11.4.5

S_1 = the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 11.4.1 or 11.4.7

For structures located where S1 is equal to or greater than 0.6g, Cs shall not be less than,

C_S min = 0.5S1/(R/Ie) Equation 12.8-6. => 0.5*0.738/ (3½/1) = 0.105.

- Cs = $0.044 S_{DS}$Ie ≥ 0.01 => 0.044*1.19867*1= 0.052 Note: for the ELF use CS min and for Drift use CS max

Now, it is possible to calculate the seismic base shear from the ASCE7, 12.8-1 formula or in another word, the seismic load can be considered as approximately five per cent from the total building weight and to be spread laterally on the structure height by Equations below:

$$F_x = C_{vx} V \qquad (12.8\text{-}11)$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (12.8\text{-}12)$$

where

C_{vx} = vertical distribution factor
V = total design lateral force or shear at the base of the structure (kip or kN)
w_i and w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to Level i or x
h_i and h_x = the height (ft or m) from the base to Level i or x
k = an exponent related to the structure period as follows:
for structures having a period of 0.5 s or less, $k = 1$
for structures having a period of 2.5 s or more, $k = 2$

For structures having a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

3.2.2. Analysis type method and parameters: -

Specify the toughness type along with the detail type using the AS1170.0, and in general, there are three methods covering all the type of structures which relies on the Earthquake Design Category EDC: -

- EDC1 analysis of static simple by taking ten per cent of the total building weight.
- EDC2 analysis of static earthquake.
- EDC3 analysis of dynamic earthquake.

The last two methods can be defined as mentioned in the figure (3.3) this earthquake elastic response most be divided by the RF factor which is a result of multiplying the ductility factor with the strength factor in order to reach the inelastic response and as mentioned in the table (3.12) Rodsin, K., Lam, N.T.K., Wilson, J.L. and Goldsworthy, H.M. (2003).

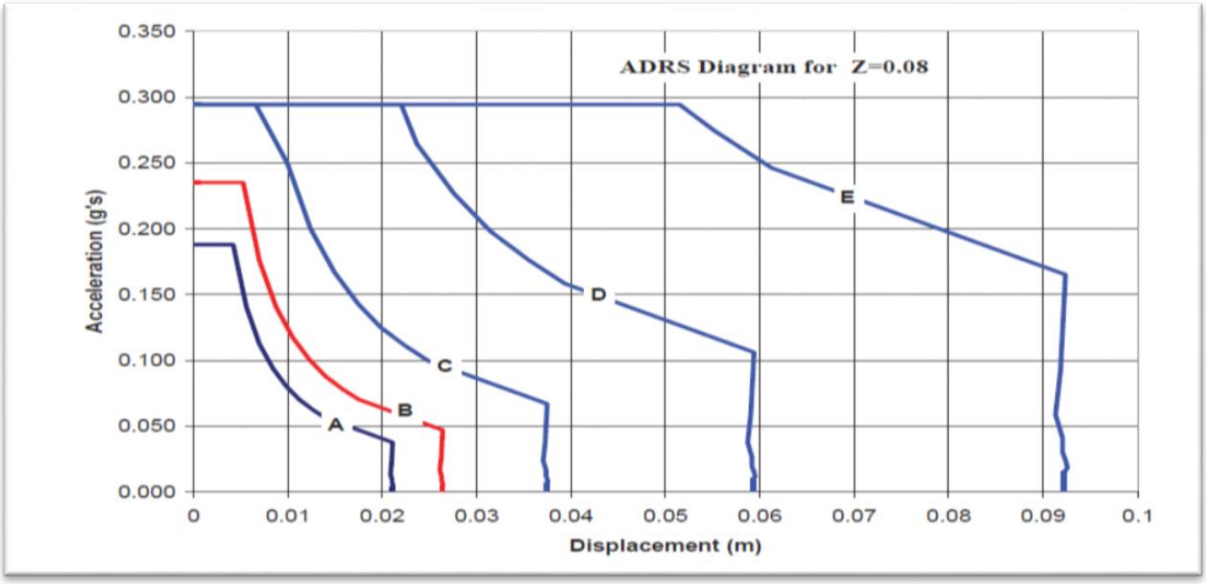


Figure (3.3) response design spectra with Z 0.08 addressed in ADRS set-up

System type	**Overstrength factor Ω**	**Ductility factor μ**	**RF factor = Ω x μ**
Ductile system	4	1.5	6
middle ductile system	3	1.5	4.5
Limited ductile system	2	1.3	2.6
Un-reinforced masonry system	1.25	1.3	1.6

Table (3.12) Over strength factor Ω and Ductility factor μ as per the AS1170.4, 07

In general, most buildings design using the first two concepts excluding the tall structures where the high mode of influence is significant. AS1170.4 agrees to check the base displacement check for seismic effects along with wind and vertical load effects.

Another design method is available such as the nonlinear/ pushover method which relies on the displacement and used for the low seismic zone and middle needs of displacement. Displacement-based design technique DB is a very simple way to check structure attitude using the ultimate limit state and its look like a massive improvement on the further indirect force-based technique FB by implementing the ductile factor along with the overstrength factor. In this technique, it is necessary to module the building as a single degree of freedom SDOF and the seismic structure attitude evaluated throughout checking the displacement needs with the calculated displacement capacity of the building. It is possible to get the capacity displacement of the structure Δc from this method where engineers estimate displacement because of growing the horizontal force till the building failed which presumed to occur when the building failed to resist the gravity loads and collapse. The outcome of the displacement forces is usually called the pushover figure which represents the building capacity of deformation, and it is possible to transfer it into an accelerate-displace figure by regularising the shear base respecting the structure mass. Estimation in growing the transferred capacity curvature depend on the material type without including the inelastic and elastic deformation of the building composed of the defection of the P-delta and fixable footing effects. To assess the structure, it is required to compare the max. Displacement needs PDD with the capacity displacement Δc and incases the capacity more than the demand; the structure is safe. Otherwise, it's suggested to use the spectrum capacity technique CSM to assess the structure and its required to keep the CSM above the Δc to consider the structure safe. The used damping percentage is five-per-cent, and as per the new research, the damping in Australia does not exceed the ten per-cent.

Then, it is required to demonstrate the hazard factor site sub-soil class, the height of the structure, probability factor, and the site sub-soil type which contain five types, (A) Hard rock, (B) Rook, (C) shallow, (D) Deep, and (E) very soft. For the hazard factor, it is equal to the peak ground motion acceleration impacts with the period return of five hundred years

Importance level, type of structure (see Clause 2.2)	(k_pZ) for site sub-soil class				Structure height, h_n (m)	Earthquake design category
	E_e or D_e	C_e	B_e	A_e		
1	—				—	Not required to be designed for earthquake actions
Domestic structure (housing)	—				Top of roof ≤8.5	Refer to Appendix A
					Top of roof >8.5	Design as importance level 2
2	≤0.05	≤0.08	≤0.11	≤0.14	≤12 >12, <50 ≥50	I II III
	>0.05 to ≤0.08	>0.08 to ≤0.12	>0.11 to ≤0.17	>0.14 to ≤0.21	<50 ≥50	II III
	>0.08	>0.12	>0.17	>0.21	<25 ≥25	II III
3	≤0.08	≤0.12	≤0.17	≤0.21	<50 ≥50	II III
	>0.08	>0.12	>0.17	>0.21	<25 ≥25	II III
4	—				<12 ≥12	II III

which is connected to Peak Ground

Table (3.13) of Earthquake Design Selection Categories (AS1170.4, 7, T2.1)

Velocity (PGV) and the factor is 0.1g equal to PGV=75mm/sec. (see Table (2.4)). Lam, N.T.K. and Wilson, J.L. (1999).

The American code has another way of calculation, the Ss and the S_1 can be obtained from the USGS report, and then it is essential to specify the soil type and long-period transition period T_L. While the risk targeted the max, consideration earthquake MCE_R response spectral acceleration and the coefficient site will be as per tables 3.14, and 15 and the values will be 1.0 and 1.3 for Fa and Fv respectively. Using equation 1.17 throughout 20, the below outcomes is the spectral designed acceleration limits:

$S_{MS} = FaSs$

$S_{M1} = FvS_1$

$S_{DS} = \frac{2}{3} S_{MS}$

$S_{D1} = \frac{2}{3} S_{M1}$

The assumption of the type of this building regarding a risk category for an earthquake, occupancy category is as per table 3.15 and based on table 3.16 the important factor can be determined. The seismic category design which depends on the short response period acceleration limit and one second can be implemented from table 3.17 and the Seismic Design Category (SDC).

Using Table 12.2-1 from the ASCE7, the coefficient design and seismic forces resistance factor system can be specified by determining the response modification coefficient R, deflection amplification coefficient Cd, and the Over Strength Factor Ω_0 and the detail requirements described in the ASCE7 and table 2.18 will lead us to use the proper analysis method wither using the equivalent lateral force procedure (static), the response spectrum procedure (dynamic) or both.

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9[a]	Seismic Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding 2 stories above the base	P	P	P
	Structures of light frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft in structural height	P	P	P
	Structures exceeding 160 ft in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

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

Table (3.14) seismic category design which depends on the short response period

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

acceleration limit and one second respectively

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

Table (3.15) Allowable Analyse Procedures

To determine the earthquake action using the equivalent lateral force method (ELF), the Australian code suggested a chain of the parameter to be calculated: -

3.2.2.1. Loads and load combinations

Suitable load cases and the combination should be specified before starting the analysis. As per the code, its suggested to take section 12.5.3&4, and that means, using four type of load directions, two orthogonal with negative and positive in each direction which will cause torsion eventually. In total, it will be required eight load combinations along with accidental torsion forces on the building. It is important to use the thirty per cent in each opposite direction in these calculations which will lead to increase the number of the combination at lease sixteen combo. Using chapter two from the code, two combinations can be found in the strength design and allowable stress design. For the first one as shown below Building Seismic Safety Council, (2009), 2009 NEHRP Recommended Seismic Provisions for Buildings and Other Structures, prepared for the Federal Emergency Management Agency, Washington, DC.

1.2D + 1.0E + 1.0L + 0.2S

0.9D + 1.0E+1.6H

It is allowed to take the live load as fifty per cent in case the load exceed the five KN, and it is possible to take the earth load when the structure contains a basement. It is important to know the ASCE7 divided the seismic load into two portions horizontal and vertical and as below

$Eh = \rho QE$ where QE earthquake load effect, ρ redundancy factor

$Ev = 0.2SDSD$

Because of substituting the equation into the general combination

$(1.2 + 0.2SDS) D + \rho QE + 0.5L + 0.2S$

$(0.9 - 0.2SDS) D + \rho QE$

Taking S_{DS} = 1.19867 and ignoring the basic snow combinations for strength design become:

$1.44D + 0.5L + \rho QE$

$0.66D + \rho QE$

Redundancy factor ρ, obtained from Section 12.3.4, so ρ is 1.0 and the final combination is

$1.44D + 0.5L + 1.0QE$

$0.66D + 1.0QE$

On the other hand, the Australian code suggested to take AS1170.0 section 4.2.1&2 that means, using four type of load directions, two orthogonal with negative and positive in each direction which will cause torsion eventually. In total, it will be required eight load combinations along with accidental torsion forces on the building. It is important to use the thirty per cent in each opposite direction in these calculations which will lead to increase the number of the combination at lease sixteen combo. Using chapter two from the code, two combinations can be found in the strength design and allowable stress design. For the first one as shown below:

4.2 COMBINATIONS OF ACTIONS FOR ULTIMATE LIMIT STATES

4.2.1 Stability

The basic combinations for the ultimate limit states used in checking stability (see Clause 7.2.1) shall be as follows where the long-term and combination factors are given in Table 4.1:

(a) For combinations that produce net stabilizing effects ($E_{d,stb}$):

$E_{d,stb} = [0.9G]$ permanent action only (does not apply to prestressing forces)

(b) For combinations that produce net destabilizing effects ($E_{d,dst}$):

(i) $E_{d,dst} = [1.35G]$ permanent action only (does not apply to prestressing forces)

(ii) $E_{d,dst} = [1.2G, 1.5Q]$ permanent and imposed action

(iv) $E_{d,dst} = [1.2G, W_u, \psi_c Q]$ permanent, wind and imposed action

A3 (v) $E_{d,dst} = [G, E_u, \psi_E Q]$ permanent, earthquake and imposed action

(vi) $E_{d,dst} = [1.2G, S_u, \psi_c Q]$ permanent action, actions given in Clause 4.2.3 and imposed action

NOTE: Combination factors for prestressing forces are given in the appropriate materials design Standard.

4.2.2 Strength

The basic combinations for the ultimate limit states used in checking strength (see Clause 7.2.2) shall be as follows, where the long-term and combination factors are given in Table 4.1:

(a) $E_d = [1.35G]$ permanent action only (does not apply to prestressing forces)

(b) $E_d = [1.2G, 1.5Q]$ permanent and imposed action

(c) $E_d = [1.2G, 1.5\psi_\ell Q]$ permanent and long-term imposed action

(d) $E_d = [1.2G, W_u, \psi_c Q]$ permanent, wind and imposed action

(e) $E_d = [0.9G, W_u]$ permanent and wind action reversal

A3 (f) $E_d = [G, E_u, \psi_E Q]$ permanent, earthquake and imposed action

(g) $E_d = [1.2G, S_u, \psi_c Q]$ permanent action, actions given in Clause 4.2.3 and imposed action

A3 NOTES:

1 Combination factors for prestressing forces are given in the appropriate materials design Standard.

2 Refer to AS/NZS 1170.1, Clause 3.3.

3.2.2.2. Seismic Ground Motion Parameters

As per the USGS, the S_s = 1.798 g, S_1 = 0.738 g (see report below), soil type = C (from the question), and long-period transition period T_L=8 seconds. While the risk targeted the max.

Consideration earthquake MCE_R response spectral acceleration and the coefficient site will be as per tables 2.13, and 14 and the values will be 1.0 and 1.3 for Fa and Fv respectively. Using equation 1.17 throughout 20, the below outcomes is the spectral designed acceleration limits:

$S_{MS} = FaSs = 1.0(1.798) = 1.798$

$S_{M1} = FvS_1 = 1.3(0.738) = 0.9594$

$S_{DS} = \frac{2}{3} S_{MS} = 1.19867$

$S_{D1} = \frac{2}{3} S_{M1} = 0.6396$

The assumption of the type of this building regarding a risk category for an earthquake, occupancy category will be II (as per table 3.16) and based on table the important factor will be considered as one. The seismic category design which depends on the short response period acceleration limit and one second can be implemented from table 2.17, and the Seismic Design Category (SDC) will be D.

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

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

Table (3.16) seismic category design which depends on the short response period acceleration limit and one second respectively

Using Table 12.2-1 from the ASCE7, the coefficient design and seismic forces resistance factor system can be specified by determining the response modification coefficient R, deflection amplification coefficient Cd, and the Over Strength Factor Ω_0 which is 3½, 2½, and 2.25 respectively and the detail requirements described in section 14.4 from the ASCE7. These outcomes adopted based on our section of the frame system which is "BEARING WALL SYSTEMS by using Intermediate reinforced masonry shear walls."

Based in the ASCE7n code, the smallest analysis procedure can be found in table 2.18 which indicate to us that using the equivalent lateral force procedure (static) and the response spectrum procedure (dynamic) is compulsory. Alternatively, building regularity divided into two components vertical and horizontal regularity and can be found in table 2.19&20. (Applied Technology Council, 1978).

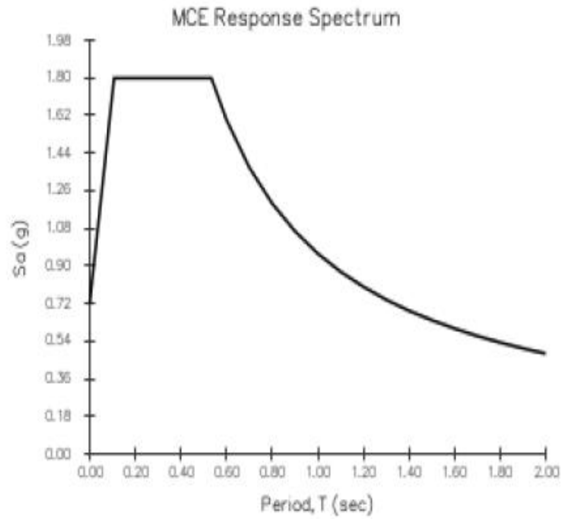
USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document	ASCE 7-05 Standard (which utilizes USGS hazard data available in 2002)
Site Coordinates	33.7217°N, 116.21936°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Occupancy Category	I/II/III

USGS–Provided Output

S_S =	1.798 g	S_{MS} =	1.798 g	S_{DS} =	1.199 g
S_1 =	0.738 g	S_{M1} =	0.959 g	S_{D1} =	0.640 g



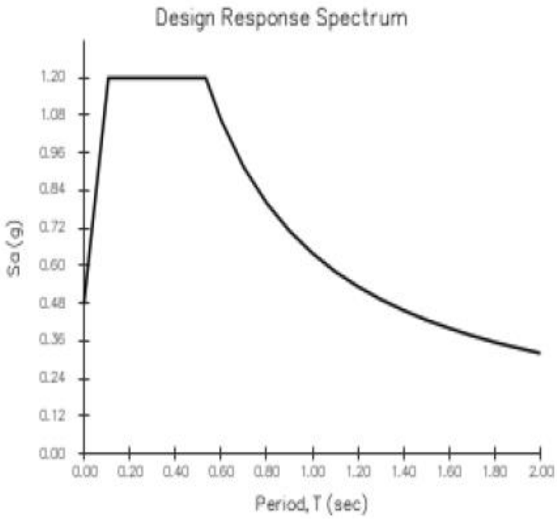


Figure (3.4) outcomes from the USGS

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity:** Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F B, C, D, E, and F C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity:** Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity:** Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity:** Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one having a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offset Irregularity:** Out-of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.	12.3.3.3 12.3.3.4 12.7.3 Table 12.6-1 Section 16.2.2	B, C, D, E, and F D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel System Irregularity:** Nonparallel system irregularity is defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Table (3.17) Allowable Analyse Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9[a]	Seismic Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding 2 stories above the base	P	P	P
	Structures of light frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft in structural height	P	P	P
	Structures exceeding 160 ft in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

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

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Stiffness-Soft Story Irregularity:** Stiffness-soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	**Stiffness-Extreme Soft Story Irregularity:** Stiffness-extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
2.	**Weight (Mass) Irregularity:** Weight (mass) irregularity is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
3.	**Vertical Geometric Irregularity:** Vertical geometric irregularity is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity:** In-plane discontinuity in vertical lateral force-resisting elements irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting beam, column, truss, or slab.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity:** Discontinuity in lateral strength–weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity:** Discontinuity in lateral strength–extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

Table (3.18) Horizontal Structural Irregularities

From the above table, the building is irregular horizontal in the Re-entrant Corner Irregularity condition, and it is possible that the structure will be irregular horizontally in the torsion definition which will be check later. The analysis can decide the vertical regularity.

As per the AS1170.4, it is essential to follow the procedure mentioned below to start the analysis. So, the first step is to find the probability factor (K_P) as shown in table 3.19 and our case, K_P is 1.3 since we will design with a probability of 1/1000 year. The next important factor we should specify it is the hazard factor (Z) which will be 0.09 based on the selected area. Then, we can use table 3.20 to determine the ECD earthquake design category which will be III after we selected the soil type which is C. Finally, and after following figure 3.5, the code instructed us to use the dynamic analysis by following the section-7.

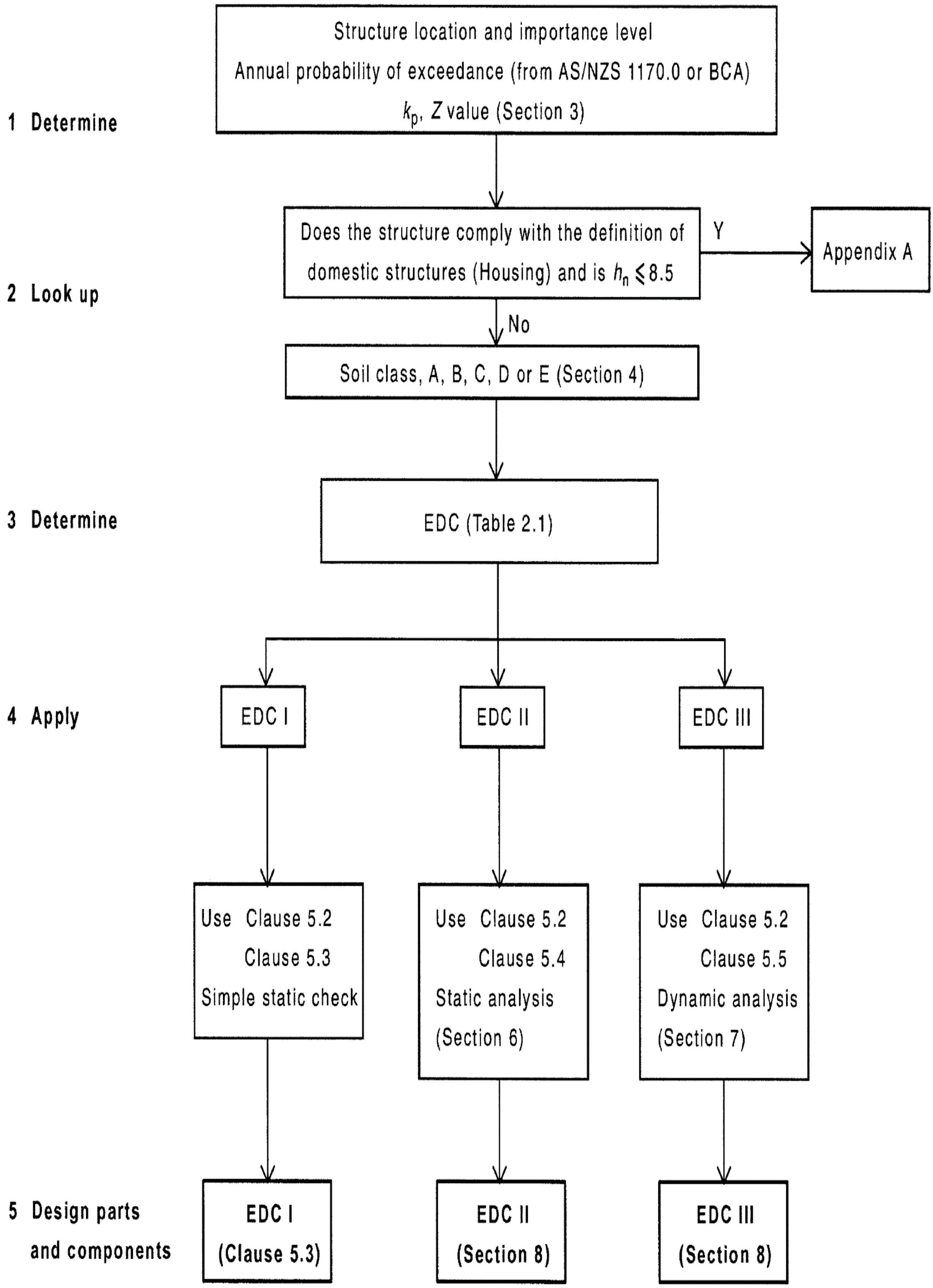


Figure (3.5) Design procedure as per the AS1170.4

Importance level, type of structure (see Clause 2.2)	$(k_p Z)$ for site sub-soil class				Structure height, h_n (m)	Earthquake design category
	E_e or D_e	C_e	B_e	A_e		
1	—				—	Not required to be designed for earthquake actions
Domestic structure (housing)	—				Top of roof ≤8.5	Refer to Appendix A
					Top of roof >8.5	Design as importance level 2
2	≤0.05	≤0.08	≤0.11	≤0.14	≤12 >12, <50 ≥50	I II III
	>0.05 to ≤0.08	>0.08 to ≤0.12	>0.11 to ≤0.17	>0.14 to ≤0.21	<50 ≥50	II III
	>0.08	>0.12	>0.17	>0.21	<25 ≥25	II III
3	≤0.08	≤0.12	≤0.17	≤0.21	<50 ≥50	II III
	>0.08	>0.12	>0.17	>0.21	<25 ≥25	II III
4	—				<12 ≥12	II III

NOTES:

1 Values for k_p and Z are given in Section 3. Site sub-soil class are given in Section 4.

2 A higher earthquake design category or procedure may be used in place of that specified.

3 Height (h_n) is defined in Clause 1.5. For domestic structures refer to Appendix A.

4 In addition to the above, a special study is required for importance level 4 structures to demonstrate they remain serviceable for immediate use following the design event for importance level 2 structures.

Table (3.19) finding the EDC earthquake design category

Location	Z	Location	Z	Location	Z
Adelaide	0.10	Geraldton	0.09	Port Augusta	0.11
Albany	0.08	Gladstone	0.09	Port Lincoln	0.10
Albury/Wodonga	0.09	Gold Coast	0.05	Port Hedland	0.12
Alice Springs	0.08	Gosford	0.09	Port Macquarie	0.06
Ballarat	0.08	Grafton	0.05	Port Pirie	0.10
Bathurst	0.08	Gippsland	0.10	Robe	0.10
Bendigo	0.09	Goulburn	0.09	Rockhampton	0.08
Brisbane	0.05	Hobart	0.03	Shepparton	0.09
Broome	0.12	Karratha	0.12	Sydney	0.08
Bundaberg	0.11	Katoomba	0.09	Tamworth	0.07
Burnie	0.07	Latrobe Valley	0.10	Taree	0.08
Cairns	0.06	Launceston	0.04	Tennant Creek	0.13
Camden	0.09	Lismore	0.05	Toowoomba	0.06
Canberra	0.08	Lorne	0.10	Townsville	0.07
Carnarvon	0.09	Mackay	0.07	Tweed Heads	0.05
Coffs Harbour	0.05	Maitland	0.10	Uluru	0.08
Cooma	0.08	Melbourne	0.08	Wagga Wagga	0.09
Dampier	0.12	Mittagong	0.09	Wangaratta	0.09
Darwin	0.09	Morisset	0.10	Whyalla	0.09
Derby	0.09	Newcastle	0.11	Wollongong	0.09
Dubbo	0.08	Noosa	0.08	Woomera	0.08
Esperance	0.09	Orange	0.08	Wyndham	0.09
Geelong	0.10	Perth	0.09	Wyong	0.10
Meckering region				**Islands**	
Ballidu	0.15	Meckering	0.20	Christmas Island	0.15
Corrigin	0.14	Northam	0.14	Cocos Islands	0.08
Cunderdin	0.22	Wongan Hills	0.15	Heard Island	0.10
Dowerin	0.20	Wickepin	0.15	Lord Howe Island	0.06
Goomalling	0.16	York	0.14	Macquarie Island	0.60
Kellerberrin	0.14			Norfolk Island	0.08

Table (3.20) seismic category design which depends on the short response period acceleration limit and one second respectively

Based in the AS1170.4 code, the smallest analysis procedure can be found in section 6.1 which indicate that we have to find the time period of the structure and another structure requirement to be used in the equivalent lateral force procedure (static) and the response spectrum procedure (dynamic) is compulsory. Paulay T. and Priestley M.J.N. (1992).

3.2.2.3.Response Spectra and Spectral Shape Factor C_h (T): -

Elastic response spectra, designers commonly used the response acceleration spectra to show the earthquake motion which is represented by the acceleration/displacement or acceleration/velocity curvatures in contradiction of period response of a SDOF oscillator measured to characterize the building Figure which is created by estimating the response of one mass oscillator with five percentage of damping see figure (3.6).

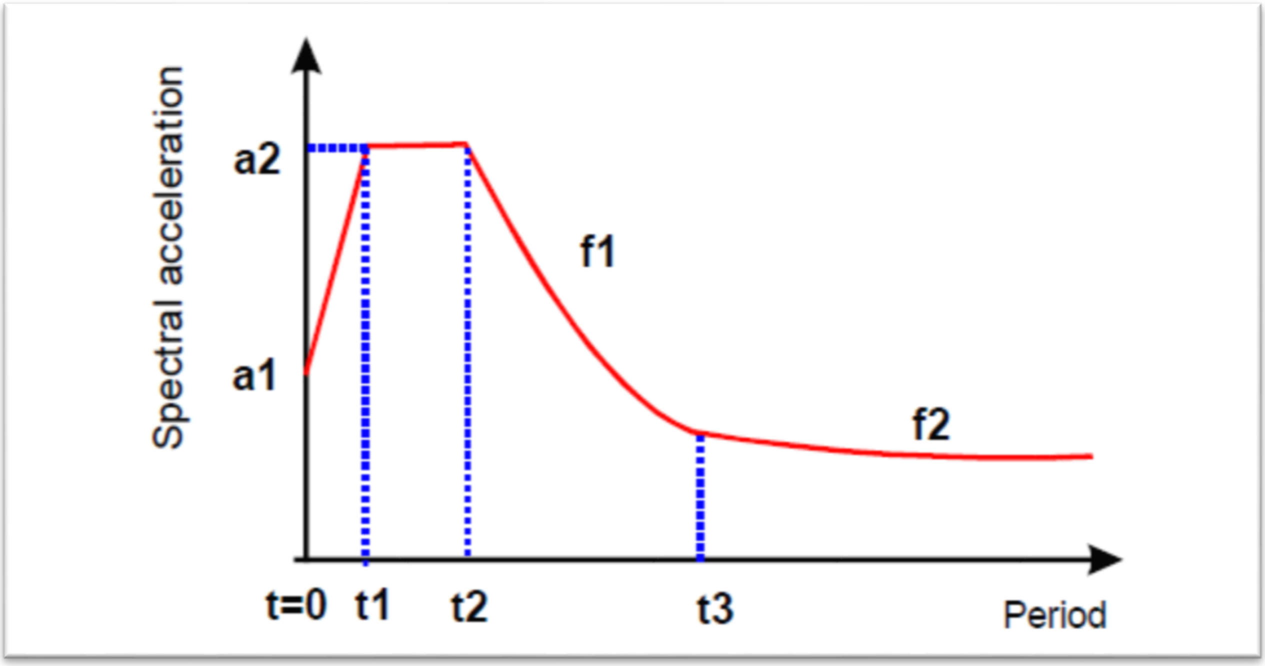


Figure (3.6) Parameter Acceleration Spectrum Response

Response modification; Based on the Australian code AS1170.4,2007, the response spectra design updated meaningfully along with improvements in response acceleration calculation, velocity, and significantly displacement for a specified position and site. Also, the low-moderate seismic zone reflected better than before. The response factor for buildings R_f standardised and became much more accessible for design engineers to use it mainly for the push-over non-linear curvature in order to obtain a better calculation whenever necessary.

The philosophy of the building response is a comprehensive subject that can be defined in deepness but fast, instead of detailed. The connection between the ductility to building behaviour ratio and a factor of reduction R_f can be illustrated in figure (3.7). Lam & Wilson proved that the factor of building response Rf defined by using a factor of over-strength Ω and factor of ductility μ with the same New Zealand notation. Griffith, M.C., Wu, Y. and Oehlers D. (2003).

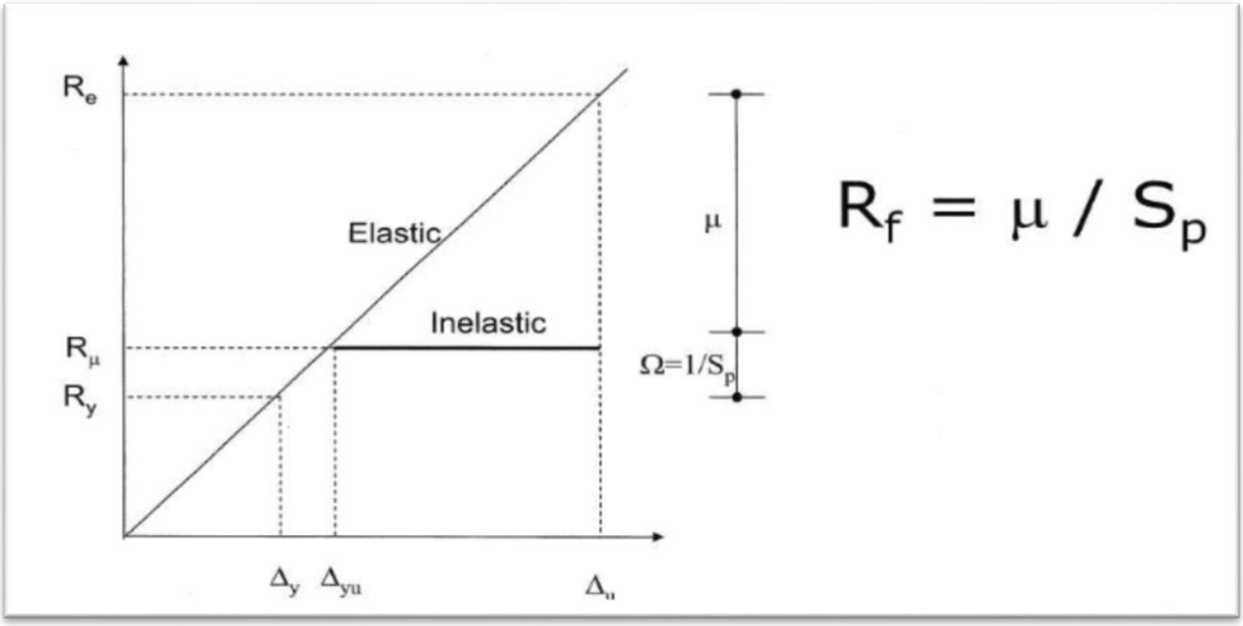


Figure (3.7) comparison of the Rf and μ/Sp relationship

There are a few divisions inside the ASCE7 designed response spectrum (fig.3.8) these divisions located between the ($T_0 = 0.2S_{D1}/S_{DS}$) and ($T_S = S_{D1}/S_{DS}$), the acceleration response is fix and identical to S_{DS}. Continuous velocity division overlays the period group of (T_s-T_L) while the acceleration response of this group is proportionate to (1/T) and one second period is equivalent to S_{D1}.

For the Long period of spectrum response, it can be identified base on the T_L limitation, the period that represents the moving from continuous velocity division to continuous displacement division of the design spectrum response. The accelerated response in the continuous displacement division ($T \geq T_L$), is proportionate to($1/T^2$) and T_L values can be obtained from ASCE7 (Fig.22,12 to 16) which is plotted based on two-stage techniques, the relationship between the seismic degree and the recognized T_L and then the modal degree from seismic hazard classification motion ground for two seconds period.

To calculate the design response spectrum using the ASCE7 without the needs specific site motion ground, curvature can be created as figure 3.8 and follow the procedure below:

- find the Sa, design spectral acceleration response for period lower than T_0 as mentioned below:

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right)$$ EQ. (1.21)

- Find the SA, design spectral acceleration response for a period more than T_0 and lower or same as T_S which is the same as S_{DS}.
- find the Sa, design spectral acceleration response for a period more than T_S and lower same as T_L as mentioned below

$$S_a = \frac{S_{D1}}{T}$$ EQ. (1.22)

- find the Sa, design spectral acceleration response for a period more than T_L as mentioned below

$$S_a = \frac{S_{D1}T_L}{T^2}$$ EQ. (1.23)

T = the essential period of the building in the second

T_L = long period changeover period in second exposed in ASCE7, Figures (22, 12 – 22, 16). $T_0 = 0.2\frac{S_{D1}}{S_{DS}}$ And $T_S = \frac{S_{D1}}{S_{DS}}$

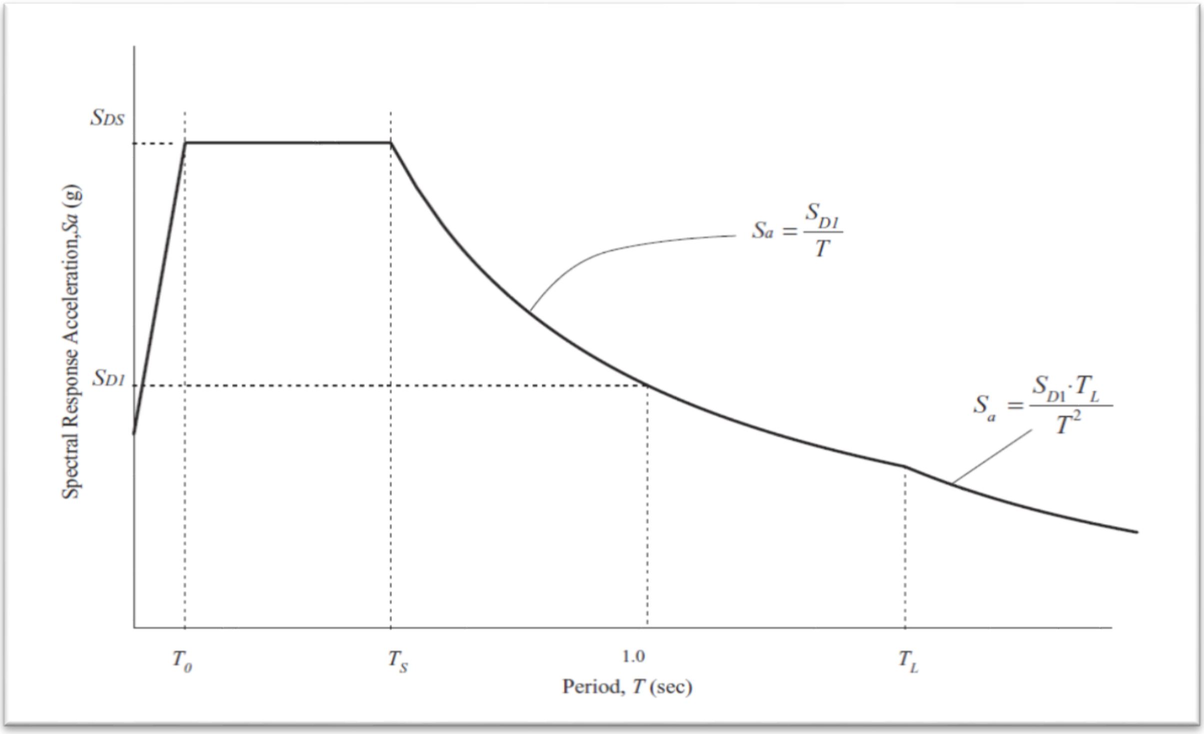


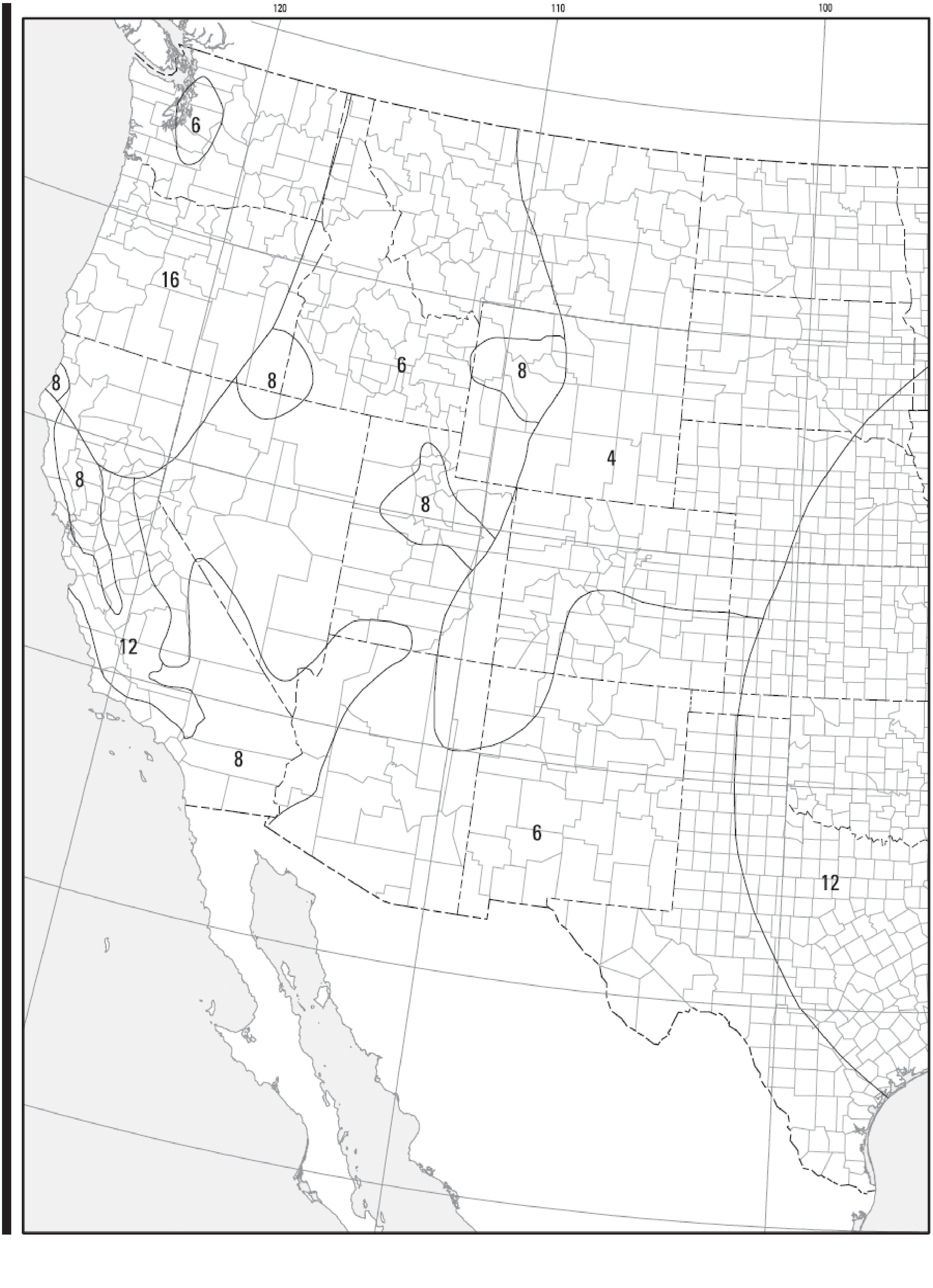
Figure (3.8) response spectrum design

Based on the American slandered, The Spectral Acceleration Design limits to design structure implemented for earthquake purpose needed 2/3 MCER spectral response which is described in sec11.4.4. By obtaining the additional limits SDS & SD1 and as shown in the equation below.

$$S_{DS} = \frac{2}{3} S_{MS}$$ EQ. (1.19)

$$S_{D1} = \frac{2}{3} S_{M1}$$ EQ. (1.20)

While the time period for 1.0 second which is based on using the site position, the long period transaction for one second T_L can be specified using the figure above (figure 2.10).


120
110
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12

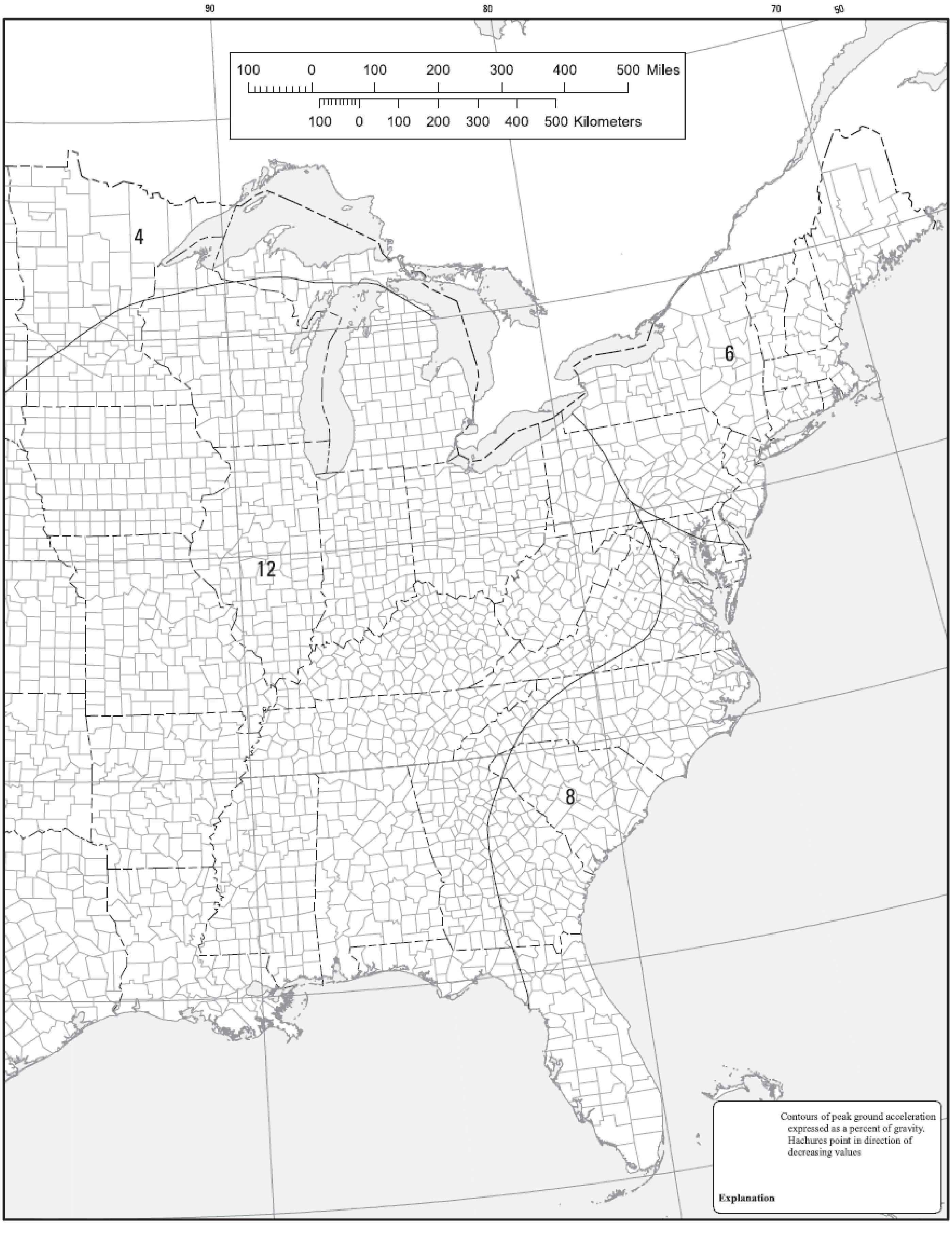


Figure (3.9) long Period 1.0 second TL value

The PGA from the ASCE7 implemented to estimate the possible for soil liquefaction/ strength damage and to specify lateral earth stress for retaining walls and basement design. By having the site passion, the values of the MCER motion ground can be found from ASCE7 figure 22-8. Adopting the PGA value and site classification, the FPGA coefficient can be found in line with sec.11.8.3, table 11.8-1, and equation 11.8-1, PGAM can be found. The value adopted to estimate the possible soil liquefaction/ strength damage. The PGA used to estimate the seismic dynamic lateral earth stress for retaining walls, and basement design is calculated as

$$2/3\ PGAM = 2/3\ PGA\ g \qquad \text{EQ. (1.24)}$$

ASCE7, Sec.11.8.3 mentioned that PGA specified by whichever a site precise instigation considering the effects soil amplification or from Equation 1.25, with MCE_G PGA is found from national maps of PGA for bedrock B Site Class times F_{PGA} site coefficient gain for other site classes PGA_M.

$$PGA_M = F_{PGA}\ PGA \qquad \text{EQ. (1.25)}$$

Alternatively, building regularity divided into two components vertical and horizontal regularity and can be found in table 2.19&20. (Applied Technology Council, 1978).

Also, the Australian code required to contain the ductility capacity as a minimum equivalent to the matching assumed reduction force factor wither the designer used the in-elastic or reduced spectra. When the Sp/μ value reduces or in another term, increasing the factor of building response and μ/Sp, the building will imbibe the growing energy and consequently is calculated for a lesser amount of direct actions nevertheless for extra plastic volume capacity.

System	Ductility (μ)	Over-strength (Ω)	response factor $R_f = \mu * \Omega$
URM	1.25	1.3	1.6
Limited Ductility	2	1.3	2.6
Moderate Ductility	3	1.5	4.5
Ductile	4	1.5	6

Table (3.21) μ and Ω used in the AS1170.4, 2007 without using it in the notation

The AS1170.4,2007 introduced a new classification and name it “Limited Ductility” which contain a ductility μ is two, and this needs a basic detail and specification of standard materials from the AS3600, 2001 Concrete Structures. At the same time, the μ of the “Moderate Ductility” is three which needs special details as mentioned in the AS3600, 2001 Concrete Structures. The last type is the “Full Ductility” with ductility μ is four which is eliminated from the AS1170.4,2007, and it is possible to study it using the New Zealand standard NZ1170.5, where advanced procedures are implemented to obtain plastic capacity and possible ductility at the connection point and specific hinges. It is highly complicated to make the details in order to reach the desired ductility on the other hand and regarding the ductility value one, the building assessed to be full elastic during the full actions. Below in figure (3.10), the relationship between the ductility and the strength presented.

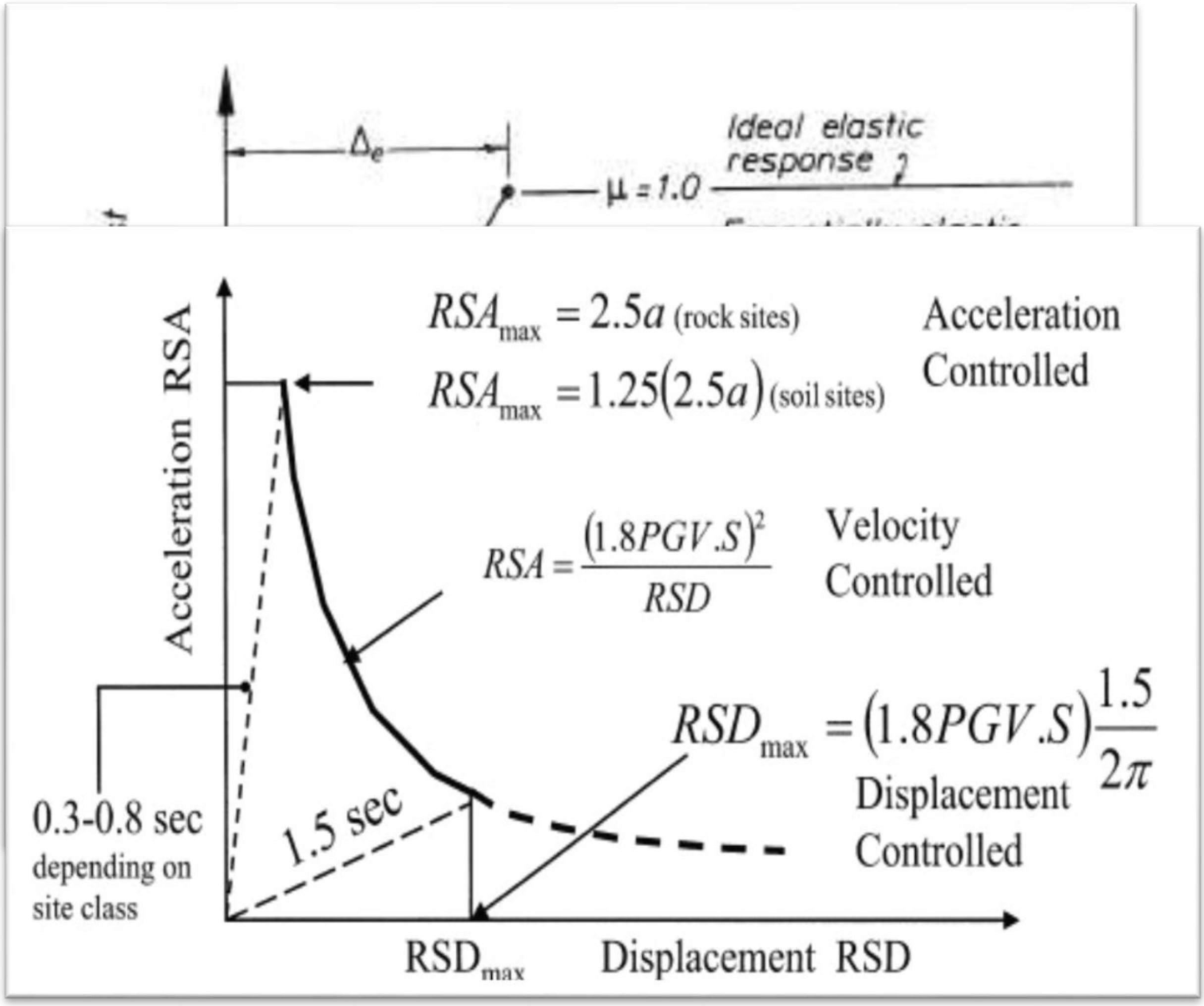


Figure (3.10) comparison of the Rf and μ/Sp relationship

The AS117.4, 2007 presented the spectral shape factor taking into the consideration, the building period and condition of the site. The site reverberation contains a significant impact on the ground motion magnifying.

The old code AS1170.4,1993 is conventional regarding response displacement and velocity raised at the same time with the rise of the natural period which is not physically correct. Figure (3.11) show us the response spectra equation of each stage. In this figure, the location coefficient of the controlled acceleration zone of response spectrum is Fa, and the location coefficient of the displacement-controlled zone and the velocity of the response spectrum is Fv.

Figure (3.11) required curvature symmetrical to the AS1170.4 model

RSVmax = 1.8*PGV*Fv

RSDmax = $\frac{T2}{2\pi}$ *RSVmax

PGV= peak ground velocity = 750KpZ.

Wilson & Lam designated the RSAmax as three times the kpZFa value, using three instead of two and helve indicated a good knowledge phenomenon of high magnification spectral in short period array with incorporate earthquakes.

Calculating the acceleration response spectrum equation for the AS11700.4, 2007 is a revise from the 1993 version which is more accurate for the response level to illustrate the resonance.

The spectral shape factor as a function of period C_h (T) can be taken from the AS1170.4 table 6.8 or figure 6.4 as shown below

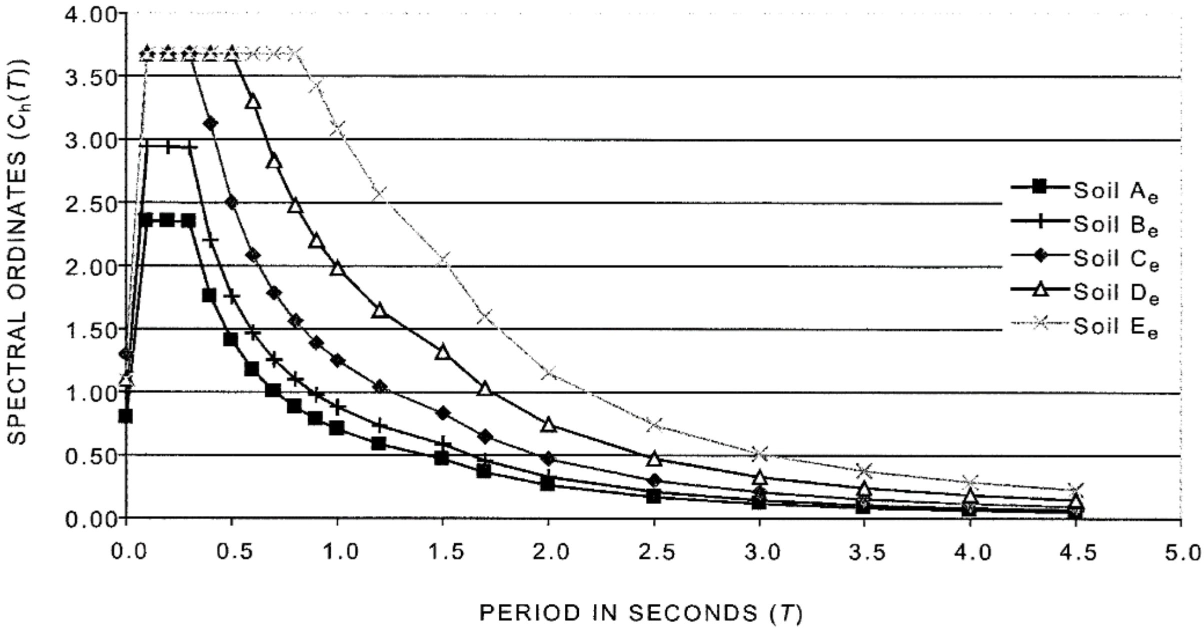


Figure (3.12) normalised response spectra for site SUB-Soil class6

	Site sub-soil class				
Period (seconds)	A_e Strong rock	B_e Rock	C_e Shallow soil	D_e Deep or soft soil	E_e Very soft soil
0.0	2.35 (0.8)*	2.94 (1.0)*	3.68 (1.3)*	3.68 (1.1)*	3.68 (1.1)*
0.1	2.35	2.94	3.68	3.68	3.68
0.2	2.35	2.94	3.68	3.68	3.68
0.3	2.35	2.94	3.68	3.68	3.68
0.4	1.76	2.20	3.12	3.68	3.68
0.5	1.41	1.76	2.50	3.68	3.68
0.6	1.17	1.47	2.08	3.30	3.68
0.7	1.01	1.26	1.79	2.83	3.68
0.8	0.88	1.10	1.56	2.48	3.68
0.9	0.78	0.98	1.39	2.20	3.42
1.0	0.70	0.88	1.25	1.98	3.08
1.2	0.59	0.73	1.04	1.65	2.57
1.5	0.47	0.59	0.83	1.32	2.05
1.7	0.37	0.46	0.65	1.03	1.60
2.0	0.26	0.33	0.47	0.74	1.16
2.5	0.17	0.21	0.30	0.48	0.74
3.0	0.12	0.15	0.21	0.33	0.51
3.5	0.086	0.11	0.15	0.24	0.38
4.0	0.066	0.083	0.12	0.19	0.29
4.5	0.052	0.065	0.093	0.15	0.23
5.0	0.042	0.053	0.075	0.12	0.18
Equations for spectra					
$0 < T \le 0.1$	$0.8 + 15.5T$	$1.0 + 19.4T$	$1.3 + 23.8T$	$1.1 + 25.8T$	$1.1 + 25.8T$
$0.1 < T \le 1.5$	$0.704/T$ but ≤ 2.35	$0.88/T$ but ≤ 2.94	$1.25/T$ but ≤ 3.68	$1.98/T$ but ≤ 3.68	$3.08/T$ but ≤ 3.68
$T > 1.5$	$1.056/T^2$	$1.32/T^2$	$1.874/T^2$	$2.97/T^2$	$4.62/T^2$

* Values in brackets correspond to values of spectral shape factor for the modal response spectrum and the numerical integration time history methods and for use in the method of calculation of forces on parts and components (see Section 8)

Table (3.13) spectral shape factor as a function of period Ch (T)

The structural ductility factor (μ) is the second required parameter along with the structural performance factor (S_P) which can be determined either through the provided data from the material standard or as mentioned in the Australian code AS1170.4 table 6.5(A&B) in case the data not available apart from that, for a precise building, its allowed to determine this parameter as a nonlinear static for analysis Griffith, M.C., Wu, Y. and Oehlers D. (2003).

Type of structure	μ	S_p	μ/S_p	S_p/μ
Tanks, vessels or pressurized spheres on braced or unbraced legs	2	1	2	0.5
Cast-in-place concrete silos and chimneys having walls continuous to the foundation	3	1	3	0.33
Distributed mass cantilever structures, such as stacks, chimneys, silos and skirt-supported vertical vessels	3	1	3	0.33
Trussed towers (freestanding or guyed), guyed stacks and chimneys	3	1	3	0.33
Inverted pendulum-type structures	2	1	2	0.5
Cooling towers	3	1	3	0.33
Bins and hoppers on braced or unbraced legs	3	1	3	0.33
Storage racking	3	1	3	0.33
Signs and billboards	3	1	3	0.33
Amusement structures and monuments	2	1	2	0.5
All other self-supporting structures not otherwise covered	3	1	3	0.33

Table (3.14) structure ductility factor (μ) and the structural performance factor (SP) - basic structures

Structural system	Description	μ	S_p	S_p/μ	μ/S_p
Steel structures					
	Special moment-resisting frames (fully ductile)*	4	0.67	0.17	6
	Intermediate moment-resisting frames (moderately ductile)	3	0.67	0.22	4.5
	Ordinary moment-resisting frames (limited ductile)	2	0.77	0.38	2.6
	Moderately ductile concentrically braced frames	3	0.67	0.22	4.5
	Limited ductile concentrically braced frames	2	0.77	0.38	2.6
	Fully ductile eccentrically braced frames*	4	0.67	0.17	6
	Other steel structures not defined above	2	0.77	0.38	2.6
Concrete structures					
	Special moment-resisting frames (fully ductile)*	4	0.67	0.17	6
	Intermediate moment-resisting frames (moderately ductile)	3	0.67	0.22	4.5
	Ordinary moment-resisting frames	2	0.77	0.38	2.6
	Ductile coupled walls (fully ductile)*	4	0.67	0.17	6
	Ductile partially coupled walls*	4	0.67	0.17	6
	Ductile shear walls	3	0.67	0.22	4.5
	Limited ductile shear walls	2	0.77	0.38	2.6
	Ordinary moment-resisting frames in combination with a limited ductile shear walls	2	0.77	0.38	2.6
	Other concrete structures not listed above	2	0.77	0.38	2.6
Timber structures					
	Shear walls	3	0.67	0.22	4.5
	Braced frames (with ductile connections)	2	0.77	0.38	2.6
	Moment-resisting frames	2	0.77	0.38	2.6
	Other wood or gypsum based seismic-force-resisting systems not listed above	2	0.77	0.38	2.6
Masonry structures					
	Close-spaced reinforced masonry†	2	0.77	0.38	2.6
	Wide-spaced reinforced masonry†	1.5	0.77	0.5	2
	Unreinforced masonry†	1.25	0.77	0.62	1.6
	Other masonry structures not complying with AS 3700	1.00	0.77	0.77	1.3

* The design of structures with $\mu > 3$ is outside the scope of this Standard (see Clause 2.2)

† These values are taken from AS 3700

Table (3.15) structure ductility factor (μ) and the structural performance factor (SP) - basic structures

3.2.2.4.Probability factor and Hazard factor

Also, it's essential to determine the probability factor appropriate for the limit state under consideration (K_P) for the annual exceedance and the hazard factor (Z), as per the AS1170.4 section-3 the probability factor can be taken as mentioned in the in table 3.16 and regarding the hazard factor it can be obtained from table 3.17 and in case the location isn't mentioned in the table, its recommended to use figure 3.2(A to F). There is an overall overview shown in figure 3.2(G) as well.

PROBABILITY FACTOR (k_p)

Annual probability of exceedance P	Probability factor k_p
1/2500	1.8
1/2000	1.7
1/1500	1.5
1/1000	1.3
1/800	1.25
1/500	1.0
1/250	0.75
1/200	0.7
1/100	0.5
1/50	0.35
1/25	0.25
1/20	0.20

NOTE: The annual probability of exceedance in Table 3.1 is taken from the BCA and AS/NZS 1170.0.

Table (3.16) probability factor (KP)

Finally, it is essential to calculate the variation period (T), which differs based on the mode of shaking being measured or, the crucial natural period of the building as specified in Clause 6.2.3.

6.2.3 Natural period of the structure

The fundamental period of the structure as a whole (T_1, fundamental natural translational period of the structure) in seconds, including all the materials incorporated in the whole construction, may be determined by a rigorous structural analysis or from the following equation:

$$T_1 = 1.25 k_t h_n^{0.75} \quad \text{for the ultimate limit state} \qquad \ldots 6.2(7)$$

where

k_t = 0.11 for moment-resisting steel frames

= 0.075 for moment-resisting concrete frames

= 0.06 for eccentrically-braced steel frames

= 0.05 for all other structures

h_n = height from the base of the structure to the uppermost seismic weight or mass, in metres

The base shear obtained using the fundamental structure period (T_1) determined by a rigorous structural analysis shall be not less than 80% of the value obtained with T_1 calculated using the above equation.

Clause 6.2.3 for the natural period of the building

Location	Z	Location	Z	Location	Z
Adelaide	0.10	Geraldton	0.09	Port Augusta	0.11
Albany	0.08	Gladstone	0.09	Port Lincoln	0.10
Albury/Wodonga	0.09	Gold Coast	0.05	Port Hedland	0.12
Alice Springs	0.08	Gosford	0.09	Port Macquarie	0.06
Ballarat	0.08	Grafton	0.05	Port Pirie	0.10
Bathurst	0.08	Gippsland	0.10	Robe	0.10
Bendigo	0.09	Goulburn	0.09	Rockhampton	0.08
Brisbane	0.05	Hobart	0.03	Shepparton	0.09
Broome	0.12	Karratha	0.12	Sydney	0.08
Bundaberg	0.11	Katoomba	0.09	Tamworth	0.07
Burnie	0.07	Latrobe Valley	0.10	Taree	0.08
Cairns	0.06	Launceston	0.04	Tennant Creek	0.13
Camden	0.09	Lismore	0.05	Toowoomba	0.06
Canberra	0.08	Lorne	0.10	Townsville	0.07
Carnarvon	0.09	Mackay	0.07	Tweed Heads	0.05
Coffs Harbour	0.05	Maitland	0.10	Uluru	0.08
Cooma	0.08	Melbourne	0.08	Wagga Wagga	0.09
Dampier	0.12	Mittagong	0.09	Wangaratta	0.09
Darwin	0.09	Morisset	0.10	Whyalla	0.09
Derby	0.09	Newcastle	0.11	Wollongong	0.09
Dubbo	0.08	Noosa	0.08	Woomera	0.08
Esperance	0.09	Orange	0.08	Wyndham	0.09
Geelong	0.10	Perth	0.09	Wyong	0.10
Meckering region				**Islands**	
Ballidu	0.15	Meckering	0.20	Christmas Island	0.15
Corrigin	0.14	Northam	0.14	Cocos Islands	0.08
Cunderdin	0.22	Wongan Hills	0.15	Heard Island	0.10
Dowerin	0.20	Wickepin	0.15	Lord Howe Island	0.06
Goomalling	0.16	York	0.14	Macquarie Island	0.60
Kellerberrin	0.14			Norfolk Island	0.08

Table (3.17) hazard factor (Z) for a specific area in Australia

3.2.3. TORSION

The building response in horizontal has huge compilations of the produced forces in the system of lateral support because of the torsional effects.

Approximate Formula: - amplification or not for the eccentric static amongst the stiffness and mass centre not essential which is different from old code (AS1170,93) which is suitable since the usual centre story stiffness not possible to recognise with exactness and in realism and the estimation of the amplification factor needs boring evaluates. In case there is a building have asymmetric stiffness and mass horizontally, the horizontal analysis of seismic loads gives zero response torsion. Even though, variance stiffness along with doubt of probable torsional of motion ground might occur a response torsion even when the building is fully symmetric.

Moreover, to ensure the building will resist the min. Torsion resists, and building stiffness; the Australian code recommend applying an accidental torsion in placing an earthquake load with an eccentricity of **± 0.1** from the centre of the building while the ASCE7 purpose **± 0.05**. It is conformist to undertake that all masses of the building moved throughout the horizontal path with the concept of (+/-) at the time and examining the most severe torsional moment and consider it in the analysis.

Estimating the mass displacement effects using the dynamic analysis is not practical at all, the dynamic features of the building system will change as per the mass positions. So, the horizontal seismic eccentric accident considered in the mass centre.

Giving a direct evaluation of the mention above is impossible. However, it is essential to understand that in theory for the full symmetric structures, the accidental moment torsion significantly doubled. AS1170.4 have the more protective technique to estimate the accidental moment torsion especially for the symmetric structures and keeping the same performance of effects for the un-symmetric structures. The simplified technique showed for structures less than fifteen meters in category II zone, removed the needs for the torsion and increased the load multipliers. Gad, E.F., Chandler, A.M., Duffield, C.F. and Hutchinson, G.L. (1999).

3.2.4. *Vertical Regularity: -*

The building capacity affected by the moment and shear pattern as shown in figure (3.13). The outcome of this result is posted elastic distortion which is occurred at every level which causes elements degradations, more distortion, and soft story mechanisation with total collapse as a result. The purpose of this check is to avoid all sudden variations in total stiffness or strength at any particular level. In case the opposite happened, further analysis to be implemented to ensure the capacity distortion of the post-elastic is enough. Where such provisions are not met, then a more detailed analysis will be needed to ensure that post-elastic deformation capacity at each level can be met without unacceptable loss of strength or post-elastic deformation demands more than their capacity. It is also recommended to avoid the curtailment of steel either in columns or shear walls in one level and try to make gradually upon each a set of the level to avoid any familiar soft story.

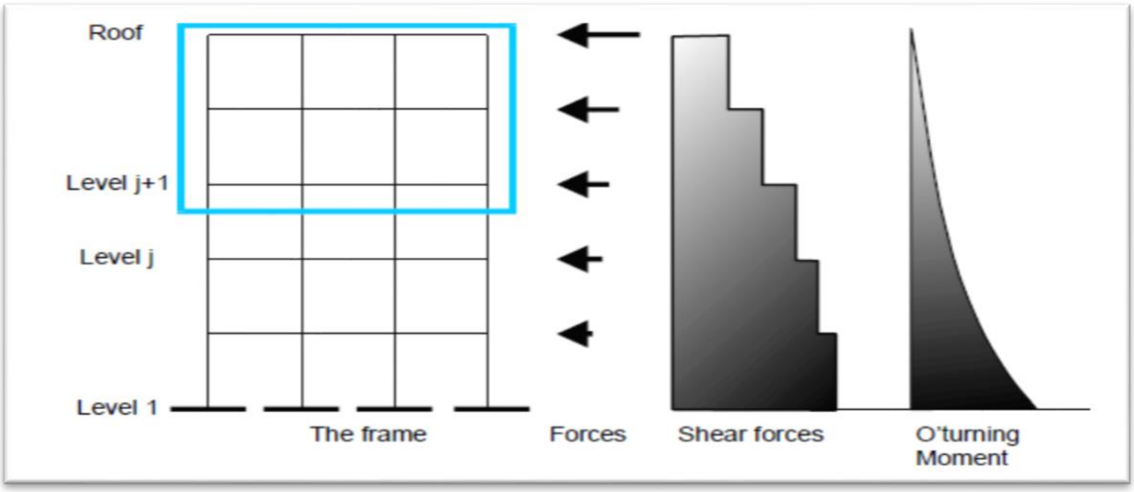


Figure (3.13) pattern load with internal building actions

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Stiffness-Soft Story Irregularity:** Stiffness-soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	**Stiffness-Extreme Soft Story Irregularity:** Stiffness-extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
2.	**Weight (Mass) Irregularity:** Weight (mass) irregularity is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
3.	**Vertical Geometric Irregularity:** Vertical geometric irregularity is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity:** In-plane discontinuity in vertical lateral force-resisting elements irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting beam, column, truss, or slab.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity:** Discontinuity in lateral strength–weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity:** Discontinuity in lateral strength–extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

Table (3.18) vertical Structural Irregularities

The American code has the same concept and identifies the vertical irregularity as shown in the table above.

3.2.5. Horizontal Regularity:

In general, there are three dimensions of motion created throughout the earthquake action which is mitigated into two horizontal perpendicular dimensions only and the vertical considered to be ignored. It is also possible to found torsion and twist effects on the building which is obtained either as the straight occupation of the motion ground inputs, differences in the three-dimensional division of the mass seismic or due to the in-plan irregularity.

In order to contain this problem, arranging the columns/shear walls in a way to resist the lateral forces adequately by reduce the eccentricity between the centre of gravity and the centre of mass as much as possible and adding ten percent (10%) of eccentricity to avoid any misesteemed eccentricity which will provide the building a level of safety to resist the seismic effects. Gad, E.F., Chandler, A.M., Duffield, C.F. and Hutchinson, G.L. (1999).

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity:** Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F B, C, D, E, and F C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity:** Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity:** Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity:** Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one having a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offset Irregularity:** Out-of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.	12.3.3.3 12.3.3.4 12.7.3 Table 12.6-1 Section 16.2.2	B, C, D, E, and F D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel System Irregularity:** Nonparallel system irregularity is defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Table (3.19) Horizontal Structural Irregularities

3.2.6. Floor Diaphragms: -

Also, it is essential to design the flooring system to resist the seismic effects by linking the floor with the lateral effects using the diaphragm floor in every floor level. It is also acceptable to the horizontal action techniques like beams or trusses in case of big opening in the structure. This action is usually taken into consideration during the design stage, and it is vital to allow the plane stress density inside the floor diaphragm about openings over the provision of diagonal angle reinforcements within flooring. It essentially to pay attention to the connection details between the vertical and horizontal members in order to transfer the seismic effects accurately.

3.3. Equivalent lateral force analysis (static): -

For the American code, the analysis considered as an essential requirement to design any type of building or structure to do all the needful assessments for the structure. The weight of the building can be easily calculated from the ETABs software and as shown in table 3.20 and the suggested damping to be considered five per cent either it is static nor dynamic.

Story	*Total Weight in each story (Kn)*	*SUM*
Story 11-70	22642.8	1358568
Story 10	133670.285	160404.342
Story 1- 9	22356.0663	241445.516
TOTAL	1760418	

Table (3.20) total weight of the building extracted from ETABs for each tower

For the vibration building period, it can be obtained from the ASCE7 using formula 12.8-7 as shown below

$T_a = C_t\ h^X$ where h is the building height, Ct and X can be taken from table 3.21:

Structure Type	C_t	x
Moment-resisting frame systems 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:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731)[a]	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a]Metric equivalents are shown in parentheses.

Table (3.21) Values of Approximate Period Parameters as per the ASCE7

Thus, Ta = 0.0488 * $210^{0.75}$ = 2.692 Seconds and to find the upper limit for the calculated period taken from the formula Cu*Ta and the Cu from table 3.23 and consequently, 2.692*1.4 = <u>3.768</u> Seconds. The time period extracted from the ETABs software was 8.91 seconds which is more reasonable and can be used effectively in our calculation.

Design Spectral Response Acceleration Parameter at 1 s, 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 (3. 22) Coefficient for Upper Limit on Calculated Period as per the ASCE7

The next step is to calculate the seismic response coefficient as mentioned in section 12.8.1.1 and as below:

CS min < CS < CS Max

- CS max = $\frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$ Equation 12.8-2. => 1.19867 (3½/1) = 0.342
- Since T=3.768 < T_L=8 So,

CS = $\frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$ for T ≤ TL Equation 12.8-3 => 0.6396/8.91 (3½/1) = 0.020

- for structures located where S1 is equal to or greater than 0.6g, Cs shall not be less than,

C_S min = 0.5S1/(R/Ie) Equation 12.8-6. => 0.5*0.738/ (3½/1) = 0.105.

- Cs = $0.044S_{DS}Ie \geq 0.01$ => 0.044*1.19867*1= 0.052 Note: for the ELF use CS min and for Drift use CS max.

Now, it's possible to calculate the seismic base shear from the ASCE7, 12.8-1 formula which lead to

V=0.052 *1760418= 91541.73612Kn or in another word, the seismic load can be considered as approximately five percent from the total building weight and to be spread laterally on the structure height by Equations below:

$$F_x = C_{vx}V \qquad (12.8\text{-}11)$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (12.8\text{-}12)$$

k = 0.75 + 0.5T = 0.75 + 0.5(3.768) = 2.634.

On the other hand, and regarding the Australian code, this analysis considered as an essential requirement to design any type of building or structure to do all the needful assessments for the structure. The weight of the building can be easily calculated from the ETABs software and as shown in table 3 and the suggested damping to be considered five per cent either it is static nor dynamic.

Story	*Total Weight in each story (Kn)*	*SUM*
Story 11-70	22642.8	1358568
Story 10	133670.285	160404.342
Story 1- 9	22356.0663	241445.516
TOTAL	1760418	

Table (3.23) total weight of the building extracted from ETABs for each tower

The natural time period of the structure can be determined by following section 6.2.3. From the code by following the formula below:

$$T_1 = 1.25 k_t h_n^{0.75} \quad \text{for the ultimate limit state}$$

where

k_t = 0.11 for moment-resisting steel frames

= 0.075 for moment-resisting concrete frames

= 0.06 for eccentrically-braced steel frames

= 0.05 for all other structures

hn = height from structure-based on the uppermost seismic weight or mass, in the meter.

Thus, T1 = 0.05 * $210^{0.75}$ = 2.785 seconds which is lower than the American code in milliseconds, and there is no safety coefficient mentioned. The time period extracted from the ETABs software was 7.43 seconds which is more reasonable and can be used effectively in our calculation and as noted it is lower than the American code.

The next step is to calculate the spectral shape factor as mentioned previously in section1.4. A table 2.8-a, the value can be taken as .25. Then, it is important to specify the structure ductility factor (μ) and the structural performance factor (SP) - basic structures using table 2.8-b from this work. The result will be, μ=2 and Sp=0.77. The next stage is to find the design action coefficients which is: -

$K_P Z C_h$ (T1) Sp/ μ ➔ 1.3x0.09x0.25x0.77/2 = 0.012 or in another word 1.2% of the building weight. However, the code mentioned clearly in section 5.5.2.1 that the minimum acceptable is five per cent of the total weight.

So, the V will be 0.05x 1760418= 88020Kn. Which is less than the American code and to calculate the vertical separation of the horizontal forces, it is recommended to follow section 6.3 and to use the below procedure:

Fi=$K_{F,i}$ V and $K_{F,i}$ can $\dfrac{W_i h_i^k}{\sum_{j=1}^{n}\left(W_j h_j^k\right)}$ be fine from =

Moreover, as shown in the table below taking K=2 which is less than ASCE-7

The below schedule showing the base shear forces which is showing a huge difference between the American the Australian code which is approximately forty per cent more. Paulay T. and Priestley M.J.N. (1992).

3.4. Modal Response Spectrum Analysis (Dynamic): -

Before the analysis, it iessential to specify the dynamic features if the building such as the mass, stiffness and damping. ETABS can calculate stiffness and mass and the seismic weight will be calculated as a semi-rigid diaphragm not like in the past where it was much easier to consider the slabs as a rigid to reduce the degree of freedom since we have powerful computers can calculate faster than before by using the finite element computation method. The total weight of the structure is calculated using computers. American Society of Civil Engineers, (2010), ASCE 7-10: Minimum Design Loads for Buildings and Other Structures, Reston, VA.

Moreover, for damping, the assumption has been taken to consider the damping percentage as five per cent whether its static or dynamic analysis and the combining method for each response modal in the response spectrum technique will be via the complete quadratic combination CQC as a substance of the square root of the sum of the squares SRSS.

It is essential to compute the natural mode along with natural period vibration of the building of which is possible to extract from the building ELF response in both directions. Mode frequencies and shapes automatically calculated by ETABs. Figure 2.15 shows the first two modes obtained from ETABs also the

Modal and frequencies are shown in table 2.24. American Society of Civil Engineers, (2010), ASCE 7-10: Minimum Design Loads for Buildings and Other Structures, Reston, VA.

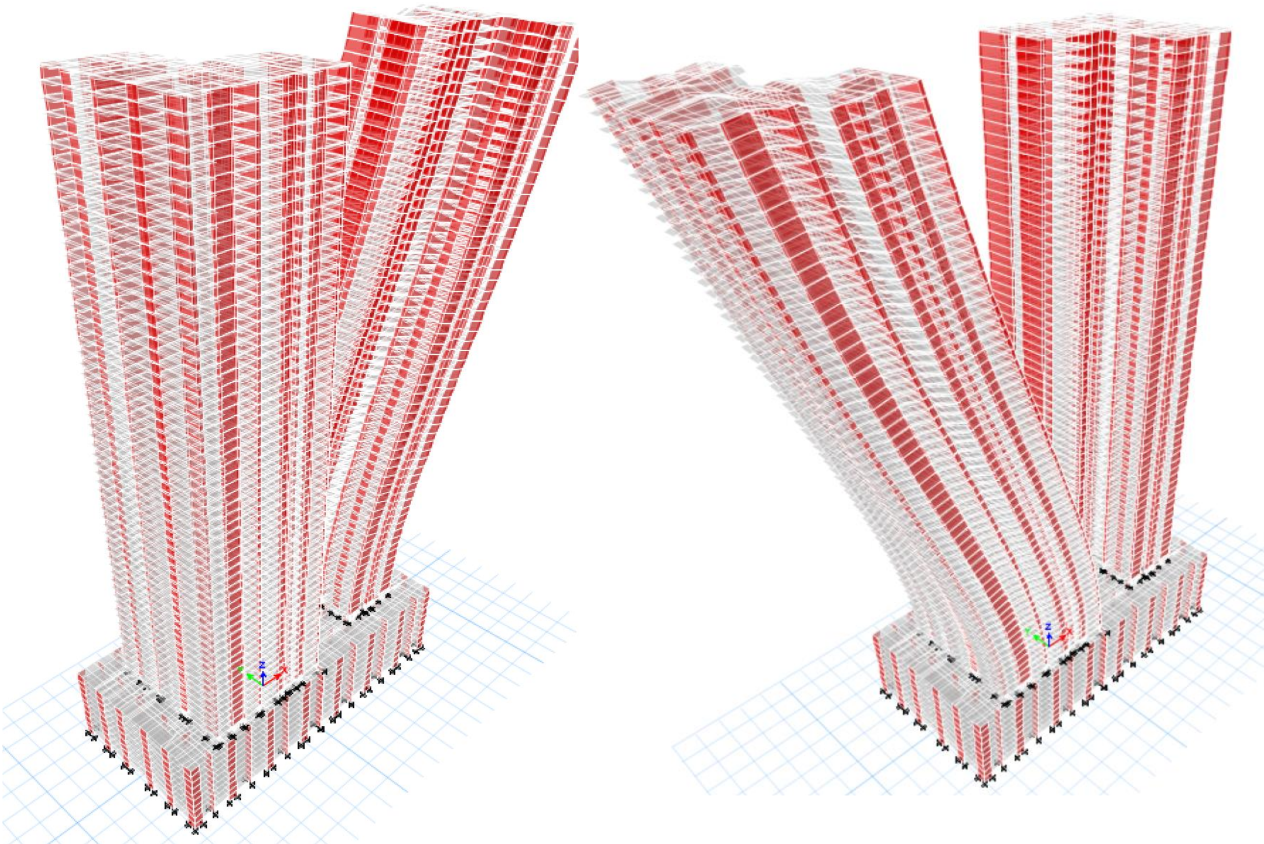


Figure (3.14) mode shape of MRS using ETABs

The Response spectrum coordinates and computation of modal forces obtained using section 11.4.5. This spectrum contains three portions for periods less than TL = 8.0 sec.:

- For periods less than T_0:

$$S_a = 0.6\frac{S_{DS}}{T_0}T + 0.4S_{DS}$$

- For periods between T_0 and T_S:

$$S_a = S_{DS}$$

- For periods greater than T_S:

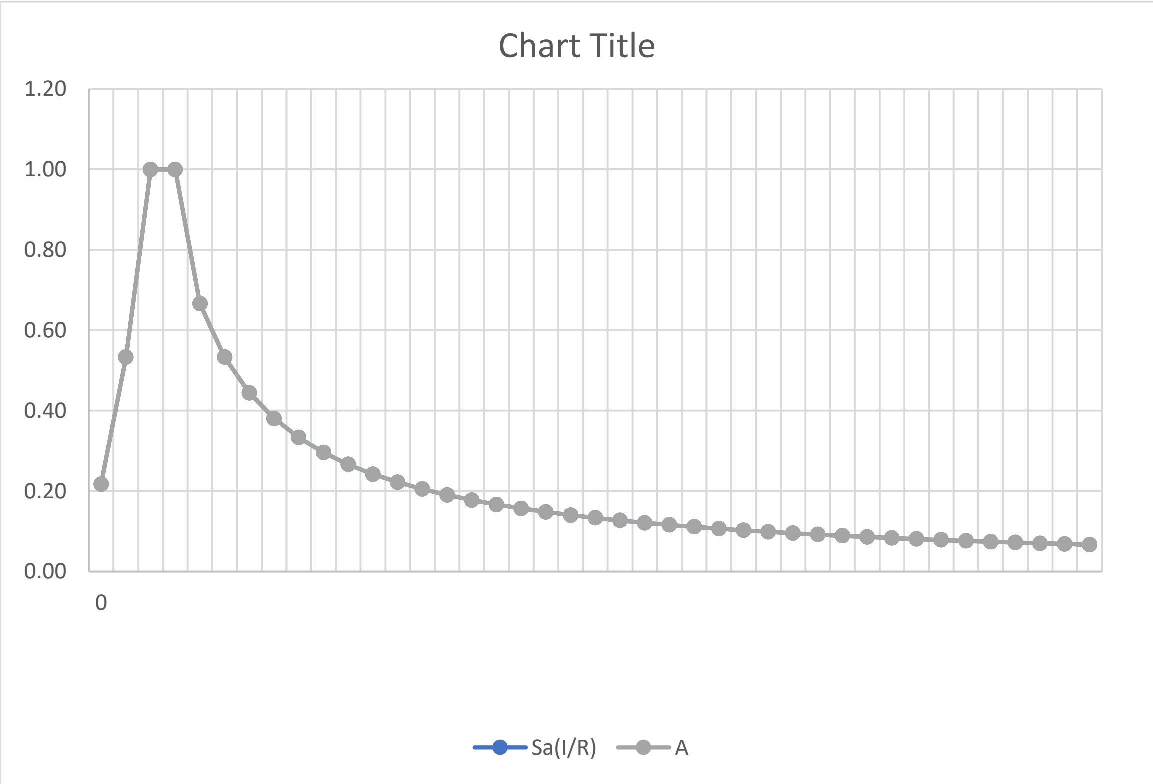
$$S_a = \frac{S_{D1}}{T}$$

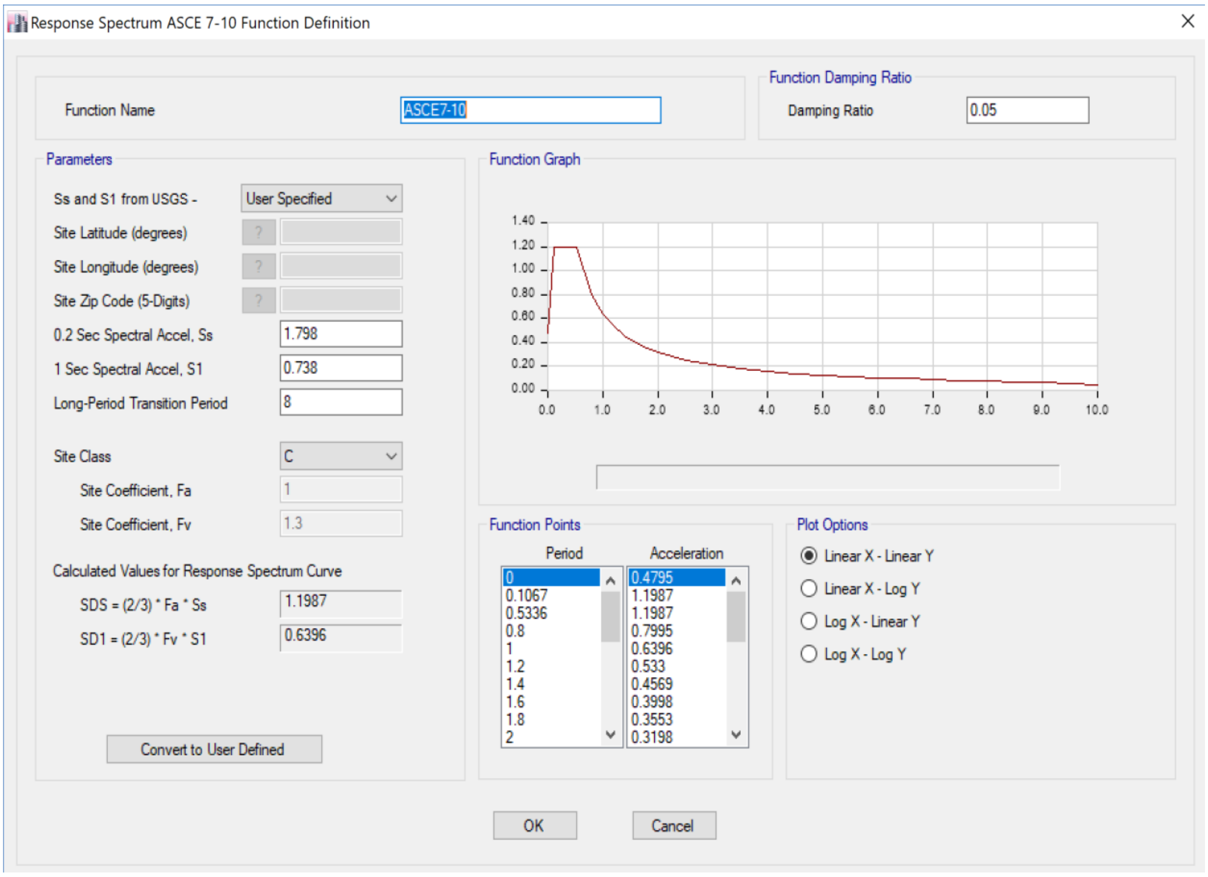
where $T_0 = 0.2S_{D1}/S_{DS}$ and $T_S = S_{D1}/S_{DS}$.

With *SDS* = 1.19867 and *SD1* = 0.6396, *TS* = 0.533 seconds and *T0* = 0.106 seconds. The computed response spectrum coordinates for several period values are shown in figure 11.4-1 from the ASCE7 and figure 2.13 obtained from ETABs.

Tm	Sa	Sa(I/R)
0	0.49	0.14
0.106To	1.20	0.34
0.533Ts	2.25	0.64
0.533	2.25	0.64
0.8	1.50	0.43
1	1.20	0.34
1.2	1.00	0.29
7.8	0.15	0.04
8	0.15	0.04

Table (3.24) Response Spectrum Coordinates





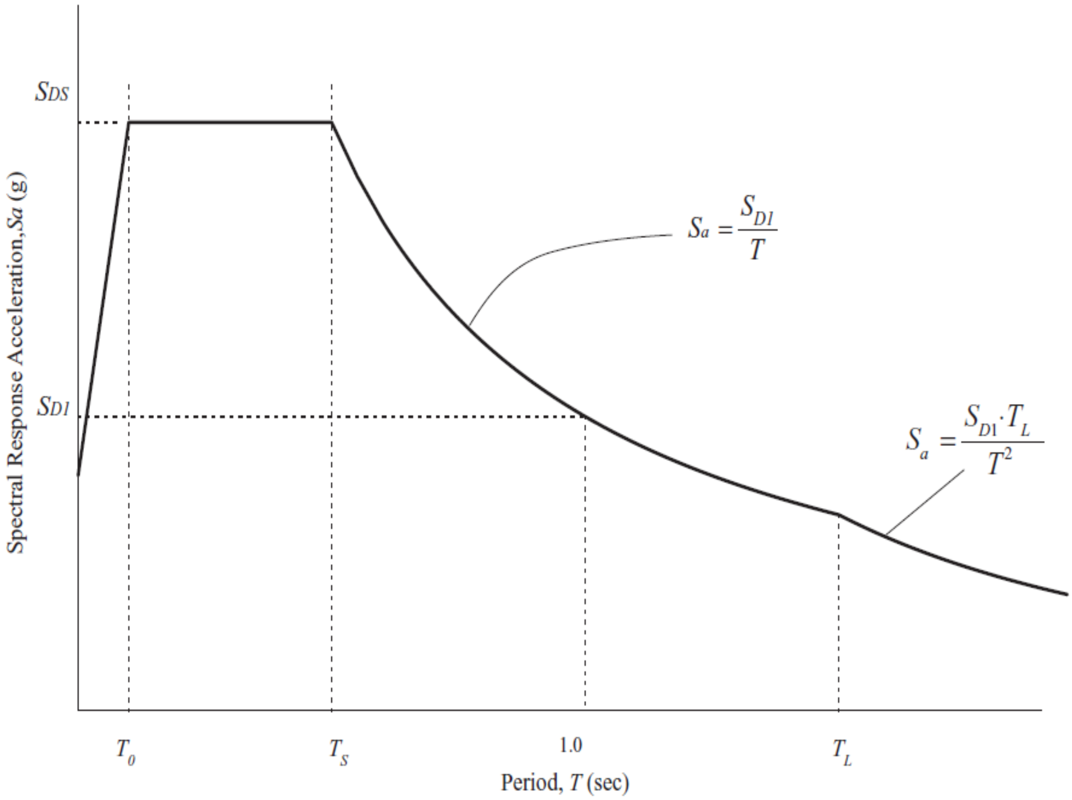


FIGURE 11.4-1 Design Response Spectrum.

Figure (3.15) response spectrum obtained from ETABs

Regarding the Australian code, it is essential to compute the natural mode along with natural period vibration of the building of which is possible to extract from the building ELF response in both directions. Mode frequencies and shapes automatically calculated by ETABs. Figure 2.15 shows the first two modes obtained from ETABs also the modal and frequencies are shown in table 3.24. Chandler, A.M., Lam, N.T.K. and Sheikh, M.N. (2002).

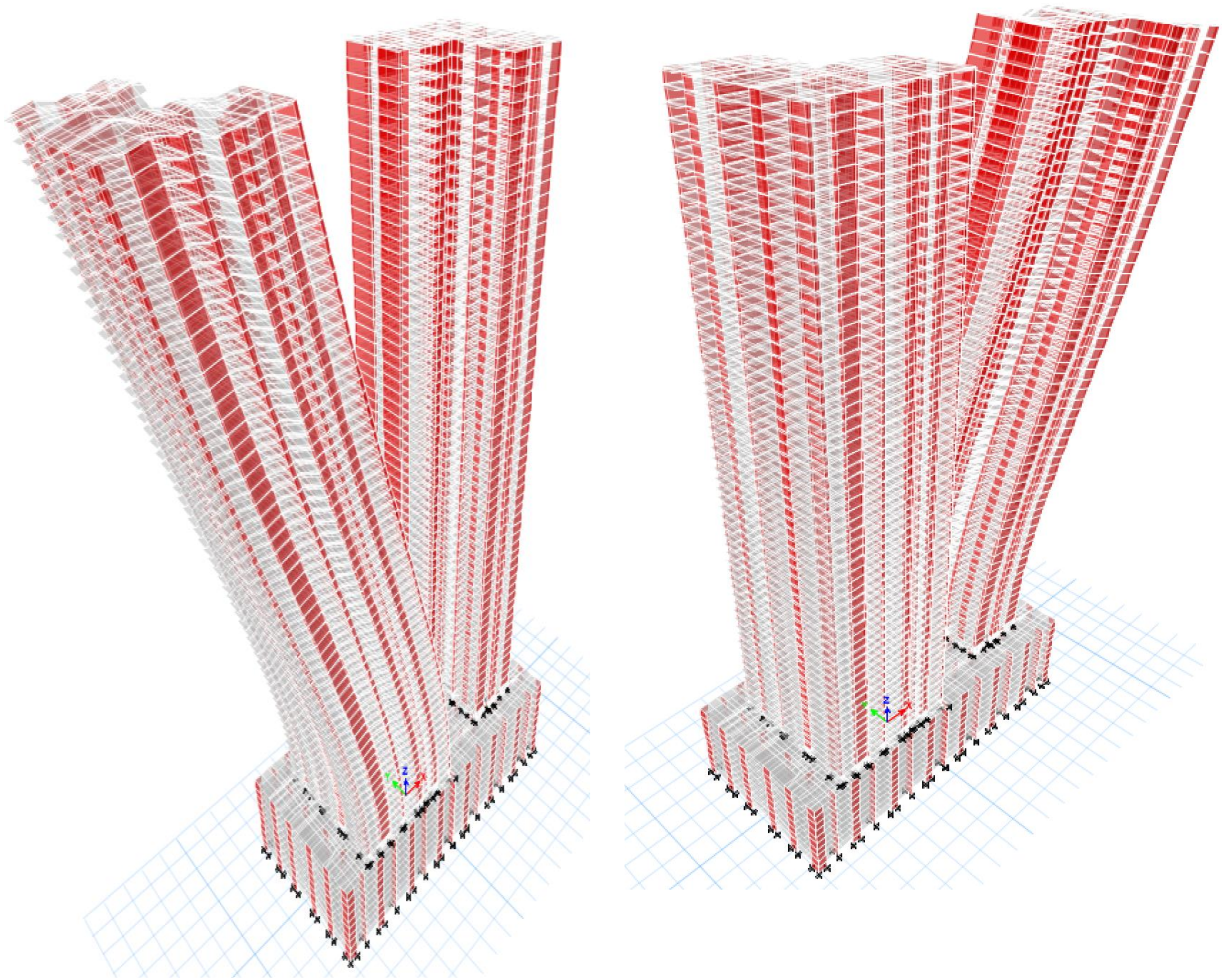


Figure (3.16) mode shape of MRS using ETABs

The Response spectrum coordinates and computation of modal forces obtained using section 11.4.5. This spectrum contains three portions for periods less than TL = 8.0 sec.:

$$RSA = RSA_{\max} = C_1\ \mathrm{F_a}\ \mathrm{R_p} Z\ \ (\mathrm{T} \leq \mathrm{T_1})\ \ (2\mathrm{a})$$

$$RSA = \frac{C_2 R_p Z F_V}{T^n} \qquad (\mathrm{T} > \mathrm{T_1}) \qquad (2\mathrm{b})$$

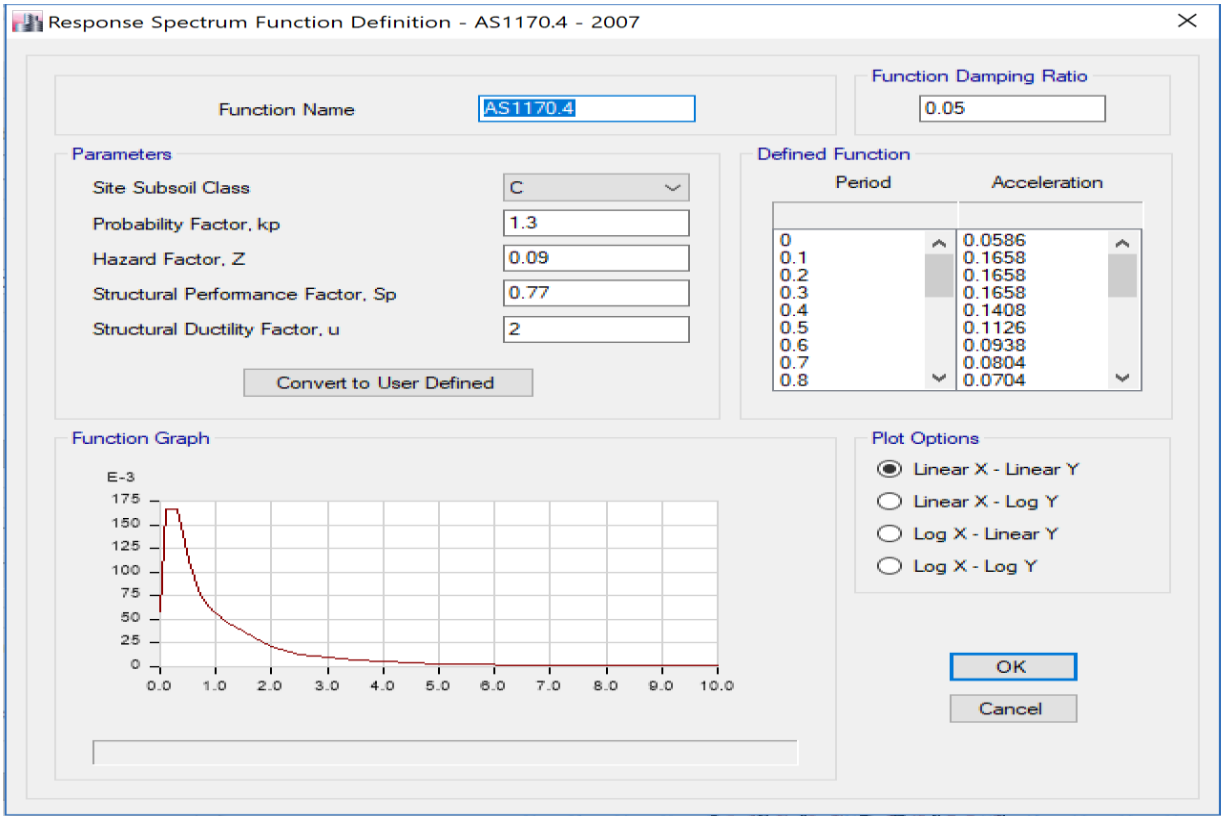


Figure (3.17) response spectrum obtained from ETABs

Chapter four

4. RESULTS

The significance of this chapter is to review the outcomes analysis using the ETABS software along with some manual calculation to celebrate the two modules and check the outcome if it is safe or not as per each code.

The seismic analysis and design to stand against earthquake effects can be obtained by using the proper strength, stiffness, and ductility. To do so, there are several calculation methods such RESPONSE SPECTRUM METHOD (smooth methods), TIME HISTORY METHOD which depends on the actual collected data from actual earthquakes, and EQUIVALENT LATERAL FORCES ELF which can calculate the adopted lateral inertial forces extracted from the ground wave motions.

The primary objective of this work makes a seismic design comparison between the Australian and American codes with the current requirements for each code. A high-rise building contains seventy floors will be adapted to face a differ provisions of seismic load using the mentioned Examine and compare the seismic design provisions for the American society of civil engineers ASCE7 published in 2016 and the Australian AS-1170 code, Specify the seismic loads for the sixty floors building. The used design spectra, response modification factors, comparing the seismic design loads which are established from the considered codes, assess the effect of the overstrength factors and the accelerations spectral on the seismic design loads.

With the purpose to assess and evaluate the theoretical outcome of the mentioned ASCE and AS method, a case study of 210m vertical height of three symmetric buildings with shared podium implemented to compare the two codes outcome.

The building is a seventy floors tower. The typical plan is three L-shapes planted on ten story of podium plan with unparalleled alignment which is noteworthy. Reinforced concrete walls implemented along with exterior columns frames connected by beams for each level. The slabs assumed to be a hollow core slab with thickness 330mm and 10mm as a topping slab.

4.1. Base shear calibrations

Regarding the American code, after analysis finished from the ETABs and combining all the data using the CQC methods, the outcomes as shown in Table 2.26 indicated that the dynamic is below the static, and as per the code section12.9-4.1, the required scaling is 85% from the static but since the building is high and we need to be very sure, we can take it 99%. Building Seismic Safety Council, (2009), 2009 NEHRP Recommended Seismic Provisions for Buildings and Other Structures, prepared for the Federal Emergency Management Agency, Washington, DC.

Check for Dynamic & Static Base Shear(X-Direction)				
Trial	Static base shear (VSx)Kn	dynamic base shear (VDx)Kn	Ratio (VSx/VDx)	check
1st	-75907.2027	271.6386	279.44189	RE-check
2nd	-426516.1641	426515.3543	1.0000019	Achieved
Check for Dynamic & Static Base Shear(Y-Direction)				
Trial	Static base shear (VSy)Kn	dynamic base shear (VDy)Kn	Ratio (VSy/VDy)	check
1st	-75907.2027	253.32	299.64947	RE-check
2nd	-426516.1751	426523.7911	0.9999821	Achieved
Check for Dynamic & Static Base Shear(Z-Direction)				
1st	299.6494659			
2nd	1.000001899			

Table (4.1) Check of Dynamic & Static Base Shear

On the other hand, using the Australian code and after analysing finish from the ETABs and combining all the data using the CQC methods, the outcomes as shown in Table 2.26 indicated that the dynamic is below the static and as per the code (NZS1170.5 section 5.2.2.2), the required scaling is 80% from the static but since the building is high and we need to be very sure, we can take it 99%.

<table>
<tr><th colspan="5">Check for Dynamic & Static Base Shear(X-Direction)</th></tr>
<tr><td>Trial</td><td>Static base shear
(VSx)Kn</td><td>dynamic base shear
(VDx)Kn</td><td>Ratio
(VSx/VDx)</td><td>check</td></tr>
<tr><td>1st</td><td>-23058.4629</td><td>234.7486</td><td>98.22619986</td><td>RE-check</td></tr>
<tr><td>2nd</td><td>-23058.4629</td><td>23059.3618</td><td>0.999961018</td><td>Achieved</td></tr>
<tr><th colspan="5">Check for Dynamic & Static Base Shear(Y-Direction)</th></tr>
<tr><td>Trial</td><td>Static base shear
(VSy)Kn</td><td>dynamic base shear
(VDy)Kn</td><td>Ratio
(VSy/VDy)</td><td>check</td></tr>
<tr><td>1st</td><td>-23058.4597</td><td>275.7072</td><td>83.633868</td><td>RE-check</td></tr>
<tr><td>2nd</td><td>-23058.4597</td><td>23058.4572</td><td>1.0000001</td><td>Achieved</td></tr>
<tr><th colspan="5">Check for Dynamic & Static Base Shear(Z-Direction)</th></tr>
<tr><td colspan="2">1st</td><td colspan="3">269.6788359</td></tr>
<tr><td colspan="2">2nd</td><td colspan="3">1.000001119</td></tr>
</table>

Table (4.2) Check of Dynamic & Static Base Shear

The base shear outcome shows a massive difference between the two codes and approximately ten per cent different from the American code the max. Story shear can be reviewed as shown in the table below:

Story	static		Dynamic	
	AS1170.4	**ASCE7**	**AS1170.4**	**ASCE7**
T1-Story70	382.1586	7030.8673	347.6027	6376.2742
T1-Story69	852.477	15728.305	771.502	14189.3905
T1-Story68	1309.2617	24175.467	1179.6267	21709.1466
T1-Story67	1752.7105	32376.015	1572.5417	28946.1974
T1-Story66	2183.0206	40333.595	1950.8891	35912.6579
T1-Story65	2600.39	48051.862	2315.3937	42622.1871
T1-Story5	5.2377	105.4924	301.9512	5984.1546
T1-Story4	7.4562	149.923	751.8169	14860.2583
T1-Story3	8.7041	174.9152	1048.0087	20665.6123
T1-Story2	9.2587	186.0228	1157.4881	22719.2466
T1-Story1	9.3973	188.7997	1175.2988	22946.3368

Table (4.3) static and dynamic forces as per the Australian and American code

4.2. Drift and P-delta effects

Based on the code section 12.9.4.2. American code, the calculated drift and displacement must have scaled as well as the base shear using the same scaling from table 2.28 which can be implemented based on section 12.8.6-2 which will calculate as *CuTa*.

Implementing section 12.9.4.2, formula 12.8-6 which are controlled by Cs required us to use formula qw.8-5, and that means the drift must escalate by the Cd value which is 2.25.

As previously, the drift story shows the alterations in displacement at the mass centre on each floor, and the vertical displacement estimate of that point at below level. These numbers specified each mode and combined with CQC. As shown in the tables, the allowable drift not exceeded at any level. American Society of Civil Engineers, (2010).

Cd=	2.25		**Time Period (T) =**		**8.91**	**seconds**		
D1								
Story	**Diaphragm**	**Load Case/Combo**	**the point displacement in direction Y in each story δ (Uymm)**	**Height (mm)**	**story drift Δs mm**	**elastic drift Δm mm**	**Allowable Drift (Δa mm)**	**check (Δm< Δa)**
Story70	D1	SPX Max	661.73	3000	16.20	25.51	75.00	Achieved
Story69	D1	SPX Max	645.53	3000	11.25	17.71	75.00	Achieved
Story13	D1	SPX Max	5.30	3000	2.75	4.33	75.00	Achieved
Story12	D1	SPX Max	2.55	3000	2.55	4.02	75.00	Achieved

Table (4.4) Check of story drift in the y-direction

V_X (kN)	U_X mm	Δ_X mm	Θ_X	check
28922.6611	**94.741**	**12.195**	**0.127296635**	**P-DELTA**
29073.7485	**82.546**	**11.58**	**0.122754024**	**P-DELTA**
29208.2828	**70.966**	**10.915**	**0.117522183**	**P-DELTA**
29326.6574	**60.051**	**10.199**	**0.11155713**	**P-DELTA**
29429.1696	**49.852**	**9.43**	**0.1048019**	**P-DELTA**
29690.4159	**17.311**	**5.776**	**0.068522162**	**NO P-DELTA**
29729.7176	**11.535**	**4.707**	**0.056762354**	**NO P-DELTA**
29829.6878	**3.276**	**3.276**	**0.040754853**	**NO P-DELTA**
29934.6312	**0**			

Table (4.5) Check of P-delta in the x-direction

There is a massive difference between the American and Australian code in term of allowing the drift of each story, section 5.5.4 Australian code mentioned the maximum allowable is 1.5% from the story height while the American code table 12.12-1 came with more detail and more specific for each structure case and elements and based on the seismic zone as well.

As previously, the drift story shows the alterations in displacement at the mass centre on each floor, and the vertical displacement estimate of that point at below level. These numbers specified each mode and combined with CQC. As shown in the tables, the allowable drift is not exceeded at any level. (Paulay T. and Priestley M.J.N. (1992).

The below table shows the drifts computed from each of the analyses. The ELF drifts are significantly higher than those determined using modal response spectrum analysis which is showing a vast difference and higher values in the American than the Australian.

Story	**Static**		**Dynamic**	
	ASCE7	AS1170.4	ASCE7	AS1170.4
T1-Story70	0.033955	0.001851	0.030282	0.001719
T1-Story69	0.034054	0.001856	0.030372	0.001724
T1-Story68	0.034194	0.001864	0.030503	0.001732
T1-Story67	0.03438	0.001873	0.030677	0.001743
T1-Story66	0.034605	0.001885	0.030888	0.001756
T1-Story65	0.034863	0.001899	0.031132	0.001771
T1-Story5	4.00E-06	3.75E-07	0.000255	3.40E-05
T1-Story4	7.00E-06	4.24E-07	0.000615	4.40E-05
T1-Story3	8.00E-06	3.58E-07	0.00079	3.90E-05
T1-Story2	7.00E-06	1.60E-07	0.000705	1.80E-05

Table (4.6) static and dynamic drift as per the Australian and American code

Based on the ASCE section 12.9.6, P-delta effects must check by ELF method. This implies that such effects should *not* be specified by outcomes from the modal response spectrum analysis. Thus, the results already are shown and discussed.

But, the effects can be calculated by the outcomes of the response spectrum assessment when the drift/ displacements and shear calculated from response spectrum, by not scaling the base shear as shown in table 4.7 below which shows P-delta effects and that required a dynamic analysis which already happened (Stehle, J.S., Goldsworthy, H.M. and Mendis, P., 2001).

μ/S_p		2.60	**Time Period (T) =**		**9.21**	**second**		
D1								
Story	**Diaphragm**	**Load Case/Combo**	**displacement in direction Y in each story δ (Uymm)**	**Height (mm)**	**story drift die mm**	**elastic drift di mm**	**Allowable Drift (Δa mm)**	**check** (Δm< Δa)
Story70	D1	QYN	252.56	3000	4.73	12.28	45.00	Achieved
Story69	D1	QYN	247.84	3000	4.57	11.88	45.00	Achieved
Story68	D1	QYN	243.26	3000	4.59	11.92	45.00	Achieved
Story67	D1	QYN	238.68	3000	4.61	11.98	45.00	Achieved

Table (4.7) Check of story drift in the y-direction

Regarding the P-delta is the same concept as the American code and there is no different at all as shown in the section below.

both codes have almost the same concept of evaluating the P-delta effect with more conservative in the American code and more criteria involved in the formula.

6.7.3 *P*-delta effects

6.7.3.1 *Stability coefficient*

For the inter-storey stability coefficient (θ) calculated for each level, design for *P*-delta effects shall be as follows:

(a) For $\theta \leq 0.1$, *P*-delta effects need not be considered.

(b) For $\theta > 0.2$, the structure is potentially unstable and shall be re-designed.

(c) For $0.1 < \theta \leq 0.2$, *P*-delta effects shall be calculated as given in Clause 6.7.3.2,

$$\theta = d_{st} \sum_{j=i}^{n} W_j \Big/ \left(h_{si} \mu \sum_{j=i}^{n} F_j \right) \qquad \ldots 6.7(2)$$

where

i = level of the structure under consideration

h_{si} = inter-storey height of level i, measured from centre-line to centre-line of the floors

4.3. Participant mass ratio

Both codes determine the same mass ratio to resist the seismic load (section 12.9.1ASCE7 and 7.4.2 AS1170.4) by taking a minimum of 90% of the mass to participate as shown in table 4.8 and 4.9. The Australian code also mentioned all the elements not participated in the seismic effects should be excluded from the analysis along with all elements there fundamental period is less than five per cent.

Case	Mode	Period(sec)	Sum (UX)	Sum (UY)
Modal	1	8.91	0.2497	0.0054
Modal	2	8.892	0.5008	0.0113
Modal	3	7.887	0.5064	0.2407
Modal	4	7.832	0.5126	0.4743
Modal	5	5.097	0.5127	0.4886
Modal	6	5.013	0.5129	0.5007
Modal	7	2.105	0.568	0.5014
Modal	8	2.083	0.6243	0.5021
Modal	9	1.812	0.6251	0.5517
Modal	10	1.797	0.6259	0.6028

Table (4.8) Check of participant mass ratio

Case	Mode	Period(sec)	Sum (UX)	Sum (UY)
Modal	1	9.155	0.2536	0.0058
Modal	2	9.136	0.5086	0.0123
Modal	3	8.141	0.5146	0.2435
Modal	4	8.08	0.5213	0.4792
Modal	5	5.307	0.5214	0.4953
Modal	6	5.219	0.5216	0.509
Modal	7	2.164	0.5774	0.5098
Modal	8	2.141	0.6343	0.5105
Modal	9	1.868	0.6353	0.5604
Modal	10	1.853	0.6361	0.612

Table (4.9) Check of participant mass ratio

All the computed results for the modal response spectrum method of analysis was based on the first 20 modes of the model. The accumulated effective modal mass for the first 20 modes is in the near 90% of the total mass of the structure, which is accepted. It is recommended to take three modes in each floor two transitional and one torsional, but we used only 26 because we do not have a powerful computer.

4.4. Accidental torsion and orthogonal loading effects

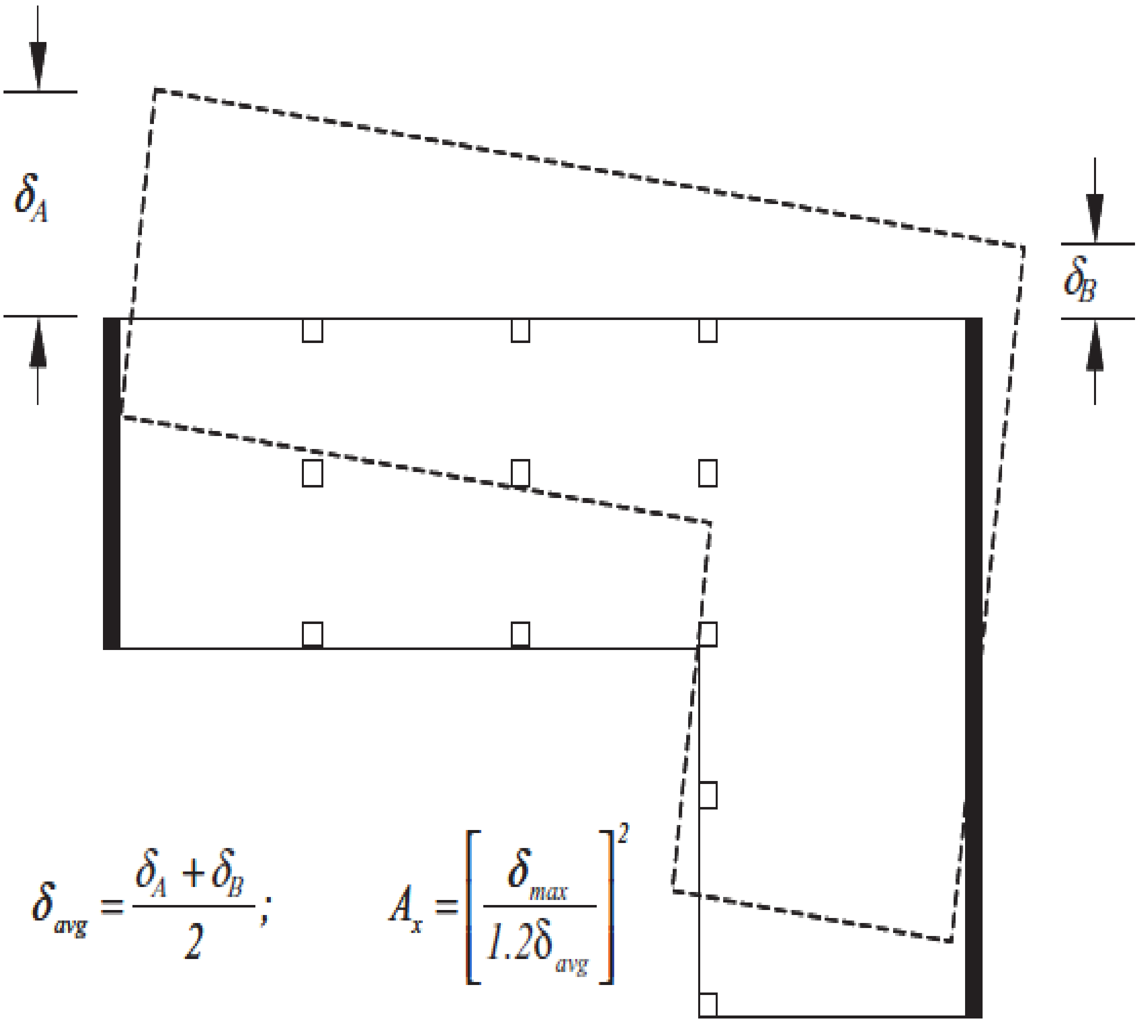
Since the analysis was from the ELF, two effects must be implemented the first is based on section 12.5 which state to take hundred per cent in one direction and thirty per cent in the orthogonal direction of the forces. The second is by implementing section 12.8.4.2 for $\frac{\Delta_{max}}{\Delta_{avg}} \geq 1.2$

regular/irregular torsion by implementing five per cent of eccentricity on the displacement mass source in both directions for SDC C, D, E, or F, using section 12.8.4.3. The accidental eccentricity must increase as below:

Δmax = maximum story drift at the edge of the floor diaphragm.

Δ_{Avg} = average drift at the centre of the diaphragm (see Standard Figure 5-A-1).

The Ax should be $A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2$ and between (1 to 3) (Building Seismic Safety Council, (2009).



Figure(4.1) Torsional Amplification Factor, Ax

So,

If $\delta_{Avg} < 1.2$ then no torsion, If $1.2 < \delta_{Avg} < 1.4$ then torsion, If $\delta_{Avg} > 1.4$ then Extreme torsion. See table2.31.

Story	Load Case/Combo	Maximum (mm)	Average (mm)	Drift Ratio (Max/Avg.)	Irregularity		Ax	Used Ax	Ecc. * used Ax	Max. Used Ecc.
Story70	QXN	0.0010	0.0010	1.33	.	irregular	1.24	1.24	0.06	0.0676
Story2	QXN	0.0010	0.0010	1.02	.	regular	0.72	1.00	0.05	
Story1	QXN	0.0010	0.0010	1.02	.	regular	0.72	1.00	0.05	
Story70	SPX	0.0010	0.0010	1.02	.	regular	0.72	1.00	0.05	
Story69	SPX	0.0010	0.0010	1.02	.	regular	0.72	1.00	0.05	

Table (4.10) Check of torsion in X-direction

Story	Load Case/Combo	Maximum (mm)	Average (mm)	Drift Ratio (Max/Avg.)	Irregularity		Ax	Used Ax	Ecc. * used Ax	Max. Used Ecc.
Story70	QYN	0.0010	0.0010	1.05	.	regular	0.76	1.00	0.05	0.0804
Story69	QYN	0.0010	0.0010	1.05	.	regular	0.76	1.00	0.05	
Story68	QYN	0.0010	0.0010	1.05	.	regular	0.76	1.00	0.05	
Story69	SPY Max	830.5310	647.9900	1.28	.	irregular	1.14	1.14	0.06	

Table (4.11) Check of torsion in Y-direction

The calculation shows irregular in the x-direction, and extreme irregular in Y-direction and that reflected in the calculation.

For the Australian code, Since the analysis was from the ELF, two effects must be implemented the first is based on section 5.4.2.1 which state to take hundred per cent in one direction and thirty per cent in the orthogonal direction of the forces. Second, check the accidental eccentricity must increase as below: -

$$\frac{\Delta_{max}}{\Delta_{avg}} \geq 1.2$$

δmax = maximum story drift at the edge of the floor diaphragm.

δAvg = average drift at the centre of the diaphragm.

So,

If δAvg < 1.2 then no torsion, If 1.2 < δAvg torsion. See table2.40&41.

Story	Load Case/Combo	Direction	Maximum(mm)	Average (mm)	Drift Ratio (Max/Avg.)	Ax	Used Ax	Ecc.* used Ax	Max. Used Ecc.
Story70	QXN	X	350.999	334.142	1.05	regular	0.77	1.00	0.05
Story69	QXN	X	345.678	328.705	1.052	regular	0.77	1.00	0.05
Story68	QXN	X	340.322	323.242	1.053	regular	0.77	1.00	0.05
Story67	QXN	X	334.927	317.749	1.054	regular	0.77	1.00	0.05
Story66	QXN	X	329.48	312.215	1.055	regular	0.77	1.00	0.05

Table (4.12) Check of torsion in X-direction

Story	Load Case/Combo	Direction	Maximum (mm)	Average (mm)	Drift Ratio (Max/Avg.)	Ax		Used Ax	Ecc. * used Ax
Story70	QYN	Y	324.546	260.95	1.244	irregular	1.07	1.07	0.05
Story69	QYN	Y	319.469	256.311	1.246	irregular	1.08	1.08	0.05
Story68	QYN	Y	314.365	251.657	1.249	irregular	1.08	1.08	0.05
Story67	QYN	Y	309.232	246.984	1.252	irregular	1.09	1.09	0.05
Story66	QYN	Y	304.057	242.288	1.255	irregular	1.09	1.09	0.05
Story65	QYN	Y	298.833	237.563	1.258	irregular	1.10	1.10	0.05
Story5	SPY	Y	0.182	0.166	1.092	regular	0.83	1.00	0.05
Story4	SPY	Y	0.168	0.154	1.093	regular	0.83	1.00	0.05
Story3	SPY	Y	0.131	0.12	1.092	regular	0.83	1.00	0.05
Story2	SPY	Y	0.079	0.073	1.087	regular	0.82	1.00	0.05
Story1	SPY	Y	0.028	0.026	1.078	regular	0.81	1.00	0.05

Table (4.13) Check of torsion in Y-direction

The calculation shows irregular in the x-direction with a maximum value of 1.903 which will lead to having 0.099 eccentricities in both directions, and that reflected in the calculation

4.5. Natural time period: -

Based on the above subject, there are significant different between the American and the Australian code in calculating the natural time period as described previously and the results show 3.44 for the American code and 2.69 for the Australian code for the approximate method while the ETABs calculate the accurate time and also it shows a difference of 8.4 and 9.3 respectively which will indicate more safety in the American code than the Australian.

4.6. Load and load combination: -

There is another difference in both codes regarding the live load; the Australian consider the 1.5 Kn/m2 while the American is 2.4 Kn/m2. On the other hand, the load combination is different as well, and at taking 1.35 for the initial dead load in the Australian and 1.4 in the American and all over the combination there are differences in the combinations, and it shows that the American have height values than the Australian code. Another difference is there for calculating the modulus of elasticity as mentioned before.

4.7. Overview of Design Loads and Column Capacities Development

The ETABs outcomes contain the moment and shear forces in the shear walls along with the displacement. As mentioned before, the time period of the building is 2.692 for the American code and 2.785 for Australian (see fig 2.18) and the participating mass is over 90%. Static and dynamic analysis has been used, and for the static, the combined forces have been done as mentioned in the formula below:

$F_1 = \sqrt{(Fx)^2 + (0.3Fy)^2}$ and $F_2 = \sqrt{(Fy)^2 + (0.3Fx)^2}$, where e $F_{1\&2}$ is the elastic shear, moment or deflection and Fix &y is the moment, shear or deflection in each direction.

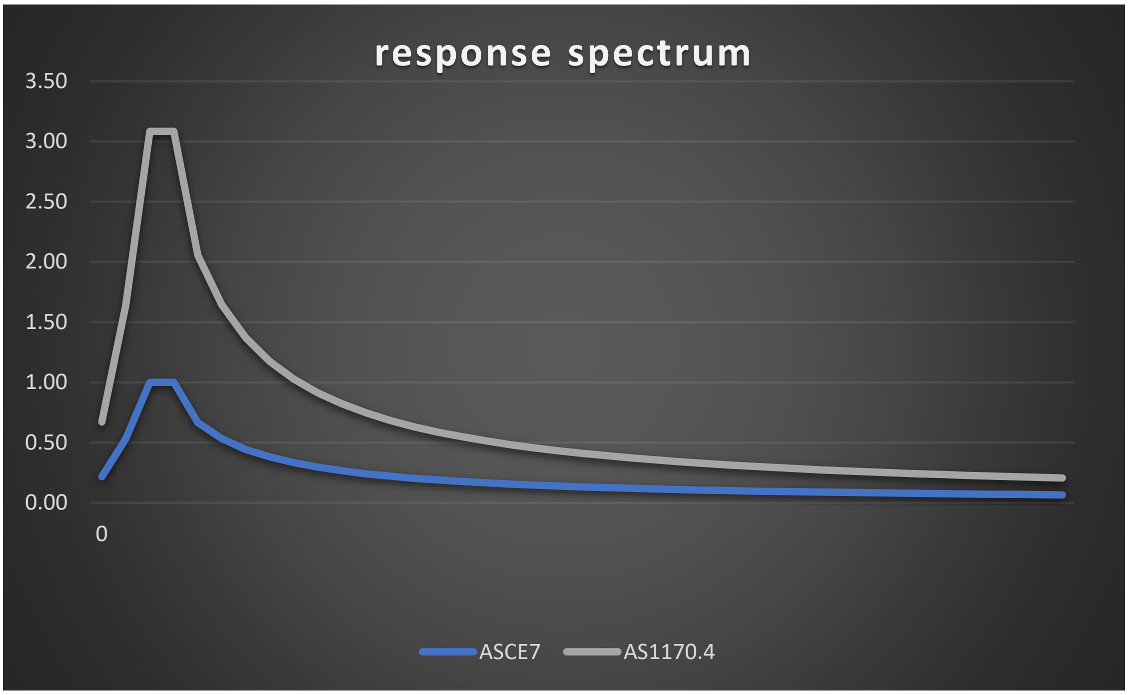


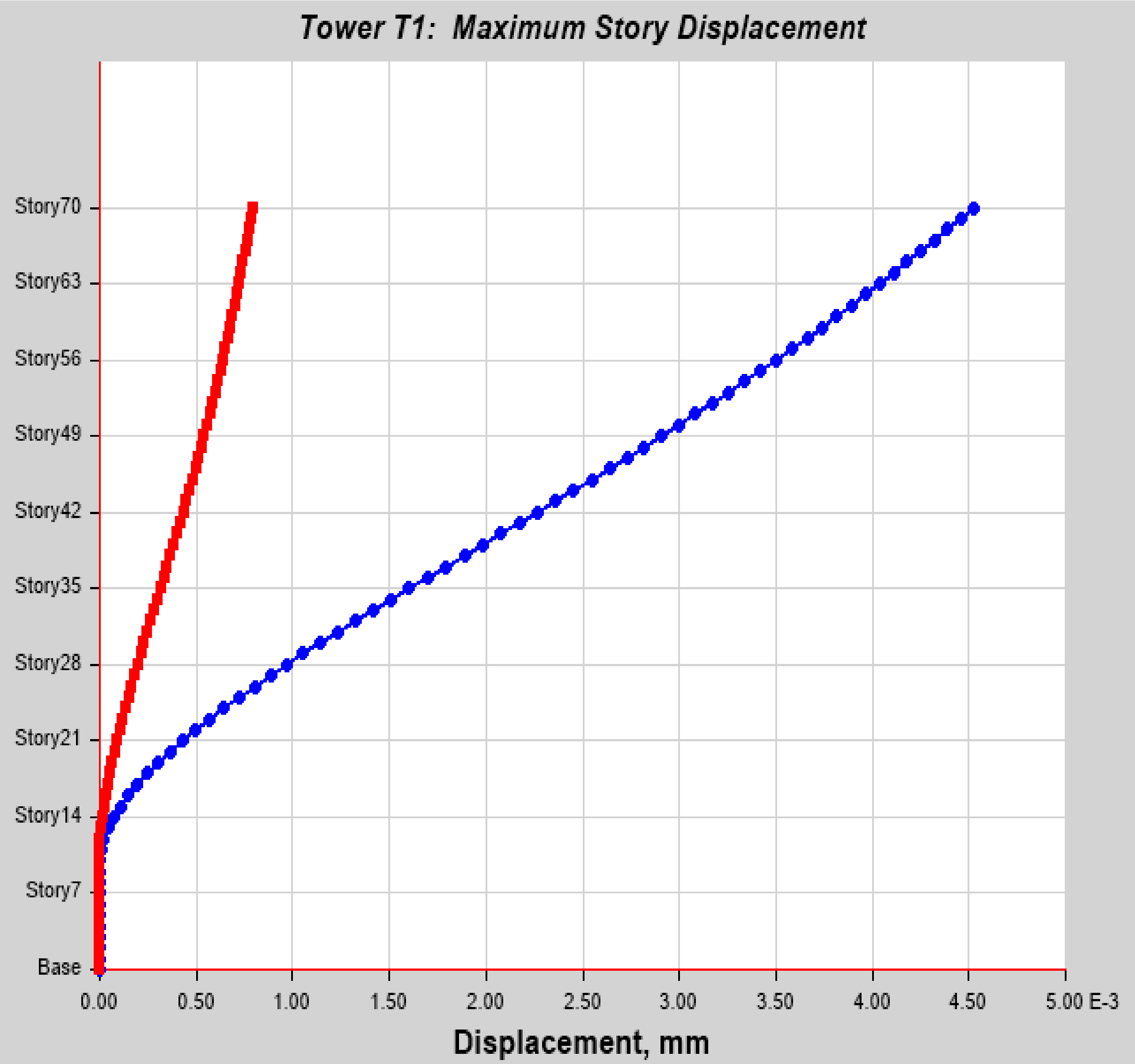
Figure (4.2) response spectrum as per the American and Australian code

4.8. Seismic Displacement Check

The seismic displacement comparison between the two codes shows a significant difference as shown in the graph below, 660mm for ASCE7 and 252mm for the AS1170.4 which a substantial different

Figure (4.3) Seismic Displacement as per the American code

Figure (4.4) Seismic Displacement as per the Australian code



4.9. Flexure and shear

Based on the above outcomes, the different from the two codes have been investigated. The discrepancy in outcomes is majorly referred to the used techniques used to specify the used overstrength factor.

Regarding the analysed building, the seismic design forces created from the ASCE7 and AS1170 are not conformable. The design numbers of those two codes are slightly different, mainly because of the overstrength influence and the modification response factors. The mentioned codes contain different design loads since constant acceleration ground motion of the 475yrs return period are different to create the seismic forces.

In general, the ASCE7 are more conservative than the AS1170 and that evident in the base shear outcomes which shows at least 20% more than the Australian code and at the same time, the Australian code gives more safety in terms of calculating the time period which gave more time period than the American in a few milliseconds. Regarding the soil type and dealing with soil types, both codes had almost the same category.

It is important to mention that each code has his method of calculating the seismic load wither it is static or dynamic, and the Australian code is much more comfortable than the American code but since the American code is more conservative, its common global used by the engineers.

4.10. Ductility Demand

The used ductility displacement for every wall is 262mm in Australian and 361mm in American code which is accepted in both codes, and this different allowable in codes came based on the used forces and safety factors in each.

It is clear to us after using both codes that the Australian code applied loads values less than the American, and that will lead use less factor of safety in the Australian than the American.

Chapter five

5. DISCUSSION OF RESULTS

5.1. Base shear

There is a distinct and essential different between the two codes where the American code is more conservative in the subject mainly and using 85% of the static effects if the structure system is regular while the Australian is 80% which increase the amount of the base shear and indicate that American code is more protective while the Australian is more economical. On the other hand, the case of irregular are the same but the used coefficients along with the formulas to calculate the base shear based on the equivalent lateral force are entirely different, and the American gives a higher number than the American nearly 10% more than the Australian code.

5.2. Drift, P-delta effects and Participant mass ratio

The American code contain more details and criteria than the Australian in terms of drift calculation by categorised the building type on the time history based while the Australian code is more generalized and have only one type and time history formula, the American code considered the response modification factor, importance factor, design spectral response acceleration parameter at a period of 1.0 s, fundamental period of the structure, long-period transition period, the mapped maximum considered earthquake spectral response acceleration parameter to calculate the P-delta effect as essential variables while the Australian code relies on the consideration level only.

The two codes have the same criteria and there no different. The recommended value for both codes is ninety per cent from the total mass should be involved during the seismic load calculation.

5.3. Accidental torsion effects

Both codes implemented the same procedure by relying on the maximum drift story on each edge diaphragm floor divided on average and check it with 1.2 value and considering the zone category, and if it's below this value, there will be no accidental torsion and if its more, both code will consider it as torsional, the American code add a new category if the value is more than 1.4 to be extreme torsion.

5.4. Natural time period: -

Both codes contain nearly the same category with steel buckling- resistance brace frame for the American code and Australian. Each code using the same formula with a different coefficient which affecting the outcome of the time period. American code is more describable and specific than Australian, and the outcomes profoundly different which can be seen in our outcome approximately forty per cent higher in American code.

5.5. Load and load combination: -

In general, significant or difference found between the American and the Australian code regarding load and combination. American code shows more conservative than the Australian. This differences cloud reach thirty-per-cent more in loads and ten per cent for the load combination safety factors. Moreover, this will be reflected on the displacement of the building which showed a clear difference between the two codes and the displacement is twenty per cent higher in the American than the Australian.

5.6. Flexure and shear

The difference between the codes occurred due to the overstrength factor. The design of the two codes is slightly different, because of the overstrength influence and the modification response factors. The mentioned codes contain different design loads since constant acceleration ground motion of the 475yrs return period are different to create the seismic forces.

Overall, the ASCE7 are more conservative than the AS1170 and that evident in the base shear outcomes which shows at least 20% more than the Australian code and at the same time, the Australian code gives more safety in terms of calculating the time period which gave more time period than the American in a few milliseconds. Regarding the soil type and dealing with soil types, both codes had almost the same category.

It is important to mention that each code has his method of calculating the seismic load wither it is static or dynamic, and the Australian code is much more comfortable than the American code but since the American code is more conservative, its common global used by the engineers.

5.7. Ductility Demand

The used ductility displacement for every wall is 262mm in Australian and 361mm in American code which is accepted in both codes, and this different allowable in codes came based on the used forces and safety factors in each.

It is clear to us after using both codes that the Australian code applied loads values less than the American, and that will lead use less factor of safety in the Australian than the American.

CHAPTER SIX

SUMMARY AND CONCLUSION

Below, is the conclusion and outcomes from the comparison between the American and the Australian code based on the seismic design which is adopted based on analysing a seventy-floors building: -

- The calculated natural time period using the Australian code is higher than the American which will make the Australian code safer since the structure exposed to more period.
- The used imposed, and the dead load factor for the load combination in the Australian code is less than the American code which will lead to fewer forces outcomes, and the P-delta effects will be less as well.
- The Australian code covers up to 200m only for the wind calculation and its need more study in order to cover higher high limit.
- The Australian code takes ±0.1b for the eccentric centre of mass while the American take it ±0.05b for the torsional effects.
- The Australian code allows for the 90% different from the static and dynamic while the need 100%.
- The drift limit in the Australian code is 1.5% while the American is 2-2.5% which is logical since the loading in the Australian is less.
- The final base shear is 20% more in the American than the Australian.
- Australian code required to take at least 5% of the structure weight to be used in the horizontal load effects while the American required 4%.

Chapter Seven

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- Gad, E.F., Chandler, A.M., Duffield, C.F. and Hutchinson, G.L. (1999). ***“Earthquake ductility and over strength in domestic structures”***, Journal of Structural Engineering and Mechanics, Vol. 8. On the other hand, the American code identifies the irregularity as shown below.

APPENDIX

Australian code

Story	*Total Weight in*	*SUM*
T1- Story 11-70	22643	1358568
T1- Story 10	133670	160404
T1- Story 1- 9	22356	241446
TOTAL	1760418	

$$F_i = k_{F,i}V$$

$$= \frac{W_i h_i^k}{\sum_{j=1}^{n}\left(W_j h_j^k\right)}\left[k_p Z C_h\left(T_1\right)\frac{S_p}{\mu}\right]W_t$$

k= 2

Kf,i= 0

V= 88021 Kn

Equivalent Lateral Forces for Building Response in X and Y Directions								
Level	W (Kn)	h (m)	*f. Hight (m)*	$W h^k$	$K_{F,i}$	F (Kn)	V (Kn)	M (Kn.m)
70	22643	210	3	998547480	0	3677	3677	11032
69	22643	207	3	970221337	0	3573	3573	10720
68	22643	204	3	942302765	0	3470	3470	10411
67	22643	201	3	914791763	0	3369	3369	10107
66	22643	198	3	887688331	0	3269	3269	9808
65	22643	195	3	860992470	0	3171	3171	9513
64	22643	192	3	834704179	0	3074	3074	9222
63	22643	189	3	808823459	0	2979	2979	8936
62	22643	186	3	783350309	0	2885	2885	8655
61	22643	183	3	758284729	0	2793	2793	8378
60	22643	180	3	733626720	0	2702	2702	8105
59	22643	177	3	709376281	0	2613	2613	7838
58	22643	174	3	685533413	0	2525	2525	7574
57	22643	171	3	662098115	0	2438	2438	7315
56	22643	168	3	639070387	0	2354	2354	7061
55	22643	165	3	616450230	0	2270	2270	6811
54	22643	162	3	594237643	0	2188	2188	6565
53	22643	159	3	572432627	0	2108	2108	6325
52	22643	156	3	551035181	0	2029	2029	6088
51	22643	153	3	530045305	0	1952	1952	5856
50	22643	150	3	509463000	0	1876	1876	5629
49	22643	147	3	489288265	0	1802	1802	5406
48	22643	144	3	469521101	0	1729	1729	5188
47	22643	141	3	450161507	0	1658	1658	4974
46	22643	138	3	431209483	0	1588	1588	4764
45	22643	135	3	412665030	0	1520	1520	4559
44	22643	132	3	394528147	0	1453	1453	4359
43	22643	129	3	376798835	0	1388	1388	4163
42	22643	126	3	359477093	0	1324	1324	3972
41	22643	123	3	342562921	0	1262	1262	3785
40	22643	120	3	326056320	0	1201	1201	3602
39	22643	117	3	309957289	0	1142	1142	3425
38	22643	114	3	294265829	0	1084	1084	3251
37	22643	111	3	278981939	0	1027	1027	3082
36	22643	108	3	264105619	0	973	973	2918
35	22643	105	3	249636870	0	919	919	2758
34	22643	102	3	235575691	0	868	868	2603
33	22643	99	3	221922083	0	817	817	2452
32	22643	96	3	208676045	0	769	769	2306
31	22643	93	3	195837577	0	721	721	2164

30	22643	90	3	183406680	0	675	675	2026
29	22643	87	3	171383353	0	631	631	1894
28	22643	84	3	159767597	0	588	588	1765
27	22643	81	3	148559411	0	547	547	1641
26	22643	78	3	137758795	0	507	507	1522
25	22643	75	3	127365750	0	469	469	1407
24	22643	72	3	117380275	0	432	432	1297
23	22643	69	3	107802371	0	397	397	1191
22	22643	66	3	98632037	0	363	363	1090
21	22643	63	3	89869273	0	331	331	993
20	22643	60	3	81514080	0	300	300	901
19	22643	57	3	73566457	0	271	271	813
18	22643	54	3	66026405	0	243	243	729
17	22643	51	3	58893923	0	217	217	651
16	22643	48	3	52169011	0	192	192	576
15	22643	45	3	45851670	0	169	169	507
14	22643	42	3	39941899	0	147	147	441
13	22643	39	3	34439699	0	127	127	381
12	22643	36	3	29345069	0	108	108	324
11	22643	33	3	24658009	0	91	91	272
10	133670	30	3	120303257	0	443	443	1329
9	22356	27	3	16297572	0	60	60	180
8	22356	24	3	12877094	0	47	47	142
7	22356	21	3	9859025	0	36	36	109
6	22356	18	3	7243365	0	27	27	80
5	22356	15	3	5030115	0	19	19	56
4	22356	12	3	3219274	0	12	12	36
3	22356	9	3	1810841	0	7	7	20
2	22356	6	3	804818	0	3	3	9
1	22356	3		201205	0	1	1	0
Total	1693443			23900281699	1	88020		

Check Irregularity of Soft Story

Table 12.3-2 Vertical Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Stiffness-Soft Story Irregularity:** Stiffness-soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	**Stiffness-Extreme Soft Story Irregularity:** Stiffness-extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F

Story	story stiffness in X-direction	$K_i/K_{i+1} \geq 0.7$	Check	$K_{mi} = avg(K_{i-1,i-2,i-3})$	$K_i/K_{mi} \geq 0.8$	Check
70	72225.79	-	-			
69	159963.18	2.21	regular			
68	243336.07	1.52	regular			
67	321998.20	1.32	regular	158508.34	2.03	regular
66	395926.57	1.23	regular	241765.82	1.64	regular
65	465130.75	1.17	regular	320420.28	1.45	regular
64	529755.32	1.14	regular	394351.84	1.34	regular
63	590001.65	1.11	regular	463604.22	1.27	regular
62	646128.88	1.10	regular	528295.91	1.22	regular
61	698423.08	1.08	regular	588628.62	1.19	regular
60	747546.48	1.07	regular	644851.21	1.16	regular
59	793966.38	1.06	regular	697366.15	1.14	regular
58	837524.59	1.05	regular	746645.31	1.12	regular
57	878477.16	1.05	regular	793012.48	1.11	regular
56	917117.81	1.04	regular	836656.04	1.10	regular
55	953783.69	1.04	regular	877706.52	1.09	regular
54	988356.41	1.04	regular	916459.55	1.08	regular
53	1021269.94	1.03	regular	953085.97	1.07	regular
52	1052545.42	1.03	regular	987803.35	1.07	regular
51	1082448.20	1.03	regular	1020723.92	1.06	regular
50	1111296.03	1.03	regular	1052087.85	1.06	regular

49	1139212.92	1.03	regular	1082096.55	1.05	regular
48	1166240.76	1.02	regular	1110985.72	1.05	regular
47	1192582.20	1.02	regular	1138916.57	1.05	regular
46	1218508.56	1.02	regular	1166011.96	1.05	regular
45	1244215.55	1.02	regular	1192443.84	1.04	regular
44	1269897.70	1.02	regular	1218435.44	1.04	regular
43	1294569.93	1.02	regular	1244207.27	1.04	regular
42	1318512.93	1.02	regular	1269561.06	1.04	regular
41	1342900.45	1.02	regular	1294326.85	1.04	regular
40	1367911.23	1.02	regular	1318661.10	1.04	regular
39	1393987.96	1.02	regular	1343108.20	1.04	regular
38	1421096.80	1.02	regular	1368266.55	1.04	regular
37	1449440.45	1.02	regular	1394332.00	1.04	regular
36	1479262.72	1.02	regular	1421508.40	1.04	regular
35	1510843.82	1.02	regular	1449933.33	1.04	regular
34	1544506.75	1.02	regular	1479849.00	1.04	regular
33	1580628.45	1.02	regular	1511537.76	1.05	regular
32	1619648.51	1.02	regular	1545326.34	1.05	regular
31	1662081.71	1.03	regular	1581594.57	1.05	regular
30	1708534.01	1.03	regular	1620786.22	1.05	regular
29	1759725.40	1.03	regular	1663421.41	1.06	regular
28	1816522.30	1.03	regular	1710113.71	1.06	regular
27	1879984.42	1.03	regular	1761593.90	1.07	regular
26	1951431.04	1.04	regular	1818744.04	1.07	regular
25	2032535.67	1.04	regular	1882645.92	1.08	regular
24	2125460.06	1.05	regular	1954650.38	1.09	regular
23	2233048.64	1.05	regular	2036475.59	1.10	regular
22	2359117.34	1.06	regular	2130348.12	1.11	regular
21	2508894.01	1.06	regular	2239208.68	1.12	regular
20	2689733.21	1.07	regular	2367020.00	1.14	regular
19	2912358.65	1.08	regular	2519248.19	1.16	regular
18	3193155.91	1.10	regular	2703661.96	1.18	regular
17	3557795.16	1.11	regular	2931749.26	1.21	regular
16	4049754.53	1.14	regular	3221103.24	1.26	regular
15	4749870.13	1.17	regular	3600235.20	1.32	regular
14	5820766.43	1.23	regular	4119139.94	1.41	regular
13	7700900.43	1.32	regular	4873463.70	1.58	regular
12	11537006.99	1.50	regular	6090512.33	1.89	regular
11	31967399.25	2.77	regular	8352891.28	3.83	regular
10	18273601.22	0.57	Soft	17068435.56	1.07	regular
9	9979837.08	0.55	Soft	20592669.15	0.48	soft
8	8505115.21	0.85	regular	20073612.51	0.42	soft
7	7994646.09	0.94	regular	12252851.17	0.65	soft
6	7806401.99	0.98	regular	8826532.79	0.88	regular
5	7940937.47	1.02	regular	8102054.43	0.98	regular
4	8142221.64	1.03	regular	7913995.18	1.03	regular
3	8799502.75	1.08	regular	7963187.03	1.11	regular
2	10815677.15	1.23	regular	8294220.62	1.30	regular
1	23791238.82	2.20	regular	9252467.18	2.57	regular

Story	Diaphragm	Load Case/Combo	the point displacement in direction X in each story δ (Uxmm)	Hieght(mm)	story drift ⊿e mm	elastic drift ⊿ mm	Allowblw Drift (Δa mm)	check (Δm< Δa)
T2-Story70	D1	SPX Max	810.34	3000.00	13.03	33.88	45.00	achived
T2-Story69	D1	SPX Max	797.31	3000.00	13.27	34.50	45.00	achived
T2-Story68	D1	SPX Max	784.04	3000.00	13.34	34.68	45.00	achived
T2-Story67	D1	SPX Max	770.70	3000.00	13.43	34.91	45.00	achived
T2-Story66	D1	SPX Max	757.27	3000.00	13.53	35.19	45.00	achived
T2-Story65	D1	SPX Max	743.74	3000.00	13.66	35.51	45.00	achived
T2-Story64	D1	SPX Max	730.08	3000.00	13.80	35.87	45.00	achived
T2-Story63	D1	SPX Max	716.29	3000.00	13.94	36.25	45.00	achived
T2-Story62	D1	SPX Max	702.34	3000.00	14.10	36.67	45.00	achived
T2-Story61	D1	SPX Max	688.24	3000.00	14.27	37.10	45.00	achived
T2-Story60	D1	SPX Max	673.97	3000.00	14.44	37.54	45.00	achived
T2-Story59	D1	SPX Max	659.54	3000.00	14.62	38.00	45.00	achived
T2-Story58	D1	SPX Max	644.92	3000.00	14.79	38.46	45.00	achived
T2-Story57	D1	SPX Max	630.13	3000.00	14.97	38.93	45.00	achived
T2-Story56	D1	SPX Max	615.15	3000.00	15.15	39.40	45.00	achived
T2-Story55	D1	SPX Max	600.00	3000.00	15.27	39.70	45.00	achived
T2-Story54	D1	SPX Max	584.73	3000.00	15.39	40.02	45.00	achived
T2-Story53	D1	SPX Max	569.34	3000.00	15.56	40.47	45.00	achived
T2-Story52	D1	SPX Max	553.77	3000.00	15.73	40.90	45.00	achived
T2-Story51	D1	SPX Max	538.04	3000.00	15.89	41.31	45.00	achived
T2-Story50	D1	SPX Max	522.15	3000.00	16.04	41.71	45.00	achived
T2-Story49	D1	SPX Max	506.11	3000.00	16.18	42.07	45.00	achived
T2-Story48	D1	SPX Max	489.93	3000.00	16.31	42.40	45.00	achived
T2-Story47	D1	SPX Max	473.62	3000.00	16.43	42.71	45.00	achived
T2-Story46	D1	SPX Max	457.20	3000.00	16.53	42.97	45.00	achived
T2-Story45	D1	SPX Max	440.67	3000.00	16.61	43.20	45.00	achived
T2-Story44	D1	SPX Max	424.05	3000.00	16.69	43.38	45.00	achived
T2-Story43	D1	SPX Max	407.37	3000.00	16.74	43.52	45.00	achived
T2-Story42	D1	SPX Max	390.63	3000.00	16.77	43.61	45.00	achived
T2-Story41	D1	SPX Max	373.86	3000.00	16.79	43.65	45.00	achived
T2-Story40	D1	SPX Max	357.07	3000.00	16.78	43.63	45.00	achived
T2-Story39	D1	SPX Max	340.29	3000.00	16.75	43.56	45.00	achived
T2-Story38	D1	SPX Max	323.53	3000.00	16.70	43.42	45.00	achived
T2-Story37	D1	SPX Max	306.83	3000.00	16.63	43.23	45.00	achived
T2-Story36	D1	SPX Max	290.21	3000.00	16.52	42.96	45.00	achived
T2-Story35	D1	SPX Max	273.68	3000.00	16.39	42.62	45.00	achived
T2-Story34	D1	SPX Max	257.29	3000.00	16.23	42.21	45.00	achived
T2-Story33	D1	SPX Max	241.06	3000.00	16.05	41.72	45.00	achived
T2-Story32	D1	SPX Max	225.01	3000.00	15.82	41.14	45.00	achived
T2-Story31	D1	SPX Max	209.19	3000.00	15.57	40.49	45.00	achived
T2-Story30	D1	SPX Max	193.62	3000.00	15.29	39.74	45.00	achived
T2-Story29	D1	SPX Max	178.33	3000.00	14.96	38.90	45.00	achived
T2-Story28	D1	SPX Max	163.37	3000.00	14.60	37.97	45.00	achived
T2-Story27	D1	SPX Max	148.77	3000.00	14.21	36.93	45.00	achived
T2-Story26	D1	SPX Max	134.56	3000.00	13.77	35.80	45.00	achived
T2-Story25	D1	SPX Max	120.79	3000.00	13.29	34.55	45.00	achived
T2-Story24	D1	SPX Max	107.51	3000.00	12.77	33.19	45.00	achived
T2-Story23	D1	SPX Max	94.74	3000.00	12.20	31.71	45.00	achived
T2-Story22	D1	SPX Max	82.55	3000.00	11.58	30.11	45.00	achived
T2-Story21	D1	SPX Max	70.97	3000.00	10.92	28.38	45.00	achived
T2-Story20	D1	SPX Max	60.05	3000.00	10.20	26.52	45.00	achived

T2-Story19	D1	SPX Max	49.85	3000.00	9.43	24.52	45.00	achived
T2-Story18	D1	SPX Max	40.42	3000.00	8.61	22.38	45.00	achived
T2-Story17	D1	SPX Max	31.82	3000.00	7.72	20.08	45.00	achived
T2-Story16	D1	SPX Max	24.09	3000.00	6.78	17.63	45.00	achived
T2-Story15	D1	SPX Max	17.31	3000.00	5.78	15.02	45.00	achived
T2-Story14	D1	SPX Max	11.54	3000.00	4.71	12.24	45.00	achived
T2-Story13	D1	SPX Max	6.83	3000.00	3.55	9.24	45.00	achived
T2-Story12	D1	SPX Max	3.28	3000.00	3.28	8.52	45.00	achived
T2-Story11	D1	SPX Max	0.00	3000.00	0.00	0.00	45.00	achived
T2-Story10	D1	SPX Max	0.00	3000.00	0.00	0.00	45.00	achived

American code.

Story	Total Weight in	SUM
T1- Story 11-70	22642.8	1358568
T1- Story 10	133670.285	160404.342
T1- Story 1- 9	22356.0663	241445.516
TOTAL	1760418	

$$F_x = C_{vx} V \quad (12.8\text{-}11)$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad (12.8\text{-}12)$$

k= 2.634
Cs= 0.052
V= 91541.7286

Equivalent Lateral Forces for Building Response in X and Y Directions								
Level	W (Kn)	h (m)	*f. Hight (m)*	$W h^k$	C_{vx}	F (Kn)	V (Kn)	M (Kn.m)
70	22643	210	3	29624822223.57	0.05052	4624.57	4624.6	13873.7238
69	22643	207	3	28523053758.53	0.04864	4452.58	4452.6	13357.75
68	22643	204	3	27447070101.25	0.04681	4284.62	4284.6	12853.8516
67	22643	201	3	26396633694.84	0.04501	4120.64	4120.6	12361.9174
66	22643	198	3	25371505700.23	0.04327	3960.61	3960.6	11881.8354
65	22643	195	3	24371445969.95	0.04156	3804.50	3804.5	11413.4933
64	22643	192	3	23396213020.98	0.03990	3652.26	3652.3	10956.7779
63	22643	189	3	22445564006.58	0.03828	3503.86	3503.9	10511.5755
62	22643	186	3	21519254687.12	0.03670	3359.26	3359.3	10077.7717
61	22643	183	3	20617039399.77	0.03516	3218.42	3218.4	9655.25154
60	22643	180	3	19738671027.03	0.03366	3081.30	3081.3	9243.89919
59	22643	177	3	18883900964.08	0.03220	2947.87	2947.9	8843.59826
58	22643	174	3	18052479084.80	0.03078	2818.08	2818.1	8454.23162
57	22643	171	3	17244153706.43	0.02941	2691.89	2691.9	8075.6814
56	22643	168	3	16458671552.76	0.02807	2569.28	2569.3	7707.82898
55	22643	165	3	15695777715.86	0.02677	2450.18	2450.2	7350.55499
54	22643	162	3	14955215616.10	0.02550	2334.58	2334.6	7003.73927
53	22643	159	3	14236726960.47	0.02428	2222.42	2222.4	6667.26086
52	22643	156	3	13540051699.16	0.02309	2113.67	2113.7	6340.99797
51	22643	153	3	12864927979.99	0.02194	2008.28	2008.3	6024.82797
50	22643	150	3	12211092101.01	0.02082	1906.21	1906.2	5718.62737
49	22643	147	3	11578278460.66	0.01974	1807.42	1807.4	5422.27178
48	22643	144	3	10966219505.73	0.01870	1711.88	1711.9	5135.6359
47	22643	141	3	10374645676.71	0.01769	1619.53	1619.5	4858.5935
46	22643	138	3	9803285350.46	0.01672	1530.34	1530.3	4591.01737
45	22643	135	3	9251864779.98	0.01578	1444.26	1444.3	4332.7793
44	22643	132	3	8720108031.07	0.01487	1361.25	1361.3	4083.75009
43	22643	129	3	8207736915.62	0.01400	1281.27	1281.3	3843.79944
42	22643	126	3	7714470921.36	0.01316	1204.27	1204.3	3612.79599
41	22643	123	3	7240027137.66	0.01235	1130.20	1130.2	3390.60724
40	22643	120	3	6784120177.14	0.01157	1059.03	1059	3177.09956
39	22643	117	3	6346462092.76	0.01082	990.71	990.71	2972.13808
38	22643	114	3	5926762289.99	0.01011	925.20	925.2	2775.58672
37	22643	111	3	5524727433.55	0.00942	862.44	862.44	2587.3081
36	22643	108	3	5140061348.39	0.00877	802.39	802.39	2407.16352
35	22643	105	3	4772464914.30	0.00814	745.00	745	2235.01291
34	22643	102	3	4421635953.45	0.00754	690.24	690.24	2070.71474
33	22643	99	3	4087269110.46	0.00697	638.04	638.04	1914.12601
32	22643	96	3	3769055723.86	0.00643	588.37	588.37	1765.10217
31	22643	93	3	3466683688.46	0.00591	541.17	541.17	1623.49707

30	22643	90	3	3179837307.39	0.00542	496.39	496.39	1489.16284
29	22643	87	3	2908197132.73	0.00496	453.98	453.98	1361.9499
28	22643	84	3	2651439793.59	0.00452	413.90	413.9	1241.70681
27	22643	81	3	2409237809.93	0.00411	376.09	376.09	1128.28019
26	22643	78	3	2181259390.67	0.00372	340.50	340.5	1021.51466
25	22643	75	3	1967168213.79	0.00335	307.08	307.08	921.252734
24	22643	72	3	1766623186.41	0.00301	275.78	275.78	827.334658
23	22643	69	3	1579278181.89	0.00269	246.53	246.53	739.598339
22	22643	66	3	1404781750.69	0.00240	219.29	219.29	657.879189
21	22643	63	3	1242776801.38	0.00212	194.00	194	582.009977
20	22643	60	3	1092900246.81	0.00186	170.61	170.61	511.820664
19	22643	57	3	954782610.03	0.00163	149.05	149.05	447.138218
18	22643	54	3	828047583.12	0.00141	129.26	129.26	387.78641
17	22643	51	3	712311530.26	0.00121	111.20	111.2	333.585578
16	22643	48	3	607182924.72	0.00104	94.78	94.784	284.352363
15	22643	45	3	512261706.35	0.00087	79.97	79.966	239.899412
14	22643	42	3	427138542.53	0.00073	66.68	66.678	200.035029
13	22643	39	3	351393970.65	0.00060	54.85	54.854	164.562773
12	22643	36	3	284597393.03	0.00049	44.43	44.427	133.28099
11	22643	33	3	226305885.21	0.00039	35.33	35.327	105.982251
10	133670	30	3	1039374981.78	0.00177	162.25	162.25	486.754024
9	22356	27	3	131706601.55	0.00022	20.56	20.56	61.6800668
8	22356	24	3	96576575.02	0.00016	15.08	15.076	45.2281778
7	22356	21	3	67939291.14	0.00012	10.61	10.606	31.8169322
6	22356	18	3	45267151.56	0.00008	7.07	7.0664	21.1992482
5	22356	15	3	28003980.42	0.00005	4.37	4.3716	13.1146606
4	22356	12	3	15558179.97	0.00003	2.43	2.4287	7.28611602
3	22356	9	3	7292394.57	0.00001	1.14	1.1384	3.41513165
2	22356	6	3	2506373.46	0.00000	0.39	0.3913	1.17377019
1	22356	3		403768.82	0.00000	0.06	0.063	0
Total	1693443			586412301741.56	1.00	91541.67		

Check Irregularity of Soft Story

Table 12.3-2 Vertical Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Stiffness-Soft Story Irregularity:** Stiffness-soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	**Stiffness-Extreme Soft Story Irregularity:** Stiffness-extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F

Story	story stiffness in X-direction	$K_i/K_{i+1} \geq 0.7$	Check	$K_{mi} = avg(K_{i-1,i-2,i-3})$	$K_i/K_{mi} \geq 0.8$	Check
70	72225.79	-	-			
69	159963.18	2.21	regular			
68	243336.07	1.52	regular			
67	321998.20	1.32	regular	158508.34	2.03	regular
66	395926.57	1.23	regular	241765.82	1.64	regular
65	465130.75	1.17	regular	320420.28	1.45	regular
64	529755.32	1.14	regular	394351.84	1.34	regular
63	590001.65	1.11	regular	463604.22	1.27	regular
62	646128.88	1.10	regular	528295.91	1.22	regular
61	698423.08	1.08	regular	588628.62	1.19	regular
60	747546.48	1.07	regular	644851.21	1.16	regular
59	793966.38	1.06	regular	697366.15	1.14	regular
58	837524.59	1.05	regular	746645.31	1.12	regular
57	878477.16	1.05	regular	793012.48	1.11	regular
56	917117.81	1.04	regular	836656.04	1.10	regular
55	953783.69	1.04	regular	877706.52	1.09	regular
54	988356.41	1.04	regular	916459.55	1.08	regular
53	1021269.94	1.03	regular	953085.97	1.07	regular
52	1052545.42	1.03	regular	987803.35	1.07	regular
51	1082448.20	1.03	regular	1020723.92	1.06	regular
50	1111296.03	1.03	regular	1052087.85	1.06	regular

Computation for Torsional Irregularity with ELF Loads Acting in X Direction and Torsional Moment Applied Counterclockwise(max./avg. Displacement)

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 \qquad (12.8\text{-}14)$$

where

δ_{max} = the maximum displacement at Level x computed assuming $A_x = 1$ (in. or mm)

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x computed assuming $A_x = 1$ (in. or mm)

The torsional amplification factor (A_x) shall not be less than 1 and is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

Story	Load Case/Combo	Direction	Maximum(mm)	Average(mm)	Drift Ratio(Max/Avg.)	Irregularity	Ax	Used Ax	Ecc.* used Ax	Max. used Ecc.
T1-Story70	QXN	X	0.0010	0.0010	1.33	irregular	1.24	1.24	0.06	0.0676
T1-Story69	QXN	X	0.0010	0.0010	1.34	irregular	1.24	1.24	0.06	
T1-Story68	QXN	X	0.0010	0.0010	1.34	irregular	1.24	1.24	0.06	
T1-Story67	QXN	X	0.0010	0.0010	1.34	irregular	1.24	1.24	0.06	
T1-Story66	QXN	X	0.0010	0.0010	1.34	irregular	1.24	1.24	0.06	
T1-Story65	QXN	X	0.0010	0.0010	1.34	irregular	1.24	1.24	0.06	
T1-Story64	QXN	X	0.0010	0.0010	1.34	irregular	1.24	1.24	0.06	
T1-Story63	QXN	X	0.0010	0.0010	1.34	irregular	1.24	1.24	0.06	
T1-Story62	QXN	X	0.0010	0.0010	1.34	irregular	1.24	1.24	0.06	
T1-Story61	QXN	X	0.0010	0.0010	1.34	irregular	1.24	1.24	0.06	
T1-Story60	QXN	X	0.0010	0.0010	1.34	irregular	1.24	1.24	0.06	
T1-Story59	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story58	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story57	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story56	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story55	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story54	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story53	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story52	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story51	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story50	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story49	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story48	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story47	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story46	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story45	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story44	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story43	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story42	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story41	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story40	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story39	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story38	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story37	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story36	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story35	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story34	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story33	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story32	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story31	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story30	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story29	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story28	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story27	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story26	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story25	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story24	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story23	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story22	QXN	X	0.0010	0.0010	1.34	irregular	1.25	1.25	0.06	
T1-Story21	QXN	X	0.0010	0.0010	1.35	irregular	1.26	1.26	0.06	
T1-Story20	QXN	X	0.0010	0.0010	1.35	irregular	1.26	1.26	0.06	
T1-Story19	QXN	X	0.0010	0.0010	1.35	irregular	1.27	1.27	0.06	
T1-Story18	QXN	X	0.0010	0.0010	1.36	irregular	1.28	1.28	0.06	
T1-Story17	QXN	X	0.0010	0.0010	1.36	irregular	1.28	1.28	0.06	
T1-Story16	QXN	X	0.0010	0.0010	1.37	irregular	1.30	1.30	0.06	
T1-Story15	QXN	X	0.0010	0.0010	1.38	irregular	1.31	1.31	0.07	
T1-Story14	QXN	X	0.0010	0.0010	1.39	irregular	1.33	1.33	0.07	
T1-Story13	QXN	X	0.0010	0.0010	1.40	irregular	1.35	1.35	0.07	
T1-Story12	QXN	X	0.0010	0.0010	1.38	irregular	1.33	1.33	0.07	
T1-Story9	QXN	X	0.0010	0.0010	1.01	regular	0.70	1.00	0.05	
T1-Story8	QXN	X	0.0010	0.0010	1.01	regular	0.71	1.00	0.05	
T1-Story7	QXN	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story6	QXN	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story5	QXN	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story4	QXN	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story3	QXN	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story2	QXN	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story1	QXN	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story70	SPX Max	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story69	SPX Max	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story68	SPX Max	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story67	SPX Max	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story66	SPX Max	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story65	SPX Max	X	0.0010	0.0010	1.02	regular	0.72	1.00	0.05	
T1-Story64	SPX Max	X	0.0010	0.0010	1.02	regular	0.73	1.00	0.05	
T1-Story63	SPX Max	X	0.0010	0.0010	1.02	regular	0.73	1.00	0.05	
T1-Story62	SPX Max	X	0.0010	0.0010	1.02	regular	0.73	1.00	0.05	
T1-Story61	SPX Max	X	0.0010	0.0010	1.02	regular	0.73	1.00	0.05	
T1-Story60	SPX Max	X	0.0010	0.0010	1.02	regular	0.73	1.00	0.05	
T1-Story59	SPX Max	X	0.0010	0.0010	1.02	regular	0.73	1.00	0.05	
T1-Story58	SPX Max	X	0.0010	0.0010	1.03	regular	0.73	1.00	0.05	
T1-Story57	SPX Max	X	0.0010	0.0010	1.03	regular	0.73	1.00	0.05	
T1-Story56	SPX Max	X	0.0010	0.0010	1.03	regular	0.73	1.00	0.05	
T1-Story55	SPX Max	X	0.0010	0.0010	1.03	regular	0.73	1.00	0.05	
T1-Story54	SPX Max	X	0.0010	0.0010	1.03	regular	0.73	1.00	0.05	
T1-Story53	SPX Max	X	0.0010	0.0010	1.03	regular	0.73	1.00	0.05	
T1-Story52	SPX Max	X	0.0010	0.0010	1.03	regular	0.74	1.00	0.05	
T1-Story51	SPX Max	X	0.0010	0.0010	1.03	regular	0.74	1.00	0.05	
T1-Story50	SPX Max	X	0.0010	0.0010	1.03	regular	0.74	1.00	0.05	

check of story drift in Y-Direction								
	Cd=	2.25	**Time Period (T) =**		**8.91**	**second**		
D1								
Story	**Diaphragm**	**Load Case/Combo**	**the point displacement in direction Y in each story δ (Uymm)**	**Hieght(mm)**	**story drift Δs mm**	**elastic drift Δm mm**	**Allowblw Drift (Δa mm)**	**check (Δm< Δa)**
T2-Story70	D1	SPX Max	661.73	3000	16.20	25.51	75.00	Achieved
T2-Story69	D1	SPX Max	645.53	3000	11.25	17.71	75.00	Achieved
T2-Story68	D1	SPX Max	634.28	3000	11.29	17.78	75.00	Achieved
T2-Story67	D1	SPX Max	622.99	3000	11.36	17.88	75.00	Achieved
T2-Story66	D1	SPX Max	611.64	3000	11.43	18.00	75.00	Achieved
T2-Story65	D1	SPX Max	600.21	3000	11.52	18.14	75.00	Achieved
T2-Story64	D1	SPX Max	588.69	3000	11.61	18.29	75.00	Achieved
T2-Story63	D1	SPX Max	577.08	3000	11.72	18.45	75.00	Achieved
T2-Story62	D1	SPX Max	565.36	3000	11.83	18.62	75.00	Achieved
T2-Story61	D1	SPX Max	553.54	3000	11.94	18.81	75.00	Achieved
T2-Story60	D1	SPX Max	541.60	3000	12.06	18.99	75.00	Achieved
T2-Story59	D1	SPX Max	529.54	3000	12.18	19.18	75.00	Achieved
T2-Story58	D1	SPX Max	517.36	3000	12.30	19.37	75.00	Achieved
T2-Story57	D1	SPX Max	505.06	3000	12.42	19.56	75.00	Achieved
T2-Story56	D1	SPX Max	492.64	3000	12.54	19.75	75.00	Achieved
T2-Story55	D1	SPX Max	480.10	3000	12.66	19.93	75.00	Achieved
T2-Story54	D1	SPX Max	467.44	3000	12.77	20.12	75.00	Achieved
T2-Story53	D1	SPX Max	454.67	3000	12.88	20.29	75.00	Achieved
T2-Story52	D1	SPX Max	441.79	3000	12.99	20.45	75.00	Achieved
T2-Story51	D1	SPX Max	428.81	3000	13.08	20.60	75.00	Achieved
T2-Story50	D1	SPX Max	415.73	3000	13.17	20.75	75.00	Achieved
T2-Story49	D1	SPX Max	402.55	3000	13.25	20.88	75.00	Achieved
T2-Story48	D1	SPX Max	389.30	3000	13.33	20.99	75.00	Achieved
T2-Story47	D1	SPX Max	375.97	3000	13.39	21.09	75.00	Achieved
T2-Story46	D1	SPX Max	362.58	3000	13.44	21.17	75.00	Achieved
T2-Story45	D1	SPX Max	349.14	3000	13.48	21.23	75.00	Achieved
T2-Story44	D1	SPX Max	335.66	3000	13.51	21.27	75.00	Achieved
T2-Story43	D1	SPX Max	322.16	3000	13.52	21.29	75.00	Achieved
T2-Story42	D1	SPX Max	308.64	3000	13.52	21.29	75.00	Achieved
T2-Story41	D1	SPX Max	295.12	3000	13.50	21.27	75.00	Achieved
T2-Story40	D1	SPX Max	281.62	3000	13.47	21.22	75.00	Achieved
T2-Story39	D1	SPX Max	268.15	3000	13.42	21.14	75.00	Achieved
T2-Story38	D1	SPX Max	254.73	3000	13.35	21.03	75.00	Achieved
T2-Story37	D1	SPX Max	241.37	3000	13.27	20.89	75.00	Achieved
T2-Story36	D1	SPX Max	228.11	3000	13.16	20.73	75.00	Achieved
T2-Story35	D1	SPX Max	214.95	3000	13.03	20.53	75.00	Achieved
T2-Story34	D1	SPX Max	201.91	3000	12.89	20.29	75.00	Achieved
T2-Story33	D1	SPX Max	189.03	3000	12.71	20.02	75.00	Achieved
T2-Story32	D1	SPX Max	176.32	3000	12.52	19.72	75.00	Achieved
T2-Story31	D1	SPX Max	163.80	3000	12.30	19.37	75.00	Achieved
T2-Story30	D1	SPX Max	151.49	3000	12.06	18.99	75.00	Achieved
T2-Story29	D1	SPX Max	139.44	3000	11.79	18.56	75.00	Achieved
T2-Story28	D1	SPX Max	127.65	3000	11.49	18.09	75.00	Achieved
T2-Story27	D1	SPX Max	116.16	3000	11.16	17.58	75.00	Achieved
T2-Story26	D1	SPX Max	105.00	3000	10.80	17.01	75.00	Achieved
T2-Story25	D1	SPX Max	94.20	3000	10.41	16.40	75.00	Achieved
T2-Story24	D1	SPX Max	83.79	3000	9.99	15.74	75.00	Achieved
T2-Story23	D1	SPX Max	73.80	3000	9.54	15.02	75.00	Achieved
T2-Story22	D1	SPX Max	64.26	3000	9.05	14.25	75.00	Achieved
T2-Story21	D1	SPX Max	55.22	3000	8.52	13.42	75.00	Achieved
T2-Story20	D1	SPX Max	46.70	3000	7.95	12.52	75.00	Achieved

(P-Delta) - EQy							
story	Height	P(kN)	$V_{y\ (kN)}$	$U_{X\ mm}$	$\Delta_{X\ mm}$	θ_X	check
T2-Story70	3000	18869.1956	953.601	661.726	16.196	0.106825074	P-DELTA
T2-Story69	3000	37738.391	2121.3144	645.53	11.246	0.066689147	NO P-DELTA
T2-Story68	3000	56607.5863	3250.2358	634.284	11.292	0.065555537	NO P-DELTA
T2-Story67	3000	75476.7822	4341.3322	622.992	11.355	0.065804598	NO P-DELTA
T2-Story66	3000	94345.9778	5395.6801	611.637	11.43	0.066619623	NO P-DELTA
T2-Story65	3000	113215.173	6414.4803	600.207	11.517	0.067758108	NO P-DELTA
T2-Story64	3000	132084.369	7399.0457	588.69	11.613	0.069103316	NO P-DELTA
T2-Story63	3000	150953.564	8350.7863	577.077	11.716	0.070595027	NO P-DELTA
T2-Story62	3000	169822.759	9271.1807	565.361	11.825	0.072200589	NO P-DELTA
T2-Story61	3000	188691.955	10161.7407	553.536	11.94	0.073904068	NO P-DELTA
T2-Story60	3000	207561.15	11023.97	541.596	12.058	0.07567668	NO P-DELTA
T2-Story59	3000	226430.346	11859.3214	529.538	12.178	0.077504962	NO P-DELTA
T2-Story58	3000	245299.541	12669.1555	517.36	12.3	0.07938399	NO P-DELTA
T2-Story57	3000	264168.737	13454.7047	505.06	12.42	0.081284472	NO P-DELTA
T2-Story56	3000	283037.932	14217.0434	492.64	12.54	0.083216919	NO P-DELTA
T2-Story55	3000	301907.127	14957.0713	480.1	12.657	0.085160132	NO P-DELTA
T2-Story54	3000	320776.323	15675.5065	467.443	12.772	0.087120102	NO P-DELTA
T2-Story53	3000	339645.518	16372.8929	454.671	12.88	0.08906254	NO P-DELTA
T2-Story52	3000	358514.713	17049.62	441.791	12.985	0.091015001	NO P-DELTA
T2-Story51	3000	377383.909	17705.9527	428.806	13.081	0.092935956	NO P-DELTA
T2-Story50	3000	396253.104	18342.0707	415.725	13.172	0.09485381	NO P-DELTA
T2-Story49	3000	415122.3	18958.1121	402.553	13.254	0.096740135	NO P-DELTA
T2-Story48	3000	433991.495	19554.2172	389.299	13.326	0.098586929	NO P-DELTA
T2-Story47	3000	452860.691	20130.5688	375.973	13.389	0.100400405	P-DELTA
T2-Story46	3000	471729.886	20687.4226	362.584	13.44	0.102156268	P-DELTA
T2-Story45	3000	490599.081	21225.1265	349.144	13.48	0.103859226	P-DELTA
T2-Story44	3000	509468.277	21744.1218	335.664	13.506	0.105482585	P-DELTA
T2-Story43	3000	528337.472	22244.9281	322.158	13.52	0.107037472	P-DELTA
T2-Story42	3000	547206.668	22728.1109	308.638	13.519	0.108495407	P-DELTA
T2-Story41	3000	566075.863	23194.2334	295.119	13.502	0.109842767	P-DELTA
T2-Story40	3000	584945.059	23643.8007	281.617	13.47	0.111082112	P-DELTA
T2-Story39	3000	603814.254	24077.1992	268.147	13.42	0.112183415	P-DELTA
T2-Story38	3000	622683.45	24494.6441	254.727	13.353	0.113149798	P-DELTA
T2-Story37	3000	641552.645	24896.1419	241.374	13.266	0.113951222	P-DELTA
T2-Story36	3000	660421.841	25281.4783	228.108	13.16	0.114591815	P-DELTA
T2-Story35	3000	679291.036	25650.2371	214.948	13.034	0.115059097	P-DELTA
T2-Story34	3000	698160.232	26001.8554	201.914	12.885	0.11532247	P-DELTA
T2-Story33	3000	717029.427	26335.7108	189.029	12.714	0.115385939	P-DELTA
T2-Story32	3000	735898.623	26651.2356	176.315	12.52	0.115234817	P-DELTA
T2-Story31	3000	754767.818	26948.0399	163.795	12.301	0.114843219	P-DELTA
T2-Story30	3000	773637.014	27226.0275	151.494	12.057	0.114201279	P-DELTA
T2-Story29	3000	792506.209	27485.4745	139.437	11.786	0.113277751	P-DELTA
T2-Story28	3000	811375.405	27727.0498	127.651	11.487	0.112047854	P-DELTA
T2-Story27	3000	830244.6	27951.7511	116.164	11.16	0.110494327	P-DELTA
T2-Story26	3000	849113.796	28160.7471	105.004	10.801	0.108558651	P-DELTA
T2-Story25	3000	867982.992	28355.1294	94.203	10.413	0.106251286	P-DELTA
T2-Story24	3000	886852.187	28535.6062	83.79	9.992	0.103513099	P-DELTA
T2-Story23	3000	905721.383	28702.1978	73.798	9.536	0.100305433	P-DELTA
T2-Story22	3000	924590.578	28854.0277	64.262	9.045	0.096611836	NO P-DELTA
T2-Story21	3000	943459.774	28989.3278	55.217	8.518	0.092406309	NO P-DELTA
T2-Story20	3000	962328.969	29105.7894	46.699	7.951	0.087628358	NO P-DELTA