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Post Fire Dynamic Analysis of Steel Buildings Subject to Wind Loadings in South of Iraq

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Abstract: Post-fire analysis of a six-story square steel building under the action of dynamic wind forces in the south of Iraq is presented. The nonlinear time-history analysis using direct integration method is accomplished by SAP 2000 V16 program while geometric nonlinear parameters are included. The post fire deformations values and their configurations along building are based on available literature that related to post fire deformations of steel buildings at 550^{o}Cwith reducing of yield stress and modulus of elasticity) by 10% due to fire. In the present study, two post fire scenarios are considered, with wind speed of 42 m/s, according to Iraqi meteorological and IQS.301 standards. The aim of this study is to investigate the post-fire dynamic analysis of a multi-story steel moment-resisting building subjected to dynamic along-wind loads. Variations of base shear, base moment, drift ratio, and displacement are considered for discussion and comparison. Its concluded that the presence of deformation in building after fire and the reduction in yield stress and modulus of elasticity increased the base shear, base moment and drift ratio as average by about 12.5, 20& 64%respectively under the effect wind load. The fire deformations may be critical and the structural decision for the building safety should be done via structural analysis of the building taking into consideration different parameters related to post fire effects.

Keywords: Post-fire, steel building, dynamic wind analysis, along wind.

1. Introduction

Steel has been used for the framework of buildings for almost 200 years due to the cost of material and labor aspects. The occurrence of a fire in a steel structure would lead to a significant deterioration in material strength and stiffness as shown in Fig. (1) and consequently cause large losses both in lives and property because the physical properties of steel are temperature dependent [1]. Modern construction very often uses steel structural framing to carry the structural loading making, therefore, the study of its response to fire affects a necessity [3].

Fig.1 Deformation shape (Mill of Misan) after fire.

Although safety from fires is the most important, but the restoration of structures damaged by fires is important to save costs of removing the damaged structure and constructing of new one. Since 1960s, the researchers are focused on the mechanical properties of the material steel structures subjected to fire, taking into consideration both low strength steel (Dill (1960), Diggeset al.(1966); Tide(1998), Kirbyet et al. (1986), Smithet et al. (1981), Outinen and Makelainen, (2004), Lee et al. (2012)) and high strength steels (Qiang et al. (2012, 2013), Tao et al.(2013), Chiew et al.(2014)), while cold-formed (Gunalan and Mahendran (2014)) and stainless steel (Wang et al. (2014)) have gained considerable attention as well. Iua et al. 2005 [1٢], studied an eight-storey composite frame. The maximum temperature was approximately800oC and the frame cools to ambient temperature. They concluded that the structure after cooling to ambient temperature, the lateral displacement increases inward to 155.28mm in the column and 220.17 mm in the mid beam while 192.01mmin edge beam, this fire analysis due to the thermal contraction force of the beam member under cooling.

Smith et al. 1981 [1٦], studied the factors which can cause steelwork to distort and collapse during fires. Laboratory-based experiments were described in which the strengths of various grades of steel were determined at elevated temperatures. The effects on mechanical properties of heating steelwork to temperatures in the range 100-1000oC and cooling back to ambient temperature have been assessed.

1.2 Visible Deformation Due to Fire:

Due to the thermal elongations coupled with reductions in steel strength and stiffness that occur at elevated temperatures, imperfections, crookedness, or force eccentricity can initiate visible local flange and/or web buckling, or overall member buckling, above about 315 °C with complete restrain from thermal expansion. Buckling is very likely to occur at temperatures in the 650 to 760 °C range, when the strength and stiffness are less than 50 percent of their nominal ambient values. Past experiences indicated that local buckling often can occur quite suddenly at, and above, this temperature range. In addition to these buckling distortions of the member, the steel will experience increasing end rotation and vertical deflections during the fire from the existing dead and live loads[4, 5]. While connection behavior is different than main member behavior when the temperature increases because of their relative compactness. The axial force developed by a restrained member will impose large forces on the end connections. Generally, the beam will buckle or deform to accommodate the axial force. Under these conditions, connection distress is easy to identify when steel beam cools, if the connection has fractured, the steel beam will pull away from the adjacent member revealing the damage.

All structural materials can suffer damage as a result of a severe fire, because they lose strength and expand when heated to elevated temperatures. The structure is then deformed and the deformation depends upon the applied load and support conditions [1]. When steel structures are exposed to fire, strength and stiffness are decreased with increasing temperature, the reduction for normal strength by 10% of the initial strength, whereas for high strength at least 75% of the strength is regained on cooling from temperatures above 600oCMaraveas, 2014 [6]. In this study, the mechanical properties (yield strength Fy, elastic modulus E) of steels after fire is considered 90% of that before fire.

1.3 The Wind:

The wind is essentially the large-scale horizontal movement of free air or it means the motion of air in the atmosphere. It plays an important role in the design of tall structures because it exerts loads on the building. Wind is caused by air flowing from high pressure to low pressure. Since the earth is rotating, however, the air does not flow directly from high to low pressure, but it is deflected to the right (in the northern hemisphere; to the left in the southern hemisphere), so that the wind flows mostly around the high and low-pressure areas[2,7].The motion of wind is turbulent. A concise mathematical definition of turbulence is difficult to give, except to state that it occurs in wind flow because air has a very low viscosity about one-sixteenth that of water. Any movement of air at speeds greater than 2 to 3 mph (0.9 to 1.3 m/s) is turbulent, causing particles of air to move randomly in all directions. This is in contrast to the laminar flow of particles of heavy fluids, which move predominantly parallel to the direction of flow. For structural engineering purposes, the velocity of wind can be considered as having two components: a mean velocity component that increases with height and a turbulent velocity that remains the same over height shown in Fig. (2) [8]. Similarly, the wind pressures, which are proportional to the square of the velocities.

Fig.2 Variation of wind velocity with time.

In this study, along wind component is considered in the analysis, in which the wind velocities are obtained from Iraqi Code IQS301 (Iraqi Code for forces and loadings) [9], which corresponds to the 3 second-gust speed at 10 m above ground in open terrain. The basic design wind speeds for Iraq which it's clear that basic wind speed for the south of Iraq is 42 m/sec.

2. Problem Modeling

The structure consists of six bays in both x-direction and in y-direction; each bay is (5x5m) center to center with a total height of 19 m. Thus, have similar plan dimensions in XY plane of 30x30m as shown in Fig. (3). Typical floor to floor heights measured from centers of beams is 4 m for the ground floor and 3m for the other stories. The column bases are modeled as fixed at the ground level. The mass of each element is assumed to be concentrated at the nodes.The distribution of element mass is equally divided between nodes. The mass of the the structure includes the applied dead load, including the self-weight, and live load. The mass is equal to the weight defined by the dead and live load divided by the gravitational acceleration (g). All the beams and columns members in the studied buildings are W-shape.

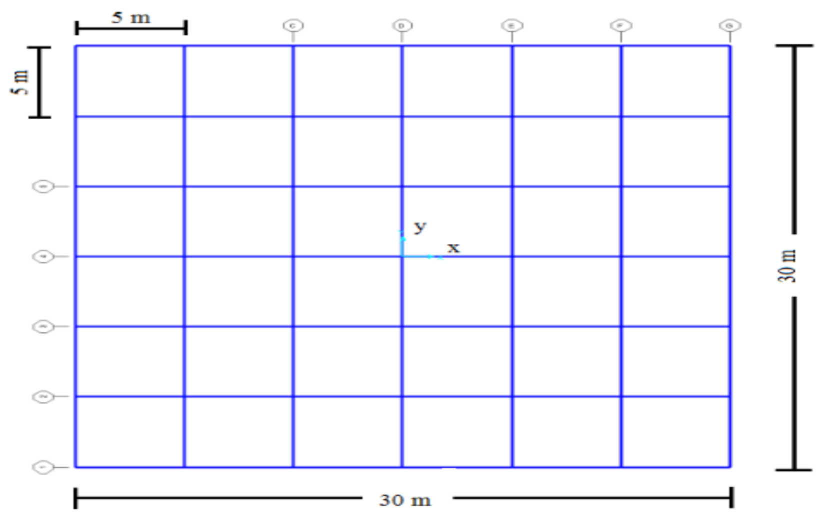

Fig.3 X-Y Plan (Top view cross sections of the beams -W14x38;
Columns from 1 to 3 story-W18x106 and 4 to 6 story -W16x36).

2.1 Loading Conditions:

Dead Load: the roof dead load is taken as (4kN/m²). Assuming steel deck slab resist on W-shape beams. Live Load: ASCE/SEI 7–05 [10], live load is taken as (6kN/m²) for industrial building. The dynamic wind records used in the present study has maximum wind speed 42m/s to represent the wind load-time history method applied loads on nodes for each story in x- direction during 45sec.

2.2 Material Properties:

The properties of the material of steel before fire used for both beams and columns are presented in Table (1) while the steel properties after fire are presented in Table (2) Constant damping ratio of (ζ = 0.02) is assumed and α and β are coefficients representing mass and stiffness proportional damping 0.046, 0.008 respectively[11]. The stress-strain curve for steel after fire is shown in Fig. (4).

Table 1. Steel properties before fire.

Item	Description	Unit	Value
Fy	Minimum yield stress	Ksi / Mpa	36 / 250
Es	Modulus of elasticity	GPa	210
ρs	Density	kN/m³	77
vs	Poisson's ratio	——	0.3

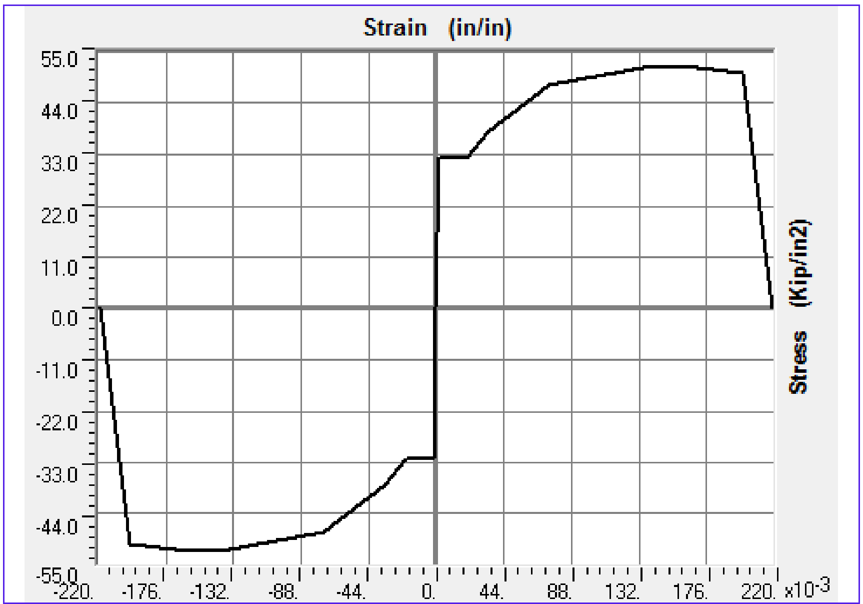

Fig.4 Stress-strain curve after fire reduced 10% from original.

Table 2. Steel properties after fire.			
Item	Description	Unit	Value
Fy	Minimum yield stress	Ksi / Mpa	32.4 / 225
Es	Modulus of elasticity	GPa	189
ρs	Density	kN/m³	77
vs	Poisson's ratio	——	0.3

2.3 Post-fire buildings configurations:

Three cases were studied; Case F0 is the state of the building before being exposed to fire but within the influence of gravity and wind loads. While the Cases (Case F1 and Case F2) states of the building after fire happen, where case F1 of the building cool back was fire on stories third to sixth in the four bays of the center of the building, while Case F2 a fire was from the first to the third in the corner of the building, they are shown in Fig. (5 and 6). The data of the deformations were taken from Iua, et al. [12] which is for approximately 550°C, the maximum beamed flection is L/60andmaximumcolumn lateral displacement is L/40were put in the SAP2000 as a values (where L is length of member). With reducing of yield stress and modulus of elasticity) by 10% due to fire temperature and these values are reduced from bottom in place of fire to top of building.The fire deformations divided into two components the first is deflection that is including the bent of beam after fire at the building on the fire area in the storys that included fire while the second component after fire is call a displacement of column under the effect of fire and they values of these deformations are presented in Table (3).

Table 3. Deformations after fire [8].

Member Types	Quantity	Deformation Types	Value
Column	Fully (Δ)	Displacement	L/60
Beam	Fully (Δ)	Deflection	L/40

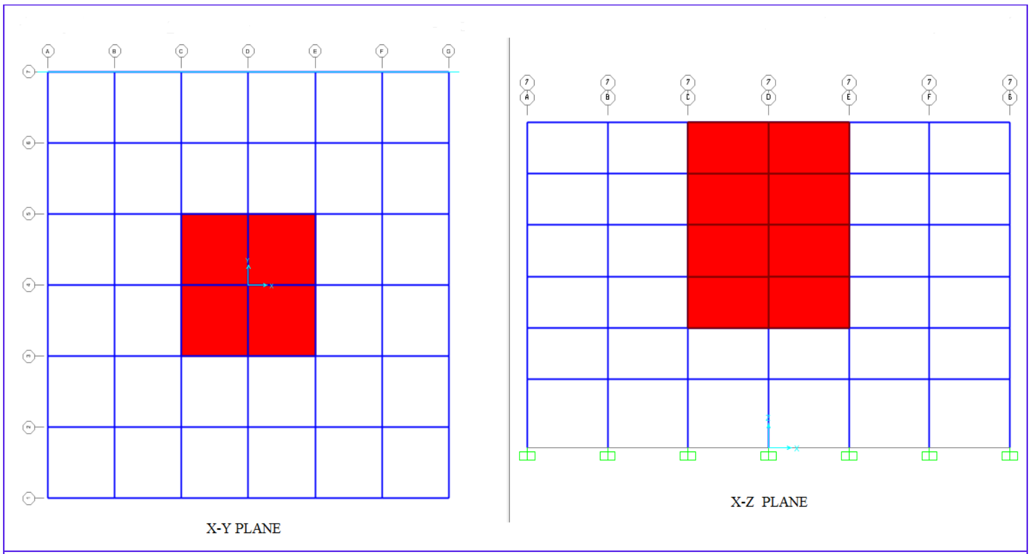

Fig. 5. Fire in Case F1.

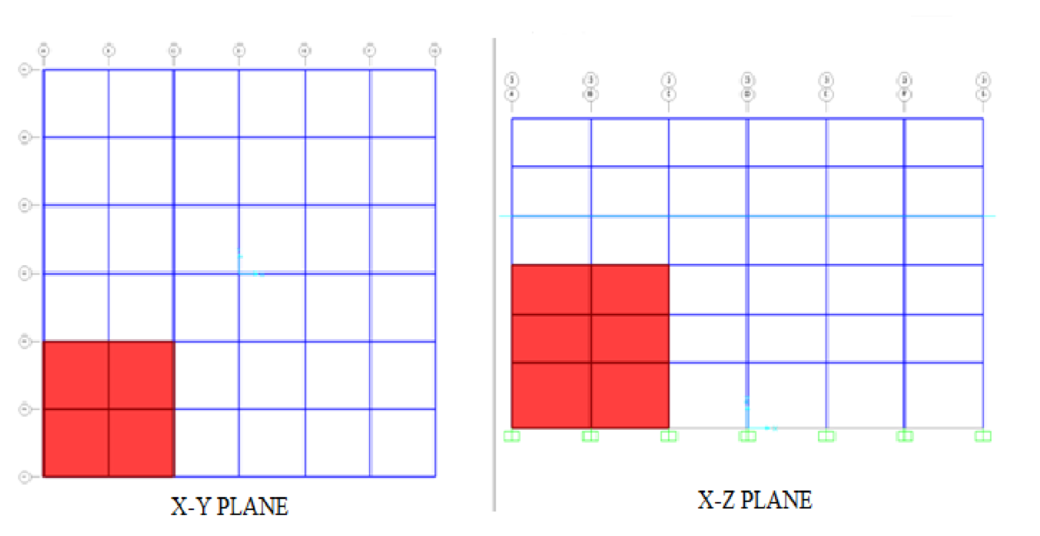

Fig. 6. Fire in Case F2.

4. Calculation of Loading and Displacements Tolerances

4.1 Dynamic Time history of wind loads:

Based on scaled time-histories of wind velocity (42m/s), the dynamic wind loads on building are calculated by equation (2) in which the time-histories amplitudes change with height via equation (1).

$$vx(t) = v10(t) \times \left(\frac{x}{10}\right)^{\frac{1}{\alpha}} \tag{1}$$

Where:

X: distance from ground level in a meter.

V10(t): reference velocity, at 10 m above the base in m/sec.

Vx(t): the wind velocity at any height (x) in m/sec.

α: power-law coefficient commonly 0.16[13].

$$F=0.5\,\bar{\rho}\,(V(x,t))^2\,C_d\,A \tag{2}$$

Where

F: dynamic wind load.

A: area upon which wind acts (m^2).

C_d: drag coefficient (equal to 1.3 [8]).

V (x, t): velocity of the wind at any level (x), at any time (t).

Thus, according to equation (2) the wind velocities are converted to forces by multiplying by projected area for each story, then the calculated per story is divided equally along nodes of projected face and modeled it as joint.

The variation of static wind speed with height of the building, 19m is shown in Fig. (6). While the time-history of wind forces due to wind speeds at third story showed Figs. (7). Time history was taken from Krauthammer [15]to represent along wind speed the velocity at any level can be found by using the power law wind formulaWhich showed that the shape of time-histories of wind velocities and forces are slightly difference due to nonlinear variation of velocity with height according to power law.

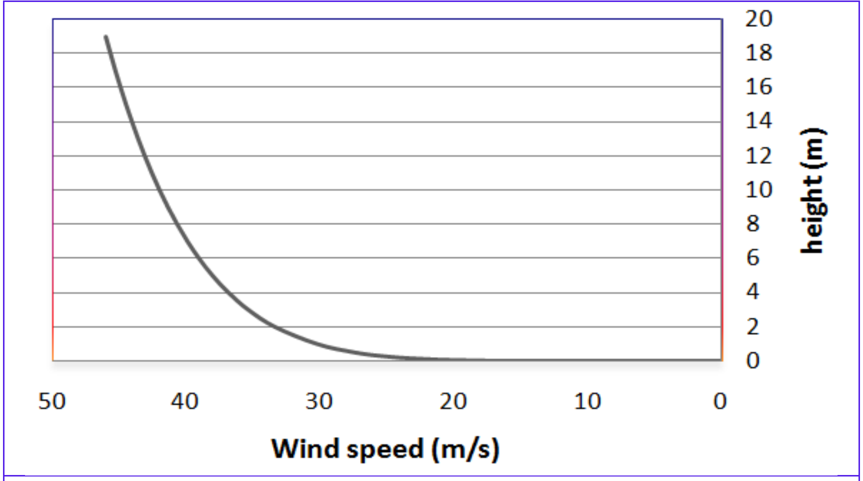

Fig. 6. Variation of wind velocity with height.

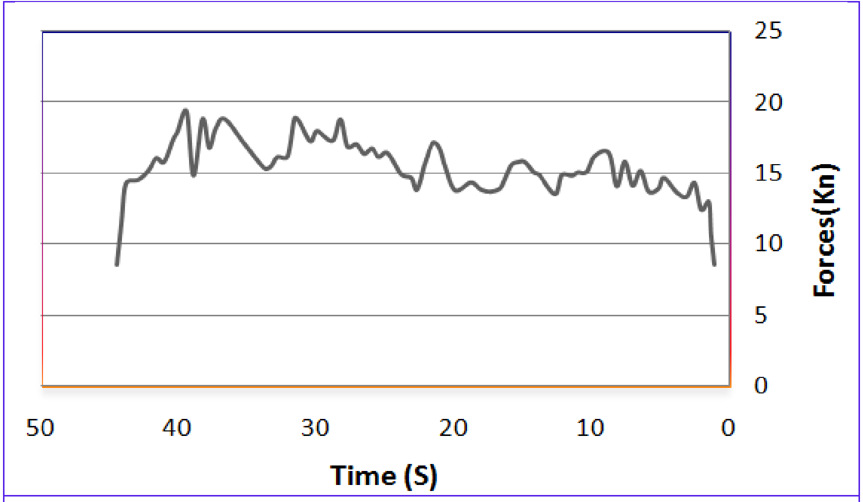

Fig. 7. Dynamic wind loads for third floor.

4.2 Structural tolerances for wind load

According to BS 8110-Part 2: 1985 [14] the maximum allowable displacement is calculated as h/500, where h is the story height. Thus, for a building with height of h=19m, the maximum top displacement (drift story) should be less than h/500=38mm so that the building is considered within allowable limit under wind load.

5. Results and discussions

The Multi-story steel building of 19m height under the action of dynamic force is analyzed before and after fire for thre ecases (Case F0 Case F1andCase F2) with different bays and story fires. The dynamic wind load was applied using time-history analysis technique. Analysis is adopted, using nonlinear direct integration using the finite element analysis of SAP2000 V16 software. The responses of buildings are investigated through several parameters such as the maximum base shear, base moment, drift ratio and displacement in x-dir.

5.1 Post-fire analysis, deformation (Δ)

In this case, the post-fire behavior of steel building is studied via comparison the results with that before fire for building cases (Case F0Case F1 and Case F2) using nonlinear analysis, where wind based on basic wind speed, V = 42 m/s at 10m above ground are used.

5.2.1 Base shear in x-dir. for Case F0,Case F1&Case F2:

From Table (4) it can clearly be seen that the maximum base shear of Case F0 under the effect of wind loads equal to 763kN while the maximum due to the same wind for Case F1and Case F2 868.69, 847kN respectively, the difference between Case F0 and CaseF1are 14%whilethe difference between Case F0and Case F2 are 11%.So there is slightly difference in time-history where there is sharply of oscillation in wind after fire as compared with before fire due to the fire that making reduced on yield stress and modulus of elasticity 10% from before fire as shown in Figs.(9) to (12).

Table 4. Base shear x-dir. before and after fire.

Output Case	Base shear_x CaseF0	Base shear_x CaseF1	Base shear_x CaseF2
Text	KN	KN	KN
DynamicWind	763	868.69	847.211

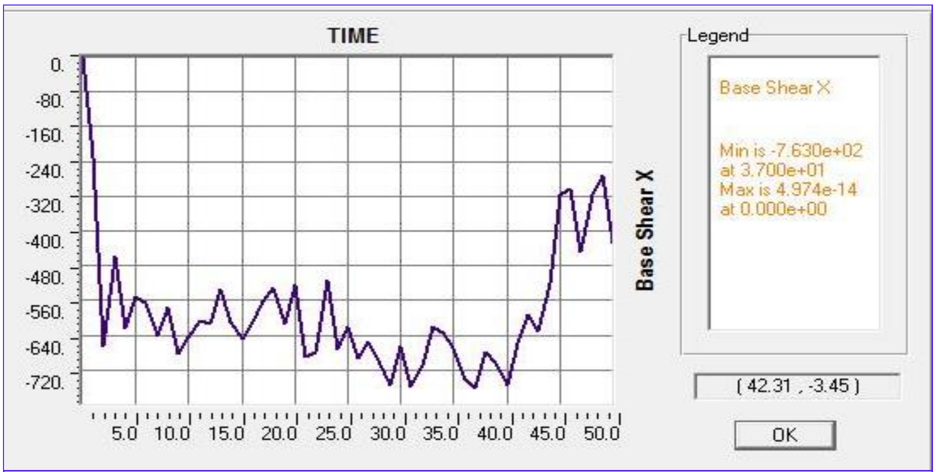

Fig. 9. Base shear in x-dir. due to wind, Case F0.

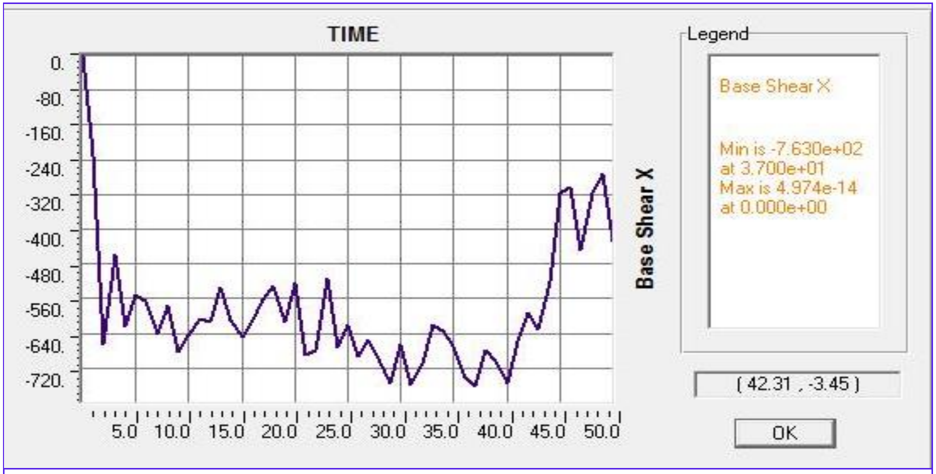

Fig. 10. Base shear in x-dir. due to wind, Case F1.

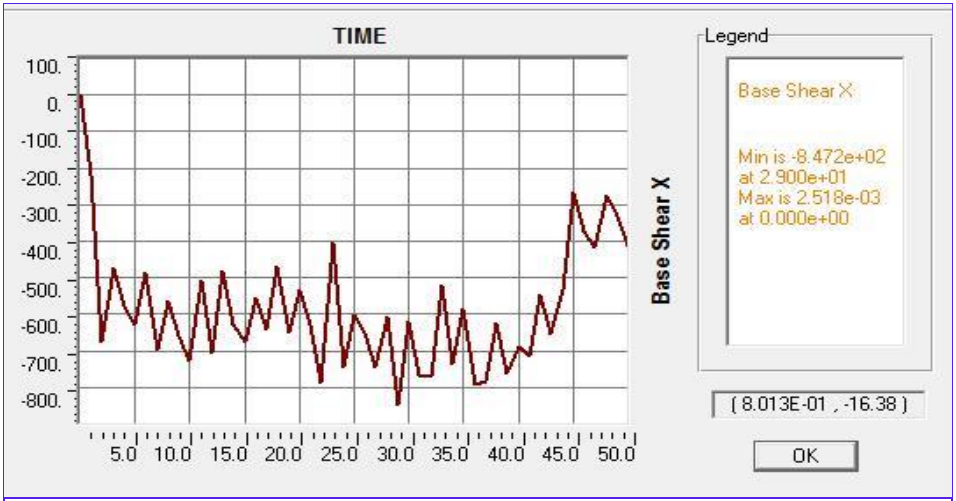

Fig. 11. Base shear in x-dir. due to wind, Case F2.

5.2.2 Base moment in y-dir. for Case F0, Case F1andCase F2:

Similar to base shear,The maximum base moment after fireof Case F0 under the effect of wind is equal to 10170kN.m while the maximum due to the same wind before firefor Case F1 and Case F2 are equal to 12157.7, 12302kN.m respectively as shown in Table (5), the difference between Case F0 and Case F1are 19.5%, the difference between Case F0 and Case F2 are 21%and there is slightly difference in time-history where there is sharply of oscillation in wind after fire as compared with before fire due to the fire that making reduced on yield stress and modulus of elasticity 10% from before fire as shown in Figs.(13) to (15).

Table (5): Base moment y-dir. before and after fire.			
Output Case	**Base Moment_y CaseF0**	**Base Moment_y CaseF1**	**Base Moment_y CaseF2**
Text	KN.M	KN.M	KN.M
Dynamic Wind	10170.1	12157.7	12302

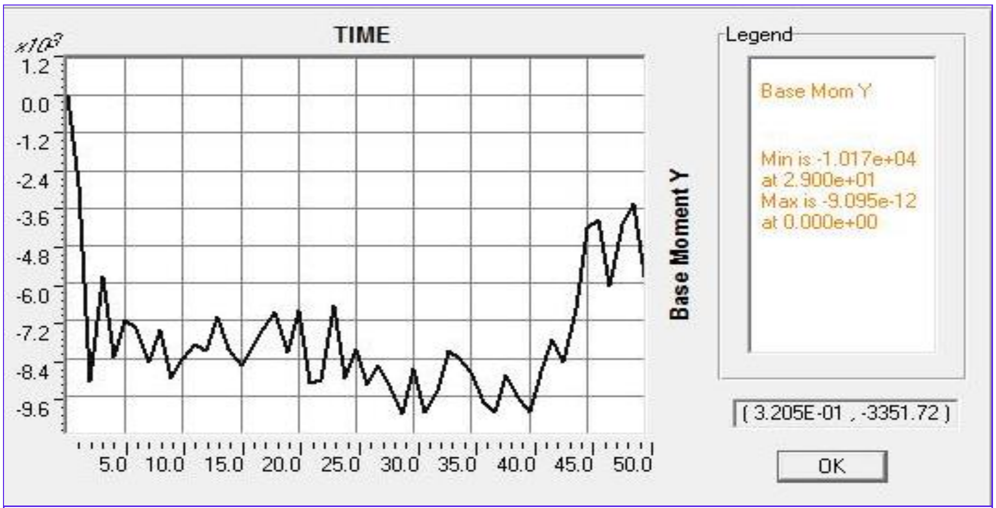

Fig. 13. Base moment in y-dir. due to wind, Case F0.

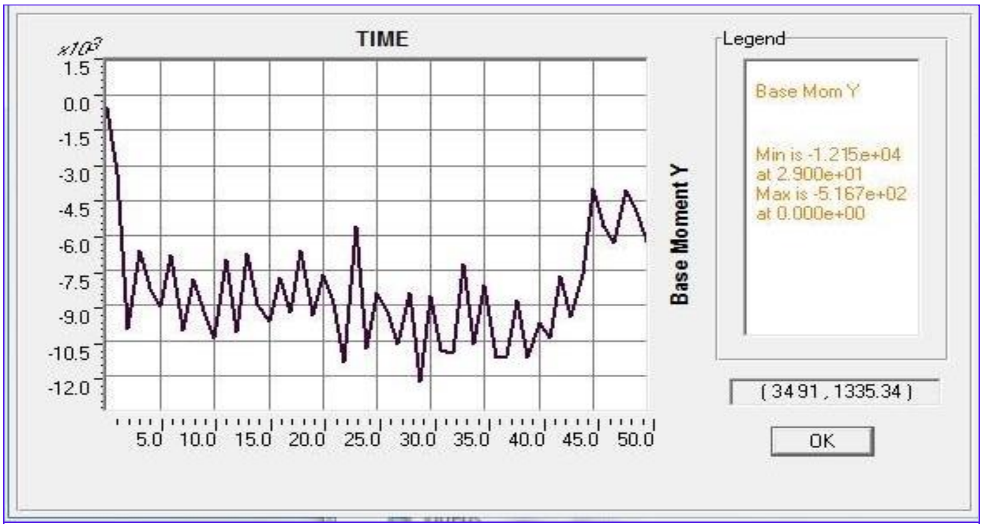

Fig. 14. Base moment in y-dir. due to wind, Case F1.

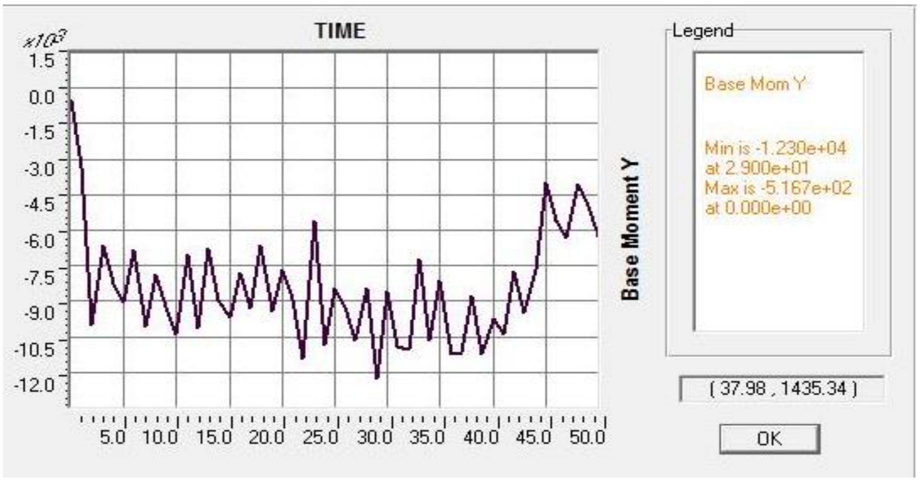

Fig. 15. Base moment in y-dir. due to wind, Case F2.

5.2.3 Max. drift ratio in x-dir. for Case F0, Case F1andCase F2:

The peak drift ratio response obtained from the numerical model for Case F0 in one principal directions x-dir. of the building was compared with Case F1 &Case F2results as shown in Fig. (17). The maximum drift ratio in fourth story before firefor CaseF0 and after fire for Case F1 under the effect of windloads equal to 0.127%, 0.204% respectively, the difference between them is 61% while for building CaseF2 after fire the peak drift ratio in first story is 0.214% and the difference between CaseF0 and Case F2 is 68%.So,there is clearly difference in maximum drift ratio before and after fire due to the fire

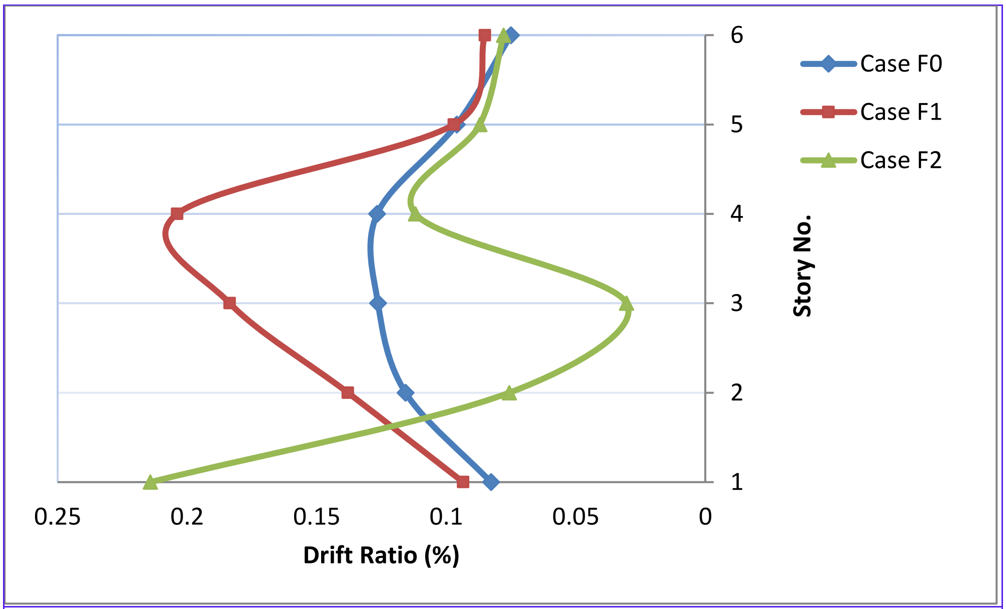

Fig. 17. Drift ratio % vs. story number for Case F0, Case F1&Case F2.

damage in building which led to reduced 10% of Fy and E after fire.

5.2.7 Max. displacements in x-dir. for Case F0, Case F1andCase F2:

The maximum displacement on each level is listed in Table (9). The maximum displacement in last story before fire for CaseF0 and after fire for CaseF1 &CaseF2 under the effect of wind loads equal to 19.5, 24.96 &20mm respectively, the difference between before and after fire for CaseF0 and Case F1 is 27%whilethe difference betweenCaseF0 and Case F2, them is 3%due to the yield less than 10% from origin on fire area.

It's found that is slightly difference in maximum displacement in Case F2 due to the maximum occurred in last story that is not included fireinvertCaseF1but there is clearly difference in the first and second stories are increased as compared with Case F0 due to the fire which making to reduced 10% of Fy and E after fire as shown in Fig. (24). According to BS 8110-Part 2: 1985 the maximum allowable deflection is calculated as h/500, therefore, maximum allowable displacement value for building height of 24.96m is 38mm. The maximum value of displacement in serviceability limit condition obtained for dynamic wind loads are less than allowable (38 mm) for criteria failure, so the building in safe in all cases.

Table 9. Max. displacement before and after fire.

Case F0	Case F0	Case F1	Case F2
Text	**Ux**	**Ux**	**Ux**
Story NO.	**mm**	**mm**	**mm**
1	3.31	3.739	8.568
2	6.78	7.877	10.839
3	10.57	13.383	11.75
4	14.37	19.5	15.107
5	17.25	22.41	17.718
6	19.5	24.96	20.053

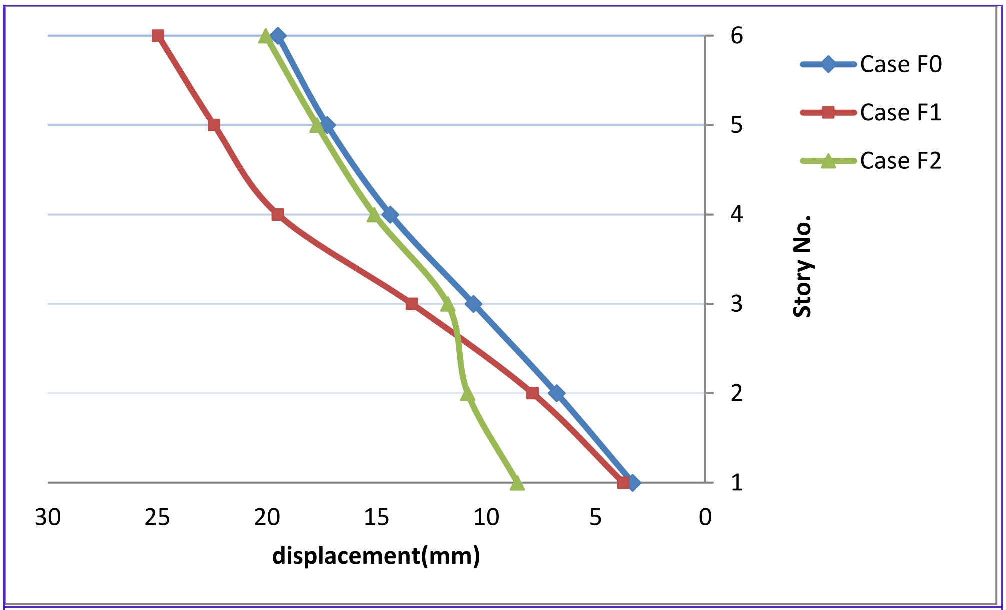

Fig. 24. Max. displacements vs. story number for Case F0, Case F1&Case F2.

6. Conclusions

From the present theoretical study and depending on its results the following points are concluded:

1. Base shear from post-fire states (Case F1&Case F2) are large than before fire state (CaseF0) by 14 & 11% respectively under the effect wind load.

2. Base moment from post-fire states (Case F1& Case F2) are large than before fire state (CaseF0) by 19.5&21% respectively due to fire deformations and their configurations. This yields the fact that base shear and base moment affected by both post fire deformation and also their configuration along building (post fire scenario).

3. Drift ratio from post-fire states (Case F1& Case F2) are large than before fire state (CaseF0) by 61 & 68% respectively under the effect wind load.

4. The displacement from post-fire states (Case F1& Case F2) are large than before fire state (CaseF0) by 27 & 3% respectively at same conditions there are considerable differences at stories affected by temperature. Thus the drift ratio and displacement are affected by both post fire deformation and also their configuration along building (post fire scenario).

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References

[1] Kirby B.R., Lapwood D.G. And Thomson G. 1986, "The reinstatement of fire damaged steel and iron framed structures ", *British Steel Corporation, Swindon Laboratories, UK*.

[2] Dawood A. O. , Sangoor A. J. and , Al-Rkaby A.J. ", 2020, " Case Studies in Construction Materials, Behavior of tall masonry chimneys under wind loadings using CFD technique"

[3] KonstantinosMiamis, "A Study of the Effects of High Temperature on Structural Steel Framing," M.Sc. Thesis, Purdue University, Indiana.

[4] Tide R. H. R. , 1998, "Integrity of Structural Steel after Exposure to Fire" *AISC Engineering Journal*, First Quarter.

[5] Richard G. Gewain, Nestor R. Iwankiw and FaridAlfawakhiri, 2003, “Facts for Steel Buildings fire,” *ASCE*..

[6] ChrysanthosMaraveas. , 2014 "Post-Fire Mechanical Properties of Structural Steel," *National Steel Structures Conference*, Tripoli, Greece.

[7] BungaleS.Taranth, 2004 " Wind and Earthquake Resistant Buildings Structural Analysis and Design", 1stEdition, CRC Press.

[8] Taranath, B. S., Wind and Earthquake, 2005, "Resistant Buildings, Structural Analysis and Design", *Marcel Dekker*, ISBN: 0-8247-5934-6.

[9] Iraqi Standards IQ.301, 2014, "Iraqi Code for forces and loadings ", *Iraqi Ministry of Building and Construction*.

[10] ASCE/SEI 7-05, 2005, " Minimum Design Loads for Buildings and Other Structures", *American Society of Civil Engineers*.

[11] Side Reran and S.H.Mahure, 2014, ” Study of Seismic and Wind Effect on Multi Story R.C.C. Steel and Composite Building” I*nternational Journal of Engineering and Innovative Technology (IJEIT)*, 3(12), June, 78-83.

[12] Chi Kin Iua, Siu Lai Chanb, Xiao XiongZhaa., 2005, "Nonlinear pre-fire and post-fire analysis of steel frames "*Engineering Structures*, Elsevier.

[13] Hideki Kikumoto, RyozoOoka, Hirofumi Sugawara, 2015, "Variations in the power-law index with stabilityandheight for wind profiles in the urban boundary layer." *ICUC9*, June 15.

[14] BS 8110-2:1985, 1985 , “Structural use of concrete - Part 2: Code of practice for special Circumstances".

[15] Krauthammer, T., "A Numerical Study of Wind-Induced Tower Vibrations ", 1987, *Computers & Structures*, Vol. 26, No. 1/2, pp. 233-241.

[16] Smith, C.I., Kirby, B.R., Lapwood, D.G., Cole, K.J., 1981, "Cunningham, A.P. and Preston, R.R., ―There Instatement of Fire Damaged Steel Framed Structures‖" , *Fire Safety Journal*, Vol. 4 No. 1,pp. 21-62.