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Constant Displacement Iteration Algorithm for Non-Linear Static Pushover Analysis

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Abstract: The paper presents formulation and implementation of a new static push-over analysis algorithm for the seismic rehabilitation of structure, in accordance with "NEHRP Guidelines for the Seismic Rehabilitation of Buildings".
The concept of non-linear pushover analysis is described and one problem of steel structure with piping racks are examined. Earlier work contributed pushover analysis by controlled node method. Here we go with the different method with new problem statement.
The sample structure is steel framed structure with piping racks with the G+10 story building. The final results give us the target displacement.
Keywords: Non-Linear, Static, Pushover, Analysis, Algorithm.

I. INTRODUCTION

The method of nonlinear analysis is used to test structures under gravity loading and a lateral load pattern with monotonic displacement control, and it is also known as pushover analysis.

Pushover analysis is one of the streamlined approaches for non-linear technique to evaluate seismic structural deformation for existing structures. It is carried out for the base fixed structure to examine the yield displacement and succeeding inelastic behaviour.

In several current guidelines for retrofit seismic design, pushover analysis is applied mainly to evaluate the seismic capacity of existing structures.

According to FEMA (NEHRP guidelines for the seismic rehabilitation of buildings) nonlinear analysis is a valuable method for determining the inelastic strain and deformation needs of a structure as well as for identifying design flaws in the structure. Because nonlinear analysis is primarily concerned with elastoplastic materials and large displacements, the superposition effect cannot be used in this case.

When performing a pushover study, a sequence of consecutive elastic analyses is performed and then superimposed to approximate the force-displacement curve of the entire structure.

II. NSP (NONLINEAR STATIC PROCEDURE)

Under the Nonlinear Static Procedure (NSP), a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and forces are determined.

The nonlinear load-deformation characteristics of individual components and elements of the building are modelled directly. The building's mathematical model is subjected to lateral forces or displacements that increase monotonically until it either the target displacement gets exceeded or the structural failure.

The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The target displacement may be calculated by any procedure that accounts for the effects of nonlinear response on displacement amplitude; one rational procedure is presented in FEMA 273Section 3.3.3.3.

A. Target Displacement

The target displacement δt for a building with rigid diaphragms (Section 3.2.4) at each floor level shall be estimated using an established process that accounts for the likely nonlinear response of the building. From roof displacement and base shearwe get capacity curve as shown in Fig. 1 which gives target displacement.

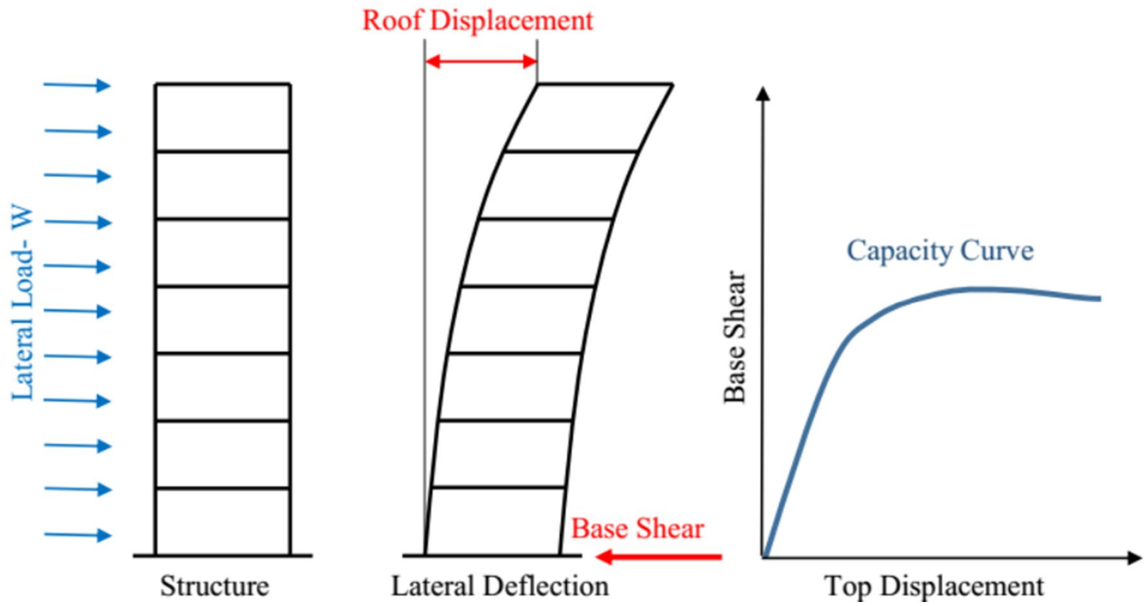

Figure.1. Behaviour of Structure

As per mentioned in FEMA273target displacement is $\partial t = C0*C1*C2*C3*Sa*(Te^2/4\prod^2)*g$ This formula is described in FEMA 273 so we can calculate values of all the coefficient factors as per given in a code.

III. PROBLEM STATEMENT

Nonlinear static analysis has two methods to do the pushover analysis from that one is the displacement coefficient method which is very long and heavy method to do. To calculate target displacement, have to go through the FEMA 273 formulas.

For existing structure, it is the one of the convenient methods to calculate the target displacement.it is very important to find at which load the structure can be collapse or get the cracks. Loads coming on the structure is different at every time so the displacement of the structure is also different at each time here, we find the solution on that problem by finding the target displacement of the structure. So, we can get the maximum loads that structure can bear.

IV. REVIEW OF DISPLACEMENT COEFFICIENT METHOD

Many compromises were required to transform research results into the FEMA 273/356 nonlinear static procedure (Krawinkler). The C1 factor as defined in FEMA 356 is smaller than research indicates, as noted in FEMA 274.

Miranda (2001) points out that the C1 term should be derived from oscillator response values and not from the R-µ-T relations that are based on these responses, to avoid statistical bias in the results.

MacRae and Tagawa (2001) note that the coefficient C2 should approach unity as the strength of the pinched system approaches the strength required for elastic response.

Song and Pincheira (2000) find that the FEMA 273 recommendations provide conservative estimates of the displacement amplification factors for degrading oscillators with periods greater than 0.3 sec on firm soils, and are unconservative at shorter periods.

Lew and Kunnath (2000) compare demands computed using the LSP, LDP, NSP, and NDP of FEMA 273 with the acceptance criteria of the document for an instrumented 7-story reinforced concrete frame building (the Holiday Inn, Van Nuys, California) subjected to ground motions having a 10% probability of exceedance in 50 years, as developed for the FEMAfunded SAC1 project. A triangular load pattern was used in the pushover analysis, and member plastic rotations were calculated from chord rotations as suggested in FEMA 273.

The beam plastic rotation demands determined in this way were similar to the mean beam plastic rotations determined by nonlinear dynamic analysis, with pushover analysis underestimating the plastic rotation demands in the columns relative to those determined by nonlinear dynamic analysis, particularly in the upper stories. As a part of the current work for the ATC-55 project, the Displacement Coefficient Method as presented in FEMA 356 was evaluated for a wide range of parametric values.

V. PROBLEM SOLUTION

The peak displacement in a nonlinear system is estimated as the peak displacement in an elastic system (Keff = Kinitial) multiplied by a series of coefficients. Of primary interest here are the coefficients C1, the ratio of the peak displacement in the inelastic system and the peak displacement in the elastic system having the same period of vibration; C2, which accounts for the effect of pinching in the loaddeformation relation; and C3, which accounts for second-order (P-Delta) effects. FEMA 356 is the primary source of documentation for the Displacement Coefficient Method (DCM).
For calculating target displacement some terms are considered which is given in FEMA 273.

A. *Control Node*

The NSP requires definition of control node in a building. These Guidelines consider the control node to be the centre of mass at the roof of a building; the top of a penthouse should not be considered as the roof. The displacement of control node is compared with the target displacement a displacement that characterizes the effects of earthquake shaking.

B. *Lateral Load Patterns*

Lateral loads shall be applied to the building in profiles that approximately bound the likely distribution of inertia forces in an earthquake. For three-dimensional analysis, the horizontal distribution should simulate the distribution of inertia forces in the plane of each floor diaphragm. For both two- and three-dimensional analysis, at least two vertical distributions of lateral load shall be considered.
The first pattern, often termed the uniform pattern, shall be based on lateral forces that are proportional to the total mass at each floor level. The second pattern, termed the modal pattern in these Guidelines, should be selected from one of the following two options:

1) A lateral load pattern represented by values of Cvx given in FEMA 273, which may be used if more than 75% of the total mass participates in the fundamental mode in the direction under consideration; or
2) A lateral load pattern proportional to the story inertia forces consistent with the story shear distribution calculated by combination of modal responses using (1) Response Spectrum Analysis of the building including a sufficient number of modes to capture 90% of the total mass, and (2) the appropriate ground motion spectrum.

C. *Period Determination*

The proposed direction's effective fundamental period Te shall be calculated using the force-displacement relationship of the NSP. The nonlinear relation between base shear and displacement of the target node shall be replaced with a bilinear relation to estimate the effective lateral stiffness, Ke, and the yield strength, Vy, of the building. The effective lateral stiffness shall be taken as the secant stiffness is calculated at a base shear of 60% of the yield strength. The effective fundamental period Te shall be calculated as:

$$Te = Ti\sqrt{\frac{Ki}{Ke}}$$

Where:
Ti = Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis
Ki = Elastic lateral stiffness of the building in the direction under consideration
Ke = Effective lateral stiffness of the building in the direction under consideration

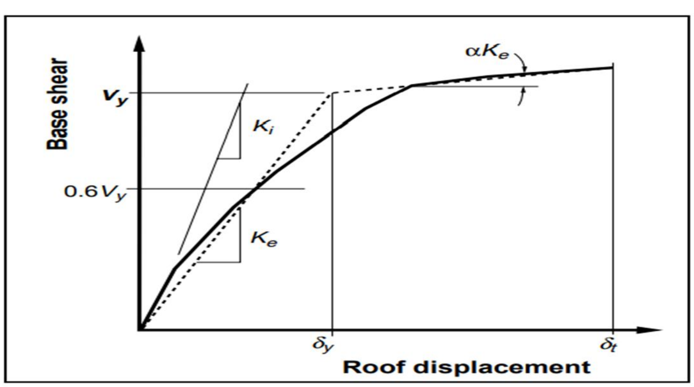

Figure.2.Calculation of Effective Stiffness, Ke

D. Analysis of three-dimensional Models

Static lateral forces shall be imposed on the three-dimensional mathematical model corresponding to the mass distribution at each floor level. Independent analysis along each principal axis of the three-dimensional mathematical model is permitted unless multidirectional evaluation is required.

E. Analysis of two-dimensional Models

Mathematical models describing the framing along each axis (axis 1 and axis 2) of the building shall be developed for two-dimensional analysis. If multidirectional excitation effects are to be considered, component deformation demands and actions shall be computed for the following cases: 100% of the target displacement along axis 1 and 30% of the target displacement along axis 2; and 30% of the target displacement along axis 1 and 100% of the target displacement along axis 2.

VI. DETAIL DESCRIPTION OF PROPOSED ALGORITHM

The assemble algorithm in the figure is the NSP solution algorithm where the ∂t is the target displacement, lateral load F, Ki is the elastic lateral stiffness of building in the direction under consideration, Ke is the effective lateral stiffness of building in the direction under consideration. QG is the gravity load which is equal to 0.9 of dead load. Initially Ke, Ki and F is on the zero and the load vector Q is equal to QG.

A. First Incremental Step

The first step of the algorithm is the incremental step which is significantly different from subsequent steps. In this step the analysis of structure is done on the only gravity load QG. Before this step all the factors are set to be zero. If the Q < F then the increment in the Q is 0.6Q which is new F. such iterative step is first step in nan linear static analysis.

B. Successive Step

In the incremental step whatever the F gets after increment in it this will be added in QG and get the value of the Q if it is greater than F then the iterative scheme is started. The total external load is used to calculate effective lateral stiffness.

C. Iterative Scheme

Now from the successive step we got the exact value of the F. Now from this step here the elastic lateral stiffness of the building by the formula of the EI/L. From the calculated stiffness matrix and the final force Q calculate the defection Δu. Now the next step is calculating the effective lateral stiffness of the building Ki which is equal to Qi/Δu.

For the calculating the target displacement first have calculate effective fundamental period of building Te which is equal to Ti*$\sqrt{\frac{\text{Ki}}{\text{Ke}}}$. Where the Ti is the fundamental period which is equal to $C^t(hn)^{0.75}$ here the hn is total height of building from support. For calculating deflection, we have to calculate Sa which is depend on the To and Ti. There is two condition of the 0<Ti<0.2To for this condition Sa=(Sxs/Bs)(0.4+3*Ti/To) and for the second condition Ti>To , Sa=Sxs/B1To. The target displacement

$$\partial t = CO*C1*C2*C3*Sa*(Te^2/4\prod^2)*g.$$

Now the modification factors C= Co*C1*C2*C3 now we are designing for the specific steel framed structure which is on the hard rock which is class A. It is designed for the maximum considered earthquake of (BSE 2). For the G+10 Building frame Co is the 1.5 which is given in the table below which is mentioned in FEMA 273.

Number of Stories	Shear Buildings		Other Buildings
	Triangular Load Pattern (1.1, 1.2, 1.3)	Uniform Load Pattern (2.1)	Any Load Pattern
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

Table 1. Modification Factor C0

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Now the next C1 is calculated from the two conditions of the effective fundamental time period and fundamental period Ti<0.1,1.5 and Ti=> To,1.0. The third modification factor is C2 is also calculated from the time period which is given in the table in FEMA 273.

Performance Level	$T_i = 0.1\text{sec}$		$T_i \geq T_0$ sec	
	Farming Type 11	Farming Type 22	Farming Type 11	Farming Type 22
Immediate occupancy level	1.0	1.0	1.0	1.0
Life safety level	1.3	1.0	1.2	1.0
Collapse prevention level	1.5	1.0	1.3	1.0
1. Structures in which more than 30% of the story shear at any level is resisted by any combination of the following components, elements, or frames: ordinary moment-resisting frames, concentrically-braced frames, frames with partially-restrained connections, tension-only braces, unreinforced masonry walls, shear-critical, piers, and spandrels of reinforced concrete or masonry				
2. All frames not assigned to Framing Type 1.				
3. Linear interpolation shall be used for intermediate values of T				

Table 2. Modification factor C2

From the above table C2 = 1.0. Moving forward the forth modification factor is C3 which is equal to 1.0+ $|\alpha|$ (R – 1)$^{3/2}$/ Te where $|\alpha|$ = Ke/Ki and R= Sa/(Vy ⁄ W)*(1/C0).

From such way we calculate value of Co, C1, C2, C3 values and calculate value of C which is helps to calculate target displacement. That displacement is more than the displacement we considered for calculate Ki then replace the target displacement ∂t to the Δu and calculate Ki again and calculate ∂t that which is repetitive process where the value of ∂t and Δu has less difference till then this process will continually repeat. Finally, we get the target displacement.

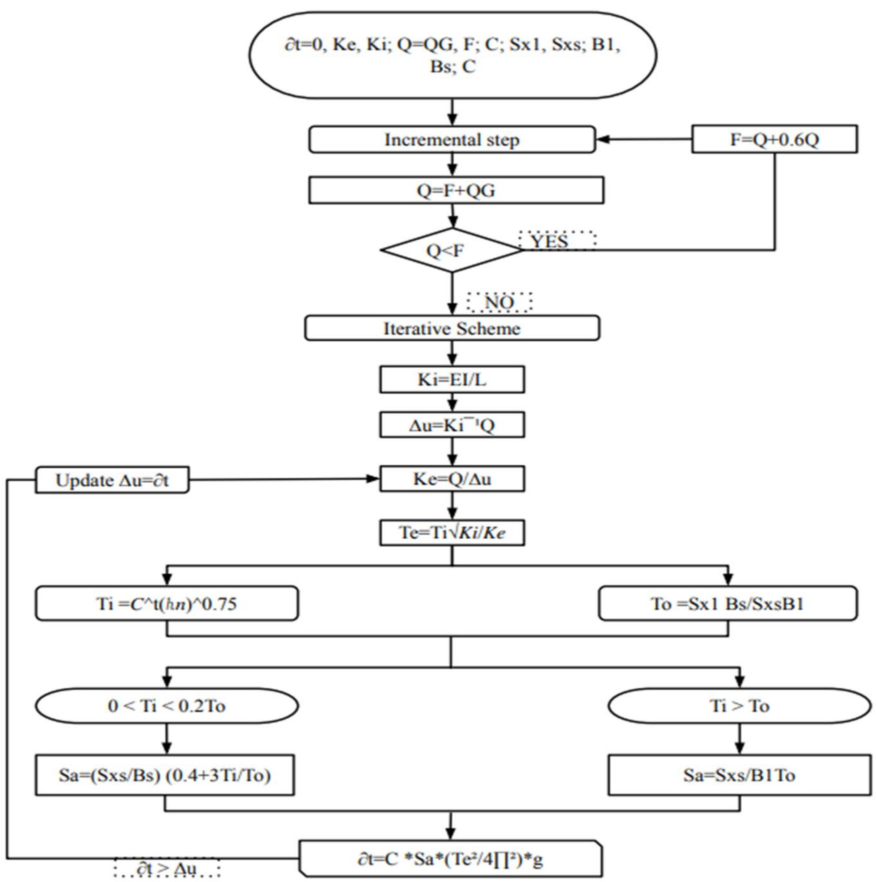

Figure.3. Solution Algorithm

VII. APPLICATION

A. Manual Calculation Of 2D Structure

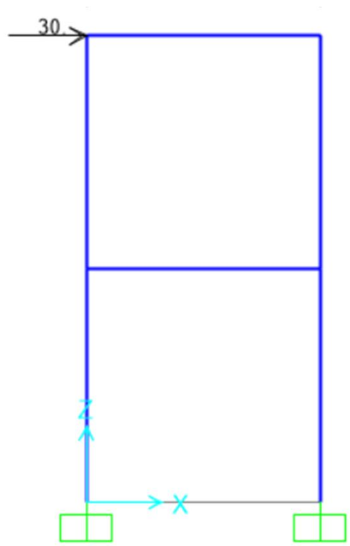

Figure.4: 2D frame

1) Pushover Analysis Of G+1 2D Framed Structure

This is the 2D framed concrete structure with column and beam dimension of 300mm*300mm with length and height 4m each. Lateral load F is applied 30Kn on the top beam of first story now the target displacement is calculated as follows. We will follow the steps of algorithm, first step

Incremental step

Value of F = 30 KN

QG = 9.81 KN

Now,

$$Q = F + QG$$

Q = 39.81 KN

39.81 KN > 30 KN

Q > F so it will be the **Successive step**

Iterative Step

Now moving forward, we have to first calculate elastic lateral stiffness and effective lateral stiffness for that we use sub-assemblage method.

Elastic lateral stiffness,

$$Ki = \sum[(\frac{12EIc}{H^3})(\frac{\sum Kbt + \sum Kbb}{4Kc + \sum Kbt + \sum Kbb})]$$

Kbt : Stiffness of beam at top of column which is equal to $\frac{Ibt}{L}$

Kbb : Stiffness of beam at bottom of column which is equal to $\frac{Ibb}{L}$

Kc : Stiffness of column which is equal to $\frac{Ic}{H}$

Formula for ground story will be changed because there is no bottom beam so it will be,

$$Ki = \sum[(\frac{12EIc}{H^3})(\frac{\sum Kc + \sum Kbt}{4Kc + \sum Kbt})]$$

Now,

$$E = 5000 * \sqrt{fck}$$
$$E = 5000 * \sqrt{25}$$
$$E = 25 \times 10^6 KN/m^2$$

$$Ic = Ib = \frac{bd^3}{12}$$
$$Ic = Ib = \frac{0.3\times0.3^3}{12}$$
$$Ic = Ib = 0.675 \times 10^{-3} m^4$$

Elastic stiffness for Ground story,

$$Kc = \frac{Ic}{H}$$
$$Kc = \frac{0.675\times10^{-3}}{4}$$
$$Kc = 0.1687 \times 10^{-3} \text{ KN/m}$$
$$Kbt = \frac{Ic}{L}$$
$$Kbt = \frac{0.675\times10^{-3}}{4}$$
$$Kbt = 0.1687 \times 10^{-3} \text{ KN/m}$$

$$Ki = \sum[(\frac{12EIc}{H^3})(\frac{\sum Kc + \sum Kbt}{4Kc + \sum Kbt})]$$
$$Ki1 = \sum[(\frac{12 \times 25 \times 10^6 \times 0.675 \times 10^{-3}}{4^3})(\frac{\sum 0.1687 \times 10^{-3} + \sum 0.1687 \times 10^{-3}}{4 \times 0.1687 \times 10^{-3} + \sum 0.1687 \times 10^{-3}})]$$
$$Ki1 = (3164.06 \times 0.3999) + (3164.06 \times 0.3999)$$
$$Ki1 = 2530.6 KN/m$$

Elastic stiffness for first story,

$$Kc = \frac{Ic}{H}$$
$$Kc = \frac{0.675\times10^{-3}}{4}$$
$$Kc = 0.1687 \times 10^{-3} \text{ KN/m}$$
$$Kbt = \frac{Ic}{L}$$
$$Kbt = \frac{0.675\times10^{-3}}{4}$$
$$Kbt = 0.1687 \times 10^{-3} \text{ KN/m}$$
$$Kbb = \frac{0.675\times10^{-3}}{4}$$
$$Kbb = 0.1687 \times 10^{-3} \text{ KN/m}$$
$$Ki = \sum[(\frac{12EIc}{H^3})(\frac{\sum Kbt + \sum Kbb}{4Kc + \sum Kbt + \sum Kbb})]$$
$$Ki2 = \sum[(\frac{12 \times25\times10^6\times0.675\times10^{-3}}{4^3})(\frac{\sum 0.1687\times10^{-3}+\sum 0.1687\times10^{-3}}{4\times0.1687\times10^{-3}+\sum 0.1687\times10^{-3}+\sum 0.1687\times10^{-3}})]$$
$$Ki2 = (3164.06 \times 0.333) + (3164.06 \times 0.333)$$
$$Ki2 = 2109.36 KN/m$$

Elastic stiffness for first story

$$Ki = Ki1 + Ki2$$
$$Ki = 2530.6 + 2109.36$$
$$Ki = 4639.96 KN/m$$

Now, Displacement Δu

$$\Delta u = \frac{Q}{Ki}$$
$$\Delta u = \frac{39.81}{4639.96}$$
$$\Delta u = 8.75 \times 10^{-3} \text{ m}$$

Now, here we go for the first iteration

$$Ke = \frac{Q}{\Delta u}$$
$$Ke = \frac{39.81}{8.75 \times 10^{-3}}$$
$$Ke = 4549.97 \text{ KN/m}$$
$$Ti = C_t \times h_n^{0.75}$$

C_t = 0.030 For concrete frame
h_n = Height of column in Ft

$$h_n = 13.123 \text{ Feet}$$
$$Ti = 0.030 \times 13.123^{0.75}$$
$$= 0.206 \text{ Sec}$$
$$Te = Ti \times \sqrt{\frac{Ki}{Ke}}$$
$$Te = 0.0206 \times \sqrt{\frac{4639.96}{4549.97}}$$
$$= 0.208 \text{ sec}$$

For calculation of response spectrum acceleration Sa we have to calculate To

$$To = \frac{Sx1 \times Bs}{Sxs \times B1}$$

Were,

$$Sxs = Fa.Ss$$
$$Sx1 = Fv.S1$$

Here we considered,
Class A – Hard rock with measured shear wave velocity, Vs > 5000Ft/sec
Also, we considered basic safety earthquake 2(BSE 2) for an maximum considered earthquake structure without exterior cladding. According to FEMA273

$$Ss = 0.89 \quad Fa = 0.8$$
$$S1 = 0.96 \quad Fv = 0.8$$
$$Sxs = 0.8 * 0.89$$
$$Sxs = 0.712$$
$$Sx1 = 0.8 * 0.96$$
$$Sx1 = 0.768$$

Viscus damping ratio β = 2%

$$Bs = 0.8$$
$$Bs = 0.8$$
$$To = \frac{0.768 * 0.8}{0.712 * 0.8}$$
$$To = 1.078 \text{ sec}$$
$$0 < Ti < To * 0.2$$
$$0 < 0.206 < 0.215$$

According to this Sa will be

$$\mathrm{Sa} = \left(\frac{\mathrm{Sxs}}{\mathrm{Bs}}\right)\left(0.4 + \frac{3\mathrm{Ti}}{\mathrm{To}}\right)$$
$$\mathrm{Sa} = \left(\frac{0.712}{0.8}\right)\left(0.4 + \frac{3*0.206}{1.078}\right)$$
$$\mathrm{Sa} = 0.866$$

According to FEMA 273 values of modification factors,

$$C_0 = 1.2$$
$$C_1 = 1$$
$$C_2 = 1$$
$$C_3 = 1$$

Now, target displacement

$$\partial_t = C_0 \times C_1 \times C_2 \times C_3 \times S_a \times \left(\frac{T_e}{2\pi}\right)^2 \times g$$
$$\partial t = 1.2 \times 1 \times 1 \times 1 \times 0.866 \times \frac{0.208 \times 0.208}{4 \times 3.14 \times 3.14} \times 9.81$$
$$\partial t = 0.0112 \text{ m}$$

Here,

$$\partial t > \Delta u$$

So, we go for the second iteration
For that replace Δu to ∂t so the value of Δu is now equal to 0.0112 m
So, the value of Ke will be

$$\mathrm{Ke} = \frac{Q}{\Delta u}$$
$$\mathrm{Ke} = \frac{39.81}{0.0112}$$
$$\mathrm{Ke} = 3554.46 \text{ KN/m}$$
$$\mathrm{Ti} = C_t \times h_n^{0.75}$$

C_t = 0.030 For concrete frame
h_n = Height of column in Ft

$$h_n = 13.123 \text{ Feet}$$
$$\mathrm{Ti} = 0.030 \times 13.123^{0.75}$$
$$\mathrm{Ti} = 0.206 \text{ Sec}$$
$$\mathrm{Te} = \mathrm{Ti} \times \sqrt{\frac{\mathrm{Ki}}{\mathrm{Ke}}}$$
$$\mathrm{Te} = 0.0206 \times \sqrt{\frac{4639.96}{3554.46}}$$
$$\mathrm{Te} = 0.235 \text{ sec}$$

For calculation of response spectrum acceleration Sa we have to calculate To

$$\mathrm{To} = \frac{\mathrm{Sx1} \times \mathrm{Bs}}{\mathrm{Sxs} \times \mathrm{B1}}$$

Were,

$$\mathrm{Sxs} = \mathrm{Fa.Ss}$$
$$\mathrm{Sx1} = \mathrm{Fv.S1}$$

Here we considered,
Class A – Hard rock with measured shear wave velocity, $\mathrm{Vs} > 5000 \mathrm{Ft/sec}$
Also, we considered basic safety earthquake 2(BSE 2) for an maximum considered earthquake structure without exterior cladding.

According to FEMA273

$$Ss = 0.89 \quad Fa = 0.8$$
$$S1 = 0.96 \quad Fv = 0.8$$
$$Sxs = 0.8 * 0.89$$
$$Sxs = 0.712$$
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Viscus damping ratio β = 2%

$$Bs = 0.8$$
$$Bs = 0.8$$
$$To = \frac{0.768*0.8}{0.712*0.8}$$
$$To = 1.078 \text{ sec}$$
$$0 < Ti < To * 0.2$$
$$0 < 0.206 < 0.215$$

According to this Sa will be

$$Sa = \left(\frac{Sxs}{Bs}\right)\left(0.4 + \frac{3Ti}{To}\right)$$
$$Sa = \left(\frac{0.712}{0.8}\right)\left(0.4 + \frac{3*0.206}{1.078}\right)$$
$$Sa = 0.866$$

According to FEMA 273 values of modification factors,

$$C_0 = 1.2$$
$$C_1 = 1$$
$$C_2 = 1$$
$$C_3 = 1$$

Now, target displacement

$$\partial_t = C_0 \times C_1 \times C_2 \times C_3 \times S_a \times \left(\frac{T_e}{2\pi}\right)^2 \times g$$
$$\partial t = 1.2 \times 1 \times 1 \times 1 \times 0.866 \times \frac{0.235 \times 0.235}{4 \times 3.14 \times 3.14} \times 9.81$$
$$\partial t = 0.014 \text{ m}$$

Here we get the target displacement is 0.14 m.

2) Pushover Analysis on FEM Based Software

The same 2D structure we will design on the FEM based software SAP 2000 and done the pushover analysis to cross check value of target displacement.

All the values which are calculated by software are

Table 3: Result in software

Item	Value
C0	0.6684
C1	1.3492
C2	1.
C3	1.
Sa	1.1
Te	0.2453
Ti	0.2231
Ki	4783.8286
Ke	3956.3177
Alpha	0.363
R	2.6781
Vy	22.1736
Weight	53.9841
Cm	1.

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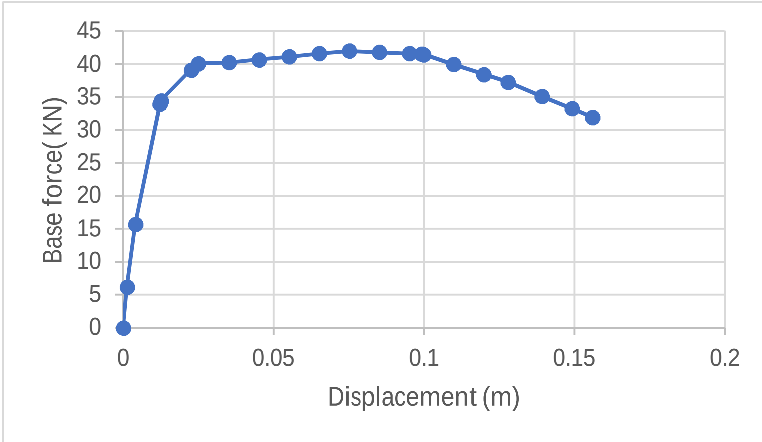

Figure.5: Pushover curve in SAP 2000

Pushover curve which shows the target displacement 0.015m. The difference between results of manual calculation and software is 6.63% which shows an algorithm successfully implements displacement coefficient method, which gives us the target displacement up to the collapse mode and thus agree with the nature of push-over strategy.

The proposed algorithm was implemented in a modern finite element program and tested on G+10 story steel frame buildings with piping racks. The sample building is located in the seismic zone IV. The building is square and symmetrical with five bays of 1.2 meter and two bays of 3 meter in width.

B. Analytical Model

FEMA 273 specifies all the pushover analysis for the building. Here we take the three-D steel frame with piping racks. Before we do the pushover analysis it is very important to check the structure is safe for all the loads and load combinations. In our case except basic loads like dead load, live load, wind load and earthquake load we have to take piping loads which are operating load, pipe empty load, friction load, pipe testing load. For those loads we take the 100 kg/m² is full load. Now the operating load is equal to full load which is also equal to testing load, pipe empty load is 60% of operating load and friction load is 30% of operating load.

The model is at seismic zone IV with zone factor Z 0.24, importance factor I 1.0, response reduction factor is R 4 it is for steel framed structure with soil type II which is medium soil and also steel is used is Fe 410 MPa. According to FEMA 273 data collection for this G+10 steel framed structure with piping rack.

Direction	No. of story	No. of DOF	∂t(mm)	No. of iteration	Max. base shear (KN)
PUX	G+10	219	116	8	9567.323
PUY	G+10	219	64	8	14143.873

Table 4. Results of pushover analysis

As per given table our algorithm is suitable for this G+10 story building so we can say that is also suitable for any kind of steel and concrete structure. We can successfully implement the given algorithm in two-dimensional and three-dimensional steel or concrete framed structure. In displacement coefficient method the structure step by step increases the displacement up to the collapse mode once the any member of the frame reaches up to collapse then the iteration step stops and we get the target displacement where the structure is safe against the collapse mode.

VIII. CONCLUSION

The presented algorithm is based upon requirements of the Nonlinear Static Procedure specified by the NEHRP Guidelines for Seismic Rehabilitation of Buildings. It is formulated for two and three-dimensional analysis, with the three-dimensional analysis initially applied and tested in this work.An algorithm successfully implements displacement coefficient method, which gives us the target displacement up to the collapse mode and thus agree with the nature of push-over strategy.

It involves the number of iterative steps which gives us the displacement up to the collapse mode. The displacement that we get is the target displacement which shows when lateral load collapse on the structure then the structure can displace up to that target displacement. Beyond the target displacement it has chances to fail the structural members or has chances to collapse the whole structure. Science existing structures instability is eliminated and also if any performance design require it can be done further after the pushover analysis.

IX. NOTATIONS

∂t	Target Displacement
F	Lateral load pattern
QG	Gravity load
Q	Load Vector
Ke	The effective lateral stiffness of building
Ki	The elastic lateral stiffness of building
Δu	The vector of nodal displacement
E	Modulus of elasticity
I	Moment of inertia
L	Length of structure
Ct	Numerical values
Hn	Height to roof level
Te	Effective fundamental period of building
Ti	Elastic fundamental period (in seconds)
To	Characteristic period of the response spectrum,
Sa	Response spectrum acceleration
Sxs	The design short-period spectral response acceleration parameter
Sx1	The design spectral response acceleration parameter at one second
G	Acceleration of gravity
R	The strength ratio
Vy	Yield strength calculated using results of NSP
W	Total dead load and anticipated live load
A	Ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force displacement relation is characterized by a bilinear relation
C0	Modification factor to relate spectral displacement and likely building roof displacement
C1	Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.
C2	Modification factor to represent the effect of hysteresis shape on the maximum displacement response
C3	Modification factor to represent increased displacements due to dynamic P-Δ effects.

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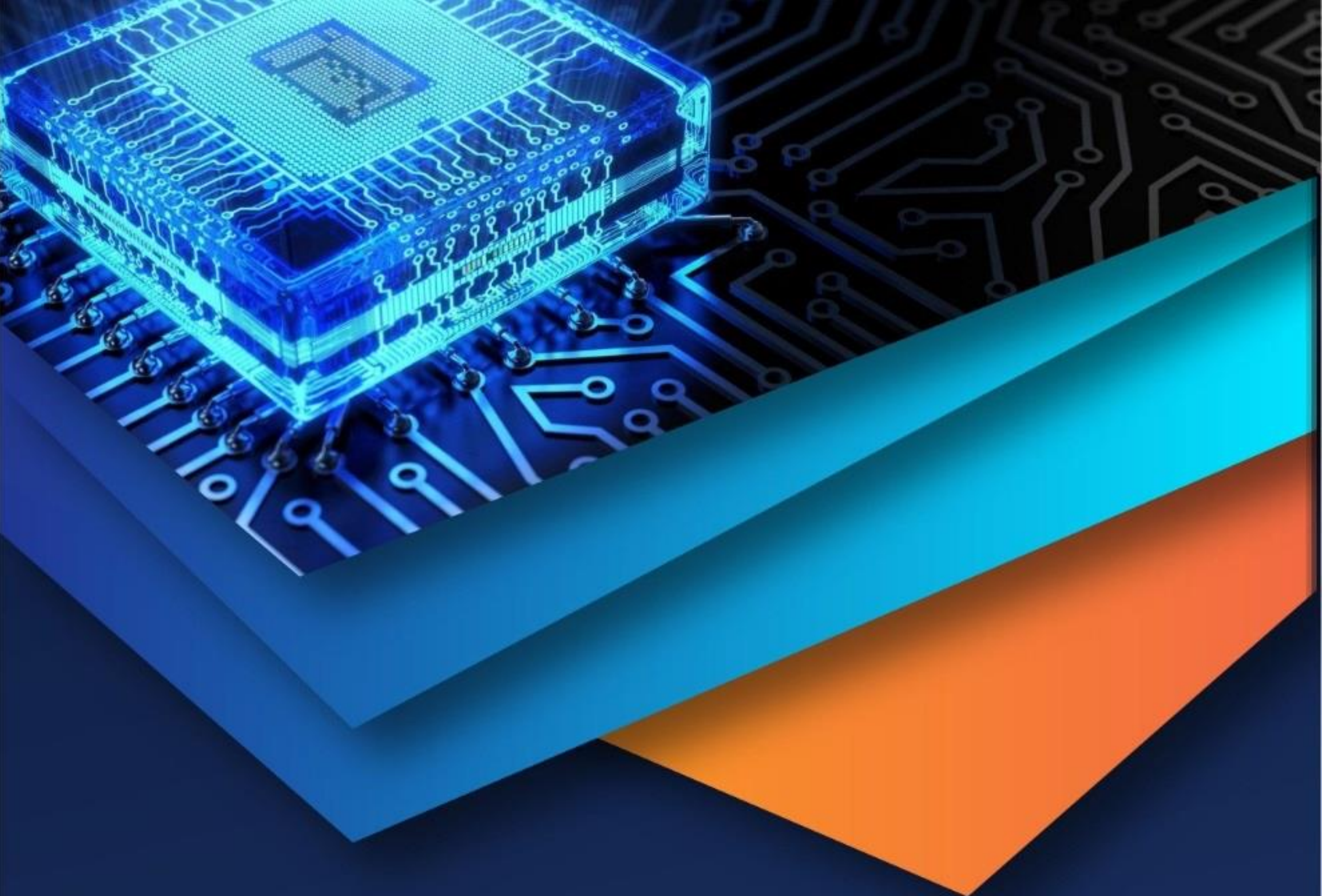

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