ADDIS ABABA INSTITUTE OF TECHNOLOGY SCHOOL OF GRADUATE STUDIES

Seismic Performance Evaluation of Steel frame buildings (A case study on existing Smart Parking System in Addis Ababa City)

A thesis submitted to the school of Graduate Studies in Partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structures)

By Abrha Gebregziabher

Advisor, Dr. Abrham Gebre

(July 2019)

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Approved by Board of Examiners

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DECLARATION

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

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ABSTRACT

Earthquake hazards have an effect on significant damage to structures and cause widespread failure throughout buildings. Therefore, the assessment of the structural behavior against possible earthquakes should be the fundamental point on the design and construction process.

The seismic response and/or a performance prediction of buildings assessed based on nonlinear static analysis as it consumes less time to analysis and provides nearly equally reliable results with nonlinear dynamic analysis.

Currently, in this research, the methodology is extended to assess the performance of 3D structural steel frames using a G+10 and G+15 Smart Parking Systems as a case study which are located in Addis Ababa city. Imperative issues regarding direction of load application, loading profile, diaphragm effects, beam to column joint model and arrangement of incremental static analysis results to generate required data for performance based evaluations have been studied.

The result of this research shows that; the curve line for capacity curve (push over curve) of both case study buildings due to applying uniform lateral load distribution is above the capacity curve of triangular lateral load distribution. To this end, Conventional pushover analysis with triangular load pattern distribution gives higher displacement while the uniform lateral load pattern gives higher base shear. The Inter-Storey drift ratio of both the case study buildings is a maximum on the lower storey of the buildings. Hence, it concluded that failure of buildings due earthquake starts on the lower storey part of the buildings.

KEY WORDS: Seismic analysis, Response spectrum analysis, Elastic analysis, Pushover analysis, Steel frame, Capacity curve, Peak Ground Acceleration.

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NOTATIONS

EBCS	Ethiopian Building Code Standard
ATC	Applied Technology Council
FEMA	Federal Emergency Management Agency
IBC	International Building Code
NEIS	US National Earthquake Information Service
MPA	Modal Pushover Analysis procedure
RSA	Response Spectrum Analysis
UBC	The University of British Colombia
PGA	Peak Ground Accelerations
AEM	Applied Element Method
ESL	Equivalent Static Load
RC	Reinforced Concrete
CP	Collapse Prevention
IO	Immediate Occupancy
LS	Life Safety
F_b	Seismic Base Shear
W	Effective seismic weight of the structure
F_i	Horizontal Force Acting on Storey i
m_i	Mass of storey i
T_e	Effective fundamental period of the structure
δ_t	Target Displacement
δ_i	Inter storey drift of storey i
h_t	Height of storey i
d_t	Displacement at the floor i
α	Yield slope
μ	structure ductility
ϕ_i	Amplitude of the Elastic First Mode
V_y	Effective yield strength
K_e	Effective lateral stiffness
R	Behavior Factor

CHAPTER - I

1. INTRODUCTION

In tall buildings, the lateral load due to earthquake was a matter of concern. These lateral forces can produce critical stresses in the structure, induce undesirable lateral stresses in the structure, and induce undesirable vibration or causes excessive lateral sway of the structure. Hence, the seismic design performance of the structure should be the main concern and Knowing the forces affecting the structures and consequently assessing the behavior of the structures in terms of parameter like drift which is the magnitude of the lateral displacement at the top of the building relative to its base against possible earthquakes should be the fundamental point on the design and construction process of the structures. In other words, the performance of the structure should be known and investigated before construction.

The structural engineering community has developed a new generation of design and rehabilitation procedures that incorporate performance based engineering concepts. This aim can be achieved only by introducing some kind of nonlinear analysis into the seismic design methodology. In a short term, the most appropriate approach seems to be a combination of the nonlinear static (pushover) analysis and the response spectrum approach. A pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral forces, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads various structural elements yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Therefore, nonlinear static analysis, or pushover analysis, has become the preferred analysis procedure for design and seismic performance evaluation purposes as the procedure is relatively simple and considers post elastic behavior. However, the procedure involves certain approximations and simplifications that some amount of variation is always expected to exist in seismic demand prediction of pushover analysis.

The main aim of this study is to investigate the general behavior of steel frames with semi-rigid connections, to compute their performances and to investigate the effects of different connection types on the performance of the steel frame. Steel is by far most useful material for building construction in the world. Today steel industry is the basic or key industry in any country. Its strength is approximately ten times that of concrete, steel is an ideal material of modern construction. It’s mainly advantages are strength, speed of erection, prefabrication. Structural steel is used in load-bearing frames in buildings, and as members in trusses, bridges, and space frames. Steel, however, requires fire and corrosion protection. In steel buildings, claddings and dividing walls are made up of masonry or other materials, and often a concrete foundation is provided. Due to its large strength to weight ratio, steel structures tend to be more economical than concrete structures for tall buildings and large span buildings and bridges. Steel structures can be constructed very fast and this enables the structure to be used early thereby leading to overall economy steel offers much better compressive and tensile strength than concrete and enables lighter constructions.

The reason for the selection of steel structures to work on is, the proofs of the worldwide investigations that steel structures' performance are better than reinforced concrete structures' and to point out the advantages of steel structures in advance, where the usage of steel as a construction material is uncommon. Also, the reason for the selection of semi-rigid connection type instead of fixed or a hinge connection type is to achieve more realistic analysis results.

This study is generally established using a Finite element Package software, Seismostruct that can make non-linear static (pushover) analysis method to assess the capacity of an existing 3D steel frames to reach an adequate level of performance related to the performance limits set by FEMA 356,2000. Firstly, the software is validated by taking single regular steel planar frame as verification problem.

The models for the non-linear analysis of steel frames are proposed in the literature. The choice of the suitable model depends on the goals of the analysis and the structural properties. For structures with flexural collapse mechanisms, member by member or global models can be used to obtain reliable predictions. So accurate modeling of the nonlinear properties of various structural elements is very important in nonlinear analysis. In the present study, beams and columns were modeled with inelastic flexural deformations using fiber based element using the Seismostruct software.

1.1. STATEMENT OF THE PROBLEM

Building's structural design practice for earthquake in Ethiopia is based on the rules and regulations of Ethiopian Building Code Standards as a reference. However, in the design and construction process, the structural design result may not be further checked rigorously by the respective authority which aggravates the problem in earthquake performance of buildings. Therefore, a performance-based procedure for seismic performance evaluation of steel moment frames is presented that allows the designer to estimate the confidence level of satisfying the performance objectives.

This is being the case, to perform appropriate design for seismic behavior of structure during the design earthquake should consider the randomness and uncertainty in the calculation of seismic demand and structure capacity must be accounted for in seismic performance evaluation.

1.2. OBJECTIVE OF STUDY

2.2.1. GENERAL OBJECTIVE

The aim of this study is to assess the seismic performance of steel frame building using the conventional nonlinear static (pushover) analysis method to have an adequate level of performance related to the life safety of its occupants in terms of control of the storey drifts and maximum storey displacements by taking case study of Smart Parking System (Steel frame buildings), Addis Ababa City.

2.2.2. SPECIFIC OBJECTIVE

The objective of this study is to know seismic performance behavior of the designed steel frame building; by performing a case study on Smart Parking system (steel frames buildings) with different stories to withstand seismic levels with certain performance levels. Particularly, the study has the following sub-objectives:

- ☞ To evaluate seismic performance of steel frame building.
- ☞ To Account the effects of Randomness and uncertainty in the calculation of seismic demand and structure capacity using seismic performance evaluation.
- ☞ To understand the effects and the accuracy of various invariant lateral load patterns (Uniform and Triangular load distribution) utilized in conventional pushover analysis to predict the behavior imposed on structures.

1.3. SCOPE OF THE STUDY

The scope of this study is to employ seismic performance assessment using non linear static analysis on two existing steel framed structures; G+10 and G+15 smart parking system located in the seismic zone III of the country, Addis Ababa. The Seismostruct 2016 package program was used as a tool of this finite element analysis is carried out by taking a single seismic load with approximate hazard level of 0.7g ground acceleration for determining the Response Spectrum.

This research work attempts to model a three dimensional (3-D) framed structures subjected to earthquake loading and then employ a detail study on their seismic responses to easily see the effect by varying the number of parameters involved in the analysis and the damage index will investigated through lateral drift profile of the models and verify the analysis results in terms of absolute maximum storey displacement, maximum inter-storey drifts and storey ductility demands. To this end, the structural performance assessment of the case study steel framed structures applying fiber beam column model and conventional pushover analysis using the above mentioned software was based on the document provided by Client; Addis Ababa City Government Transport Programs Management Office considering all data specifications and detail working drawings regardless of soil structure interaction, bond slip and effects of vertical movement of the vehicles through the mechanical lift system.

CHAPTER II

2. LITERATURE REVIEW

2.1. GENERAL

To provide a detailed review of the literature related to modeling of structures in its entirety would be difficult to address in this chapter. In this section a brief review of previous studies on the Seismic performance of steel structure and application of the pushover analysis are presented and focused on recent contributions related to pushover analysis of steel frames and past efforts most closely related to the needs of the present work.

2.2. SEMI-RIGID CONNECTIONS

The steel frame performs as an integrated structure deriving its strength from the individual elements, which are properly located to transmit loads throughout the structure. The connections' behavior has an effect on the frame performance, because they are the basic elements and integrated part of a steel frame.

In 1994, the Load and Resistance Factor Design (LRFD) specification of the American Institute of Steel Construction (AISC) categorizes two basic types of steel frame construction as follows:

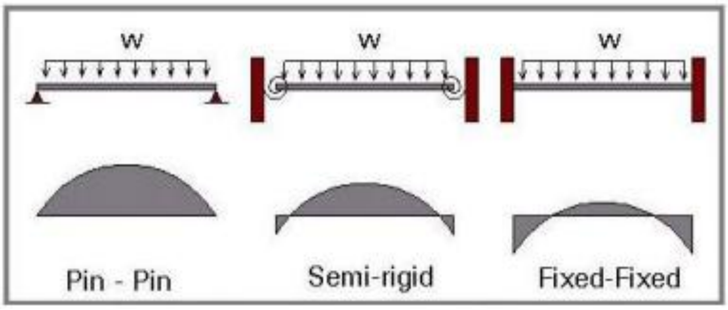


Figure 2.1: Comparison of Semi-Rigid Connections vs. Pinned and Fixed Connections (McGuire, 1995)

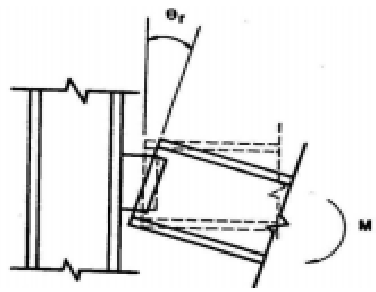


Figure 2.2: Rotation of Connections under Applied Moment

Most of the connections that fasten beam to column using angles, plates, welds, and bolts are deformable and perform a non-linear behavior between fully fixed and perfectly pinned conditions. So it is more reasonable to classify all connections under the classification of semi-rigid, whereas rigid and pinned conditions being special cases.

Exact determination of the relative restraint of beam to column connection is important for both strength and serviceability of structural frames. Overestimating the connection restraint can result in underestimating lateral sway and underestimating the connection restraint can lead to underestimating forces developed in the beams and columns. Both conditions can affect structural stability.

The primary distortion of a steel beam to column connection is due to in plane bending moment, which yields in a rotational deformation. A connection under an applied moment rotates through angle θ_r, which is the angle between beam and column from their original position (Figure 2.2). This moment also causes a destabilization effect on frame stability since additional drift will occur as a result of the decrease in effective stiffness of the members which the connections are attached to. Increased frame drift triggers the P-Δeffect and the overall stability of the frame will be affected [1].

2.2.1. SINGLE WEB-ANGLE CONNECTIONS AND SINGLE PLATE CONNECTIONS

Single web-angle connection consists of an angle either bolted or welded to both the column and the beam web as shown in Figure 2.3. Different from this, single plate connections use the plate instead of angle (Figure 2.4). This connection type requires less material than a single-web angle connection. In the design of this kind of connection, generally, the single web-angle connections have moment rigidity equal to about one-half of the web double web-angle connections [1].

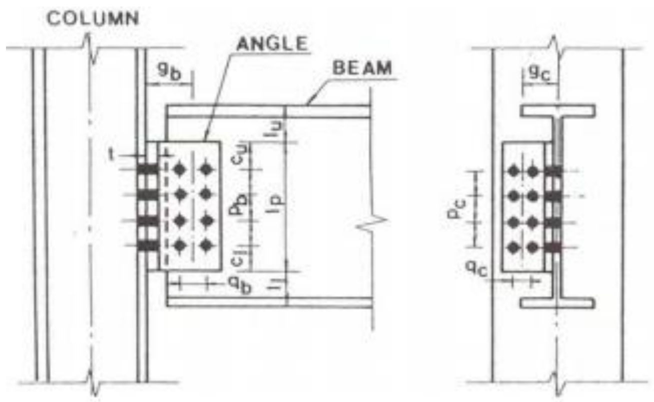


Figure 2.3: Single Web-Angle Connection

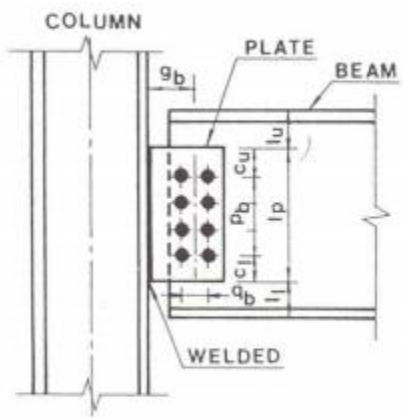


Figure 2.4: Single Plate Connection

2.2.2. DOUBLE WEB-ANGLE CONNECTIONS

Double web-angle connections consist of two angles either bolted or riveted to both the column and the beam web as shown in Figure 2.6. Today, the double web-angle connections with high-strength bolts are used popularly. The connection rigidity of this type is stiffer than that of the single web-angle and single plate connections.

2.2.3. TOP AND SEAT ANGLE CONNECTIONS WITH DOUBLE WEB ANGLE

A typical top and seat angle connection with double web angle is shown in Figure 2.5. Top and seat angle connection with or without double web angle are semi-rigid connections. Double web angle is used to improve the connection restraint characteristics of top and seat angle connections. In order to evaluate inherent ductility offered by the flexural deformation capacities of both the flange and the web angles in the legs attached to the column under static and earthquake loading [1].

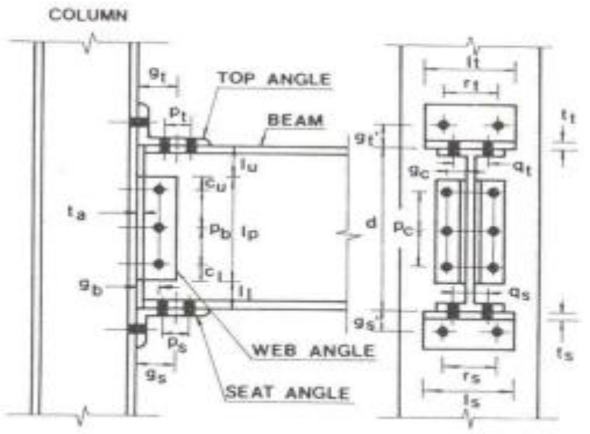


Figure 2.5: Top and Seat Angle Connections with Double Web Angle

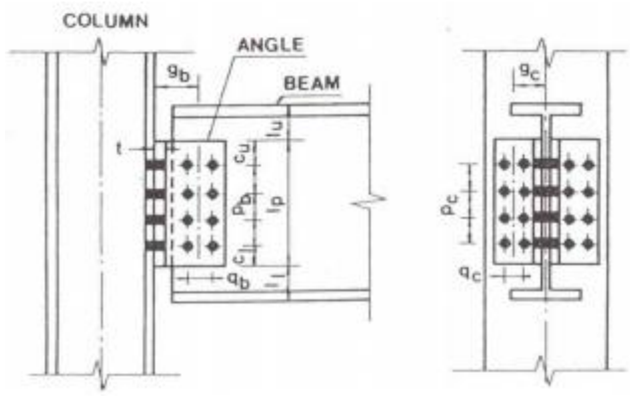


Figure 2.6: Double Web-Angle Connection

2.2.4. TOP AND SEAT ANGLE CONNECTIONS

A typical top and seat angle connection is shown in Figure 2.7. In this type, the seat angle transfers only vertical reaction and should not give significant restraining moment on the end of the beam. On the other hand, the top angle is merely for lateral stability and is not considered to carry any gravity loads. However, according to experimental results, these connections will resist some end moment of the beam [1].

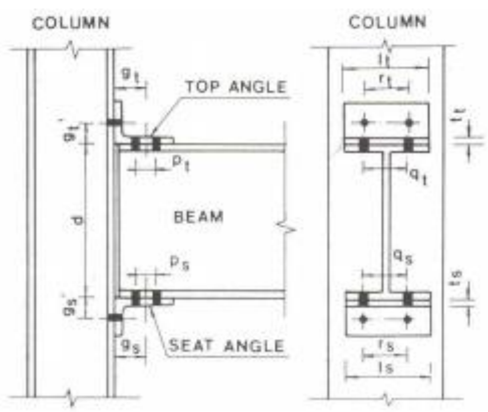


Figure 2.7: Top and Seat Angle Connections

2.2.5. EXTENDED END – PLATE CONNECTIONS AND FLUSH END-PLATE CONNECTION

In general, the end-plate connections are welded to the beam end along both the flanges and the web in the fabricator's shop and bolted to the column in the field.

The end-plate connection has been used extensively since the late 1960s. The extended end-plate connections are classified into two types – either on the tension side only or on both the tension and the compression sides, as shown in Figure 2.8. A typical flush end-plate connection is shown in Figure 2-9. Since both extended and flush end-plate connections are considered as type FR rather than type PR connections, they have often been used as means of transferring beam end moment to the column).

The extended end-plate connection on the tension side only is commonly used. The extended end-plate connection on both sides is preferred when the frame structure is subjected to moment reversal, as during severe earthquake loading. While the flush end-plate connection is weaker than the extended end-plate connection, this connection type is often used in roof details. The behavior of the end-plate connections depends on whether the column flange near the connection is stiffened or not.

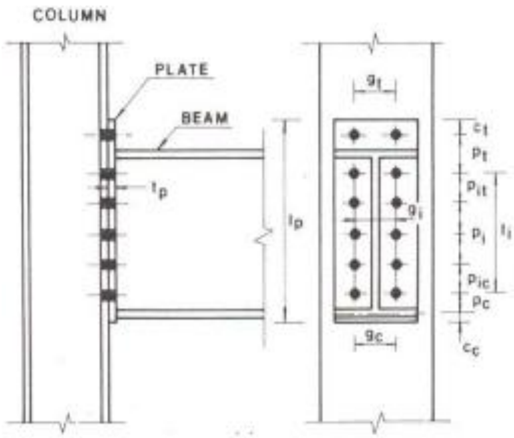


Figure 2.8: Extended End-Plate Connections (Tension Side Only)

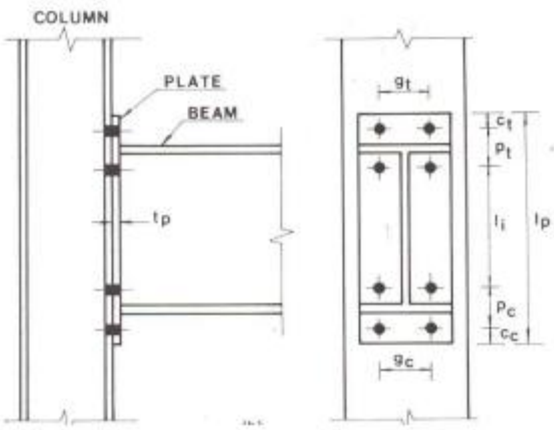


Figure 2.9: Extended End-Plate Connections (Tension and Compression Sides)

2.3. PUSHOVER ANALYSIS

The Non – Linear Static Procedure or Pushover Analysis is defined as a non – linear static approximation of the response a structure will undergo when subjected to dynamic earthquake loading. The static approximation consists of applying a vertical distribution of lateral loads to a model which captures the material non–linearity of an existing or previously designed structure, and monotonically increasing those loads until the peak response of the structure is obtained on a base shear vs. roof displacement plot as shown in Figure 2.10.

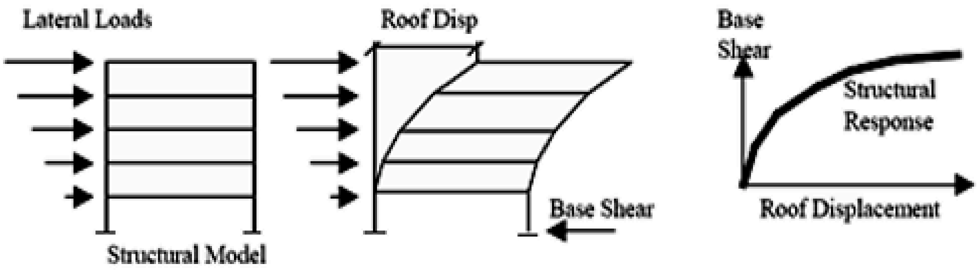


Figure 2.10: Static Approximation Used In the Pushover Analysis

The desired condition of the structure after a range of ground shakings, or Building Performance Level, is then decided upon by the owner, architect, and structural engineer. The Building Performance Level is a function of the post event conditions of the structural and non – structural components of the structure. Some common Building Performance Levels are shown in Figure 2.11 [3].

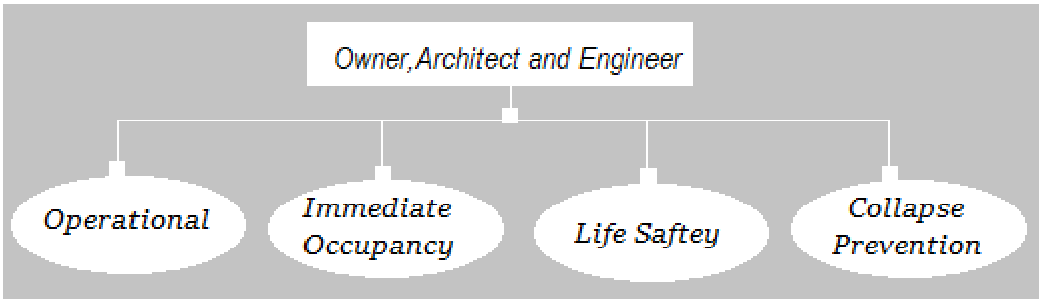


Figure 2.11: Building Performance Level

Based on the desired Building Performance Level, the Response Spectrum for the design earthquake may be determined. The Response Spectrum gives the maximum acceleration, or Spectral Response Acceleration, a structure is likely to experience under the design ground shaking given the structure’s fundamental period of vibration. This relation is shown qualitatively in Figure 2.12. [3].

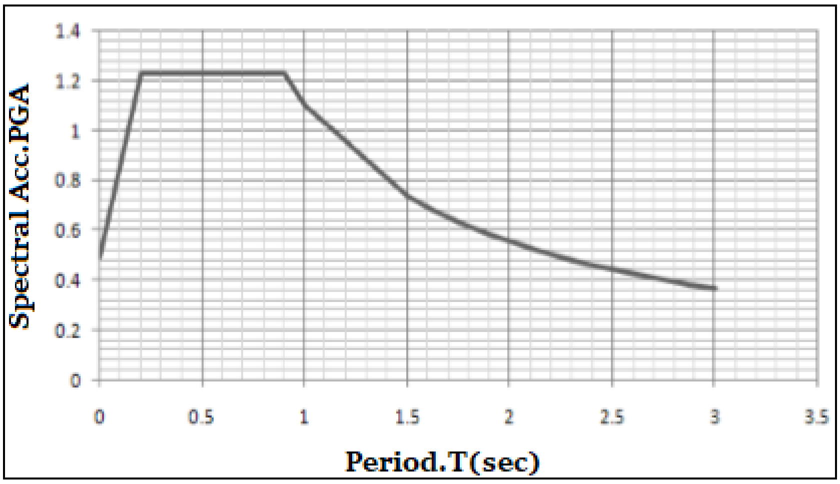


Figure 2.12: Response Spectrum

From the Response Spectrum and Base Shear vs. Roof Displacement plot, the Target Displacement, δt, may be determined. The Target Displacement represents the maximum displacement the structure will undergo during the design event. One can then find the maximum expected deformations within each element of the structure at the Target Displacement and redesign them accordingly. The Target Displacement is shown qualitatively in Figure 2.13 [3].

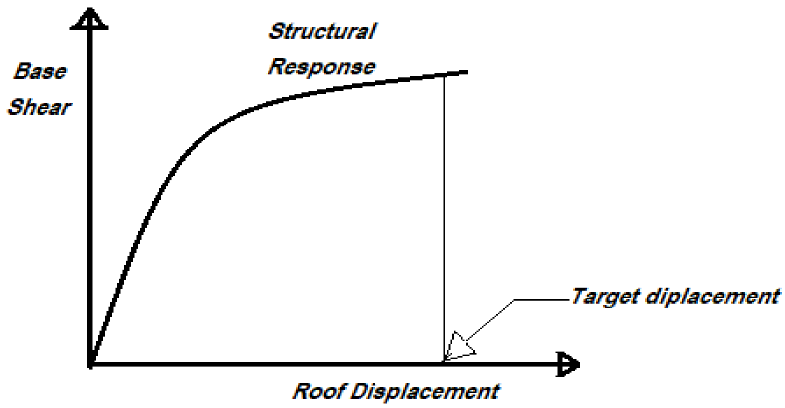


Figure 2.13: Target Displacement

The analysis involves applying horizontal loads, in a prescribed manner, to a model of the structure incrementally; i.e. pushing the structure; and plotting the total applied shear force and associated lateral displacement at each increment, until the structure reaches a limit state or collapse condition. In this method, forcing functions, expressed either in terms of horizontal forces or displacements, are applied to the lateral action - resisting system. Static forces or displacements are distributed along the height of the structure so as to simulate the inertia forces or their effects. The forcing functions are increased in intensity and the pushover analysis (PA) terminates when the ultimate capacity corresponding to a set of ultimate limit states reached. These forcing functions correspond to one or more mode shapes. If the pattern of forcing function (loads or displacements) is kept constant throughout the analysis, the method is referred to as conventional pushover. If the pattern changes to account for variations in the mode shapes of the structure in the inelastic range, the method is referred to as adaptive pushover [3].

Conventional pushover is an inelastic static analysis method in which the idealized representation of the structure is subjected to constant gravity loads and to monotonically increasing lateral force or displacement pattern (also termed 'forcing function') of a constant shape. Because the structural model accounts directly for effects of both material inelasticity and geometric non - linearity, the Pushover analysis is a capacity estimation method under a set of functions that represent inertial effects from the earthquake. This method is capable of shedding light on design weaknesses that elastic analysis cannot detect. For example in the equivalent static analysis or in the simplified code method, weaknesses such as storey mechanisms cannot be readily detected. However, such satisfactory predictions of seismic demands are mostly restricted to low and medium-rise structures in which inelastic action is distributed throughout the height of the structure [4].

None of the invariant force distributions can account for the contributions of higher modes to response, or for a redistribution of inertia forces due to structural yielding and the associated changes in the vibration properties of the structure. To overcome these limitations, several researchers have proposed adaptive force distributions that attempt to follow more closely the time-variant distributions of inertia forces [5].A modal pushover analysis (MPA) procedure that includes the contributions of several "modes" of vibration, thus providing superior accuracy in estimating inelastic seismic demands on buildings, while retaining the conceptual simplicity and computational attractiveness of the procedure with invariant force distribution, has recently been developed [6]. It seems reasonable because the lateral force distribution for each mode appears to be the most rational choice among all invariant distributions of forces and it provides results for elastic buildings that are identical to the well-known response spectrum analysis (RSA) procedure. Since no single load pattern can capture the variability in the local demands expected in a design earthquake, the use of at least two load patterns that are expected to bound inertia force distributions is recommended [7]. Most common load distributions are, a uniform distribution, a linear load distribution and that recommended by the FEMA 356 guidelines in which the normalized storey load is a function of the floor height, h, and the fundamental period of the structure. The load pattern suggested by FEMA 356 applies increased lateral forces to the upper levels of the building. This distribution is intended to capture the higher mode effects in the seismic response. The results of the pushover analyses using the baseline model suggest that the predicted failure mechanism is relatively insensitive to the load distribution. A similar failure mechanism was observed for different load distributions (uniform, triangular and FEMA 356). [8]

➢ Ghobarah A. et al., (1997) the control of inter storey drift can also be considered as a means to provide uniform ductility over the stories of the building. A storey drift may result in the occurrence of a weak storey that may cause catastrophic building collapse in a seismic event. Uniform storey ductility over all stories for a building is usually desired in seismic design.

R. Hasan and L. Xu, D.E. Grierson (2002) conducted a simple computer-based pushover analysis technique for performance-based design of building frameworks subject to earthquake loading. And found that rigidity-factor for elastic analysis of semi-rigid frames, and the stiffness properties for semi-rigid analysis are directly adopted for pushover analysis.

➢ X.-K. Zou et al., (2005) presented an effective technique that incorporates Pushover Analysis together with numerical optimization procedures to automate the Pushover drift performance design of reinforced concrete buildings. PBD using nonlinear pushover analysis, which generally involves tedious computational effort, is highly iterative process needed to meet code requirements.

➢ Oğuz, Sermin (2005) Ascertained the effects and the accuracy of invariant lateral load patterns Utilized in pushover analysis to predict the behaviour imposed on the structure due to randomly Selected individual ground motions causing elastic deformation by studying various levels of Nonlinear response. For this purpose, pushover analyses using various invariant lateral load Patterns and Modal Pushover Analysis were performed on reinforced concrete and steel moment resisting frames covering a broad range of fundamental periods. The accuracy of approximate Procedures utilized to estimate target displacement was also studied on frame structures. Pushover analyses were performed by both DRAIN-2DX and SAP2000. The primary observations from the study showed that the accuracy of the pushover results depended strongly on the load path, the characteristics of the ground motion and the properties of the structure.

➢ Mehmet et al., (2006), explained that due to its simplicity of Push over analysis, the structural engineering profession has been using the nonlinear static procedure or pushover analysis. Pushover analysis is carried out for different nonlinear hinge properties available in some programs based on the FEMA-356 and ATC-40 guidelines and he pointed out that Plastic hinge length (Lp) has considerable effects on the displacement capacity of the frames. The orientation and the axial load level of the columns cannot be taken into account properly by the default-hinge properties (Programme Default).

➢ Shuraim et al., (2007) summarized the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 has been utilized for the evaluation of existing design of a building, in order to examine its applicability. He conducted nonlinear pushover analysis shows that the frame is capable of withstanding the pre-assumed seismic force with some significant yielding at all beams and columns.

➢ Girgin. et., (2007),Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is computationally and conceptually simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure.

➢ Karavasilis et. al. (2008) presented a parametric study of the inelastic seismic response of plane steel moment resisting frames with steps and setbacks. A family of 120 such frames, designed according to the European seismic and structural codes, was subjected to 30 earthquake ground motions, scaled to different intensities. The main findings of this paper are as follows. Inelastic deformation and geometrical configuration play an important role on the height-wise distribution of deformation demands. In general, the maximum deformation demands are concentrated in the tower-base junction in the case of setback frame and in all the step locations in the case of stepped frames. This concentration of forces at the locations of height discontinuity, however, is not observed in the elastic range of the seismic response.

➢ A.Kadid and A. Boumrkik (2008), proposed use of Pushover Analysis as a viable method to assess damage vulnerability of a building designed according to Algerian code. Pushover analysis was a Series of incremental static analysis carried out to develop a capacity curve for the building. Based on capacity curve, a target displacement which was an estimate of the displacement that the design earthquake would produce on the building was determined. The extent of damage Experienced by the structure at this target displacement is considered representative of the Damage experienced by the building when subjected to design level ground shaking. Since the Behaviour of reinforced concrete structures might be highly inelastic under seismic loads, the global inelastic performance of RC structures would be dominated by plastic yielding effects and consequently the accuracy of the pushover analysis would be influenced by the ability of the Analytical models to capture these effects.

➢ P.Poluraju and P.V.S.N.Rao (2011), has studied the behaviour of framed building by conducting Push over Analysis, most of buildings collapsed were found deficient to meet out the requirements of the present day codes. Then G+3 building was modelled and analyzed, results obtained from the study shows that properly designed frame will perform well under seismic loads.

➢ Narender Bodige, Pradeep Kumar Ramancharla (2012), modelled a 1 x 1 bay 2D four storied building using AEM (applied element method). AEM is a discrete method in which the elements are connected by pair of normal and shear springs which are distributed around the elements edges and each pair of springs totally represents stresses and deformation and plastic hinges location are formed automatically. Gravity loads and laterals loads as per IS 1893-2002 were applied on the structure and designed using IS 456 and IS 13920. Displacement control pushover analysis was carried out in both cases and the pushover curves were compared. As an observation it was found that AEM gave good representation capacity curve. From the case studies it was found that capacity of the building significantly increased when ductile detailing was adopted. Also, it was found that effect on concrete grade and steel were not highly significant.

2.4. SMART PARKING SYSTEMS

Now-a-day there is an increase in the number of vehicles on road. It is very easy to park vehicle on road or when vehicle are moving. But, when the vehicle stops, we cannot leave vehicle on road or unsafe place. For proper storage of vehicle when not in use "parking space" is needed. For any person or cargo moving in a vehicle, a terminal facility is essential both at the origin and the destination. When the person has to stop route for some purpose other than traffic related, the vehicle needs some halting facility, without disturbing traffic flow otherwise on the street. Such a facility is called parking [14].

It is also noteworthy that a personal vehicle is on the move hardly for 2 to 3 hours in a day, while for the remaining period it is parked at the residence or destination and sometimes in route. Even commercial vehicles will be found to be parked for about 60% of time on an average. With the increased ownership and usage of private vehicles in the form of automobiles and motorized two-wheelers, parking has become an essential fact of this age, particularly in urban areas Thus, due to increase in population there is increase in vehicle demand and we require more parking space thus in control by constructing multi parking building or road street[14].

Parking building is used for safe guarding the user vehicle. Successful parking structures meet the user demands. It includes feeling safe and also knowing that their cars are in a secure environment. In provision of parking area is unplanned so there is a need to make an improvement in the parking area. Every vehicle owner would like to park his vehicle as close as possible to his destination, so as to minimized walking distance. The acceptable walking distance is 50 to 150 m and also depends on pedestrian side walk facility available. Parking structures have become important elements in today's urban and suburban environments. Owners have realized that parking services represent the first and last impression a visitor receives of the facility and that can be a pivotal factor when consumers decide where to do business. Even more, owners and designers both are acknowledging that parking structures must be designed specifically for the types of visitors that structure will serve, based on the facilities they support and the flow of daily traffic. The need to create a parking structure that precisely fits the needs of the users cannot be stressed enough. Unless the facility is user-friendly, projecting a safe, secure, and easy to use environment, parkers will find other options. These needs have become too vital to their peace of mind to be left unmet by the owner and designer. As a result, creating the best parking structure for the site, users, and budget requires a careful balance of all elements and a logical plan from start to finish. [14].

Multi-storey car parks have a number of unique features that distinguish them from other buildings or structures. A lack of understanding and recognition of these distinct characteristics by designers and those responsible for inspection and maintenance is believed to be the major cause of many of the common problems identified in these structures. Parking structures are generally classified as either "static" or "automated." The automated parking are more common in Europe while static is the most prevalent type of parking structure in the United States. The two types of ramps that can be used are straight ramp and curve ramp. Five types of layout that can be used in traditional parking structure include parallel packing, perpendicular/ angle 90^o, angle 60^o, angle 45^o. The floor level system can be flat on the same floor; can be split level or staggered floor systems or sloping floor systems. For the design aspect, there are numerous configurations of multi-storey car parks featuring different arrangements of deck and ramp [15].

The smart parking systems were initially implemented in Europe, USA and Japan but later on as the other countries started developing; these smart parking systems are being installed in these countries as well. With further advancement in the smart parking systems the problem of finding vacant spaces and all the hassle is going to deprive [15].

CHAPTER III

3. PUSHOVER ANALYSIS AND BEHAVIOR FACTOR

3.1. PUSHOVER ANALYSIS- AN OVERVIEW

The use of the nonlinear static analysis (pushover analysis) came in to practice in 1970's but the potential of the pushover analysis has been recognized for last two decades years. This procedure is mainly used to estimate the strength and drift capacity of existing structure and the seismic demand for this structure subjected to selected earthquake. This procedure can be used for checking the adequacy of new structural design as well. The effectiveness of pushover analysis and its computational simplicity brought this procedure in to several seismic guidelines (ATC 40 and FEMA 356) and design codes (Euro code 8) in last few years.

Pushover analysis is defined as an analysis where in a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. Target displacement is the maximum displacement (elastic plus inelastic) of the building at roof expected under selected earthquake ground motion. The structural Pushover analysis assesses performance by estimating the force and deformation capacity and seismic demand using a nonlinear static analysis algorithm. The seismic demand parameters are storey drifts, global displacement (at roof or any other reference point), storey forces, and component deformation and component forces. The analysis accounts for material inelasticity, geometrical nonlinearity and the redistribution of internal forces. Response characteristics that can be obtained from the pushover analysis are summarized as follows:

- ✓ Estimates of force and displacement capacities of the structure. Sequence of the member yielding and the progress of the overall capacity curve.
- ✓ Estimates of force (axial, shear and moment) demands on potentially brittle elements and deformation demands on ductile elements.
- ✓ Estimates of global displacement demand, corresponding inter-storey drifts and damages on structural and non-structural elements expected under the earthquake ground motion considered.
- ✓ Sequences of the failure of elements and the consequent effect on the overall structural stability.
- ✓ Identification of the critical regions when the inelastic deformations are expected to be high and identification of strength irregularities (in plan or in elevation) of the building. Pushover analysis delivers all these benefits for an additional computational effort (modeling nonlinearity

and change in analysis algorithm) over the linear static analysis. Step by step procedure of pushover analysis is discussed next.

3.2. PUSHOVER ANALYSIS PROCEDURE

Pushover analysis can be performed as either Force-controlled or Displacement-controlled depending on the physical nature of the load and the behavior expected from the structure. Force controlled option is useful when the load is known (such as gravity loading) and the structure is expected to be able to support the load. Displacement controlled procedure should be used when specified drifts are sought (such as in seismic loading), where the magnitude of the applied load is not known in advance, or where the structure can be expected to lose strength or become unstable.

Some computer programs (e.g. Seismostruct, DRAIN-2DX, nonlinear version of SAP2000, ANSYS) can model nonlinear behavior and perform pushover analysis directly to obtain capacity curve for two and/or three dimensional models of the structure. When such programs are not available or the available computer programs could not perform pushover analysis directly (e.g. ETABS, SAP90), a series of sequential elastic analyses are performed and superimposed to determine a force displacement curve of the overall structure. A displacement-controlled pushover analysis is basically composed of the following steps:

i. A two or three dimensional model that represents the overall structural behavior is created.
ii. Bilinear or tri-linear load-deformation diagrams of all important members that affect lateral response are defined.
iii. Gravity loads composed of dead loads and a specified portion of live loads are applied to the structural model initially.
iv. A pre-defined lateral load pattern which is distributed along the building height is then applied.
v. Lateral loads are increased until some member(s) yield under the combined effects of gravity and lateral loads.
vi. Base shear and roof displacement are recorded at first yielding.
vii. The structural model is modified to account for the reduced stiffness of yielded member(s).
viii. Gravity loads are removed and a new lateral load increment is applied to the modified structural model such that additional member(s) yield. Note that a separate analysis with zero initial conditions is performed on modified structural model under each incremental lateral load. Thus, member forces at the end of an incremental lateral load analysis are obtained by adding the forces from the current analysis to the sum of those from the previous increments. In other words, the results of each incremental lateral load analysis are superimposed

ix. Similarly, the lateral load increment and the roof displacement increment are added to the corresponding previous total values to obtain the accumulated values of the base shear and the roof displacement.

x. Steps vii, viii and ix are repeated until the roof displacement reaches a certain level of deformation or the structure becomes unstable.

xi. The roof displacement is plotted with the base shear to get the global capacity (pushover) curve of the structure (Figure 3.1).

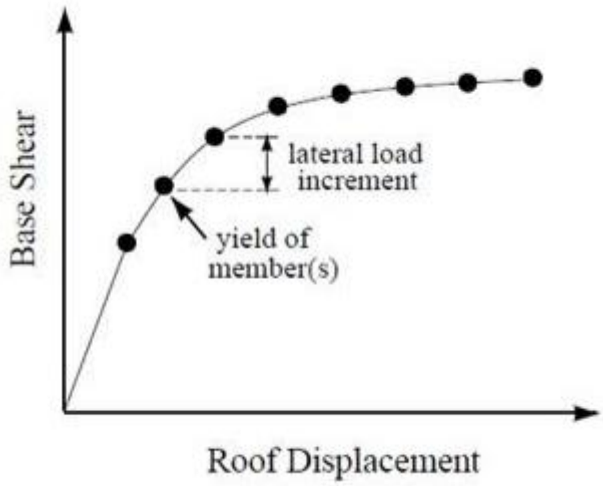


Figure 3.1: Global Capacity (Pushover) Curve of Structure

3.3. LATERAL LOAD PROFILE

The analysis results are sensitive to the selection of the control node and selection of lateral load pattern. In general case, the centre of mass location at the roof of the building is considered as control node. In pushover analysis selecting lateral load pattern, a set of guidelines as per FEMA 356 is explained in Section 2.5.2. The lateral load generally applied in both positive and negative directions in combination with gravity load (dead load and a portion of live load) to study the actual behavior. Different types of lateral load used in past decades are as follows.

- ***Uniform Lateral Load Pattern***

The lateral fore at any storey is proportional to the mass at that storey.

$$F_i = {}^{M_i}/_{\sum M_i},$$ where; F_i, Force at i^{th} storey

M_i, Mass at i^{th} storey

- ***Elastic First Mode Lateral Load Pattern***

The lateral force at any storey is proportional to the product of the amplitude of the elastic first mode and the mass at that storey, i.e

$$F_i = {M_i \Phi_i}/{\sum M_i \Phi_i} ,$$

where;

F_i, Force at i^{th} storey
M_i, Mass at i^{th} storey
Φ_i, amplitude of the elastic first mode at *i-th* storey

(Note: The storey forces are normalized with the base shear to have a total base shear equals to unity).

3.4. USE OF PUSHOVER RESULTS

Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is computationally and conceptually. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure.

The expectation from pushover analysis is to estimate critical response parameters imposed on structural system and its components as close as possible to those predicted by nonlinear dynamic analysis.

Pushover analysis provides the following information on many response characteristics that can't be obtained from an elastic static or elastic dynamic analysis.

- Inter storey drifts are estimates and its distribution along the height.
- Determination of force demands on brittle members, are axial force demands on columns, beam-column connections are moment demands.
- Deformation demands of determination for ductile members.
- In location of weak points identification in the structure (or potential failure modes).
- Effort of an action strength deterioration of individual members on the behavior of structural system.
- In plan or elevation identification of strength discontinuities that will lead to changes in dynamic characteristics in the inelastic range.
- Verification of the completeness and adequacy of load path.
- Pushover analysis also exposes design weaknesses that may remain hidden in an elastic analysis. They are storey mechanisms, excessive deformation demands, irregularities strength and overloads on potentially brittle members.

3.5. LIMITATION OF PUSH OVER ANALYSIS

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, pushover predictions are accuracy and limitations of current pushover procedures must be identified. Selection of lateral load patterns and identification of failure mechanisms for estimate of target displacement due to higher modes of vibration are important issues that affect the accuracy of pushover results. The mass center of roof displacement structure is used as target displacement. The estimation of target displacement accurate associated with specific performance objective affect the accuracy of seismic demand predictions of pushover analysis. Target displacement is the global displacement expected in a design earthquake. The estimate of target displacement, identification of failure mechanisms due to higher modes of vibration are important issues that affect, selection of lateral load patterns the accuracy of pushover results.

3.6. BEHAVIOR FACTOR (R)

The behavior factor (R) is the ratio of the strength required to maintain the structure elastic to the inelastic design strength of the structure [9]. In other words, it is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra. It is found through Push over analysis. The behavior factor, R, accounts for the *inherent ductility, over strength of a structure and difference in the level of stresses considered in its design.* FEMA (1997), UBC (1997) suggests the R factor in force-based seismic design procedures. It is generally expressed in the following form taking into account the following three components;

$$R = R_\mu . R_s . Y; \qquad (3.1)$$

$$R_\mu = \frac{V_e}{V_y} \quad , \quad R_s = \frac{V_y}{V_s} \quad , \quad Y = \frac{V_s}{V_w} \qquad (3.2)$$

Hence, $R = \frac{V_e}{V_y} * \frac{V_y}{V_s} * \frac{V_s}{V_w} = \frac{V_e}{V_w}$ (3.3)

Where; R_μ is the ductility dependent component also known as the ductility reduction factor.

R_s is the over-strength factor ,and

Y is termed the allowable stress factor.

V_e,V_y, V_s and V_w correspond to the structure's elastic response strength, the idealized yield strength, the first significant yield strength and the allowable stress design strength, respectively as shown in the Figure 3.2.

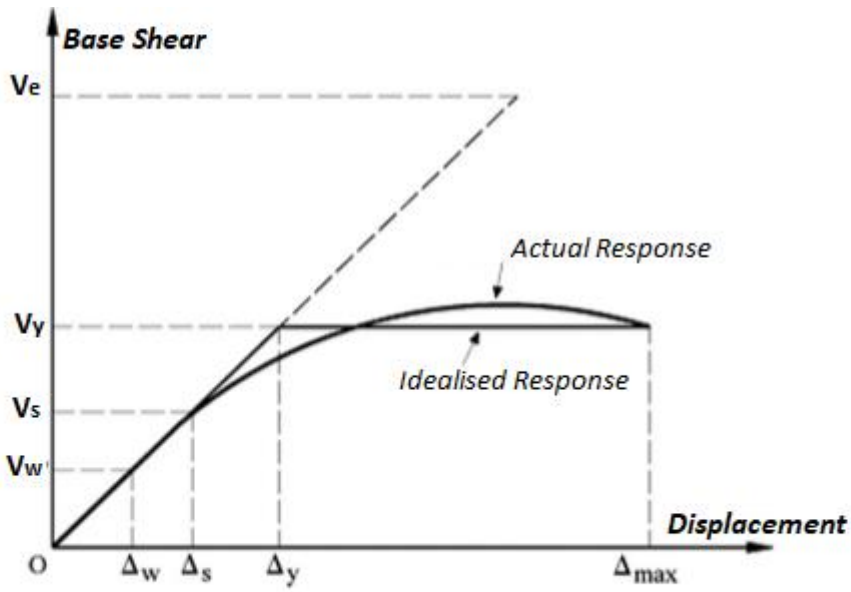


Figure 3.2: Typical Pushover response curve for evaluation of behavior factor, R

The structure ductility, μ, is defined in as maximum structural drift ($\triangle_{max}$) and the displacement corresponding to the idealized yield strength ($\triangle_{y}$) as:

$$\mu = \frac{\triangle_{max}}{\triangle_{y}} \qquad (3.4)$$

CHAPTER IV

4. METHOD OF ANALYSIS AND VERIFICATION OF THE STRUCTURAL ANALYSIS SOFTWARE FOR STEEL FRAMES

4.1. METHODOLOGY

4.1.1. TARGET DISPLACEMENT

According to FEMA356, the target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The appropriate estimation of target displacement point is a very important task in the seismic performance assessment of structures. Equation 4.1 represents a basic relation that is used to calculate the target displacement, δt, at each storey level (FEMA440, 2005),

$$\delta_t = C_o\, C_1\, C_2\, C_3 S_a\, \frac{T_e^2}{4\pi^2}\, g \tag{4.1}$$

Where; C_o is modification factor used to relate spectral displacement of an equivalent SDOF system to the roof displacement of the structure MDOF system obtained applying one of the following methods,

- ✓ The easiest way is using *Table 3-2* of FEMA 356 to determine the appropriate value of the modification factor.
- ✓ The first modal participation factor obtained at the level of the control node.
  A shape vector consistent to the deflected shape of the structure at the target displacement can be used.

C_1 is modification factor to relate expected maximum inelastic displacements to displacements obtained for linear elastic response.

FEMA440 states that the modification factor (C_1) estimates maximum displacements of inelastic SDOF Oscillator and estimated the value using the improved/simplified *Equation 4.2* that can be used for most structures.

$$C_1 = 1 + \frac{R-1}{aT_e^2} \tag{4.2}$$

Where; T_e: Effective fundamental period of the structure,

a: Constant parameter for different site classes. , is equal to *130, 90 and 60* in site class *B, C and D* respectively.

R: Ratio of elastic strength demand;

$$R = \frac{S_a W}{V_y} \tag{4.3}$$

Where; S_a: Response spectrum acceleration, g,

V_y: Yield strength obtained by performing nonlinear static pushover analysis

W: Effective seismic weight of the structure

C_2 is modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. FEMA 440 states that this modification factor has limitation in considering the effects of strength degradation and recommend that the $C2$ coefficient is as *equation 4.4.*

$$C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 \tag{4.4}$$

C_3 is modification factor to represent increased displacements due to dynamic P-Δ effects.

Values of C_3 shall be calculated using Equation 4.6-7.

- For structures with negative post yield stiffness

$$C_3 = 1 + \frac{|\alpha|R - 1}{T_e} \tag{4.6}$$

Where; α: Ratio of post-yield stiffness to effective elastic stiffness

- For structures with positive post yield stiffness

$$C_3 = 1, \tag{4.7}$$

4.1.2. COMPUTING AND DEFINING INELASTIC FRAME ELEMENTS

In this study, for performing nonlinear analysis of the structure through finite element software, SeismoStruct which has capable of predicting the large displacement behavior of space frames under static or dynamic loading, taking into account both geometric nonlinearities and material inelasticity were selected.

Modeling of structure and, defining geometric and material nonlinearities are the main differences among the other available programs; SAP 2000, PERFORM 3D.

4.1.2.1 Defining inelastic frame elements in SeismoStruct Software

SeismoStruct software is capable of considering both global and local sources of geometric and material nonlinearities. The current unknown deformation of the elements have been described by attaching local chord system to each finite element as shown in *Figure 4.1* and it rotates and translates with the element. Geometric nonlinearities are also available in SAP2000 software for nonlinear time-history analysis and P-delta plus large displacements effects are taken into account by the program.

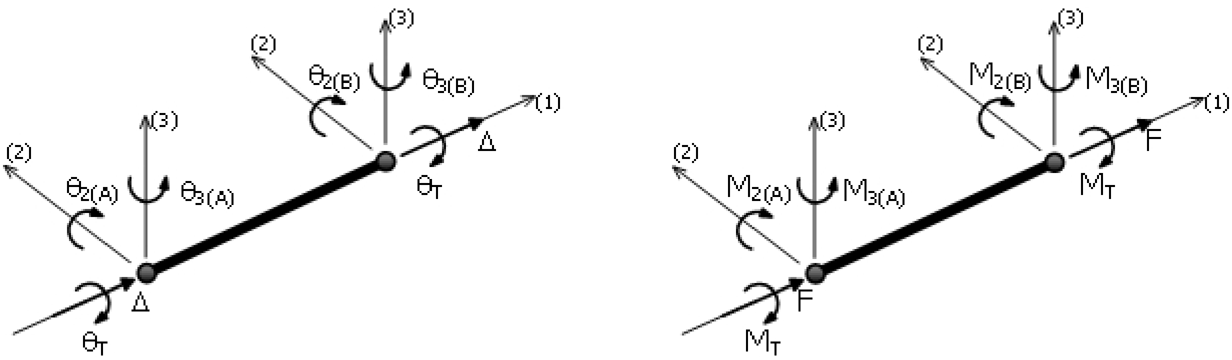


Figure 4.1: Local sources of geometric nonlinearities (SeismoStruct, 2016)

For characterizing of the nonlinear response of the system, defining of material elasticity to elements is very important task and it can be defined in elements through distributed and concentrated plasticity. In SeismoStruct software, material inelasticity of the elements is made of so called fiber modeling approach in which the element has been subdivided into many segments.

The section is discretized in sufficient quantity of fibres and the response of sections are obtained through the integration single fiber’s response of individual fibres (typically 100-150) (SeismoStruct, 2016). But, SAP2000 software does not take into account the material nonlinearity of the elements.

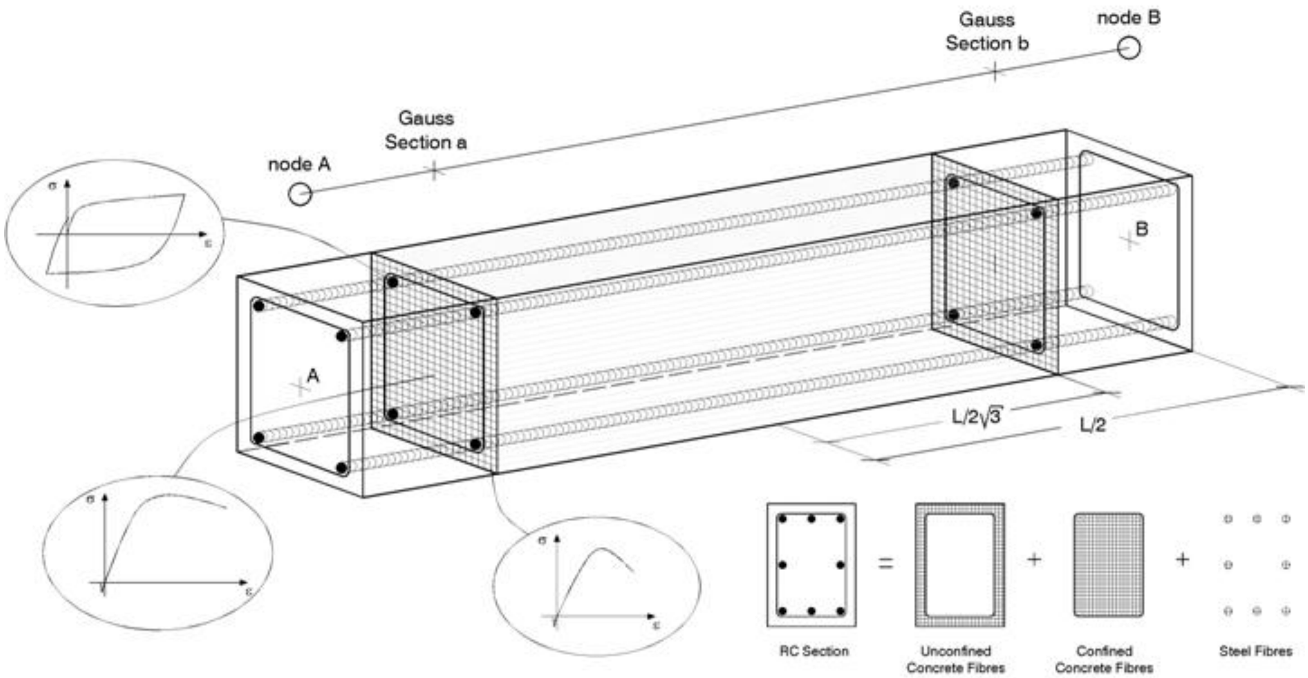


Figure 4.2: Material inelasticity (Seismostruct, 2016)

In SeismoStruct software, many various material types are available while in SAP2000 program, only elastic material such as orthotropic and isotropic can be used.

4.1.2.2 Defining inelastic frame elements in SAP2000

In nonlinear analysis, to consider nonlinear structural behavior of each element, plastic hinges are assigned at the both ends of beam–column elements. FEMA (2000c) has given information about nonlinear plastic hinge properties of all of the structural elements.

SAP2000 implements the plastic hinge properties and computes it automatically from section and material properties based on given criteria in ATC-40 or FEMA-356. In SAP2000, three kinds of hinge properties are available. They are User-Defined hinge properties, Auto Hinge Properties, and Program Generated Hinge properties. Inverse of User-Defined hinge properties, Auto hinge properties cannot be modified and viewed because the default properties are section dependent. In user-defined hinge properties, it is required to define moment curvature data for each element and moment rotation relationship for each section.

In program Generated Hinge Properties, the software combines its built-in criteria with the defined section properties for each object to generate the final hinge properties which means that you do considerably less work defining the hinge properties because you do not need to define every hinge.

CSI SAP2000 program is able to displays the plastic hinges behavior at each step of the change process (SAP2000, 2006).

Inel and Ozmen (2006) state that for RC buildings, the difference between the results of pushover analysis by using Auto hinge and User-Defined properties is very little for new buildings and more for old ones (more appropriate for rehabilitation objectives). Since the aim of this study is assessing the nonlinear behavior of RC buildings that designed according new design code. M3 and P-M2-M3 interaction hinges were used for defining hinges at the beginning and ending points of the beams and columns, respectively. Therefore, Hinge properties are assigned to the frame elements based on FEMA-356 generalized force-deformation relation model as shown in *Figure 4.3.*

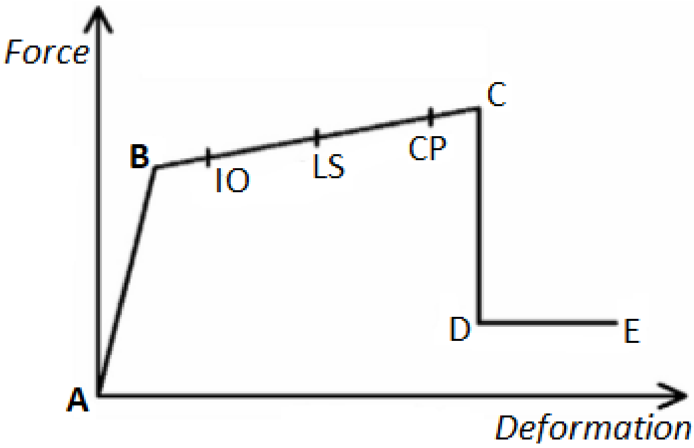


Figure 4-3: The relationship of Force-deformation of a typical plastic hinge (FEMA356, 2000)

4.1.3. STRUCTURAL PERFORMANCE LIMITS STATES

According to FEMA 356, the discrete Structural Performance Levels, that is going to be studied, are Collapse Prevention (CP), Life Safety (LS) and Immediate Occupancy (IO).

i) Immediate occupancy (IO)

In this level, the post-earthquake damage should be in the level that the structure remains safe to occupy and stays harmless to inhabit. The basic seismic and vertical force-resisting systems of the structure retain the pre-earthquake design stiffness and strength of the construction. Therefore, Slight damage to the structure has occurred, that can be easily repaired, is observed.

ii) Life Safety (LS)

In this level, the post-earthquake damage should be in the level that the structure has suffered considerable damage, nevertheless retains a margin opposing onset of partial or total collapse. Some elements and components of structure are severely damaged and there is risk of injury to life. It should be possible to repair the structure and repairing may be less economical when compared to complete reconstruction.

iii) Collapse Prevention (CP)

In this level, the post-earthquake damage should be in the level that the building is on the verge of experiencing partial or total collapse. The structure has suffered Comprehensive damage, as well as encompassing momentous degradation in the strength and stiffness of the seismic load resisting system. The construction is not safe for re-occupancy and could not be technically practical to repair.

iv) Collapse (C)

In this level, the post-earthquake damage should be in the level that the structure is at the total collapse and fails to satisfy any criteria that mentioned above, thus this means that re-occupancy of the structure should not permitted because the structure is at the collapse level.

Drift Levels

Inter-storey drift profiles were used to achieve valuable results data on the failure mechanism and illustrate influence of yielding derived from the inelastic procedures that is directly correlated to non-structural and structural damage (FEMA440, 2005).

In this work, according to (FEMA356, 2000), three limit states including collapse prevention (CP), life safety (LS) and immediate occupancy (IO) will be defined on Appendix A, Table A-1.

4.1.4. IDEALIZING FORCE-DISPLACEMENT CURVE

In nonlinear static pushover analysis, the calculated relationship between displacement of a control node and base shear should be idealized to the bilinear curve in order to calculate effective yield strength,Vy the effective lateral stiffness, Ke, and the yield slope, α, of the structure as illustrated in Figure 4.4. At the origin is starting point of initial linear portion of the idealized force-displacement curve and maximum base shear is the last point of the second linear portion. The effective lateral stiffness, Ke should be considered as the secant stiffness obtained a base shear force equal to 60% of the effective yield strength of the building. The post-yield slope,α, should be identified by a line segment which passes through the substantial curve at the obtained target displacement (FEMA440, 2005).

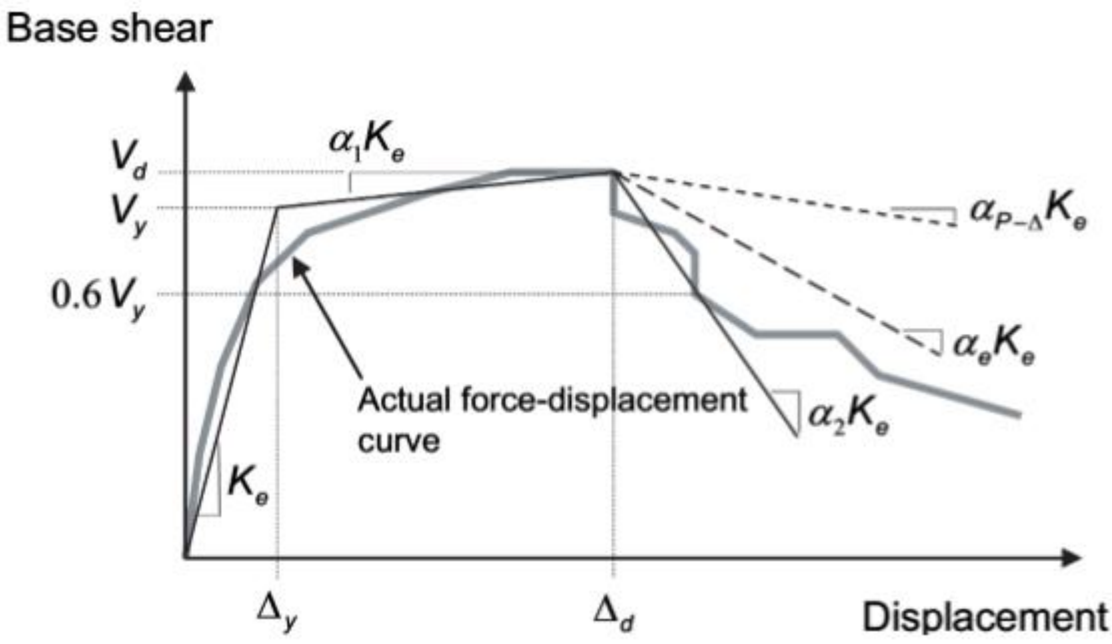


Figure 4.4: Idealized force-displacement curve for Non static analysis (FEMA440, 2005)

4.2. VALIDATION OF THE STRUCTURAL ANALYSIS SOFTWARE FOR STEEL FRAMES

The Finite Element package (SeismoStruct) used in the present work for nonlinear analysis is verified with experiment results of response of Steel structures under reproduction of horizontal incremental loads carried out by the *P.Magudeaswaran et al.,(2016) International Journal of Advanced Engineering Technology*.

The experimental data on a full-scale steel structural model subjected to incremental loads up to serious strength deterioration will be compared with analytical estimate, and thus the program evaluated to simulate the behaviour of steel frames under the horizontal loads.

To address this, two verification examples (Bare frame and Braced frame) were presented in this sub topic to verify the analysis results of Seismostruct software. In the analysis of steel material, the nonlinearity material behavior is approximately accounted using the implementation of fiber section. There are two important factors in the modeling of discrete member using fiber section model that are a member discretization along the element length and a section discretization into many fibers. The member discretization should be done as the requirement of stiffness based formulation in a nonlinear analysis problem. The fiber discretization should be done as well due to the constant stress assumption of each fiber.

4.2.1. BARE FRAME VALIDATION

Validation Data

The geometric detail of bare frame is 920mm * 50mm * 1050mm and its section detail of frame is 100mm * 50mm * 4mm.The materials used for casting of deck slab are Coarse aggregate (25mm), fine aggregate (sand), 53 grade cement, binder, PVC pipe, U-Hook, Wood. The size of form work is 1240mm X 940mm. The deck slab is casted for C20 mix, which has the concrete mix of 1:1.5:3 and Water/Cement ratio of 0.5. Deck slab is casted along with frames for experimental investigations.

The bare frame is provided with a fixed support at its bottom and horizontal load is applied at top left corner of the frame and the deflection is computed for each load step. According to experiment [10], the bare frame is subjected to a horizontal load of 50kN, so that the frame gets deflected and the maximum deformation occurs at the top of the section.

The overall dimension and structural configuration can be seen in *Figure 4.5.*

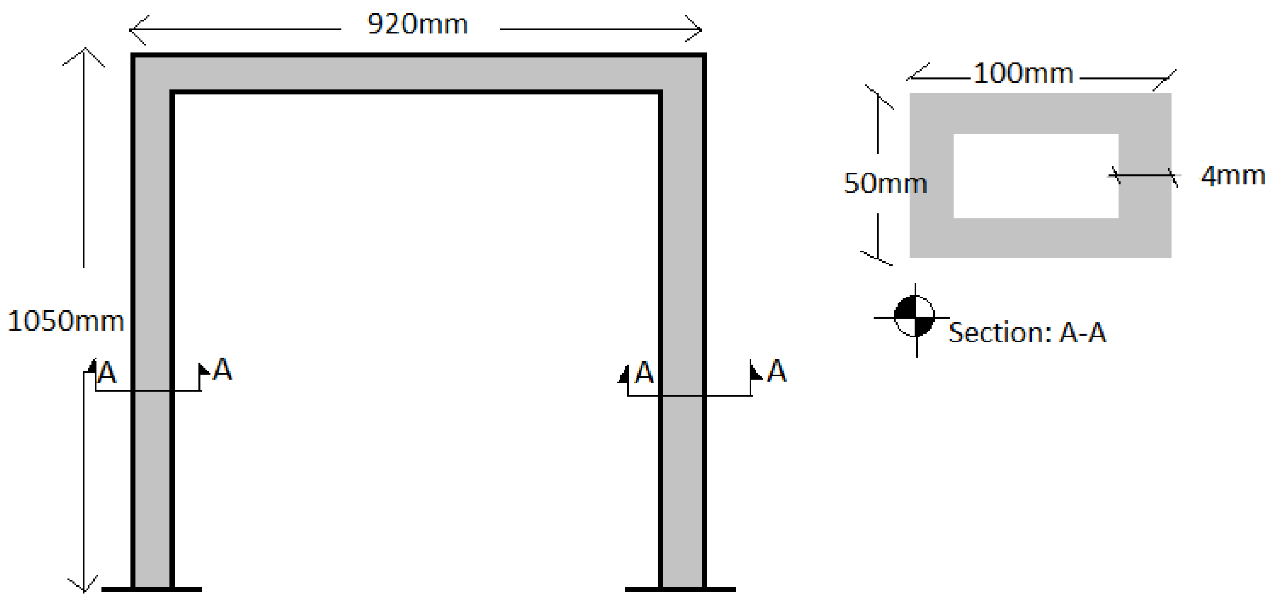


Figure 4.5: Validation of bare frames[10]

Validation bare frame model

The *figure 4.6* shows Seismostruct model of the bare frame used to verify the analytical result of the program with the experimental output.

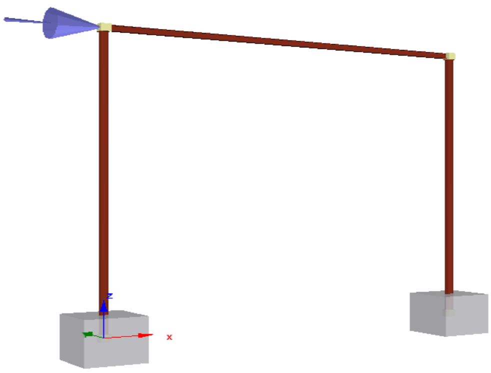


Figure 4.6: Validation Bare frame Model (Seismostruct model)

Seismostruct 2016

As depicted on Figure 4-8 there is some variation between capacity curves from Seismostruct 2016 with the experiment output which have acceptably similar result. The pushover curve of the program is similar with the experiment value till the yielding zone of the material and shows that Program capacity curve could be below the experiment one starting from yield point up to the ultimate capacity of the frame. But the material models should capture the real behavior of materials as much as possible through the performance assessment as could be seen from the Seismostruct results. Figure 4.7 shows the capacity curves for load patterns and deformed shapes at first rupture of steel with the corresponding roof displacement.

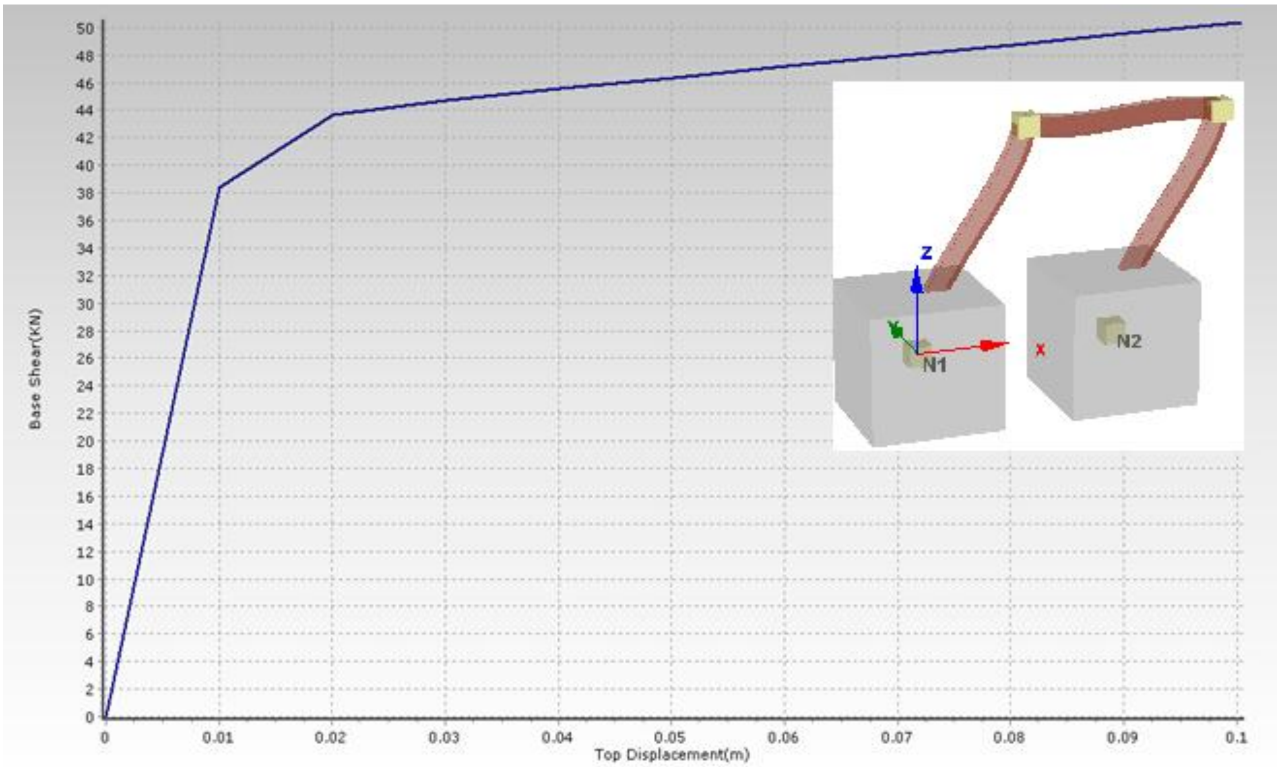


Figure 4.7: Seismostruct Capacity Curve and frame model at Steel Rupture

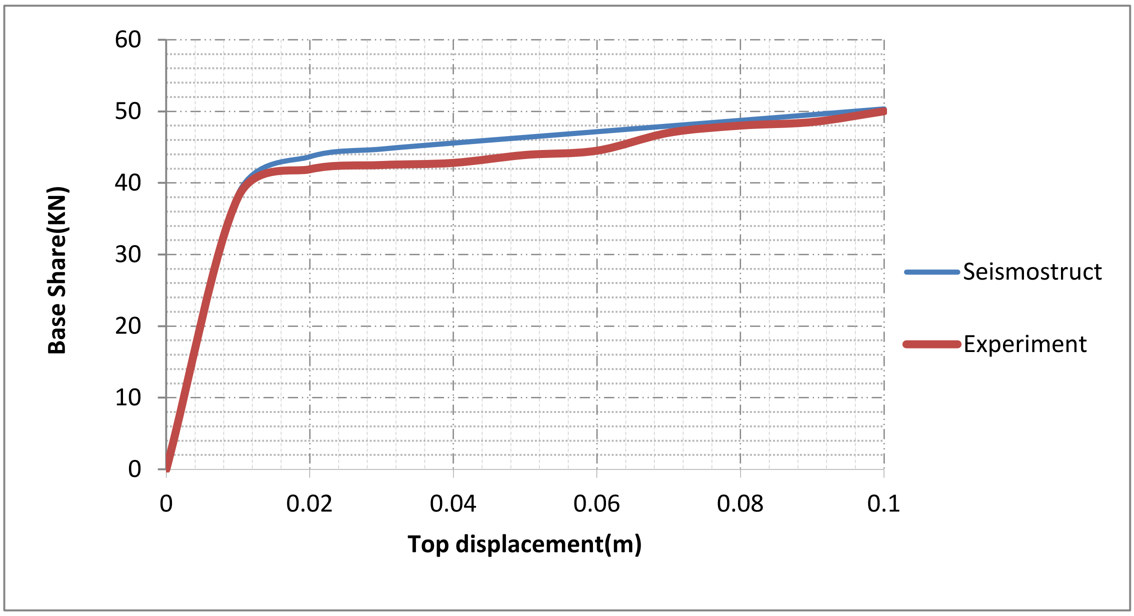


Figure 4.8: Validation bare frame model Results of Seismostruct 2016

4.2.1. BRACED FRAME VALIDATION

Validation Data

The geometric detail of braced frame is 920mm * 50mm *1050mm with bracing members of 8mm diameter rod. The box section detail of both frames is 100mm * 50mm *4mm.The materials used for casting of deck slab are Coarse aggregate (25mm), fine aggregate (sand), 53 grade cement, binder, PVC pipe, U-Hook, Wood. The size of form work is 1240mm * 940mm. The deck slab is casted for C20 mix, which has the concrete mix of 1:1.5:3 and Water/Cement ratio of 0.5. Deck slab is casted along with frames for experimental investigations.

The bare frame is provided with a fixed support at its bottom and horizontal load is applied at top left corner of the frame and the deflection is computed for each load step. According to experiment [10], the bare frame is subjected to a horizontal load of 60kN, so that the frame gets deflected and the maximum deformation occurs at the top of the section.

The overall dimension and structural configuration can be seen in *Figure 4.9.*

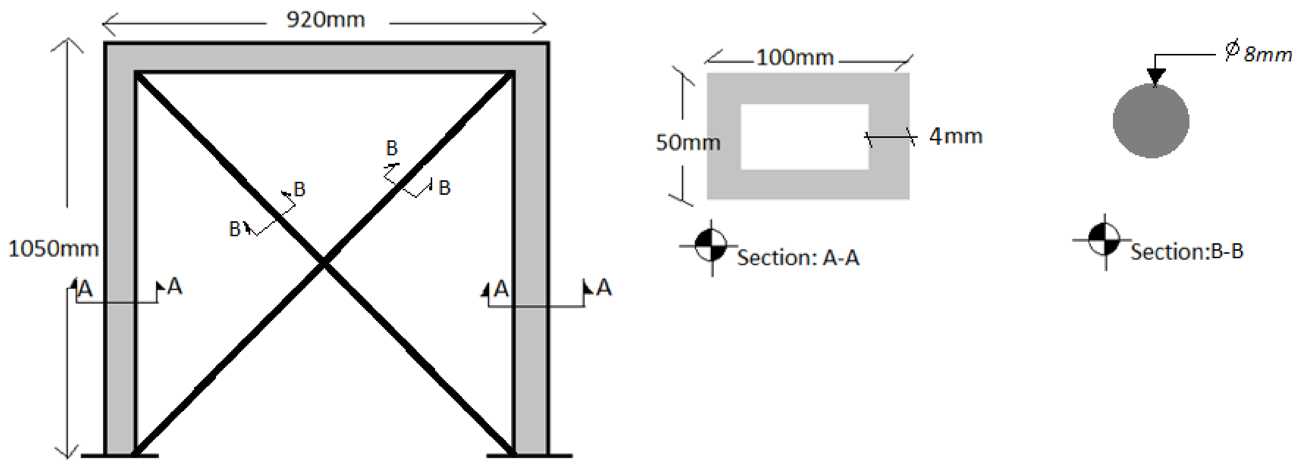


Figure 4.9: Validation of braced frame [10]

Validation braced frame model

The figure 4.10 shows seismostruct model of the braced frame used to verify the analytical result of the program with the experimental output.

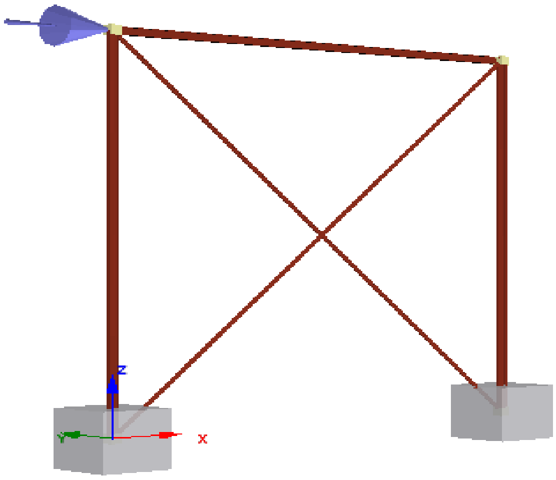


Figure 4.10: Validation Braced frame Model (seismostruct model)

Seismostruct 2016

As depicted on Figure 4.12 there is some variation between capacity curves from Seismostruct 2016 with the experiment output which have acceptably similar result. The pushover curve of the program is similar with the experiment value till the yielding zone of the material and shows that Seismostruct capacity curve could be above the experiment one starting from yield point of the frame. But the material models should capture the real behavior of materials as much as possible through the performance assessment as could be seen from the Seismostruct results. Figure 4.11 shows the capacity curves for load patterns and deformed shapes at first rupture of steel with the corresponding roof displacement.

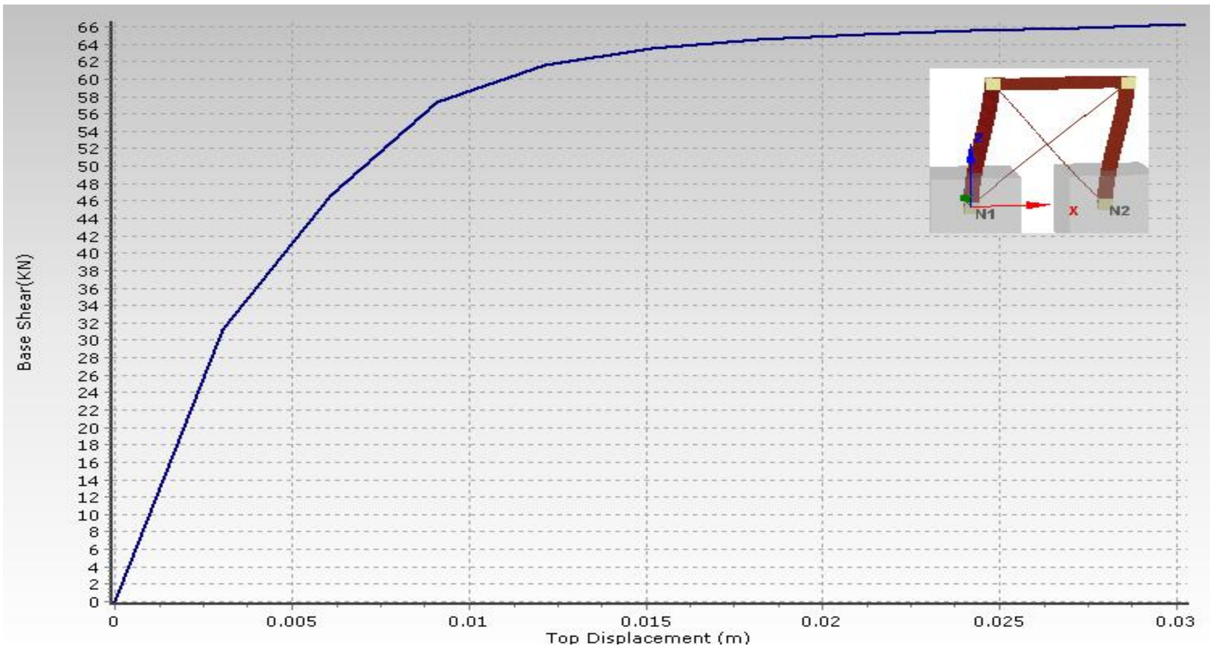


Figure 4.11: Seismostruct Capacity Curve and frame model at Steel Rupture

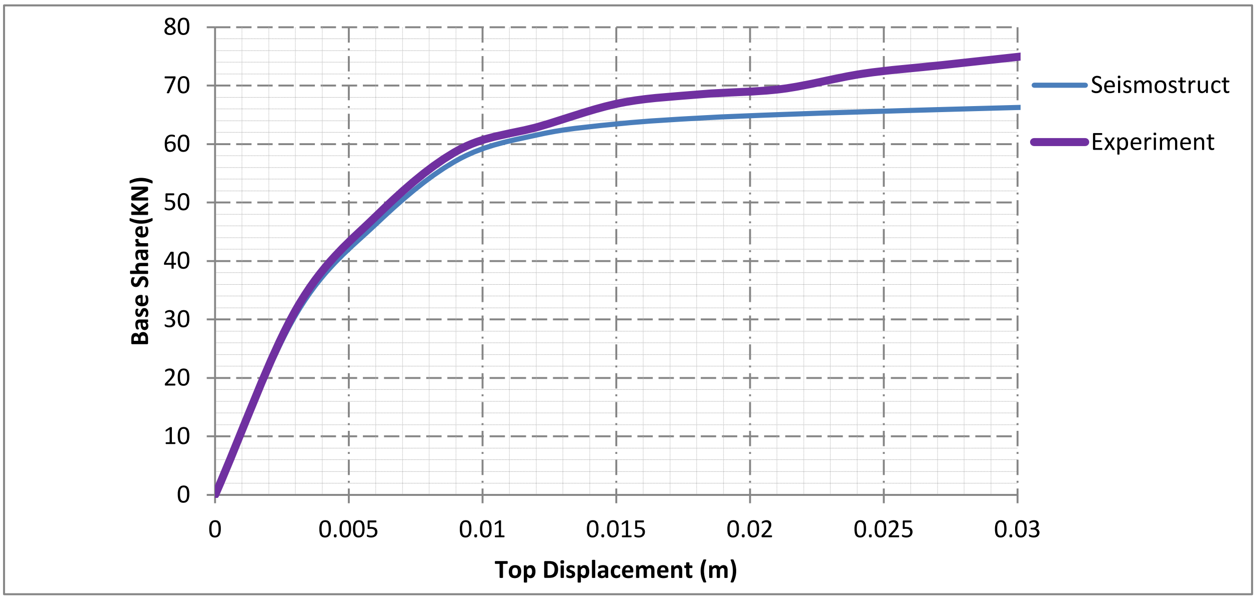


Figure 4.12: Validation braced frame model Results of Seismostruct 2016

From the above output we can wrap up that the ability of the finite-element program adopted within the present work to simulate nonlinear static response of steel frame structures to seismic loads has been proven by comparing analytical estimates; Seismostruct out put to the experimental result by the P. *Magudeaswaran et l.,(2016).*

CHAPTER V

5. PERFORMANCE ASSESSMENT OF THE CASE STUDY BUILDING

5.1 CASE STUDY BUILDING

The buildings selected for the study is a regular steel structure with G+10 and G+15 of Smart Parking System located in Addis Ababa city which is categorized as Zone Three (3) according to the seismic hazard map of Ethiopia. The aim of the case study primarily to provide additional results and thus allow for a better understanding of the performance of static procedures, with respect to recently proposed research works.

5.1.1. BUILDING DESCRIPTION

5.1.1.1. 15-Storey frame

The building is 6.60m x 22.80m in plan (Figure 5.1 Plan view) and 40.38m in elevation and floor-to-floor height of 2.29m for the odd storeys, 2.40m for the even storeys and for the top level storey of 3.00m (Figure 5.2: Front Elevation View and Figure 5.3: Side Elevation View). Both the interior and exterior bays of the structure contain vertical bracing system, while the level of the floor system also contain the horizontal bracing system for the purpose of lateral load resisting system in addition to the moment resisting framed system. The bays are each 2.70m wide, with three bays along the N-S direction uses as the main gate of the building, the rest six bays with 2.45m along the same direction and one bay with 6.60m along the E-W direction.

The structure system of the building is Steel beams and columns frame system with 235 MPa steel wide flange sections. For the moment resisting frames (MRF) columns are connected every two levels through moment- and uplift-resistant connections the center-line of the beam to column joint as shown in the figure 5.15. The frames are assumed fixed at their ground level from horizontal displacement and with diaphragm constraint at each storey level and rigid beam column joints. As per their identification name of Cross-sectional detail of beams, columns and bracings member are given under *Table 5.1.*

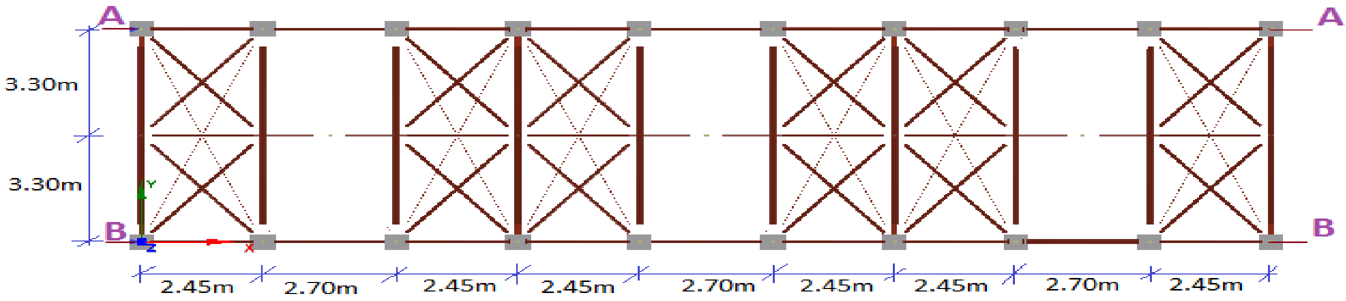


Figure 5.1: Plan view of 15-Storey frame [12]

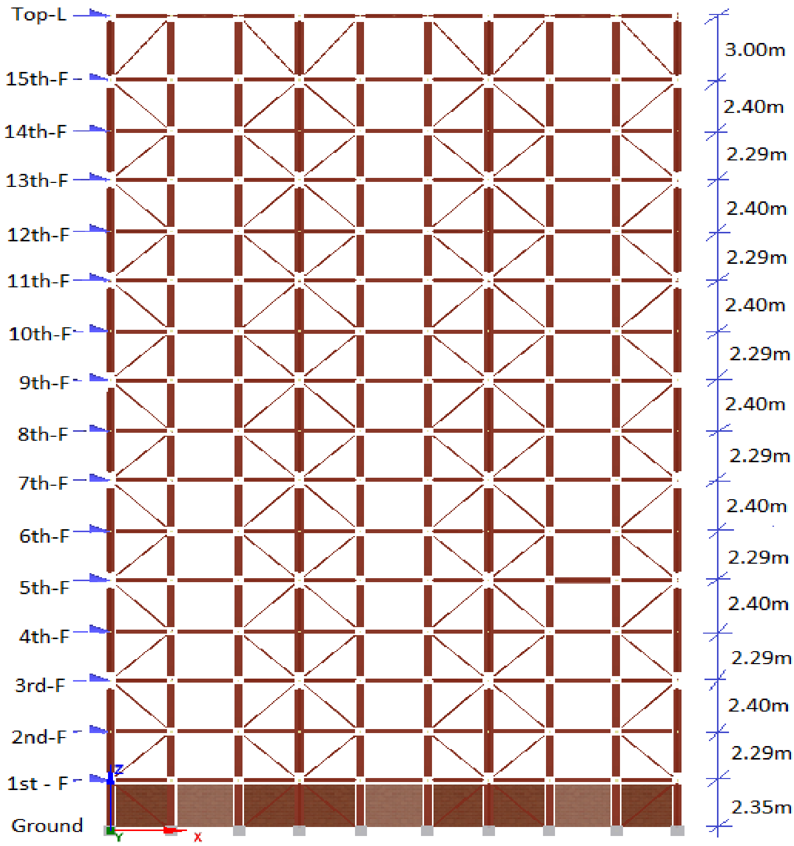


Figure 5.2: Front Elevation View (Axes A-A and B-B) of 15-Stranstorey frame [12]

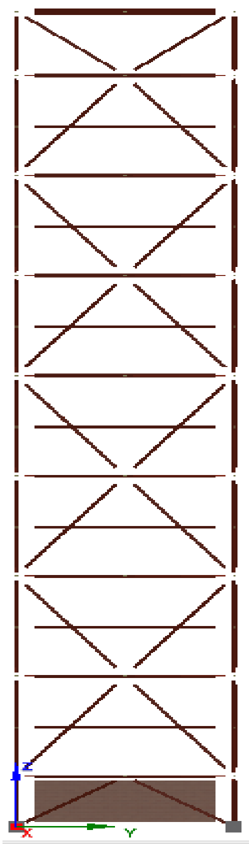


Figure 5.3: Side Elevation View (Axes A-B) of 15-Storey frame [12]

Member		Section(mm)	Member		Section(mm)	Strength(MPa)
Column	Gz1	HN 400*200*8*13	Bracing	Zc1	L70*5	235
	Gz2	HN 350*175*7*11		Zc2	Square Pipe 120*80*4	
Beam	GL1	H 150*150*7*10		Zc3	Square Pipe 80*80*5	
	GL2	HN 200*100*5.5*8		Zc3a	Square Pipe 120*120*4	
	GL3	HN 400*200*8*13		Zc4	Square Pipe 150*150*5	
	BL	HN 400*200*8*13		Zc4*	Solid Round Steel (18ϕ)	

Table 5.1: Member properties for of 15-Storey frame [12]

5.1.1.2. 10-Storey frame

The building is 6.55m x 30.40 m in plan (Figure 5.4: Plan view) and 26.4m in elevation and floor-to-floor height of 2.29m for the odd storeys, 2.40m for the even storeys and for the top level storey of 3.00m (Figure 5.5: Front Elevation View and Figure 5-6: Side Elevation View). Both the interior and exterior bays of the structure contain vertical bracing system, while the level of the floor system also contain the horizontal bracing system for the purpose of lateral load resisting system in addition to the moment resisting framed system. The bays are each 2.70m wide, with four bays along the N-S direction uses as the main gate of the building, the rest six bays with 2.45m along the same direction and one bay with 6.55m along the E-W direction.

The structure system of the building is Steel beams and columns frame system with 235 MPa steel wide flange sections. For the MRF columns are connected every two levels through moment- and uplift-resistant connections the center-line of the beam to column joint as shown in the figure 5.15. The frames are assumed fixed at their ground level from horizontal displacement and with diaphragm constraint at each storey level and rigid beam column joints. Table 5.2: gives the Cross-sectional detail of beams, columns and bracings member as per their identification name.

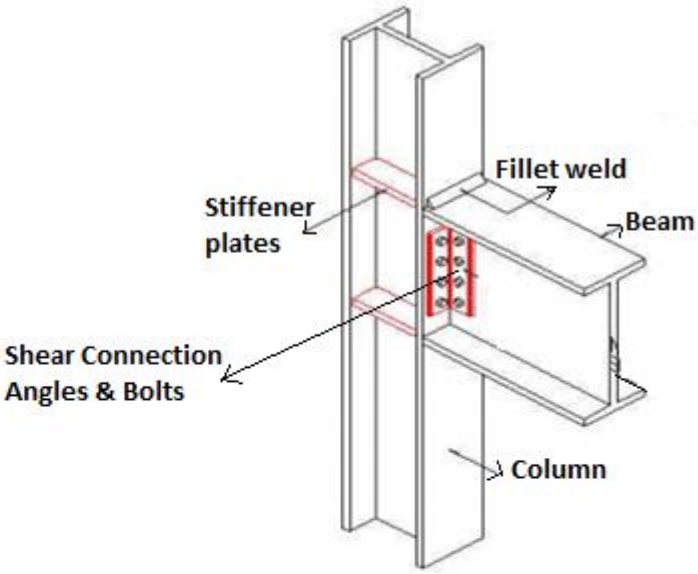


Figure 5.15: Detail of moment and uplift resistant (rigid) connections

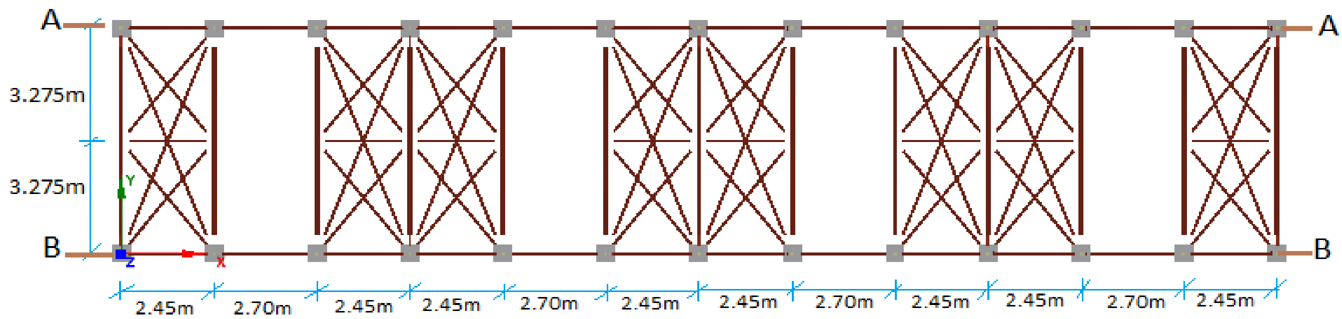


Figure 5.4: Plan view of 10-Storey frame [12]

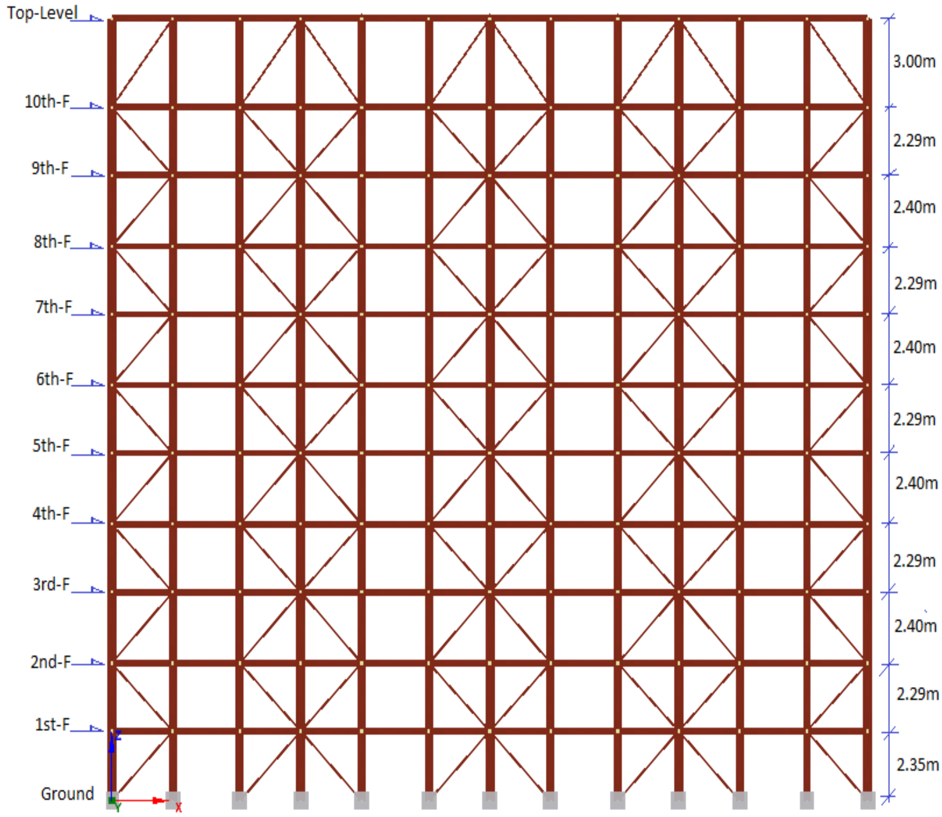


Figure 5.5: Front Elevation View (Axes A-A and B-B) of 10-Storey frame [12]

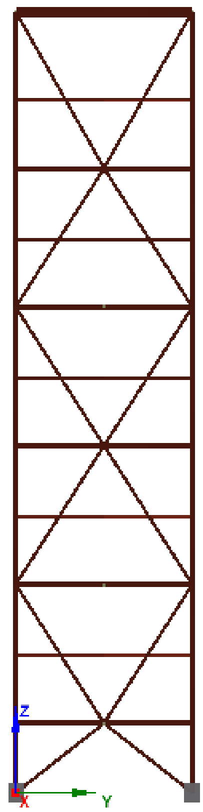


Figure 5.6: Side Elevation View (Axes A-B) of 10-Storey frame [12]

Member		Section(mm)	Member		Section(mm)	Strength(MPa)
Column	Gz1	HN 350*175*7*11	**Bracing**	Zc1	RHS 100*100*5	235
	Gz2	HN 300*150*6.5*9		Zc2	RHS 80*80*3	
Beam	GL1	H 150*150*7*10		Zc2a	RHS 80*80*5	
	GL2	HN 200*100*5.5*8		Zc3	RHS 120*120*4	
	GL3	RHS 120*80*4		Zc4	Solid Round Steel (18ф)	

Table 5.2: Member properties of 10-Storey frame [12]

5.1.2. ASSUMPTIONS

The following assumptions were made to formulate the research problem and develop the analyses models

- Steel moment-frame buildings are designed to resist earthquake ground shaking based on the assumption that they are capable of extensive yielding and plastic deformation, without loss of strength.
- **Fully Restrained Moment-Resisting Connections;** Nonlinear analysis models of structures with fully restrained connections should be based on the assumption that the connection provides a fully rigid interconnection between the beam and column, located at the centerline of the column, until the connection panel zone, beam or column yields, or a total inter storey drift angle (obtained from Table 4-12, FEMA 350) occurs. In this case, rigid beam-column joints were assumed for modeling and the capacity of adjacent components was not limited by the joint strength.
- No bond-slip
- No shear failure
- Considering infill wall effect at the bottom of the structure and diaphragm action on each floor of the framed structure.

Input Parameters

- The design spectrum from ES EN1998-1-1:2015 was scaled by peak ground accelerations shown in the table 5.3 here in below;

Sub – Soil classification	Peak ground accelerating	Importance Class	Coefficient of Damping
Sub-soil class A.	PGA 0.10g	Importance Class - I	5%
Sub-soil class B.	PGA 0.10g		

Table 5.3: Parameters to draw design Spectrum

Lateral Load Pattern

Different invariant lateral load patterns were utilized to represent the likely distribution of inertia forces imposed on the frames during an earthquake and the two types of loading profile were compared, *Uniform and Triangular* lateral load patterns.

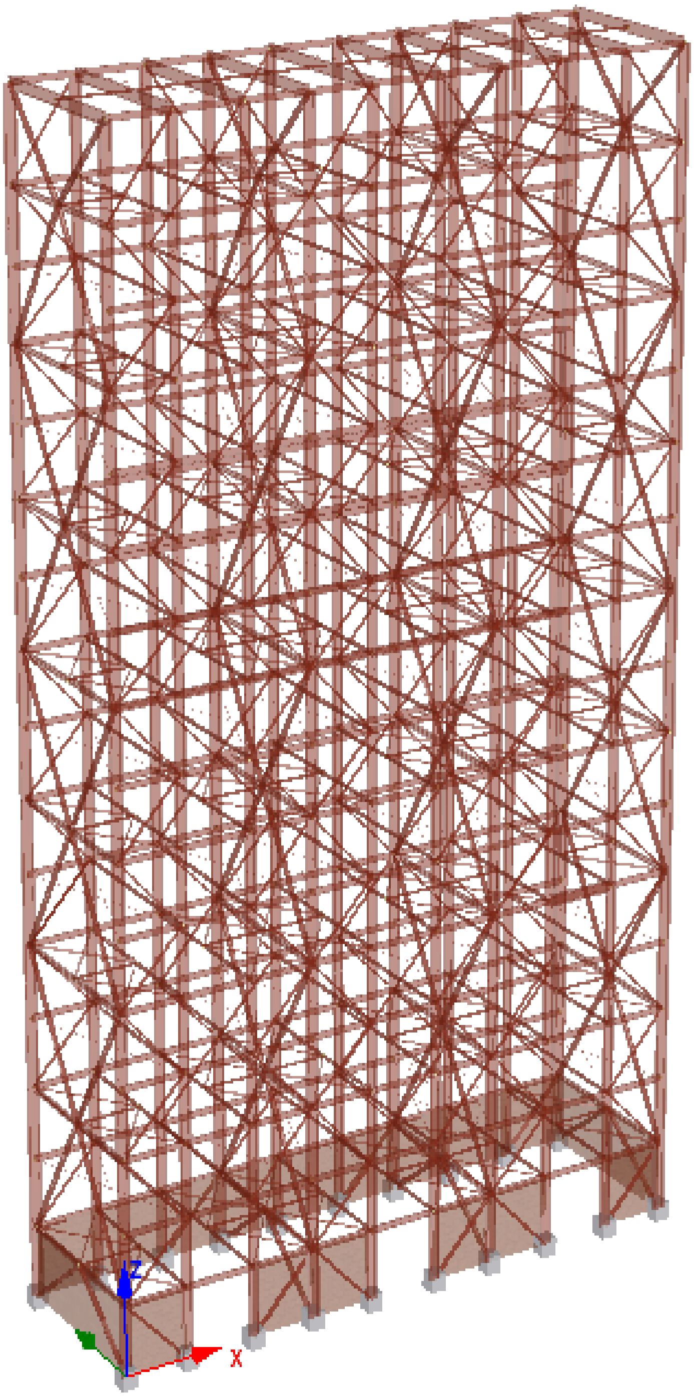
5.2 MODELING APPROACH

5.2.1. MATHEMATICAL/ANALYTICAL TOOL

All the analyses are performed through the finite element analysis program Seismostruct (Seismosoft, 2016) where Three-dimensional models of the analyzed structures have been implemented. Seismostruct is a fiber element-based program for seismic analysis of framed structures, freely downloadable from the internet and accepts static loads (either forces or displacements) as well as dynamic (accelerations) actions and has the ability to perform eigenvalues, nonlinear static push over (conventional and adaptive), nonlinear static time history analysis, nonlinear dynamic analysis and incremental dynamic analysis.

The program incorporates both local (beam-column effects) and global (large displacements/rotations effects) taking in to account material inelasticity and geometric nonlinearity as well as the interaction between axial force and transverse deformation of the element. The spread of material inelasticity along the member length is explicitly represented trough the employment of a fibre modeling approach, implicit in the formulation of the inelastic beam-column frame elements adopted in the analysis.

5.2.2. THREE DIMENSIONAL (3-D) MODEL OF THE CASE STUDY BUILDINGS



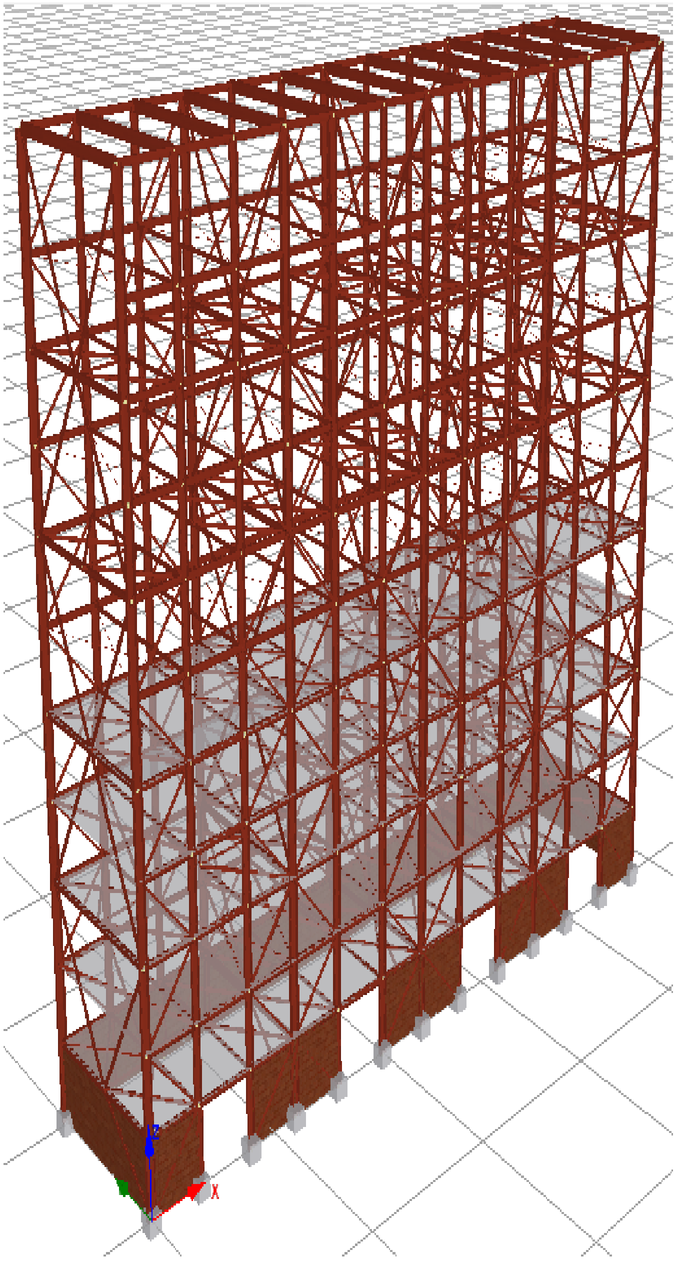


Figure 5.7: Three dimensional structural model (Seismostruct 2016) of 15-Storey frame [12]

Figure 5.8: Three dimensional structural model (Seismostruct 2016) of 10-Storey frame [12]

5.3 RESULTS AND DISCUSSION

Analyses were conducted to evaluate the performance of Steel structures under seismic loading with different bracing type and shear wall. Results of Response Spectrum Analysis have been used to observe and compare floor response of all the models in terms of the following parameters.

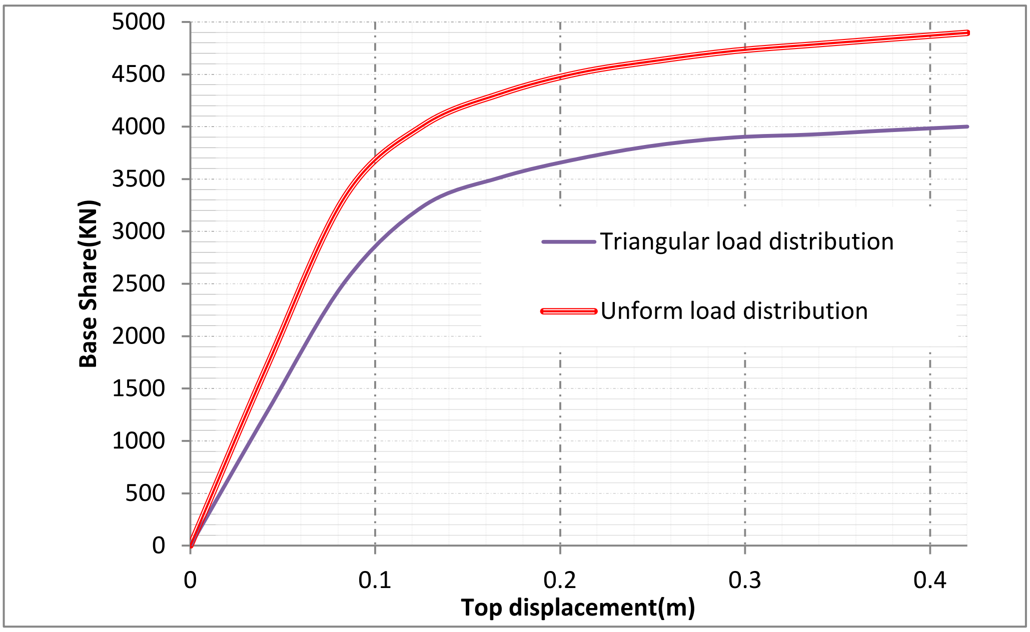
a) Capacity Curve
b) Storey displacements
c) Storey Drift.

The comparison of results in terms of the above parameters was given in terms of tables & graphs in the coming paragraph

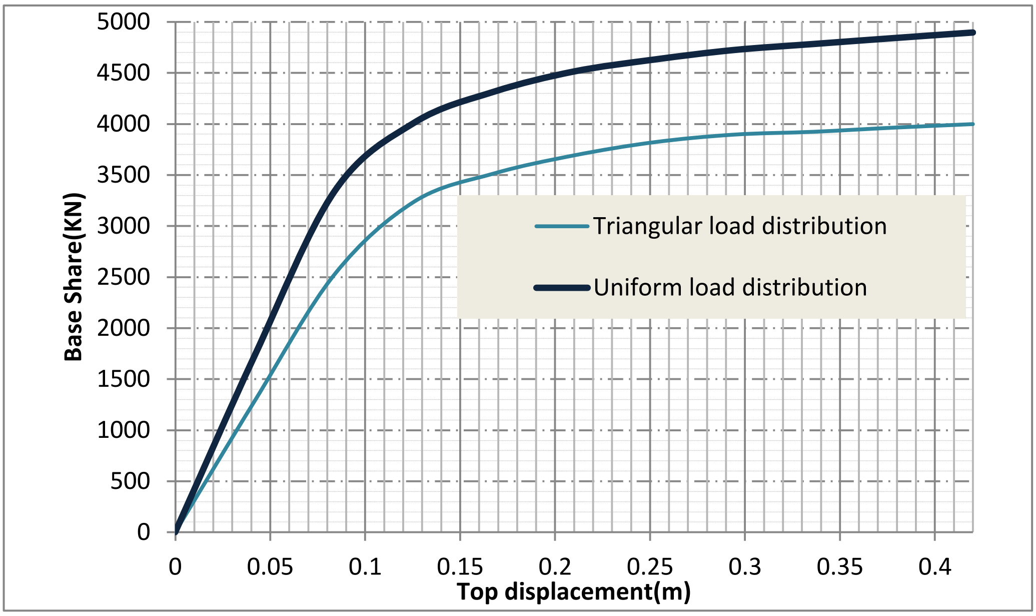
5.3.1. CONSTRUCTION OF CAPACITY CURVE OF STEEL FRAME'S RESPONSE

One of the most important steps in post-processing of nonlinear structural analysis is to achieve the capacity curve (base shear versus top displacement). The capacity curve can reveal important features of structural response, includes yield displacement, total strength and initial stiffness estimation of the structure. Thus, it is important and crucial to compare Capacity curves for case study frames were obtained from the pushover analyses using Triangular and uniform lateral load patterns.

Capacity curves are the load-displacement envelopes of the structures and represent the global response of the structures. The results of capacity curves for case study frames were obtained from the pushover analyses using Triangular and uniform lateral load patterns are shown in Figure 5.9-10.

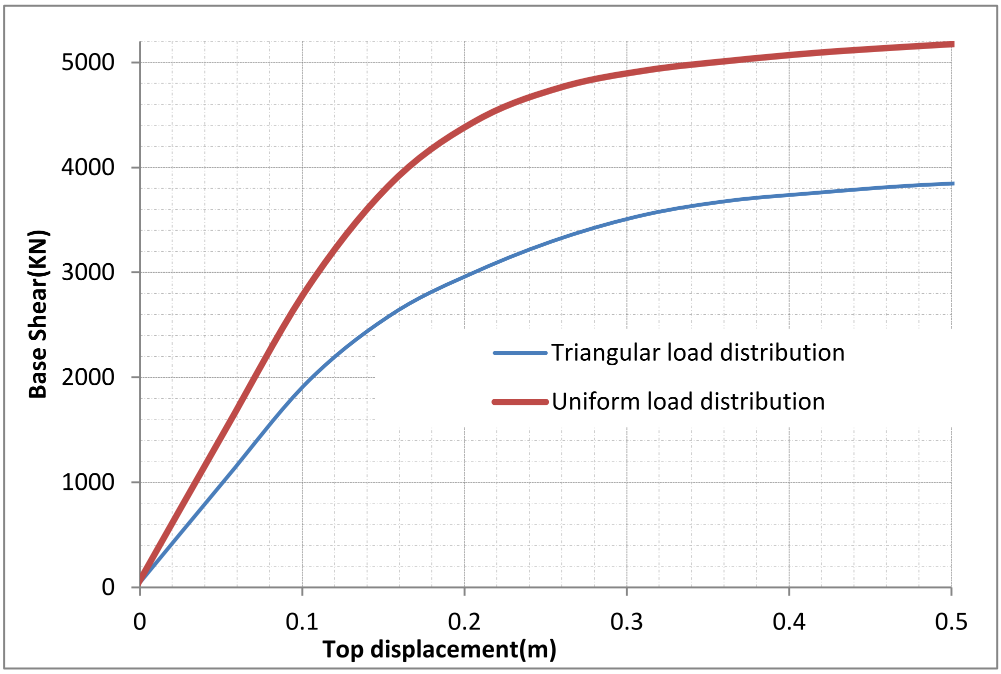


Sub soil A,PGA 0.10g

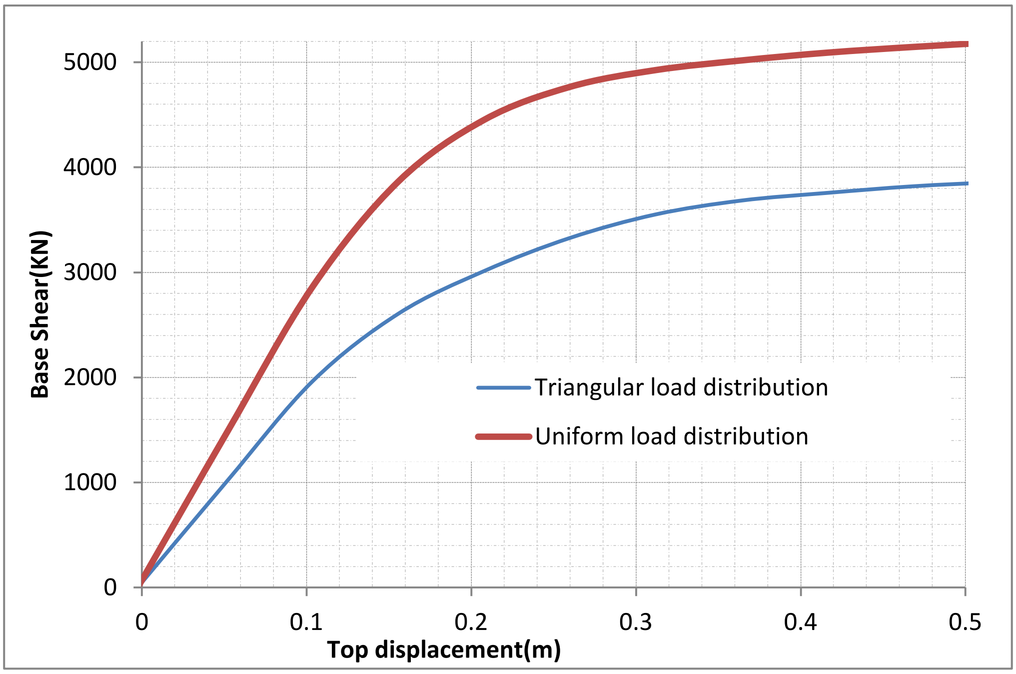


Sub soil B,PGA 0.10g

Figure 5.9: Capacity Curve of steel frame for 10 storey buildings case with Triangular and Uniform load distribution



Sub soil A,PGA 0.10g



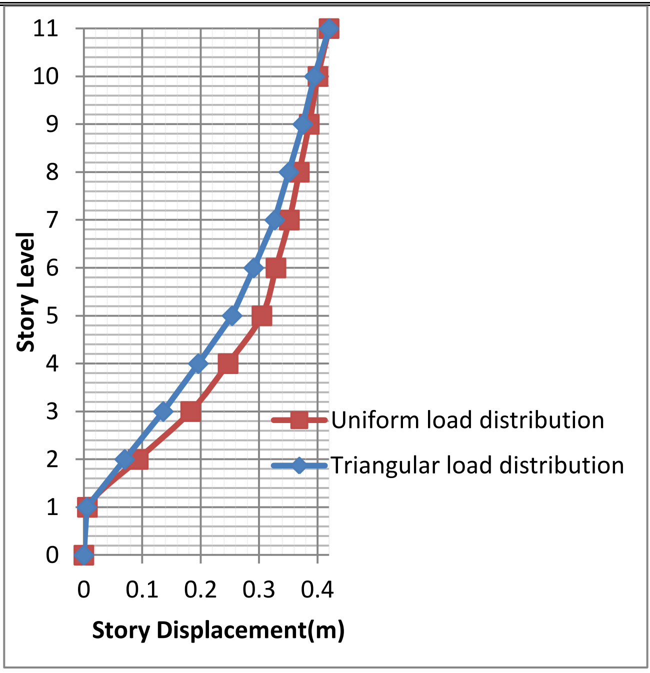
Sub soil B,PGA 0.10g

Figure 5.10: Capacity Curve of steel frame for 15 storey buildings case with Triangular and Uniform load distribution

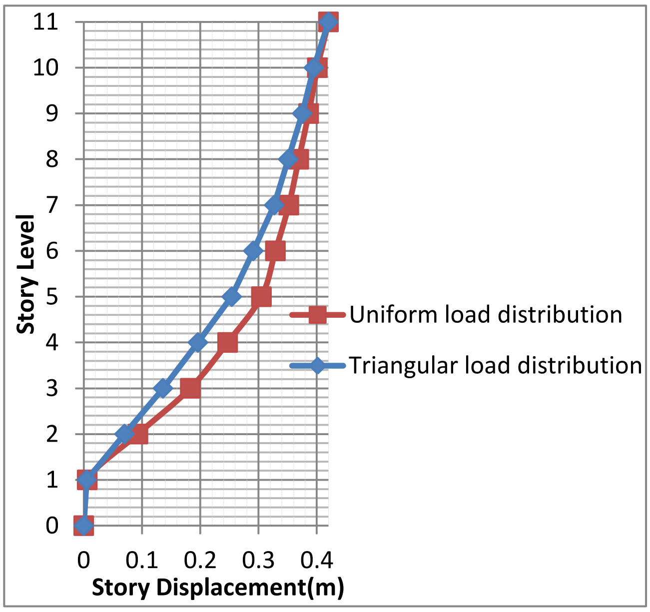
5.3.2. STOREY DISPLACEMENTS

Storey displacement is the absolute value of displacement of the storey under action of the lateral forces and which is helpful for estimating the inert storey drift of the case study building and the same used for seismic performance evaluation purposes.

The figure 5.11-5.12, below shows displacement profile at the performance point in which statically calculated force demands are used to find the distribution of points over height of the buildings.

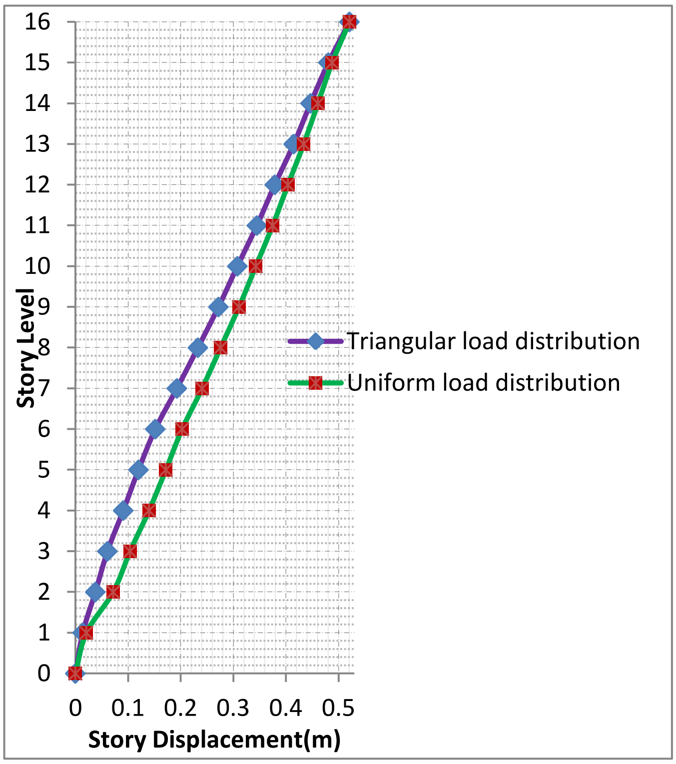


Sub soil A,PGA 0.10g

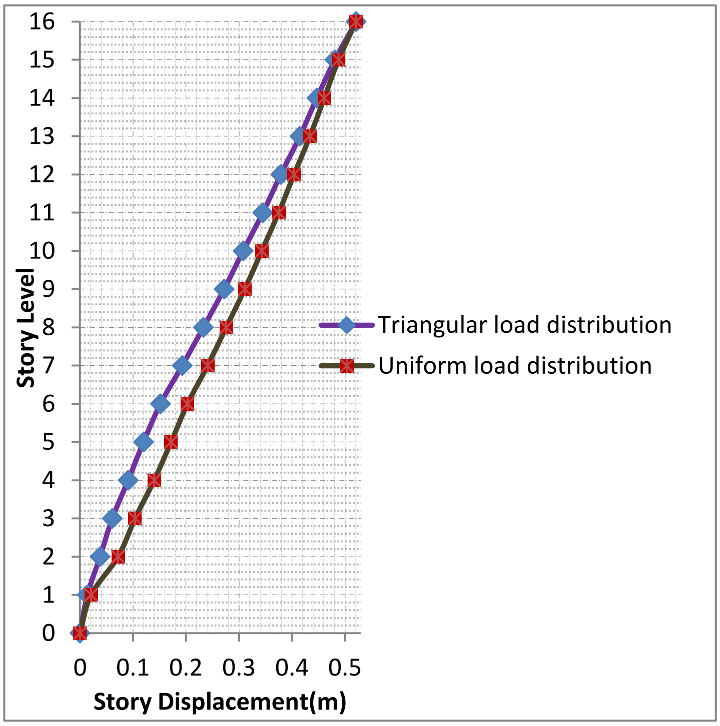


Sub soil B,PGA 0.10g

Figure 5.11: Storey displacements of steel frame for 10 storey buildings case with Triangular and Uniform load distribution



Sub soil A,PGA 0.10g



Sub soil B,PGA 0.10g

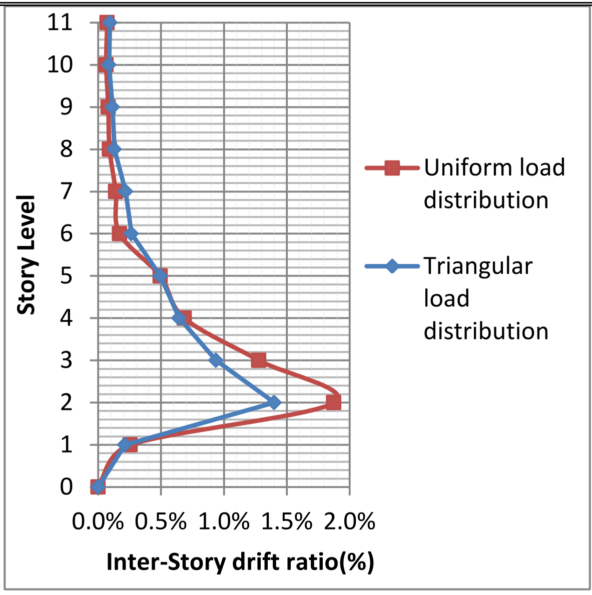
Figure 5.12: Storey displacements of steel frame for 15 storey buildings case with Triangular and Uniform load distribution

5.3.3. INTER-STOREY DRIFT REQUIREMENT

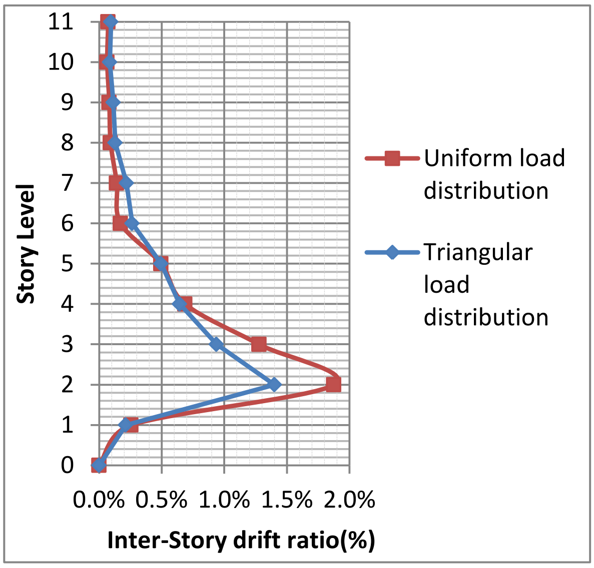
Lateral (storey) drift is the amount of sideway between two adjacent stories of a building caused by lateral (wind and seismic) loads. The precise estimation of inter-storey drift ratio and its distribution along the height of the structure is very decisive for seismic performance evaluation purposes since the structural damage is directly related to the inter-storey drift ratio. Therefore, at each instant of the push over analysis, the storey drift is evaluated using the following equation;

$$\delta_i(t) = \frac{d_i - d_{i-1}}{h_i},$$

where; δ_i = is inter storey drift, h_i = is the height of storey i;
d_i = is the displacement at the floor i;
d_{i-1} = is the displacement at the floor $i-1$



Sub soil A,PGA 0.10g



Sub soil B,PGA 0.10g

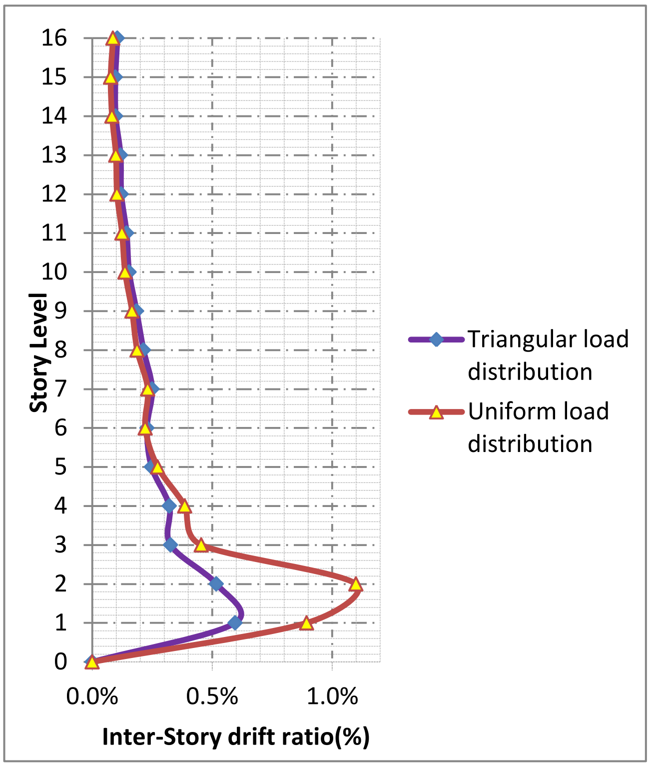
Figure 5.13: Inter-Storey drift ratio of steel frame for 10 storey buildings case with Triangular and Uniform load distribution

Figure 5.13 shows nonlinear height wise distribution of stiffness shifted inelastic demands to the upper stories with respect to the inverted triangular and uniform lateral load distribution acting to the positive of X- direction considering the magnitude of Peak ground acceleration (PGA 0.10g) and Class of the sub soil (class A and class B), though approximately the same storey displacement was displayed regarding the two types of the sub soil class to the software *(see figure 5.11).*

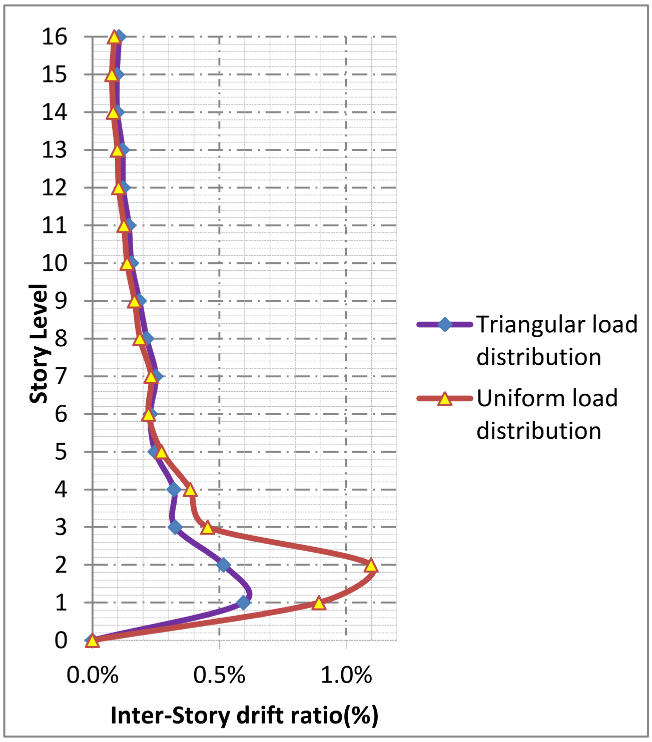
Based on the response limits for storey drift ratio described in APPENDIX-A, Table A-1: Structural Performance Levels and Damage for steel Structure -Vertical Elements, the performance level of the building was determined as shown in table below.

Lateral Load pattern: Triangular/inverted shape to the +ve X direction				
Storey Level	Storey Drift	IO	LS	CP
12	0.095%	Ok!	Ok!	Ok!
11	0.085%	Ok!	Ok!	Ok!
10	0.114%	Ok!	Ok!	Ok!
9	0.128%	Ok!	Ok!	Ok!
8	0.219%	Ok!	Ok!	Ok!
7	0.264%	Ok!	Ok!	Ok!
6	0.494%	Ok!	Ok!	Ok!
5	0.643%	Not Ok!	Ok!	Ok!
4	0.938%	Not Ok!	Ok!	Ok!
3	1.401%	Not Ok!	Ok!	Ok!
2	0.213%	Ok!	Ok!	Ok!
1	0.000%	Ok!	Ok!	Ok!
Lateral Load pattern: Uniform to +ve X direction				
12	0.072%	Ok!	Ok!	Ok!
11	0.064%	Ok!	Ok!	Ok!
10	0.081%	Ok!	Ok!	Ok!
9	0.091%	Ok!	Ok!	Ok!
8	0.140%	Ok!	Ok!	Ok!
7	0.171%	Ok!	Ok!	Ok!
6	0.494%	Ok!	Ok!	Ok!
5	0.686%	Not Ok!	Ok!	Ok!
4	1.278%	Not Ok!	Ok!	Ok!
3	1.875%	Not Ok!	Not Ok!	Ok!
2	0.255%	Ok!	Ok!	Ok!
1	0.000%	Ok!	Ok!	Ok!

Table 5.3: Storey-Drift Requirement of steel frame for 10 storey buildings



Sub soil A,PGA 0.10g



Sub soil B,PGA 0.10g

Figure 5.14: Inter-Storey drift ratio of steel frame for 15 storey buildings case with Triangular and Uniform load distribution

Figure 5.14 shows nonlinear height wise distribution of stiffness shifted inelastic demands to the upper stories. Based on the response limits for storey drift ratio described in APPENDIX-A, Table A-1: Structural Performance Levels and Damage for steel Structure -Vertical Elements, the performance level of the building was determined as shown in table below.

Lateral Load pattern: Triangular/inverted shape to the +ve X direction				
Storey Level	Storey Drift	IO	LS	CP
16	0.105%	Ok!	Ok!	Ok!
15	0.097%	Ok!	Ok!	Ok!
14	0.098%	Ok!	Ok!	Ok!
13	0.118%	Ok!	Ok!	Ok!
12	0.121%	Ok!	Ok!	Ok!
11	0.143%	Ok!	Ok!	Ok!
10	0.154%	Ok!	Ok!	Ok!
9	0.185%	Ok!	Ok!	Ok!
8	0.214%	Ok!	Ok!	Ok!
7	0.250%	Ok!	Ok!	Ok!
6	0.228%	Ok!	Ok!	Ok!
5	0.247%	Ok!	Ok!	Ok!
4	0.322%	Ok!	Ok!	Ok!
3	0.327%	Ok!	Ok!	Ok!
2	0.517%	Not Ok!	Ok!	Ok!
1	0.596%	Not Ok!	Ok!	Ok!
Lateral Load pattern: Uniform to +ve X direction				
16	0.086%	Ok!	Ok!	Ok!
15	0.077%	Ok!	Ok!	Ok!
14	0.082%	Ok!	Ok!	Ok!
13	0.098%	Ok!	Ok!	Ok!
12	0.103%	Ok!	Ok!	Ok!
11	0.124%	Ok!	Ok!	Ok!
10	0.137%	Ok!	Ok!	Ok!
9	0.166%	Ok!	Ok!	Ok!
8	0.187%	Ok!	Ok!	Ok!
7	0.231%	Ok!	Ok!	Ok!
6	0.221%	Ok!	Ok!	Ok!
5	0.273%	Ok!	Ok!	Ok!
4	0.386%	Ok!	Ok!	Ok!
3	0.455%	Ok!	Ok!	Ok!
2	1.099%	Not Ok!	Ok!	Ok!
1	0.894%	Not Ok!	Ok!	Ok!

Table 5.4: Storey-Drift Requirement of steel frame for 15 storey buildings

CHAPTER VI

6. SUMMARY AND CONCLUSION

6.1. CONCLUSION

In this research work, the seismic performance assessment with non linear static analysis on two existing steel framed structures; G+10 and G+ 15 smart parking systems are studied. The Seismostruct 2016 package program is used by taking a single seismic load developed from artificially generated ground motion history that best fits the response spectrum of ES EN1998-1-1:2015 with approximate hazard level of 0.10g ground acceleration. The seismic performance of the case studies are assessed according to FEMA 356, 2000 and Capacity Spectrum Method proposed in ATC-40, 1996 using the pushover analysis results that are fundamental period of buildings, absolute maximum storey displacement, maximum storey drift and storey ductility demands.

Therefore, conclusions from this study are drawn as follows:

- The curve line for capacity curve (push over curve) of both case study buildings due to applying uniform lateral load distribution is above the capacity curve of triangular lateral load distribution. To this end, Conventional pushover analysis with triangular load pattern distribution gives higher displacement while the uniform lateral load pattern gives higher base shear.
- The capacity of the case study building; G+10 Smart Parking System is below the force demand for class A and B with uniform lateral load distribution at IO and LS and above the force demand for soil class A and B with triangular lateral load distribution at LS and CP of structural performance level set by FEMA356 - 2000. Therefore, The G+10 Smart Parking System complies the considering the triangular lateral load distribution based on structural performance level set by the FEMA 356-2000.
- The capacity of the other case study building; G+15 Smart Parking System is below the force demand for soil class A and B with both lateral load distribution at IO and above the force demand for soil class A and B with both lateral load distribution at LS and CP of structural performance level set by FEMA356 - 2000. Therefore, The G+15 Smart Parking System complies the LS considering both type of lateral load distribution for demand of 0.10g based on structural performance level set by the same.
- Uniform lateral load pattern mostly emphasized demands in lower stories over demands in upper stories as observed in story displacement, inter-story drift ratio predictions while triangular lateral load patterns predicted the response more homogenously over the frame height. Therefore,

seismic demand predictions of triangular lateral load patterns were better than uniform loading predictions for all frames at all deformation levels.

- The triangular load patterns yielded almost same predictions of global capacity curve, story displacement, inter-story drift ratio for low to mid-rise frames since the variation in height-wise distribution of triangular lateral load patterns is negligible for low to mid-rise frames. Therefore, any triangular lateral load pattern could be used to predict seismic demands of low to mid-rise frames.
- The assessment of the structural behavior against possible earthquakes should be the fundamental point on the design and construction process. In other words, the performance of the structure should be known and investigated before construction.

6.2. RECOMMENDATION

Based on this study, the following are the main points that deserve attention for future study.

- In this study, the cases study of building structure fulfils plan and elevation regularity was studied. So, the behavior of irregular structures under seismic loads has been left for future study.
- The effect of wind on the seismic behaviour of braced frame and shear walled frame is left for future study.
- Effect of the connection details to make the most suitable design to achieve desired performance, serviceability and optimum member criterias while assessing seismic performance has been left for future study.
- Additionally the effect of soil-structure interaction needs to be adequately addressed.

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APPENDIX

APPENDIX- A: Seismic Performance Criteria

Target Building Performance Levels

A target Building Performance Level shall consist of a combination of a Structural Performance Level selected from the levels specified in Section 1.5.1 and a Nonstructural Performance Level selected from the levels specified in Section 1.5.2. The target Building Performance Level shall be designated alphanumerically in accordance with Section 1.5.3(FEMA 356, 2000).

Target Building Performance Levels Building performance is a combination of the performance of both structural and nonstructural components.

Table C1-2 and Table C1-3 (FEMA 356, 2000), describes the approximate limiting levels of structural and nonstructural damage that may be expected of buildings rehabilitated to the levels defined in the standard. On average, the expected damage would be less. For comparative purposes, the estimated performance of a new building subjected to the level of shaking is indicated. Performance descriptions in Table C1-3 are estimates rather than precise predictions, and variation among buildings of the same target Building Performance Level must be expected.

Structural Performance Levels and Ranges

The Structural Performance Level of a building shall be selected from four discrete Structural Performance Levels and two intermediate Structural Performance Ranges defined in this section.

The discrete Structural Performance Levels are;

- ✓ ***Immediate Occupancy (S-1):*** defined as the post-earthquake damage state that remains safe to occupy, essentially retains the pre-earthquake design strength and stiffness of the structure, and is in compliance with the acceptance criteria specified in this standard for this Structural Performance Level.
- ✓ ***Life Safety (S-3):*** defined as the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse in compliance with the acceptance criteria specified in this standard for this Structural Performance Level.
- ✓ ***Collapse Prevention (S-5):*** defined as the post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse in compliance with the acceptance criteria specified in this standard for this Structural Performance Level.

The intermediate Structural Performance Ranges are;

- ✓ ***Damage Control Range (S-2):*** defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Immediate Occupancy Structural Performance Level (S-1)
- ✓ ***Limited Safety Range (S-4):*** defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Collapse Prevention Structural Performance Level (S-5).

The drift values given in Table A-1 are typical values provided to illustrate the overall structural response associated with various Structural Performance Levels (FEMA 356, 2000).

Element	Type	CP	LS	IO
Steel moment frame	Drift	5%transient	2.5%transient	0.7%transient
Braced steel frame	Drift	2%transient	1.5%transient	0.5%transient

Table A-1: Structural Performance Levels and Damage for steel Structure -Vertical Elements

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