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Experimental and numerical study on the collapse failure of long-span transmission tower-line systems subjected to extremely severe earthquakes

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Abstract. A long-span transmission tower-line system is indispensable for long-distance electricity transmission across a large river or valley; hence, the failure of this system, especially the collapse of the supporting towers, has serious impacts on power grids. To ensure the safety and reliability of transmission systems, this study experimentally and numerically investigates the collapse failure of a 220 kV long-span transmission tower-line system subjected to severe earthquakes. A 1:20 scale model of a transmission tower-line system is constructed in this research, and shaking table tests are carried out. Furthermore, numerical studies are conducted in ABAQUS by using the Tian-Ma-Qu material model, the results of which are compared with the experimental findings. Good agreement is found between the experimental and numerical results, showing that the numerical simulation based on the Tian-Ma-Qu material model is able to predict the weak points and collapse process of the long-span transmission tower-line system. The failure of diagonal members at weak points constitutes the collapse-inducing factor, and the ultimate capacity and weakest segment vary with different seismic wave excitations. This research can further enrich the database for the seismic performance of long-span transmission tower-line systems.

Keywords: long-span transmission tower-line system; failure; severe earthquakes; scale model; shaking table tests, numerical studies

1. Introduction

It is widely recognized that electricity transmission systems are important lifeline engineering components that affect every aspect of modern society. For the long-distance transmission of electricity, these systems must inevitably cross large rivers or deep valleys; under these circumstances, long-span transmission tower-line systems are required. Because of their large span and immense height, long-span transmission tower-line systems have a greater flexibility than an "ordinary" overhead transmission tower-line system, which indicates that they should possess a stronger capacity to resist collapse failure during an earthquake. Nevertheless, transmission towers have frequently failed and even collapsed in past major earthquakes, such as the 1994 Northridge earthquake (Hall *et al.* 1994), the 1995 Kobe earthquake (Shinozuka 1995), the 1999 Chi-Chi earthquake (NCREE 1999) and the 2008 Wenchuan earthquake (Tian *et al.* 2016a). The collapse failure of a transmission tower-line system not only causes direct economic losses but also impacts the entire power grid, hindering post-earthquake relief efforts. Therefore, it is essential to investigate the seismic responses and collapse failure mechanisms of long-span transmission tower-line systems subjected to severe earthquakes.

In recent decades, extensive research, including analytical, numerical and experimental studies, has been conducted to investigate the seismic responses of transmission tower-line systems. Simplified analytical models were established to investigate the seismic responses of coupled transmission tower-line systems (Li *et al.* 2005, Kempner and Smith 1984, Ghobarah and Aziz 1996), and some significant conclusions were drawn. However, these theoretical results could have been influenced by the assumptions made in the analysis. To capture the responses of such structures more genuinely, numerical simulations were subsequently introduced to investigate the seismic responses of transmission tower-line systems (Bai *et al.* 2011, Park *et al.* 2016, Tian *et al.* 2018a) Compared with analytical studies, numerical simulations have a higher precision and can easily solve nonlinear problems. In addition to analytical and numerical studies, limited shaking table tests were also performed to study the realistic performance of transmission tower-line systems subjected to seismic excitations. Through shaking table tests, the influences of transmission lines (Kotsubo *et al.* 1985) and seismic spatial variations (Tian. 2016b, 2017a, 2018b) on the dynamic responses of transmission tower-line systems were investigated, thereby providing significant references for analytical and numerical studies.

A critical review of the relevant literature reveals that numerous investigations have already been performed on the seismic responses of transmission tower-line systems. Recently, these research interests have shifted toward the collapse of transmission tower-line systems. Albermani *et*

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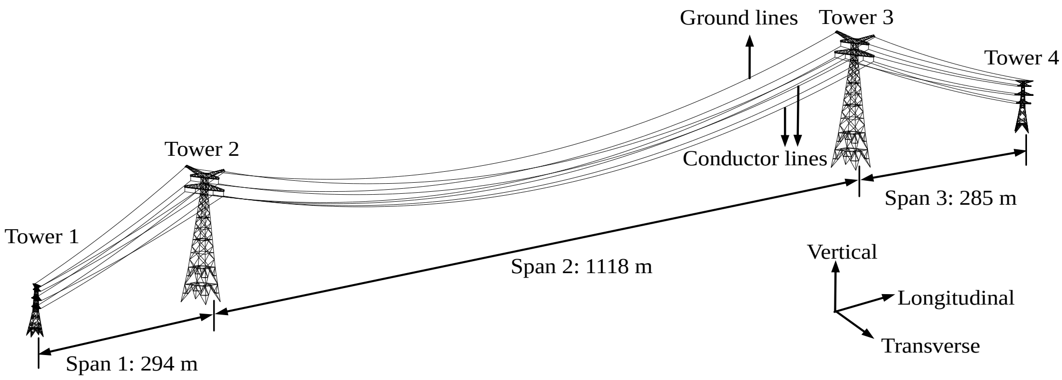
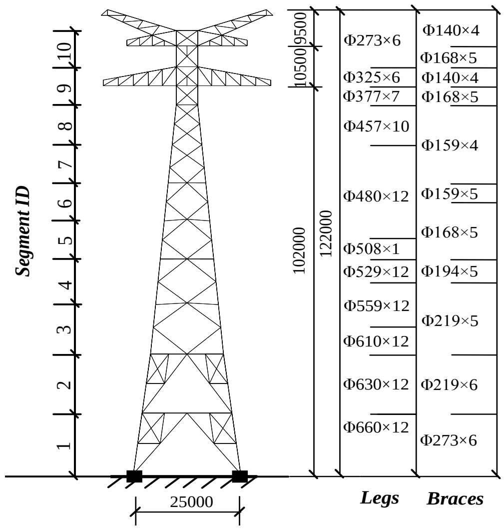


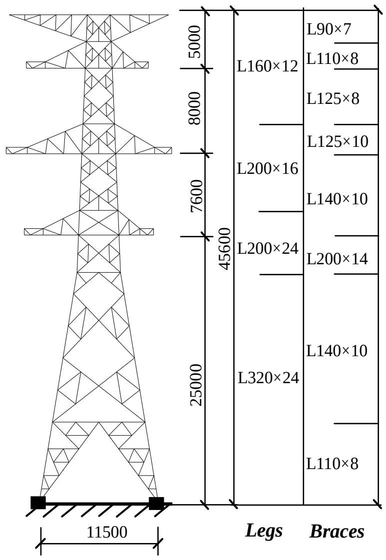
Fig. 1 Sketch of the selected prototype

al. (2009) investigated the ultimate capacity of a power transmission tower by proposed nonlinear analytical techniques, in which both geometric and material nonlinearity were considered. Eslamlou and Asgarian (2017) performed a nonlinear dynamical analysis of the progressive collapse of a power transmission tower, evaluated the key structural member of the power transmission tower and effectively predicted that key member. Wang *et al.* (2003) proposed an analytical procedure to study the progressive collapse of a transmission tower-line system; the results indicate that the collapse mode and vulnerable points can be obtained by using the proposed procedure. Tian *et al.* (2016a, 2017b) carried out a collapse analysis to study the ultimate capacity of a power transmission tower-line system under earthquake excitations. Zheng *et al.* (2017, 2018) proposed an explicit dynamic analysis method to calculate the progressive collapse of a high-rise power transmission tower structure; the results show that this method is suitable for calculating the seismic collapse of a power transmission tower. As mentioned above, the collapse failure of a transmission tower-line system has been extensively investigated by using numerical methods. Nevertheless, no experimental studies have been reported. Consequently, there is an urgent need to investigate the collapse failure of long-span transmission tower-line systems through experimentation, such as shaking table tests.

Previous works (Tian *et al*. 2016b, 2017a, 2018b) studied only the seismic response by shaking table tests; that is, tests were not conducted until failure. Therefore, to fill this gap, this research experimentally and numerically studies the collapse failure of long-span transmission tower-line systems subjected to multicomponent seismic excitations. A 1:20 scale long-span transmission tower-line system model is tested using the three-table array at the shaking table laboratory of Central South University, China. A corresponding numerical simulation is also conducted using the incremental dynamic analysis in ABAQUS software. The remainder of this paper is organized as follows. Section 2 introduces the prototype of the experimental model. Section 3 describes the design and construction of the scaled experimental model. Section 4 shows the experimental results and analyses the failure process of a transmission tower; the collapse failure mechanism of the transmission tower is studied numerically, and the influences of different ground motions on the ultimate capacity and the weakest segment of the transmission tower are investigated in Section 5. Finally, Section 6 summarizes the major conclusions of the present study.



(a) Towers 2 and 3



(b) Towers 1 and 4

Fig. 2 Elevations of the transmission towers (mm)

2. Selection of prototype

A 200 kV electricity transmission system is selected in this research. This system is designed for seismic hazards with a probability of exceedance of 10% in 50 years, which corresponds to a peak ground acceleration (PGA) of 0.2 g in the original design. The entire system extends approximately 58.4 km and crosses the Yellow River (the 6th longest river in the world) in Shandong Province, China. As this research focuses on a long-span transmission tower-line system, the subsystem crossing the Yellow River is examined separately from the entire system as the prototype.

Fig. 1 depicts a sketch of the selected prototype. As illustrated, the selected prototype consists of four supporting towers and three spans of transmission lines. The lengths of the three spans (designated as Spans 1, 2 and 3 in Fig. 1) are 294, 1118, and 285 m, respectively, and the longest span (Span 2) crosses the Yellow River. Additionally, the four towers are designated as Towers 1 through 4; among them, Towers 1 and 4 are tension-type towers providing the transmission lines with a tension force, while Towers 2 and 3 are suspension-type towers supporting the transmission lines with a vertical force.

Fig. 2 illustrates the elevations of the two kinds of supporting towers. As shown, the total height and root span of a suspension-type tower are 122 m and 45.6 m, respectively, while those for a tension-type tower are 45.6 m and 11.5 m, respectively. Furthermore, detailed cross-section information of the leg and brace members for the towers is also given in Fig. 2. As shown, the suspension-type towers are constructed by using steel tubes, while the tension-type towers consist of angle steel; these steel tubes and angle steel beams are manufactured from Q345 and Q235 steel. The suspension-type towers are selected as the primary objective of this research because they exhibit larger dynamic responses than the tension-type towers. Thus, the body of a suspension-type tower is divided into 10 segments (see Fig. 2) along the height of the tower to precisely locate the failure position. For the transmission lines, conductor lines are supported at two cross arms at elevations of 112.5 m and 102 m, while ground lines are fixed at the tops of the suspension-type towers. The properties of the conductor and ground lines are tabulated in Table 1.

3. Design and construction of the experimental model

Table 1 Properties of the conductor and ground lines

Category	Conductor line	Ground line
Designation	LHBGJ-400/95	OPGW-180
Outer diameter (mm)	29.14	17.85
Modulus of elasticity (GPa)	78000	170100
Cross-sectional area (mm^2)	501.02	175.2
Mass per unit length (kg/km)	1856.7	1286
Thermal expansion coefficient (1/℃)	18.0E-6	12.0E-6

(a) The simplified models of Towers 1 and 4

(b) The experimental models of Towers 2 and 3

Fig. 3 Supporting towers of the experimental model

After the selection of the prototype, an experimental model of the long-span transmission tower-line system is designed and constructed in this section. The experimental model is tested at the shaking table laboratory of Central South University equipped with an array of three identical 6-degrees of freedom (DOF) shaking tables. The detailed performance of these shaking tables can be found in (Tian, *et al.* 2017a). It should be noted that the maximum allowable height for a specimen is 15 m. Additionally, only three shaking tables are currently available, although the prototype includes four supporting towers. Therefore, since this research focuses on the seismic responses of Towers 2 and 3, only Towers 2 and 3 are placed on shaking tables, while Towers 1 and 4 are mounted on the rigid floor of the laboratory.

Considering the spatial constraints of the laboratory, a reduced-scale experimental model of the prototype is developed in this research. The scale factors of the supporting towers and transmission lines are 1/20 and 1/40, respectively, which are determined based on Buckingham's π theorem (Sedov 1959). To prevent repetition, the detailed procedure used to determine the scale factors of the experimental model is not discussed here and can be found in a previous study (Tian *et al.* 2018b). As mentioned above, Towers 1 and 4 are not the primary objectives in this research; thus, they are considered through simplified models, as shown in Fig. 3(a). The simplified models are designed to ensure that the heights and stiffness values along the longitudinal and transverse directions are identical to those determined during the design of the experimental models. As shown in Fig. 3(b), the experimental models of Towers 2 and 3 are built using stainless steel tubes. The diameter of the steel tubes utilized in the construction varies from 4 mm to 31 mm, while the wall thickness ranges from 0.2 mm to 0.6 mm. The members of these experimental models are connected by welding, and steel blocks are installed along the height of the tower to achieve the target artificial mass per segment (Zhang 1997). Steel wires with diameters of 3.56 mm and 2.19 mm are utilized to model the conductor and ground lines, respectively, in the experimental model. To realize the target artificial mass of transmission lines, stainless steel chains are installed along each steel wire. Fig. 4 shows a field photo of the assembled experimental model of the transmission tower-line system.

As mentioned above, Towers 2 and 3 constitute the primary research objective of this research. Therefore, these two towers are selected for the installation of instrumentation to record the acceleration response and stress in selected members. Fig. 5 depicts the instruments outfitted on Towers 2 and 3. Strain gauges are attached to the leg members to record the strain data, and accelerometers are mounted along the height of the transmission tower to record the longitudinal and transverse acceleration responses.

4. Model experiment

The site condition is a key factor for selecting the input ground motions in the dynamic test. Considering that the site condition of the transmission tower-line system is classified as class II, a typical natural seismic record, namely, the Imperial Valley wave (El Centro Array #9, 1940), is adopted in this shaking table test according to the Code for Seismic Design of Electrical Installations (GB 50260-2013 2013). The detailed acceleration information of the seismic record is downloaded from the Pacific Earthquake Engineering Research Center (PEER, http://peer.berkeley.edu/). Fig. 6 shows the acceleration time histories of the three seismic components and the corresponding acceleration response spectra with a damping ratio of 2%. The specific input mode and directions of the seismic wave can be found in previous work (Tian *et al.*

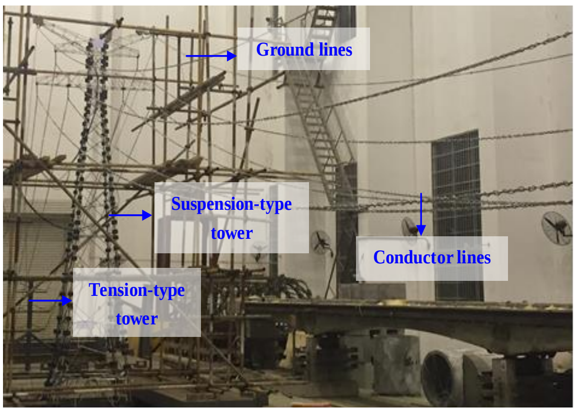


Fig. 4 Transmission lines of the experimental model

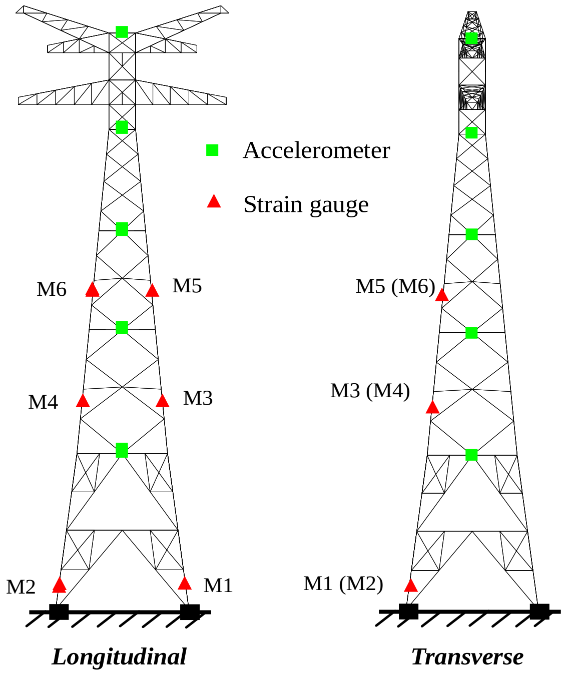


Fig. 5 Instrumentation of Towers 2 and 3

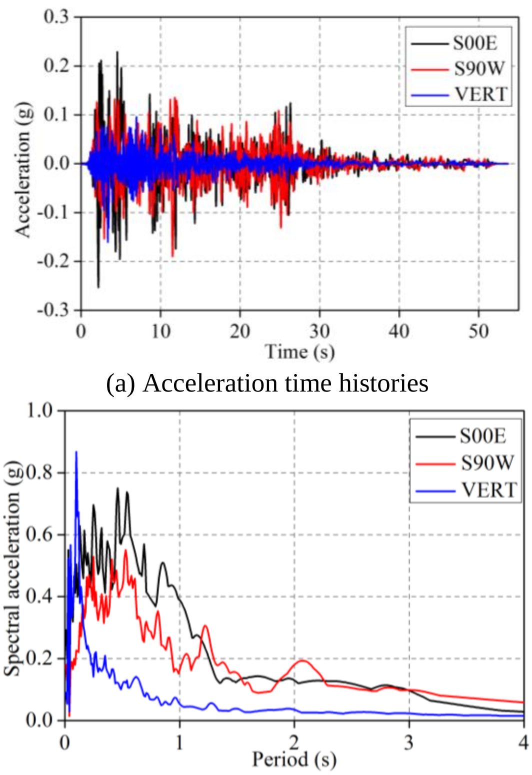
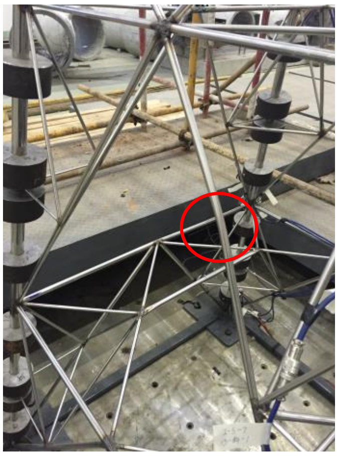


Fig. 6 Acceleration time histories and the corresponding response spectra of the El Centro wave



(a) Buckling of elements

(b) Fracturing of elements

Fig. 7 Two different damage patterns of elements

2018b). To attain the failure state of the transmission tower, the PGA of the longitudinal direction component is amplified gradually starting from 0.1 g with an increment of 0.1g, and an identical modulation coefficient is applied to the other two components of ground motion. Note that the seismic wave mentioned above is aimed at the prototype; thus, the amplitude and time step of each seismic wave should be modified based on the scale factors for the acceleration and time, respectively, for use in the experimental model.

The shaking table tests of the experimental model are carried out as expected. With regard to safety in the laboratory, these tests investigate only the failure of the transmission tower and not its collapse. During the field experiment, no visible damage appears when the test model is subjected to shock waves with accelerations ranging from 0.1 g to 0.5 g. Nevertheless, intrinsic damage begins to appear suddenly under 0.6 g and progressively develops with an increase in the seismic intensity. Major damage eventually occurs and propagates under 0.8 g. Fig. 7 illustrates two different damage patterns, namely, the buckling and the fracturing of elements. Fig. 8 and Fig. 9 show the cumulative damage processes of Towers 2 and 3 at accelerations ranging from 0.6 g to 0.8 g, respectively, where the elements after buckling and fracturing are highlighted in red and blue, respectively. The corresponding detailed process is described as follows.

(1) Under a PGA of 0.6 g, the diagonal members in Segment 2 of Towers 2 and 3 exhibit compression buckling and start to lose their bearing capacity, impacting the transmission path of the internal force of the transmission towers (Fig. 8(a) and Fig. 9(a)).

(2) The damaged areas of the members continue to expand outward when the PGA is 0.7 g. A few members adjacent to the failed members begin to buckle in succession in Towers 2 and 3 (Fig. 8(b) and Fig. 9(b)).

(3) Under a PGA of 0.8 g, the damage situations of the members in both towers are aggravated, and fractures are observed, as shown in Fig. 8(c) and Fig. 9(c). Seven diagonal members ultimately fractured in Tower 2, while only one member fractured in Tower 3.

As mentioned above, the buckling of members leads to the redistribution of the internal force of the transmission towers, whereas the fracturing of members seriously undermines the bearing capacity of the towers. Note that large-scale damage eventually occurred in the diagonal members of Tower 2, as the fracturing of members makes it difficult to withstand stronger seismic excitations. Therefore, for the sake of safety, 0.8 g, which is 4 times greater than the design PGA, is taken as the collapse PGA. In addition, the test results show that Segments 2 and 3 are more sensitive to seismic excitations than other segments, and the diagonal members are the most prone to being damaged, which means that more attention should be paid to diagonal members. Additionally, due to the asymmetry of the prototype, the length of Span 1 is different from that of Span 3; as a result, the destruction of Tower 2 is more serious than that of Tower 3.

5. Numerical research

For a comparison with the shaking table tests, the collapse failure of the transmission tower-line system is analyzed numerically in ABAQUS software. To accurately capture the genuine behavior of the structure, the Tian-Ma-Qu material model (Tian *et al.* 2017b, 2018a, 2019), which can consider the nonlinear behaviors of members, is utilized. Damping ratios of 2% and 1% are assumed for the transmission towers and transmission lines, respectively. Additionally, the same seismic wave and excitation pattern as those used in the shaking table tests are adopted in this section. Subsequently, the collapse failure process of the transmission tower-line system is simulated, and the results are compared with those obtained from the abovementioned shaking table tests. Finally, the failure states under different ground motions are discussed.

5.1 Verification of the finite element (FE) model

As shown in Fig. 10, a three-dimensional (3D) FE model of the transmission tower-line system is established in ABAQUS. In this model, the elastic modulus, yield stress, mass density and Poisson's ratio of the steel are equal to 2.01×10^{11} Pa, 235 MPa for Q235 (345 MPa for Q345), 7800 kg/m^3 and 0.3, respectively. The transmission towers and transmission lines are modeled by beam elements (B31)

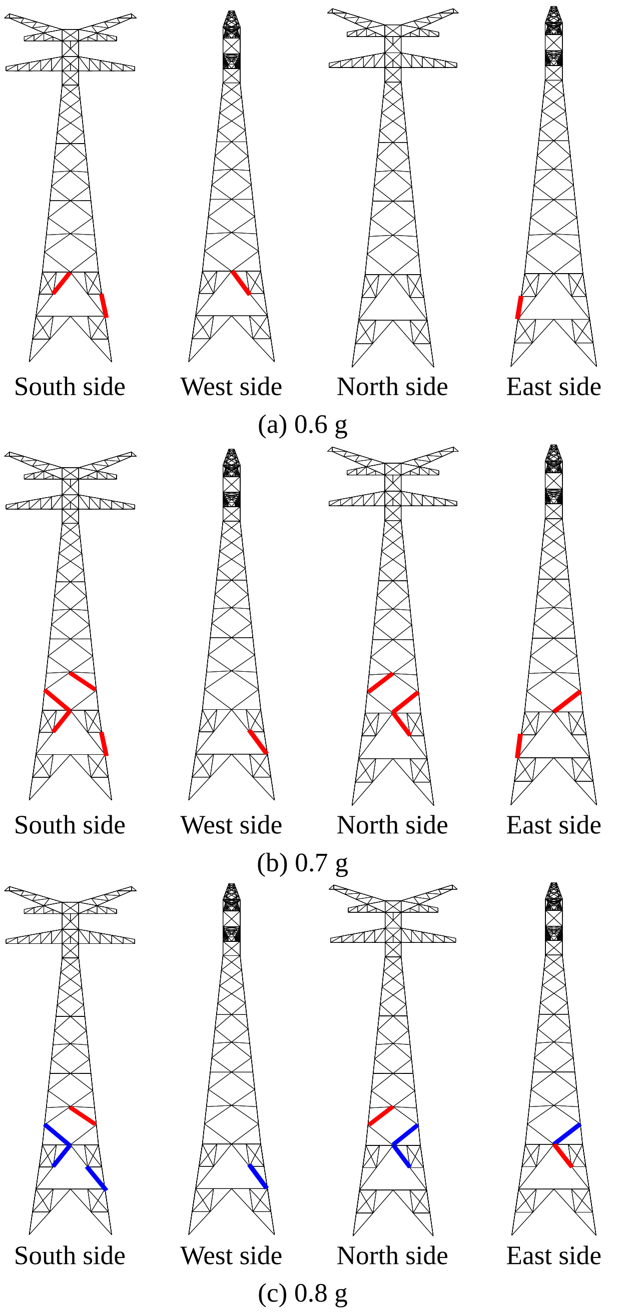


Fig. 8 Cumulative damage processes of Tower 2 with an increase in the earthquake intensity

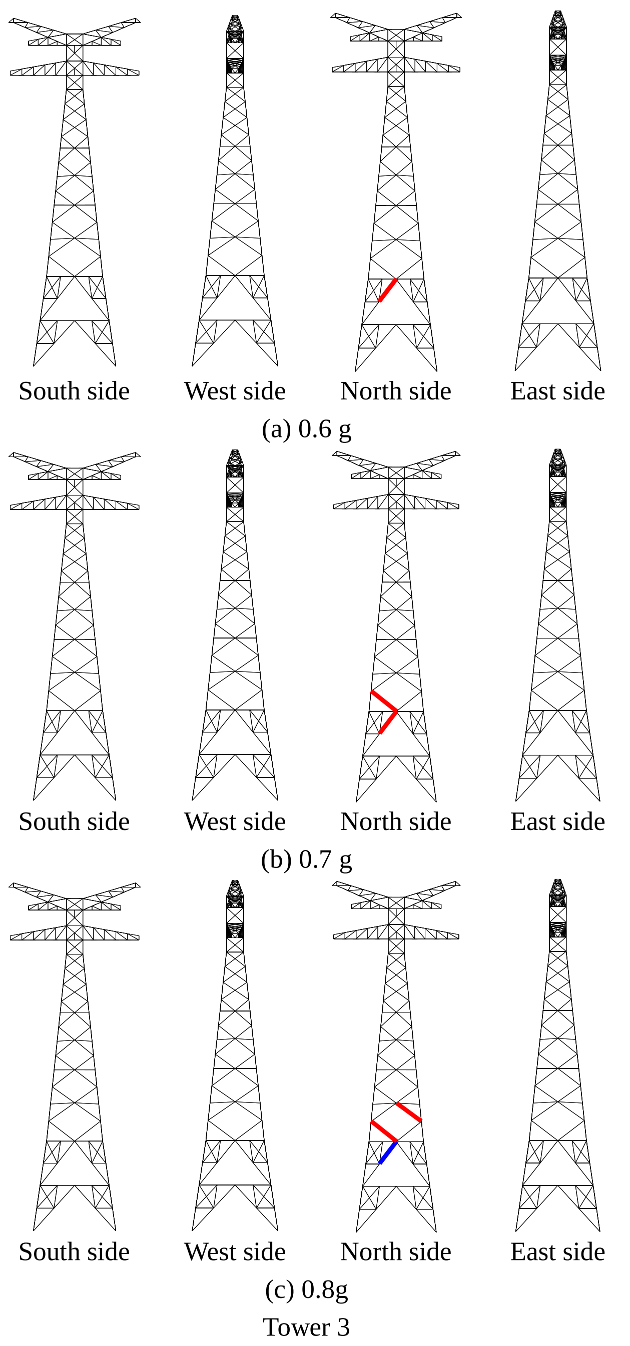


Fig. 9 Cumulative damage processes of Tower 3 with an increase in the earthquake intensity

and truss elements (T3D2), respectively. For Towers 2 and 3, there are a total of 1140 members and 431 nodes. The supports of the towers are assumed to be fixed. Based on an eigenvalue analysis, the first frequencies of a suspension-type tower along the longitudinal and transverse directions are 1.285 Hz and 1.308 Hz, respectively, whereas the fundamental frequencies along the longitudinal and transverse directions of the experimental model extracted by white noise excitation are 1.482 Hz and 1.491 Hz, respectively; the corresponding relative errors are 13.9 and 11.9%, respectively. In other words, the experimental model shape agrees well with the analytical results.

To further demonstrate the accuracy of the FE model, the peak absolute accelerations and peak stresses in the FE model are compared with the shaking table test results. Given a scale factor of 1.0, the acceleration and stress responses from the test data are the same as those from the prototype. Figs. 11 and 12 show comparisons between the numerical and experimental results of Towers 2. The original design PGA (i.e., 0.2 g) is adopted for the PGA of the input excitation, and both the longitudinal and the transverse responses of the system are given. As shown, the FE model can accurately capture the trend of the acceleration response along the height, although some differences from the exact values are observed, especially above 80 m. However, for the peak absolute stress, the relative errors are within 10%. Therefore, these results can

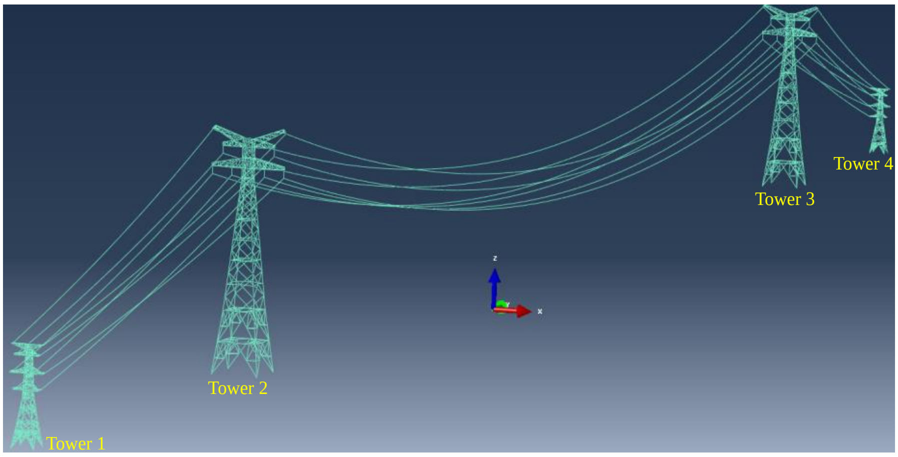
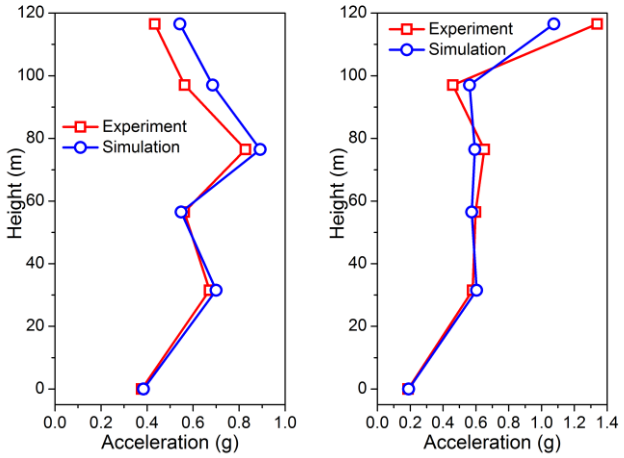


Fig. 10 FE model of the prototype



(a) Longitudinal direction (b) Transverse direction

Fig. 11 Comparison of the peak acceleration responses along the height of the transmission tower

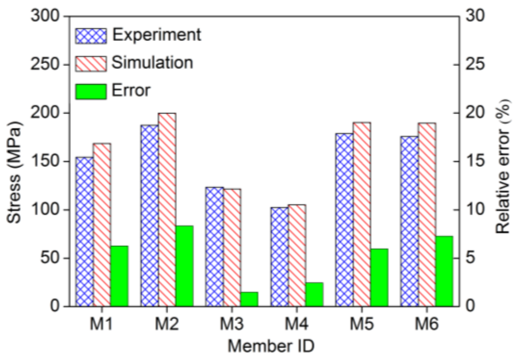


Fig. 12 Comparison of the peak stresses in different members

be considered acceptable given that errors in the test design, selection of materials and manufacturing of the model are unavoidable. This FE model lays the foundation for further simulating the failure of the tower-line system.

5.2 Collapse failure mechanism analysis

In this subsection, an incremental dynamic analysis (IDA) is carried out to investigate the collapse failure process under the El Centro wave mentioned above. The minimum PGA for the FE model structure to undergo progressive collapse is ultimately determined to be 0.75 g. Fig. 13 illustrates the typical failure mode of a tower-line system subjected to an El Centro wave (PGA=0.75 g). Note that the different colors represent the different damage situations of members; blue denotes an intact member, while red indicates that the member has failed. It is clear that the failed members are distributed mainly throughout Segments 2 through 5 of Tower 2. Relatively few failed members are also found in Segments 2 and 3 of Tower 3. The extremely large deformation of Tower 2 eventually leads to the collapse of the model. Considering that the extent of damage suffered by Tower 2 is even higher than that suffered by Tower 3, the failure process of Tower 2 will be studied in detail in the following sections.

Fig. 14 shows the time history curves of the horizontal and vertical displacements at the top of Tower 2 during the collapse process as well as the deformed shape of the structure at specific moments (i.e., T1, T2, T3 and T4). When the time t is less than T1, some members are slightly damaged, and no member fails, indicating that the whole transmission tower is still almost elastic. When the time t is T1 and T2, diagonal members 799 and 611 in Segment 2 fail successively and lose their bearing capacity. Once the diagonal members fail, the internal forces of the transmission tower are redistributed, and the seismic load is transferred to the adjacent members, which are then destroyed due to the additional seismic load. Note that the horizontal displacements at the corresponding moments are only 0.166 m and 0.172 m under normal operating conditions. Immediately afterwards, at t=T3, a large number of diagonal members in Segments 2 through 4 fail, and the internal forces continue to be redistributed to other structural members. Note that the horizontal displacement suddenly increases at this time, reaching 1.3 m; this indicates that the tower has been severely damaged. Suddenly, when the time t reaches T4, additional connected members begin to fail; moreover, Segments 2 through 5 have lost the ability to transfer internal forces, causing the entire tower to reach its load-carrying capacity and begin to tilt. Meanwhile, it is worth noting that the horizontal displacement has exceeded 6 m beyond the normal

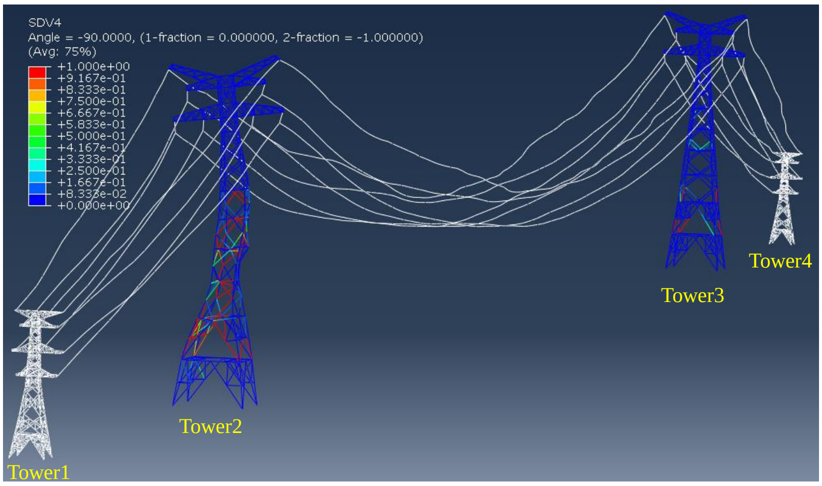


Fig. 13 Typical collapse mode of the transmission tower under A1 earthquake ground motion

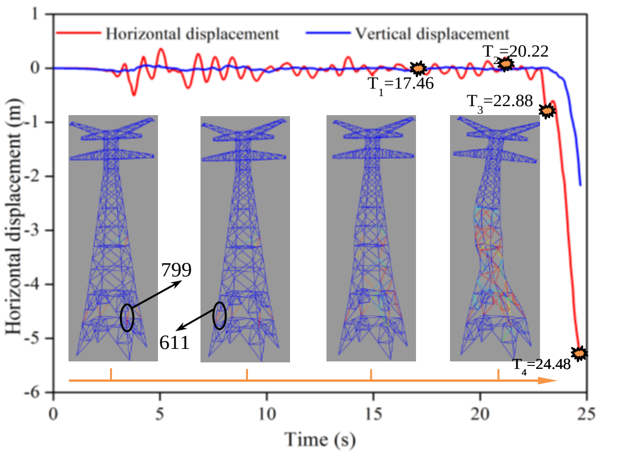


Fig. 14 Time history curves of the displacement at the top of the structure and the deformed shape of the structure at specific moments

operating conditions, showing that the tower has collapsed.

Obviously, when subjected to the El Centro ground motion, the ultimate collapse of the model structure is due to the failure of Tower 2, and the damage is hosted mainly in Segments 2 through 5 of Tower 2. Based on the analysis above, Segments 2 through 5 are regarded as potentially weak points, i.e., points at which the collapse of the tower can be initiated. Therefore, more attention should be paid to these weak points during the design and reinforcement of transmission towers. Note that damage first occurs in the diagonal members of Segment 2 during the collapse process; however, this segment is the first line of defense within the model structure against an earthquake, and hence, this damage leads to the eventual collapse of the entire structure. Therefore, the failure of a single member could rapidly lead to the failure of adjacent members and thus the collapse of the structure.

As mentioned above, the collapse PGA values of the system determined by numerical simulation and experimentation are 0.75 g and 0.8 g, respectively, which are very close. Fig. 8 and Fig. 14 demonstrate that the diagonal members in Segment 2 of Tower 2 undergo damage first, and Segments 2 to 5 suffer the most damage

Table 2 Summary of seismic wave records

ID	Event	Station	Year	Magnitude/M	PGA (g)
A1	Borrego	El Centro Array #9	1942	6.5	0.267
A2	Kern County	Taft Lincoln School	1952	7.36	0.180
A3	San Fernando	San Onofre-So Cal Edison	1971	6.61	0.323
A4	San Fernando	San Juan Capistrano	1971	6.61	0.276
A5	Northridge	Villa Park-Serrano	1994	6.6	0.239
A6	Artificial	–	–	–	–

in both cases. With regard to the collapse failure mechanism, the result is the same for the simulation and experimental analysis: the damage of a single member or of a few members could rapidly lead to the failure of adjacent members in quick succession; consequently, the load-bearing capacity of the transmission tower decreases with an increase in the extent of damage, and thus, the long-span transmission tower-line system loses its load-bearing capacity and completely collapses when a sufficient number of members fail. Considering the consistency of the collapse PGA, the weak points and the collapse failure mechanism, the numerical simulation method based on the Tian-Ma-Qu material model is deemed reliable; therefore, this method can be used to simulate the dynamic response and failure process of a transmission tower-line system.

5.3 Collapse analysis under different ground motions

Considering the distinct differences in the spectral responses of different ground motions, the collapse mode and weak points of the structure may differ greatly when subjected to different ground motions. Therefore, it is necessary to investigate the effects of the input ground motion on the collapse model of a transmission tower-line system. Based on the Code for Seismic Design of Electrical Installations (GB 50260-2013 2013), five other natural seismic records (designated as A1 through A5) and one artificial seismic record (designated as A6) are selected, and

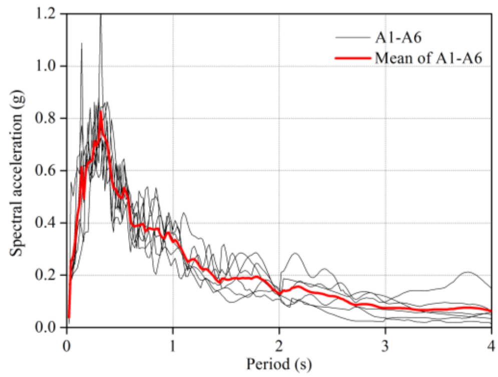


Fig. 15 Comparison of the six response spectra with the target design spectrum

Table 3 Collapse statistics of the model structure under different earthquake excitations

ID	Design PGA (g)	Collapse PGA (g)	Maximum horizontal displacement (m)	Damage position	Position of the first member to fail
A1	0.2	0.78	6.47	Segments 2 through 6	Segment 5
A2	0.2	0.8	6.59	Segments 2 through 5	Segment 2
A3	0.2	0.75	5.37	Segments 2 through 7	Segment 2
A4	0.2	0.73	5.90	Segments 2 through 6	Segment 2
A5	0.2	0.7	6.20	Segments 2 through 7	Segment 5
A6	0.2	0.69	6.99	Segments 2 through 5	Segment 2

their detailed information can be found in Table 2. Fig. 15 plots the acceleration spectra and mean spectrum of these ground motions.

Failure simulations are conducted for the transmission tower-line system when subjected to the six ground motions. Fig. 16 illustrates the time history curve of the horizontal displacement at the top of the tower and the final deformed shape of the tower under each seismic wave. As shown in Fig. 16, all the maximum horizontal displacements at the top of the tower exceed 5 m beyond normal operating conditions, demonstrating that the entire tower reaches its load-carrying capacity and starts to collapse. Table 3 lists the collapse states and damage positions of the transmission tower-line system under different seismic excitations. Fig. 16 and Table 3 demonstrate that Segments 2 through 5 are severely damaged in all cases, and Segments 6 and 7 also suffer a certain amount of damage in some cases, thereby confirming the weakness of these segments. At the same time, the probabilities that the first member to fail is Segment 2 and Segment 5 are 71.4% and 28.6%, respectively, indicating that the vulnerability of Segment 2 in the transmission tower is higher than those of the other segments. Moreover, the maximum and minimum collapse PGA values among the seven ground motions are 0.8 g and 0.69 g, respectively, which indicates that the ultimate capacity of the transmission tower-line system varies greatly under different seismic waves. These factors demonstrate that different ground motions have a great

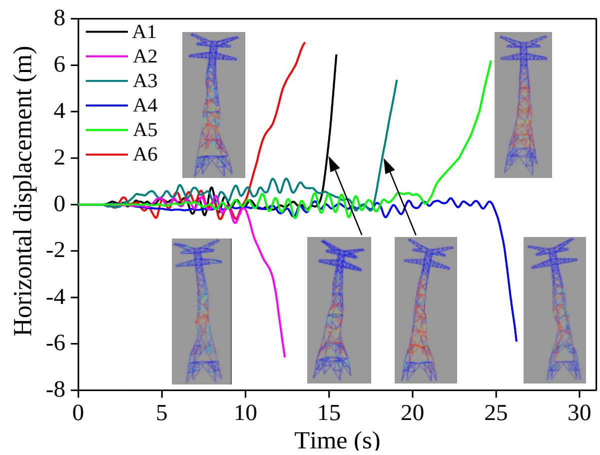


Fig. 16 Time histories of the horizontal displacements at the top of the tower and the collapse modes under different ground motions

influence on the weak points and ultimate capacity of transmission tower-line systems and thus cannot be ignored.

6. Conclusions

A combined experimental and numerical approach is adopted to focus on the failure analysis of a long-span transmission tower-line system under multicomponent earthquake excitations. A 1:20 scale model of a transmission tower-line system is constructed and tested on an array of shaking tables by subjecting it to the El Centro wave. Moreover, numerical analysis is conducted in ABAQUS and compared with the test results. Additionally, the influences of different ground motions on the failure mode are also investigated. Based on the experimental and numerical results, the following significant conclusions are drawn:

- The first frequencies of the experimental model agree well with the numerical simulation results, and the acceleration and stress response curves are consistent. The developed FE model and user-defined material model are reasonable for simulating the genuine behavior of a transmission tower subjected to earthquake loads.
- Based on the shaking table test and numerical simulation results, Segment 2 is more susceptible to failure, and the damage probability reaches 71.4%; this segment therefore requires more attention during the design and maintenance of the transmission tower.
- The collapse failure mechanism determined from the numerical simulation is very similar to that ascertained from the shaking table test: the collapse of the transmission tower-line system is caused by the failure of diagonal members at weak points. The failure of a single member or of a few members could rapidly lead to the failure of adjacent members in quick succession, eventually resulting in the collapse of the entire structure.
- Different ground motions have an important effect on the ultimate capacity and weakest segment of the

transmission tower and thus cannot be ignored in the seismic analysis of such structures.

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References

Albermani, F., Kitipornchai, S. and Chan, R.W.K. (2009), "Failure analysis of transmission towers", *Eng. Fail. Anal.*, **16**(6), 1922-1928. https://doi.org/10.1016/j.engfailanal.2008.10.001.

Bai, F.L., Hao, H., Bi, K.M. and Li, H.N. (2011), "Seismic response analysis of transmission tower-line system on a heterogeneous site to multi-component spatial ground motions", *Adv. Struct. Eng.*, **286**(3), 565-585. https://doi.org/10.1260/1369-4332.14.3.457.

Eslamlou, S.D. and Asgarian, B. (2017), "Determining critical areas of transmission towers due to sudden removal of members", *Case Stud. Eng. Fail. Anal.*, **9**, 138-147. https://doi.org/10.1016/j.csefa.2015.09.005.

GB 50260-2013 (2013), Code for Seismic Design of Electrical Installations, China Architecture & Building Press, Beijing, China.

Ghobarah, A., Aziz, T.S. and El-Attar, M. (1996), "Response of transmission lines to multiple support excitation", *Eng. Struct.*, **18**(12), 936-946. https://doi.org/10.1016/S0141-0296(96)00020-X.

Hall, J.F., Holmes, W. and Somers, P. (1994), "Northridge Earthquake, January 17, 1994", Preliminary Reconnaissance Report.

Kempner, L.J. and Smith, S. (1984), "Cross-rope transmission tower-line dynamic analysis", *J. Struct. Eng.*, ASCE, **110**(6), 1321-1335. https://doi.org/10.1061/(ASCE)0733-9445(1984)110:6(1321).

Kotsubo, S., Takanishi, T., Uno, K. and Sonoda, T. (1985), "Dynamic tests and seismic analysis of high towers of electrical transmission line", *Tran. JPN Soc. Civil Eng.*, **15**, 72-75.

Li, H.N., Shi, W.L., Wang, G.X. and Jia, L.G. (2005), "Simplified models and experimental verification for coupled transmission tower-line system to seismic excitations", *J. Sound Vib.*, **286**(3), 565-585. https://doi.org/10.1016/j.jsv.2004.10.009.

NCREE (1999), "Investigation report of Chi-Chi Earthquake", National Center for Research on Earthquake Engineering.

Park, H.S., Choi, B.H., Kim, J.J. and Lee, T.H. (2016), "Seismic performance evaluation of high voltage transmission towers in South Korea", *KSCE J. Civil Eng.*, **20**(6), 2499-2505. https://doi.org/10.1007/s12205-015-0723-3.

Sedov, L. (1959), *Similarity and Dimensional Methods in Mechanics*, CRC Press.

Shinozuka, M. (1995), "The Hanshin-Awaji Earthquake of January 17", 1995 Performance of Lifelines, Technical Report NCEER, 95 US National Center for Earthquake Engineering Research.

Tian L., Ma, R.S. and Qu, B. (2018a), "Influence of different criteria for selecting ground motions compatible with IEEE 693 required response spectrum on seismic performance assessment of electricity transmission towers", *Eng. Struct.*, **156**, 337-350. https://doi.org/10.1016/j.engstruct.2017.11.046.

Tian, L., Gai, X. and Qu, B. (2017a), "Shake table tests of steel towers supporting extremely long span electricity transmission lines under spatially correlated ground motions", *Eng. Struct.*, **132**, 791-807. https://doi.org/10.1016/j.engstruct.2016.11.068.

Tian, L., Gai, X., Qu, B., Li, H. and Zhang, P. (2016a), "Influence of spatial variation of ground motions on dynamic responses of supporting towers of overhead electricity transmission systems: An experimental study", *Eng. Struct.*, **128**, 67-81. https://doi.org/10.1016/j.engstruct.2016.09.010.

Tian, L., Ma, R.S., Li, H.N. and Wang, Y. (2016b), "Progressive collapse of power transmission tower-line system under extremely strong earthquake excitations", *Int. J. Struct. Stab. Dyn.*, **16**(7), 1550030. https://doi.org/10.1142/S0219455415500303.

Tian, L., Ma, R.S., Qiu, C.X., Xin, A.Q., Pan, H.Y. and Guo, W. (2018b), "Influence of multi-component ground motions on seismic responses of long-span transmission tower-line system: An experimental study", *Earthq. Struct.*, **15**(6), 583-593. https://doi.org/10.12989/eas.2018.15.6.583.

Tian, L., Pan, H.Y. and Ma, R.S. (2019), "Probabilistic seismic demand model and fragility analysis of transmission tower subjected to near-field ground motions", *J. Constr. Steel Res.*, **156**, 266-275. https://doi.org/10.1016/j.jcsr.2019.02.011.

Tian, L., Pan, H.Y., Ma, R.S. and Qiu, C.X. (2017b), "Collapse simulations of a long span transmission tower-line system subjected to near-fault ground motions", *Earthq. Struct.*, **13**(2), 211-220. https://doi.org/10.12989/eas.2017.13.2.211.

Wang, W.M., Li, H.N. and Tian, L. (2013), "Progressive collapse analysis of transmission tower-line system under earthquake", *Adv. Steel Constr.*, **9**(2), 161-172.

Zhang, M.Z. (1997), "Study on similitude laws for shaking table tests", *Earthq. Eng. Eng. Vib.*, **17**(2), 52-58.

Zheng, H.D. and Fan, J. (2018), "Analysis of the progressive collapse of space truss structures during earthquakes based on a physical theory hysteretic model", *Thin Wall. Struct.*, **123**, 70-81. https://doi.org/10.1016/j.tws.2017.10.051.

Zheng, H.D., Fang, J. and Long, X.H. (2017), "Analysis of the seismic collapse of a high-rise power transmission tower structure", *J. Constr. Steel. Res.*, **134**, 180-193. https://doi.org/10.1016/j.jcsr.2017.03.005.

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