Evaluation of Seismic Performance of Steel Lattice Transmission Towers

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Abstract

The electricity transmission systems are an important lifeline for modern societies. They are used for overhead power lines as supporting structures. Transmission towers are designed to meet electrical and structural requirements. They are designed according to the weight of conductors and environmental effects such as wind and ice loads. They also considered other extraordinary stresses such as cable breakage and ice-breaking effects. Because of a common perception that transmission line (TL) towers show low sensitivity to earthquakes, the effects of the earthquake in TL tower construction are not considered. For this reason, TL towers are investigated with regard to the seismic performance in this study. The principal objectives of this research are: i) to assess the sensitivity of typical TL towers to earthquake loads, ii) to retrofit an existing steel lattice tower using a new section Centre To Center (CTC). In this study, a finite element model of a representative 154 KV transmission tower in Turkey was performed using a set of 10 recorded earthquake ground movements. The four-legged square TL tower has been analyzed and designed for Turkey, Eskisehir seismic zone considering 42.95 m height using finite element (FE) software. Therefore, a new section Centre To Center (CTC) type has been designed and the failed sections have been replaced with a designed section using the SAP2000 section designer. The results show that the load of failure increased after retrofitting. The retrofitting method was effective and easily conducted in fields.

Keywords: Earthquake Damage; Non-linear Dynamic Analysis; Retrofitting; Seismic Evaluation; Transmission Tower.

1. Introduction

A transmission steel tower is a high-rise structure, known as an electricity tower. It used to carry overhead lines [1]. Electrical engineering advancement shows the need to support heavy conductors that led to the current existing towers. Transmission line towers are high rise-structures, the height of well-above the side dimensions. These are space frames made of steel profiles, which have a separate foundation for every leg. The customer sets the elevation of the transmission tower and the engineer designs the overall configuration, element, and connection details.

The power generated in power stations was transmitted via transmission power lines and transported by transmission power line towers. The transmitting power line towers cost 35 to 45% of the overall expenditure of the transmission network. So, the largest economy must be achieved in its design and installation [2]. Despite the advanced sensor technology and seismological academic efforts, earthquakes stay unpredictable.

In Turkey, where most of the total population lives in urban areas, for instance, Eskisehir city, which is in northwestern Turkey, is a quiet active zone. Eskisehir’s city center lies in the 2nd and 3rd-degree earthquake zones.
The experience of powerful earthquakes in populated areas will have major catastrophic consequences of damage to buildings and residential areas. The physical damage of earthquakes to structures causes minor effects, for example, power, water, or gas outages. Most times, the secondary impact causes more socioeconomic losses than losses from straight structural damage [3].

For the indispensable dependence on electricity in modern society, transmission networks must cover many regions [4]. The ultimate tested capacity is crucial to power transmission systems that is recognized in society as lifeline construction [5]. In the designing stage of a transmission tower, we consider wind loads to be the controlling side load which overrides a load of earthquakes since the transmission tower is in principle light structures. For instance, the National Electrical Safety Code (2012) and in the United States ASCE guidelines (2009) demand the wind impact to be taken into consideration in the design, instead of the earthquake effect.

Therefore, the cross-section of structural elements is determined by taking into account the weight of the tower, the tension in the transmission cable, and the wind load. Despite that, we have noticed that the transmission tower is exposed to severe earthquakes [6]. The lateral displacement of transmission towers, which can be defined as a tall structure, should be limited to acceptable levels under seismic loads. It is essential to have an excellent understanding of the transmission tower’s seismic vulnerability to show proper earthquake management [7]. The primary goal will offer a more comprehensive understanding of performing lattice steel transmission towers under earthquake load.

We organize the paper as follows; in Section 2 we present the literature review and a brief discussion of several earlier studies. Section 3 describes the method used in this article, the structural features of the transmission line tower that are selected to study, the selection method of ground motion records, and the nonlinear time-history analysis conducted to find the ability of the tower under earthquake ground motions. Section 4 presents a design using AISC360-16 and the failed sections replacement with a designed section. In the last section, we discuss the results.

2. Scientific Literature Review

In response to that mentioned in the Introduction, extensive investigations have been carried out in recent decades to study the seismic performances of transmission tower systems, including analytical, experimental [8], and numerical [9] studies. An alternative model of material that can recognize the non-linear behavior of steel members, when axial cyclic loads applied, was developed further in [10] and applied to stimulate the ongoing failure of TL towers during earthquakes [11]. However, the limited studies [12] have aimed to study the seismic risk evaluation of the TL tower. Wu and Pantelides (2019) carried out a study that presents the seismic evaluation of deficient bridge bent under nonlinear static and dynamic analysis in which a nonlinear time history analysis was conducted in the study to evaluate the seismic performance. The research shows that the method used for the repair of RC bridge bents was effective [13].

Moon et al. (2009) performed a semi-scaled substructure test to assess the behavior and the fault position of an existed transmission tower exposed to wind loads. He observed from the experiment that local regional buckling occurred on the two legs elements where they had been exposed to compression. To avoid associated regional buckling and irregular deformation, the elements must be enlarged or braces must be added to weak joints [14].

Alam and Santhakumar [15] carried out a test of load on a 34-meter elevated transmission tower under a 200 kV capacity. They discovered that the total bending of the tower legs and the transverse components caused the tower to fail. In the results of the tests, they proposed reduction of the highest slenderness ratio of 150 to 110, as stated in the steel transmission tower’s design code [16]. We can find few works in the literature of seismic behavior of transmission towers.

Lei and Chien (2009) examined the dynamic behavior of a coupled tower conductor-system during powerful earthquake movements. They implemented a detailed finite-element model of structural towers and the overhead line, taking into account the geometric nonlinearities and the interaction model of a soil-structure in the numerical simulations. The authors stated that neglecting the contribution to overhead lines of the earthquake reaction to support towers would lead to errors in estimating the largest strength of supporting tower elements [17].

Chen et al. (2014) summarized the state-of-the-art on dynamics and control of vibration in transmission tower systems. In the article, they carried out and discussed the investigations into dynamic reactions to transmission line towers under ice, wind, and earthquake load conditions. Considering the seismic behavior of transmission towers, the authors made out the following conclusion: a) The transmission lines spans are large compared to today’s civil engineering constructions, so the effects of multiple excitations must be in details; b) The common failure of transmission line tower systems show that the load patterns given in the codes still do not represent extreme load conditions, for example, those resulting from seismic ground movements and the designed method based on them. The static analysis is a restricted and dynamic analysis of the interaction of Tower-line systems [18].
3. Method

3.1. Description of the Structural Model

The selected tower was a suspension (tangent) tower. It is used for straight runs or with a line deviation of 0° to 2°. The model of the tower bracing is a Diamond lattice system as seen in Fig.1. This tower model has been selected because it is one of the most used models. The sections used for the tower are steel angles made from S355 which is a non-alloy European standard (EN 10025-2) structural steel. Fig.2 displays the finite element model of the studied TL system which comprises three towers and four spans of transmission lines. As seen, the system is symmetrical with the middle spans and spans on the side of 400 m and 200 m.

The transmission tower has been modeled on AutoCAD and then imported to SAP2000 (SAP 2000, computer, and structures). We chose this program because of its dynamic analysis capabilities. The transmission tower is fixed at the base. The height of the tower is 42.95 meter and the base width of the tower is 4 meters. Figures 3 and 6, show the geometry of the 154 KV tower.
3.2. Steel and Section Profiles

For the aim of this study, the members have been modeled on the size of the profiles and the grade in steel for guaranteed realistic contributions to the dead load on the models. In this study, analysis of steel connections has not been considered, so elements including holes of bolts or bolts have been omitted. Figure 4 shows the Dummy bars used in the vertical plane.

The tower used in this study was modeled with different steel equal angles (L.-profile) sizes. Table 1 lists the member sections used in the study. Figure 5 shows instances of L-profile arrangements.
Figure 6. The geometry of 154 kV lattice tower (all units in millimeters)
Table 1. List of member profiles used in the study

| Section                | No. | Profile size     | Unit Weight (Nm²) | Area (mm²) | Radius of Gyration Rx = Ry (mm) | Ix=Iy (cm⁴) | Wx=Wy (cm²) |
|------------------------|-----|------------------|-------------------|------------|---------------------------------|-------------|-------------|
| Peak legs              | 1   | L60x60X5         | 44.83             | 581.9      | 18.245                          | 19.4        | 4.45        |
|                        | 2   | L120x120X11      | 195.22            | 2537       | 36.641                          | 340.6       | 39.41       |
|                        | 3   | L120x120X11      | 195.22            | 2537       | 36.641                          | 340.6       | 39.41       |
|                        | 4   | L120x120X11      | 195.22            | 2537       | 36.641                          | 340.6       | 39.41       |
|                        | 5   | L120x120X11      | 195.22            | 2537       | 36.641                          | 340.6       | 39.41       |
|                        | 6   | L150X150X14      | 310               | 4004       | 46.308                          | 845.4       | 78.33       |
| Cage legs              | 8   | L50X50X5         | 36.99             | 480.3      | 15.106                          | 11          | 3.05        |
|                        | 10  | L80X80X7         | 83.28             | 1082       | 24.355                          |             |             |
|                        | 12  | L70X70X7         | 72.50             | 939.7      | 21.214                          | 42.4        | 8.43        |
|                        | 14  | L100X100X10      | 147.54            | 1900       | 30.78                           | 177         | 24.7        |
|                        | 16  | L80X80X7         | 83.28             | 1082       | 24.355                          |             |             |
| Cage Primary Bracing   | 18  | L60x60X5         | 44.83             | 581.9      | 18.245                          | 19.4        | 4.45        |
|                        | 20  | L80X80X7         | 83.28             | 1082       | 24.355                          |             |             |
|                        | 22  | L70X70X7         | 72.50             | 939.7      | 21.214                          | 42.4        | 8.43        |
|                        | 24  | L100X100X8       | 119.49            | 1551       | 30.555                          | 145         | 20          |
|                        | 26  | L70X70X7         | 72.46             | 939.7      | 21.214                          |             |             |
| Cage Horizontal Bracing| 32  | L60x60X5         | 44.83             | 581.9      | 18.245                          | 19.4        | 4.45        |
|                        | 33  | L80X80X10        | 116.35            | 1511       | 24.064                          | 87.7        | 15.5        |
| Top Cross Arm Beams    | 37  | L70x70X6         | 62.69             | 812.7      | 21.302                          | 36.9        | 7.27        |
| Middle Cross Arm Beams | 38  | L100X100X10      | 147.54            | 1915       | 30.376                          | 177         | 24.7        |
| Bottom Cross Arm Beams | 44  | L60x60X6         | 53.17             | 684        | 18.468                          | 22.9        | 5.25        |
|                        | 45  | L80X80X7         | 83.28             | 1082       | 24.355                          |             |             |
| Main legs              | 49  | L150X150X14      | 310.00            | 4004       | 46.308                          | 845.4       | 78.33       |
|                        | 50  | L150X150X14      | 310.00            | 4004       | 46.308                          | 845.4       | 78.33       |
|                        | 51  | L180X180X16      | 426.74            | 5504       | 55.681                          | 1682        | 129.7       |
|                        | 52  | L180X180X18      | 476.77            | 6191       | 54.9                            | 1866        | 144.7       |
|                        | 53  | L200X200X18      | 531.70            | 6911       | 61.336                          | 2600        | 180.6       |
|                        | 54  | L200X200X20      | 587.91            | 7635       | 61.107                          | 2854        | 199.3       |
|                        | 60  | L200X200X20      | 587.91            | 7635       | 61.107                          | 2854        | 199.3       |
| Leg Primary Bracings   | 55  | L120X120X11      | 195.22            | 2537       | 36.641                          | 340.6       | 39.41       |
|                        | 56  | L100X100X12      | 174.91            | 2271       | 30.169                          | 207         | 29.2        |
|                        | 57  | L100X100X10      | 147.54            | 1900       | 30.78                           | 177         | 24.7        |
|                        | 58  | L100X100X10      | 147.54            | 1900       | 30.78                           | 177         | 24.7        |
|                        | 59  | L100X100X8       | 119.49            | 1551       | 30.555                          | 145         | 20          |
|                        | 60  | L100X100X8       | 119.49            | 1551       | 30.555                          | 145         | 20          |
|                        | 614 | L100X100X8       | 119.49            | 1551       | 30.555                          | 145         | 20          |
| Leg Horizontal Bracing | 610 | L60x60X5         | 44.83             | 625        | 20.217                          | 19.4        | 4.45        |
|                        | 510 | L70X70X6         | 62.69             | 812.7      | 21.302                          | 36.9        | 7.27        |
|                        | 410 | L60x60X5         | 44.83             | 625        | 20.217                          | 19.4        | 4.45        |
|                        | 310 | L70X70X6         | 62.69             | 812.7      | 21.302                          | 36.9        | 7.27        |
|                        | 110 | L80X80X7         | 83.28             | 1082       | 24.355                          |             |             |
| Secondary Bracings     | a   | L35X35X3         | 15.70             | 203.7      | 10.605                          | 2.3         | 0.9         |
|                        | b   | L40X40X4         | 23.74             | 307.9      | 12.052                          | 4.49        | 1.55        |
|                        | c   | L45X45X4         | 26.98             | 349.3      | 13.568                          | 6.47        | 1.97        |
|                        | d   | L50X50X4         | 30.12             | 389.3      | 15.182                          | 9.01        | 2.47        |
|                        | e   | L50X50X5         | 36.98             | 480.3      | 15.106                          | 11          | 3.05        |
|                        | f   | L60x60X5         | 44.83             | 581.9      | 18.245                          | 19.4        | 4.45        |
3.3. Material Properties

The TL tower designed according to S355 for each structural member has been shown in Table 2.

| Property                  | Symbol | Unit   | Value      |
|---------------------------|--------|--------|------------|
| Modulus of elasticity     | E      | (MPa)  | $210 \times 10^3$ |
| Yield strength            | $F_y$  | (MPa)  | 355        |
| Ultimate strength         | $F_u$  | (MPa)  | 510        |
| Poisson’s ratio           | $\nu$  | (-)    | 0.3        |

3.4. Load Cases

The loading calculations on the tower, according to the Turkish Standard International Electrical Commission (TS IEC) 60826 standard, and a set of load cases have been taken into consideration for the transmission tower design in this study. This article concerned with static and dynamic earthquake loads applications, so we calculated the load cases involving wind and ice load cases and broken wire conditions.

When defining wind direction, the following terminology was used: transversal for loads which act on the tower side; longitudinal direction for loads which act on the tower face (in the line’s direction); vertical direction for loads that act downwards or upwards. Figure 7 illustrates these terms and conventions.

![Figure 7. Load direction conventions](image)

In all cases, several load effects on the TL tower have been analyzed to simulate the actual properties of the wind angles and other load cases. Table 3 shows the examined load cases. We have defined the details of the 20 load cases as combinations of different loads as seen in Figure 8.

| Case # | Load case description                                      |
|--------|------------------------------------------------------------|
| 1      | Unstable loads                                             |
| 2      | Unstable loads                                             |
| 3      | Right top and middle conductor coupling                    |
| 4      | Right top conductor closing single circuit                 |
| 5      | Right middle and bottom conductor coupling                 |
| 6      | Right middle conductor closing single circuit              |
| 7      | Right top and bottom conductor coupling                    |
8. Right bottom conductor coupling single circuit
9. Earth wire broken
10. Earth wire broken
11. Even wind
12. One vertical wind single circuit
13. Maximum angle drawing
14. Maximum angle drawing
15. Unbalanced icing
16. Unbalanced icing
17. Down opening
18. Down opening
19. Net up lifting opening
20. Down opening

Load case description: 1. UNSTABLE LOADS
Case #: 1

Load case description: 2. UNSTABLE LOADS
Case #: 2

Load case description: RIGHT TOP AND MIDDLE CONDUCTOR COUPLING
Case #: 3

Load case description: RIGHT TOP CONDUCTOR CLOSING SINGLE CIRCUIT
Case #: 4
Load case description: RIGHT MIDDLE AND BOTTOM CONDUCTOR COUPLING
Case #: 5

Load case description: RIGHT MIDDLE CONDUCTOR CLOSING SINGLE CIRCUIT
Case #: 6

Load case description: RIGHT TOP AND BOTTOM CONDUCTOR COUPLING
Case #: 7

Load case description: RIGHT BOTTOM CONDUCTOR COUPLING SINGLE CIRCUIT
Case #: 8

Load case description: 1. EARTH WIRE BROKEN
Case #: 9

Load case description: 2. EARTH WIRE BROKEN
Case #: 10
| Load case description | Case # |
|-----------------------|--------|
| EVEN WIND             | 11     |
| ONE VERTICAL WIND SINGLE CIRCUIT | 12 |
| 1. MAXIMUM ANGLE DRAWING | 13 |
| 2. MAXIMUM ANGLE DRAWING | 14 |
| 1. UNBALANCED ICING   | 15     |
| 2. UNBALANCED ICING   | 16     |
3.5. Details of Seismic Parameters

The design spectrums created and scaled the ground motions. They were selected to fit the design spectrum within the duration of interest [20]. We should note that a place with high acceleration spectral values (e.g. Eskisehir Osmangazi University (ESOGU)) has been maintained as shown in Table 4.

| Province  | Location | Seismic Data |
|----------|----------|--------------|
|          |          | (S0) | (S1) | (PGA) | (PGV) |
| Eskisehir| ESOGU    | 0.703 | 0.186 | 0.298 | 17.934 |

This study used a set of ten ground motion registers from the Pacific Earthquake Engineering Research (PEER) Ground Motion Database (2019). In the current study, we take the time series with the greatest peak of ground acceleration for the horizontal component into account. We describe these ground motions data and parameters of concern for the structural dynamic analysis in Table 5, which presented the coefficients Fs and Fv specified in Turkey Earthquake Hazard Interactive Web Application for site class ZA. Table 6 presents 10 selected Earthquake records. Therefore, Figure 9 shows the acceleration-time diagrams of the selected records motions from PEER Ground Motion Database (2019).
Table 5. Coefficients Fs and Fv for site class ZA

| Local Soil Effect Coefficient for the short period region, Fs | Local Soil Effect Coefficient for 1.0 second period, F1 |
|-------------------------------------------------------------|---------------------------------------------------|
| $S_r \leq 0.25$ | $S_r = 0.20$ |
| 0.8 | 0.8 |
| $S_r = 0.25$ | $S_r = 0.30$ |
| 0.8 | 0.8 |
| $S_r = 0.50$ | $S_r = 0.50$ |
| 0.8 | 0.8 |

Table 6. Earthquake records selected – horizontal component

| Record No. | Earthquake | Date       | Station                  | Record Duration | Fault type |
|------------|------------|------------|--------------------------|-----------------|------------|
| P0856      | Landers    | 28.06.1992 | 21081 Amboy              | 50              | SS         |
| P0969      | Northridge | 17.01.1994 | 24611 LA- Temple & Hope  | 40              | RN         |
| P0859      | Landers    | 28.06.1992 | 32057 Baker Fire Sation  | 50              | SS         |
| P1108      | Kocaeli, Turkey | 17.08.1999 | Mecidiyekeyo               | 44              | SS         |
| P0740      | Loma Prieta | 18.10.1989 | 57064 Fremont- Mission San Jose | 39.9 | RO         |
| P0903      | Northridge | 17.01.1994 | 24157 LA – Baldwin Hills  | 40              | RN         |
| P0988      | Northridge | 17.01.1995 | 90009 N.Hollywood – Goldwater Can | 21.9 | RN         |
| P1100      | Kocaeli, Turkey | 17.08.1999 | Goyunk                    | 25.5            | SS         |
| P0763      | Loma Prieta | 18.10.1989 | 1686 Fremont – Emerson Court | 39.7 | RO         |
| P0818      | Landers    | 28.06.1992 | 5070 North Palm Springs  | 70              | SS         |
Ten of those records are typical of soil class ZA which has a soil profile distinguished by dense soil and hard rock. Every record of the ground motions examined in the study has to be scaled for the sake of matching the defined design spectrum at the structural fundamental period (T1). Turkey Earthquake Building Code (TBDY-2018) provisions need that the mean of the 5% damped response spectra of ground motion records should match or be above the target spectra over the defined interval. According to TBDY-2018, the average of 5% damped for soil movement records over the defined interval must match or exceed the target spectra. Figure 10 shows the Horizontal Elastic Design Spectrum which was obtained from the following relationship:

**Design Spectral Acceleration Coefficients**

\[ S_{DS} = S_D F_S = 0.703 \times 0.8 = 0.562 \]
\[ S_{D1} = S_1 F_1 = 0.186 \times 0.8 = 0.149 \]

- **S\(_{DS}\)**: Short period design spectral acceleration coefficient.
- **S\(_{D1}\)**: Design spectral acceleration coefficient for the second period.
\[ S_{ae}(T) = \begin{cases} 0.4 + 0.6 \frac{T}{T_A} S_{DS} & (0 \leq T \leq T_A) \\ S_{DS} & (T_A \leq T \leq T_B) \\ \frac{S_{DS}}{T^2} & (T_B \leq T \leq T_L) \end{cases} \]

\[ S_{ae}(T) = \frac{S_{D1} T_L}{T^2} & (T_L \leq T) \]

\[ T_A = 0.2 \frac{S_{D1}}{S_{DS}}, \quad T_B = \frac{S_{D1}}{S_{DS}}, \quad T_L = 6s \]

\[ T_A = 0.053(s), \quad T_B = 0.265(s), \quad T_L = 6.00(s) \]

**Figure 10. Horizontal Elastic Design Spectrum**

The current study included this provision to scale the selected records of soil movement. Figure 11 shows the scale acceleration response spectrum of the ZA class. It has been noticed that only the frequency content was scaled for the acceleration response spectrum. Figure 12 presents the flowchart of the adopted method.

**Figure 11. Design response spectrum for 5% damping, site Class ZA (TBDY-2018), and the response spectra of selected scaled records**
4. Results

4.1. Natural Frequency Analysis

We carry out the model analysis for the design using the eigenvector method. Once directing the modular analysis, it is guaranteed that the outcomes incorporate a sufficient degree of structural mass that has been done by remembering enough modes for the analysis to catch 90% of the mass participation at every one of the three displacement directions. The mass participation proportion shows the level of the basic mass for the model taking part in providing guidance and mode. It appears in Table 7 that mode 330 is achieved.

Table 7. Modal mass participation ratios

| Mode No. | UX   | UY   | UZ   | SumUX | SumUY | SumUZ |
|----------|------|------|------|-------|-------|-------|
| 1        | 0.000| 0.520| 0.000| 0.000 | 0.520 | 0.000 |
| 2        | 0.536| 0.000| 0.000| 0.536 | 0.520 | 0.000 |
| 3        | 0.000| 0.000| 0.000| 0.536 | 0.521 | 0.000 |
| 4        | 0.000| 0.237| 0.000| 0.536 | 0.757 | 0.000 |
| 5        | 0.000| 0.000| 0.001| 0.536 | 0.757 | 0.001 |
| 6        | 0.250| 0.000| 0.000| 0.786 | 0.757 | 0.001 |
| 7        | 0.000| 0.000| 0.002| 0.786 | 0.757 | 0.003 |
| 8        | 0.000| 0.000| 0.000| 0.786 | 0.757 | 0.003 |
| 9        | 0.000| 0.000| 0.000| 0.786 | 0.757 | 0.003 |
| 10       | 0.000| 0.000| 0.008| 0.786 | 0.757 | 0.011 |
| 11       | 0.000| 0.005| 0.000| 0.786 | 0.763 | 0.011 |
| 25       | 0.019| 0.009| 0.000| 0.920 | 0.867 | 0.019 |
| 150      | 0.000| 0.000| 0.000| 0.991 | 0.991 | 0.704 |
| 239      | 0.000| 0.000| 0.000| 0.994 | 0.994 | 0.827 |
| 330      | 0.000| 0.000| 0.001| 0.996 | 0.995 | 0.900 |
Table 8 shows the modal periods and natural frequencies of the modes listed in Table 7.

| Mode No. | Period (s) | Frequency (Hz) | Shape       |
|----------|------------|----------------|-------------|
| 1        | 0.410      | 2.434          | Longitudinal|
| 2        | 0.400      | 2.500          | Transversal |
| 3        | 0.133      | 7.517          | Torsion     |
| 4        | 0.123      | 8.123          | Longitudinal|
| 5        | 0.118      | 8.456          | Vertical    |
| 6        | 0.114      | 8.756          | Transversal |
| 7        | 0.088      | 11.326         | Vertical    |
| 8        | 0.088      | 11.328         | Vertical    |
| 9        | 0.088      | 11.329         | Vertical    |
| 10       | 0.071      | 13.978         | Vertical    |
| 11       | 0.0686     | 14.578         | Longitudinal|
| 25       | 0.059      | 16.852         | Transversal |
| 150      | 0.019      | 53.730         | Vertical    |
| 239      | 0.012      | 81.652         | Longitudinal|
| 330      | 0.008      | 120.940        | Vertical    |

As it should be with any mode in which the mode shape is longitudinal, transverse or vertical head, this mode is caused by the corresponding direction of displacement. For torsional shaped, mass participation in all directions is zero. We estimate the response of the structure by the initial four transverse and longitudinal modes.

Here, we separate transverse mode shapes for actual interest as we use the high dynamic wind load in this direction. The four modes 2, 6, 20, and 25 are shown in Figures 13 to 16 where their mass participation is higher than 1% for the direction of transverse. Point out that we show the figures in their easiest ‘wireframe’ form for clarification owing to the structure’s height. Therefore, the figures have also been scaled to show the mode forms better.

![Figure 13. Mode 2 - frequency = 2.500 Hz](image1)

![Figure 14. Mode 6 – frequency = 8.756Hz](image2)

![Figure 15. Mode 20 – frequency = 16.521 Hz](image3)

![Figure 16. Mode 25– frequency = 16.852 Hz](image4)
4.2. Design using AISC 360-16 Code

The study has been carried out using SAP2000 that can perform both static and dynamic analysis as well as using necessary combinations. In this study, the AISC360-16 code, which is the most suitable for Turkey earthquake building code (TBDY-2018), has been used.

Load cases are a set of load and boundary conditions used to define particular loading conditions. In the study, we have taken the load cases which have been used to study the linear response from a structure subjected to different loading conditions from Turkey Electricity Distribution (TEDAS).

4.3. Design with the Same Sections Which Have Been Taken From TEDAS

We designed the section members of the transmission tower. Therefore, several sections have not passed as seen in Figure 17. 18 steel frames failed the stress/capacity check as illustrated in Table 9, which provides the summary data for the designed frames used in the current work.

![Figure 17. The failed sections of the transmission tower](image)

| Frame No. | Design Section | Ratio | Combo |
|-----------|----------------|-------|-------|
| 7         | L80X7          | 1.82  | COMB1 |
| 13        | L80X7          | 1.98  | COMB2 |
| 47        | L80X10         | 1.35  | COMB1 |
| 53        | L80X10         | 1.76  | COMB2 |
| 199       | L70X7          | 1.15  | COMB11|
| 626       | L60X5          | 1.69  | COMB2 |
| 635       | L80X7          | 1.08  | COMB1 |
| 637       | L80X7          | 1.08  | COMB11|
| 654       | L80X7          | 1.17  | COMB2 |
| 713       | L70X7          | 1.41  | COMB14|
| 714       | L70X7          | 1.08  | COMB2 |
| 715       | L70X7          | 0.95  | COMB2 |
| 716       | L70X7          | 1.43  | COMB10|
| 771       | L50X5          | 1.96  | COMB2 |
| 772       | L50X5          | 2.52  | COMB14|
| 773       | L50X5          | 2.11  | COMB2 |
| 774       | L50X5          | 2.52  | COMB14|
| 1199      | L60X6          | 12.73 | COMB2 |
4.4. Modeling and Parametric Analysis of Bolted Connections on Retrofitted Transmission Tower Members

The useful retrofitting method for attaching parallel reinforcement components to critical members (Figure 18) has been used on transmission line towers in Turkey and worldwide.

Figure 18. Method of retrofitting

In this approach, a critical leg component and parallel reinforcement members are connected by cross-shaped bolt connections (Figure 19.a). The cross-shaped connections transfer large external loads from the critical legs member of the reinforcement components so that less loads transport in critical legs and they increase the total load carrying the amount that can be contained. Splicing joints (Figure 19, b) are used in grid transmission towers to connect the critical lengths of leg members. Therefore, cruciform and joint connections are significant factors within the structural behavior of the legs of retrofitted transmission towers.

Figure 19. Connection of transmission tower

(a) Cruciform connection
(b) splice joint

Experimental investigation in the joint connections provided the concept of "slip resistance" relying on a set of effecting factors, for instance, bolt arrangement, properties of the material surface, material strength, and torque values.

Relying on this study, ‘Ungkurapinan et al. (2003)’ carried out experimental research on the load transfer behavior of screw connections with the same steel angles. In this work, we developed experimental and theoretical expressions of the load shift. We then used this load switching behavior for screw connections in the fault analysis of network towers [21].

Studies on cross-connections have been carried out too. Experimental research showed that the load transfer behavior of cross-connections depends on the screw arrangement. ‘Zhuge et al. (2012)’ has extended these research results from the numerical analysis [22]. ‘Mills et al. (2012)’ also suggested a retrofitted powerline tower test. Moreover, he concluded that the cross-connection in a transmission tower system has sufficient load transmission capability.

However, the influencing variables and error paths of the combination of cross and joint connections have not been examined. Therefore, the authors have investigated the load transfer behavior of cross-connections and joints using experimental tests and compared the results with models that were developed in software called ABAQUS. They then carried out a parametric study, taking into account friction coefficients, screw numbers, and torque values using the confirmed numerical models [23].
Lu et al. (2014) presented in their study the load shifting behavior of the cross and joint connections to the legs of retrofitted gear towers. ABAQUS has been used for designing numerical models to make a pretense of experimental tests and in a parametric study, then they used the number of verified models. Depending on both the experimental and numerical test results, they found the assembly specification affecting the load shifting behavior, Figure 20, displays failure mode of CTC section type [24].

4.5. Replace the Failed Sections in the Tower with New Sections

In our study, we replaced the failed sections with the CTC section type. Figure 21 presents the section designed with SAP2000 software. The comparative data summary of the designed steel using AISC 360-16 is highlighted in Table 10.

| Frame No. | Design Section | Ratio | Replaced section | Ratio |
|-----------|----------------|-------|------------------|-------|
| 7         | L80X7          | 1.82  | CTC 80x7         | 0.58  |
| 13        | L80X7          | 1.98  | CTC 80x7         | 0.56  |
| 47        | L80X10         | 1.35  | CTC 80x10        | 0.41  |
| 53        | L80X10         | 1.76  | CTC 80x10        | 0.61  |
| 199       | L70X7          | 1.15  | CTC 70x7         | 0.10  |
| 626       | L60X5          | 1.69  | CTC 60x5         | 0.21  |
| 635       | L80X7          | 1.08  | CTC 80X7         | 0.10  |
| 637       | L80X7          | 1.08  | CTC 80X7         | 0.10  |
| 654       | L80X7          | 1.17  | CTC 80X7         | 0.10  |
| 713       | L70X7          | 1.41  | CTC 70X7         | 0.10  |
| 714       | L70X7          | 1.08  | CTC 70X7         | 0.10  |
| 715       | L70X7          | 0.95  | CTC 70X7         | 0.50  |
| 716       | L70X7          | 1.43  | CTC 70X7         | 0.08  |
| 771       | L50X5          | 1.96  | CTC 50X5         | 0.10  |
| 772       | L50X5          | 2.52  | CTC 50X5         | 0.09  |
| 773       | L50X5          | 2.11  | CTC 50X5         | 0.07  |
| 774       | L50X5          | 2.52  | CTC 50X5         | 0.09  |
| 1199      | L60X6          | 12.73 | CTC 60X6         | 0.40  |
5. Conclusions

The seismic study of transmission towers in Turkey developed a set of time history analyses using non-linear finite element (FE) models on towers and a set of 10 recorded earthquake ground motions. We shall conclude that the designed transmission towers are not safe from earthquakes in Turkey.

For the 154 kV transmission towers, the buckling developed in the steel members is further probable to occur to steel sections in the cage (top part) of the towers. A relationship has been determined between the transmission tower height and the seismic vulnerability for transmission towers.

The results show that the load of failure increased after retrofitting. The retrofitting method was effective and easy to be conducted in fields. Disadvantages of the current research, which are the effect of connection design parameters, bolt arrangement, properties of the material surface, material strength, and torque values, on the connection model, have not been discussed.

Potential improvements on the study are as follows; i) We can use a transmission tower more than once, also we should do an optimization study to keep the weight as low as possible; ii) Analysis and design of the transmission tower can be carried out by different engineering softwares, then comparison will lead us to the best results; iii) For a more accurate examination of the seismic performance of the transmission towers, a non-linear time-history analysis considering the interaction of soil-structure will be required.

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7. Conflicts of Interest

The authors declare no conflict of interest.

8. References

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