Strongly Nonlinear Response Analyses on Steel Frame Structure with Large-scaled and Super-detailed Numerical Modeling

Project Representative
Yoichi Mukai — Department of Architecture, Graduate School of Engineering, Kobe University

Authors
Yoichi Mukai — Department of Architecture, Graduate School of Engineering, Kobe University
Yasunori Mizushima — Research and Development Institute, Takenaka Corporation
Masahito Ohno — Building Design Department, Osaka Main Office, Takenaka Corporation
Tomoharu Saruwatari — Engineering Technology Division, JSOL Corporation
Kenzo Taga — Department of Architecture, Graduate School of Engineering, Kobe University
Makoto Maruta — Department of Architecture and Production Design Engineering, Graduate School of Science and Engineering, Shimane University
Atsuo Takino — Department of Residential Environment, Faculty of Human Life and Environment, Nara Women's University
Yoshikazu Kanai — Department of Urban Environment and Information Sciences, Graduate School, Maebashi Institute of Technology

In general structural designing of building constructions, seismic responses are estimated by numerical design calculation by using simplified frame model or lumped-mass model. However, modeling parameters of those configuring elements are quantified under considering definite nonlinearity with partial specimens off-line experimental results, so that, it is difficult to estimate adequate and reliable seismic responses in strongly nonlinear ranges with those simplified analytical models. In the same way, difference between macroscopic modeling such as frame models used in the general structural design process and microscopically modeling such as detailed FEM models reproducing the individual components' shapes of structures are not elucidated. In this study, the dynamic response of a detailed FEM model is compared to that of a frame model. The FEM model is made to finely reproduce the building's shape as precisely as possible and the frame model was roughly composed of beam elements. Two cases in the frame models are evaluated whether the P-δ effect was considered or not. Difference between those frame models and the detailed FEM model in large nonlinear ranges is investigated. As a result, it is assured that the frame model of neglecting P-δ effect tend to underestimate the deflection in these ranges, while the residual deflection of the frame model considering P-δ effect is larger than that of the detailed FEM model because of stiffness degradation.

Keywords: Seismic response, FEM analysis, Strongly nonlinearity, Collapse mechanism, Super-detailed modeling

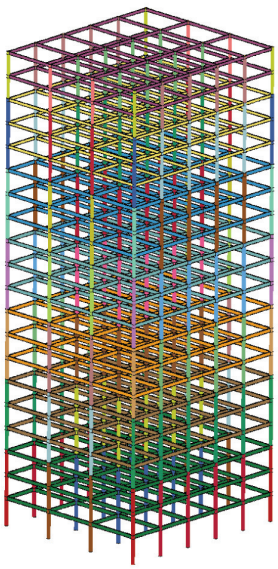
1. Introduction

In the building design for preventing large-scale urban disasters, to simulate the actual dynamic behavior of building structures during severe earthquakes is very important. In those cases, it is need to evaluate the actual behavior of buildings against severe earthquakes in highly non-linear ranges. General building design is operated by considering as simple models such as multi-mass system models or frame models to represent structural properties of buildings. However, these simple models may not be sufficient to evaluate severe earthquakes response of target structures because of the following two reasons. Firstly, the elements (beam elements or springs) employed in simple models are essentially assumed to express linear or weak non-linear responses [1]. Secondly, each designer (or researcher) may use a different analysis model since they are modeled on the basis of many engineering judgments. For example, evaluations of the stiffness of composite beams in steel frames are different among each engineer or analysis software [2],[3]. This difference is the result of simplifications of structures' shape. On the other hand, full FEM models that reproduce the structure's shape as precisely as possible can resolve these individual styled differences. Although 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]. And also,

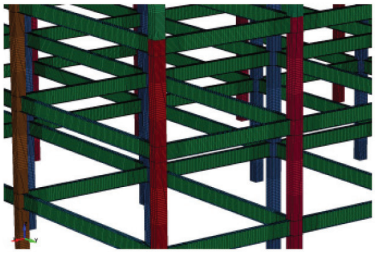
differences existing in dynamic responses in strongly nonlinear ranges between simple models and full FEM model are not clarified. In this study, the responses of simple models are compared to those of the full FEM model [7],[8],[9]. Target of the analysis is a 20-story steel structure that can be modeled with a full FEM model of approximately ten million elements as seen in Fig. 1.

2. Analysis modeling

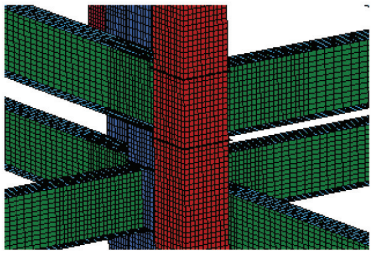
The structure is designed so that the maximum story angle is less than 1/200 under *Ai* distribution (confirming Japanese seismic design code). The material of steel members is SN490, so the yield strength is 357 MPa. The columns have box-section, beams have H-shape section and all members are designed to satisfy *FA* rank of structural frame design grade. This structure is evaluated using the following three kinds of numerical models: 1) a frame model neglecting the P-δ effect (Frame), 2) a frame model considering the P-δ effect (Frame P-d) and 3) a full FEM model (FEM). The frame models are treated as pseudo-3-D models. In the two frame models, the strength degradation of the steel members is not considered (this assumption is generally used in Japanese structural design methods). Each member is modeled by a beam element with a rigid-plastic spring 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 is evaluated by interaction with the axial force. On the other hand, the FEM model as shown in Fig.1 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 shell elements are iso-parametric elements having four nodes, one integration point in-plane and four integration points in the cross-sectional direction. The solid elements are iso-parametric solid elements that have eight nodes and one integration point in the center of elements. Geometric non-linearity such as P-δ 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 model 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



(a) Whole view

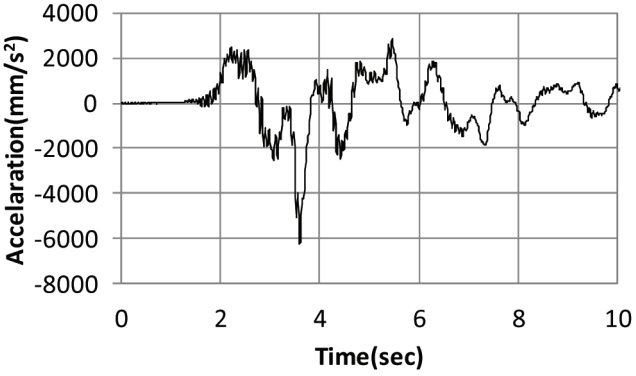


(b) Close-up view of framing

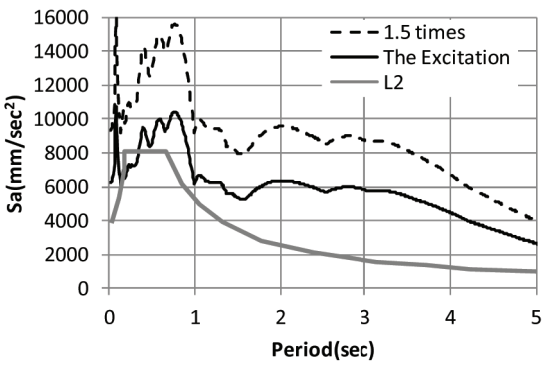


(c) Close-up view of connection of beam-column

Fig. 1 Configurations of the FEM model for the 20-story steel building.

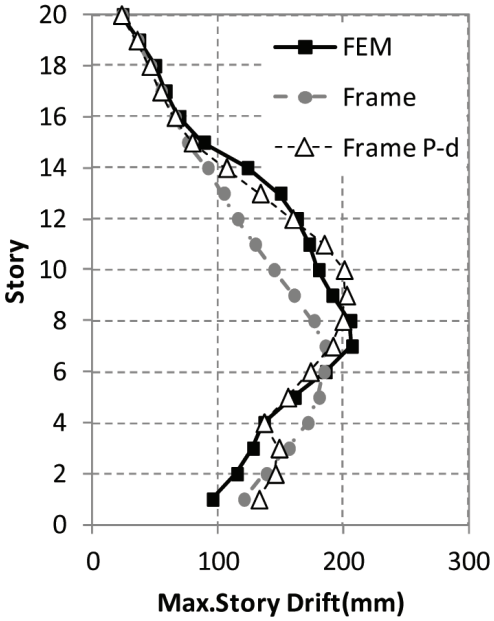


(a) Time history

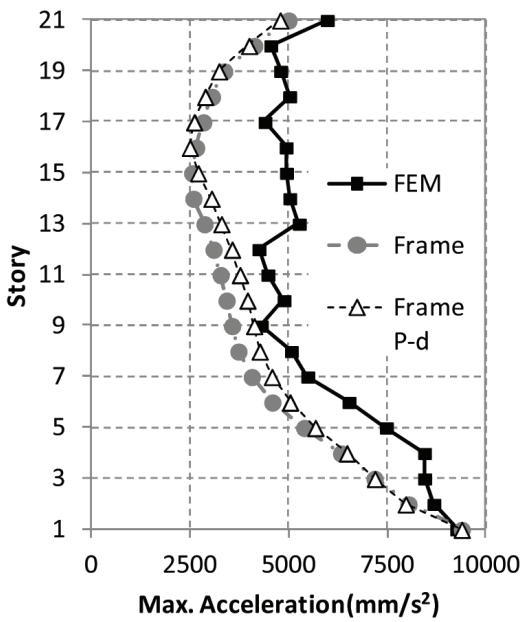


(b) Acceleration response spectrum

Fig. 2 Excitation of seismic response analyses.



(a) Story Drift Distributions



(b) Acceleration Distributions

Fig. 3 Maximum responses.

mixed hardening is used [1]. Ottosen's fracture criterion model is adopted for the concrete material model [10]. 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 [11].

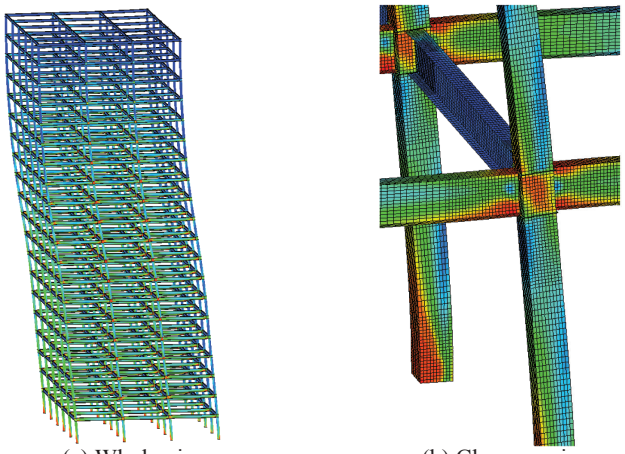
3. Analysis results and considerations

Seismic response analyses are carried out on those three numerical models. The time history and acceleration response spectrum of the excitation are shown in Fig. 2. The excitation exceeds the Level-2 earthquake defined in Japanese seismic design code as very rarely occurring. So that, we analyze a case in which the excitation is scaled to 1.5 times acceleration in this study. The execution time was approximately 77 h by 64 cores of the Earth Simulator 2. Maximum responses are shown in Fig. 3. In all models, the structure did not reach to collapse. The acceleration response of the FEM model was the largest among the three models. As one reason for this, it seems that local vibrations might have affected to that response.

The deformation in the middle stories of the frame model that neglects P-δ effect was smaller than that of the other two models because of consideration of the P-δ effect. On the other hand, deformation at the lower stories estimated by the FEM model was smaller than that of both frame models. In which, the plasticity of the joint panel zone is not considered in frame models. In addition, local buckling of beam flange and shear buckling of the web have been occurred in FEM. These phenomena could not be considered in frame models. These are considered to be part of the reasons of difference of response among three models. The Mises's equivalent stress distribution of the FEM model is shown in Fig. 4. In this model, the strength deterioration is not considered steel material model. However, the structure's shape is reproduced and geometrical non-linearity is considered because steel member's strength deterioration due to local buckling can be expressed. Thus, the full FEM model which reproduces the structure's shape may adequately express the collapse phenomenon without special effort.

4. Conclusion

Analyses of a 20-story steel building structure using frame models and a detailed FEM model were executed and compared. No models reached to collapse against very severe earthquake motion used in this study. When the story drift reached to approximately 200 mm, the deformation of frame model neglecting P-δ effect is smaller than other models considering P-δ effect. Although the maximum story drift of models considering P-δ effect are almost the same, residual deformation of the frame model considering P-δ was larger than that of FEM. The simplifications of structures' shape have been affected to dispersion of the evaluation of the responses in strongly non-linear ranges.



(a) Whole view (b) Close-up view

Fig. 4 Mises's equivalent stress distribution.

References

[1] T. Ine et al.,“Elasto-plastic Finite Element Analysis of Steel Square Pipe Column with Local Buckling under Cyclic Lateral Loading”, Summaries of Technical Papers of Annual Meeting of Architectural Institute of Japan B-1, pp.285-286, 2008 (in Japanese).

[2] SNAP ver.5 Technical Manual, Kozo System Co., Tokyo, Japan, 2009 (in Japanese).

[3] Design Recommendations for Composite Constructions, Architectural Institute of Japan, Tokyo, Japan, 2010 (in Japanese).

[4] T. Miyamura et al., “Comparison between Virtual Shaking Table Test of Super-Highrise Steel Frame Using E-Simulator and Beam Element Analysis”, Summaries of Technical Papers of Annual Meeting of Architectural Institute of Japan B-1, pp.291-292, 2010 (in Japanese).

[5] Y. Mizushima et al., “Large-Scale Simulation of the Dynamic Behavior of Building Structure”, Summaries of Technical Papers of Annual Meeting of Architectural Institute of Japan B-1, pp.401-402, 2009 (in Japanese).

[6] S. Takeda et al., “Application of Impact Analysis Code based on the Explicit Time Integration to the Shaking Table Tests of Full-Scale Six Story RC Building, Part 2”, Annals of Maebashi Institute of Technology, Vol.14 , pp.17-24, 2011 (in Japanese).

[7] Y. Mizushima, Y. Mukai, M. Ohno, and T. Saruwatari, “A Study on Strong Non-linearity Analysis with Large-scale and Detailed FEM Models -Comparison of Dynamic Responses of Frame and Lumped Mass Models-”, Proc. on International Symposium on Earthquake Engineering, JAEE, Vol.1, pp.517-524, 2012.

[8] Y. Mizushima, Y. Mukai, M. Ohno, and T. Saruwatari, “Large-Scaled Simulation of 20-Sotry Steel Building”, Proc. on the Nat. Cong. of Theoretical and Applied Mechanics 2013, pp. (GS01-05)1-2, 2013 (in Japanese).

[9] Y. Mizushima, Y. Mukai, M. Ohno, and T. Saruwatari, “A Study on Response to Severe Earthquake of Steel Structure by Large-Scaled Simulation”, Proceedings of Computational Engineering Conference of JSCES, Vol.18, pp. (G-9-5)1-3, 2013 (in Japanese).

[10] W. F. Chen, *Plasticity in Reinforced Concrete*, Maruzen Co. Ltd., Tokyo, Japan, 1985.

[11] B. J. Broadhouse, “The Winfrith Concrete Model in LS-DYNA3D”, Report: SPD/D(95)3, Structural Performance Dept., AEA Technology, Winfrith Technology Centre, U.K., 1995.

鋼構造建築構造骨組の大規模・超詳細モデリングによる強非線形応答解析

プロジェクト責任者
向井　洋一　　神戸大学　大学院工学研究科　建築学専攻

著者
向井　洋一　　神戸大学　大学院工学研究科　建築学専攻
水島　靖典　　株式会社　竹中工務店　技術研究所　先端技術研究部
大野　正人　　株式会社　竹中工務店　大阪本店　設計部
猿渡　智治　　株式会社　JSOL　エンジニアリング本部
多賀　謙蔵　　神戸大学　大学院工学研究科　建築学専攻
丸田　　誠　　島根大学　大学院総合理工学研究科　建築・生産設計工学領域
瀧野　敦夫　　奈良女子大学　生活環境学部　住環境学科
金井　喜一　　前橋工科大学　大学院工学研究科　環境・情報工学専攻

構造物の衝撃応答解析に効率的な陽解法による衝撃解析コードを用い､建築構造物の大規模･超詳細モデリングにより、巨大地震を想定した建築物の終局挙動解析を行うことが本研究の目標である。通常の建築構造設計段階ではフレームモデルや多質点系モデルといった主要部材単位のマクロモデルにより構成される系の地震時挙動評価を行う。しかしながら、これらのモデルでは、次のような形状の単純化に起因する問題点を有し、大規模な地震時挙動の正確な予測が困難であると考えられる。すなわち、①これらの単純化モデルに用いられる要素は、線形範囲から弱非線形範囲程度までの挙動を予測することを念頭に置いていること、②こうしたマクロモデルを構築するためには、その形状を単純化する過程で様々な工学的判断が介在すること、である。

一方で、形状の単純化を行わずに直接的に構造をモデル化する手法として FEM による解析が挙げられる。ただし、扱う要素数が膨大となるため、建築分野では従来 FEM 解析は主に部材実験の再現解析等に用いられることが主流であり、建物全体をモデル化し動的な解析がなされた例は数例しかない。本研究では、鉄骨造建築物を対象に、建物全体を FEM でモデル化した時と単純モデルを用いた時の強非線形域での応答の差異について明らかにしていくために、以下のような建物モデルについて応答の比較解析を行った。

(1) 鋼構造 4 階建て低層建築物について、FEM 解析モデルとマルチフレームモデル、集中質量系モデルによる応答解析結果との比較検討を行った。これらのモデルについて弾性応答域での各モデルの応答が整合するようにチューニングを行っても、降伏直後～大変形に至るにつれて、FEM モデルによる詳細解析では、脆弱層の変形進行が顕著となることが示された。なお、集中質量系モデルは進行性破壊を生じやすい一方で、その発生位置の特定などに関する結果の信頼性と解析の安定性に疑問がある。またフレームモデルでは、解析が安定する一方、脆弱層の変形進行を生じにくく評価する傾向が認められた。

(2) 鋼構造 20 階建て高層建築物のモデルについて、FEM 解析モデルと擬似 3D フレームモデルを用いた動的応答解析結果の比較を行った。フレームモデルについては、P-δ 効果を考慮した場合としない場合の比較を合わせて行った。いずれのモデルに関しても、一般の建物の構造設計段階で考慮される入力レベルを大きく超える地震動に対しても倒壊には至っておらず、最大加速度応答には大きな相違は見られなかった。一方、最大層間変形について、各モデル間で大きな差異が見られ、P-δ 効果を考慮したフレームモデルで、層間変形が最大となった。さらに、最下層では FEM モデルの方がフレームモデルと比較して最大せん断力応答が大きく生じた。これは、梁の耐力がスラブの合成効果により大きくなったことが一因と考えられる。

キーワード:地震応答解析, 有限要素解析, 強非線形領域, 倒壊メカニズム, 超詳細モデル