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INVESTIGATION ON APPLICABILITY OF SUBSTITUTE BEAM-COLUMN FRAME FOR DESIGN OF REINFORCED CONCRETE SWAY FRAMES

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ABSTRACT- *Although Ethiopian Building Code Standard, EBCS-2, is based on the Eurocode, EC-2, there are some clear differences between the two codes, most notably, with respect to the provisions for the design of slender columns in sway frames. The provision in the EBCS -2 for columns in sway frames is based on the American concrete institute, ACI; however, the former introduces the concept of the substitute frame, which is not in the ACI, to determine the stiffness of columns for the determination of the critical load, Ncr, thereby the sway moment magnification factor. This research is thus intended to investigate the suitability of the substitute frame for the intended purpose by comparing the design internal actions obtained based on the ACI and EBCS sway moment magnification provisions with the iterative P-Δ second order analysis, which is believed to be a more realistic approach, by taking different types of building frames subjected to different loading conditions. The results of the investigation reveal that, though the results are on the unsafe side, the provision in EBCS-2 yields design moments close to the iterative P-Δ second order analysis, except for the case of frames with vertical irregularities where it deviates by 6.4%.*

Keywords-*Sway frames, slender columns, substitute beam-column frame, ETABS, critical load, sway moment magnification, second-order analysis.*

I. INTRODUCTION

Columns are vertical structural members supporting axial compressive loads, with or without moments. They mainly support vertical loads transferred from floors and roof and transfer the loads to the foundations. Although columns are mainly meant for their axial compression capacity, they, in many cases, are subjected to bending moments about one or both axes of the cross section due to eccentric loading or transverse loading.

Because of the occurrence of these moments, the axial load capacity of columns, which they are intended for, decreases substantially. Interaction diagrams are usually used to describe the interaction between moment and axial load in a column, and determine the failure loads.

The maximum moment in a column could happen at the ends as in columns of sway frames or somewhere at the span of the column in between the two ends as in slender columns of nonsway frames. The analysis and design of columns in sway and nonsway frames have distinct procedures given in codes. However, the analysis and design procedures given in the Ethiopian Building Code Standard, EBCS-2[4] for the design of slender columns in sway frames need detail investigations.

II. BACKGROUND OF THE STUDY

It is well known that the Ethiopian Building Code Standard, EBCS-2- Part 1[4] is based on the previous versions of Euro code, EC-2[5]. As a result, the two Codes are very similar, with only few exceptions in some parts of the Codes. One of the sections where EBCS-2[4] deviates significantly from EC-2[5] is with respect to the provisions for the design of columns in sway frames.

EC-2[5] gives detailed simplified design provisions for slender reinforced concrete columns that may be considered as isolated columns. These are individual columns with articulation in non-sway structures, slender bracing elements, and columns with restrained ends in a non-sway structure. Corresponding provisions for the design of columns in sway frames are not provided by EC-2[5]. According to EC-2[5], such columns are to be designed using the more rigorous approach based on the results of a second order global analysis.

The EBCS-2[4] seems to be more complete in this respect, because it gives additional simplified procedures for the design of columns in sway frames. A closer look into the provisions reveals that they are based on the corresponding procedures according to the American Concrete Institute, ACI [1]. The interrelationships between the two provisions, however, are not immediately obvious because of some significant differences in the procedures such as the concept of the substitute frame adopted by EBCS-2[4] for column stiffness computation.
Therefore the design of slender reinforced concrete columns in sway frames has long been a controversial subject among practicing structural engineers with lack of consensus with regard to its suitability as a design tool or even the validity of the results. [10]
Zerayohannes G. [10] has tried to address this issue through his paper "Influence of ACI Provisions for the Design of Columns in Sway Frames on EBCS-2:1995"; however, only one frame has been used to compare the results with the results of the ACI provision. It is thus very important to make a detailed investigation on the validity of the results obtained from the provision in EBCS-2[4] by comparing them with the provision in ACI [1] and iterative P-Δ second order analysis results, by taking different sway frame models of varying story number and height for different load conditions.

III. DESIGN PROVISIONS FOR SLENDER COLUMNS IN SWAY FRAMES ACCORDING TO ACI AND EBCS CODES

3.1. Introduction

Column moments due to symmetric gravity loads do not cause appreciable sway. They are magnified when the column deflects by an amount δ relative to its original straight axis such that the moments at points along the length of the column exceed those at the ends. This is referred to as the member stability effect or P-δ effect, where the lower case refers to deflections relative to the chord joining the ends of the column. Such column end moments should not be magnified by P- δ moments.
Column moments due to lateral loads, on the other hand, cause appreciable sway. They are magnified by the P-□ moments resulting from sway deflections, δ of the beam-column joints in the frame from their original undeflected locations. This is referred to as the P-Δ effect or lateral drift effect.
Treating the P-δ and P-Δ moments separately simplifies design. The nonsway moments frequently result from a series of pattern loads. The pattern loads can lead to a moment envelope for the nonsway moments. The maximum end moments from the moment envelope are then combined with the magnified sway moments from a second-order analysis or from a sway moment-magnifier analysis.
The two codes, ACI and EBCS, seem to have similar provisions for design of slender columns in sway frames but they do have some clear differences in some aspects. One of these major differences is the introduction of the substitute beam-column frame in the EBCS for the determination of the effective column stiffness in sway frames to calculate the critical buckling loads.
A detail and closer view of the provisions of the two codes for the design of slender columns is thus necessary to investigate the acceptability of the results obtained from the substitute beam-column frame given in EBCS-2.

3.2. Moment Magnification Procedure for Sway Frames According to ACI

3.2.1. Factored Load Combinations

Three different load cases shall be considered.

Case 1: Gravity and wind loads, U = 0.75(1.4D + 1.7L) + (1.6W), (3.1)

For wind loads that did not include a directionality factor, 1.6W drops to 1.3W.

Assuming it did not: U = 1.05D + 1.275L 1.3W. (3.2)

Case 2: Gravity and EQ loads, U = 0.75(1.4D + 1.7L) + (1.0E), (3.3)

U = 1.05D + 1.275L + 1.0E, (3.4)

Case 3: Gravity loads only, U = 1.4D + 1.7L (3.5)

3.2.2. Check whether a Story is Sway or Not

According to ACI 318-08 Section 10.10.5.2, a story in a frame can be assumed nonsway if.

$$Q=\frac{\sum P_U \Delta_O}{V_U l_c} \leq 0.05 \qquad (3.6)$$

Where, Q = stability index

$\sum P_U$ = total factored axial load in the story

Δ_o = 1st order relative deflection between the top and bottom of that story due to V_u

l_c = story height

NB: P_u, Δ_o and V_u shall be obtained from elastic first-order analysis using section properties prescribed in ACI 318-08, section 10.10.4.1.

3.2.3. Check the Stability of the Structure as a Whole under Gravity Loads Only

In addition to load combinations involving lateral loads, the strength and stability of the structure as a whole under gravity loads shall be considered. ACI 318-05, Section 10.13.6 states that side-sway buckling will not be a problem if the following conditions are satisfied.

a) When δ sMs is computed from second-order elastic analysis, i.e. method (i) of section 3.2.5, the ratio of second-order lateral deflections to first-order lateral deflections for factored dead and live loads plus factored lateral loads applied to the structure shall not exceed 2.5;

b) When δ sMs is computed from equation (3.12), the value of $\boldsymbol{Q}$ computed using $\sum P_U$ for factored dead and live loads shall not exceed 0.60 which is equivalent to δ_s = 2.5;

c) When δ sMs is computed from equation (3.13), δ_s computed using $\sum P_U$ for 1.4D + 1.7L and $\sum P_U$ based on 0.40 EI/ (1 + β_d), shall be positive and shall not exceed 2.5.

In a), b) and c) above, β_d shall be taken as the ratio of the total sustained axial loads to the total axial loads, as defined in ACI 318-05, Section 10.13.6.

d) According to ACI318-08, however, the check is made simply by limiting the ratio of the total moment including second order effects to first-order moments to 1.40.

3.3. Moment Magnification Procedure for Sway Frames According to EBCS

3.3.1. Factored Load Combinations

Three different load cases shall be considered.

Case 1: Gravity and wind loads, Sd = S (1.20(G + Qvk + Qhk) (3.7)

Sd = 1.20D + 1.20L ± 1.20W (3.8)

Case 2: Gravity and earthquake loads, Sd =0.75(1.30D + 1.60L) ± 1.0E (3.9)

Sd =0.975D + 1.20L ± 1.0E (3.10)

Case 3: Gravity loads only, Sd = S (1.30G + 1.60Qvk) (3.11)

3.3.2. Check whether a Story is Sway or Not

According to Section 4.4.4.2 of EBCS-2, 1995, a story in a given frame may be classified as non-sway story if:

$$\frac{N_{sd}}{N_{CR}} \leq 0.1$$

Beam-and-column type plane frames in building structures with beams connecting each column at each story level may be classified as non-sway story if:

$$\frac{N\delta_{sd}}{HL} \leq 0.1$$

Where, in both equations,

N_{Sd}, N = total factored axial load in the story,

Ncr = story buckling load,

H = total horizontal reaction (shear) at the bottom of the story,

δ = first-order relative deflection between the top and bottom of that story due to the design loads (vertical and horizontal), plus the initial sway imperfection,

L = story height.

The displacement δ shall be determined based on stiffness values for beams and columns appropriate to Ultimate Limit State.

3.3.3. Check the Stability of the Structure as a Whole under Gravity Loads Only

EBCS-2 section 4.4.8.1(1) states that all frames shall have adequate resistance to failure in a sway mode, but it does not place any explicit limit on δ_s or the critical load ratio as in the ACI

IV. RESULTS AND DISCUSSIONS

Four different types of frames have been analyzed according to the ACI and EBCS sway moment magnification provisions for the intended purpose. The results obtained from the two provisions have been compared with iterative P-Δ analysis results for the corresponding load combinations. The analysis outputs of each frame have been summarized and discussed in the preceding sections. One can refer the appendices for detail analyses of the frames.

4.1. Five Story Regular Building

The detail analysis of this frame has been shown in chapter 4 as a design example. The results obtained based on the ACI and EBCS sway moment magnification provisions as well as the iterative P-Δ analysis are summarized in table 5.1 below. The comparison of the results is shown in the table as a percent change. Figure 5.2 also shows the results in graphical form.

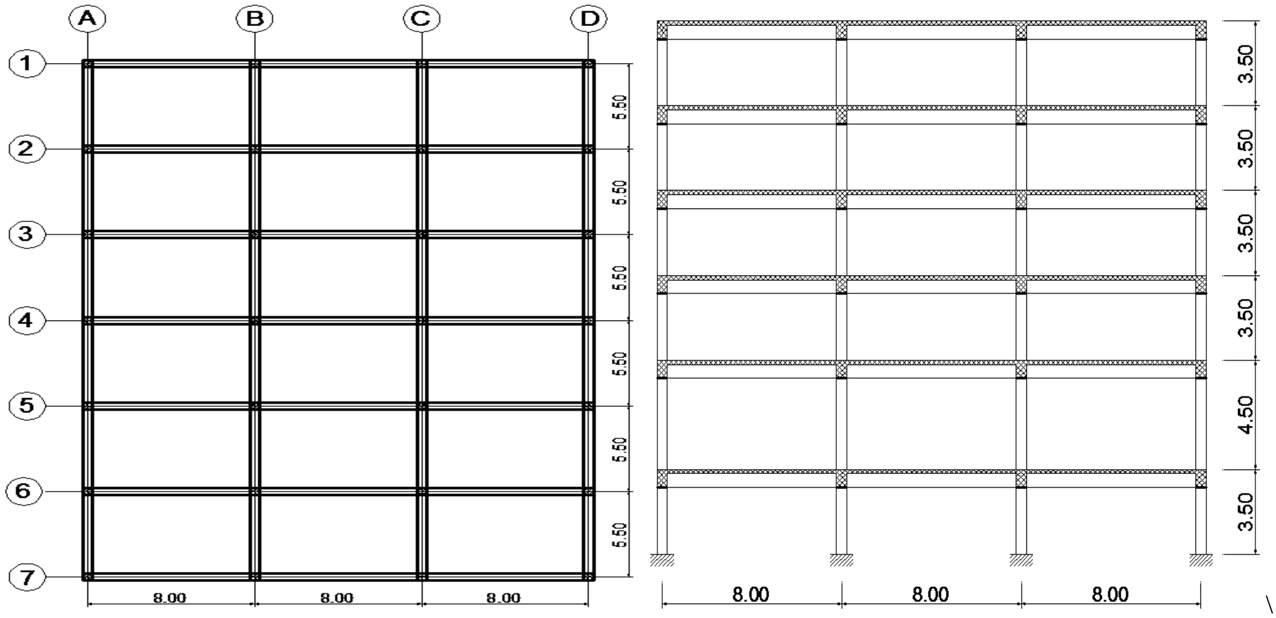


Fig. 4.1 Plan and section of a five story regular building

ACI	δs	Design Action Effects	Exterior Columns				Interior Columns			
			MM	Iterative P-Δ Outputs	ETABS P-Δ Outputs	% Change	MM	Iterative P-Δ Outputs	ETABS P-Δ Outputs	% Change
Load case 1	1.254	P (kN)	1184.26	1187.5	1180.38	-0.273	2175.68	2175.92	2175.91	-0.011
		M (kN-m)	157.4	154.12	152.06	2.128	97.57	92.61	89.03	5.356
Load case 2	1.254	P (kN)	1338.88	1351.77	1348.98	-0.954	2192.99	2194.03	2193.96	-0.047
		M (kN-m)	329.72	311.60	307.05	5.815	375.16	345.99	339.04	8.431
EBCS										
Load case 1	1.199	P (kN)	1280.18	1283.14	1282.1	-0.231	2346.62	2346.83	2346.83	-0.009
		M (kN-m)	156.76	156.04	154.22	0.461	86.89	85.92	82.65	1.129
Load case 2	1.098	P (kN)	1259.09	1270.83	1269.05	-0.924	2045.73	2046.66	2046.67	-0.045
		M (kN-m)	292.71	300.6	298.41	-2.625	326.06	338.76	335.42	-3.749

Table 4.1 Comparison of sway moment magnification and iterative P-Δ analysis outputs

Where: δ_s = Sway moment magnification factor

MM = Results of the Sway moment magnifier method provisions,

Iterative P-Δ = Results of iterative P-Δ analysis method (calculated manually)

Etabs P-Δ = Results of Etabs 9.7.4 software iterative P-Δ analysis

Load case 1 = gravity and wind loads

$$=\begin{cases} 1.05D + 1.275L \pm 1.3W.\,according\ to\ ACI \\ 1.20D + 1.20L \pm 1.3W,\,according\ to\ EBCS \end{cases}$$

Load case 2 = gravity and earthquake loads

$$=\begin{cases} 1.05D + 1.275L \pm 1.0E.\,according\ to\ ACI \\ 0.97D + 1.20L \pm 1.0E,\,according\ to\ EBCS \end{cases}$$

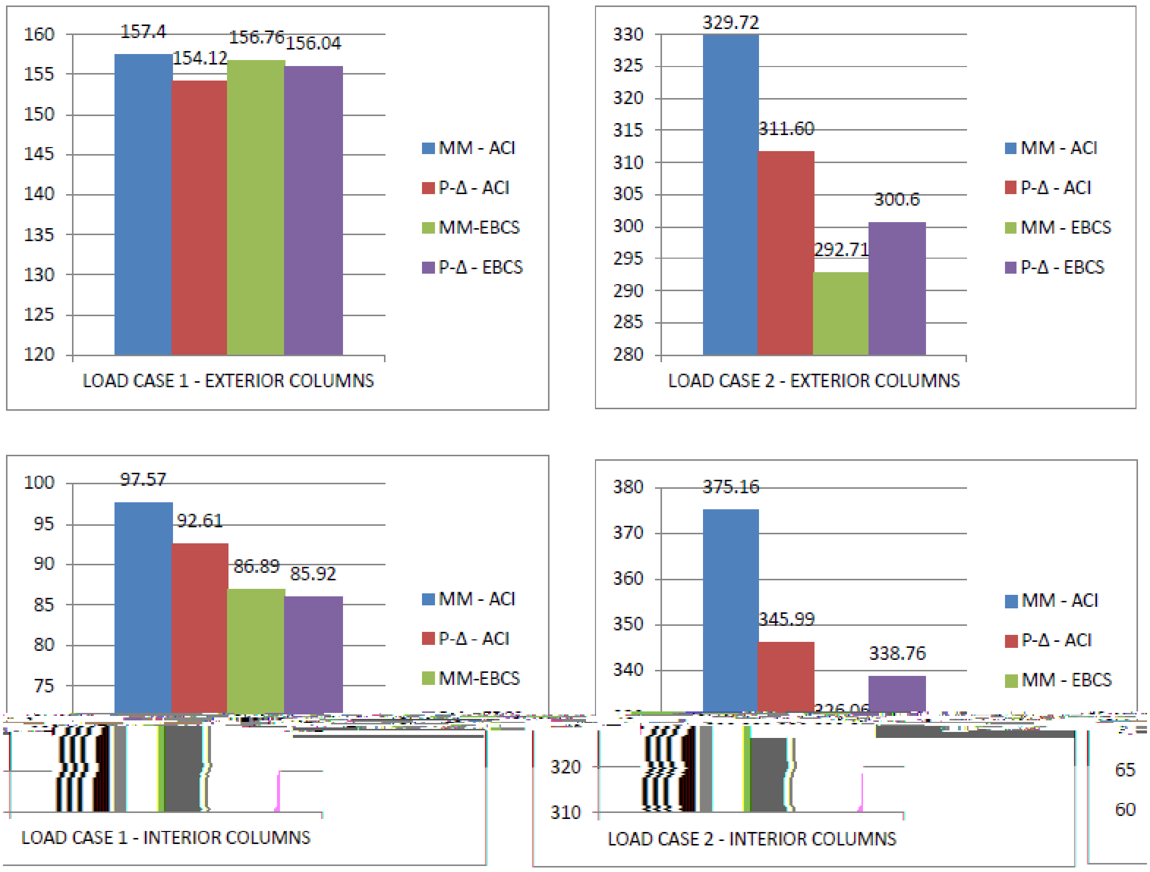


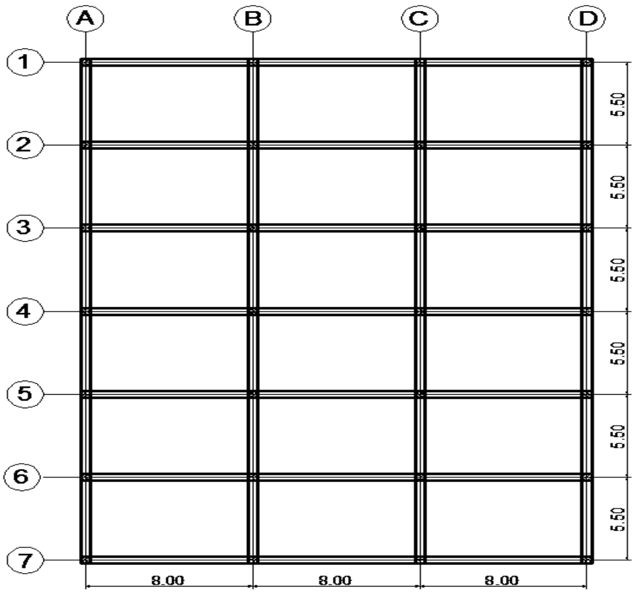
Fig. 4.2 Comparison of ACI and EBCS provision results with iterative P-Δ analysis results

From table 5.1 and fig. 5.2 one can see that:

- ✓ The sway moment magnification method provision of the EBCS gives a closer result to the iterative P-Δ analysis results than the ACI provisions; numerically:
  - For load case 1: 0.461% vs. 2.128% deviation for exterior columns, and 1.129% vs. 5.356% for interior columns.
  - For load case 2: 2.625% vs. 5.815% deviation for exterior columns, and 3.749% vs. 8.431% for interior columns
- ✓ For load case 2, however, the results of the EBCS provision are smaller than the iterative P-Δ analysis results.

4.2. Nine Story Regular Building Frame

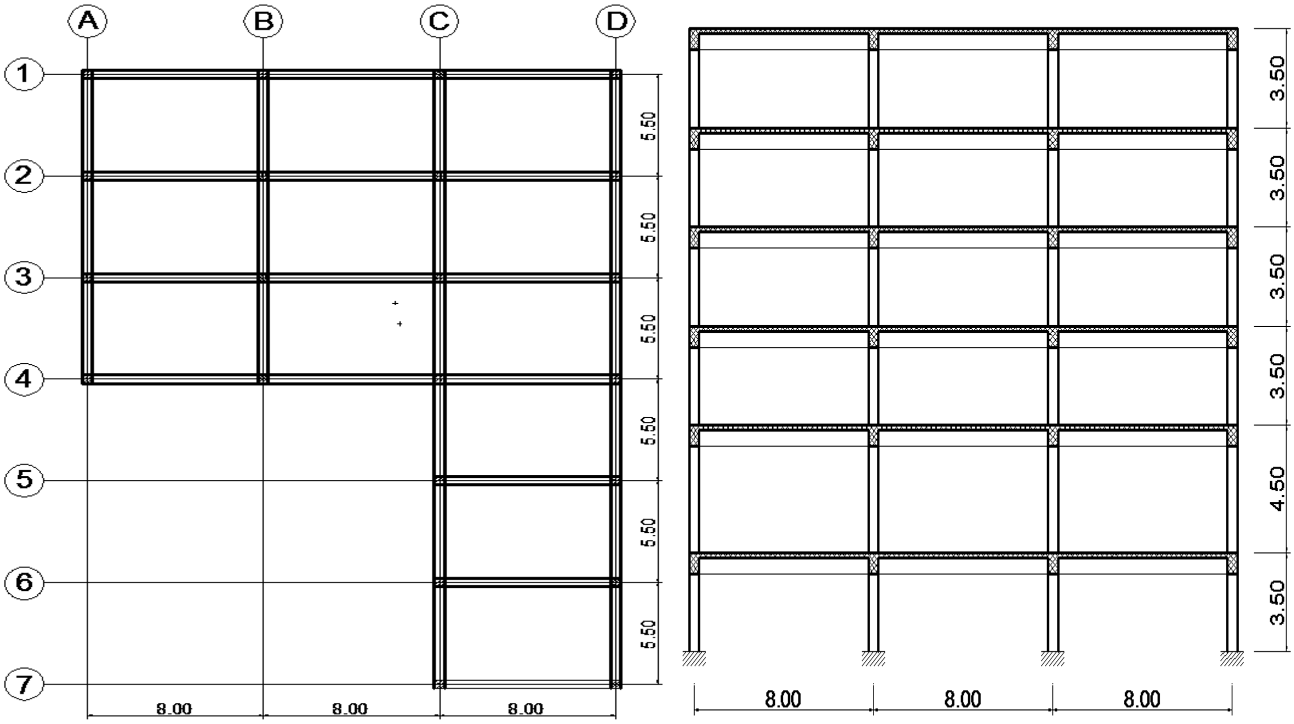
Refer appendix A for the detailed analysis of this frame. Only the results obtained based on ACI and EBCS sway moment magnification provisions are summarized and compared with the iterative P-Δ analysis results for the corresponding load combinations in table 5.2 below. The comparison of the results is shown in the table as a percent change. Figure 5.4 also shows the results in graphical form.



a) Plan b) Section

Fig. 4.3 Plan and section of a nine story regular building

ACI	**δs**	**Design Action Effects**	**Exterior Columns**				**Interior Columns**			
			MM	**Iterative P-Δ Output**	**ETABS P-Δ Output**	**% Change**	**MM**	**Iterativ e P-Δ Output**	**ETABS P-Δ Output**	**% Change**
Load case 1	1.183	P (kN)	2436.04	2447.7	2443.39	-0.476	4172.81	4173.32	4173.36	-0.012
		M (kN-m)	235.91	224.14	225.85	5.251	195.84	187.91	178.52	4.220
Load case 2	1.183	P (kN)	2829.91	2866.42	2857.14	-1.274	4200.58	4202.42	4202.39	-0.044
		M (kN-m)	466.01	439	436.47	6.153	584.48	548.04	532.23	6.649
EBCS										
Load case 1	1.114	P (kN)	2620.1	2630.74	2626.78	-0.404	4478.64	4492.69	4492.75	-0.313
		M (kN-m)	225.4	229.4	224.03	-1.744	166.56	172.05	163.37	-3.191
Load case 2	1.087	P (kN)	2667.57	2700.83	2694.41	-1.231	3919.43	3921.09	3921.19	-0.042
		M (kN-m)	419.08	431.59	425.12	-2.899	518.97	536.16	526.18	-3.206



a) Plan b) Section

Figure 4.6 also shows the results in graphical form.

ACI	δs	Design Action Effects	Exterior Columns				Interior Columns			
			MM	**Iterative P-Δ Output**	**ETABS P-Δ Output**	**% Change**	**MM**	**Iterative P-Δ Output**	**ETABS P-Δ Output**	**% Change**
Load case 1	1.157	P (kN)	1424.28	1443.01	1438.57	-1.298	2312.79	2314.57	2314.43	-0.077
		M (kN-m)	392.45	385.75	385.45	1.737	451.27	450.45	440.91	0.182
EBCS										
Load case 1	1.088	P (kN)	1345.65	1362.97	1359.95	-1.271	2158.77	2160.41	2160.4	-0.076
		M (kN-m)	367.74	382.58	378.71	-3.879	424.04	445.85	440.2	-4.892

Table 4.3 Comparison of sway moment magnification and iterative P-Δ analysis outputs

Where: Load case 1 = gravity and earthquake loads

$$=\begin{cases} 1.05D + 1.275L \pm 1.0E.\, according\ to\ ACI \\ 0.97D + 1.20L \pm 1.0E,\, according\ to\ EBCS \end{cases}$$

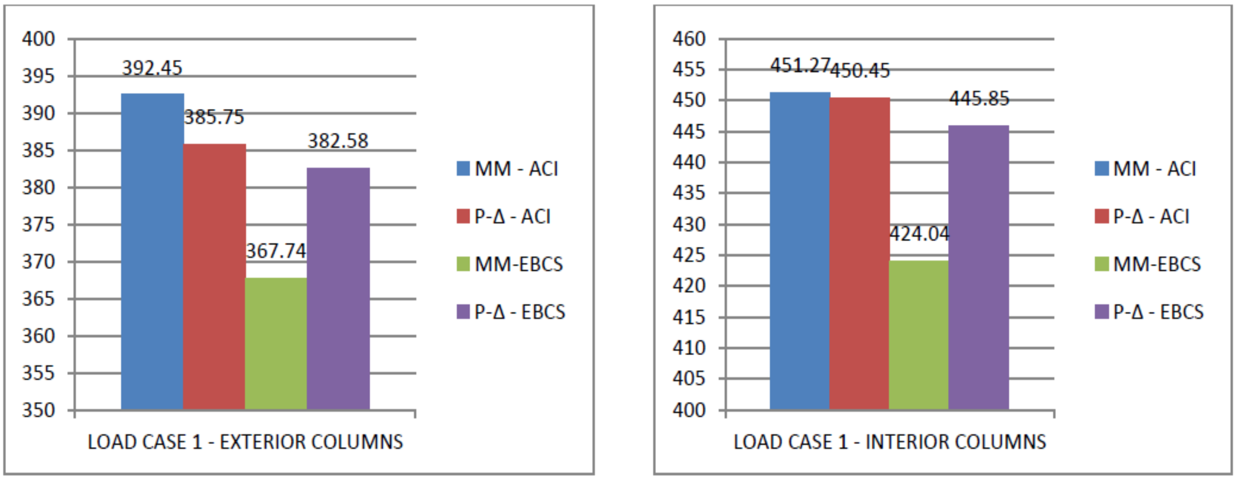


Fig. 4.6 Comparison of ACI and EBCS provision results with iterative P-Δ analysis results

From table 4.3 and fig. 4.6 one can see that:

- ✓ The sway moment magnification provision of the ACI gives a closer result to the iterative P-Δ analysis results than the EBCS provisions. Numerically, 1.737% vs. 3.879% deviation for exterior columns, and 0.182% vs. 4.892% for interior columns.
- ✓ The results of the EBCS provision are smaller than the iterative P-Δ analysis results.

V. CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

From this research the following conclusions have been made

1. Generally, the ACI provisions give more conservative results (higher design axial load and design moment) than those of the EBCS provisions reflecting the differences in load combinations used in the two codes. However, when designing structures for gravity and wind loads, the axial loads obtained from EBCS provisions are higher than those from ACI provisions.
2. In all the building frames considered, except the case with plannar irregularity, the EBCS provision gives results closer to the iterative P-Δ analysis than the ACI provision, although the results are, almost always, on the unsafe side.
3. Unlike the ACI provision, the sway moment magnification provision of the EBCS gives design moments smaller than the iterative P-Δ analysis outputs, with maximum deviation of 6.365% for the nine story frame with vertical irregularity.
4. Results of the design examples also show that the sway-moment magnification factors from EBCS provision are slightly less than the ACI sway moment magnification factors in all cases.
5. While using the sway moment magnification provision of the EBCS for designing slender columns in sway frames, one has to recall that the sway-moment magnification factor is different for different load conditions. This is because of the introduction of the substitute frame which has to be designed for the load combination under consideration to determine the effective stiffness, critical load and hence the sway moment magnification factor.
6. The provision in EBCS does not give any explicit limit as in the ACI for checking frame stability under gravity loads only; though it requires the check to be made.

5.2. Recommendations

1. When using the EBCS provision for the design of slender columns of reinforced concrete sway frames, the author recommends increasing the design moments by 3 -7%, higher values for buildings with irregularities (up to nine stories), which does not significantly affect the overall economy of the structure while ensuring safety. However, further study is needed to give exact correction factors for different frames.
2. When using the sway moment magnification method provisions of the ACI and the EBCS for the design of slender columns of sway frames with irregularities, precaution should be made since the reliability of the results decreases with irregularities.
3. The author recommends the following limits for checking the possibility of sidesway buckling under gravity loads only, which are equivalent to the limits in ACI 318-05.
   i. When □ sMs is computed from second-order elastic analysis, the ratio of second-order lateral deflections to first-order lateral deflections for factored dead and live loads plus factored lateral loads applied to the structure shall not exceed 2.5;
   ii. When □ sMs is computed using the sway moment magnification procedure, □ s computed by equation (3.33) using □ NSd for 1.3D + 1.6L and □ Ncr based on

$$EI_e = \frac{0.2E_cI_c + E_sI_s}{1+\beta_d}$$, shall be positive and shall not exceed 2.5.

   iii. The critical load ratio NSd/Ncr, NSd computed using NSd for 1.3D + 1.6L and

N_{cr} based on $EI_e = \dfrac{0.2E_cI_c + E_sI_s}{1+\beta_d}$ shall not exceed 0.60, which is equivalent to □ s = 2.5.

In **i)**, **ii)** and **iii)** above, d shall be taken as the ratio of the total sustained axialloads to the total axial loads.

iv. As in ACI318-08, the above three checks can be ignored simply by limiting the ratio of the total moment including second-order effects to first-order moments to 1.40.

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