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Evaluating the effect of Non-Uniform Subsidence on Behave of Steel Cooling Towers under Wind Load

Shahryar Heidari* and Abdulrahim Jalali

Department of Civil engineering, Maragheh Branch, Islamic Azad University, Maragheh, Iran; shahryar_haidari@yahoo.com, Jalali@tabrizu.ac.ir

Abstract

Background/Objectives: This study was aimed to evaluate the effect of non-uniform subsidence on behave of steel cooling towers under wind load. **Methods/Statistical Analysis:** This study was done through Kato model and tower behave was studied after applying different loads. The tower height is 170 meters above the ground. The maximum radius was 60 meters at the bottom and the minimum radius was 36.5 in the upper part. Then the tower behavior were analyzed under loading in nonlinear static load cases. **Findings:** The results showed that the steel cooling tower had elastic behavior against the wind but components of lower levels of tower had plastic behaviour and eventually will lead to the destruction of the tower. Alsothe most tensions occures in the columns. Most effect and increasing of tension occure in columns with increasing the wind load. Balances No 5, 8, 15 are best balances for implementation the stiffener.

Keywords: Cooling Towers, Non-Uniform, Steel, Wind Load

1. Introduction

Low subsidence in soil under the foundation of a structure or rotating foundation and structres (like a rigid material) wont create new tensions in the structure. But non-uniform subsidence of foundation can creat considerable tension in structures and foundation.As a result, non-uniform subsidence is considered as a structural loading. So in different regulations of structures designing, there are coefficients for considering corresponding troops with non-uniform subsidence of structures and its combining with other loading factors. The difference of subsidence in the sandy soil is approximately equal to the maximum subsidence but in clay soil the difference of subsidence is lower than maximum subsidence[1]. Cooling towers are one of the great human inventions and due to the specific issues in the analysing and designing are interesting for researchers and engineers.In thermal power plants, power generation and petrochemicaln, machines heat should be transferred to the external environment to prevent increasing the temperature in different parts of the plant.One of empirical research in field of non-uniform subsidence was done by[2]. Rao studied non-uniform subsidence too(1992)[3]. Akhtari studied non-uniform subsidence through provided formula by kaloza and matza on mathematical model $\omega = \Delta U \cos(n\theta)$in arak cooling tower[4]. Kato et al., studied critical internal forces in various modes and noted that tensions were decreased highly in all parts compared with linear mode but the deformation of tower was increased[5]. This study was aimed to evaluate the effect of non-uniform subsidence on behave of steel cooling towers under wind load.

2. Materials and Methods

This research was done through Kato model and tower behave was studied after applying different loads. The geometry of tower is as follows:

The tower height is 170 meters above the ground. The maximum radius was 60 meters at the bottom and the minimum radius was 36.5 in the upper part. Tower had hyperbolic shape to height of 122 meters and it had cylinder shape to height of 170 meters after the balance. Equation of each balance radius of the tower is as follows:

$$r = \begin{cases} 36.5\sqrt{1+\left(\dfrac{z-122}{c}\right)^2} & x<0 \\ 36.5 & x \geq 0 \end{cases} \tag{1}$$

* Author for correspondence

Z:tower balance of the ground,C:radius of curvature of the hyperbolic tower =93.51.Towergeometry is shown in Figure 1[5].

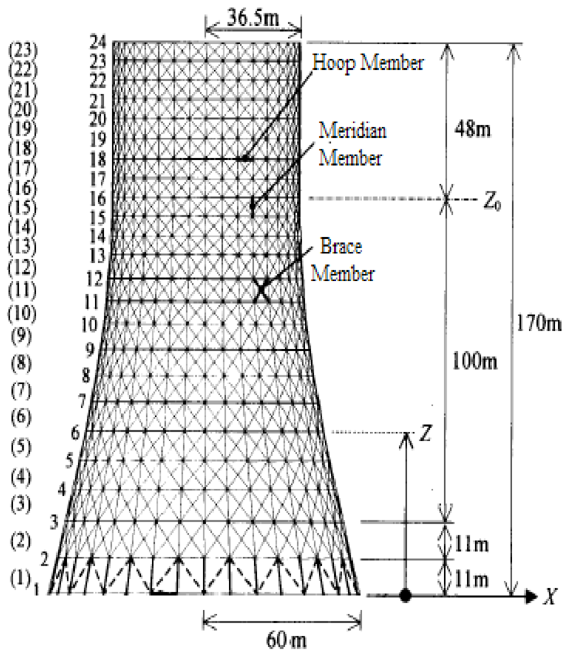

Figure 1. Geometry of cooling tower[5].

Table 1. Specification of cooling towers sections[5]

Member	Balance	Cross section (Cm^2)	Moment of inertia (Cm^4)
(Hoop Members)	H24-24	96.3	59000
	H16-23	74.4	23100
	H06-15	40.2	8050
	H04-05	80.4	16100
	H02-03	364	256850
(Meridiomal Member)	M17-23	33.7	4100
	M11-16	109	40600
	M08-10	149.2	66000
	M02-07	200.2	159000
	M01	280	336500
(Brace Member)	B20-23	33.7	4100
	B16-19	50	6070
	B03-15	80.4	16100
	B02	88.6	21900
	B01	87.5	26900

Due to the length of used parts in steel cooling towers, the members should possess high zirasion radius to keep emaciation in limit. Truss compound tools were used to acheieve this purpose, but these tools were inchmeal replaced by the weight of used steel were decreased due to using of tubular sections in cooling tower. All of segment are tubular sections in *Kato* model. Profile of used sections are mentoined in Table 1[5]. Beams and columns are rigidly connected to each other in *Kato* model. The connecting of bracing members is in articulation type and the members are connected in the middle part so that buckling length is reduced by half. Columns and braces are connected to each other in articulation type infirst row and the members also act axially.

Type of used steel in the profiles of the cooling tower is "steel ST-37" with yield stress 2,400 kN/cm^2 and modulus of elasticity is 2,000,000 kgN/cm^2.

2.1 Buckling of Compression Members

One of the most important issues in structures is buckling in the pressure member. Euler equation was used for controling of buckling of cooling tower members so this equation is presented as maximum thrust force of pressure (Pcr) for each member.

$$p_{cr} = \frac{\pi^2 EI}{L_e^2} \quad (2)$$

Assuming a uniform axial stress in each member:

$$\sigma_{cr} = \frac{\pi^2 EI}{AL_e^2} \quad (3)$$

L_e is the effective length depending on the support conditions. Support and values of effective length for different support conditions is presented[6].

According to Figure 2, L_e is effective length for bracing members in fact the support conditions of beams and columns were between (1) and (2) status, also due to this issue, effective length for each member is 0.075 L. According to equation (3) and Table 1, calculated values of σ_{cr} for all members is provided in Tables 2 to 4.

According to obtained values of σ_{cr} compressive stress didn't reach to step buckling before submission in all members $\left(2400\frac{kgf}{cm^2} = yield\ \sigma > critical\ \sigma\right)$.

In other words, all members of the towers are thick and there wasn't any problem in terms of elastic buckling instability during the loading.

Table 2. Buckling stress values in beams of cooling tower

Balance	Columns			
	Length (m)	Cross section (Cm^2)	Moment of inertia (Cm^4)	Buckling stress ($KNcm^{-2}$)
1	11.12	363	256850	20012
2	10.49	363	256850	22490
3	9.95	80.4	16100	7102
4	9.48	80.4	16100	7816
5	9.05	40.2	8050	8589
6	8.68	40.2	8050	9317
7	8.35	40.2	8050	10070
8	8.06	40.2	8050	10828
9	7.79	40.2	8050	11565
10	7.57	40.2	8050	12247
11	7.42	40.2	8050	12769
12	7.30	40.2	8050	13193
13	7.23	40.2	8050	13460
14	7.18	40.2	8050	13625
15	7.17	74.4	23100	21213
16	7.17	74.4	23100	21213
17	7.17	74.4	23100	21213
18	7.17	74.4	23100	21213
19	7.17	74.4	23100	21213
20	7.17	74.4	23100	21213
21	7.17	74.4	23100	21213
22	7.17	74.4	23100	21213
23	7.17	96.3	59000	41859

Table 3. Buckling stress values in columns of cooling tower

Balance	Columns			
	Length (m)	Cross section (Cm^2)	Moment of inertia (Cm^4)	Buckling stress ($KNcm^{-2}$)
1	11.50	280	336500	31899
2	11.46	200.2	159000	21224
3	10.38	200.2	159000	25870
4	9.31	200.2	159000	32177
5	9.27	200.2	159000	32428
6	8.21	200.2	159000	41369
7	8.18	200.2	159000	41696
8	8.14	149.2	66000	23413
9	8.11	149.2	66000	23604
10	8.08	149.2	66000	23788
11	7.05	109	40600	26335
12	7.03	109	40600	26473
13	6.01	109	40600	36170
14	6.00	109	40600	36258
15	6.00	109	40600	36302
16	6.00	109	40600	36308
17	6.00	33.7	4100	11859
18	6.00	33.7	4100	11859
19	6.00	33.7	4100	11859
20	6.00	33.7	4100	11859
21	6.00	33.7	4100	11859
22	6.00	33.7	4100	11859
23	6.00	33.7	4100	11859

Table 4. Buckling stress values in braces of cooling tower

Balance	Braces			
	Length (m)	Cross section (Cm^2)	Moment of inertia (Cm^4)	Buckling stress ($KNcm^{-2}$)
1	8.11	87.5	26900	9226
2	8.00	88.6	21900	7625
3	7.77	80.4	16100	6549
4	7.19	80.4	16100	7650
5	6.64	80.4	16100	8957
6	6.48	80.4	16100	9425
7	5.97	80.4	16100	11073
8	5.84	80.4	16100	11573
9	5.73	80.4	16100	12051
10	5.62	80.4	16100	12496
11	5.54	80.4	16100	12893
12	5.12	80.4	16100	15106
13	5.07	80.4	16100	15404
14	4.70	80.4	16100	17897
15	4.68	80.4	16100	18044
16	4.67	50	6070	10971
17	4.67	50	6070	10972
18	4.67	50	6070	10972
19	4.67	50	6070	10972
20	4.67	33.7	4100	10996
21	4.67	33.7	4100	10996
22	4.67	33.7	4100	10996
23	4.67	33.7	4100	10996

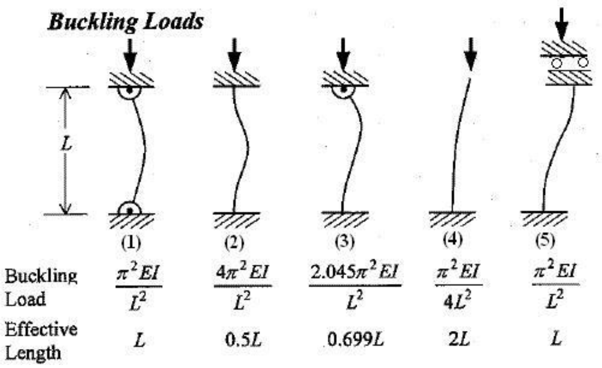

Figure 2. Figure of buckling load and the effective length for different support types[6].

2.2 Modeling in Abaqus Software

Abaqus software was used for modeling the steel cooling tower between *CSI, Abaqus,* and *OpenSEES*[7]. In this research, steel was defined by using "*steel101*" recipes. The stress-strain diagram of steel is presented in Figure 3. All of defined sections are fiber section type in model. Modeled sections by Fiber are the most complete sections which determine properties of finite element sections by using the fibers well. Figure 4 indicated a sample of fiber element sections which defined in tower[8]. Existence changing location of different member is reason of one factor of non-linear behave structures which caused extra forces P-Δ force in member. Another factor of the behave is changing location of nodes compared to the initial status. Usually the software considers the effects of changing the location as stiffness matrix addition the main stiffness matrix which called the geometric stiffness matrix. Option P-Δ should be selected in Abaqus software to consider the effects of P-Δ in section of definition the geometric stiffness matrix. Option *corotational* should be selected for considering the simultaneous effects P-Δ and shift nodes. *Corotational* geometric stiffness matrix was used in studied model.

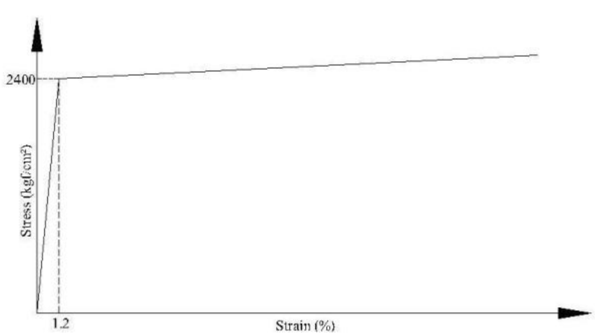

Figure 3. Diagram stress-strain of used steel in cooling tower.

Nonlinear beam column was used for defining the elements of cooling tower.This element is one of the most complete non-linear elements in *Abaqus* software. It determined the plastic hinge along the length of member by defining a series of plastic joint.

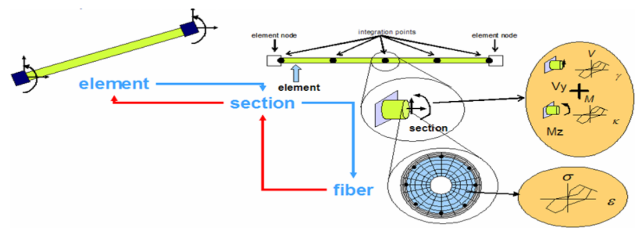

Figure 4. Model fiber section in cooling tower[8].

3. Results and Discussion

The wind load created the most tensions in members and it is determinative load in metal cooling tower.VGB regulations is one of the most respected regulations for desighning the cooling tower.The instruction of the regulations in wind loading is as follows[9]:

Wind load applied to each point of the cooling tower can be obtained from the following equation:

$$W(z,\theta)=\varphi.q_0(z).c_p(\theta) \qquad (4)$$

In the above equation: φ is dynamic magnification factor to consider the dynamic effects of wind load which it is calculated according to diameter of gorge, minimum natural frequency of tower and wind pressure on the edge of the tower. There isnt any formula for metal cooling tower. The dynamic magnification factor is 1 value in this research.

(z) Q_0 is base wind pressure in balance z of cooling tower. The instruction of VBG regulations is used for calculating q_0 (z) on base of regulations 5 in the regulations, z is the height of each part of tower compared to the base level.

$$Q_0(z)=0.9\times\left(\frac{z}{100}\right)^{0.22}\frac{kN}{m^2} \qquad (5)$$

Cp (θ) is coefficient of pressure distribution on the circumference of the shell. The coefficient is calculated in VBG instruction in terms of cover surface roughness (Table 5). According to the metal cooling tower is covered by corrugated sheets No 1, k / a or surface roughness is a great number on base of Figure 5 so K1.0 curve is used for calculating Cp (θ) (Table 6). Therefore, in this research the relation 6 is used to calculate Cp (θ)

$$C_p(\theta)=\begin{cases} 1-2.0\left(\sin\frac{90}{70}\theta\right)^{2.267} & ;\ 0\le\theta\le 70 \\ -1+0.5\left\{\sin\left[\frac{90}{21}(\theta-70)\right]\right\}^{2.395} & ;\ 70\le\theta\le 90 \\ -0.5 & ;90\le\theta\le 180 \end{cases} \tag{6}$$

Table 5. Table of curves pressure distribution as a function of surface roughness[9]

Roughness	Minimum pressure (min $C_{p)}$	Curve
0.006 to 0.010	-1.3	K 1.3
0.010 to 0.016	-1.2	K 1.2
0.016 to 0.025	-1.1	K 1.1
0.025 to 0.100	-1.0	K 1.0

Usually, the most transformations occurs against the wind load in cooling tower. The calculated wind load by using VBG regulatins in each balance and each angle was centralized on base of the load level of each node in the radial direction. The caused tension by wind load of structural members are provided at different heights in this section. The results are presented in Figures 6 to 12 in absolute value status.

According to diagrams 6 to 8 under the effect of wind load, the tensions of all members is in elastic range. the created tensions in the columns on the most critical balance: 60% σy, in braces about:30% and in beams about:20% σy. There are some fractures in altitudinal distribution of tensions as the charts show. The fractures were created due to sudden changes in cross-section of members similar behavior against gravity load and against wind load. For example the most prominent point of fracture Figure 6 occurred in level 4 of altitudinal distribution of beams. In this level the cross section was decreased to about 80% from floor 3 to 4 and lead to sudden increasing of tension. In this study for better understanding the behave of the tower againts the increasing wind load, the load was incrementally applied to tower for magnification factor of different wind load (q) and the results of tensions were calculated and presented in Figures 9 to 12.

To increase reliability of structure againts the loads, stiffener can be used at critical levels. These balances posses higher transience so balances No 5, 8, and 15 are best balances for implementation the stiffener. Even though, the gust factor decreases with the increase in height of water tower, the gust pressures increase with height of the water tower. The same is also valid for multistoried tall frames[10,11].

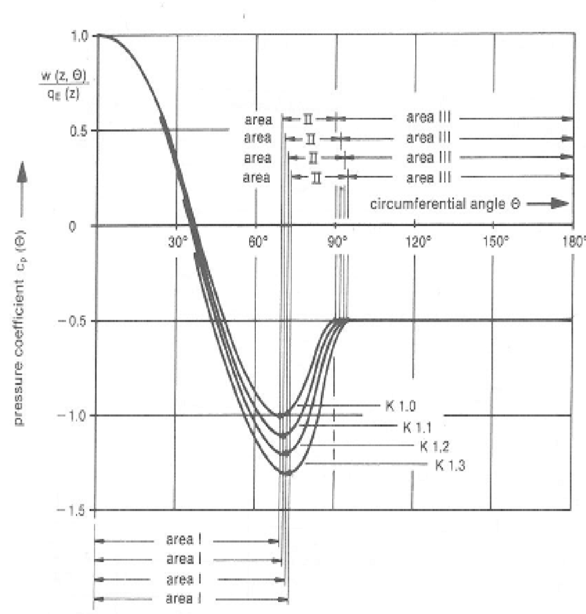

Figure 5. Amount of CP (θ) according to the VGB regulations[9].

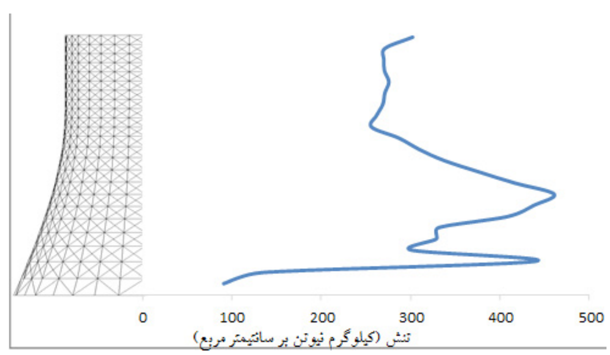

Figure 6. Axial tension of beams under the effect of wind load.

Table 6. Function of pressure curve CP (θ)[9]

Curve	Minimum pressure (min C_p)	Zone 1	Zone 2	Zone 3	pressure
1.3k	1.3	$-1.0+2.0\left(\sin\frac{90}{70}(\theta)\right)]]^{2.267}$	$-1.0+0.5[(\sin\frac{90}{21}(\theta-70))]]^{2.395}$	0.5	0.66
1.2k	-1.2	$-1.0+2.1\left(\sin\frac{90}{71}(\theta)\right)]]^{2.239}$	$-1.1+0.6[(\sin\frac{90}{22}(\theta-71))]]^{2.395}$	-0.5	0.64
1.1k	-1.1	$-1.0+2.2\left(\sin\frac{90}{72}(\theta)\right)]]^{2.208}$	$-1.2+0.7[(\sin\frac{90}{23}(\theta-72))]]^{2.395}$	0.5	0.60
1.0k	-1.0	$-1.0+2.3\left(\sin\frac{90}{73}(\theta)\right)]]^{2.068}$	$-1.3+0.8[(\sin\frac{90}{24}(\theta-73))]]^{2.395}$	-0.5	0.56

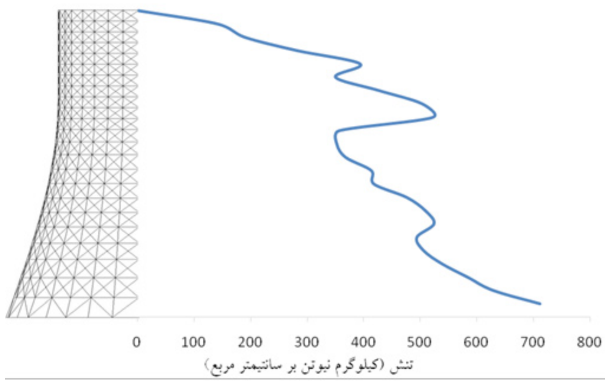

Figure 7. Axial tension of braces under the effect of wind load.

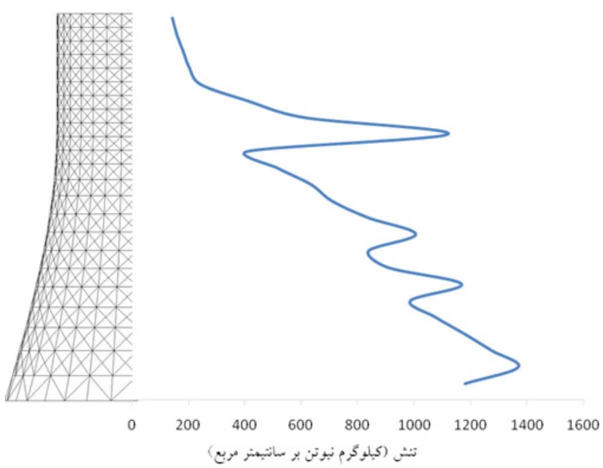

Figure 8. Axial tension of columns under the effect of wind load.

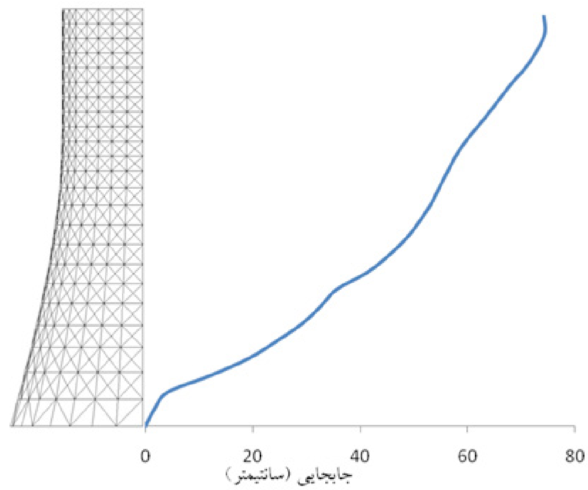

Figure 9. Lateral displacement of cooling tower under the effect of combined loads.

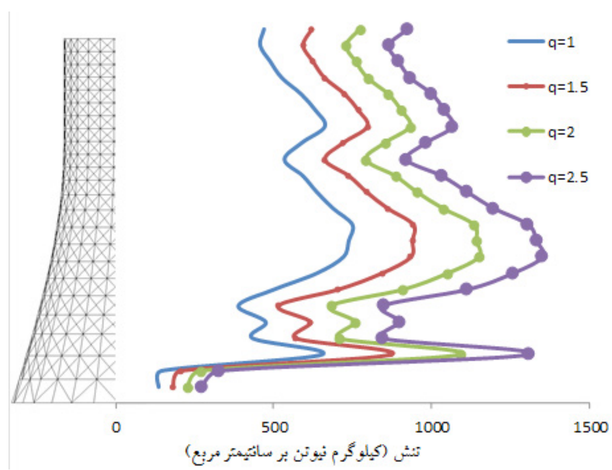

Figure 10. Axial stress of beams under the effect of combined loads for magnification factor of different load.

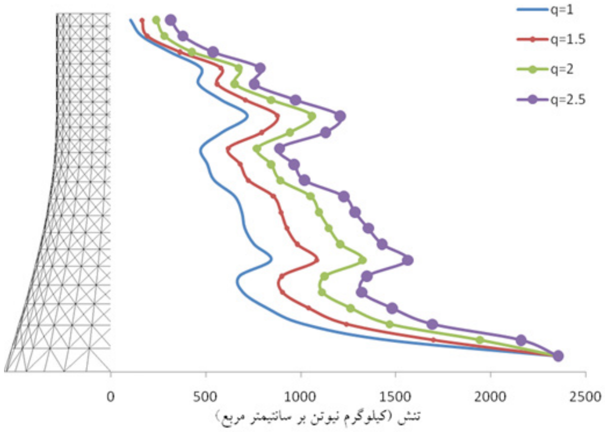

Figure 11. Axial stress of braces under the effect of combined loads for magnification factor of different load.

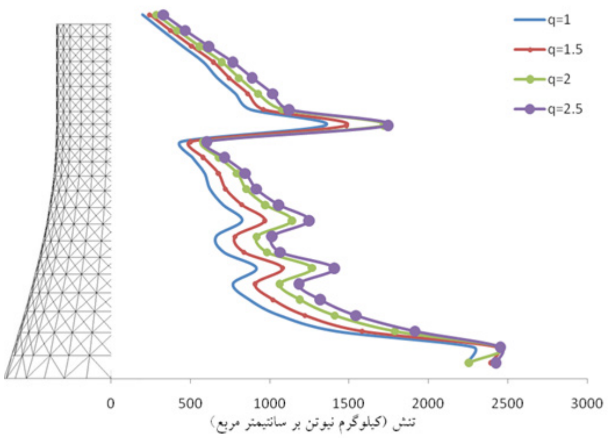

Figure 12. Axial stress of columns under the effect of combined loads for Magnification factor of different load.

4. Conclusion

Under the effect of wind load,the tension is in elastic range for total members.Also the most tensions occures in the columns.Most effect and increasing of tension occure in columns with increasing the wind load.Balances No 5,8, and 15 are best balances for implementation the stiffener. Sudden changes in cross-section of members causes the fractures of structures against the wind load.

5. Acknowledgment

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