SEISMIC EFFECT OF MEMBERS FRACTURE ON TRUSS TOWER

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Summary

The ultimate strength of truss tower structures is governed by buckling of tubular members which easily leads to the member fracture. This paper proposes the post-fracture analysis methods for truss structures composed with tubular members with high diameter-to-thickness ratios, and a study on the collapse mechanism of such truss towers after buckling and fracture of members is investigated using incremental dynamic analysis (IDA) analysis. Truss towers without stack members exhibited collapse after buckling of main columns or fracture of diagonal members.

Keywords: Truss Tower, Collapse Analysis, Incremental Dynamic Analysis, Member Fracture, Cyclic Loading

1. Introduction

Although truss towers used for telecommunication, electric power transportation, supporting structures for stacks in power plants have been designed against wind loads, a higher level of safety is recently required because of the increase of anticipated large seismic inputs. The ultimate strength of these structures is governed by tubular member buckling which easily leads to member fracture [1]. This paper proposes the post-fracture analysis methods for truss structures composed with tubular members with high diameter-to-thickness ratios (Fig. 1), and a study on the collapse mechanism of such truss towers (Fig. 2) after the buckling and fracture of members is investigated, using IDA (Ref. [2]) analysis. The continuous column effect of the stack for the tower structures is also discussed.

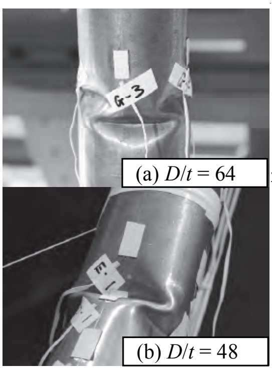
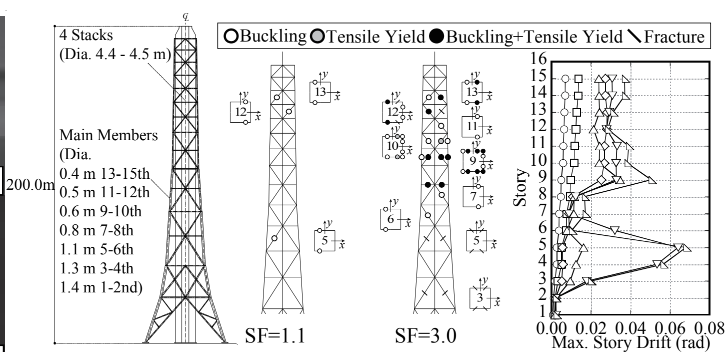


Fig1: Local Buckling



(a) Model (b) Result

Fig. 2: Truss Tower Model and Result

2. IDA for Tower Structure Integrating Modified Fracture Method

Integrating the modified fracture method to the time history analysis program, the IDA for a tower structure (Fig. 2) was carried out. The SF represents the scale factor of the ground motion. Fig. 2(b) shows the damaged CHS (Circular Hollow Section), and the maximum story drifts distribution. The diagonal CHS in 13th, 12th and 5th story underwent buckling in the SF = 1.1 firstly, and the diagonal CHS in 12th, 9th, 5th and 3rd suffered fracture in the SF = 3.0. In the SF = 3.0 the story drift in 9th and 5th story was significantly larger than the other stories due to the CHS fracture. However, the collapse of the tower was not confirmed when the CHS member underwent fracture up to 4.2 times of the elastic limit state.

Fig. 3 illustrates in the truss tower excluding the stacks, a large story drift in 12th story was confirmed due to the diagonal CHS fracture in the SF = 1.0, and the collapse occurred in the 9th story due to the main CHS yielding in the SF = 2.0.

Fig. 4 shows the IDA curve of the truss tower excluding stacks. The collapse was defined on the basis of the FEMA-350 [3]. When the direction of the ground motion is 0 deg., the collpase occured at the SF =0.9, on the other hand when the direction is 45 deg., the collapse occurred at the SF = 1.8. The story drift of the truss tower does not significantly increase over the elastic limit as the SF of the ground motion increased, which indicated the plastic damage does not necessarily lead to collapse of the whole structure. The collapse occurred approximately 1.5 times elastic limit.

3. Conclusions

1) The local plastic strain of tubular members over 40 *D*/*t* ratios moved to the edges of local buckling zones, which is well evaluated by the proposed strain amplification factors including the modification factors reflecting *D*/*t* value.
2) The main columns of stack truss towers exhibited the CHS fracture after buckling at 4.2 times elastic state; however not collapsed due to the continuous column effect of stack members.
3) Truss towers without stack members exhibited collapse after buckling of main columns or fracture of diagonal members. The collapse occurred approximately 1.5 times elastic limit.

4. References

[1] TAKEUCHI T., and MATSUI R., "Cumulative Cyclic Deformation Capacity of Circular Tubular Braces under Local Buckling", *ASCE Journal of Structural Engineering*, Vol. 137, No. 11, 2011, pp. 1311-1318.

[2] FEDERAL EMERGENCY MANAGEMENT AGENCY., "Recommended seismic design criteria for new steel moment-frame buildings", *FEMA-350*, 2000.6

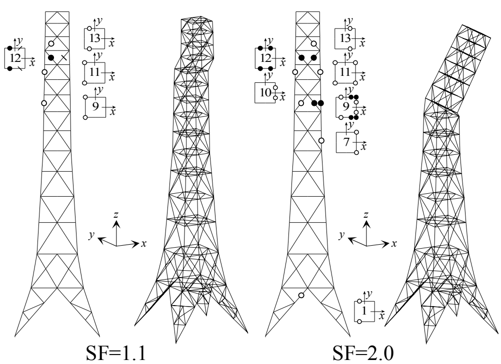


Fig. 3: Analysis Results

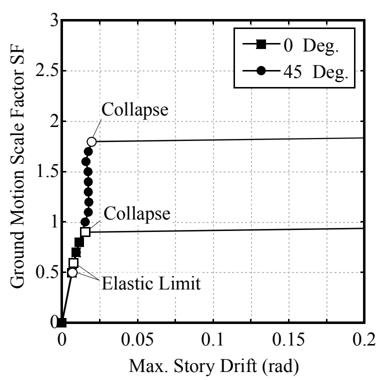


Fig. 4: IDA Curve