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Stability of Concrete Moment Resisting Frames in View of Current Codes Requirements

Mahmoud A. Mahmoud, Ashraf Osman

***Abstract*—**In this study, the different approaches currently followed by design codes to assess the stability of buildings utilizing concrete moment resisting frames structural system are evaluated. For such purpose, a parametric study was performed. It involved analyzing group of concrete moment resisting frames having different slenderness ratios (height/width ratios), designed for different lateral loads to vertical loads ratios and constructed using ordinary reinforced concrete and high strength concrete for stability check and overall buckling using code approaches and computer buckling analysis. The objectives were to examine the influence of such parameters that directly linked to frames' lateral stiffness on the buildings' stability and evaluates the code approach in view of buckling analysis results. Based on this study, it was concluded that, the most susceptible buildings to instability and magnification of second order effects are buildings having high aspect ratios (height/width ratio), having low lateral to vertical loads ratio and utilizing construction materials of high strength. In addition, the study showed that the instability limits imposed by codes are mainly mathematical to ensure reliable analysis not a physical ones and that they are in general conservative. Also, it has been shown that the upper limit set by one of the codes that second order moment for structural elements should be limited to 1.4 the first order moment is not justified, instead, the overall story check is more reliable.

***Keywords*—**Buckling, lateral stability, p-delta, second order.

I. Introduction

LATERAL stability of buildings is considered as one of the major concerns that should be accounted for in designing structures, especially, those supporting heavy gravity loads or having high slenderness ratios [1]. Such worries are attributed to the fact that instability of a building due to failure of accommodating the regular vertical/lateral forces places people's life at extreme risk and have economic impact that might be intolerable.

In recent years, during many extreme events of lateral loading, several buildings have collapsed or experienced certain level of damage due to the excessive lateral deformations that arises from second order effects especially with the use of high strength materials in constructing these buildings which made most of them relatively slender. Consequently, many researchers [1]-[4] requested that this second order effects and the permanent deformations after the exposure to lateral forces need to be limited in the very beginning of the design stage. Such requests and concerns had been covered into the design codes [6]-[10] to come out with different limitations to control buildings lateral drift and ensure that they have sufficient lateral stiffness. However, most of these limitations are extremely conservative without clear link between the stated stability limits and the regular allowable limits of lateral deformations. Furthermore, some of these limits are applied on the elements level not on the global level which is totally unjustified. Therefore, in this study the stability of moment resisting concrete frames are studied. The different limits imposed by codes [6]-[10] for stability of these buildings are checked and compared with the results of buckling analysis [11] taking into consideration the second order effect and building overall buckling to determine their applicability and their level of conservatism.

Mahmoud A. Mahmoud is Structural Engineer with Dar-Alhandasah, Cairo, Egypt and is graduate student with Structural Engineering Department, Cairo University, Egypt (e-mail: Mahmoud.Abdelwahab@dargroup.com).
Ashraf Osman is Professor with Structural Engineering Department, Cairo University, Egypt (e-mail: Ashraf.Osman@dargroup.com).

II. Global Buckling and Lateral Stability

A. Buckling Phenomena

The "Buckling" term generally has the meaning of instability which leads to failure. In structural engineering, however, this term is used particularly to describe the phenomena of having a progressive increase in lateral deformation until failure due to high level of compressive stresses. The local buckling, which is happened to the individual structural elements, has been proved that it is directly related to the member stiffness. Consequently, the bending stiffness of the structural member could be expressed as, where $\boldsymbol{E}$ is the elasticity modulus of the member's material and $\boldsymbol{I}$ is the second moment of area, moment of inertia, of the member.

In 1757, mathematician Leonhard Euler derived a formula to show the maximum compression load which could initiate the buckling failure mode of an individual element; this load became known as the critical buckling load or "Euler's Load", which is equal to:

$$P_{cr} = \frac{\pi^2 EI}{L^2} \tag{1}$$

where $\boldsymbol{P_{cr}}$ is the critical buckling load "Euler's Load", $\boldsymbol{\pi}$ is a mathematical constant and $\boldsymbol{KL}$ is the effective member length. This theory can be conceptually applied for the whole building and every system has its own "Euler's load". However, the buckling failure of the whole building is not exactly the same as that of individual members. The main concern in buildings is that vertical loads approach the global critical load leading to instability or infinite magnification to lateral deformations/straining actions. Therefore, most popular design codes considered magnification factors due to second order effect as the suitable check for the overall stability of the building.

B. Overall critical load for Buildings

The lateral deformations of the building are directly related to the lateral forces. However, the vertical forces' influence is not less important. Initial movements caused by the lateral forces in the first order analysis lead to additional moments as a result of the new obtained eccentricity of the vertical loads. The new, magnified, moments are called the second order moments. However, an early assessment of the influence second order effects on the primary forces would be helpful to decide on the necessity of providing such type of analysis.

The early assessment is subject to determination of the overall critical load of the building which is usually obtained by a global buckling analysis. More simplified hand calculations, however, could be provided to give a quick and preliminary overview about the status of the building under study.

The simplified method [5] is allowing three modes of deformation to take place in the structure. These are bending and shear deformations in the vertical structure and rotation at its base. The related stiffness parameters will be indicated by ***EI*** for bending, ***GA*** for shear and ***C*** for the rotational stiffness at base. For simplicity, all three stiffness parameters were compiled in one parameter ***K*** in the pinned column model as shown in Fig. 1.

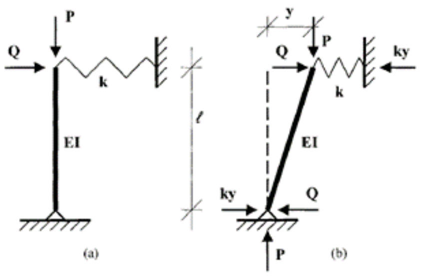

Fig. 1 Pinned column with lateral spring support; (a) initial position and (b) deflected [5]

The pinned column has a bending stiffness ***EI***, has a hinged support at the base and a spring of stiffness ***K*** at top. The stiffness ***K*** replaces all lateral displacement restraining stiffness such as bending stiffness, shear stiffness and rotational stiffness at base. The structure is subjected to vertical load ***P*** and lateral load ***Q*** at top. If the spring has an infinite stiffness, then the critical load will simply be the "Euler's load" as shown in (1). If the spring, however, has a finite stiffness, then there will be equilibrium which could be expressed as:

$$Ql + Py = kyl \tag{2}$$

Then the horizontal displacement at top equal

$$y = \frac{Ql}{kl - p} \tag{3}$$

Instability occurs when the denominator of (3) becomes zero. Hence, the critical load could be defined as:

$$P_{cr} = kl \tag{4}$$

Prior to the second order effect, the lateral displacement was a result of the lateral force ***Q*** only

$$y_o = \frac{Q}{k} \tag{5}$$

Hence,

$$ky_o = Q \tag{6}$$

From all previous equations we can express the second order deformation equation as:

$$y = \frac{kl}{kl - P} y_o = \frac{P_{cr}}{P_{cr} - P} y_o = \frac{1}{1 - \frac{P}{P_{cr}}} y_o \tag{7}$$

Accordingly, as $\boldsymbol{P=P_{cr}}$; the second order lateral displacement will become ∞. Which, mathematically, indicates instability and, physically, means getting into the buckling failure mode.

III. Lateral Stability Checks in Design Codes

Various approaches were provided in different design codes for checking the lateral stability of MRF. Examining these approaches indicated that they vary in their bases. The following section discusses these approaches which were evaluated later in this study.

A. Approach (1)

This approach was employed by the ACI 318-05 code [6]. According to clause 10.11.1of this code, the lateral stiffness of the structural elements shall be modified to consider the cracking zones due to vertical loads in lateral analysis. These modifications can be done by reducing the section moment of inertia for the different structural elements by a certain percent according to element type such as using 0.35 $\boldsymbol{I_g}$ for beams, 0.70 $\boldsymbol{I_g}$ for columns, 0.70 $\boldsymbol{I_g}$ for uncracked walls, 0.35 $\boldsymbol{I_g}$ for cracked walls and 0.25 $\boldsymbol{I_g}$ for slabs.

For sway frames; if there are sustained lateral loads which are causing permeant lateral displacement the modified moments of inertia values need to be divided by (1+$\boldsymbol{\beta_d}$) Where $\boldsymbol{\beta_d}$ is the ratio of the maximum factored sustained shear to the maximum factored story shear. However, it has been stated clearly that the above values need to be divided by (1+$\boldsymbol{\beta_d}$) prior to proceeding with the lateral stability checks mentioned in same clause, where $\boldsymbol{\beta_d}$ which used either for stability checks or for non-sway frames checks is the ratio of the maximum factored sustained axial load to the maximum factored axial load.

After applying the mentioned stiffness modifiers, the following check need to be carried out to ensure the structure stability as a whole and to determine the second order magnification factors for straining actions: According to this check, the ratio of second-order lateral deflection to first order lateral deflection for factored dead and live loads plus factored lateral load applied to the structure shall not exceed **2.5** and $\boldsymbol{\delta_s M_s}$ is computed using a second-order elastic analysis based on member modified stiffness. Alternatively, $\boldsymbol{\delta_s M_s}$ can be computed using the ratio of $\sum \boldsymbol{P_u}/\sum \boldsymbol{P_c}$ using (8) and the value of $\boldsymbol{\delta_s}$ computed using $\sum \boldsymbol{P_u}$ and $\sum \boldsymbol{P_c}$ corresponding to the factored dead and live loads shall be positive and shall not exceed **2.5**.

$$\delta_s M_s = \frac{M_s}{1 - \dfrac{\sum P_u}{0.75 \sum P_c}} \geq M_s \tag{8}$$

Another way for determining the second order magnification factor and providing the necessary stability check is computing $\boldsymbol{\delta_s M_s}$ using the value of $\boldsymbol{Q}$ where,

$$\delta_s M_s = \frac{M_s}{1 - Q} \geq M_s \tag{9}$$

And $\boldsymbol{Q}$ could be determined as

$$Q = \frac{\sum P_u \Delta_o}{V_{us} l_c} \tag{10}$$

where $\boldsymbol{\Sigma P_u}$ and $\boldsymbol{V_{us}}$ are the total factored vertical load and the horizontal story shear, respectively, in the story being evaluated, and $\boldsymbol{\Delta_o}$ is the first-order relative lateral deflection between the top and bottom of that story due to $\boldsymbol{V_{us}}$, while $\boldsymbol{l_c}$ is the floor height.

The value of $\boldsymbol{Q}$ shall not exceed **0.60**. However, $\boldsymbol{\delta_s}$ calculated in this way shall not exceed **1.5**; otherwise it shall be calculated using any of the other mentioned two alternatives. Also, it was clearly stated that if $\boldsymbol{Q}$ is **0.20** or less no need for further stability checks.

All mentioned checks are about the whole stability of the structure and are required despite the fact that different limits under lateral loads should be satisfied [9].

B. Approach (2)

This approach was employed by ACI 318-08 [7], according to clause 10.10.2.1 of this code, the limit of the acceptable magnification in structural members' moments due to second order effect has been changed to the new limit of **1.4** time the moment due to first order analysis. It's been stated, however, that the **1.4** time limit is applicable to the individual elements instead of considering the check to be applicable to the whole story as per ACI 318-05 [6] and the older versions of the American code.

Second-order effects need to be checked when slenderness isn't negligible. All members shall be designed considering second-order effects and according to clause 10.10.2, the designer can use any of the accepted design procedures, as per clauses 10.10.3 to 10.10.5, despite the previous limitations for some of these procedures which have been set in older code versions. The designer can choose between the nonlinear second-order analysis, elastic second-order analysis or moment magnification procedure up to his preference. However, in any of the three accepted ways of considering second-order effects in design, the designed members shall satisfy the following two conditions which have been stated under clauses 10.10.2.1 and 10.10.2.2:

Firstly, total moment including second-order effects in compression members, restraining beams, or other structural members shall not exceed **1.4** times the moment due to first-order effects. This check could be performed using the elastic analysis results using sections with stiffness modification to account for cracking, such as using 0.35 $\boldsymbol{I_g}$ for beams, 0.70 $\boldsymbol{I_g}$ for columns, 0.70 $\boldsymbol{I_g}$ for uncracked walls, 0.35 $\boldsymbol{I_g}$ for cracked walls and 0.25 $\boldsymbol{I_g}$ for slabs. These factors need to be divided by $(1+\boldsymbol{\beta_{ds}})$ when there is a sustained lateral load on the building.

Secondly, the second-order effects shall be considered along the length of compression members. The analytical programs may be used to evaluate magnification of the end moments only, but not along the element's length. Therefore, it shall be permitted to account for these effects using the moment magnification procedure for non-sway frames outlined in clause 10.10.6 of this code. However, the non-sway procedure emphasizes on using a modified stiffness in calculation of the magnified moments considering the axial sustained load factor $\boldsymbol{\beta_{dns}}$.

In the commentary of the same clause, R10.10.2.1, it was mentioned that the new limit has been introduced according to analytical researches on reinforced concrete showed that the probability of stability failure increases rapidly when the stability index exceeds **0.2**, which is equivalent to a secondary-to-primary moment ratio of **1.25**. However, in ASCE/SEI 7-05 [8] the maximum value of the stability coefficient $\boldsymbol{\theta}$, which is close to the ACI [6], [7] stability coefficient $\boldsymbol{Q}$, is **0.25**. This value is equivalent to a secondary-to-primary moment ratio of **1.33**. The upper limit of **1.4** on the secondary-to-primary moment ratio was chosen considering the above. Moreover, the excessive **P-Δ** effects where secondary moments are more than **23** percent of the primary moments may introduce singularities into solutions to the equations of equilibrium, indicating physical structural instability. Nevertheless, the code did not clearly mention that the new limit supersedes the old check. Instead it was conservatively mentioned that by providing the new limit on the second-order moment, it is unnecessary to retain the stability check given in 10.13.6 of the 2005 code [6] despite the fact that the 2005 code's check was a global story check not an individual structural element check.

C. Approach (3)

This approach was followed by ASCE 7-05 [8], according to clause 12.8.6 of this code, the inelastic (amplified) lateral deflection could be calculated according to

$$\delta_x = \frac{C_d \delta_{xe}}{I} \tag{11}$$

where C_d is the deflection amplification factor, δ_{xe} is the deflection determined by an elastic analysis and I is the importance factor.

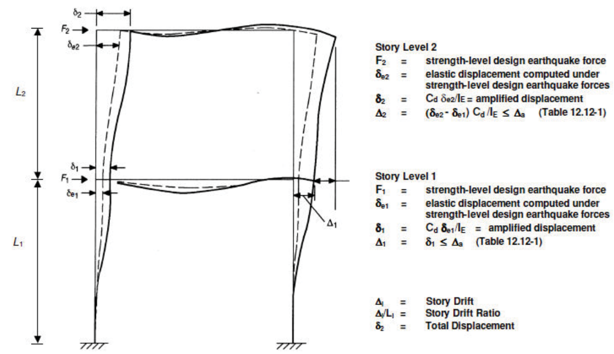

Fig. 2 Story drift determination [9]

According to clause 12.8.7, which considering the overall P-Delta check, it has been sated that the P-Delta effect could be neglected if θ, as per (12) is equal to or less than **0.10**.

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{12}$$

where P_x is the total vertical design load at and above Level x (kN); where computing P_x, no individual load factor need exceed **1.0**, Δ is the design story drift occurring simultaneously with V_x (mm), V_x is the seismic shear force acting between levels x and x-1 (kN), h_{sx} is the story height below level x (mm) and C_d is the deflection amplification factor. However, the upper limit for θ has been defined as:

$$\theta_{\max} = \frac{0.5}{\beta C_d} \leq 0.25 \tag{13}$$

where β is the ratio of shear demand to shear capacity for the story between Levels x and x − 1. This ratio is allowed to be conservatively taken as **1.0**. Where the stability factor (θ) is greater than **0.10** but less than or equal to θ_{max}, the incremental factor related to P-delta effects on displacements and member forces shall be determined by analysis. Alternatively, it is permitted to multiply displacements and member forces by 1.0/ (1 − θ). Where θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned. Where the P-delta effect is included in an automated analysis, shall still be satisfied, however, the value of θ computed from using the results of the P-delta analysis is permitted to be divided by (1 + θ) before checking.

D. Approach (4)

This approach was followed by ASCE 7-10 [9], according to this code all previously mentioned equations/limits in ASCE 7-05 [8] are still valid except the following change to by adding the importance factor I_e to the numerator as shown in (14):

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \tag{14}$$

E. Approach (5)

The Euro code [10] has adopted the overall story check similar to ASCE 7-05 and ASCE 7-10 [8], [9]. However, the limit has been slightly moved to be **0.30** in EC8 instead of **0.25** in ASCE.

$$\theta = \frac{P_{tot} d_r}{V_{tot} h} \leq 0.30 \tag{15}$$

where P_{tot} is the total cumulative gravity load at the story considered in the seismic design situation; d_r is the design inter-story drift, evaluated as the difference of the average lateral displacements ds at the top and bottom of the story under consideration, V_{tot} is the seismic story shear and h is the inter-story height.

IV. Research Methodology

In order to evaluate the different approaches stated by design codes for checking the stability of moment resisting frames, 48 hypothetical structures were developed. The buildings were categorized into four sets. The buildings were modeled and analyzed using the computer program for structural analysis and design (ETABS-2015) [11]. All buildings adopted in this research are symmetric in plan, and as such, avoid any torsional effect.

The first set included 12 buildings having different aspect ratios (height/width) ranging from 1:1 to 1:8, utilizing concrete strength of 35 MPa and reinforcement having yield stress of 420 MPa as construction material. This set have been detailed for high seismic zone having S_1 = 0.29 and S_s = 0.74.

The second set includes 12 buildings having different aspect ratios (height/width) ranging from 1:1 to 1:8, utilizing concrete strength of 35 MPa and reinforcement having yield stress of 420 MPa as construction material. This set have been detailed for low seismic zone having S_1 = 0.10 and S_s = 0.25.

The third set includes 12 buildings having different aspect ratios (height/width) ranging from 1:1 to 1:8, utilizing concrete strength of 70 MPa and reinforcement having yield stress of 550 MPa as construction material. This set have been detailed for high seismic zone having S_1 = 0.29 and S_s = 0.74.

The fourth set includes 12 buildings having different aspect ratios (height/width) ranging from 1:1 to 1:8, utilizing concrete strength of 70 MPa and reinforcement having yield stress of 550 MPa as construction material. This set have been detailed for low seismic zone having S_1 = 0.10 and S_s = 0.25. Table I lists these sets and the used loading criteria [9].

It is worth to mention here that sets 3 and 4 are different from sets 1 and 2 in the used material strength. These sets have been developed to investigate the effect of using high strength materials on the lateral stability of the buildings.

All these buildings were chosen to be intermediate moment resisting frames (IMRF) and the buildings were designed in accordance with ASCE & ACI design codes [6-9] requirements with regard to satisfying story drift and elements strength. All buildings in every set were designed according to the same loading criteria for vertical and lateral loads considering an approach of minimizing the dimensions of the structural elements, while satisfying strength and inter-story drift requirements.

Upon finalizing the design of the buildings, they were checked for stability requirements using the different approach previously discussed for sake of comparison and were analyzed for overall buckling [11].

TABLE I
DIFFERENT ANALYSIS SETS, MODELS DIMENSIONS AND ASPECT RATIOS

Analysis Set	Group	Aspect Ratio	Model I.D.	Base Width/ Length (m)	Overall Bldg. Height (m)	Num. of Floors	Seismic Zone		f_c' / f_y (mPa)
							S_1	S_s	
1	**A**	1:1	A_{11}	40	40	10	0.29	0.74	35/420
	B	1:2	B_{11}	40	80	20			
			B_{12}	30	60	15			
			B_{13}	20	40	10			
	C	1:3	C_{11}	40	120	30			
			C_{12}	20	60	15			
	D	1:4	D_{11}	30	120	30			
			D_{12}	20	80	20			
			D_{13}	15	60	15			
	E	1:5	E_{11}	20	100	25			
	F	1:6	F_{11}	20	120	30			
	G	1:8	G_{11}	15	120	30			
2	**A**	1:1	A_{21}	40	40	10	0.1	0.25	35/420
	B	1:2	B_{21}	40	80	20			
			B_{22}	30	60	15			
			B_{23}	20	40	10			
	C	1:3	C_{21}	40	120	30			
			C_{22}	20	60	15			
	D	1:4	D_{21}	30	120	30			
			D_{22}	20	80	20			
			D_{23}	15	60	15			
	E	1:5	E_{21}	20	100	25			
	F	1:6	F_{21}	20	120	30			
	G	1:8	G_{21}	15	120	30			
3	**A**	1:1	A_{31}	40	40	10	0.29	0.74	70/550
	B	1:2	B_{31}	40	80	20			
			B_{32}	30	60	15			
			B_{33}	20	40	10			
	C	1:3	C_{31}	40	120	30			
			C_{32}	20	60	15			
	D	1:4	D_{31}	30	120	30			
			D_{32}	20	80	20			
			D_{33}	15	60	15			
	E	1:5	E_{31}	20	100	25			
	F	1:6	F_{31}	20	120	30			
	G	1:8	G_{31}	15	120	30			
4	**A**	1:1	A_{41}	40	40	10	0.1	0.25	70/550
	B	1:2	B_{41}	40	80	20			
			B_{42}	30	60	15			
			B_{43}	20	40	10			
	C	1:3	C_{41}	40	120	30			
			C_{42}	20	60	15			
	D	1:4	D_{41}	30	120	30			
			D_{42}	20	80	20			
			D_{43}	15	60	15			
	E	1:5	E_{41}	20	100	25			
	F	1:6	F_{41}	20	120	30			
	G	1:8	G_{41}	15	120	30			

Typical/Roof Floor S.D.L. = 2.0/3.5 kN/m^2
Typical/Roof Floor L.L. = 4.8 kN/m^2
Soil type for all sets “B”
Risk Category “III”
Importance Factor “1.25”
S.D.C. = “C”

V. Loading Criteria and Buildings Aspects

A. Dimensions and Aspect Ratios of the Buildings

For analyzed buildings, the heights of the floors were set to 4m and the maximum number of floors in any of these structures has been limited to 30 stories which represents the reasonable upper limit for using concrete moment resisting frames as a sole lateral resisting system for buildings, all bays width was taken to be 5m in span. Table I shows the different groups of the developed hypothetical structures.

B. Loading Criteria

All models are designed according to the shown loading criteria in Table I. The default ultimate design combinations according to ASCE 7-10 [9] have been used.

C. Lateral Resisting System

The chosen lateral force resisting system for all models according to the seismic design category is intermediate moment resisting frames (IMRF) with the following lateral design factors [9] shown in Table II.

TABLE II
LATERAL FORCE RESISTING SYSTEM

Seismic Force resisting system	Response Modification Coefficient R	Overstrength Factor Ω_o	Deflection Amplification Factor C_d
IMRF	5.0	3.0	4.5

No limitation on building height with S.D.C. "C"

VI. Study Results

In this section the global behavior of the studied buildings will be presented. The results of the models considering different approaches to determine the second order magnification factors will be discussed and the change in behavior due to variation in height, lateral forces and material strength will be examined.

A. Different Magnification Factors Due to 2nd Order

Lateral stability of the buildings and second order magnification factors have been investigated/checked as per ACI/ASCE [6]-[9] design codes. A sample of the results shown in Fig. 3. Different methods of calculating magnification factor for each building in all sets have been introduced. The (Δ_2/Δ_1) values are representing the ratio of overall 2nd order lateral displacement to overall 1st order lateral displacement, (M_2/M_1) is the global story moment magnification, (θ_{1max}) is the stability factor required to be checked according to ASCE [8-9] not considering p-delta in analysis, (θ_{2max}) is the stability factor required to be checked according to ASCE [8-9] with p-delta considered in analysis and (***Buckling Magnification***) [11] is the magnification factor determined using the results of the advanced buckling analysis results.

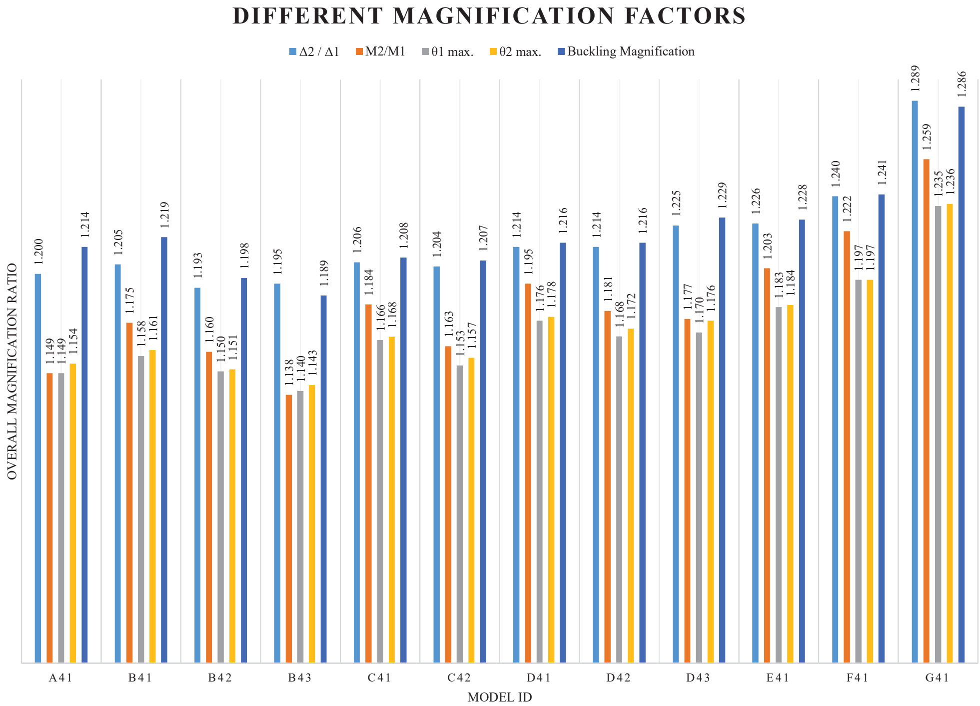

Fig. 3 Different approaches to determine overall 2nd order magnification

Down to the base of the second order magnification which is a direct result to the ratio of the applied vertical loads to the global buckling capacity of the building, as discussed earlier, buckling analysis is the most accurate way to assess the actual behavior of the building. Considering the results shown in Fig. 3, a noticeable similarity between buckling analysis results and the overall displacement magnification factor is clearly presented. Hence, the overall displacement magnification factor can be adopted as the most reliable traditional check to evaluate the building stability performance.

B. Effect of the Building's Aspect Ratio

Aspect ratio is one of the most logical terms affecting buildings' slenderness. The results obtained from analyzing all sets of the hypothetical models indicated that, generally, magnification due to second order effect is directly proportioning to the width/height ratio of the buildings. However, in some structures of low aspect ratio, increasing the building height/number of floors while keeping the aspect ratio constant tends to improve building's lateral stiffness and the overall performance of the building under lateral loads.

Examining Fig. 4, it is evident that second order magnification factors are directly proportioned to the aspect ratio of buildings having same, in plan, dimensions. However, this behavior could be clearly noted in low seismic zones.

For instance, buildings which comprise of three bays in plan (15m width) have been investigated under two different aspect ratios, 15:60 (1:4) and 15:120 (1:8), and it is clearly shown that no change in the magnification factors among buildings belonging to sets 1 and 3 that were designed for considerably high seismic forces was noted whereas, a noticeable variation in magnification values has been obtained for buildings belonging to sets 2 and 4 that were designed for low seismic forces, this variation reached a maximum value of **5.2%** due to the change in the building's aspect ratio for set 4.

Buildings which comprise of four bays in plan (20m width) have been investigated under five different aspect ratios 20:40 (1:2), 20:60 (1:3), 20:80 (1:4), 20:100 (1:5) and 20:120 (1:6). Same behavior, similar to buildings comprise of 3 bays, was observed with an exception to the first model with the aspect ratio of 1:2. This model has minimum lateral stiffness due to low level of vertical and lateral loads adopted in design, which increased the magnification factor.

The observed performance pattern of the previously mentioned buildings is applicable to most of the studied structures as shown in Fig. 4. Accordingly, it can be concluded that; aspect ratio is an important factor affecting the magnitude of the second order magnification. However, it shouldn't be considered as the sole factor for judging the building's stability. Other factors such as the level of lateral forces used for designing the buildings and strengths of materials can be effective too.

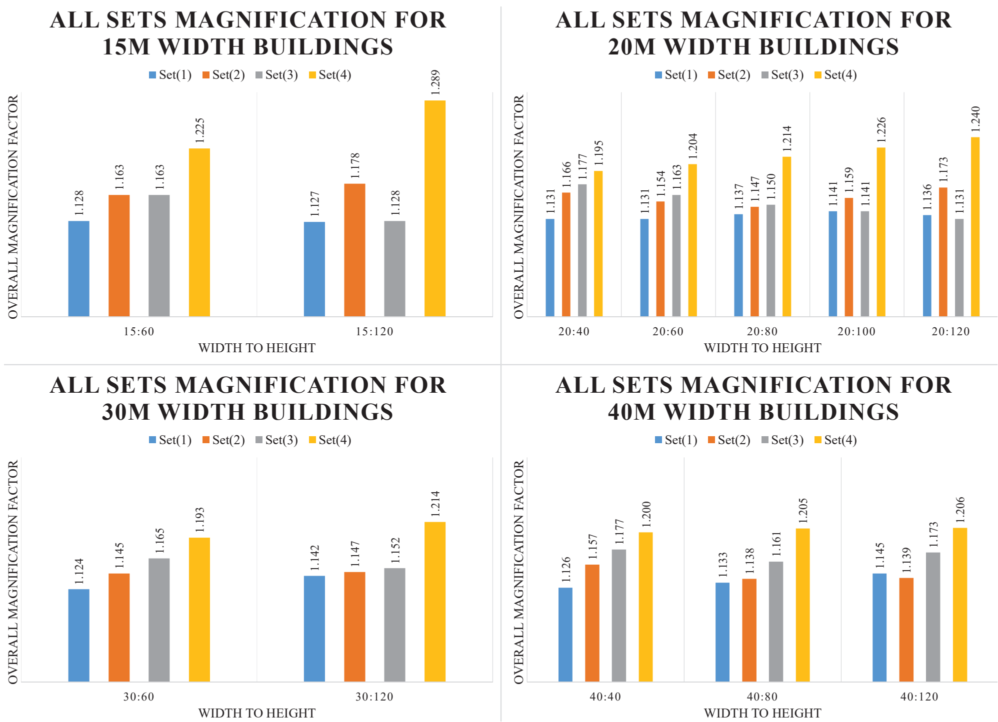

Fig. 4 Second-order magnification with respect to buildings' dimensions

C. Different Seismic Zones (Level of Design Lateral Load)

To examine the effect of increasing the level of lateral forces used for designing the buildings on their stability, different levels of seismicity have been adopted in this study for designing the buildings. As stated earlier, the only difference between set 1 and set 2 is the level of seismicity. Similarly, the difference between set 3 and set 4.

Fig. 5 shows the effect of this parameter on the 2nd order magnification factors. It is evident that buildings in low seismic zones are more susceptible to high 2nd order magnification than those designed for higher lateral loads. This is attributed to the minimal lateral stiffness which has been produced from designing the buildings located in low seismic zones for low lateral forces.

Comparing the obtained results from models of set 1 and models of set 2, it could be clearly noted that magnification factors have increased with a maximum percentage of **4.53%**. Similarly, but more severely, change in magnification factors between models of set 3 and models of set 4 have been noted to range from **1.53%** up to **14.27%**.

D. Effect of Using High Strength Materials

The effect of using high strength material in the construction of the building on their stability was checked. For such purpose, buildings belonging to sets 3 and 4 were designed using high strength materials. The only differences between set 1 and set 3 are the concrete strength and reinforcement yield stress. Similarly, the difference between set 2 and set 4.

Fig. 5 shows the effect of such parameter on the 2nd order magnification factors. It is evident that structures utilizing high strength materials are more susceptible to high 2nd order magnification. This is a result of minimizing the structural elements cross sections which have been produced from design of such buildings.

Comparing the obtained results from models of set 1 and models of set 3, it could be clearly noted that magnification factors have increased with a maximum percentage of **4.53%**. Similarly, but more severely, change in magnification factors between models of set 2 and models of set 4 have been noted to range from **2.49%** up to **9.42%**.

E. Individual Element Checks

According to ACI 318-08 [7], the stability check, which is mandatory to be carried out, is about the individual elements' straining actions. As discussed earlier and with reference to the commentary of this code, it was clearly stated that this check is based on the overall story stability check approach adopted by ASCE 7-05 [8] and that ACI committee has chosen the value of **1.4** [7] as the upper limit for the 2nd order / 1st order moments in each lateral resisting structural element for simplicity.

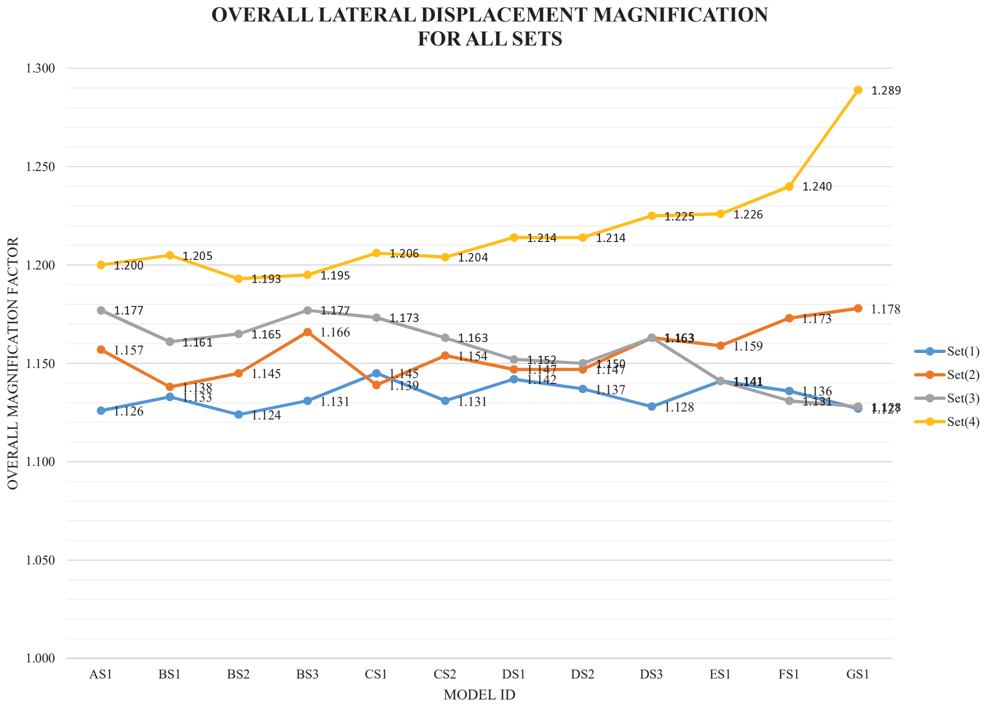

Fig. 5 Overall displacement magnification for all sets

Despite the fact that, no clear justification has been provided for converting the overall story check in ASCE [8], [9] into an individual members check in ACI [7], the practical problem here can be briefly stated as; the percentage of beams and columns with 2nd order to 1st order straining actions exceeding the **1.4** limit are unrealistic even though for buildings which are not slender or even having a considerably high magnification factor according to all other methods of calculation for the 2nd order effect. Figs. 6-9 are neatly presenting the different percentages of such members in all models that have been developed in this study. With no exceptions, all models have a considerable number of elements that need to be resized according to ACI code [7]. Percentage of columns/beams out of this limit in models of set 1 was ranging between **42.55%** and **76%** for beams and **28.64%** and **60%** for columns. For set 2 models the percentage is ranging from **65.56%** to **98.67%** for beams and from **31.85%** to **87.50%** for columns. For set 3 models the percentage is ranging from **37.45%** to **90.67%** for beams and from **35.20%** to **65.42%** for columns. And finally for set 4 models the percentage is ranging from **77.00%** to **98.50%** for beams and from **39.51%** to **89.17%** for columns. Accordingly, and based on the results obtained from this study it can be concluded that such individual elements check is not reliable and need to be scrutinized.

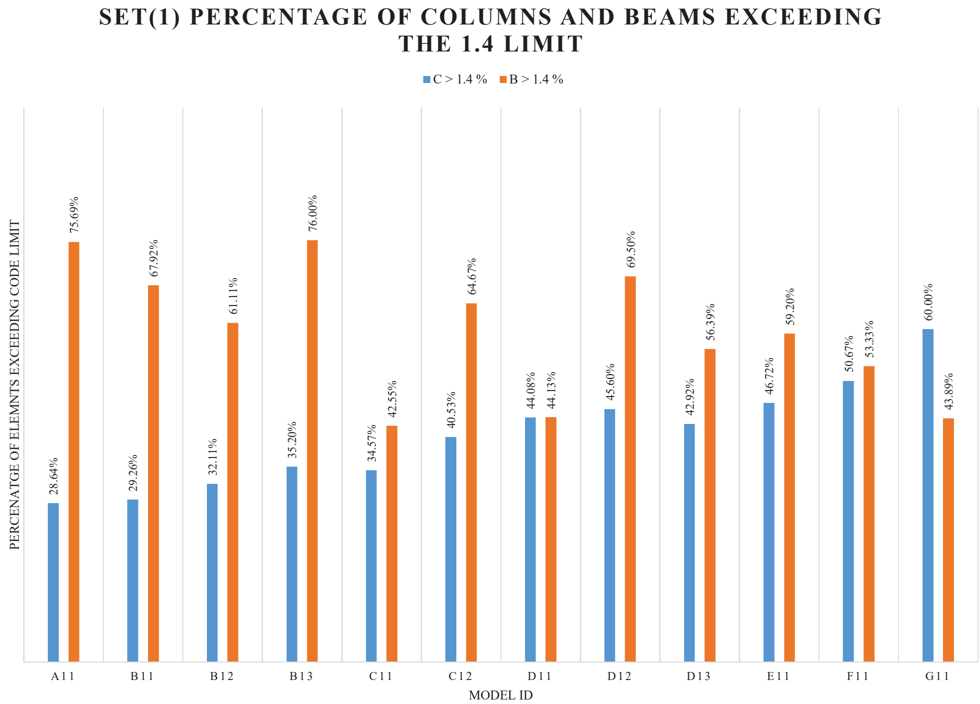

Fig. 6 Columns and beams with 2nd order/1st order moments exceeding 1.4 (set 1)

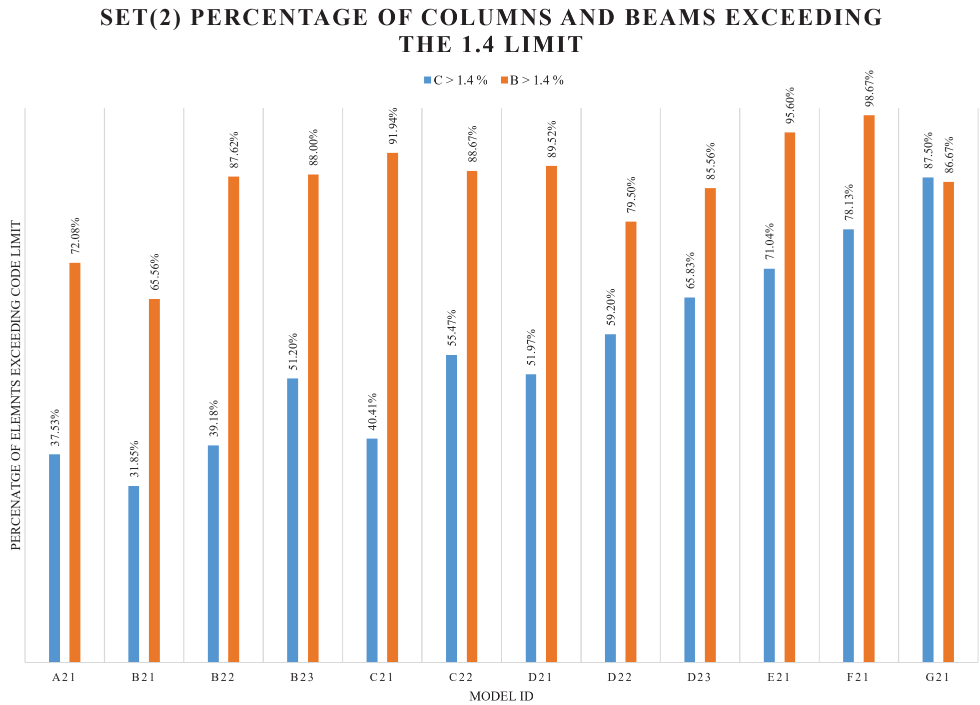

Fig. 7 Columns and beams with 2nd order/1st order moments exceeding 1.4 (set 2)

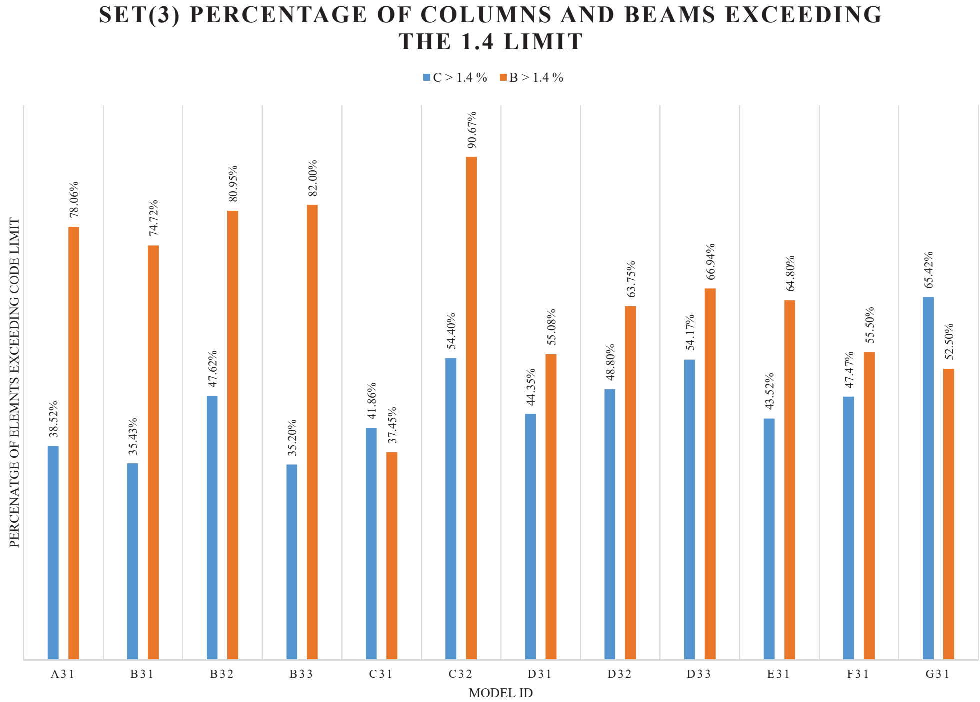

Fig. 8 Columns and beams with 2nd order/1st order moments exceeding 1.4 (set 3)

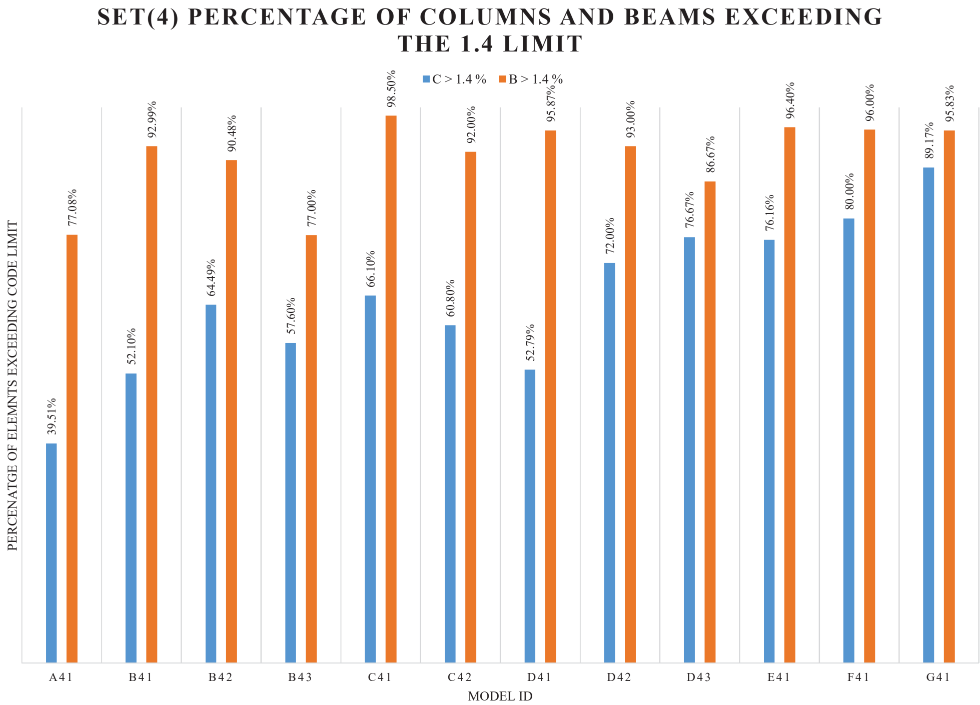

Fig. 9 Columns and beams with 2nd order/1st order moments exceeding 1.4 (set 4)

VII. Conclusion

Based on the performed study the following can be concluded:

- The term instability in most of design codes is a mathematical instability in second-order analysis and not physical instability. The actual upper limit of the overall vertical load to avoid the buckling failure mode can be defined as the global buckling load of the building which is similar to the individual Euler load but with the use of the global stiffness.

- Buildings with high slender aspect ratios, more than **1:4**, tend to be more sensitive for second order effects. In such cases the stability check is mandatory.
- Second order magnification is more likely to be increased in low seismic zones. This is a direct result of providing relatively weak overall stiffness of the buildings in such zones due to the minimal design lateral loads.
- Second order magnification is more likely to be increased while utilizing high strength construction materials. This is a direct result for reduced building lateral stiffness due to reducing structural elements' dimensions that would be provided by design.
- The limit stated by recent ACI [7] code asking for limiting the ratio of structural element's moment calculated from second order analysis to that calculated from first order analysis to be less than or equal to **1.4** is unjustified and conservative.

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