Case Studies in Engineering Failure Analysis xxx (2016) xxx–xxx

Contents lists available at ScienceDirect

Case Studies in Engineering Failure Analysis

journal homepage: www.elsevier.com/locate/csefa

Case study

# Determining critical areas of transmission towers due to sudden removal of members

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A R T I C L E I N F O

*Article history:*
Received 10 August 2015
Received in revised form 22 September 2015
Accepted 30 September 2015
Available online xxx

*Keywords:*
Progressive collapse
Critical areas
Impact factor
Capacity-to-demand ratio

A B S T R A C T

In this study, the structural susceptibility of a 400 kV power transmission tower subjected to progressive collapse and methods of determining the critical areas of above mentioned structure are investigated. OpenSees program is used for numerical modeling and nonlinear dynamic analysis of the tower which considers the buckling possibility of compression members and the plasticity in the cross sections as well. First, the progressive collapse analysis is performed and the results are reported as time history diagrams. Then, the impact factor of members' removal and the capacity-to-demand ratio are calculated for different failure scenarios of structural members due to the results of preliminary analysis of progressive collapse. The critical areas of the transmission tower through impact factor and capacity-to-demand ratio are determined so that it will be more feasible to propose retrofitting methods for the damaged structure in order to reduce the future risks. For the studied sample transmission tower, impact factors and capacity-to-demand ratios of 41% of APM cases can predict same critical areas.



## 1. Introduction

Progressive collapse has increasingly attracted the attention of civil engineers since the casualty of the Ronan Point residential apartment building in London in 1968 caused by gas explosion [1]. Progressive collapse is the spread of an initial failure from a member to another one which leads to partial or total collapse of a structure [2]. Some of possible abnormal loads which can cause progressive collapse arise from the following events: design errors, heavy object collision, fire, explosion, accidental overload, lack of proper connectivity, etc. [3]. Due to high voltage and thickness of the conductors as well as their considerable weight, failure in structural members of transmission towers such as leg members or bracing ones may occur. This is because of the events such as lightning and other natural catastrophes such as earthquakes which can cause twisting or even collapse of either tower itself or other towers in line. Hence, it is necessary that the safety of transmission towers is accurately investigated from the point view of progressive collapse [4]. The main causes of failure in transmission towers may be cable rupture during storm, improper behavior of a member or a connection and an explosion near tower.

Currently some codes and guidelines have attempted to address the issues of this type of collapse [5]. The American Society of Civil Engineers (ASCE) 7-05 [2] is the standard which describes details of progressive collapse. The GSA [6] and UFC [7] are both progressive collapse analysis and design guidelines which are used to reduce progressive collapse effects

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http://dx.doi.org/10.1016/j.csefa.2015.09.005

in structures after an abnormal loading. The GSA methodology diminishes the potential of progressive collapse in structures based on Alternate Path Method (APM). The method defines scenarios in which one of the members is removed and the damaged structure is analyzed and its response is studied. Powell [8] utilized various analysis procedures and found that the impact factor of 2 regulated in the linear static analysis can display very conservative result. Ruth et al. [9] found that a factor of 1.5 better represents the dynamic effect especially for steel moment frames. Kim and Kim [10] studied the progressive collapse-resisting capacity of steel moment frames by using APM recommended in the GSA and UFC guidelines and observed that when a nonlinear dynamic analysis was conducted, it led to larger structural responses. Fu declared that under the same general conditions, a member removal at an upper level will induce larger vertical displacement than a member removal at ground level [11,12]. El Kamari et al. [13] studied the deflection of a part of the Roissy Charles de Gaulle Airport and compared the results with the ones measured on site and the ones predicted by the design. They simulated a progressive collapse by reducing the rigidity of the elements which yielded and explained the different incidents of the collapse. They also concluded that construction defects, poor quality of the concrete and an improper design caused the collapse. Siriwardane [14] proposed a simple technique to locate the damage region of railway truss bridges based on measured modal parameters. As this periodical modal parameter measurement was based only on acceleration under usual moving train loads, it was concluded that the proposed method could provide an inexpensive damage locating technique. Al-Bermani and Kitipornchai [15] have applied advanced nonlinear analysis for transmission towers. They emphasized that the problem is more complicated by the spatial nature of the configuration and by the fact that individual components are asymmetric angle shapes that are eccentrically connected. Then, the elements undergo uniaxial loading and biaxial bending effects, which are impossible to model using conventional 3D elastic truss-type methods. In Robert and Lemelin [16], nonlinear analysis results are compared between a truss model and a frame model where the tower legs are represented with frame elements while the secondary bracing members (redundant members ignored in linear analysis) were also taken into consideration. Both models yielded similar results but the frame method is more preferable. Ramesh et al. [17] discussed the dynamic effects of progressive member failure of truss structures. Their method is to replace the damaged member by the adequate external force functions at its end joints. They observed that sudden buckling failures can cause significant stress redistributions near adjacent members and might cause a second member failure, and possibly trigger progressive collapse.

Since the transmission towers are so sensitive structures and their failure can cause considerable financial and life losses, it is obligatory to check the different parts of the tower. The aim of this study is to a some method for determining the critical areas of a 400 kV transmission tower so that the related members may be retrofitted in specific occasions. Impact factors and capacity-to-demand ratios are calculated for various failure scenarios and for different elevation levels of removal. The obtained ratios are used to appraise the critical areas of the structure.

## 2. Modeling the 400 kV power transmission tower

The configuration and members' sections of the 400 kV suspension tower is shown in Fig. 1. This structure, a 400 kV tower designed for 0–2° line deviation, is selected as a sample structure for studying the progressive collapse in this research [18]. Steel with yield stress of 255 MPa is considered for all members. Nonlinear finite element software OpenSees [19] is used for modeling and analyzing the structure. A kinematic two-line stress–strain curve is considered for modeling the behavior of steel in the elements. According to Fig. 2, strain hardening of 1% is considered for inelastic phase. Nonlinear beam-column elements with fiber sections are considered in modeling of cross sections. In addition, the effect of large deformations is taken into account using corotational transformation in geometric stiffness matrix. Having some information such as conductor data, weather data, wind and weight spans, insulator data and other factors, the loading of the tower of interest is calculated and the loads are applied to the structure to get the results [20,21]. In addition to the main model used in this study, an elastic model of the prototype structure was constructed using SAP2000 structural analysis software [22]. In this simple model, elements were modeled as elastic beam-column members. It will be discussed that the results from this model are used to validate the OpenSees model in the elastic range.

### 2.1. Verification

The selected model for the verification is a similar but different structure taken from the article of Prasad Rao et al. [18]. In the study by Prasad Rao et al., the axial forces for a few structural elements during some tests were reported to study the failures in detail. These force amounts were presented for 75% of the design loads, namely the failure load of the tower in the test. They used NE-NASTRAN software [23] to model the above mentioned 400 kV tower, and also to obtain the values of axial forces at the legs of the first panel, the legs of the second panel, and the bracing members in first panel. The values of axial forces in these members taken from different models are presented in Table 1. As observed, the first column of Table 1 represents the reported forces by Prasad Rao et al. [18]. The values obtained from both SAP2000 and OpenSees models in this study are in close agreement with the reported values [18]. The percentage errors between the first column and the other two are also incorporated into Table 1. In addition, comparable displacements at different joints, as well as natural periods of vibration are obtained from the two models in this study. The natural period of the first

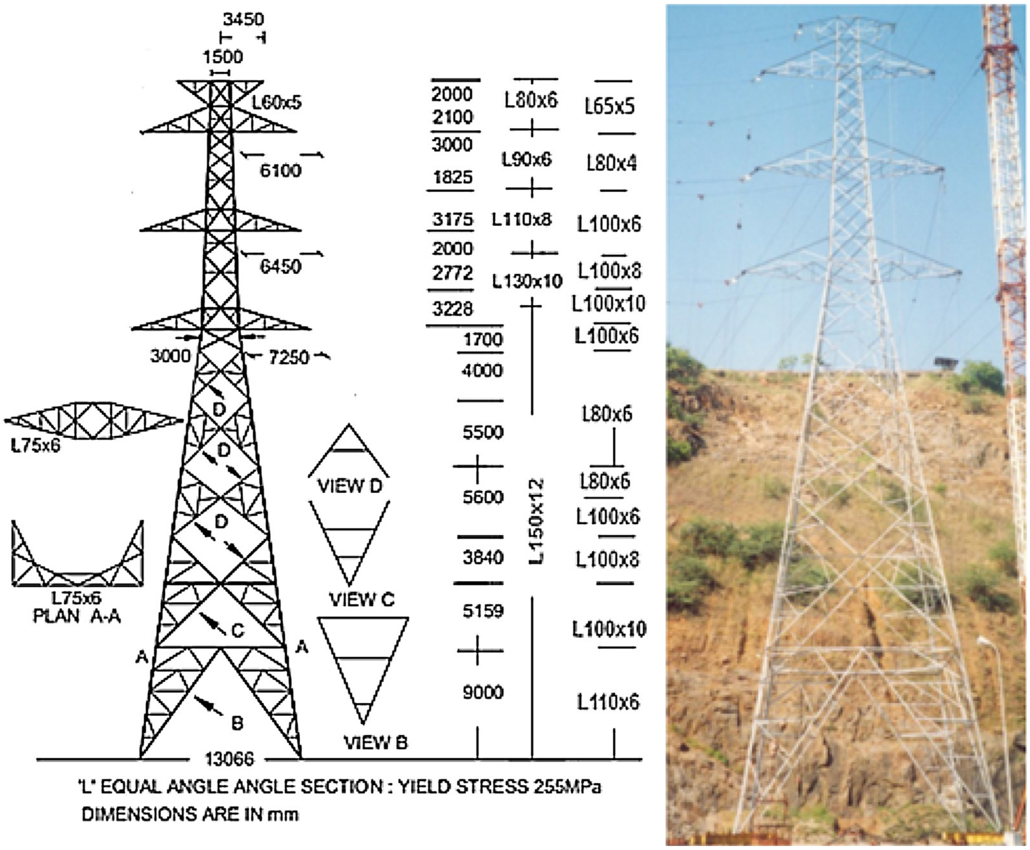


**Fig. 1.** Configuration of the 400 kV transmission tower.

mode taken from different software is 0.54. Table 2 compares the displacement of a node at the top of the structure in different software used in the present study.

## 3. Progressive collapse analysis

As the three dimensional model of transmission tower is complicated, Fig. 3 displays some views of it to illustrate the different members of the structure through removal scenarios. In this research the gravity loads are linearly increased during

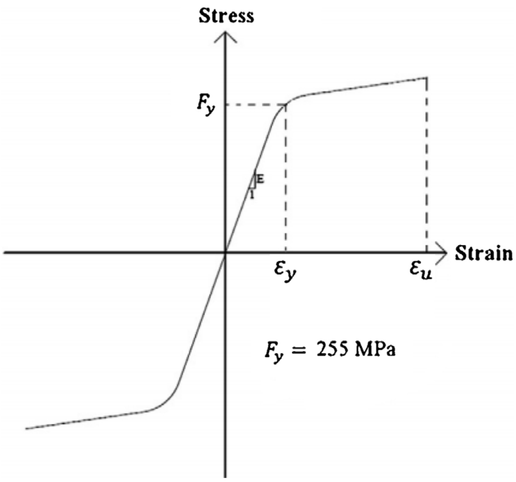


**Fig. 2.** Stress–strain curve of structural steel.

Please cite this article in press as: Eslamlou SD, Asgarian B. Determining critical areas of transmission towers due to sudden removal of members, Case Stud Eng Fail Anal (2016), http://dx.doi.org/10.1016/j.csefa.2015.09.005

**Table 1**
Verification of the results of the program.

| Non-linear analysis force at test failure load in kN | | | Percentage errors (%) | | Failed member location |
|---|---|---|---|---|---|
| (A) NE-NASTRAN | (B) Sap2000 | (C) OpenSees | A&B | A&C | |
| 520 | 517 | 522.14 | 0.58 | 0.41 | Leg in first panel |
| 505 | 495 | 502.67 | 2.02 | 0.46 | Leg in second panel |
| 35 | 35 | 34.7 | 0 | 0.86 | Bracing in first panel |

**Table 2**
Displacement at top of the tower.

| | SAP2000 | OpenSees |
|---|---|---|
| Transverse | 312.4 mm | 306.2 mm |
| Longitudinal | 0.019 mm | 0.023 mm |
| Vertical | 45.7 mm | 43.15 mm |

5 s to reach ultimate values, and then are maintained unchanged for 2 s to avoid exciting dynamic effects. Once the gravity loads have been fully applied at 7th second, the related elements to the APM case are removed, and the subsequent response of the structure as well as the behavior of it are then investigated. The simulations are conducted with 2% mass and stiffness proportional damping. The results of progressive collapse analysis are presented in the form of time history charts. As shown in Fig. 4, the horizontal axis indicates time in seconds and the vertical axis is the axial force in Newton. Time history diagrams can be drawn for all failure scenarios, including the removal of the legs, the bracing members and the conductors. The axial force of the members before removal of selected ones, the maximum axial force that usually occurs after removing such members which is called as the maximum demand on the time history charts, are extracted from the above mentioned time history charts. Table 3 shows the list of APM analysis cases considered in this study together with the members which are removed in each case.

In Fig. 5(a), the impact of sudden removal of leg members on their adjacent ones are compared through time history diagrams. The members of interest have been chosen from different heights, and the scenarios for the legs and the braces separately are shown in increasing order in terms of height of removal. Consequently, it is feasible to check the effect of elevation level of removal on the diagrams. The axial force of L3 in the APM case 1 spikes from 139 kN to a peak value of 873.5 kN before settling down at a steady value of 446.7 kN. It can be concluded that in the removal scenarios of leg members, the lower the elevation level of removal is, the more the adjacent members are affected. However, it is not so wise to only check the max values of time history charts to determine the critical areas of the tower. Fig. 5(b) displays the time history of axial load due to sudden removal of bracing members with different elevation level. It seems that the failure scenario No. 12 has the greatest impact on the adjacent members in comparison with the other bracing members' scenarios of failure. The axial force of B40 in the APM case 12 spikes from 12.7 kN to a peak value of 15.7 kN before settling down at a steady value of 13.5 kN.

## 4. Determination of the critical areas of 400 kV transmission tower

There are some methods to determine more susceptible areas of transmission towers to progressive collapse using impact factors and capacity-to-demand ratios after element removal. As mentioned before, time history charts cannot help to exactly determine the critical areas of the transmission tower. However, the axial force of the members before removal of selected ones, the maximum axial force that usually occurs after removing such members which is called the maximum demand on the time history charts, are extracted from the above mentioned time history charts. As a result, such data are used for obtaining the impact factor and the capacity-to-demand ratio which better predict the critical areas of the tower.

### *4.1. Impact factor of members' removal*

In order to calculate this factor, the maximum of axial force of a member in the case of nonlinear dynamic analysis of sudden removal of another member is divided by its axial force before removal. That is, the maximum demand is divided by the steady state value of time history charts. Eq. (1) displays how to calculate the above mentioned factor.

$$\mathrm{IF} = \frac{\text{Peak Value of Internal Force}}{\text{Steady Value of Internal Force (Before Removal)}} \tag{1}$$

Fig. 6 displays impact factor values for adjacent braces in different cases of removal of legs and bracing members. Note that the scenarios of failure are chosen from different elevation levels so that there are three removal scenarios from both leg members and bracing ones. The same diagram could be drawn for adjacent legs in different scenarios of failure.

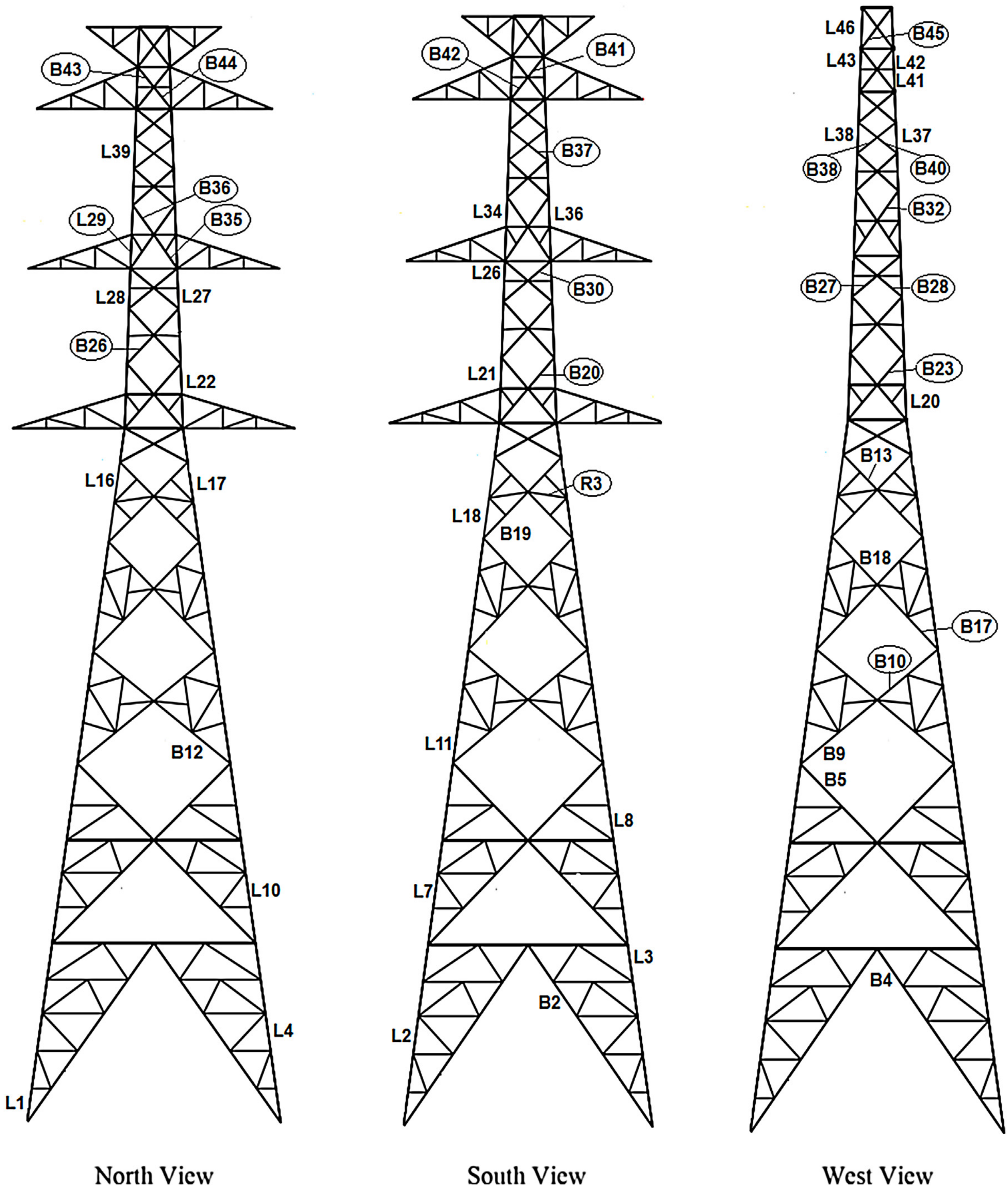


**Fig. 3.** Different views of the suspension tower.

Fig. 7(a) compares the maximum values of impact factors for leg and bracing members in the cases of legs removal. Besides, Fig. 7(b) compares the maximum values of impact factors for leg and bracing members in the cases of removal of bracing members. Note that in Fig. 7(a) and (b), the horizontal axis is corresponding to different APM cases, and the vertical one displays the maximum of impact factors calculated in each removal scenario.

In Figs. 8(a) and (b), the impact of increasing the elevation level of removal is investigated on the max IF. Fig. 8(a) displays the variations of max IF by increasing the elevation level of legs removal. That is, all of the failure scenarios are chosen out of the APM cases of legs. However, Fig. 8(b) displays the variations of max IF by increasing the elevation level of bracing members' removal. The max IF almost decreases by increasing the elevation level of legs removal for both leg and bracing

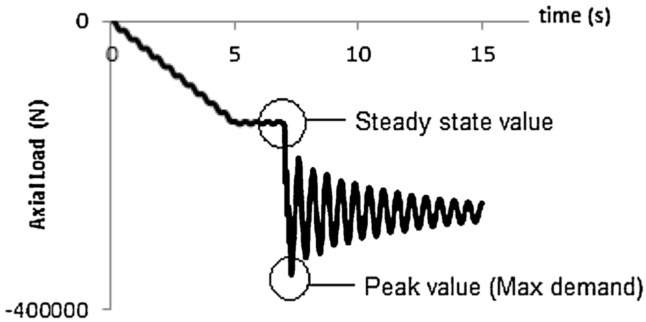


**Fig. 4.** A sample for time history of axial load.

**Table 3**
List of removal scenarios.

| APM case/scenario | Removed member type | Elements removed |
|---|---|---|
| 1 | Leg | L1 |
| 2 | Leg | L10 |
| 3 | Leg | L16 |
| 4 | Leg | L22 |
| 5 | Leg | L27 |
| 6 | Leg | L39 |
| 7 | Leg | L43 |
| 8 | Brace | B2 |
| 9 | Brace | B10 |
| 10 | Brace | B19 |
| 11 | Brace | B28 |
| 12 | Brace | B37 |

members. However, the max IF increases by increasing the elevation level of bracing members' removal while investigating on adjacent leg ones. There are not any considerable variations for adjacent bracing members in the same diagram.

### 4.2. Detection of capacity-to-demand (CoD) ratio

Another method of determination of the critical areas of the desired tower is the capacity-to-demand ratio. For calculating the capacity-to-demand ratio, the investigated member's capacity is divided by the maximum amount of axial force from nonlinear dynamic analysis result of the member. The least value of the mentioned above ratio may be a good sign of the crittical area especially when the ratio is less than 1. Eq. (2) displays how to calculate the CoD ratio.

$$\mathrm{CoD} = \frac{\text{Member's Nominal Capacity}}{\text{Peak Value of Internal Force}} \tag{2}$$

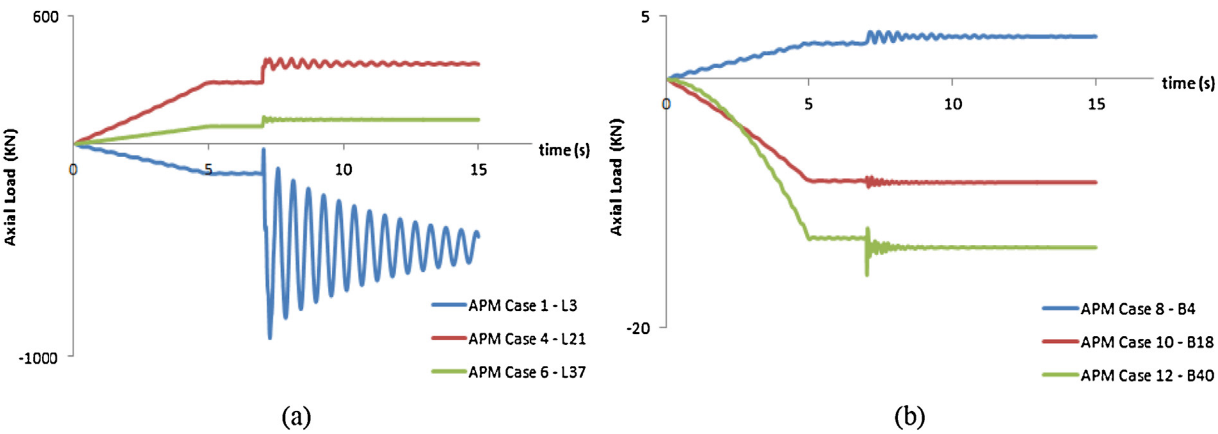


**Fig. 5.** (a) Time history of legs' removal and (b) time history of brace members' removal.

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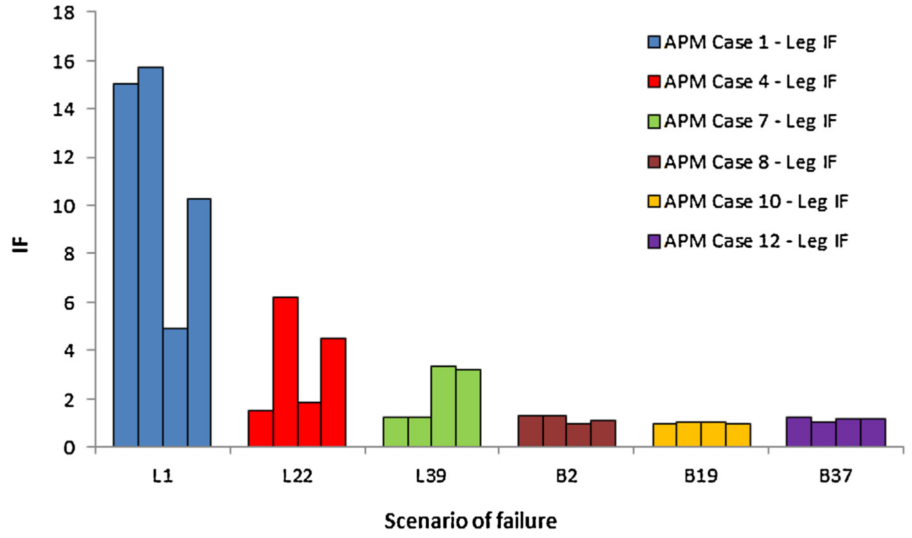


**Fig. 6.** Impact factor values for brace members in various APM cases.

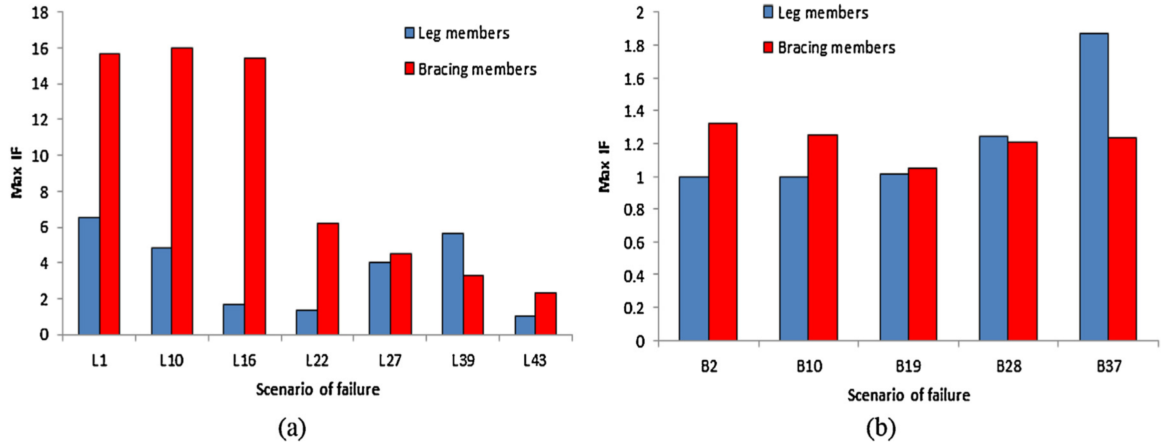


**Fig. 7.** (a) Max IF values during removal scenarios of legs and (b) max IF values during removal scenarios of brace members.

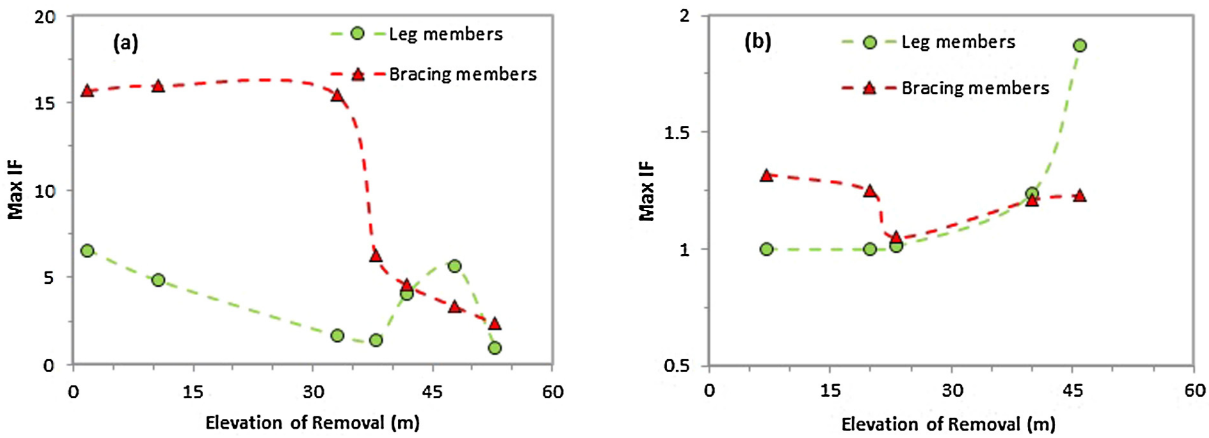


**Fig. 8.** (a) Max IF vs. elevation level of removal during removal of legs and (b) Max IF vs. elevation level of removal during removal of brace members.

Fig. 9 displays the values of capacity-to-demand ratio for adjacent leg members in different cases of removal of leg and bracing members. The significant point is that the scenarios of failure are chosen from different elevation levels so that there are three removal scenarios from both leg members and bracing ones to be compared. The same diagram could be drawn for adjacent bracing members in different scenarios of failure.

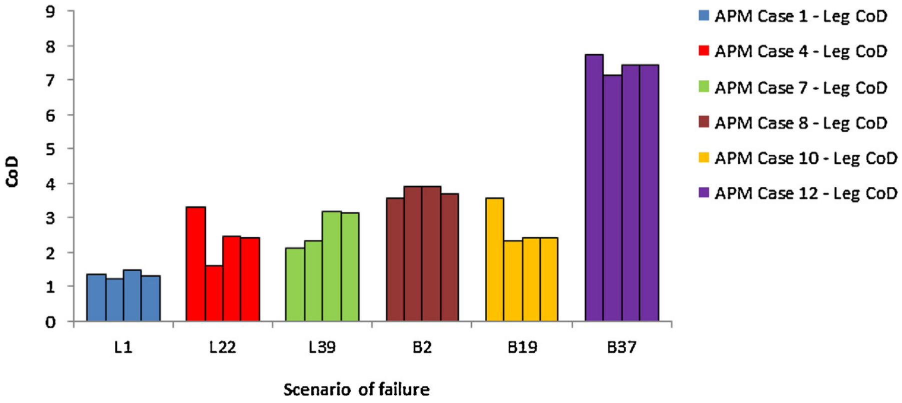


**Fig. 9.** CoD ratios for leg members during various APM cases.

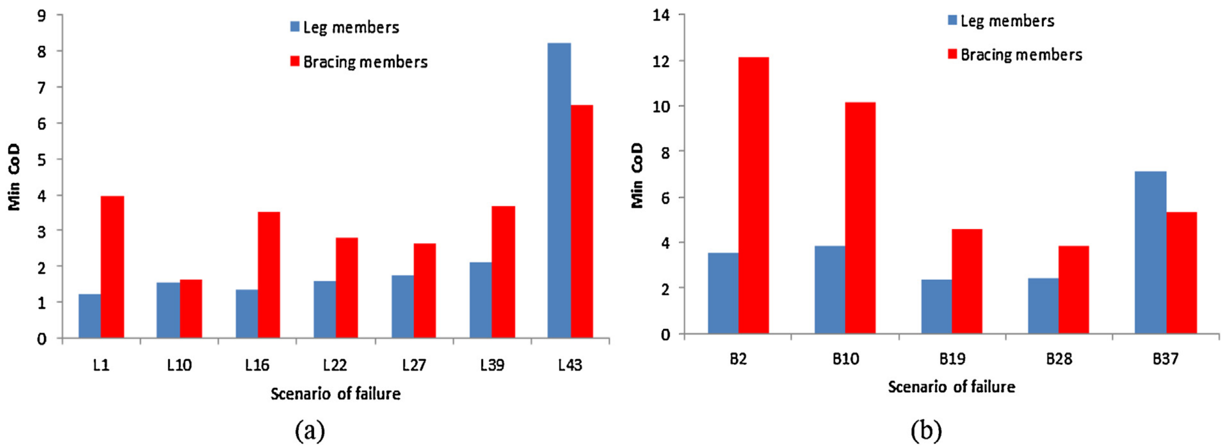


**Fig. 10.** (a) Min CoD values during removal scenarios of legs and (b) Min CoD values during removal scenarios of brace members.

In Fig. 10(a), the minimum values of CoD ratio for leg and bracing members in the cases of removal of legs are compared. Also, Fig. 10(b) compares the minimum values of CoD ratio for leg and bracing members in the cases of bracing members' failure. Note that in Figs. 10(a) and (b), the horizontal axis is corresponding to different APM cases, and the vertical one displays the minimum of capacity-to-demand ratios calculated in each removal scenario.

Figs. 11(a) and (b) illustrate the effect of the elevation level of removal on minimum of CoD values. Fig. 11(a) displays the variations of min CoD by increasing the elevation level of legs removal so that all of the failure scenarios are chosen out of the APM cases of legs. However, Fig. 11(b) shows the variations of min CoD by increasing the elevation level of bracing members'

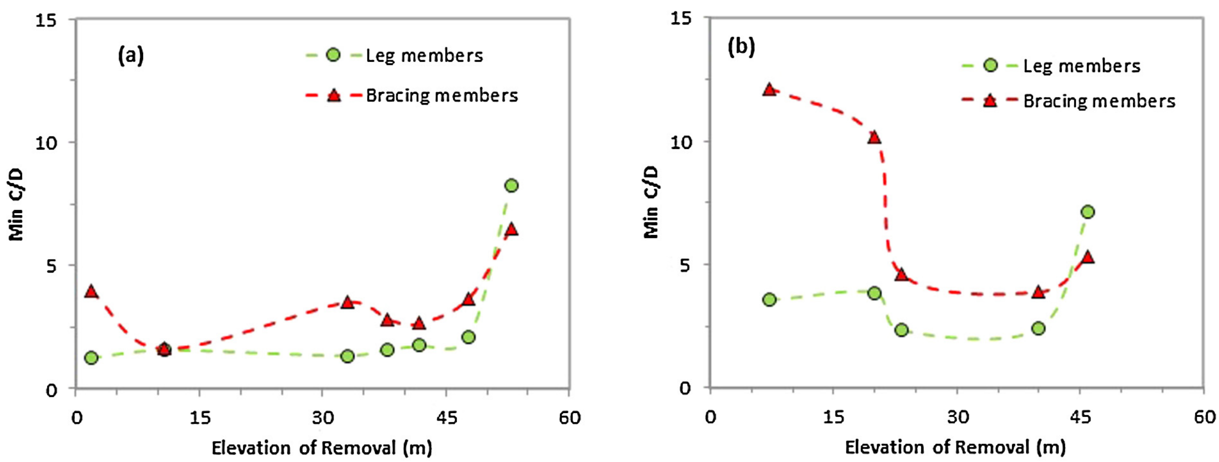


**Fig. 11.** (a) Min CoD vs. elevation level of removal during removal of legs and (b) Min CoD vs. elevation level of removal during removal of brace members.

removal. The minimum of CoD almost increases by increasing the elevation level of legs removal for both leg and bracing members. However, the minimum of CoD decreases by increasing the elevation level of bracing members' removal while investigating on adjacent bracing ones. There is only a considerable increase for the leg members in the last part of the same diagram which improves safety.

## 5. Conclusion

Transmission towers are susceptible to progressive collapse. In this paper, progressive collapse analysis of a sample transmission tower is carried out and factors such as impact factors and capacity-to-demand ratios are calculated for various scenarios of failure and for different elevation levels of removal. The maximum IF value decreases by increasing the elevation level of legs removal. This means that the maximum value of this factor is in lower elevations in the APM cases of legs removal. Furthermore, the minimum CoD increases by increasing the elevation level of legs removal so that it corresponds with the above mentioned control. The max IF values increase by increasing the elevation level of bracing members' removal in adjacent legs. However, the minimum CoD remains almost unchangeable, and their values are more than critical limit. Consequently, there is not any risk of progressive collapse in that area. Table 4 compares the members with max IF values and the members with minimum CoD values. As observed, there is a list of members for each of the APM cases. The maximum values of IF and the minimum values of CoD occur in the above mentioned members. In some of the APM cases, the maximum IF and the minimum CoD are observed in the same members so that such members are the most critical members of the tower which needs more attention. However, other cases show different members for maximum IF and minimum CoD values. Since the CoD ratio uses the capacity of the member to determine the critical areas of the tower, it is most likely to get

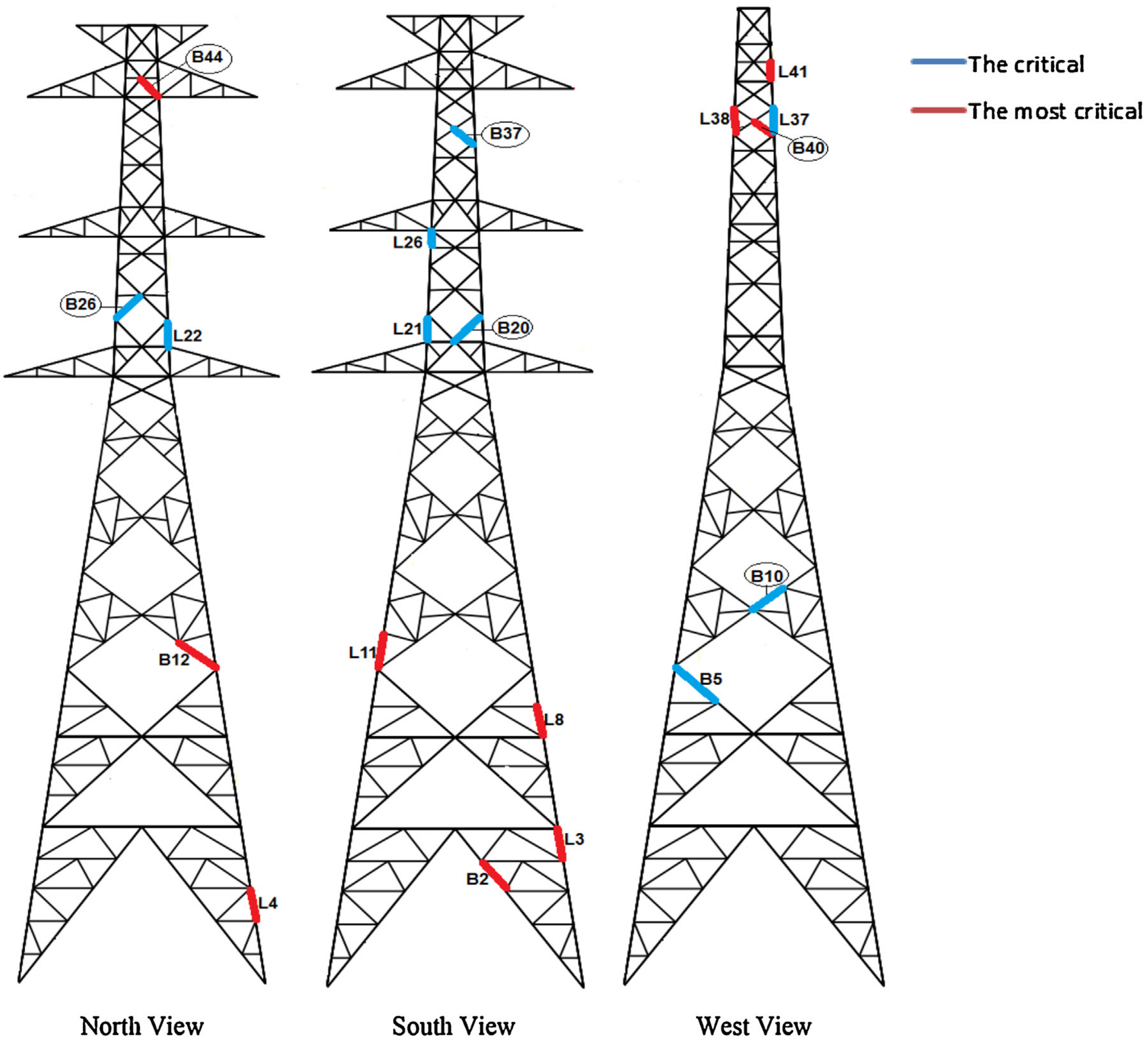


**Fig. 12.** The most critical and the critical members of the tower.

**Table 4**
Comparison of critical members taken from max IF and min CoD values.

| Scenarios/APM case | Maximum IF | | Minimum CoD | |
|---|---|---|---|---|
| | Leg members with max IF | Bracing members with max IF | Leg members with min CoD | Bracing members with min CoD |
| L1 | L3 | B2 | L3 | B2 |
| L10 | L8 | B9 | L8 | B10 |
| L16 | L20 | B13 | L21 | B20 |
| L22 | L20 | B23 | L21 | B26 |
| L27 | L38 | B30 | L26 | B37 |
| L39 | L38 | B41 | L37 | B37 |
| L43 | L41 | B44 | L41 | B44 |
| B2 | L4 | B4 | L4 | B5 |
| B10 | L11 | B12 | L11 | B12 |
| B19 | L18 | B18 | L22 | B20 |
| B28 | L28 | B27 | L26 | B37 |
| B37 | L38 | B40 | L38 | B40 |

more accurate results from this ratio in comparison with the impact factor. Consequently, the remaining members in the fourth and the fifth columns of Table 4 stand for the critical elements. By comparing the values of impact factor and capacity-to-demand ratio in cases of both leg and bracing members' removal, it can be concluded that in almost 41% of scenarios, both factors can predict the same critical areas of the tower. Fig. 12 displays the critical elements and the most critical ones highlighted in different colors. There are of course other methods to determine the critical and sensitive areas of such structures for which a broader study is needed.

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