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Determine Seismic Coefficients of Structural Using Pushover Nonlinear Analysis

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Abstract

This study focus push-over analysis technique for performance-based design of steel building frame works subjected to earthquake loading using SPA2000. Through the use of a plasticity-factor that measures the degree of plasticisation, the standard elastic and geometric stiffness matrices for frame elements are progressively modified to account for nonlinear elastic–plastic behaviour under constant gravity loads and incrementally increasing lateral loads. The proposed analysis technique is illustrated for two steel frameworks of solid and hollow member properties. This research studies are aimed at analysing the comparison between hollow and solid frames. The technique is based on the conventional displacement method of elastic analysis. The analytical procedure developed is to estimate the inelastic deformations of beams, columns and connections are validated by incorporating the same in pushover analysis. It has been observed that the inelastic displacement of the structure is within the collapse prevention level.

Keywords: Seismic Performance, Pushover analysis, Capacity curves, Inelastic deformations, Capacity curve, Behavior factor

1. Introduction

Pushover analysis is a static, nonlinear method structure's capability to resist the seismic demand in which the size of the structural loading is performance is related on the manner that the capacity gradually increased in conformity with a definite is able to handle the demand. Its mean that the structure predefined pattern. With the increase in the magnitude of must have the capacity to resist demands of the loading, weak links and destruction modes of the earthquake such that the performance of the structure is structure are found [19]. This method provides adequate data on seismic demands inflict by the specific ground motion on the structural system and its components [4,11,12]. The inelastic material and member deformation specifications [7,8] are essential for dependable simulation of inelastic behavior of the structure. In the present study pushover analysis of an existing 2- story steel building is perform by combine inelastic material behavior and allocate inelastic effects to plastic hinges at member ends. The analysis is accomplishing with finite element analysis software (SAP2000) [14], and gravity loads are applied

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initially. A predefined lateral load pattern which is distributed along the structure height is then applied. The lateral forces [6,13] are increased until some member's yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional member's yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with respect to base shear to get the global capacity curve. The capacity curve procure is evaluate with passable levels such as immediate occupancy life safety and collapse prevention for the evaluation of performance of the building.

2. SEISMIC EFFICIENCY ASSESSMENT

The assessment of the seismic efficiency of buildings is necessary to determine a passable solution in terms of capacity and performance. seismic efficiency assessment can be performing by conducting static, modal and dynamic response analysis of a structure. amid this, the static pushover analysis is becoming a popular tool for seismic performance assessment of existing and new structures.

3. PUSHOVER ANALYSIS

The pushover analysis can be used to evaluate the envisage performance of a structural system by appraise its strength and deformation demands for design earthquakes by means of static inelastic analysis, and comparing these demands with available capacities at the performance levels of attention. In pushover analysis, the seismic demands are estimated by the nonlinear static analysis of structure subjected to monotonically increasing lateral forces varying through the height of the structure. The analysis is carried out by applying the gravity loads followed by lateral loading along a direction starting at the end of the gravity push [14]. The structure is pushed until either a predetermined target displacement [3] is reached or it collapse. The reliable post-yield material model and inelastic member deformations are extremely important in the nonlinear analysis. The evaluation is based on an assessment of important parameters, including global drift, inter-story drift, inelastic element deformations (either absolute or normalized with respect to a yield value), deformations between elements, and element and connection forces (for elements and connections that cannot sustain inelastic deformations). The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces occurring when the structure is subjected to inertia forces that no longer can be resisted within the elastic range of structural behavior. The two key steps in applying this method, i.e. lateral force distribution and target displacement are based on the assumption that the structure's response is mainly from the fundamental mode, and that the mode shapes remain unchanged after structure gets into the inelastic region [6].

3.1. PERFORMANCES TARGET

The seismic efficiency of a building is measured by the state of damage under a determined level of seismic risk. The state of damage is measured by the drift of the roof and the displacement of the structural elements. Before the analysis of a building, a target performance level of the building and level of seismic risk are selected. A performance objective specifies the desired seismic performance of the building. Seismic performance is described by designating the maximum allowable damage state (performance ground motion). A performance objective may include consideration of damage states for several levels of ground motion. The selection of the two levels is based on recommended guidelines for the type of the building, economic consideration and engineering judgment. [1]

3.1.1. CHOOSING OF EFFICIENCY TARGET

The efficiency *target* of an analysis is the *choosing* of a building efficiency level under a specified earthquake. If the objective includes two building efficiency levels under two earthquake *target*, then it is a dual level efficiency objective. Similarly, there can be multiple level performance objectives. A basic safety objective (BSO) is defined as the dual requirement of life safety under design basis earthquake (DBE) and collapse prevention under maximum considered earthquake (MCE). The aim of BSO is to have a low risk of life threatening injury during a moderate earthquake (as defined by DBE) and to check the collapse of the vertical load resisting system during a severe earthquake (as defined by MCE). For analysis of multistory buildings in India, collapse prevention under MCE can be selected. It is a partial performance objective as per FEMA356. Unless the earthquake level of DBE as per IS: 1893-2002 is comparable to the level defined based on the probabilistic method, it is not prudent to check life safety under DBE. It may be noted that checking one performance level will not meet the damage control requirement for frequent earthquake.

3.2. I MPORTANT FACILITY OF EFFICIENCY EVALUATION OF STRUCTURE

Two significant features of performance evaluation of buildings are demand and capacity. Demand is the representation of earthquake ground motion and capacity is a representation of the structure's ability to resist the seismic demand. Performance is dependent on the manner that the building is able to handle the demand.

3.2.1. CAPACITY

The overall capacity of a building depends on the strength and deformation capacities of single element of the building. In order to obtain capacities beyond the elastic limits some form of nonlinear analysis is required. This procedure uses a series of sequential elastic analyses superimposed to approximate a force-displacement capacity diagram of the overall structure. The capacity curve is constructed to represent the first mode response of the structure based on the assumption that the fundamental mode of vibration is generally the governing response of the structure. This is valid for buildings with fundamental periods of vibration up to 1 second. For more flexible buildings with fundamental period greater than 1 second, higher modes need to be considered.

3.2.2. Demand

Demand is the delegation of earthquake ground motion and capacity is a representation of the structure's ability to resist the seismic demand. There are three methods to establish the demand of the building. They are (a) capacity spectrum method, (b) identical displacement method and (c) displacement factor method [2,3].

3.2.3. ASSESSMENT BASED ON NONLINEAR PUSHOVER ANALYSIS

Pushover analysis is a nonlinear static analysis in which the magnitude of the lateral load is slowly increased on an increasing basis, sustaining a predefined distribution pattern along the height of the building. The enlarge in magnitude of the loads, results in weak links and failure modes of the building. Pushover analysis can obtain the accurate behavior of a building, including the ultimate load and the maximum inelastic deformations. At each load step, the base shear and the roof displacement can be plotted to generate the pushover curve. It results in estimation of maximum base shear that the structure is capable of resisting. Further, it can also estimate of global stiffness in case of regular buildings.

3.3. EFFICIENCY LEVELS OF STRUCTURE AND ELEMENTS

A building efficiency level is a combination of the efficiency levels of the structural and the non-structural components. An efficiency level details a limiting damage situation which may be considered satisfactory for a given building and a given ground motion. The structural performance levels are designated using names and numbers, while non-structural performance levels are designated using names and letters. The performances levels are discrete damage states identified from a continuous spectrum of possible damage states and shown in Fig.1 [2,3]

The structural performances levels are as follows.

1. Immediate occupancy (IO)
2. Life safety (LS)

3. Collapse prevention (CP)

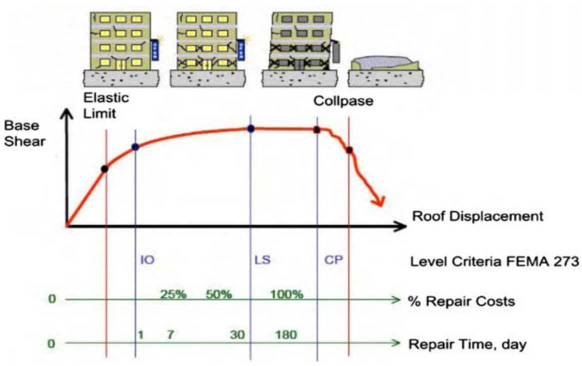

Figure 1:Performance Levels as per FEMA 356

3.3.1. Immediate Occupancy (IO)

The structural performance level, immediate occupancy, means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical- and 3 lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low. Although some minor structural repairs may be appropriate, these are generally not required prior to re-occupancy. [2,3]

3.3.2. Life Safety (LS)

The structural performance level, life safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to reoccupancy. [2,3]

3.3.3. COLLAPSE PREVENTION (CP)

The structural performance level, collapse prevention, means the post-earthquake damage state in which the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system, large permanent lateral deformation of the structure, and to a limited extent of degradation in vertical load carrying capacity. However, all significant components of the gravity-load resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. It may not be technically practical to repair the structural components and is not safe for re-occupancy, as aftershock activity could induce collapse. The three levels are arranged according to decreasing performance of the lateral load and vertical load resisting systems. A target performance is defined by a typical value of the roof drift, as well as limiting values of the deformation of the structural elements. To determine whether a building meets a specified performance objective, response quantities from the pushover analysis should be compared with the limits for each of the performance level [2,3].

3.3.4. Roof Drifts for different Performance Levels of Structure Typical values of roof drifts for the three performance levels are as follows [2]

1. Immediate Occupancy: Transient drift is about 0.7% or negligible permanent drift.
2. Life Safety: Transient drift is about 2.5% or 1% permanent drift.
3. Collapse Prevention: 5% transient or 5% permanent drift. [2,3]

3.3.5. Seismic Hazard Levels

In a probabilistic method, an earthquake level is defined with a probability of exceedance in a specified period. The following three levels are commonly defined for buildings with a design life of 50 years [3]

1. Serviceability earthquake: 50% probability of exceedance in 50 years.
2. Design basis earthquake (DBE): 10% probability of exceedance in 50 years.
3. Maximum considered earthquake (MCE): 2% probability of exceedance in 50 years. In IS: 1893-2002, the zone factor Z corresponds to MCE. The values of Z are evaluated based on a deterministic method. It cannot be directly related to the definition given above. The factor 2 in the denominator of Z is used so as to reduce the MCE zone factor to the factor for DBE. A partial load factor of 1.5 is applied to DBE in the load combination. [2,3]

3.4. PUSHOVER ANALYSIS USING SAP2000

The pushover analysis of a typical steel frame of a 3-storey steel building, designed for zone V, is carried out by using SAP2000 (Integrated software for Structural Analysis and Design). In SAP2000, the default-hinge model assumes the same deformation capacity for all columns regardless of their axial load and their weak and strong axis orientation. SAP2000 takes the average values of hinge properties instead of carrying out detailed calculation for each member. But, the hinge properties depend on the type of element, material property, shear span ratio and the axial load on the element. To account for this, in the present study, user-defined hinge properties obtained from the yield, plastic and ultimate rotation characteristics (θy, θp, θum) of typical elements estimated as per fundamental concepts along with [7,8] are adopted. Using this method, the inelastic hinge effects of the columns, beams and connections of the 3-storey building is analysed. The force-deformation behaviour of hinges such as IO, LS and CP are defined and also incorporated in SAP2000. A user defined stress-strain curves with post yield behaviour is also incorporated in SAP2000, while carrying out the analysis. From the nonlinear static analysis, the capacity curves (variation in maximum base shear and roof displacement capacities) are generated and discussed in the following sections.

3.4.1. Inelastic Member Deformations

Beams, columns and connections are considered to be the basic structural elements in steel moment-resisting frames. These elements are expected to undergo large inelastic deformations and a significant number of inelastic cycles when subject to a severe earthquake ground motion. In seismic evaluation procedure, the mechanical behaviour of these elements in the inelastic range is required to assess the performance. The element /structure will collapse when the earthquake induced inelastic deformations are larger than the element/structure can tolerate. Hence, an analytical procedure is developed to estimate the inelastic deformations of beams, columns and joints in the absence of such methods in Indian context. Tables I-III give details about the proposed moment rotation relations. In all cases, the ultimate moment is taken as 3% of ultimate stiffness of elastic slope and a residual strength ratio of 0.6 is assumed.

3.4.2. Inelastic Behaviour of Beam-Column Connections/ Joints

The inelastic joint behaviour is very much in need for the simulation of failure mechanism in steel structures. The moment-resisting frames have a large number of dissipative zones, located near the beam-column connections/joints. Hence the numerical simulations of connections are equally important as they play a key role in computation of ultimate load. Since connections frequently are located at points of maximum shear and moment, the details must assure that the performance that is assumed in design and analysis are one and the same. According to the basic response, they are generally classified as (1) flexible connections (pinned); (2) rigid connections (fixed); and (3) semi-rigid connections. The idealized approximation of the moment-rotation behaviour of most realistic connections may be simulated as semi-rigid [5,9,10]

4. The behavior factor (R)

The method of Uang 1991 is pursue in this research for the determination of R factor. The following parameters are defined based on the capacity curves obtained from static pushover analysis where the symbols are as shown in Figure 2. [18]

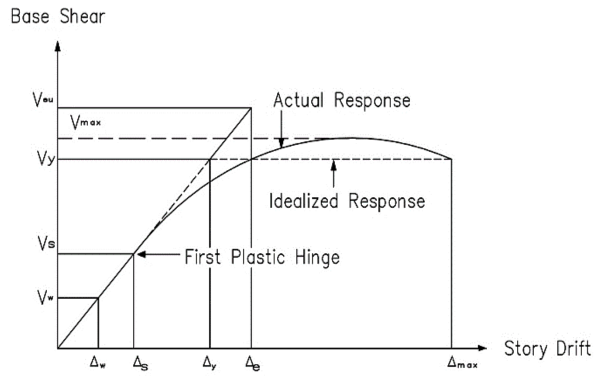

Figure 2: Seismic demand parameters

4.1. Structural Ductility (μs)

The structural ductility ratio (μ s) is defined as the ratio of maximum story drift (Δmax) to story drift at general yield (Δy).

$$\mu = \frac{\Delta u_{max}}{\Delta_y}$$

4.2. Added resistance Factor (Ω)

This factor measures the reserved strength in the structure from the formation of the first plastic hinge (Vs) to the general yield point (Vy).

$$\Omega = \frac{V_y}{V_s}$$

4.3. Ductility Based Force Reduction Factor

(Rμ) The structural ductility is responsible for dissipating hysteretic energy of earthquake which results in reducing the maximum elastic seismic force (elastic base shear, Veu) to general yield point (Vy) at failure.

Thus

$$R_u = \frac{V_{eu}}{V_y}$$

4.4. Allowable Stress Factor (γ)

The allowable stress factor (Y) is defined as the ratio of base shear (structural strength) at formation of the first plastic hinge (Δs) to the strength at allowable working design shear (Δw).

$$Y = \frac{\Delta_s}{\Delta_w}$$

4.5. Behavior Factor (R)

The behavior factor (R) used when calculating the seismic forces for building design is used to achieve the balance between resistance and energy dissipation capacity and would be defined as the value of elastic base shear (Veu) divided by the allowable working design shear (Vw).

$$R = \frac{V_{eu}}{V_w} = \left(\frac{V_{eu}}{V_y} * \frac{V_y}{V_s} * \frac{V_s}{V_w}\right) = (R_u * \Omega * \gamma)$$

5. Numerical example

5.1. Two-Story Building Details

The two-story building, 20m by 20m in plan and 6m in elevation with EBF brace to Lateral restraint is considered [4]. The bays are 5m in both the directions with five bays in the north-south (N-S) direction and five bays in the east-west (E-W) direction. The material used for columns and beams are of steel with yield stress $2.039\text{x}10^{10}$. Typical floor-to floor heights are 3.0m. Dead load and live load in the first story and roof are respectively $600\frac{kgf}{m^2}$, $250\frac{kgf}{m^2}$ and $200\frac{kgf}{m^2}$, $100\frac{kgf}{m^2}$ .The building is designed for gravity and seismic loads with a live load of. The seismic mass of the structure is due to various components of structure, including the steel framing, floor slabs, ceiling/flooring, roofing etc. In the present study the capacity curve with default hinge and user defined hinges are generated by assuming the connections as rigid. The modal properties of the first three modes are given in Table

Calculation seismic Coefficients (c) according to Iranian Code No.2800 4th-edition [16]:

*V = C *W*
*C = A*B *I / R*

A= 0.35, I= 1, R= 7, H= 6 m, W=1627139.67 kgf
Type of Ground (soil) = II
T0 = 0.1, Ts = 0.5, S = 1.5

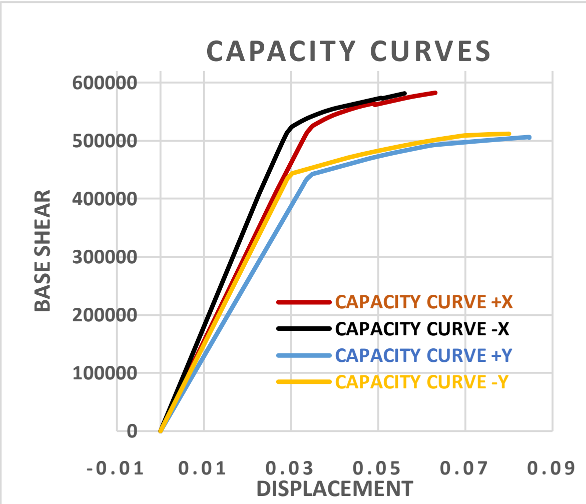

Figure 3: Capacity Curves For ALL Directions

$T = 1.25 * 0.05 * H \wedge 0.75$
$T = 1.25 * 0.05 * 6 \wedge 0.75 = 0.2396$
$T0 < T < Ts \quad \Rightarrow B = S + 1 = 1.5 + 1 = 2.5$
$C = 0.35 * 2.5 \times 1 / 7 = 0.125$

Table 1: NONLINEAR TARGECT DISPLACEMNT

DIRECTION	TARGECT DISP
+X	0.063
+Y	0.095
-X	0.056
-Y	0.08

6. RESULTS AND DISCUSSIONS

The capacity curve obtained via the pushover analysis is shown in Fig.3. The results are verified with published literature [18]. The difference in results may be due to difference in the applied lateral force and its estimation in different directions. In the present study the lateral forces have been estimated by using seismic coefficient method as per Iranian Code No.2800 4th-edition for Seismic. The zone is considered as high relative risk with type II soil. The analysis performs by representing the target displacements and the suggestion inelastic member behaviour with rigid connection. The sequence of hinge formation observed during the analysis is shown according to Table 1 that describe value of target displacement to get capacity curve for all direction. At the end of interaction severe hinges are observed in first and roof floor beams and columns and which gives an insight in structural behaviour and understanding. It may be concluded that under a severe earthquake the first floor beams and ground floor columns retrofit may not meet all the structural requirements of the life safety level. Table 2 shows the inelastic response displacements of the frame. It is observed that inelastic displacement of the structure is within collapse prevention as fig 4,5,6,7.

Table 2:INELASTIC RESPONSE DISPLACEMENTS

DIRECTION	STORY 1	STORY2 (ROF)
(+X)	0.0435	0.063
(+Y)	0.0655	0.09578
(-X)	0.0429	0.056
(-Y)	0.067	0.08

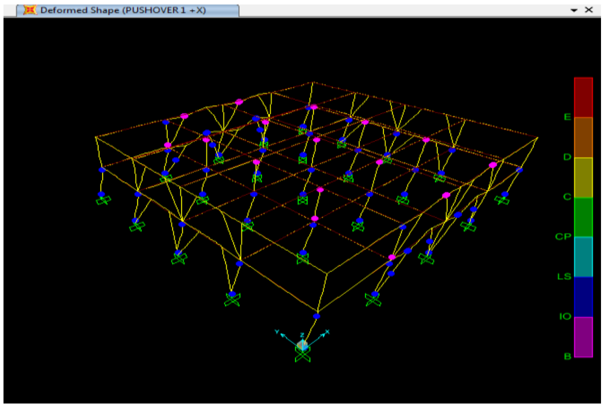

Figure 4: Capacity Curve for +X Direction

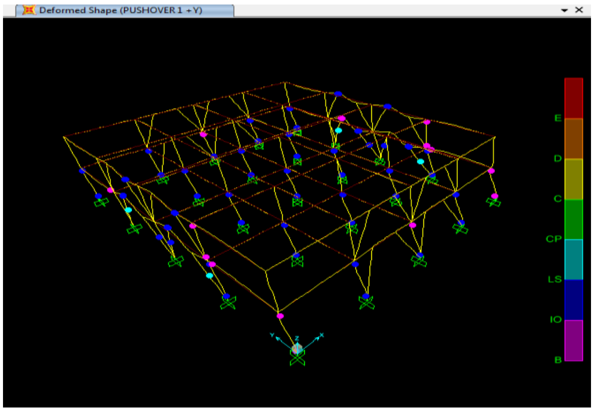

Figure 5: Capacity Curve for +Y Direction

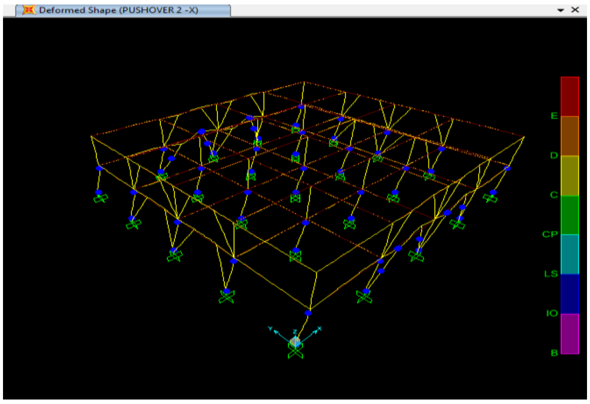

Figure 6: Capacity Curve for -X Direction

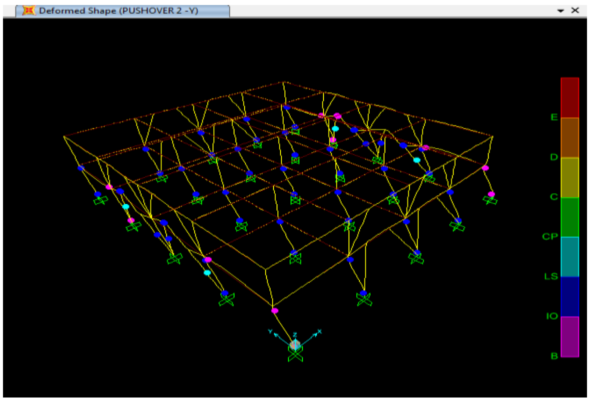

Figure 7: Capacity Curve for -Y Direction

Table 3: PARAMETERS of SHEAR BASE AND DISPLACEMENT IN DIFFRENT CONDITIONS

	Linear Analysis		First Plastic Hing		Yeld Building		Fail Building	
DIRECTION	V_d	Δ_d	$V\ s$	Δ_s	$V\ v$	Δ_y	V_u	$\Delta\ u$
+X	203392	0.01309	511404	0.03339	527351	0.03395	582397	0.063
+Y	203392	0.01572	454211	0.04015	465223	0.03595	507105	0.0951
-X	203392	0.01128	521855	0.02993	522399	0.02896	580999	0.056
-Y	203392	0.01363	457380	0.03667	457938	0.03068	511962	0.08

In table 3 (V_d, Δ_d),((V_s, Δ_s),(V_y, Δ_y), (V_u, Δ_u) are reagent base shear and displacement in various condition Respectively.

Table 4: SEISMIC PARAMETERS OF STRUCTURE

DIRECTION	K	Ω	Δu - Δy	μ	Y	Rμ	k1	k2	Cd	R
+X	2E+07	0.9623	0.0291	1.8557	2.6944	1.6466	1.6E+07	2E+06	4.81143	4.2694
+Y	1E+07	0.8955	0.0591	2.6448	2.5544	2.0712	1.3E+07	708329	6.04955	4.7374
-X	2E+07	0.9678	0.027	1.9336	2.6539	1.6933	1.8E+07	2E+06	4.96626	4.349
-Y	1E+07	0.8367	0.0493	2.6076	2.6908	2.0531	1.5E+07	1E+06	5.8711	4.6226

K1, k2 are Initial Stiffness and last Stiffness Respectively and corresponding elastic Structural stiffness and plastic Structural stiffness which can be calculated by calculating the area below the capacity curve. After the determination capacity curves of structure Using its data we can get the parameters of the seismic of building. For this purpose, we should find the coordinates of the point where the first plastic joint is formed and then using static shear force (V_d) and period of structure (T) Than to determine reduction factor (Rμ) by using Newmark and Hall methods. Finally, after calculating other seismic parameters as (Cd), the coefficient of structural behavior(R) was obtained.so total behavior factor is smallest value of (R) in the table 4.

7. Conclusions

1. Static pushover analysis is an efficient and quick method to study the nonlinear behavior of structures under seismic loads compared to the inelastic dynamic analysis which is complex and time consuming.
2. Static pushover analysis produces capacity curves (base shear versus story total drift) which can be used to calculate the seismic demand parameters

3.The smallest value obtained for the behavior factor (R) is 4.26 for +X direction. This value is on the conservative side when compared with the safer value of 4.73 for +Y direction for non-dissipative structures.

4. Based on the analysis results it is observed that inelastic displacement of the structure is within the collapse prevention level.

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Appendix A.

Symbols in table 4:

K=Stiffness inelastic structure

Ω=Added resistance

μ=Ductility factor
Y=Permitted stress factor
Rμ=Reduction factor
K1=Initial Stiffness
K2=last Stiffness
Cd=Displacement Resonant Coefficient
Δu **-** Δy = plastic deformation