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Performance evaluation of a transmission tower by substructure test

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ABSTRACT

In this paper, a half-scaled substructure test was performed to evaluate the failure mode of an existing transmission tower subjected to wind load. A loading scheme was devised to reproduce the dead and wind loads acting on the prototype transmission tower. The load was enforced on the model structure using two actuators and a triangular jig mounted on the reduced model. Preliminary numerical analysis was carried out to evaluate the stability and member force of the specimen for the design load. When the substructured transmission tower was loaded by 270% of its maximum allowable buckling load, local buckling occurred in joints of leg members with weak constraints. From the experimental results, such as load–displacement curves, displacements, and strains of members, it was concluded that the local buckling was due to the additional eccentric force caused by unbalanced deformation of the specimen.

1. Introduction

Recently, demand on electrical power has been increasing around the world and many large-scale transmission towers have been newly constructed. Many transmission towers located in an open terrain are exposed to strong winds. In 2003 nine transmission towers collapsed when the typhoon 'Maemi' swept the Korean peninsular causing enormous economical loss [10]. After the disaster, the Korean Electrical Power Corporation (KEPCO) revised the design code for transmission towers, reflecting the enhanced hazard level for wind load [5,6]. Guidelines for retrofit methods such as increasing member cross-sectional area or reducing unbraced length by installation of brace members were also proposed [5,6]. Many existing transmission towers in Korea have been retrofitted based on the enhanced design load and the recommended retrofit methods.

Albermani et al. [2] proposed retrofitting methods such as adding diaphragm and constraining the out-of-plane deformation of each face of transmission tower, and verified the performance both experimentally and numerically. Alam and Santhakumar [1] carried out a loading test of a 34 m-high transmission tower with a capacity of 220 kV, and found that the buckling of tower leg members and cross-arm bottom members caused failure of the transmission tower. Based on the test results, they suggested the reduction of the maximum slenderness ratio, 150, regulated in the design codes [3,4] to 110. Momomura et al. [9] and Okamura et al. [11] investigated the dynamic characteristics of transmission towers built in mountainous areas based on wind-induced vibration data and numerical analysis. Kim and Lee [7] performed a loading test of a 78 m-high transmission tower constructed with circular tube sections.

In this paper, a half-scaled substructure test was performed to evaluate the behavior and failure mode of an existing transmission tower subjected to wind load. A loading scheme was devised to reproduce the gravity and wind loads acting on the prototype transmission tower. The load was enforced on the model structure using two actuators and a triangular jig mounted on the top of the specimen. Preliminary numerical analysis was carried out to evaluate the stability and member force of the specimen for the design load.

2. Test setup

2.1. Scaled model for a transmission tower

A 154 kV B2-type transmission tower of height 38.1 m illustrated in Fig. 1 was chosen as a prototype structure for experiment. In both sides of the tower 300 m-long electric wires are connected. For experiment two units in the middle of the tower were modeled with a 1/2 scale in length. In this case the cross-sectional areas are reduced to 1/4 of those of the prototype structure. The height of the test specimen is 3 m, and the plan dimensions of the bottom and top of the specimen are 2 m × 2 m and 1.5 m × 1.5 m, respectively.

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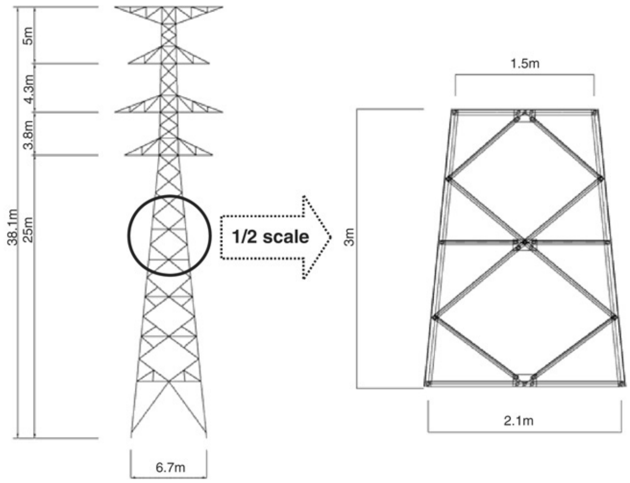

Fig. 1. A prototype transmission tower and a half scaled sub-assemblage test model.

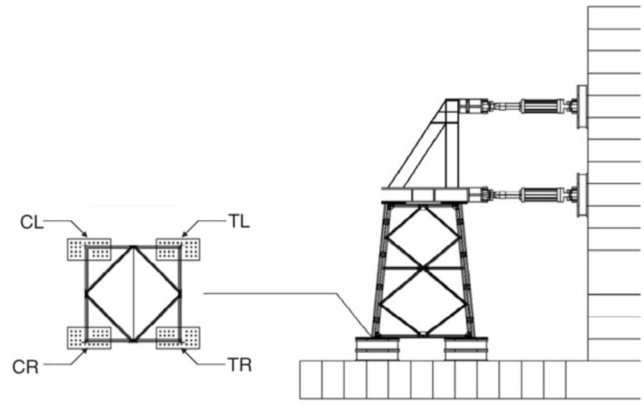

Fig. 2. Configuration of test model structure and actuators.

Table 1
Sectional properties of the prototype transmission tower

Section size (mm)	Cross-sectional area (cm^2)	Second moment of inertia (cm^4)	Radius of gyration (cm)
HL 150 × 12	34.77	304.00	2.96
L 65 × 6	7.537	12.20	1.27
L 60 × 4	4.64	6.62	1.19
L 50 × 4	3.84	3.76	0.983

Table 2
Sectional properties of the scaled model structure

Section size (mm)	Cross-sectional area (cm^2)	Second moment of inertia (cm^4)	Radius of gyration (cm)
L 75 × 6	8.727	19.00	1.48
L 60 × 4	4.692	6.62	1.19
L 45 × 4	3.492	2.69	0.88
L 45 × 4	3.492	2.69	0.88

The elements of the specimen are composed of angle sections made of SS400 steel (Fy = 240 MPa and Fu = 330 MPa). Member sizes and sectional properties of the prototype structure and the scaled test model are listed in Tables 1 and 2. Unlike the tower legs, the brace members have sections a little larger than the required ones due to the lack of the standard section satisfying the similarity law. However, since the forces in the bracing members are less than 4.5% of those of the tower legs, the inaccurate scale factors for the braces have little effect on the performance of a transmission tower. Zinc galvanized bolts with diameter of 16 mm (M16) were used instead of M20 for the same reason. To make the upper boundary conditions similar to the prototype structure, steel plates were welded to the top of the specimen and to the bottom of the jig.

2.2. Loading on the prototype and the test model structures

The loads acting on a transmission tower are as follows: vertical load; lateral longitudinal and transverse loads; and longitudinal unbalance load. The vertical load comes from mostly self-weight such as shield wires, conductors and insulators. The lateral longitudinal load is the tensile force of electric cables and shield wires. The lateral transverse load acts perpendicular to the direction of electric cables, and the wind load and the load due to angle of line deviation correspond to this category. The longitudinal unbalance load is the torsion caused by severance of transmission lines. The transmission towers are generally designed with maximum stress computed by various load combinations, including the loads described above. In this experimental study the load was enforced along the transverse direction considering the fact that the largest lateral load acts along that direction and that the experiment is carried out in a laboratory with limited number of actuators.

The load applied to the test specimen was determined following the similarity law. As the wind load is proportional to the surface area of the structure, the lateral load imposed on the test specimen is reduced to 1/4 of the design load of the prototype structure. As the weight is proportional to the volume of members, the dead load imposed on the model structure was reduced to 1/8. As the weight of the jig was similar to the dead load to be imposed on the test specimen, no additional load was applied.

Generally a loading test of full-scale transmission towers uses a wire connected to each loading point. But, in this study, two hydraulic actuators were used to apply desired loading on the specimen as shown in Fig. 2. The actuators impose both bending moment and lateral shear force to simulate the loads transmitted from the removed upper substructure through a triangular jig. This test setup has advantage in that local failure modes can be more thoroughly observed. The actuators with the capacity of 140 MN and 90 MN were placed at the height of 5 m and 3 m from the ground, respectively. The height of the specimen is 3 m, and the maximum height that an actuator can be placed in the laboratory is 5 m. Therefore a 2 m moment arm is available to impose bending moment on the specimen. However the moment arm required to meet the similarity law is approximately 8 m. To provide the specimen with equivalent bending moment the lateral force from the actuator should be increased significantly. This, however, results in unnecessarily high shear force acting on the structure. To reduce the shear force down to the design load level, another actuator was installed at the height of 3 m to apply load in the reverse direction. To carry out the experiment by displacement control, the displacement under design load was computed by numerical analysis, and the displacement was increased until failure of the structure. The test setup is depicted in Fig. 2.

2.3. Installation of strain gauges and LVDT's

The locations of strain gauges and the naming of elements and joints are shown in Fig. 3. The name of strain gauges located between the joints starts from M and the name of those placed near the joints starts from J. As the number of channels in the data logger is only 60 and there are many structural members, strain gauges were installed in limited locations. The expected failure mode of a transmission tower subjected to an overturning moment is the buckling of the compression members. According

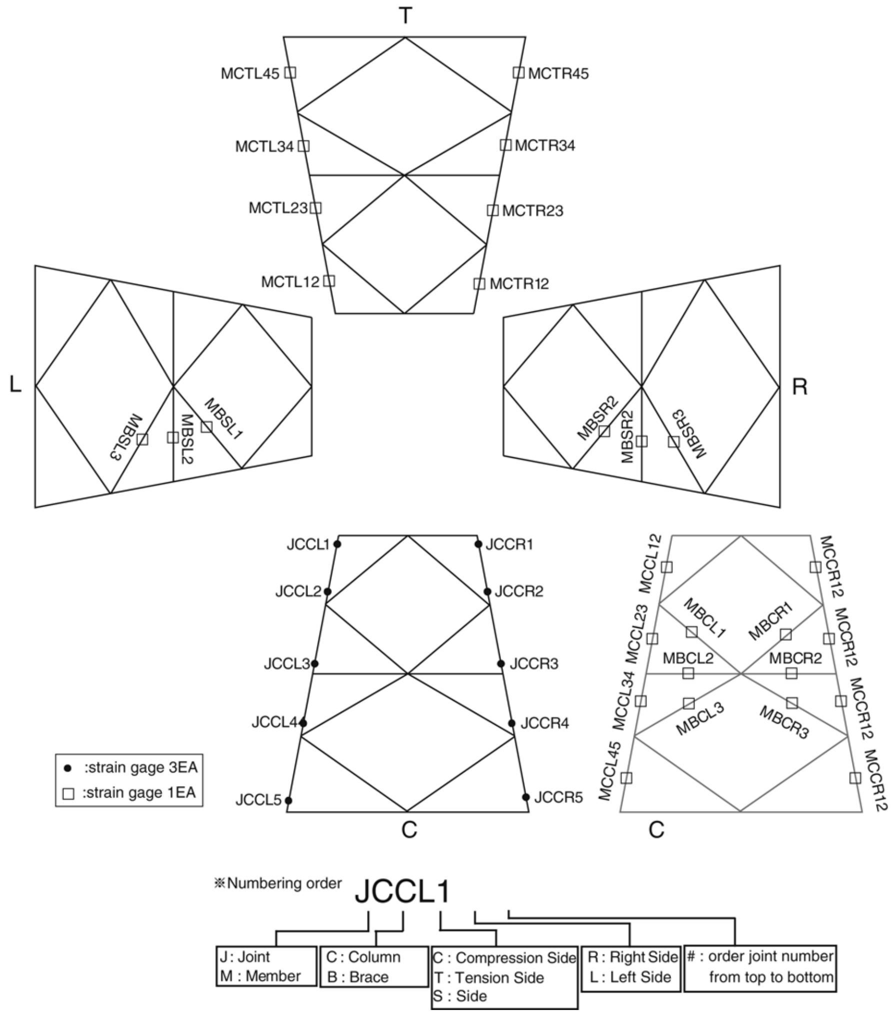

Fig. 3. Naming and location of strain gages.

to the analysis results, the compression in bracing members is significantly smaller than the buckling load, and therefore most of the strain gauges were installed in leg members subjected to compression. As no axial force exists between the connections in columns subjected to compression, strain gauges were installed close to the connections. On the other hand, a strain gauge was placed between the connections in columns subjected to tension. In the tower lags subjected to compression, three strain gauges were installed at one place, at two ends and a corner of angle sections as shown in Fig. 4, to measure bending moment and axial force imposed on the section. On the other hand, in tower legs under tension the strain gauges were installed only in the middle of the upper legs, which were expected to experience the largest member force. In bracing members, the strain gauges were installed in the middle of the members. As the displacement at the top of the specimen along the loading direction is recorded by the actuator, two LVDT's were installed perpendicular to the loading direction. In the mid-height of the specimen, two LVDT's were installed at each corner of the specimen under compression because no diaphragm action from the jig was expected (Fig. 5).

Fig. 4. Placement of strain gages near a joint of a leg member.

3. Numerical analysis

3.1. Preliminary analysis

For numerical analysis the specimen was modeled by beam elements and all elements were assumed to be rigidly connected. The triangular upper part of the jig was modeled by beam elements and the remaining lower part was modeled by shell elements. The beam and the shell elements were connected by rigid links. The numerical modeling and analysis of the model structure were carried out using the finite element analysis program package MIDAS-Civil [8]. Fig. 6 shows the analysis model of the test

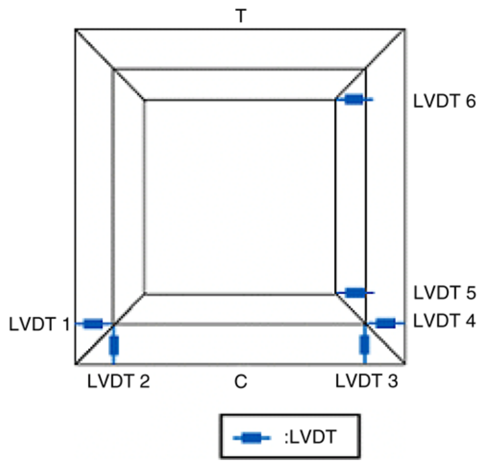

Fig. 5. Location of LVDT's.

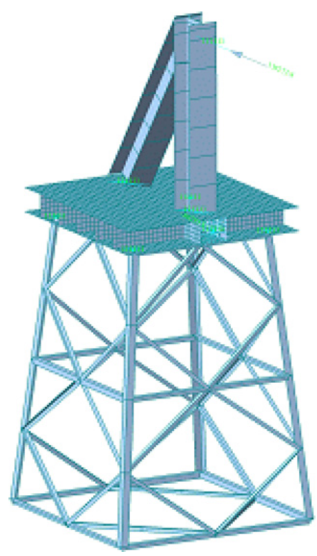

Fig. 6. Finite element modeling of the transmission tower test specimen (MIDAS Civil).

specimen. The applied load was obtained from the design load of the transmission tower (Tables 3–5).

The analysis results showed that the maximum displacement of the specimen was 3.217 mm with inter-story drift of 0.1072%. Even though the lateral displacement was not very large, significant amount of lateral drift was expected at the top of the prototype transmission tower considering the cantilever mode deformation of a transmission tower. Next the analysis results for member forces were compared with those computed by equations recommended by design guidelines for transmission towers [5,6]. The recommended equations are as follows:

– For structural members with small eccentricity (leg members, main members in cross arm)

$$t \leq 16 \quad \text{and} \quad 0 < \lambda_k < 105 : \sigma_{ka} = 1550 - 23(\lambda_k/100) - 602(\lambda_k/100)^2 \quad (1)$$

$$t \leq 16 \quad \text{and} \quad \lambda_k \geq 105 : \sigma_{ka} = 950(\lambda_k/100)^2. \quad (2)$$

– For structural members with small eccentricity (bracing members)

$$t \leq 16 \quad \text{and} \quad 0 < \lambda_k < 135 : \sigma_{ka} = 1550 - 762(\lambda_k/100) \quad (3)$$

$$t \leq 16 \quad \text{and} \quad \lambda_k \geq 135 : \sigma_{ka} = 950(\lambda_k/100)^2 \quad (4)$$

Table 3
Design load for prototype transmission tower perpendicular to the wind direction

Projected area of cross arms	1.41 (m^3)
Projected area of a tower body	9.29 (m^3)
Shear force by cross arms	4.78 (kN)
Shear force by a tower body	34.24 (kN)
Shear force by the load due to angle of line deviation	165.25 (kN)
Sum of upper shear force	198.84 (kN)
Sum of upper moment	2209.4 (kN m)

Table 4
Design gravity load on the prototype structure

Shield wire weight	14.58 (kN)
Conductor weight	85.5 (kN)
Upper tower weight	34.52 (kN)
Total weight	134.6 (kN)

Table 5
Loads imposed on the test specimen

Upper actuator force	137.2 (kN)
Lower actuator force	−88.2 (kN)
Gravity load	16.86 (kN)

where λ_k is the effective slenderness ratio of a member, l_e/γ, where l_e is the effective length and γ is the radius of gyration of a member. In transmission towers the following values are generally used for effective length: for leg members, $l_e = 0.9l$; for bracing members, $l_e = 0.8l$. The member forces of the compression members obtained from numerical analysis and the buckling strengths obtained from Eqs. (1) to (4) are presented in Table 6, where the values inside of the parentheses are the ratios of the buckling strengths obtained from numerical analysis and from the formulas. It can be observed in Table 6 that the axial forces of leg members computed by numerical analysis ranged from 80% to 90% of the allowable buckling loads. Therefore the test specimen was considered to have enough strength for the design load. When the structure is subjected to external load greater than the design load the leg members at the lower part of the specimen are most vulnerable for buckling. On the other hand, the member forces in the bracing members were less than 13% of the allowable buckling strength. This is due to the fact that the structure is mainly deformed in bending mode, and therefore it can be expected that the reinforcement of the braces will not increase the strength and stiffness of the transmission tower significantly. This observation may justify the use of braces with slightly larger cross-sectional areas than required by the similarity law in this experimental study.

3.2. Nonlinear analysis

Nonlinear analysis of the specimen was carried out using the finite element analysis program ANSYS Structural Utility. Nonlinear material property was found from the coupon test result shown in Fig. 7. Fig. 8 shows the nonlinear analysis model of the test specimen. The triangular upper part of the jig and angle members were modeled by BEAM 188. The plate steel part of the jig was considered as a rigid link since it shows rigid body motion during the experiment. The bolt connection was modeled by COMBIN7 to consider applied torque of the bolt. COMBIN7 is a 3-D pin joint which may be used to connect two or more parts of the model at a common point. Capabilities of this element include joint flexibility (or stiffness), friction, damping, and certain control features. An important feature of this element is a large deflection capability in which a local coordinate system is fixed to and moves with the joint. Fig. 9 shows the stress distribution and deformation shape of a test specimen as a result of the nonlinear analysis.

Table 6
Member forces and allowable buckling loads of compression members

Members	Length (cm)	Slenderness ratio	Allowable buckling stress ($\times 10^4$ kPa)	Allowable buckling load (kN)	Preliminary analysis results (kN)
MCCL12 MCCR12	72.7	28.4	−14.64	−127.38	−101.71 (79.9)
MCCL23 MCCR23	69.4	27.2	−14.69	−127.8	−106.38 (83.2)
MCCL34 MCCR34	81.9	32.0	−1.480	−126.18	−106.19 (84.2)
MCCL45 MCCR45	72.7	28.4	−14.50	−127.38	−112.11 (88.0)
MBSL1 MBSR1	107.7	71.8	−9.82	−34.37	3.2 (−9.3)
MBCL1 MBCR1					−1.86 (5.4)
MBSL2 MBSR2	79.50	53.0	−11.23	−39.31	0.85 (−2.2)
MBCL2 MBCR2					1.57 (−3.9)
MBSL3 MBSR3	118.1	78.7	−9.31	−32.59	−4.29 (13.2)
MBCL3 MBCR3					−2.18 (6.7)

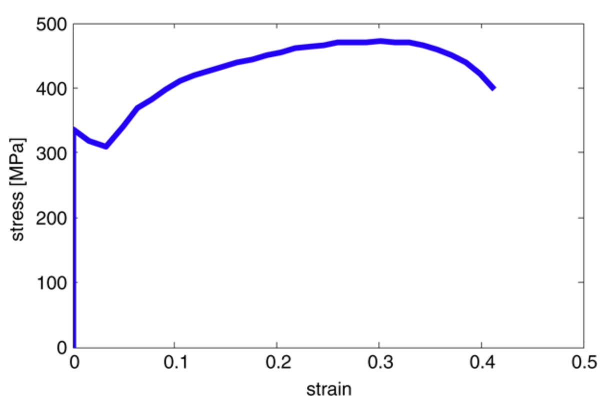

Fig. 7. Stress–strain curve of the steel angle (coupon test result).

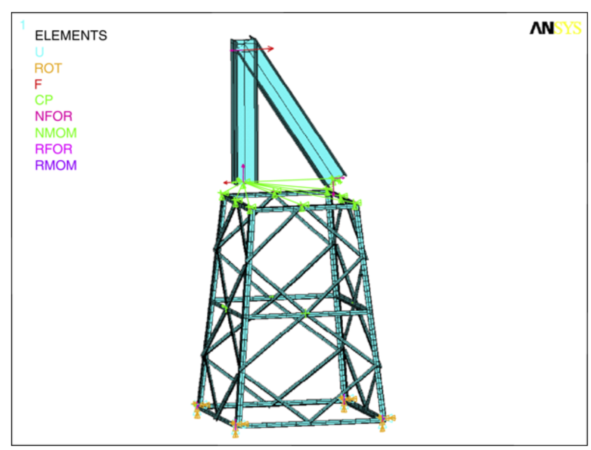

Fig. 8. The nonlinear analysis model of the transmission tower test specimen (ANSYS Structural U).

Table 7
Strains of bracing members obtained by preliminary analysis and experiment

Members	Experiment (10^{-6})	Nonlinear analysis (10^{-6})	Experiment/analysis
MBSL1	28.56	20.16	1.42
MBSR1	77.11	62.34	1.24
MBCL1	−19.04	−15.95	1.19
MBCR1	4.76	5.03	0.95
MBSL2	−2.86	−2.97	0.96
MBSR2	−10.47	7.27	−1.44
MBCL2	8.57	6.59	1.30
MBCR2	15.23	6.59	2.31
MBSL3	−28.56	−34.73	0.82
MBSR3	−27.61	−32.52	0.85
MBCL3	13.32	−6.49	−2.05
MBCR3	−24.75	−16.70	1.48

It is observed that the yield stress occurred in the “JCCL2” and “JCCR3”. This local buckling in both “JCCL2” and “JCCR3” coincides with the experimental results shown in Fig. 10. Accordingly, the nonlinear analysis results performed in this study can estimate the experimental results.

Fig. 9. Von Mises stress and deformation of nonlinear analysis results.