

A STUDY ON DAMAGE OF TRANSMISSION STEEL TOWER WITH UNEQUAL LEGS IN THE CHI-CHI EARTHQUAKE TAIWAN

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SUMMARY

The Chi-Chi Earthquake with a moment magnitude of 7.7 occurred in the central part of Taiwan on September 21 in 1999. Many steel towers had heavy damages. This paper deals with the analysis of earthquake response of 345kV steel tower to investigate the cause of damages by using FEM. And also the effect on transmission wire to dynamic behavior of strain steel tower was evaluated. From the dynamic response analysis, the possibility of damage of the steel tower by vibration was inferred and the simple substance model considering the effect on the power line was proposed.

INTRODUCTION

A severe earthquake with a moment magnitude of 7.7 occurred on September 21 in 1999, in the central part of Taiwan (23.85 north latitude and 120.81 east longitude, 6.99km depth). In the vicinity of hypocenter, a lot of human lives were lost in the earthquake. And the damage of infrastructure was very serious. The lifeline systems such as roads, electric power facilities, water supply plants, and communications networks came to a standstill for a long time. The Ministry of Education of Japan sent fact finding mission to Taiwan in October 1999. (The research representative: Professor Hirokazu Iemura, Kyoto Univ.) One of the authors of this paper investigated the damage of electric power facilities as a member of the group. This paper describes the analytical examination to find the cause of damage of 345kV steel tower for extra-high voltage power transmission and the numerical evaluation of the effect on the power line to the dynamic behavior of strain steel tower.

DAMAGE OF STEEL TOWER

A shortage of electric power in the northern region was serious in Taiwan, and the electric power of 4 million kW was supplied from the central and the southern regions to northern region by way of two routes. The first route in the east side took charge of 2/3 of total power transmission, and the second route of the west side covered 1/3. The third route was under construction to meet the increasing demand of

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electric power consumption in the northern region. The two routes were extended along the earthquake fault. For this reason, the damage of the steel towers was serious, especially near the Chungliao switching yard. It was reported that inclination and deformation of steel tower, destruction and displacement of foundation of steel towers caused by landslides¹). The outline of the damage was announced on October 8 in 1999 by Taiwan Power Company. Generally in the design of a steel tower, wind load is larger than ordinary seismic load, so the wind load controls the structural profile. The consideration on the seismic load was not important except for the design of well type foundation in Taiwan. Compared with the wind load in Japan, the average standard wind velocity V₀ in Taiwan is 44.9m/s for the important route in the eastern district, 54.4m/s in western district considering the typhoon, while V₀ in Japan is 40m/s. Generally tensile strength of a cable is larger than that of a steel tower, Tensile load is transmitted to the steel tower from the cables in both sides in the longitudinal direction when the foundation moves in landslides by the earthquake. Fig.1 shows collapsed #203 steel tower that is the largest scale in Taiwan. Judging from the photograph, the steel tower was collapsed toward the valley side. However, the steel towers of #202 and #204 standing on the both sides of #203 had no damage according to the report of Taiwan Power Company. The height of the steel tower is about 62.95m, as it is shown in Fig.2, and the distance between the legs at the foot is 12.8m. The steel tower is mostly built on the slope of mountainous area. The report says the difference of elevation of legs is about 10m. According to the earthquake observation record of Taiwan Central Meteorological Observatory, the east side of the fault was shaken hard. So it is also considered that the damage of the steel tower was possibly caused by the strong vibration.

ANALYTICAL EXAMINATION

Modeling

According to the past researches, the effect on power lines to the dynamic response of steel tower in the earthquake is not so influential in transversal direction. But, it is influential in longitudinal direction. Considering this knowledge, the steel tower was modeled based on the draft obtained from Taiwan Power Company. We prepared four types of analysis model. Model A is a three tower model consisting of one detailed tower model in the center and two simplified beam towers on each side considering the effect on power lines. The distance between two towers was assumed 550m from the information of Taiwan Power Company. The precise behavior of central tower during the earthquake can be simulated using this model. Model B1 is a single detailed tower model not considering the effect on power lines. Model B2 is a single detailed tower model considering 50% of total weight of power lines on each side. The weight of power lines is directly put on the end of the arm of the tower in this model. Model B3 is a single detailed tower model considering 50% of total weight of cables on each side too. In this model, the weight of power lines is attached to the top of the arm through the spring considering the natural period of power lines. The gross weight of the steel tower in model A and B1 is about 357kN not considering the weight of power lines. On the other hand, the gross weight of the steel tower in model B2 and B3 is about 572kN considering the weight of the power lines. Analysis models are shown in Fig.3. The foundation of the steel tower was assumed as rigid. Main members of the steel tower were modeled by three-dimensional beam element. Additional members of the steel tower were modeled by the three-dimensional truss element. The power lines were modeled by the cable element. The design criterion of a steel tower in Japan follows JEC-127, the design of a steel tower in Taiwan is in accordance with this design criterion. The spring constants of model B3 in x and z directions were estimated from the natural period of the power lines. Therefore, the mass of the power lines was considered as the concentrated mass on the node of support position of the arm in model B2 and B3. The damping ratio of steel tower in a large earthquake has not been made clear, the value of damping ratio varies from 1% to 10%. In this study, the damping ratio of members was all assumed 2%. And the damping ratio of power lines was assumed 0.4% from the past experiment. In this study, Rayleigh damping was used in the analysis. The dimension of the power lines is shown in Table 1.

Input seismic wave

The seismic waves used in the analysis are the record of TCU084 and TCU068 observed near #203 steel tower. Observed point of TCU084 is about 10 kilometers far from the epicenter and is 20 kilometers far from #203 steel tower. Maximum acceleration of TCU084 is 989gal. Large displacement of the ground induced by the fault occurred at observed point of TCU068. So long period components are prominent in this seismic wave. Fig.4 shows an acceleration response spectrum with damping ratio 2% of TCU084 and TCU068 in each direction.

Eigenvalue analysis

Before dynamic response analysis, eigenvalue analysis was conducted. Table 3 shows the results of eigenvalue analysis. Principal modes of model A are shown in Fig. 5 and Fig. 6. Comparing with Model A, Natural periods of model B3 approximately agrees with that of model A in each direction. On the other hand, natural period of transversal direction in model B1 approximately agrees with that of model A. But natural period of longitudinal direction is shorter than that of model A. Reversely natural period of longitudinal direction is longer than that of model A. But natural period of transversal direction is longer than that of model A. From above results, the steel tower and the power lines shake independently in transversal direction by the difference of stiffness. However, the steel tower and the power lines shake together in longitudinal direction. Therefore, the inertia force of power lines affects on the response of the steel tower.

Dynamic response analysis

After reproducing initial stress state by dead load, dynamic response analysis was conducted. Seismic waves were inputted in each direction separately. From dynamic response analyses using each model, time histories of axial force of main member were obtained and are shown in Fig.7, Fig.8 and Fig. 9. Maximum value of compressive axial force normalized by that of model A is shown in Fig. 10. From Fig. 7, first mode of the steel tower in transversal direction is prominent in model A. Same behaviors are confirmed in Model B1 and model B3. But the behavior of model B2 is different from other models. Judging from Fig. 10, the response is overestimated due to fixing the weight of power lines in model B2. From Fig. 8, first mode of the steel tower in longitudinal direction is prominent in model A. Same behaviors are confirmed in Model B2 and model B3. But the behavior of model B1 is different from other models. Judging from Fig. 10, the response is underestimated due to neglecting the weight of power lines in model B1. From Fig. 9, first mode of the electric wire in transversal direction is prominent in model A. Same behavior is confirmed in model B3. But the behaviors of model B1 and model B2 are different from other models. It is difficult to estimate the effect on the power lines in model B1and model B2. The result of TCU068y is omitted, because the tendency is the same as that of TCU084y. Examining the effect on power lines to the behavior of the steel tower under various conditions, it developed that model B3 can simulate the dynamic behavior of the steel tower during the earthquake as accurately as model A. The error to the response of model A was 20% in maximum.

CONCLUTION

The summary of the results is shown below.

Simplified single tower model considering the effect on power lines was proposed. The model can accurately simulate the dynamic behavior of the steel tower during the earthquake.

Heavy damage of steel tower was caused in Chi-Chi earthquake. Japan has not experienced the damage like this. Observed earthquake ground motion has equivalent energy as Hyogo-ken Nanbu Earthquake. The design specification of steel tower in Taiwan is almost the same in Japan, and the possibility of the similar damage in Japan can be considered. According to the analysis for the elucidation of the cause of the damage, large axial force arises in main member of the leg.

In the future, researches on the evaluation of the steel tower with unequal legs is important, because the steel tower is often built on a slope in mountain area. In this study, rigidity of the ground was not considered. Therefore the collection of detailed information of the steel tower is necessary in a future examination.

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Fig.1 Collapsed steel tower for extra-high voltage power transmission



Fig.2 Structural outline of 345kV steel



Fig.3 Analysis model

| Table 1 Dimensic | on of the p | power lines |
|------------------|-------------|-------------|
|------------------|-------------|-------------|

| Electric wire | Line | 2 circuits (3 lines per circuit) | |
|------------------|---|----------------------------------|--|
| | Weight per length | 62.76N/m per line | |
| | Outer diameter | 51.92mm | |
| | Tensile force during additional dead load | 141.1kN per line | |
| Earth wire | The number of wires | 2 lines | |
| | Weight per length | 7.286N/m per line | |
| | Outer diameter | 16.32mm | |
| | Tensile force during additional dead load | 19.6kN per line | |

| Model | B1 | B2 | B3 | |
|----------------------------|---------------------|---|---|--|
| Mass | | 50% of total weight of wires on each side | | |
| The position of added mass | No consideration | Fix at the top of the arm | y direction: Fix at the top of the arm x direction and z direction: Fix at the top of the arm with the spring | |

Table 2 Consideration of power line



Fig.4 Acceleration and displacement response spectrum (h=2%)

| Mode | | Model | | | | | |
|------------|----------------|---------|-------|-------|-------|-------|-------------------------------------|
| | | А | B1 | B2 | B3 | Note | |
| Power line | x direction | first | 6.189 | - | - | 6.190 | (Earth wire) |
| | | | 7.833 | - | - | | (Electric wire (upper)) |
| | | | 7.688 | - | - | 7.431 | (Electric wire (middle)) |
| | | | 7.574 | - | - | | (Electric wire (downer)) |
| Tower | x direction se | first (| 0.473 | | | 0.461 | Earth wire:13th, Electric wire:18th |
| | | | 0.469 | 0.463 | 0.780 | | Earth wire:13th, Electric wire:19th |
| | | | 0.456 | | | | Earth wire:15th, Electric wire:20th |
| | | second | 0.185 | 0.179 | 0.318 | 0.179 | |
| | у | first | 0.747 | 0.458 | 0.742 | 0.742 | Earth wire:10th, Electric wire:12th |
| | direction | second | 0.264 | 0.172 | 0.256 | 0.256 | |
| | z direction | second | 0.069 | 0.068 | 0.173 | 0.069 | |

Table 3 Natural period (sec)







Fig.5 Results of eigenvalue analysis



Fig6 Results of eigenvalue analysis



Fig.7 Time history of axial force of main member 1 (Input TCU084x)



Fig.8 Time history of axial force of main member 1 (Input TCU084y)



Fig.9 Time history of axial force of main member 1 (Input TCU068x)



Fig.10 Maximum value of compressive axial force of main member