A Study on Large-Scale Simulation of a 20-Story Steel Building Structure

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The differences in seismic responses in highly nonlinear ranges among various analytical models are not clear. These include such as frame models, and detailed Finite Element Method (FEM) models microscopically reproducing the individual components' shapes are not clear structures. In this study, the dynamic response of a detailed FEM model is compared to those of frame models. The FEM model was made to reproduce a building's shape as precisely as possible and analyzed by LS-DYNA that is commercial FE code equipped explicit method. The frame model was macroscopically composed of beam elements. Two frame models were considered; in one model, the P-delta effect was considered, and in the other model, it was neglected. The differences between those frame models and the detailed FEM model in highly non-linear ranges was investigated. As a result, it was assured that the frame model that neglected the P-delta effect tend to underestimate the deflection in these ranges, while the residual deflection of the frame model that considered the P-delta effect was larger than that of the detailed FEM model because of stiffness degradation.

1. INTRODUCTION

In order to prevent urban disasters, it is very important to simulate the actual dynamic behavior of building structures during severe earthquakes. Buildings are usually considered as simple models such as multi-mass system models or frame models to design structural properties of buildings. However, these simple models may not be sufficient to evaluate the actual behavior of buildings against severe earthquakes in highly non-linear ranges because of the following two reasons. First, the elements such as beam elements or springs are employed in simple models are essentially assumed to express linear or weak non-linear responses.1)

Second, because they are modeled on the basis of many engineering judgments, each engineer may make a different analytical model. For example, evaluations of the stiffness of composite beams in steel frames are vary with different among each engineer or analysis software.2,3)

This difference is the result of simplifications of the structures' shape. On the other hand, full FEM models that reproduce the structure's shape as precisely as possible can resolve these differences. However, there are few studies that treat the full FEM models of buildings because of the high computing costs to analyze such models.4,5,6) Hence, differences existing in dynamic responses in highly non-linear ranges between simple and full FEM models are not clear. In this study, the responses of simple models are compared to those of the full FEM model.

The target of this analysis is a 20-story steel structure that can be modeled with the full FEM model with approximately ten million elements.
2. THE MODEL

The configurations of a sample 20-story steel structure of the target of analysis are shown in Fig.1. The structure is designed so that the maximum story angle is less than 1/200 under the \( Ai \) distribution. The floor load including self-weight of the steel members is set at 7.84kN/m\(^2\). The material of steel members is SN490, so the yield strength is 357MPa. The columns have box-section, beams have H-shape section and all members are designed to satisfy FA rank of building design grade.

![Framing Plan and Framing Elevation](image)

**Fig.1 Building to be analyzed (a) Framing Plan and (b) Framing Elevation**

The structure is evaluated using the following three numerical models: a frame model that neglects the P-delta effect, a frame model that considers the P-delta effect and a full FEM model.

The frame models are treated as pseudo-3D models as shown in Fig.2.

In the two frame models, the strength degradation of the steel members is not considered. This assumption is generally used in structural design methods in Japan. Each member is modeled by a beam element with rigid-plastic springs at both ends. The springs have bi-linear characteristics, and the bending strength is determined by the maximum capacity of plastic moment in which, lateral buckling of beams is not considered. Furthermore, the bending strength of columns is evaluated by interaction with the axial force.

![Frame Models (Pseudo 3-D Frame)](image)

**Fig.2 Frame Models (Pseudo 3-D Frame)**

The full FEM model is modeled to reproduce the structure’s shape. The outline of the FEM model is shown in Fig.3.

The FEM model is composed of shell elements for steel frame members and solid elements for concrete slabs to reproduce the structure’s shape as precisely as possible. This model uses 9,046,697 elements and 10,460,942 nodes. The FEM analysis was executed by the Earth Simulator 2 at the Japan Agency for Marine-Earth Science and Technology (JAMSTEC).

The shell elements are isoparametric elements having four nodes, one integration point in-plane and four integration points in the cross-sectional direction. The solid elements are isoparametric solid elements that have eight nodes and one integration point in the center of elements. Geometric non-linearity such as P-delta effect, buckling and each member’s strength degradation due to local buckling are considered in the FEM model.
The material models employed in the FEM model, such as bi-linear model, are similar to that of the frame models because this study focuses the effects of the preciseness of the shape reproduction for the structure's responses. For the steel material model, an isotropic elastic-plastic model to consider linear kinematic-isotropic mixed hardening is used\(^1\). Von Mises yield criterion expressed in equation (1) is adopted for the yield function.

\[
\phi = \frac{1}{2} \dot{\varepsilon}_y \dot{\varepsilon}_{ij} - \frac{1}{2} \sigma_y = 0 \tag{1}
\]

Where, \( \dot{\varepsilon}_y = s_y - \alpha_{ij} \)

\[
\alpha_{ij} = \frac{2}{3} (1 - \beta) \varepsilon_p \dot{\varepsilon}_0^p \tag{1.1}
\]

\[
\sigma_y = \sigma_0 + \beta \varepsilon_p \varepsilon_{eff}^p \tag{1.2}
\]

\[
E_p = \frac{E}{E - E_i} \tag{1.3}
\]

\[
e_{eff}^p = \left[ \frac{2}{3} \varepsilon_0^p \dot{\varepsilon}_0^p \right]^{1/2} \tag{1.5}
\]

Here, \( E \) is Young modulus, \( E_p \) is Strain hardening modulus, \( E_i \) is Tangent modulus, \( \varepsilon_{eff}^p \) is Equivalent plastic strain, \( \varepsilon_y \) is Plastic strain, \( s_y \) is Deviatoric stress, \( \alpha_0 \) is Back stress, \( \sigma_y \) is Current yielding stress, \( \sigma_0 \) is Initial yield stress.

Notice that the superscript dot indicates a time differential.

For the concrete material model, Ottosen's fracture criterion expressed equation (2) is adopted.\(^7\) In the tensile region, this model has a three directional orthogonal smeared crack condition. Stress relaxation in tension depends on the fracture energy which is defined by the stress-crack width relation.\(^89)\)
\[ A \frac{J_2}{(f_c')^2} + \lambda \sqrt{\frac{J_2}{f_c'}} + B \frac{I_1}{f_c'} = 0 \tag{2} \]

Where,
\[ \lambda = K_1 \cos \left( \frac{1}{3} \cos^{-1}(K_2 \cos 3\theta) \right), \quad (\cos 3\theta > 0) \tag{2.1} \]
\[ \lambda = K_1 \cos \left( \frac{\pi}{3} - \frac{1}{3} \cos^{-1}(-K_2 \cos 3\theta) \right), \quad (\cos 3\theta < 0) \tag{2.3} \]

Here, \( A, B, K_1 \) and \( K_2 \) are the factors determined by the ratio of tensile and compressive strength, \( f_c' \) is uniaxial compressive strength, \( J_2 \) is the second invariant of the deviatoric stress, and \( I_1 \) is the first invariant of the stress tensor. The angle \( \theta \) is often referred to as the Lode angle.

The time history and acceleration response spectrum of the excitation are shown in Fig.4. The excitation exceeds the Level-2 earthquake defined in Japanese code as very rarely occurring. In addition, we analyze a case in which the excitation is scaled to 1.5 times acceleration. In this study, the first case is called Case-1.0, and the second case is called Case-1.5.

In the frame models, Newmark-\( \beta \) method (\( \beta = 0.25 \)) is used as time integration method. The time interval is 0.001 second. In the full FEM model, dynamic explicit method based on central difference is used. The time interval is automatically determined based on Courant’s criterion.

![Fig.4 Excitation](image)

(a) Acceleration Response Spectrum and (b) Time History

3. ANALYSIS RESULTS

The execution time to analyze 9.4 seconds was approximately 77 hours by 64 cores of the Earth Simulator 2.

Maximum story drift responses are shown in Fig.5. In all cases, the structure did not collapse.

In Case-1.0, each model showed similar tendencies for the maximum story drift distribution. Deformation was maximized in the middle stories. In Case-1.5, the deformation in the middle stories of the frame model that neglects P-delta effect was smaller than those of the other two models because of consideration of the P-delta effect. On the other hand, deformation at the lower stories estimated by the FEM model was smaller than those of both frame models.

Time histories of story drifts are shown in Fig.6. Comparing FEM and Frame P-d, the results do not show a large difference until approximately 5 sec. However, the residual deformation of Frame P-d became larger than that of FEM after approximately 6 sec.

Story hystereses are shown in Fig.7. As shown in Fig.7 (i) and (ii), although the initial stiffness of the stories is almost same in the FEM and frame models, the story stiffness of FEM degraded after initial yielding. It seems that this caused the difference in story drift after 6 sec.
Fig. 5 Maximum Story Drift Distributions  
(a) Case-1.0 and (b) Case-1.5

Fig. 6 Time History of Story Drift  
(i) 10F, (ii) 7F, and (iii) 1F  
(a) Case-1.0 and (b) Case-1.5
Deformations in the 7th story diagrams with plastic strain contour are shown in Fig.8. Plastic area is concentrated in ends of beams and columns as shown in Fig.8 (i) and (ii). Furthermore, beam-to-column joint panel zones have been plasticized. The plasticity of the joint panel zone is not considered in frame models. In addition, as shown in Fig.8 (iii), local buckling of beam flange and shear buckling of the web occurred. The local buckling could not be automatically considered in frame models.

The phenomena which have been neglected in frame models are considered to be par of the reason for the difference of response among three models. The maximum principal strain distribution of a concrete slab is shown in Fig.9. This describes the crack pattern of a concrete slab. The crack in the slab occurred in a direction perpendicular to the beam. This damage in a concrete slab may be related to degradation of the story stiffness described in Fig.7. The degradation because of damage to a concrete slab could not be considered automatically too.
4. CONCLUSION

In this study, the analyses of a 20-story steel building structure using frame models and a detailed FEM model were executed and compared. The execution time for the detailed FEM model was approximately 77 hours for a 9.4 sec analysis by the Earth Simulator 2. None of the models collapsed against very severe earthquake motion as used in this study. When the story drift was less than approximately 150mm, each model indicated similar tendencies towards the maximum story drift. When the story drift reached approximately 200mm, the deformation of Frame was smaller
than other models by P-delta effect. Although the maximum story drift of Frame P-d and FEM were almost the same, residual deformation of Frame P-d was larger than that of FEM. This was caused by stiffness degradation of stories after initial yielding because of damage of the concrete slab. The simplifications of the structural shape have been attributed to dispersion of the responses in highly non-linear ranges.

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