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Collapse of steel cantilever roof of tribune induced by snow loads

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Abstract. In this paper, it is aimed to present a detail investigation related to structural behavior of laterally unrestrained steel cantilever roof of tribune with slender cross section. The structure is located in Tutak town in Ağrı and collapsed on October 25, 2015 at eastern part of Turkey is considered as a case study. This mild sloped roof structure was built from a variable I beam, and supported on steel columns of 5.5 m height covering totally 240 m^2 closed area in plan. The roof of tribune collapsed completely without any indication during first snowfall after construction at midnight a winter day, fortunately before the opening hours. The meteorological records and observations of local persons are combined together to estimate the intensity of snow load in the region and it is compared with the code specified values. Also, the wide/thickness and height/thickness ratios for flange and web are evaluated according to the design codes. Three dimensional finite element model of the existing steel tribune roof is generated considering project drawings and site investigations using commercially available software ANSYS. The displacements, principal stresses and strains along to the cantilever length and column height are given as contour diagrams and graph format. In addition to site investigation, the numerical and analytical works conducted in this study indicate that the unequivocal reasons of the collapse are overloading action of snow load intensity, some mistakes made in the design of steel cantilever beams, insufficient strength and rigidity of the main structural elements, and construction workmanship errors.

Keywords: collapse; finite element model; roof; snow; steel

1. Introduction

Steel constructions are gained more importance thanks to high strength and stiffness, passing to the long openings, better cycle, fast and easy installation and erection, availability of long cantilevers, uniform ductility etc. Beside these advantages, steel structures should be carefully designed by expert engineers considering some disadvantages such as susceptibility to corrosion, low fire resistance and especially buckling and high deformation due to small and slender cross sections.

With the development of the technology, aesthetic appearance demands and the aim of long span passing bring into the use of huge, cantilever and variable built-up sections in design and construction. Beside these, the cantilever structural elements are particularly vulnerable to snow and wind loads. Compressive stresses due to the bending moments in unbraced and slender elements can lead to local or global buckling. So, wide/thickness and height/thickness ratios for flange and web should be evaluated according to the design codes. Also, the bracing elements should be projected to transfer the vertical and lateral loads into the system to prevent this slenderness.

In the last decade, the structural safety assessment of steel structures have become of increasing concern, probably as a consequence of some dramatic events registered in the world, like the sudden collapse of the public fair pavilion in Italy (Augenti and Parisi 2013), the braced steel frame in a coal-combusting electricity generating factory in China (Tong *et al*. 2009), the store hall and Katowice fair building in Poland (Biegus and Rykaluk 2009, Biegus and Kowal 2013), the space truss roof system and industrial building in Turkey (Caglayan and Yuksel 2008, Ozakgul *et al*. 2011, Piroğlu and Ozakgul 2016), the roof of gymnasium in Switzerland (Piskoty *et al*. 2013), the towers of overhead electrical lines and partial collapse of the Berlin congress hall in Germany (Klinger *et al*. 2011, Helmerich and Zunkel 2014).

In addition, numerical, theoretical and parametric studies are presented to show the buckling effect on the structural responses (Pi and Trahair 1998, Kudzys *et al*. 2009, Coz Diaz *et al*. 2010, Couto *et al*. 2016, Saoula *et al*. 2016, Shi *et al*. 2016). Also, some laboratory tests are conducted on the scale models using normal and high strength steel profiles to validate the finite element results (Ibrahim *et al*. 2015). Also, some experimental, numerical and theoretical studies are presented about the buckling response of structures and elements under variable loads (Jiang *et al*. 2013, Aktas and Balcıoglu 2014, Ghowsi and Sahoo 2015, Kuś 2015, Sofiyev and Kuruoglu 2015, Tagrara *et al*. 2015, Talebi *et al*. 2015). Fig. 1 shows the some views from the collapsed structures.

2. Description of the structure

The investigated steel cantilever roof of tribune is located in Tutak town in Ağrı, Turkey. The tribune with 500 seats is designed for province football team. The tribune

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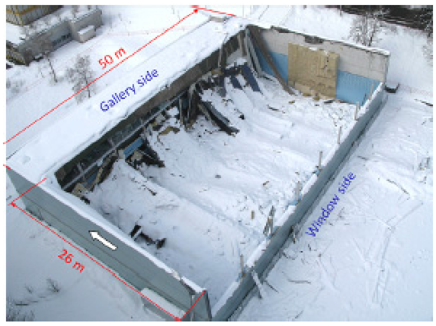
(a) Public fair pavilion in Italy

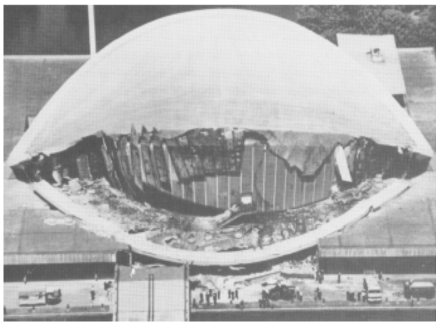
(b) Steel frame in China

(c) Store hall and Katowice fair building in Poland

(d) Space truss roof system and industrial building in Turkey



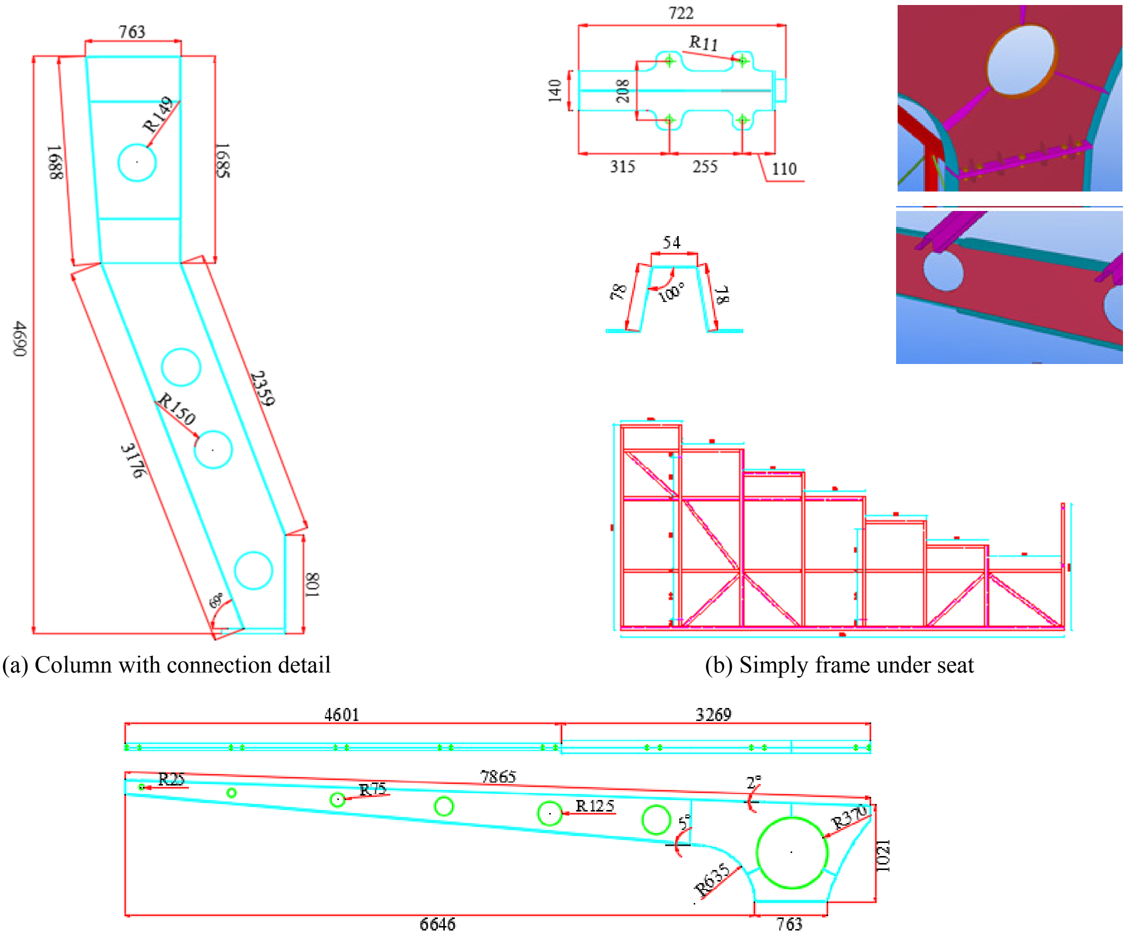
(e) Roof of gymnasium in Switzerland



(f) Collapse of Berlin congress hall

Fig. 1 Some views from the collapsed engineering structures

crosses a span of 30 m × 8 m in plan by covering a 240 m^2 area totally. The structural system of tribune consist of 11 curved columns with 5.50 m height, 11 cantilever beams with 7.87 m length, 8 purlin between two grid with 3 m length and 1.1 m interval, local bracings in the vertical direction, simply frames under seats and scarecrows. The roof of tribune attached on the top of the columns is projected with a slope of 3.5% and the lateral movements in two lateral directions are allowed to move freely, while the vertical displacements are not permitted. Each cantilever has 7 holes with different diameter on web and variable section with 16 cm and 80 cm height at the end point and beam-column connection joints, respectively. Also, top flange width of the cantilever beams has a decreasing trend from the start to end point. Some detail drawings of the designed project are shown in Fig. 2.



(a) Column with connection detail

(b) Simply frame under seat

(c) Cantilever beam section

Fig. 2 Some detail drawings of the designed project

3. Evaluation of the structure according to design codes

For the main structural elements of the cantilever roof of tribune, steel column and cantilever beams sections with varying height from 16 cm to 80 cm and flange width from 14 cm to 7 cm are selected. The related standards (TS648 1980, TEC 2007) suggest in the design that the ultimate strength and yield strength of the steel should be considered as 340 MPa and 235 MPa, respectively. Depending on these, the allowable upper stress limit is defined as 141 MPa (0.6×235 MPa) for the design calculations based on the allowable stress design (TS648 1980, TEC 2007).

According to the compulsory Turkish design codes, a check is required for the cross section properties of main structural elements to avoid potential local buckling failures in bearing members. Thus, depending on the ductility level selected for the roof structure, the section slenderness value for I section (for column and cantilever beam) is determined by

$$\frac{h}{t} \leq \frac{950}{\sqrt{\sigma_a(\sigma_a+1.2)}} \tag{1}$$

where, h is the height of section between two flange and t is thickness of web.

$$\frac{b}{t_b} \leq \frac{25}{\sqrt{\sigma_a}} \tag{2}$$

$$\frac{46}{\sqrt{\sigma_a}} \rangle \frac{b}{t_b} \rangle \frac{25}{\sqrt{\sigma_a}}$$

$$Q_s = 1.415 - 0.0166 \times \frac{b}{t_b\sqrt{\sigma_a}} \tag{3}$$

$$\frac{b}{t_b} \geq \frac{46}{\sqrt{\sigma_a}}$$

$$Q_s = \frac{1400}{\sigma_a \times \left(\frac{b}{t_b}\right)^2} \tag{4}$$

where, b is flange width, t_b is flange thickness and σ_a is yield strength. When exceeding this rate, allowable stress values on compression flange cannot exceed the $0.6 \times \sigma_a \times Q_s$ values.

According to Eq. (1), we obtained the followings

for columns

$$\frac{75.3}{0.5} \le \frac{950}{\sqrt{2.35(2.35+1.2)}} \qquad (\surd)$$

$$150.60 \le 328.91$$

for cantilever beams

$$\frac{46.8}{0.5} \le \frac{950}{\sqrt{2.35(2.35+1.2)}} \qquad (\surd)$$

$$93.6 \le 328.91$$

Thus, it can be seen that the ratio of section web height to its thickness (h/t) is suitable for the related codes. According to Eqs. (2)-(4), we have

for columns

$$\frac{14}{0.5} \le \frac{25}{\sqrt{2.35}} \qquad 28 \le 16.31 \qquad (X)$$

$$\frac{46}{\sqrt{2.35}} \rangle \frac{14}{0.5} \rangle \frac{25}{\sqrt{2.35}}$$

$$Q_s = 1.415 - 0.0166 \times \frac{14}{0.5\sqrt{2.35}} = 1.112$$

and allowable stress values can be calculated as

$$0.6 \times \sigma_a \times Q_s = 0.6 \times 2.35E5 \times 1.112 = 1.568E5 \text{ kN/m}^2$$

for cantilever beams

$$\frac{14}{0.5} \le \frac{25}{\sqrt{2.35}} \qquad 28 \le 16.31 \qquad (X)$$

$$\frac{46}{\sqrt{2.35}} \rangle \frac{14}{0.5} \rangle \frac{25}{\sqrt{2.35}}$$

$$Q_s = 1.415 - 0.0166 \times \frac{14}{0.5\sqrt{2.35}} = 1.112$$

and allowable stress values can be calculated as

$$0.6 \times \sigma_a \times Q_s = 0.6 \times 2.35E5 \times 1.112 = 1.568E5 \text{ kN/m}^2$$

It is seen that allowable stress is decreased nearly 33%. It can be seen that the ratio of flange width to its thickness (b/t_b) is not suitable for the related codes.

Also, this evaluation is carried out according to the American standards (ANSI/AISC 360-10) to better understanding for related researchers. The member properties are classified as non-slender element or slender-element sections. It is stated that the width-to-thickness ratios of its compression elements shall not exceed λ_r for a non-slender element section.

$$\lambda = \frac{b}{t} \le \lambda_r \qquad (5)$$

$$\lambda_r = 0.64\sqrt{\frac{k_c E}{\sigma_a}} \qquad (6)$$

$$k_c = \frac{4}{\sqrt{h/t_w}} \qquad (7)$$

$$0.35 \le k_c \le 0.76 \qquad (8)$$

According to Eqs. (5)-(8), we obtained the followings

for columns

$$\frac{14}{0.5} \le 0.64\sqrt{\frac{0.326 \times 2100}{2.35}} \qquad (X)$$

$$28.0 \le 10.924$$

$$k_c = \frac{4}{\sqrt{75.3/0.5}} = 0.326 \qquad (X)$$

$$0.35 \le 0.326 \le 0.76$$

for cantilever beams

$$\frac{14}{0.5} \le 0.64\sqrt{\frac{0.413 \times 2100}{2.35}} \qquad (X)$$

$$28.0 \le 12.295$$

$$k_c = \frac{4}{\sqrt{46.8/0.5}} = 0.413 \qquad (X)$$

$$0.35 \le 0.413 \le 0.76$$

It can be concluded that that these members cannot ensure the related requirements and can be classified as slender elements.

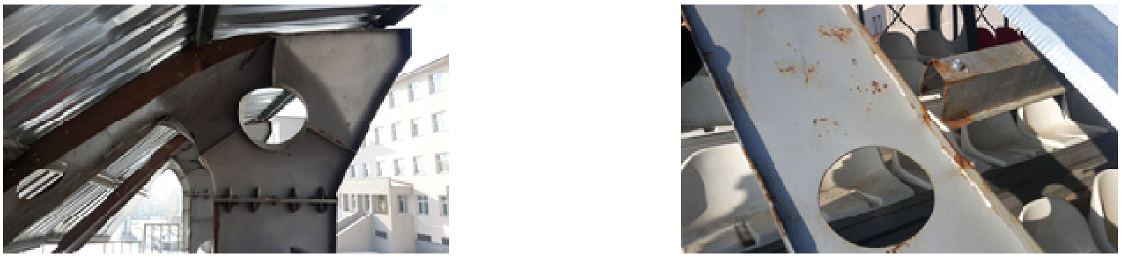
4. Collapsed tribune structure

The laterally unrestrained roof of tribune with slender cross section collapsed completely on October 25, 2015 without any indication during first snowfall after construction at midnight. Based on the information obtained from local people, media and meteorology, it was concluded that the snow load in urban areas had been measured between 80 cm and 90 cm on the ground, the storms existed frequently and the temperatures were measured under -10 °C or even -20°C utmost. Fortunately, there were no injuries sustained as a result of the collapse. The next afternoon, the football match was scheduled with 500 people. Following several investigations of the collapse, it was determined that this was an instance primarily of inadequate structural design and construction errors.

During the site investigations carried on December 11th, it was seen that the cantilever roof had been completely collapsed and 240 m^2 storage areas was affected heavily. Some views from the structure are given in Fig. 3.

The tribune is located in Tutak, Ağrı of the East Anatolia being about 1535 m high from the sea level. Belonging

Fig. 3 Some views of completely collapsed cantilever roof of tribune structure



(a) Incorrect arrangement of column-girder and girder-purlin connection

(b) Connection joints and plastic deformations

(c) Unbounded column-foundation junctions

Fig. 4 Some views from the incorrect arrangements, connections details and workmanship errors observed during site investigations

to the characteristic region IV (TS498 1987, TS EN 1991-1-3 2007) the design ground snow load is properly selected as 1.84 kN/m^2 without evaluating site specific conditions. Based on the mean bulk weight density of snow and assuming 2.0 kN/m^2 for the density of setted snow defined as a snow accumulation in several hours during night the event, the roof snow load value is derived as 1.80 kN/m^2 and 2.00 kN/m^2 corresponding to the 80 cm and 90 cm snow heights measured respectively which are 1.09 times greater than the design value. It is seen that snow load was rather high, but still considerably near the design limit. Therefore, it can be say that the snow could be the trigger of the collapse and can be designated as main reason but not alone reason for this situation. The visual findings and conformity checks pointed towards the buckling of the cantilever girder's stiffener-less web with big height and without any bracing at its supported ends to be the root failure cause.

It was observed from the Fig. 3 that the structural members in the collapsed tribune roof were laterally buckled and permanent plastic deformations were occurred. According to the site investigations, it is observed that the connection joints between column-girder and girder-purlin were not designed and constructed according to the engineering knowledge. There are not any stiffener/sheeting plates around the column-girder connection. Also, the inserts were formed on the alignment in cantilever beam (Fig. 4(a)).

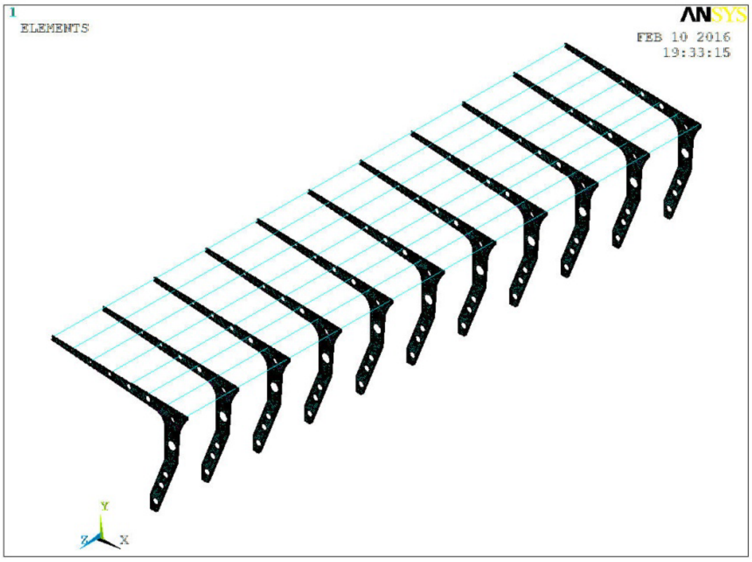
In addition, there are a lot of construction workmanship errors were observed. The joints were produced using overlapping techniques, spot welding and single-screw connections without the considering of static project (Fig. 4(b)). It can be evaluated without any comparison by limit values, the bolts, screws and spot welds were slipped and broken due to the internal forces such as shear forces, bending and torsional moments during the collapse.

Besides, the columns were not fixed to the strip foundation (Fig. 4(c)). According to the static project drawings (some drawings can be seen in Fig. 2), the connection details should be constituted using bolts. But, only welded connections were practiced considering application simplicity.

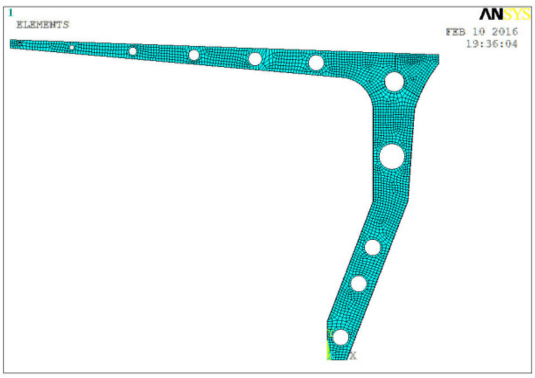
5. Finite element analyses

3D finite element model of the tribune was built in ANSYS (2014) software using the geometrical dimensions shown previously in Fig. 2. A mesh refinement study was also undertaken and a model with 46880 Shell281 and 5824 Beam189 elements was a deemed sufficient for yielding a series of numerically converged solutions. The Shell281 element has 8 nodes, with each node having three translational and rotation degrees of freedom. In defining the boundary conditions, all degrees of freedom under the columns were assumed as fixed. The finite element model of the main structural elements of tribune is given in Fig. 5.

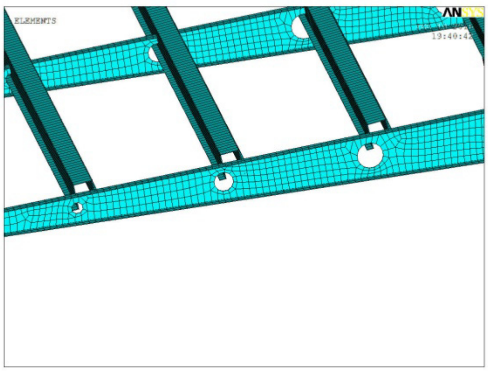
According to the earthquake design code (TEC 2007), the plant location belongs to the 2^{nd} seismic critical region i.e., earthquake acceleration should be 0.3 g, where g is the ground acceleration. Additionally, the importance factor is assumed as 1.0 based on the occupancy or type of the structure. Considering the contribution of live loads on the dead loads of the roof structure so called live load



(a) 3D view



(b) X-Y view of the system



(c) Extruded filling view

Fig. 5 The finite element model of the main structural elements of tribune

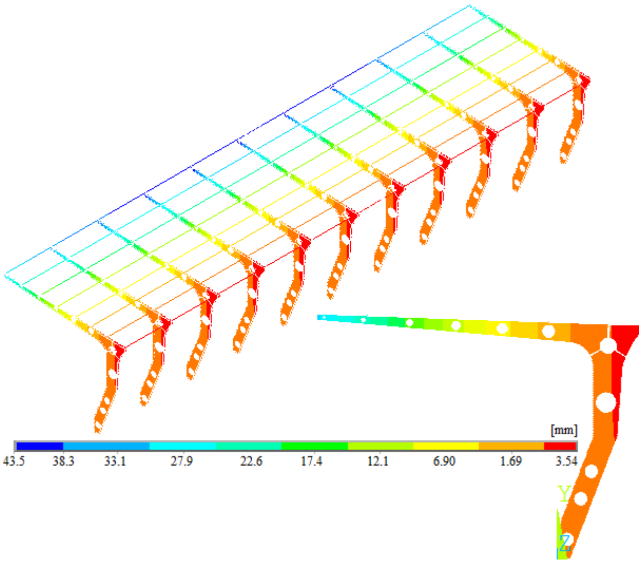


Fig. 6 Maximum displacement contours of the tribune roof under dead and snow load

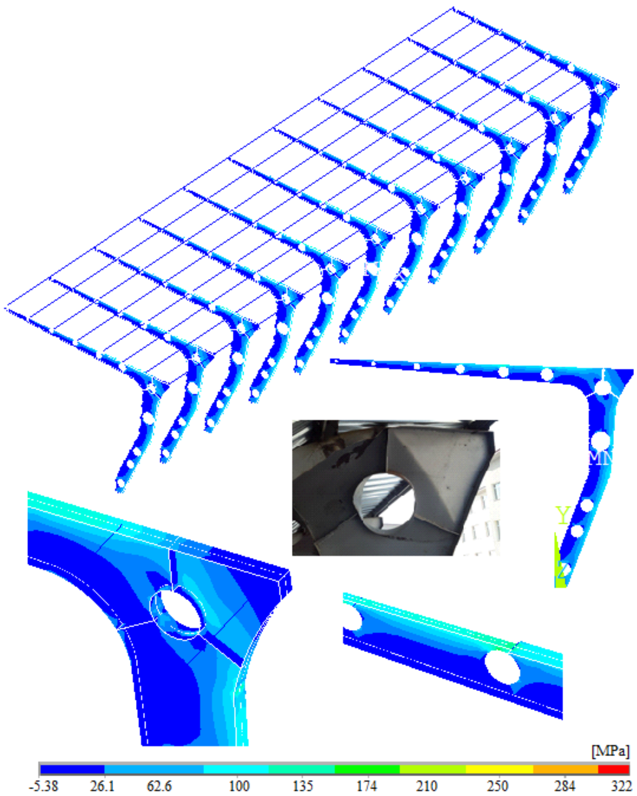


Fig. 7 Maximum tensile stress contours of the tribune roof under dead and snow load

contribution factor is considered as 0.3 in the seismic design.

The temperature change is properly considered as 25°C-30°C according to the continent climate conditions of theplant location. The tribune is located in Tutak, Ağrı of the East Anatolia being about 1535 m high from the sea level. Belonging to the characteristic region IV (TS498 1987, TS EN 1991-1-3 2007), the design ground snow load is properly selected as 1.84 kN/m^2 without evaluating site specific conditions. The parameters given in above should be considered in the design of new structures before the construction.

In this paper, it is thought that this situation is related to overdesign intensive snow load, selection of slender section during static project and construction workmanship errors. So, the structural behavior of the collapsed roof of tribune are tried to analyze under obtained snow data.

The maximum vertical displacements contour diagram of the tribune roof under snow load is shown in Fig. 6. It can be seen from the Fig. 6 that the displacements have an increasing trend from column-beam connection to endpoint of the cantilever. The displacements reach the maximum values at the endpoint of cantilever as 43.50 mm. This value is bigger than the allowable displacement limits in design codes.

$$u_{\max} \leq \frac{L}{250} \quad \text{for cantilever} \quad u_{\max} \leq \frac{7890}{250} = 31.48\ \text{mm}$$

(insufficient design for displacement)

The maximum tensile stress contour diagram of the tribune roof under dead and snow loads is shown in Fig. 7. It is seen from the Fig. 7 that maximum values of the tensile stresses occurred at web surface of column-cantilever connection section, top flange of the beam, back side of the column and section changing points between 210-280 MPa. Also, some local stress accumulations regions are observed with 322 MPa maximum stress value at the transition segment of columns. But, these points are supported by the frames under the seat. So, these local values are not

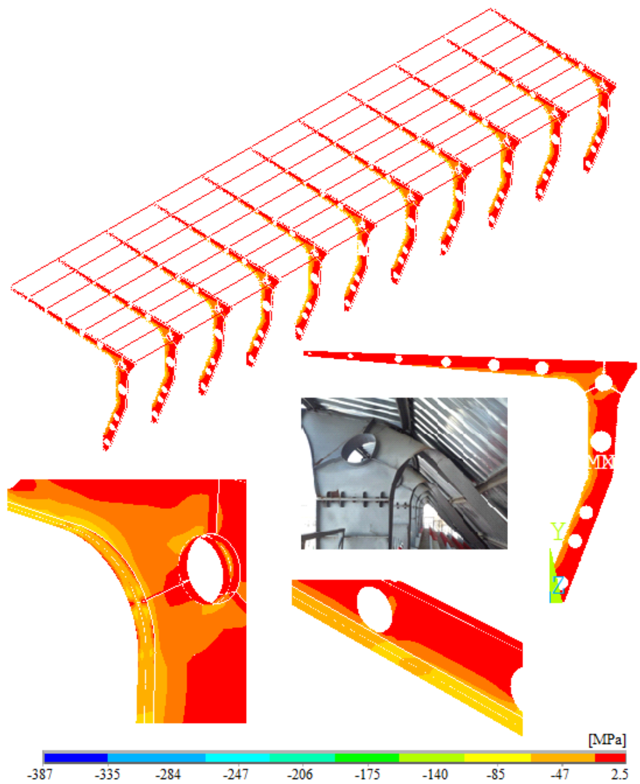


Fig. 8 Maximum compressive stress contours of the tribune roof under dead and snow load

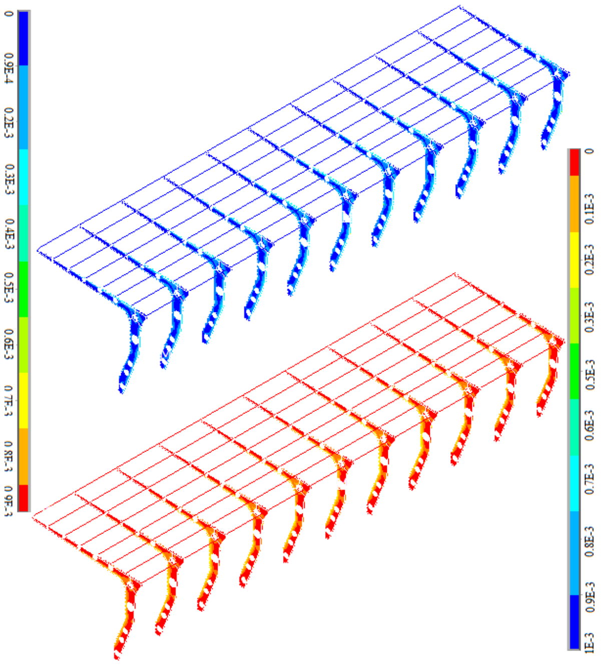


Fig. 9 Maximum tensile and compressive strain contours of the tribune roof under dead and snow load

considered to comparison.

The maximum compressive stress contour diagram of the tribune roof under dead and snow loads is shown in Fig. 8. It is seen from the Fig. 8 that maximum values of the compressive stresses occurred at web surface of column-cantilever connection section around the hole, bottom flange of the beam, front side of the column and section changing points between 150-335 MPa. Also, some local stress accumulations regions are observed with 387 MPa maximum stress value at the transition segment of columns. But, these points are supported by the frames under the seat. So, these local values are not considered to comparison.

The maximum and minimum elastic strains contour diagrams of the tribune roof under dead and snow loads are shown in Fig. 9. It is seen from the Fig. 9 that maximum and minimum elastic strains are attained as 0.90E-3 and 1.00E-3, respectively. Also, there are some strain accumulations regions with 0.30E-3 maximum strain value at the collapsed and damaged regions. Moreover, there are some strain accumulations regions with 0.50E-3 minimum strain value at the transition surfaces on the columns and cantilever beams.

The behavior and evaluation forces are tired to additionally present in graph format in Figs. 10 and 11.

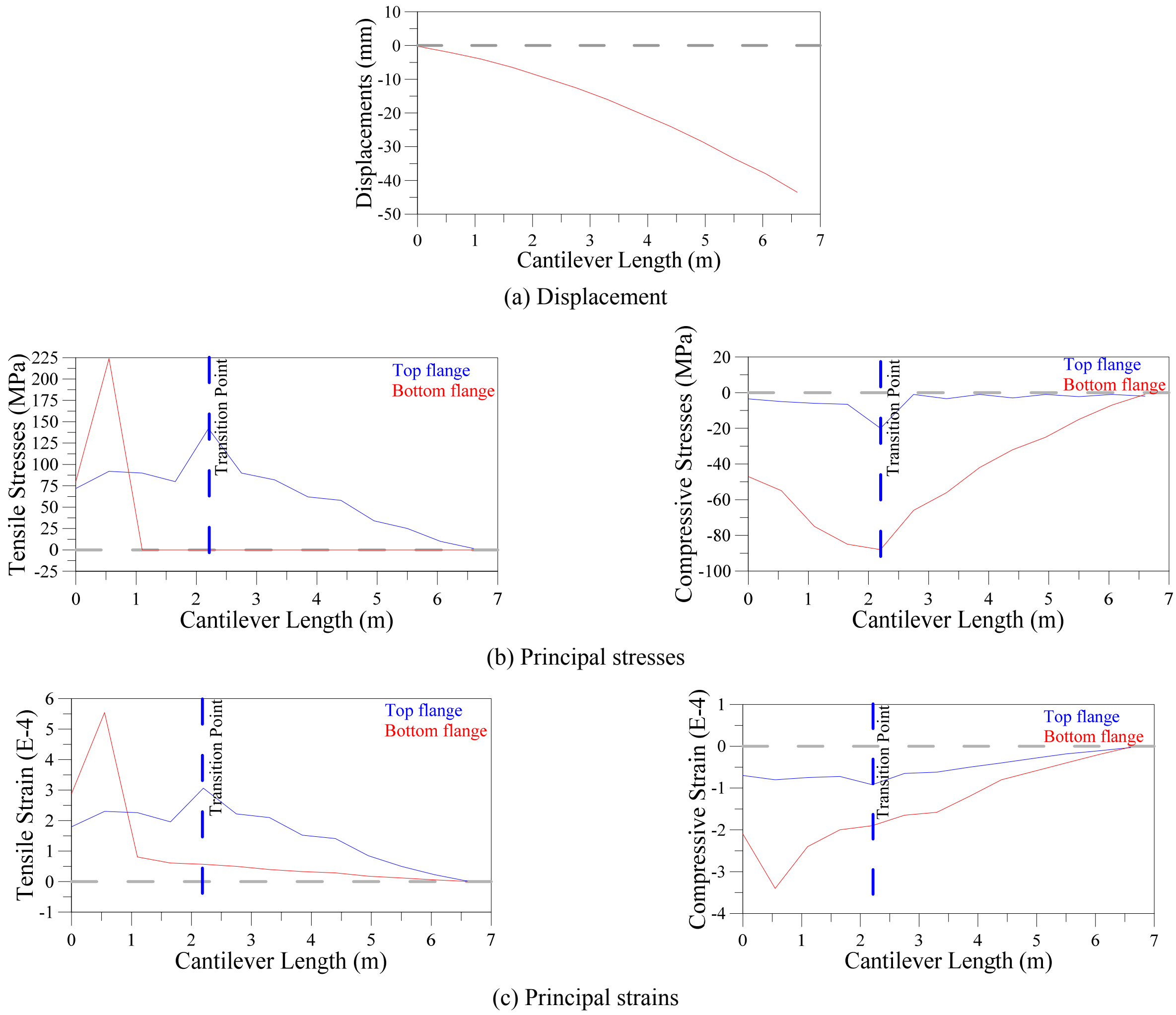
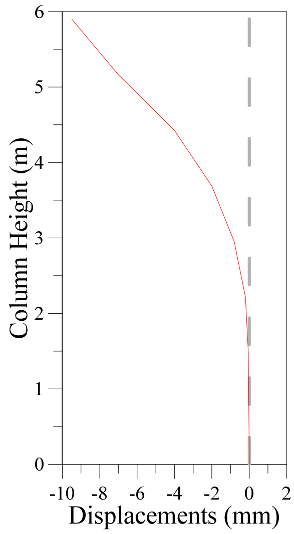
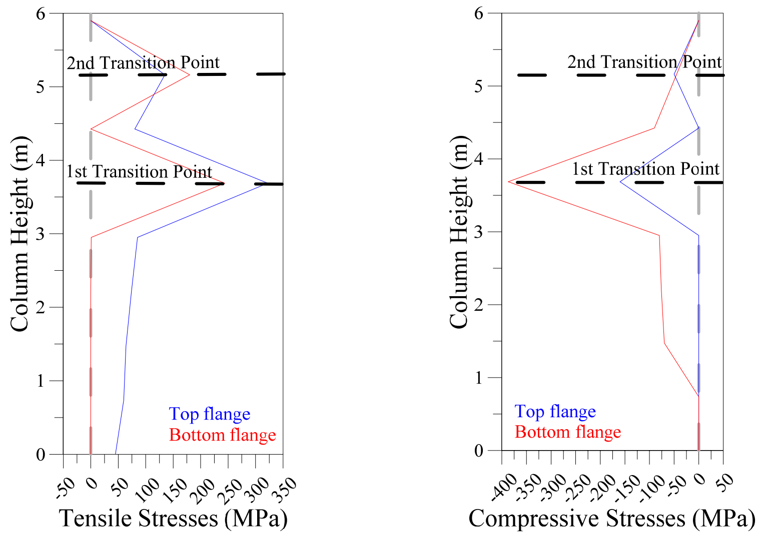


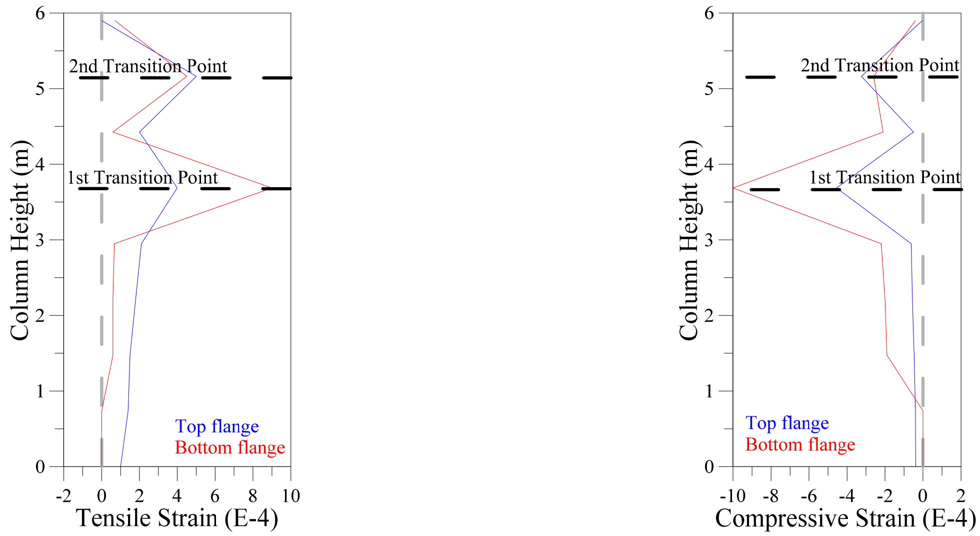
Fig. 10 Changing of displacements, principal stresses and strains with cantilever length



(a) Displacement



(b) Principal stresses



(c) Principal strains

Fig. 11 Changing of displacements, principal stresses and strains with column height

6. Conclusions

In this paper, it is aimed to present a detail visual and numerical study about the completely collapsed of tribune roof due to exceptional snowfalls and insufficient section properties by the combination of construction errors. It is clearly seen that these types of cantilever roof structures without sufficient slop are very sensitive to accumulation, bending and buckling which cause excessive deflections and loss of strength and stiffness of the construction. So, additional stiffness and stability checks are needed to ensure a stable structure without occurrence of buckling and collapses induced by the overloads or underestimated design loads.

At the end of conformity check of the tribune member, it is seen that the ratio of section web height to its thickness (h/t) is suitable for related codes, but the ratio of flange weight to its thickness (b/t_b) is not suitable for related codes.

At the end of the field investigation, it is seen that the roof snow load value is derived as 1.80 kN/m^2 and 2.00 kN/m^2 corresponding to the 80 cm and 90 cm snow heights measured respectively which are 1.09 times greater than the design value. This value is rather high, but still considerably near the design limit. Therefore, it can be say that the snow could be the trigger of the collapse and can be designated as main reason but not alone reason for this situation. The visual findings and conformity checks pointed towards the buckling of the cantilever girder's stiffener-less web with big height and without any bracing at its supported ends to be the root failure cause. The structural members in the collapsed tribune roof were laterally buckled and permanent plastic deformations were occurred. The connection joints between column-girder and girder-purlin were not designed and constructed according to the engineering knowledge. So, the bolts and welds were slipped and broken due to the shear forces, bending and torsional moments during the collapse.

(a)

(b)

(c)

(d)

Fig. 12 Similar damage conditions and collapses in Turkey

In the finite element analyses, only design snow load is considered as 1.84 kN/m^2 to emerge the reason of collapse. Seismic location effect, structure importance factor, changing of temperature, wind and live loads are not taken into account. These parameters given in above must be considered in the design of new structures before the construction. But, it is seen that the values given in related code are not adequate and should be updated to avoid the missing loads.

As a result of analyses, the maximum displacements are attained on the endpoint of cantilever as 43.50 mm. This value is bigger than the allowable displacement limits in design codes. The maximum values of the tensile stresses occurred at web surface of column-cantilever connection section, top flange of the beam, back side of the column and section changing points between 210-280 MPa. The maximum values of the compressive stresses occurred at web surface of column-cantilever connection section around the hole, bottom flange of the beam, front side of the column and section changing points between 150-335 MPa. The maximum and minimum elastic strains are attained as 0.90E-3 and 1.00E-3, respectively. The results show that the displacements, stresses and strains exceed the allowable code limits. Damage and collapse points observed during the field investigation can be verified using contour diagrams.

Finally, it is concluded that this situation is related to intensive snow load, selection of slender section during static project and construction workmanship errors. To avoid similar dramatic events, the design of engineering structures should be done carefully, the connection detail should be prepared sensitively and the construction of structures should be controlled periodically.

Similar damage conditions and collapses have been occurred in Turkey since one and half year, Yomra athletic sport complex (Fig. 12(a)), Güneysu gymnasium (Fig. 12(b)), Kalkandere football tribune (Fig. 12(c)), space roof of Of gymnasium (Fig. 12(d)) and etc. So, it is recommended that the snow loads should be considered as 1.50 kN/m^2 for the I-IV regions up to 700 m high from the sea level, 1.75 kN/m^2 and 2.25 kN/m^2 for the I-II and III-IV regions up to 1000 m, respectively. These values should be increased by 15% after 100 m sea level including engineering safety parameters.

References

Aktas, M. and Balcioglu, H.E. (2014), “Buckling behavior of pultruded composite beams with circular cutouts”, *Steel Compos. Struct., Int. J.*, **17**(4), 359-370.

ANSI/AISC 360-10 (2010), An American National Standard; Specification for Structural Steel Buildings, AISC Committee on Specifications, IL, USA.

ANSYS (2014), Swanson Analysis System, USA.

Augenti, N. and Parisi, F. (2013), “Buckling analysis of a long-span roof structure collapsed during construction”, *J. Perf. Cons. Facil.*, **27**(1), 77-88.

Biegus, A. and Kowal, A. (2013), “Collapse of halls made from cold-formed steel sheets”, *Eng. Fail. Anal.*, **31**, 189-194.

Biegus, A. and Rykaluk, K. (2009), “Collapse of Katowice fair building”, *Eng. Fail. Anal.*, **16**, 1643-1654.

Caglayan, O. and Yuksel, E. (2008), “Experimental and finite element investigations on the collapse of a Mero space truss roof structure-a case study”, *Eng. Fail. Anal.*, **15**, 458-470.

Couto, C., Vila Real, P., Lopes, N. and Zhao, B. (2016), “Numerical investigation of the lateral-torsional buckling of beams with slender cross sections for the case of fire”, *Eng. Struc.*, **106**, 410-421.

Coz Diaz, J.J., Garcia Nieto, P.J., Vilan Vilan, J.A. and Suarez Sierra, J.L. (2010), “Non-linear buckling analysis of a self-weighted metallic roof by FEM”, *Math. Comp. Model.*, **51**, 216-228.

Ghowsi, A.F. and Sahoo, D.R. (2015), “Fragility assessment of buckling-restrained braced frames under near-field earth-quakes”, *Steel Compos. Struct., Int. J.*, **19**(1), 173-190.

Helmerich, R. and Zunkel, A. (2014), “Partial collapse of the

Berlin Congress Hall on May 21st, 1980", *Eng. Fail. Anal.*, **43**, 107-119.

Ibrahim, S.A., Dessouki, A.K. and El-Saeed, S.A. (2015), "Lateral buckling behavior and strengthening techniques of coped steel I-beams", *J. Const. Steel Res.*, **108**, 11-22.

Jiang, L., Qi, J., Scanlon, A. and Sun, L. (2013), "Distortional and local buckling of steel-concrete composite box-beam", *Steel Compos. Struct., Int. J.*, **14**(3), 243-265.

Klinger, C., Mehdianpour, M., Klingbeil, D., Bettge, D., Hacker, R. and Baer, W. (2011), "Failure analysis on collapsed towers of overhead electrical lines in the region Münsterland (Germany) 2005", *Eng. Fail. Anal.*, **18**, 1873-1883.

Kudzys, A., Lukoseviciene, O. and Baltrukenaite-Kroskiene, I. (2009), "Structural safety of rolled and welded beams subjected to lateral-torsional buckling", *J. Civil Eng. Manag.*, **16**, 172-178.

Kuś, J. (2015), "Lateral-torsional buckling steel beams with simultaneously tapered flanges and web", *Steel Compos. Struct., Int. J.*, **19**(4), 897-916.

Ozakgul, K., Caglayan, O. and Tezer, O. (2011), "Investigation of buckled brace system of an existing industrial building", *Eng. Fail. Anal.*, **18**, 455-463.

Pi, Y.L. and Trahair, N.S. (1998), "Out-of-plane inelastic buckling and strength of steel arches", *J. Struct. Eng.*, **124**(2), 174-183.

Piroğlu, F. and Ozakgul, K. (2016), "Partial collapses experienced for a steel space truss roof structure induced by ice ponds", *Eng. Fail. Anal.*, **60**, 155-165.

Piskoty, G., Wullschleger, L., Loser, R., Herwig, A., Tunchschmid, M. and Terrasi, G. (2013), "Failure analysis of a collapsed flat gymnasium roof", *Eng. Fail. Anal.*, **35** 104-113.

Saoula, A., Meftah, S.A., Mohri, F. and Daya, E.M. (2016), "Lateral buckling of box beam elements under combined axial and bending loads", *J. Const. Steel Res.*, **116**, 141-155.

Shi, G., Xu, K., Ban, H. and Lin, C. (2016), "Local buckling behavior of welded stub columns with normal and high strength steels", *J. Const. Steel Res.*, **119**, 144-153.

Sofiyev, A.H. and Kuruoglu, N. (2015), "Buckling of non-homogeneous orthotropic conical shells subjected to combined load", *Steel Compos. Struct., Int. J.*, **19**(1), 1-19.

Tagrara, S.H., Benachour, A., Bouiadjra, M.B. and Tounsi, A. (2015), "On bending, buckling and vibration responses of functionally graded carbon nanotube-reinforced composite beams", *Steel Compos. Struct., Int. J.*, **19**(5), 1259-1277.

Talebi, E., Tahir, M.M., Zahmatkesh, F. and Kueh, A.B. (2015), "A Numerical analysis on the performance of buckling restrained braces at fire-Study of the gap filler effect", *Steel Compos. Struct., Int. J.*, **19**(3), 661-678.

TEC (2007), Turkish Earthquake Code; *Specification for Buildings to be Constructed in Earthquake Areas*, Government of Republic, Ankara, Turkey.

Tong, G.S., Pi, Y.L. and Bradford, M. (2009), "Buckling failure of an unusual braced steel frame supporting an electric dust-catcher", *Eng. Fail. Anal.*, **16**, 2400-2407.

TS EN 1991-1-3 (2007), Eurocode 1. Actions on Structures, Part 1-3: General Actions-Snow loads; TSE Turkish Standards Institution, Ankara, Turkey.

TS498 (1987), *Specification for Design Loads for Buildings*, TSE Turkish Standards Institution, Ankara, Turkey.

TS648 (1980), *Specification for Building Code for Steel Structures*, TSE Turkish Standards Institution, Ankara, Turkey.

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