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ANALYTICAL STUDY ON SEISMIC PERFORMANCE EVALUATION OF LONG-SPAN SUSPENSION BRIDGE STEEL TOWER

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SUMMARY

This paper presents the limit state evaluation of the steel tower structures of a long-span suspension bridge against large-scale earthquakes. A series of pushover analyses using two types of analytical models with shell and fiber elements and nonlinear dynamic analyses were conducted. The strength and damage progress characteristics obtained from two analytical models and methods were compared. Based on the analytical results, the acceptable ductility capacity for the steel tower structure exceeding the elastic limit was proposed. Furthermore, the results from both the pushover analysis and nonlinear dynamic analysis showed good agreement.

INTRODUCTION

After the Hyogo-ken Nanbu Earthquake in 1995, the Japanese seismic design code for highway bridges [1], targeting mid-span bridges, has adapted the performance-based design philosophy that the performance could range from fully operational to no critical damage depending on the earthquake level. On the other hand, existing long-span suspension bridges in Japan have been designed to remain in elastic state in case of earthquake [2]. In strait crossing projects in Japan, however, the construction of a long-span suspension bridge is investigated in the strait point where the occurrence of large-scale earthquakes is predicted. Although the wind load is generally dominant for the design of superstructure of a long-span suspension bridge, if inelastic response were not accepted in case of large-scale earthquakes, seismic load would possibly become significant to the design. Therefore, from the economical point of view, it is necessary to identify acceptable damage levels of a suspension bridge and to establish verification procedures for seismic performance against large-scale earthquakes.

Above background motivated authors to investigate the limit state of a long-span suspension bridge steel tower against large-scale earthquakes. Pushover analyses with a tower model composed of combined shell and fiber elements were conducted [3]. In this study, two types of analyses are performed; one is pushover analyses with tower models idealized by shell and fiber elements, the other is nonlinear dynamic analyses with a 3-D full bridge model idealized by fiber elements. Nonlinear force-displacement relationship, the

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damage progress, damage level and strain ratio are characterized to see the acceptable damage level of the suspension bridge steel tower. Also discussed is the difference of nonlinear behavior of the tower resulting from different analytical models and methods for the seismic performance evaluation.

PUSHOVER ANALYSES

Suspension Bridge Steel Tower Analyzed

The analyzed tower, designed as a long-span suspension bridge with 2300m in the center span, is a 288m high rigid frame structure with four lateral beams. Figure 1 shows the tower elevation and the cross-section. The tower is designed considering three load combinations: 1) a combinations relating to vehicular uses when the tower top is exposed to the maximum vertical load without wind; 2) a combinations relating to vehicular uses when the tower top is exposed to the maximum horizontal displacement without wind, and 3) a load combination, dominant one for the tower deign, relating to wind on live load. The tower is not designed for seismic load. The cross-section of the columns consists of three cells, and the longitudinal and transverse widths are 7.6m and 13.0-8.0m, respectively. The shaft plates, 36-45mm thick, have material properties of Japanese Industrial Standards (JIS) SM490Y and JIS SM570. The four lateral beams are named as the first beam at the bottom through the fourth beam at the top. The beam plates, 12-18mm thick for flanges and 30-45mm thick for webs, have material properties of JIS SM490Y at the connection with the columns and JIS SM400 at the middle of the beams.

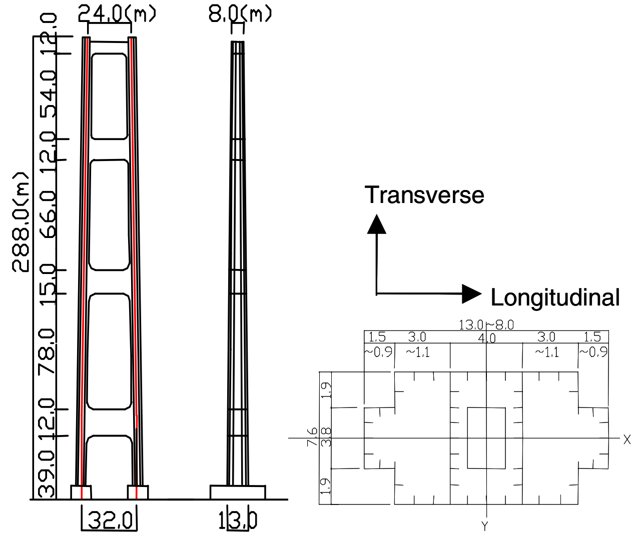


Figure 1: Tower Elevation and Cross-Section

Analytical Model

Four types of analytical models with different finite element idealization and the direction are used in the pushover analyses; the tower elements are modeled with the shell elements (Shell Model) and fiber elements (Fiber Model) in the longitudinal and transverse directions. The beam-column connections are idealized as rigid assuming them sound during earthquakes. A stress-strain relationship is assumed to be bilinear with 0.01E (E: Young's modulus) as the second gradient. For the Shell Models, half part of full structure is modeled considering the symmetrical condition, as shown in Figure 2. An elastic spring equivalent to the restraint by the main cable is attached to the tower top in the longitudinal direction models.

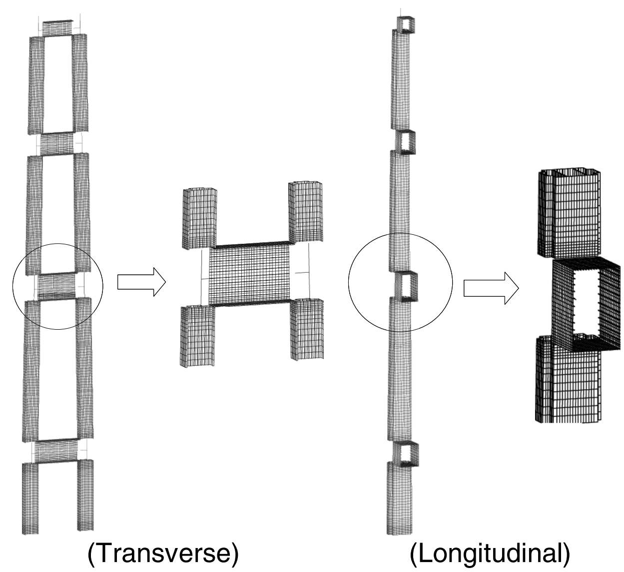
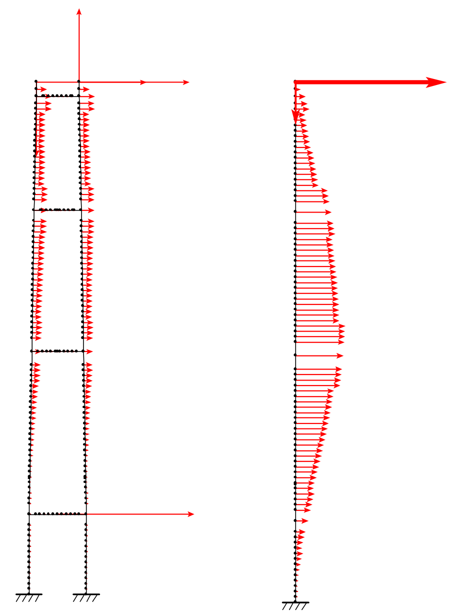


Figure 2: Shell Model

Analytical Methodology

Pushover analyses were performed by the following procedure. Seismic load was applied step by step to the analytical model which was subjected to the dead load and the cable reaction force as its initial condition. The seismic loads, as illustrated in Figure 3, were inertial force distributions obtained from linear time history dynamic analyses using the 3-D full bridge model. The inertial force distributions at the time when the bending moment at the tower base reached the maximum in the dynamic analyses were selected. For the dynamic analyses, the same input ground motion as the one for after-mentioned nonlinear dynamic analyses was used. The inertial forces were loaded to diaphragms in the Shell Models and to nodes in the Fiber Models. In addition to the inertial force, fluctuations of the cable reaction forces at the tower top were considered in the seismic loading. In the pushover analyses, both material and geometric nonlinearities were incorporated.

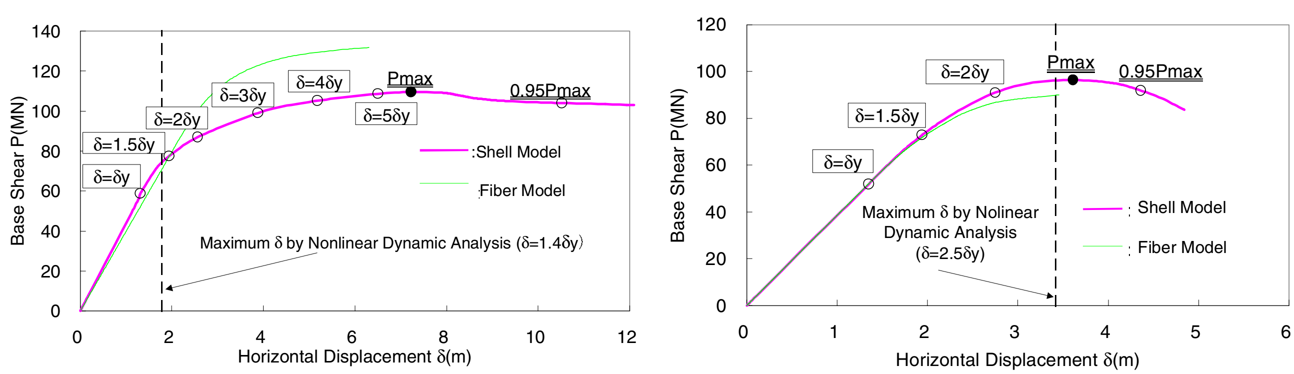


(Transverse) (Longitudinal)

Figure 3: Seismic Loading Condition

Results of Analyses

Figure 4 shows the results of base shear (P) versus horizontal displacement (δ) relationship in both directions. The displacements correspond to the ones at the tower top in the transverse direction and at the most horizontally deformed point in the longitudinal direction. δ_y designates the displacement when the first yielding develops at any shell element of the Shell Models. δ_y in the transverse and longitudinal direction were 1.3m and 1.9m, which were the displacements when the first beam and the most horizontally deformed point of the column reached yield strength, respectively. In the figure, the maximum displacements by the after-mentioned nonlinear dynamic analyses are also described.



(Transverse) (Longitudinal)

Figure 4: Base Shear (P) vs. Horizontal Displacement (δ) Relationship

In the transverse direction, P vs. δ relationship of the Shell Models was linear up to around δ=1.5δ_y. At around δ=1.5δ_y, the stiffness of the whole structure begun to deteriorate, and after δ went over 3δ_y, P scarcely increased as δ did. P reached its peak value (P_{max}) at around δ=5.5δ_y, and gradually fell to 95% of P_{max} (0.95P_{max}) at around δ=10.5δ_y. P_{max} was about 1.9 times of the base shear value at $\delta=\delta_y$. While the P vs. δ relationship of the Fiber Model was almost consistent with the one of the Shell Model during $0<\delta<1.5\delta_y$, the relationship of the Fiber Model deviated from the one of the Shell Model during $\delta>1.5\delta_y$, and P_{max} of the Fiber Model exceeded the one of the Shell Model.

In the longitudinal direction, P vs. δ relationship of the Shell Models was linear up to around $\delta=1.5\delta_y$, and after δ went over $2\delta_y$, P scarcely increased as δ did. P_{max} and $0.95P_{max}$ were generated at around $\delta=2.7\delta_y$ and $\delta=3.2\delta_y$, respectively. P_{max} was about 1.9 times of the base shear value at $\delta=\delta_y$. Although P_{max} of the Fiber Model was slightly less than the one of the Shell Model, the tendencies regarding P-δ relationships between the both models showed approximate consistency.

Figure 5 illustrates the comparisons of deformed tower shapes at $\delta=3\delta_y$ in the transverse direction and $\delta=2.5\delta_y$ in the longitudinal direction. $\delta=2.5\delta_y$ in the longitudinal direction is the maximum δ of the after-mentioned nonlinear dynamic analyses.

In the transverse direction, some local deformation could be observed especially at the second and third beams of the Shell Model, and the columns deformed more in the Shell Model than in the Fiber Model. On the other hand, in the longitudinal direction, deformed tower shapes of the both models showed good agreement.

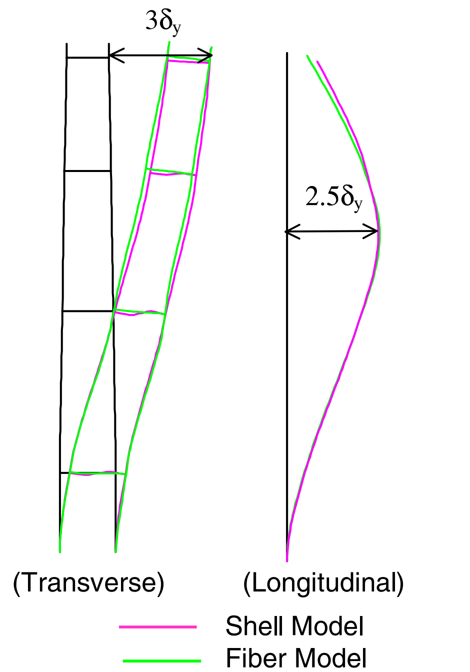


Figure 5: Comparisons of Deformed Tower Shapes

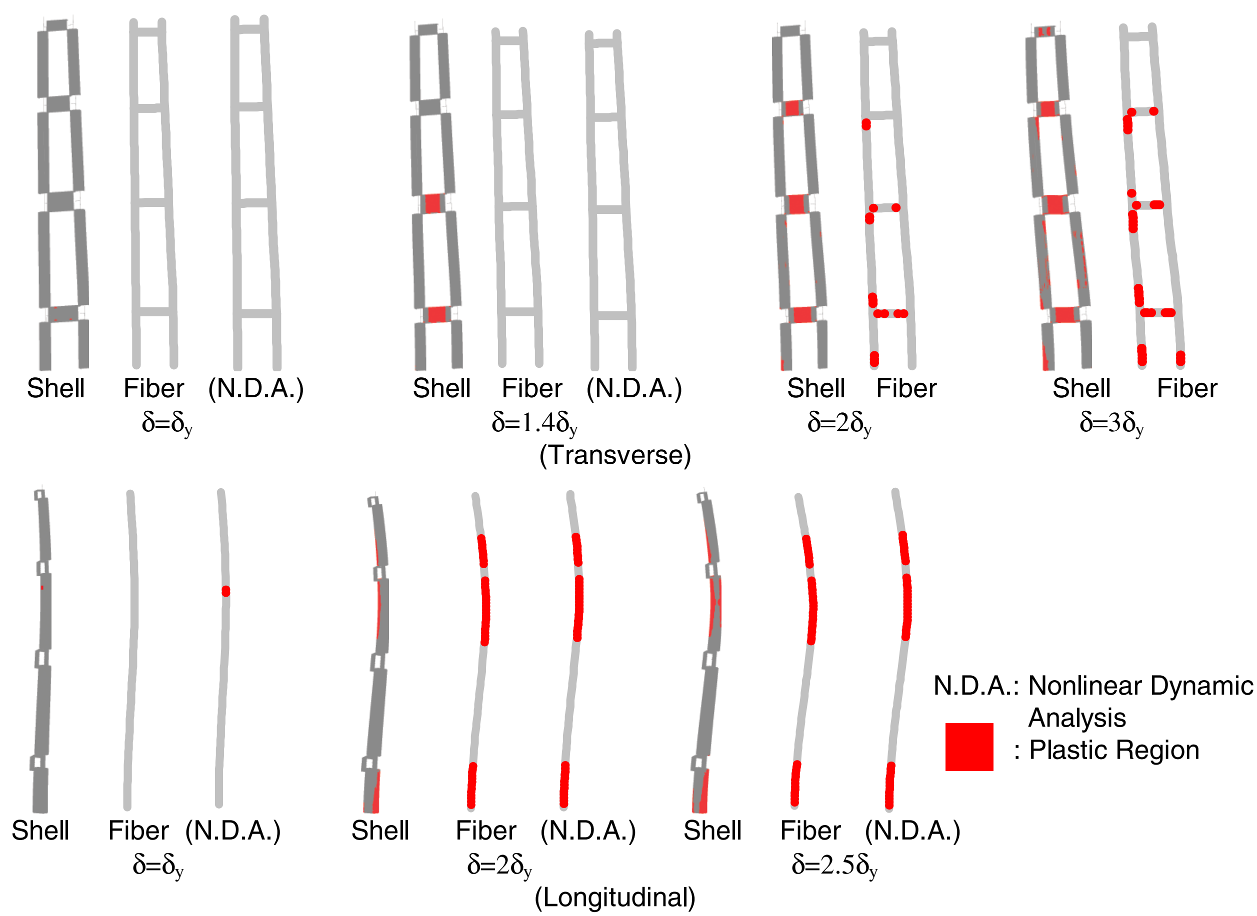


Figure 6: Comparisons of Progress in Plastic Region

Figure 6 shows the comparisons of progress of plastic region with increase of δ. Red region in the figure designates the plastic region where Von Mises yield criterion is used. The results of after-mentioned nonlinear dynamic analyses are also described.

In the transverse direction, the first yielding of the Shell Model developed at around the middle of the first beam, and then the yielding development shifted to the second and third beams. The plastic region extended to the column on compressive side at around $\delta=2\delta_y$. The first yielding of the Fiber Model was not generated until $\delta=1.4\delta_y$, and then the column on compressive side and the both ends of beams developed the plastic region at around $\delta=2\delta_y$. On the other hand, in the longitudinal direction, the first yielding of the Shell Model developed at the column below the third beam, and then the plastic region extended to the tower base and above the third beam at around $\delta=2\delta_y$. While the first yielding of the Fiber Model was not generated at $\delta=\delta_y$, the plastic region spread with increase of δ in the same manner as of the Shell Model.

Figure 7 shows the comparisons of strain ratio, defined as the ratio of resulting strain (ε) to yield strain (ε_y). The strain values are at the flanges perpendicular to the loading direction of some representative elements as indicated in the figure. The positive value indicates tension whereas the negative value indicates compression.

In the transverse direction, the same strains were generated between the both models at $\delta=\delta_y$, and then different strain ratios became to be recognized at the flange on compressive side with increase of δ. Larger values developed in the Fiber Model; the strain ratio of the Fiber Model at the tower base at $\delta=3\delta_y$ was about five whereas the one of the Shell Model was around two. On the other hand, in the longitudinal direction, no apparent differences of resulting strains at $\delta=2.5\delta_y$ could be seen between the both models, even though there were some scatters at the flange on compressive side.

Effects of Difference in Analytical Models

As discussed above, the comparisons in terms of base shear versus horizontal displacement relationship, deformed tower shape, progress of plastic region and strain ratio between the Shell Model and the Fiber Model have been done after performing pushover analyses. Based on the comparisons, the effects of difference in analytical models on strength and damage characteristics of the suspension bridge steel tower will be discussed here.

In the transverse direction, while the differences in the comparisons were small in the region of $0<\delta<1.5\delta_y$, the differences gradually became to be significant in the region of $\delta>1.5\delta_y$. The differences seem to come mainly from the nonlinearity in shear deformation at the lateral beams. As the beams of the tower have small span-to-depth ratio, shear deformation relative to the bending deformation tends to be large. The Fiber Model, not considering shear nonlinearity, was not able to simulate the shear damage generated in the Shell Model, as shown in Figure 8. It is likely that the decrease in constraint of the beams on columns due to the shear damage at the beams led to lower stiffness of base shear versus horizontal displacement relationship in the Shell Model than in the Fiber Model during $\delta>1.5\delta_y$. Nevertheless, the whole structure did not become unstable and had certain ductility after the development of the shear damage at the beams. It should be noted that the seismic performance evaluation by analyses using the Fiber Model may give results on the danger side in the region of $\delta>1.5\delta_y$ because the analysis by the Fiber Model calculated higher strength of the structure than by the Shell Model.

On the other hand, in the longitudinal direction, although detailed aspects with regard to progress in plastic region and strain ratio between the Shell Model and the Fiber Model were somewhat different,

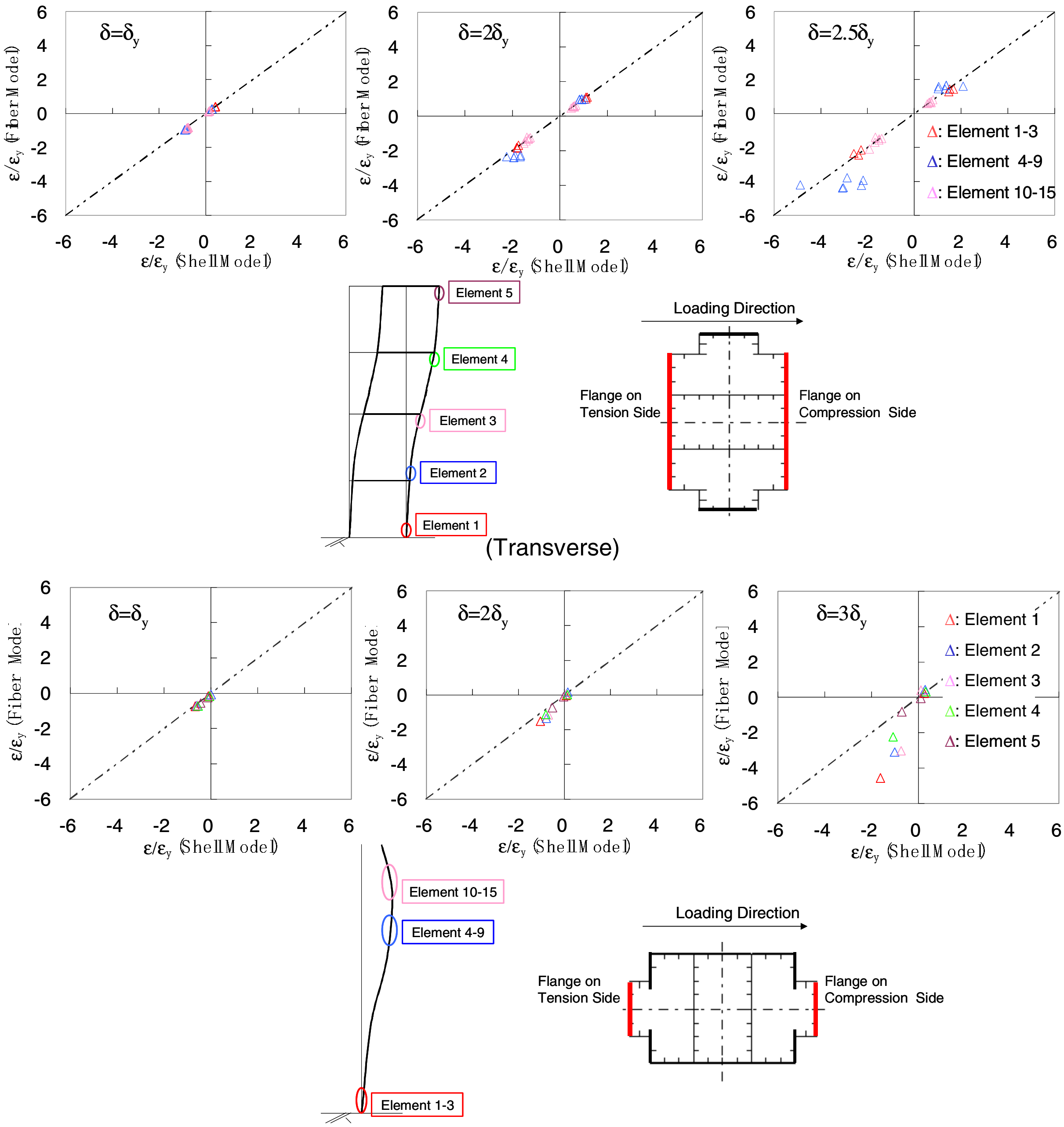


Figure 7: Comparisons of Strain Ratio

base shear versus horizontal displacement relationships and deformed tower shapes showed good agreement between the both. It is possible to say that the analyses using the Fiber Model give the results on the safe side because the strength of the structure calculated by the Fiber Model is slightly smaller than by the Shell Model.

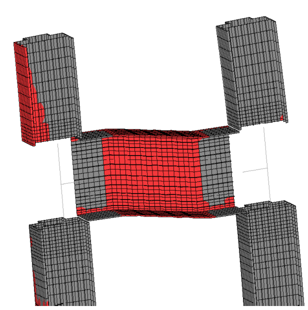


Figure 8: Shear Damage of Beam

NONLINEAR DYNAMIC ANALYSES

Analytical Model

A suspension bridge analyzed here is the same as the one analyzed by pushover analyses shown earlier. As dynamic analyses using the Sell Model is time-consuming and not practical, nonlinear dynamic analyses using 3-D full bridge model with the Fiber Model in the towers were done. The 3-D full bridge model is shown in Figure 9. The Fiber Model in the towers is identical to the one used in the pushover analyses in terms of a stress-strain relationship and element segmentation.

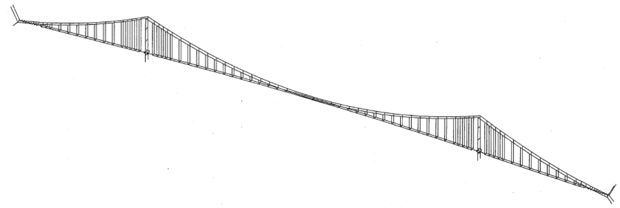


Figure 9: 3-D Full Bridge Model

Analytical Methodology

Figure 10 indicates the acceleration response spectrum of the input ground motion used for the analyses. The ground motion was estimated as a site-specific one induced from specific active faults using an empirical green function. A time integration procedure, the Newmark β method, was used for the analyses with considering both material and geometric nonlinearities. For the geometric nonlinearity, a constant geometric stiffness derived from initial axial forces was added to a linear elastic stiffness assuming that influences due to axial force fluctuations were small. Rayleigh damping was assumed in the analyses.

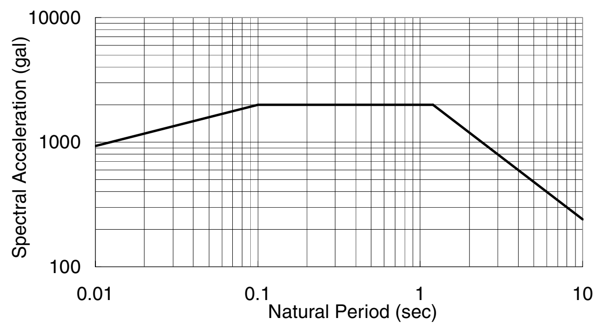
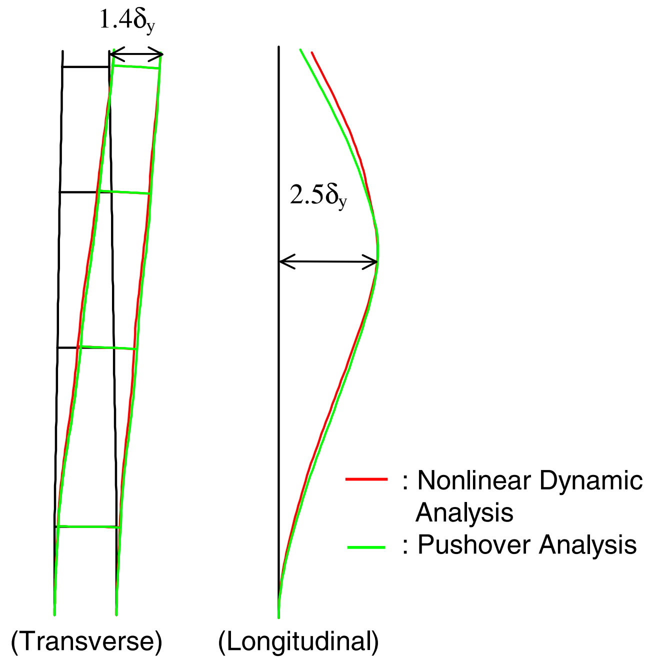


Figure 10: Acceleration Response Spectrum

Results of Analyses

As indicated in Figure 4, while the maximum δ was 1.78m ($1.4\delta_y$) in the transverse direction, the maximum δ was 3.42m ($2.5\delta_y$) and the tower deformed up to the considerable inelastic range in the longitudinal direction. Figure 11 shows the comparisons of deformed tower shapes at the maximum δ between the pushover analyses of the Fiber Model and nonlinear dynamic analyses. The figure demonstrates that the tower behaved in a similar manner in the both analytical methods. As shown in Figure 6, while the tower remained in elastic range in the transverse direction, the plastic region developed at the tower base and above and below the third beam, and the region spread with time in the longitudinal direction.

Figure 12 shows the comparisons of strain ratio at the maximum δ between the pushover analyses of the Fiber Model and nonlinear dynamic analyses. The legend symbol is given in the Figure 7 above. In the both directions, almost the same strains were generated between the both analytical methods, even though some strain values at the flanges on tension side had reverse sign (positive and negative) each other.



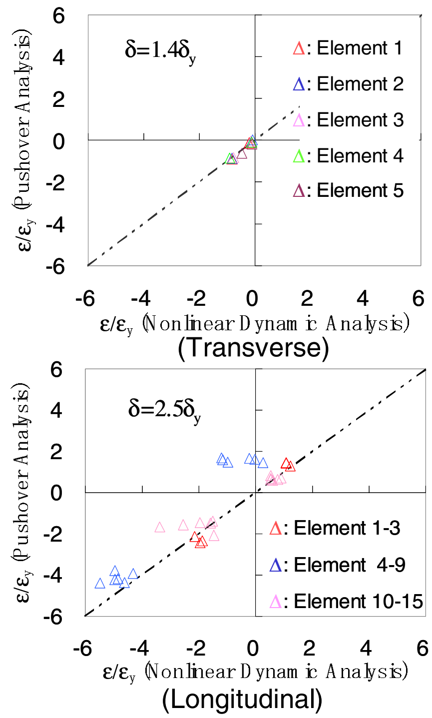


Figure 11: Comparisons of Deformed Tower Shapes **Figure 12: Comparisons of Strain Ratio**

Effects of Difference in Analytical Methods

As mentioned earlier, the comparisons in terms of deformed tower shape, progress in plastic region and strain ratio between the pushover analyses using the Fiber Model and nonlinear dynamic analyses have been done. Based on the comparisons, the effects of difference in analytical methods on strength and damage characteristics of the suspension bridge steel tower will be discussed here.

The both analytical methods generated almost the same results with regard to deformed tower shape, progress in plastic region and strain ratio in the both directions. Although the some strain values at the flanges on tension side had reverse sign (positive and negative) each other, the strain values on compression side, numerically larger than on tension side, should be focused for seismic evaluation. Accordingly, it seems reasonable to state that the effects of difference in analytical method were small within the scale of earthquake used for this study.

SEISMIC PEFORMANCE EVALUATION

One possible way for the seismic performance evaluation of a long-span suspension bridge during large-scale earthquakes is: 1) Identify the acceptable damage level at each structural element based on the nonlinear force-displacement relationship, the damage progress and the damage level obtained by pushover analyses using shell elements, which are able to simulate local buckling and shear damage; and then 2) Verify the resulting values obtained by nonlinear dynamic analyses using fiber elements within the acceptable level [4].

In the transverse direction, because the effects of difference in analytical model on nonlinear force-displacement relationship, the damage progress and the damage level during $0<\delta<1.5\delta_y$, as well as the effects of difference in analytical method, were small as descried earlier, it can be said that the suspension bridge tower analyzed suffers no critical damage against the seismic ground motion used for this study. In the longitudinal direction, because the analysis by the Fiber Model gave the results on the slightly safe side compared with the analysis by the Shell Model and the effects of difference in analytical method are small, it seems that the tower is within the ultimate state as well. Care, however, should be taken when evaluating seismic performance in the transverse direction for the unexpected earthquake, more than the ground motion used in this study, because the analysis by the Fiber Model may give the result on the danger side and the effects of difference in analytical method are not investigated yet in the range of $\delta > 1.4\delta_y$.

CONCLUSIONS

This research explored the nonlinear behavior and seismic performance evaluation of a long-span suspension bridge steel tower against large-scale earthquakes. After performing pushover analyses using two types of analytical model with shell and fiber elements, and nonlinear dynamic analyses, the following major conclusions were obtained:

(1) By the results of pushover analyses using the Shell Models, it was found that the stiffness of base shear versus horizontal displacement relationship began to decrease at $\delta=1.5\delta_y$ in the both directions and the base shear reached its peak at around $\delta=5.5\delta_y$ in the transverse direction and at around $\delta=2.7\delta_y$ in the longitudinal direction, respectively.
(2) Based on the comparisons of pushover analyses results between the Shell Model and the Fiber Model, the effects of difference in analytical model were discussed. In transverse direction, while the effects of difference in analytical model were small in the region of $0<\delta<1.5\delta_y$, the effects gradually became to be significant in the region of $\delta>1.5\delta_y$ due to shear damage of the beams. The seismic performance evaluation by analyses using the Fiber Model may give results on the danger side in the region of $\delta>1.5\delta_y$ because the analysis by the Fiber Model calculated higher strength of the structure than by the Shell Model. In longitudinal direction, although detailed aspects with regard to progress in plastic region and strain ratio between the both models were somewhat different, base shear versus horizontal displacement relationships and deformed tower shapes showed good agreement between the both. It is possible to say that the analyses using the Fiber Model give the results on the safe side because the strength of the structure calculated by the Fiber Model was slightly smaller than by the Shell Model.
(3) Based on the comparisons of results between pushover analyses and nonlinear dynamic analyses, the effects of difference in analytical method were discussed. The both analytical methods generated almost the same results with regard to deformed tower shape, progress in plastic region and strain ratio in the both directions. It seems reasonable to state that the effects of difference in analytical method are small within the scale of earthquake used for this study.
(4) It can be said that the suspension bridge tower analyzed suffers no critical damage for the seismic ground motion used for this study in the both directions. Care, however, should be taken when evaluating seismic performance in the transverse direction for the unexpected earthquake, because the analysis by the Fiber Model may give the result on the danger side and the effects of difference in analytical method are not investigated yet in the range of $\delta>1.4\delta_y$.

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