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"STABILITY P-DELTA ANALYSIS FOR OPTIMUM DESIGN AND ENHANCED STRUCTURAL PERFORMANCE OF COLUMNS: THE NECESSARY SHIFT FROM CONVENTIONAL EMPIRICAL METHODS TO SIMULATION BASED TECHNIQUE."

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Abstract

Purpose-The Second order effects, P-delta analysis in columns is performed by the empirical design method including the discrete action based on first principle, P-M curve, using the computer application program SACD. The purpose of this study is to provide the stability to the columns in a building to the P-delta effects and perform the subsequent ductility checks by SACD.

Design/Methodology/approach-The multi-story building structures are mostly analyzed by using the analytical static and Non-linear methods for different response characteristics. The much vulnerability in multi-story buildings is the existence of short columns at specific locations of the building structure. The response characteristics of these columns are evaluated by using non-linear techniques like Response spectrum method, to access the shear capacity of the column to the corresponding dynamic loading conditions. The P-delta analysis of columns of a building is mostly ignored due to the complexity of the method involved. Most researchers have tried to incorporate the P-delta effect in analysis using methods based on iteration process, which is conventional and time consuming. In this study, the SACD computer application algorithm has been used to evaluate the safety of column to the P-delta effects using the P-M curve based on first principle. In this study, the analysis is executed by discrete method based on the first principle, P-M curve. The application of this method is utilized after analyzing the columns by non-linear dynamic analysis, the response spectrum method.

Findings- The incorporation of P-delta effect to the analysis process enhances the time- period of the member and it is observed that the mode shapes change significantly as compared to the ones without P-delta effect. The design of columns in the SACD with proposed empirical design method generates much precise and improved response characteristics of structure and overcomes the ambiguity involved in the earlier adopted iterative techniques.

Originality/Value- The column system model has been analyzed for the different response characteristics using the computer application SACD bypassing the conventional iterative techniques.

Keywords- P-Delta Analysis, P-delta effects, Ductility design, System Model Column, SACD, System Modelling, Analysis

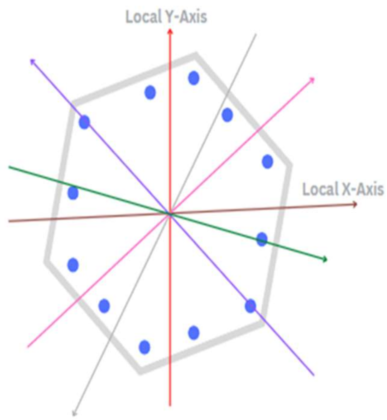
Introduction

In 3-D building complexes, the loading and stiffness is spread in a complex pattern. When the mass and stiffness of a building is spread unevenly, and the structure is unsymmetrical, then it is necessary to consider the 3-D pattern of building for the purpose of analysis. However, by considering the building in 3-D pattern for analysis, it involves many unknown variables which enhances the computation of responses and is not justified. The number of unknown variables in computation can indeed be reduced by considering some reasonable assumptions. This process is however again dependent on manual computations and hence suitable computer programs have been developed to overcome this problem [1]. The structural systems are generally analyzed for static and dynamic analysis, while the P-delta effects are not accounted for and are generally ignored by most of the researchers and structural engineers. The deformed position of a structure due to the lateral loading involves the mass maneuver, generating the second order overturning moments, P-delta effects [3]. Different techniques have been suggested to assess the second order overturning moments. Many researchers and their corresponding work and study examine the problem as geometrically non-linear and hence suggest the solutions using iteration techniques. The application of these techniques in the solution of a given problem is time consuming, inconvenient to use and sometimes these are numerically inefficient too. These iterative techniques are also not suitable for dynamic analysis of structures where the lengthening of period of vibration is caused by the p-delta effect.

In this method, the P-M Curve by first Principal approach has been used on computer application program RCDC and with certain established equations, the safety of columns to P-delta overturning moments is evaluated. This technique doesn't require the iteration process as the overall axial force at the level of a story is similar to that of a building weight above that level and remains constant during the lateral loading conditions[2, 1]. In the fundamental analytical formulation for static and dynamic analysis, the P-delta effects are included, making the effect consistent with both the linear and non-linear analytical approaches. The forces in structural members of the structure satisfy both the static and dynamic equilibrium and the additional P-delta moments are reflected in consistent with the calculated displacements [8].Generally the structures are analyzed by static and non-linear techniques and P-delta analysis is not performed and in order to omit the secondary effects of P-delta, shear walls are used at a later stage to make the structure safe to any vulnerability due to these effects, raising the cost of construction and hence eventually making the structure uneconomical[7].The P-delta effect is assessed in reinforced concrete structures with rigid joints with respect to the structural response characteristics against different loading conditions including the axial loading, moment and displacement of the structure under consideration [7, 8]. The P-delta analysis is significant in structures containing slender columns and the buildings with larger height. To assess the load carrying capacity of the columns, the axial loading in the columns need to be observed, which eventually is referred to understand the design variation in the columns of building [7].The moment in the columns must be assessed for the purpose of analyzing the trend in moment change. When a given structure is extremely flexible and is exposed to large gravity loading conditions, the P-delta effect can initiate the collapse if it is not accounted in the analytical process [4].In most of the structures, the force is generally applied with certain eccentricity along both the axes and hence there remains probability of collapse for the structural member due to the combined effects of bending and torsion [13,15].The P-delta effect exists in the beam-columns under sway effect as and when the vertical forces act on these members through the sway displacements. It has been observed that the overturning resistance capacity of the frame ranges from 2.1 to 2.6 times the required strength according to the provisions of the code, recommending the structural ductility factor of 3 [10]. Such structures with the given response reference, under the seismic design loading conditions, exhibit limited inelastic behavior, and the inelastic behavior is only exhibited for the redistribution of bending moment due to the gravity. This response characteristic of the structure indicates that the P-delta effect is negligible incase the structure exhibits ductile response characteristics and possess elastic behavioral characteristics [10] While performing the time History Analysis in the given structure through the numerical integration, due consideration must be given to recognize or locate the potential zones of plastic hinge formation, which otherwise may lead to the miscalculation for the plastic Hinge rotation, which can lead to the substantial miscalculation the lateral drift strength of the structure [10,11]. In the design of Framed Steel construction for the compression members, the effective length technique used is the first order technique including the magnification factor to compensate for the P-delta second order effect, caused by the axial and lateral loading conditions [11]. The approach involving the multiplication of first order response characteristic values by definite amplification factor values for the second order analysis of the building, followed by the definite iteration process was proposed by Avigdor et al. [12].The approach for the analytical process is ambiguous, time consuming and hence could be replaced by a computer program using the discrete method based on first principle. The approach of using the diagonal bracing to incorporate the second order effects into the first order program for plane frames. However, in this process the axial deformation of the vertical members, columns of building get affected, hence the procedure could only apply to the members of building where the axial deformation phenomenon could be neglected [12]. The techniques and approaches recommended in the direct method and the iteration process don't account for the additional moments resulting due to the deflected mode shape of the column due to the axial forces acting on it eccentrically. While modeling the geometric- stiffness matrix, the definite matrix that constitute the changes in the structure geometry, is acquired by assuming that the deflection of the vertical member, column between the two floors is straight, so as to make sure that no additional moment is induced in the members by the axial forces[12]. It must be noted that the coefficients of flexural rigidity remain unchanged by the compressive axial forces, which is justified by the said assumptions [12]. Nixon et al. [13] proposed the use of fictitious bracings system to account for the second order effects, however this technique has still its limitations including changing the vertical stiffness of the structure and the vertical component of the axial force on columns is apparent but not actually valid. The simpler methods proposed includes to account the second order effects as proportional to that of the first order effects or these structures and members

are evaluated for the second order effect based on some approximation methods and iterative techniques, resulting the ambiguity in the final response and is time consuming. The purpose of P-delta effects using the P-M curve based on first principle, which is a discrete technique for evaluation of the structure or its member like columns involves no approximations or iteration process. At the single instance in this evaluation process using the said technique, the columns can be evaluated for the second order P-delta effects, and it can be established, if the column is safe for the second order effect by ascertaining that the capacity ratio of the column is within permissible limits i.e.,≤ 1. The capacity ratio of any structural member is the ratio of its moment of resistance to its moment capacity. Deyanova et.al [20] Slender columns exhibit unique characteristics with significant bending response, resulting in larger displacements and lower ductility requirements. Their strain penetration to plastic hinge lengths ratios is smaller than the element shear span, reducing P-Delta instability risks. However, directly using existing analytical models on slender columns can overestimate deformation and energy dissipation, leading to underestimated displacement requirements, damage, and collapse probability in seismic systems. To address this, a simplified analytical approach predicts nonlinear force-deformation response and failure mechanisms in slender precast columns, accounting for rebar buckling and P-Delta effects. This approach facilitates seismic performance evaluation and cost-effective retrofit solutions for precast concrete industrial buildings [20]. Belleri et al. [21] reasserted that the weak spots of these systems lie in the connections between beams and columns, roofs and beams, and panels and structures. These connections impede the--------
rational exploitation of the strength and ductility reserves of precast elements, typically observed to function elastically until the connections give way. The failure modes experienced by these adaptable structures, whose behavior is predominantly governed by restricting second-order effects and managing displacement requirements, were associated with the loss of support from the horizontal primary elements. The connection detailing of these elements traditionally suffered from the constraints of rapid construction and overreliance on friction prior to the introduction of updated building codes. Andrea Belleri et. Al, the retrofitting solution for the vulnerable structural elements in question was carried out, but it could have been completely avoided if a second order analysis of the structural elements had been conducted during the analytical process. The vulnerabilities observed in the precast facilities, including reliance on friction capacity and other structural failures, could have been addressed and mitigated through a thorough analytical approach that considered second order effects from the beginning. This would have resulted in a more robust and resilient design, eliminating the need for retrofitting measures [22]. Mazzotta et.al. [23] The comparison and analysis of two high-rise planar structures indicate a strength-controlled pseudo elastic response, showing the effectiveness of bracing systems and outriggers in limiting drifts and second-order effects. However, these systems also lead to high floor accelerations that may harm non-structural components, occupants, and building safety. The bracing system absorbs over 80% of seismic forces, while outriggers significantly affect lateral columns by absorbing part of the earthquake-induced overturning moment as axial load [23].

The consideration of second order effect is a critical topic to work on, especially with large buildings being built across the developed and developing nations. Besides the evaluation of columns for the second order effects, special consideration is given in this research work for ductility checks of the columns. The different design techniques besides the P-M curve based on first principle for the second order effect analysis and ductility checks of columns are also discussed.



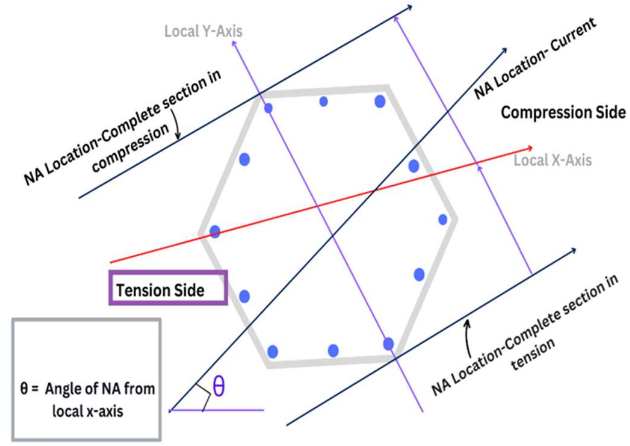
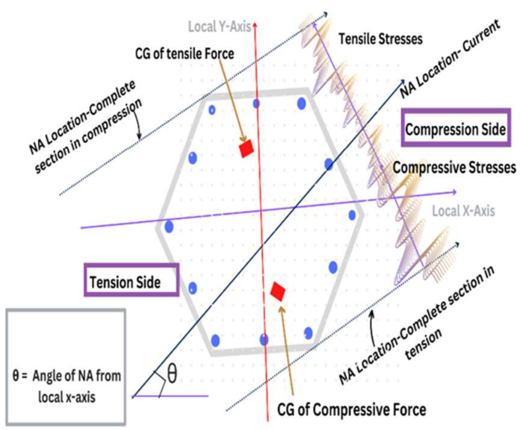


Figure.1: System Model Figure.2: Column Cross Section



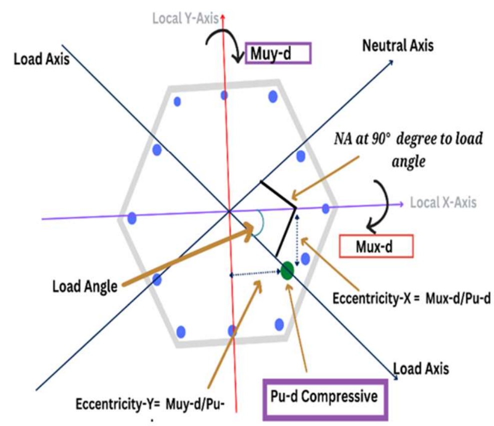


Figure.3: System Model/ Column Cross-Section

Figure.4: System Model/ Column Cross-Section

In the design of columns based on the first principle i.e., P-M curve, for any given cross section of column, there is a possibility that the neutral axis can exist in any direction. There are various arrows passing through the C.G of column, each arrow having the different direction with respect to the local x-axis as shown in fig.1.Here the angle that the neutral x-axis makes with the local x-axis is the neutral axis angle and if the P-M curves are designed for all these angles, then it is a 360-degree curve. The column cross section must be traversed from one corner end to another end so that at one edge, the entire section is in tension and as shown in fig.2. the neutral axis is located at the bottom section, then the entire section is in tension. At the same time, if the neutral axis is located at the top corner, then the entire section will be in compression. The P-M curve is generated when the neutral axis is moved parallel to itself, crossing the entire cross section. Once the section along with the neutral axis position is defined along with the reinforcement arrangement, the tensile and compressive zones of the column cross section are clearly defined as shown in fig.3. The compressive zone is where concrete is active, shown as shaded region below the neutral axis location in fig.3.The neutral axis makes angle θ with respect to the local x-axis. It can be observed that the stress-strain curve of the concrete is superimposed on the cross section and from the strain in the reinforcement bars, the stress in the reinforcement is calculated. Hence using these response characteristics, the axial capacity P for that location can be obtained and the corresponding moment for the neutral axis passing through the center of gravity and this will provide the P-M value for this neutral axis location as shown in fig 3. This technique is used in design of columns by using the Resultant action method.

In the Resultant action method, for a given cross section of column in each load combination, there are three values of design including the bending moment of the x and y axes and a vertical force, compression, or tension, in this case compressive force is considered. This compressive force acts at a certain distance from both the axes, known as eccentricity and that provides the resultant eccentricity. Hence if the C.G and the effective location where Pu acts are joined together, the load axis is obtained as shown in fig.4. The neutral axis can be assumed at right angles to it as the section is bending about that axis. The resultant eccentricity can be obtained once the resultant location has been established. Hence, we can say that instead of two bending moments acting on the cross section about x and y axes, there is a resultant moment acting about neutral axis passing through the C.G. and it will give the same effect as if two moments are acting along the cross section about x and y axes. In order to implement the Resultant action method, an assumption in the computer software is taken into consideration, which includes that the neutral axis location varies across the column cross section as mentioned earlier too, fig.5. Now a specific neutral axis location must be established where the Pu matches with the Pu design of the column under consideration as shown in graph for the P-M curve for the given neutral axis, fig.5. Once this is achieved, then the Mu- Cap at that location of the column cross-section can be obtained. The capacity ratio of the column cross-section is obtained as:

$$\text{Capacity Ratio} = \frac{\text{Mur-d acts at Neutral Axis}}{\text{Mur-Cap}} \leq 1$$

Where Mur-d = Resultant design moment in column

And Mur-Cap = Resultant Moment Capacity in column in Pu (Axial Load) design

If this condition is satisfied in the Resultant action method, then the design is safe.

Paper Contribution. The contribution of this research article is as follows:

To design a structural Element of a structure for the P-Delta effects along with linear and non-linear analytical procedure

The proposed procedure can readily be incorporated in computer programs for the elastic-plastic analysis of multistorey frames, an application which apparently has been overlooked. It can also be extended to the stability analysis of three-dimensional frames.

(III) The discrete method for P-M curve based on first principle on computer applications is adopted for the evaluation of columns, which is precise and non-iterative technique. This technique makes sure that an appropriate Capacity Ratio of structural elements is achieved.

Paper Organization. The rest of the paper is organized as follows: Section 2 describes the overall working process, along with designed algorithms. The results and discussion are illustrated in Section 3. The brief conclusion, along with future direction, is provided in Section5.

System Model

In the design based on discrete action using first principle or the P-M curve, it is assumed that the actions are discrete i.e., x and y are completely different actions. The discrete method also provides the resultant moment from design forces similar to what is given by the resultant Action Method. The resultant moment by discrete action acts about the perceived neutral axis. However, the difference is the way the P-M curve is generated by the discrete action based on the first principle. In discrete method based on first principle, the distance of C.G of the compressive force and tensile force for each neutral axis location is determined from both local x-axis and y-axis. The section capacity about those axes is computed independently and not about the axis parallel to the neutral axis location. Once the section capacity is obtained, then we have Resultant Capacity. Hence ----based on the Mx-section capacity about local x-axis (Mx-section) and My-section capacity about local y-axis (My-Section), the resultant moment capacity is obtained. It is assumed that this resultant moment might not actually act about the same axis as the neutral axis that has earlier been considered. It is in a way twisted because of the nature of the section and nonsymmetric. In this case, the effective neutral axis angle will be ϕ. The bending moment now actually acts about the effective neutral axis location. Hence in the P-M curve, the capacity is compared with the effective neutral axis and not about the actual neutral axis which is at angle the Moment Capacity about effective neutral axis can be obtained-

- Mu-Cap = $\sqrt{Mx - section^2 + My - Section^2}$

In a way, the following three equations are simultaneously satisfied,

Mur-d<=Mu-Cap	Where Mur-d = Resultant design moment in column
Mux-d <= Mx-Section	Mux-d = Design moment along x- axis in column
Muy-d <=My-Section	Muy-d = Design moment along y- axis in column

2.1 Calculations in Computer Application-Resultant Method

In this method for Resultant action on computer applications, we simply have the load angle and the corresponding resultant capacity, and the capacity ratio as shown in fig.11. It is observed from the 3D curve in the RCDC computer Applications, there is a single line that crosses through the entire cross section, indicating the capacity and the load angle corresponding to it.

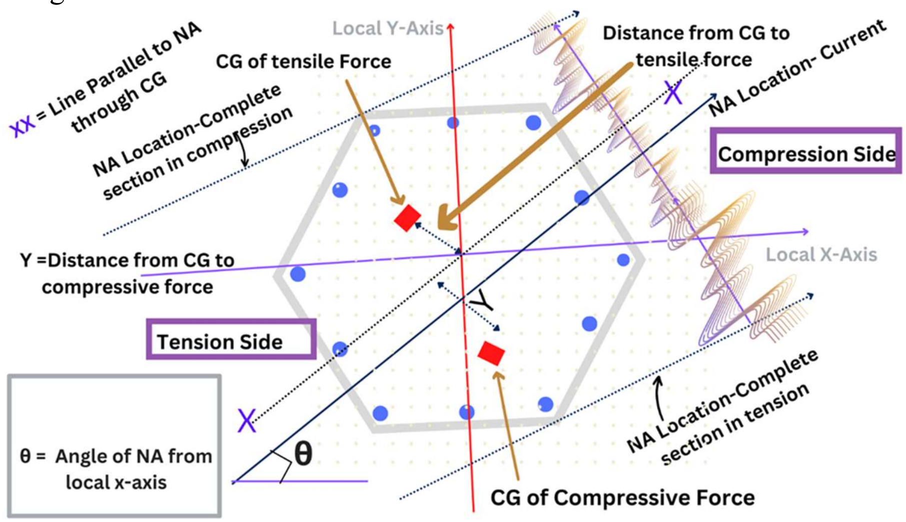


Fig.5: Resultant Action Method

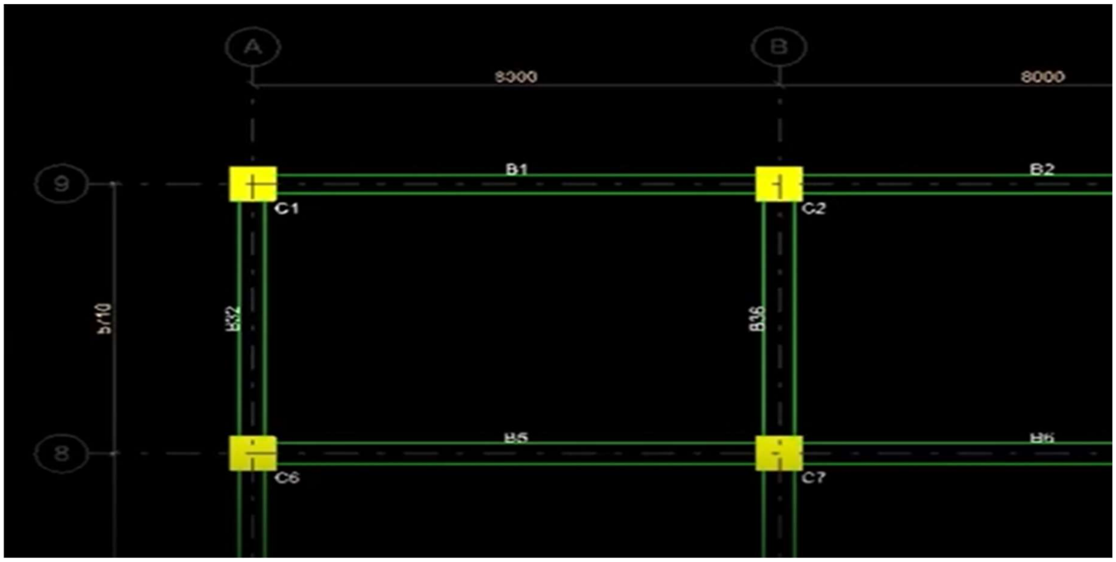


Fig.6: Plan and C7 Column Layout

2.2 Calculations in Computer Application-Discrete Method/Independent Actions

In discrete method, there are two angles which includes the load angle and the other is the effective neutral axis angle. In this case, the actual neutral axis angle θ, and the effective angle ϕ are separate. Hence the capacity in bending moment is corresponding to ϕ and not θ, as shown in fig.12. This is how a cross section of a column is designed based on the first principle. In RCDC computer application program for interaction principle or discrete action, neglecting the ductile design to compare the P-M curve for the given plan of a structure, fig.6-plan. The interaction curve of column C-7, Fig.7-interaction curve/P-M. In this case, it is observed that the actual angle of the neutral axis that is perpendicular to the load is 72.8 degree, while as the neutral axis angle itself is 71.35 degree. Hence there is a minor change or a slight eccentric angle that provides the actual neutral axis angle as shown in fig.8-interaction curve-discrete method. While we observe the critical combination, it is observed that that moment capacity is 293.77 KN-m, and the moment of resistance is 261.68 KN-m. This implies that the capacity ratio, which is the ratio of moment capacity to the moment of resistance, is 0.89. Hence the column is safe to the independent actions. The percentage of steel in case of C-7 column is 2.29.While we check the design of columns by the Resultant Method or the combined action, it is observed that the percentage of reinforcement has not changed significantly and is 2.86. Hence in general, it can be said that for a square column, both the Resultant and discrete method provide the similar design conditions, while as in case of long columns and walls, where bi-axial moment or minor axis moment is dominant, the Resultant method or the combined action provides the more economical design. In case of the Resultant Method or the combined actions, in the critical combination, the moment capacity is 294.96 KN-m, and the moment of resistance is 261.68 KN-m, and hence the capacity ratio in this case is observed to be 0.887, fig.9-interaction curve-RM. The design of column based on first principle i.e., P-M Curve is attained by using the Resultant Method. The calculations for the Resultant Method are performed by SACD through a step-by-step procedure illustrated in Figure 8:

Critical Design Forces

Axial Load Pu = 3298 KN
Moment in X-direction Mux= -77.38 KNm
Moment in X-direction, Muy = 249.50 KNm

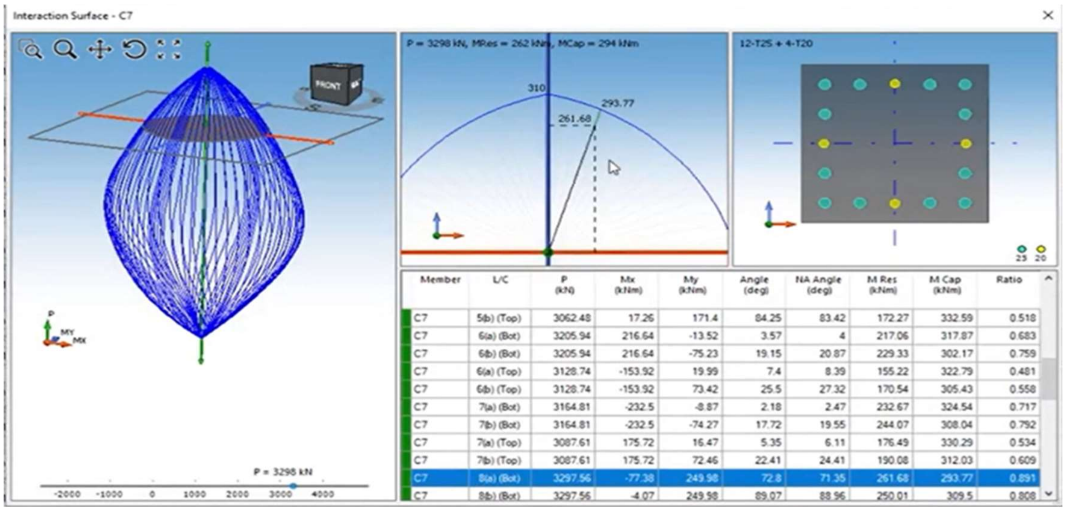


Fig.7: Interaction Curve- Discrete Method

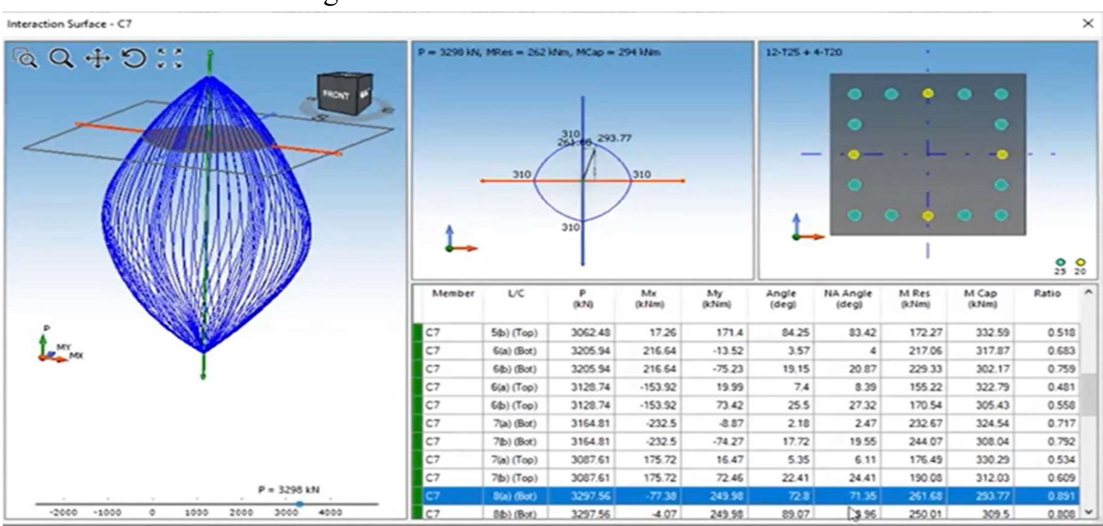


Fig.8: Interaction Curve/ PM-Curve

Resultant Moment Capacity check- Fig.8:

Moment of Resistance, MRes = 262 KN-m
Moment Capacity, MCap = 294 KN-m
Now Capacity Ratio = [MRes] ÷ [MCap] = 262/294 = 0.89 <= 1

Hence the column is safe to the critical design forces.

2.3 Calculations in Computer Application-Second Order effects

The second order effects are the additional moments and the shear that are generated within the structure due to its own deflection. These secondary order effects are classified as large PΔ and small PΔ. In case of the large Δ, the structure is acted upon by the vertical and the lateral load. The lateral load induces the lateral deflection at top of the column and this directional displacement coupled with the vertical load acting on the column generates an additional moment- PΔ. This is known as the large Δ effect. However, if in a column between two joints, it is under compression and bending at both the ends and due to its own length and the effect of lateral buckling, it may deflect horizontally in the mid depth of the section and this might induce the additional moment Pδ, known as small δ effect. These effects are important to be considered for quite a few reasons. The structure deflects under the effect of loads. The gravity loads consistently act on the structure, irrespective of the wind or earthquake loads. The columns are the most important structural component of the structure in terms of transferring the load to the soil through the foundation. Hence the columns naturally face the worst effect of the lateral loads along with the gravity loads. As we can observe that additional moments are generated due to the large PΔ and small PΔ effects, hence it is important to consider these effects. The large PΔ effect is generated in the column due to the deflection in the entire structure, while as small PΔ is comparatively local and hence may vary from one column to another, because its primarily due to the deflection of column within its own length between two floors. There are three different methods to handle the PΔ effect, which includes the Iterative technique, Equivalent Stiffness Matrix method and the Empirical Design Method. The iterative technique of analysis is the most stringent method for the analysis of column for PΔ effects. In this method, a column is acted upon by a vertical force P and a horizontal force H undergoes a deflection of Δ1 in the first iteration. Now if this Δ1 deflection is applied as a load and P and H is consistently applied on the column, the second iteration provides Δ2 and 3rd iteration provides Δ3 and so on and somewhere at 10th or at the 15th iteration, there might reach a stage where deflection in the predecessor stage iteration and the current stage of iteration is negligibly different and hence, we can say equilibrium stage is attained. This method is time consuming, uneconomical, and difficult to handle. In the equivalent Stiffness Matrix method, the effect of large Δ is captured as an effect into equivalent stiffness matrix, which then gives the ultimate forces in the state of large deflection. However, these don't match the iterative method, but these are close for most of the structures. If these methods are used in the analysis stage itself for the PΔ effect, then it is not required to be included in the design stage of column. However, if the PΔ effect is not carried out in the analysis stage, which is perfectly permitted, then we use the empirical formula from relevant codal provisions for considering the large Δ effect in the empirical design method. In the Empirical design method, for the large PΔ effect, the stability index is required to be calculated- as shown in flowchart. we have seen that in large PΔ effect is due to the entire structural behavior in terms of the deflection and not due to single column. As a result, in large PΔ effect in the empirical method, the stability index is to be calculated.

$$\text{Stability Index} = \frac{\Delta H \times Pg}{P_{\Delta H} \times (\lambda)}$$

Where ΔH = lateral deflection
Pg = Total gravity load on floor
$P_{\Delta H}$ = Total lateral load on floor or floor shear
λ = Displacement

This stability index determines whether the floor is acting in a sway condition or in a non-sway condition. If the floor is in a non-sway condition, that implies the floor is relatively rigid and will not undergo large PΔ effect, while as sway condition of the floor implies that the floor is flexible and will undergo large PΔ effect. Hence in this case the structure is required to be designed for the effect of additional moments and the shear.

2.4 Calculations in Computer Application-Second Order Effects-Empirical Design Method Calculations

In the Empirical design method, the first step is to calculate the Stability Index (Q), given by-

$$Q = \frac{\sum P_u \Delta_o}{V_{us} \times L_c}$$

Where $\sum P_u$ = Sum of vertical column loads in each story
Δ_o = Deflection due to lateral load
V_{us} = Corresponding Shear
L_c = Height of the story.

Here it can be observed that this stability index includes the effect of all the columns in each story and varies for each story. Hence it is possible that in a structure that the floors upper or lower will be subjected to a sway condition and the rest…

of the floors will be in non-sway condition. This is based on the pattern of stiffness distribution of the structure and the load distribution. Once the Stability index is obtained, we refer to the independent charts, a sway and non-sway chart (Chart fig). Referring to this chart, the local stiffness of a beam-column at a joint at bottom and top node is considered and an effective length factor is calculated. This factor is used to multiply the actual length to obtain the Actual Effective Length. Once we obtain the actual effective length, then we move a head for the moment magnification or additional moments, depending on sway or non- sway condition, there are multiple formulations in which either the moments at the joints or the moment at the mid span or all three are increased and can be used for design. Hence in this, we have a design moment and an additional moment in the empirical method.

2.5 Empirical Design Method Calculations- Implementation in SACD: The deflection and the loads data obtained in the stability index are utilized with the effective length factor in the SACD computer application, which provides the local effective length factor for each column at each joint along its major and minor axis as shown in the fig. Effective length factor. These effective length factors can be controlled by the user on SACD. Once we obtain the effective length factor, then for any given story in each direction, we obtain whether it is a sway, or a non-sway condition as shown in fig. (Global Y direction). The beam arrangement for a given column is obtained, with its corresponding stiffness value with a ψ factor equal to 3.424, which implies that the column is in a non-sway condition and the effective length factor along the minor axis is equal to 0.87, as shown in fig. (Calculation along minor axis of column). The effective length factor of the column is multiplied by the actual length, which provides the effective length which is used along with the stability index to define whether the structure is in a sway or a non-sway condition and based on it the slenderness of the column is determined as shown in fig.10 (Slenderness check). This is how the design of column is finalized by the empirical method. The basic load cases used in the analysis of the structure are first co-related and defined in the SACD. This is required because the stability index calculation is performed by the combination of the gravity and the lateral load. Once this is done, the stiffness

Calculation of each column and joint is done, and effective length factor based on standard conditions are calculated. In this study, the maximum and minimum limits for the effective length factor are provided in the SACD for standard results, as the calculated values of the effective length factor observed are less or more. It is observed that the effective length factor varies for each column, even though the stability index is performed for the entire floor.

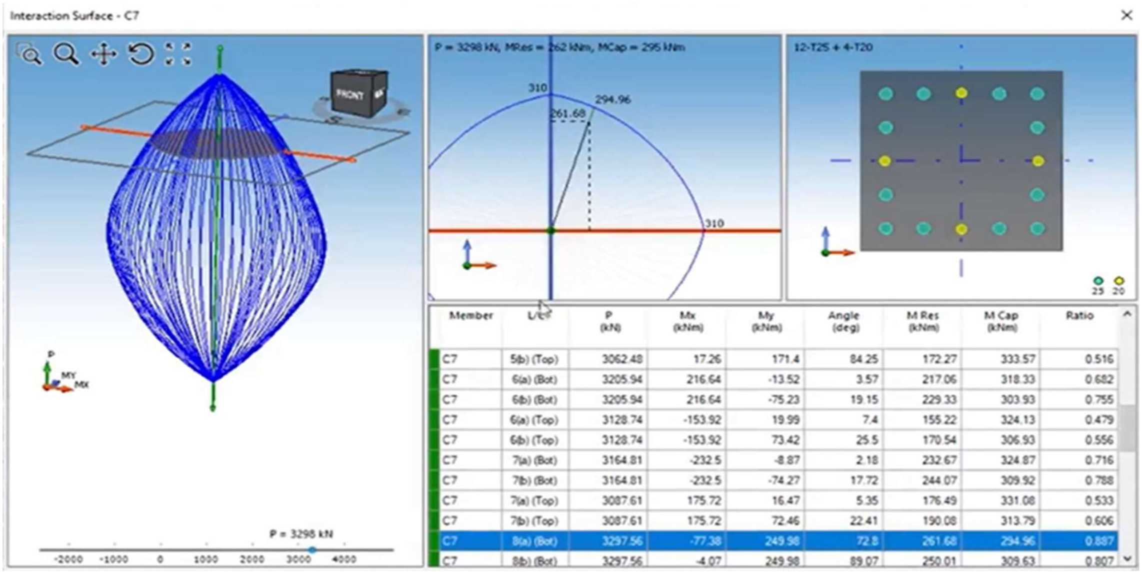


Fig.9: Interaction curve-RM.

Column Is Braced Along B
Slenderness Check along B

K	=	0.87	
r	=	202.07	mm
Klux /r	=	14.64	
M1	=	123.1	kNm
M2	=	-225.42	kNm
Min (40, 34 - 12 x (M1/M2))	=	40	
		14.64 < 40, Column not slender along B	

Fig.10: Slenderness Check

Check for Stability Index

Along D

Q = 0.018

0.018< 0.05, column shall be designed as non-sway frame (Braced)

Along B

Q = 0.033

0.033< 0.05, column shall be designed as non-sway frame (Braced)

Fig.11: Check for Stability Index

Final Critical Design Forces

Critical Case- Axial load and Biaxial Loading

Pu	=9420.12	KN
Mux	=26.63	KNm
Muy	=360.63	KNm

ϕ Pn, Max Check

Load Combination	=	[2]: 1.2(load 1: selfweight) + 1.2(load 3:SDL) + 1.6(LOAD 2:LL)	
Critical Location	=	Bottom Joint	KN
Pu	=	10751.48	KN
Mux	=	34.68	KNm
Muy	=	5.69	KNm
Pt Calculated	=	1.8	
ϕ Pn, Max	=	10758.41	KN
		Pu < *ϕ Pn, Max*	**Hence, ok**

Fig.12: Final Critical Design Check

The step-by-step method used in the calculation of effective length factor in SACD for column C-14 is depicted in this study. The Sway Stability Index is first checked for each level of the structure corresponding to the relative displacement and the story shear for the global X and Global Y directions. The column is in a non-Sway condition. Once the Sway condition is obtained, then the column stiffness at the top and bottom is obtained along with the effective length factor along the major and minor axis of the column as shown in fig.11. This stiffness calculation is again utilized by the SACD to calculate the β value at the base and the top of the column, the effective length factor obtained is used with the actual length and check whether the column is slender or non-slender, Fig.12 and if the column is slender, then the slenderness moments based on standard formulations need to be calculated.

2.5 Design of Columns- Checks for Ductility

The joint failure between the beams and columns is a critical subject to work on. The possible failure modes of the joints include the Failure of beam Rebar Anchorage, Failure of concrete in shear and formation of plastic hinge in the columns. In the earthquake lateral loading condition, the structure behaves in a bicyclic shaking condition, which may result the anchored bars in the beam anchored inside the column come out of the anchorage, known as failure of beam rebar anchorage. The joint between the beam and column may remain strong, while the column just above or below the joint might get crushed due to the excessive compressive loads, which may result in the formation of hinge in the column which is undesirable. These failures point out that if we have a strong column and weak beam, we can achieve better ductility as columns carry vertical loads from the multiple floors, while as the beam failure may be a local failure. Hence a joint or plastic hinge should form in the beam and not in the column.

2.6 Implementation in SACD- Checks for Ductility

In SACD, first it is significant to identify a joint and the connection pattern of the beams at that joint. The algorithm is performed at each building level to identify the type of beam connectivity. In SACD, the design of section for normal analysis actions is performed at the first place. The joint check for column-14 is performed and analyzed. In this check, it typically mentions the beams that are connected to the columns. The different response characteristics in the flexural strength of beam column joints includes the torsion moments, moment capacity of beam at the top and bottom and the resultant moment as shown in Table-1. The effective moment of the column is also observed to be within the permissible limits as shown in Table-1. It designs the beam locally at the joint and workout, a possible arrangement of rebar. Once we have all these characteristic responses, the shear strength capacity of the beam is added as shown in Table-2. The lower and the upper column are considered together, and their capacities are worked out to complete the ductility checks.

Results and Discussions

This section covers the overall computed results after the execution of the proposed work. The simulation environment for the proposed work is listed in the given figures and tables.

The performance of the designed model has been investigated in terms of the Resultant Actions, Discrete or Independent actions, second Order effects, Implementation in SACD and the Ductility Design. The computed results are discussed in this section.

3.1 Computed Parameters. Discrete Actions: Implementing the discrete actions on the proposed plan in the SACD computer Application, it has been observed that the actual angle of the NA normal to the load is 72.8 degree, however the NA angle itself is 71.8 degree, which implies that there is a slight eccentricity induced in the given element. In the investigation, it is observed that the column under design is safe to the second order effects or the independent actions as the Capacity Ratio is within the permissible limits.

3.2 Computed Parameters. Second Order Effects: The second order effects are implemented in the proposed work in the SACD computer application program. In this case the Stability Index of the given element is analyzed in the empirical design method. While implementing the second order effects in the SACD for the given element of the plan, the Stability Index is obtained based on the stiffness and the load distribution of the structure as shown in fig.10, where it the index value of the given element in the SACD program technique is obtained as 0.018 and 0.033 along the major and minor axis of the column and is within the permissible limits. Hence the column is designed as non-sway and braced.

Check At Beam-Column Joints Flexural Strength of Joints:

Table: 1- Moment Capacity Calculation for Beams

Beam Size	Beam angle w.r.t column Ly	Torsion Moment	Moment Capacity Beam at Top				Moment Capacity Beam at Bottom Continues----------			
(mm)	(deg)	(KNm)	Mu (KNm)	Ast. Req (sqmm)	Ast. Pro (sqmm)	Mu Cap (KNm)	Mu (KNm)	Ast. Req (sqmm)	Ast. Pro (sqmm)	Mu Cap (KNm)
300x750	0	0.24	567.07	2440.93	2533.55	585.66	0	595.71	760.08	192.64
300x750	90	0.34	679.64	3019.06	3040.26	683.61	0	595.71	760.08	192.64
300x750	180	11.33	631	2890.03	3040.26	683.61	0	595.71	760.08	192.64
300x750	270	0.11	689.84	3073.64	3460.89	760.07	0	595.71	760.08	192.64

Resultant Moment			
Top@D (KNm)	Top@B (KNm)	Bottom@D (KNm)	Bottom@B (KNm)
585.66	0	192.64	0
0	693.61	0	192.64
686.61	0	192.64	0
0	760.07	0	192.64

Table: 2- Shear Strength of joint Beams along D

Angle w.r.t column Ly (deg)	Reference Location	Width (mm)	Depth (mm)	Ast Pro top (sqmm)	Ast Pro Bottom (sqmm)
0	Right	300	750	2533.55	760.08
180	Left	300	750	3040.26	760.08

Table: 2-Shear Checks

Condition	AST-Total (sqmm)	V-Reinf (KN)	Vuy (KN)	Vj (Shear Demand) (KN)	B' (mm)	D' (mm)	Aj (sqmm)	Vn' (KN)	Vj<Vn'
Right top + Left Bottom	3293.63	1729.16	876.25	852.9	750	750	562500	5657.25	OK
Left top + Right Bottom	3800.34	1995.18	876.25	1118.93	750	750	562500	5657.25	OK

3.3 Computed Parameters. Ductility Checks: The lack of ductility leads to the failure of the joint between the beams and columns, which is critical for the structure to sustain. In the bicyclic loading condition, the columns could suffer the beam rebar anchorage. The joint between the beams and the columns might sustain, while as the column above and below it gets crushed due to the excessive compressive loading. This implies that the desired ductility in the column can be attained if we have a weak beam and a strong column, as columns sustain the vertical loads from multiple floors. In the SACD, the algorithm identifies the type of beam connectivity. In this case, the column is analyzed for different response characteristics as shown in table1 and table2. To check that the given column is safe to the shear, the effective moment of the column is evaluated by the program with all other responses including the torsion moments, moment capacity of the beam and the equivalent resultant moment. The effective moment of the element under consideration is within the permissible limits and hence is safe for the shear as shown in table 2.

Conclusion

Based on the much-needed research on the P-delta effect in analysis of the structure or its components, in this research work, a new technique on computer applications SACD has been proposed based on the Empirical Design Method-discrete action based on first principle and the PM-curve. The SACD has also been utilized in the present study to evaluate the different critical response characteristics of the structure and columns, including the Shear, Moment, and ductility checks. The scheme works on the concept of empirical equations, embedded in the computer algorithm for calculating the second order effects by simulation technique in SACD. This technique discovers the second order or the P-delta effects in the structural components by simulation and hence minimizes the much-needed efforts and time that is involved in the conventional methods based on iteration techniques. The structural component under analysis for the P-delta effects is in a non-sway condition as its corresponding stiffness value with a ψ factor equal to 3.424, which is within the permissible limits and in the slenderness check, its established that the structure is not slender. This is how the design of column is finalized in the empirical design method in SACD and is a holistic approach as compared to the conventional techniques adopted using iterative techniques. Hence in the present proposed work, the results are generated much more precise and improved in a stipulated time and overcome the ambiguity of the results involved in the Iterative techniques.

Data Availability

Data is available upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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