NONLINEAR STATIC AND DYNAMIC SEISMIC ANALYSIS OF A HYBRID HIGH-RISE STRUCTURE WITH EXPERIMENTAL VERIFICATION

Ming Cheng1 and Xilin Lu1

ABSTRACT

The prototype hybrid high-rise structure is special for its dual structural systems, so a 1/50 shaking table model was tested by the State Key Laboratory of Disaster Reduction in Civil Engineering of Tongji University to study its seismic behaviors. Then numerical models were made by 3D finite element analysis software, named MIDAS. The comparison of the analytical and experimental results verified the finite element models. In this paper, firstly, a higher structure of the same structure system was designed according to the prototype structure. Secondly, nonlinear static and dynamic analysis was carried out respectively with MIDAS to study the seismic behaviors of the higher structure. Finally, the results of the static and dynamic analysis were compared. It is found that the differences between them are acceptable and reasonable, and the nonlinear static pushover analysis and dynamic analysis are all suitable for the seismic study of this kind of structures.

1. INTRODUCTION

Under the rare occurring earthquake, structures could get into the inelastic stage because of the yielding of structure itself or some structural members. Therefore, it is reasonable and rational to do nonlinear analysis of structures, especially the tall buildings with complex structure systems, since there are additional inelastic properties for members that yield. Indeed, many nonlinear analyses of tall buildings have been carried out by researchers. Ventura and Ding used a nonlinear three-dimensional analysis to predict the state and sequence of damage of a 52-storey steel frame building in Los Angeles (Ventura and Ding, 2000). Yahyai et al. studied the seismic behavior of the 436m tall Milad Tower by a nonlinear dynamic analysis and three components of the earthquakes were considered based on three design levels (Yahyai et al. 2008).

There are many options to nonlinear analysis. One simplified nonlinear analysis is nonlinear static pushover analysis (NSPA) which assumes the dynamic performance of structures is mainly determined from the corresponding single-degree-of-freedom system. It is valid for low buildings that their first vibration mode dominates their behaviors, however its effectiveness is doubtful when apply to analysis structures whose higher mode effects are important. Recently, modal pushover analysis has been proposed to consider the higher mode effects (Pourshah et al. 2008). In addition, the powerful computers make possible the nonlinear dynamic analysis (NDA) which evaluates the time-history of structural inelastic response of a given acceleration record. It is feasible for tall buildings but costs more time and could be variation in the results if carried out for several ground motions (Powell, 2006). In this paper, the NSPA with capacity spectrum method and the NDA is applied to study the seismic behaviors of a complex 42-story-high building with dual structural systems and the results of the two methods are compared to verify if they are suitable to study this kind of structures.

1 State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University; Shanghai, China
2. DESCRIPTION OF THE STRUCTURE AND THE EXPERIMENTAL VERIFICATION

The prototype hybrid building is 36-story-high and special for its complex structural systems: the main one is a mega-frame consisted of mega-columns, strengthened floors and the outrigger truss on the top; the secondary one is consisted of the rest typical floors between the strengthened floors. The structural plans of the standard floors and the strengthened floor are showed in Fig.1 (Cheng and Lu, 2009). The total height of this building is 118.8m and the strengthened floors are located at floor 10, 19 and 28. The specialities of the structure include: the 12 triangle and lattice mega-columns at every corner; the 3 three-story-high outrigger steel truss on the top rigid-connected to the structure and the rest connections of the columns and beams are all hinged. In order to study the seismic behaviours of the structure, a 1/50 shaking table model was tested by the State Key Laboratory of Disaster Reduction in Civil Engineering of Tongji University and a 3D finite element modeling analysis were carried out by MIDAS.

![Fig. 1. Structural plans (Left is the standard floor and right is the strengthened floor)](image_url)

In the shaking table test, four types of earthquake records were used as the inputs to the shaking table, which are El Centro, Taft, EC8 (a semi-artificial record from European seismic design code) and SHW2 (an artificial accelerogram from Shanghai seismic design code). Since the test model was made by plexiglass, it only simulated the elastic responses of the structure, such as frequencies of the free vibration, displacement response under frequently and basically occurring earthquakes. It was showed that the results of the numerical analytical model and test model went well with each other and the differences were acceptable, which verified the numerical models and also proved the valid and applicable of this structural system.

In this paper, the height of the prototype structure was changed from 118.8m to 138.6m by adding 6 standard floors, and the width of the plan is 62.4m. Therefore the aspect ratio of the higher structure is 2.22 and its total floor area is approximately 73682m². Moreover, the higher structure has 42 stories and all the inter-story heights are 3.3m. The strengthened floors of the higher structure are located at floor 16, 25 and 34. Except the additional 19.8m height, the structural system of the higher structure is almost the same with the prototype one. The elevation plans of the prototype structure and higher structure are showed in Fig.2. Both NSPA and NDA were conducted to study the seismic behaviours of the higher structure in this paper and the
results were analyzed to find out whether these methods are suitable for the nonlinear analysis of this kind of structural system.

![Elevation plans](image)

Fig. 2. Elevation plans (Left is the prototype structure and right is the higher structure)

3. NUMERICAL MODEL AND PARAMETERS FOR NONLINEAR ANALYSIS

3.1 Numerical Analysis Model

The numerical finite element model of the higher structure, as showed in Fig.3, was built by the computer program called MIDAS. There are 11093 nodes, 20906 beam elements and 2374 plate elements of this model and its total mass (including the mass of the materials and loads) is 57401t. The first ten natural frequencies of the numerical model were obtained by modal analysis and the results are presented in Table 1. Specially, the damping ratio of the structure is 2% during elastic response spectrum analysis but it changes during nonlinear analysis: for determining the demand spectrum in NSPA, the damping ratio was estimated to be 4% considering that the plastic development of the structure could consume more energy and add its actual damping; for the direct integration in NDA, the Rayleigh damping matrix was applied which was calculated according to the mass and rigidity matrix. The equation of damping matrix is showed below and the parameter $\alpha$ and $\beta$ are 0.0511 and 0.0313 respectively of the numerical model.

$$[C] = \alpha [M] + \beta [K]$$

3.2 Definition of the Plastic Hinges

Elements with concentrated plastic hinge were adopted for the numerical model and the general force (moment or axial force)-general displacement (axial deformation or rotation) curve for plastic hinges is showed in Fig.4, which indicates that the stiffness after yield is 3% of the initial stiffness. Referring to the stress property of the structure, two kinds of plastic hinges were adopted during the nonlinear analysis: P-M hinge and M-M hinge. The M-M hinge is for the beam element which is mainly subjected to bending moment and only considers the interaction of the moments in two axes; while the P-M hinge is for elements subjected to both axial force
and bending moment. The flexural yield strength of this hinge is calculated considering the axial force effect. However, the interaction of bending moments in two axies is ignored by the P-M hinge for it assumes the axial force independently interacts with each moment in determining the hinge status.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Modes of Vibration</th>
<th>Period(s)</th>
<th>Frequency (Hz)</th>
<th>Mode</th>
<th>Modes of Vibration</th>
<th>Period(s)</th>
<th>Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Translation in Y</td>
<td>4.9193</td>
<td>0.2033</td>
<td>6</td>
<td>Torsion</td>
<td>0.8336</td>
<td>1.1996</td>
</tr>
<tr>
<td>2</td>
<td>Translation in X</td>
<td>4.9161</td>
<td>0.2034</td>
<td>7</td>
<td>Translation in Y</td>
<td>0.7117</td>
<td>1.4051</td>
</tr>
<tr>
<td>3</td>
<td>Torsion</td>
<td>2.6261</td>
<td>0.3808</td>
<td>8</td>
<td>Translation in X</td>
<td>0.7114</td>
<td>1.4057</td>
</tr>
<tr>
<td>4</td>
<td>Translation in Y</td>
<td>1.3919</td>
<td>0.7184</td>
<td>9</td>
<td>Translation in Y</td>
<td>0.4684</td>
<td>2.1349</td>
</tr>
<tr>
<td>5</td>
<td>Translation in X</td>
<td>1.3916</td>
<td>0.7186</td>
<td>10</td>
<td>Translation in X</td>
<td>0.4682</td>
<td>2.1358</td>
</tr>
</tbody>
</table>

In order to reflect the yield or damage mechanic of the structure, the M-M hinge was assigned on both ends of the main beams and the secondary beams; the P-M plastic hinge was assigned on columns at each floor level in NSPA and NDA. No plastic hinge was located on the top steel truss because the rigidity of the truss was large so it was assumed do not yield before the columns and beams. The total number of plastic hinges was 14076 and the assignments of plastic hinges were same in NSPA and NDA in order to compare their analysis results.

3.3 Nonlinear Static Pushover Analysis

Pushover analysis is a simple method to investigate the ultimate strength and deformation capacity of the structure after yielding and becomes a representative analysis method for Performance-Based Seismic Design. Since the results of pushover analysis are sensitive to the choice of load pattern, particularly for the high-rise buildings, two load patterns were used in this paper. The first pattern (NSPA-1) assumed the lateral force distribution as an invert triangle which was in accordance with the first mode shape of the structure; and the other pattern (NSPA-2), called uniform pattern, was based on lateral forces that are proportional to the total mass with constant acceleration response at each floor level. The two patterns of lateral force distribution are showed as Fig.5. Furthermore, the displacement control method was used and the same target roof displacement (2.77m, 1/50 of the structure height) was used for both NSPA-1 and NSPA-2. In addition, the lateral force pattern was only applied in the Y direction because of the symmetry of the structure.

Capacity spectrum method was chosen for NSPA in this paper. It uses the intersection of the capacity (pushover) curve and a reduced demand spectrum, which was determined according to Shanghai Seismic Code in this paper, to estimate the maximum displacement of the structure. The intersection is also called performance point. (ATC-40, 1996) During the analysis, the P-delta effect was considered to make the analysis results more accurate.

3.4 Nonlinear Dynamic Analysis

Nonlinear time history analysis was also adopted in this paper. Specially, the MIDAS programs use Newton-Raphson iteration method for nonlinear elements to get convergence and Newmark method for the method of direct integration during the NDA. 5 artificial earthquake records were
made corresponding to the demand spectrum in the NSPA. Also because of the symmetry, the 5 accelerograms were respectively applied to the numerical structure model only in Y direction.

Fig. 3. Numerical finite element model

Fig. 4. Force-displacement curve of plastic hinges

Fig. 5. Lateral force patterns (Left is for the NSPA-1 and Right is for the NSPA-2)

4. RESULTS AND ANALYSIS

4.1 The Pushover Curves and the Performance Points of the Numerical Model

The base shear-roof displacement relationship curves (pushover curves) of NSPA-1 and NSPA-2 in the Y direction are showed in Fig.6. It shows that the base shear of the uniform pattern is obviously larger than that of the invert triangle pattern. Moreover, the pushover curve indicates that the stiffness of structure significantly reduced during NSPA-2, which means the structure begins to yield. In addition, the performance points of the numerical model in NSPA-1 and NSPA-2 are determined by the capacity and demand spectrums as showed in Fig.7. It shows that the roof displacement of the NSPA-1 is larger than that of the NSPA-2 but the base shear is smaller than that of the NSPA-2 at their performance point respectively.
4.2 Propagation of the Plastic Hinges in NSPA

There are 3.9% of the plastic hinges in E stage and 8.1% in CP stage (the stages from A to E is according to the Fig.4) at the performance point in NSPA-1. The structure does not get into the yield stage then, which fact can also be concluded from the Fig.6. While in NSPA-2, there are 7.8% of the plastic hinges in E stage and 4.6% in CP stage at the performance point. It suggests that the whole structure gets into the yield stage, as showed in Fig.6. The propagations of the plastic hinges in both NSPA-1 and NSPA-2 were almost the same: they first appeared at the strengthened floor, next at the base of columns and then went along the height of the structure at the columns and beams.

![Fig. 6. Base shear-roof displacement curves (Left is for NSPA-1 and right is for NSPA-2)](image)

![Fig. 7. Capacity and demand spectrums of NSPA-1 and NSPA-2](image)

4.3 Displacement and Shear Force Responses of the Numerical Model

The distributions of the inter-story shear force and the inter-story drift along the structure are showed in Fig 8-9; the results of displacement and shear responses are also listed in Table 2. There are some analysis and conclusions:

(1) The inter-story shear distribution of the uniform lateral force pattern (NSPA-2) is almost triangular and its base shear is larger than that of the NSPA-1. While, the inter-story shear distribution of the NSPA-1 changes at three floors where the strengthened floors located because the rigidities of the floors are larger than the standard floors.
(2) The inter-story shear distribution of the NDA-average (the average result of the 5 NDA) is nearly smooth except a little change at the first strengthened and larger than the NSPA along the height. It dictates that the rigidity of the structure is generally uniform but the first strengthened floor maybe a weak part.

![Fig. 8. Distributions of inter-story shear force](image1)

![Fig. 9. Distributions of the inter-story drift](image2)

Table 2. Analysis results of the NSPA and NDA

<table>
<thead>
<tr>
<th></th>
<th>Max displacement (mm)</th>
<th>Max inter-story drift (mm)</th>
<th>Max inter-story drift angle</th>
<th>Max base shear (KN)</th>
<th>Min shear/weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>NDA1</td>
<td>963</td>
<td>33.1</td>
<td>1/100</td>
<td>65038</td>
<td>0.1149</td>
</tr>
<tr>
<td>NDA2</td>
<td>1002</td>
<td>32</td>
<td>1/103</td>
<td>56606</td>
<td>0.1005</td>
</tr>
<tr>
<td>NDA3</td>
<td>1211</td>
<td>39.3</td>
<td>1/84</td>
<td>65338</td>
<td>0.1155</td>
</tr>
<tr>
<td>NDA4</td>
<td>1097</td>
<td>36.3</td>
<td>1/91</td>
<td>63437</td>
<td>0.1121</td>
</tr>
<tr>
<td>NDA5</td>
<td>1118</td>
<td>37.3</td>
<td>1/88</td>
<td>56336</td>
<td>0.0996</td>
</tr>
<tr>
<td>NDA-average</td>
<td>1078.2</td>
<td>35.6</td>
<td>1/93</td>
<td>61351</td>
<td>0.1084</td>
</tr>
<tr>
<td>NSPA-1 (at the performance point)</td>
<td>1018</td>
<td>33.18</td>
<td>1/99</td>
<td>48330</td>
<td>0.0854</td>
</tr>
<tr>
<td>NSPA-2 (at the performance point)</td>
<td>862</td>
<td>28.04</td>
<td>1/118</td>
<td>56180</td>
<td>0.0993</td>
</tr>
<tr>
<td>Difference between NSPA-1 and NSPA-2</td>
<td>15%</td>
<td>15%</td>
<td>15%</td>
<td>14%</td>
<td>14%</td>
</tr>
<tr>
<td>Difference between NDA-ave and NSPA-1</td>
<td>5.60%</td>
<td>6.80%</td>
<td>6.80%</td>
<td>21%</td>
<td>21%</td>
</tr>
<tr>
<td>Difference between NDA-ave and NSPA-2</td>
<td>20%</td>
<td>21%</td>
<td>21%</td>
<td>8.40%</td>
<td>8.40%</td>
</tr>
</tbody>
</table>

(3) The inter-story drift distributions of the NSPA-1, NSPA-2 and NDA-average are all arc shape in general; however, there are little changes at strengthened floors of the NSPA-1 and NDA-average. Specially, the change at the first strengthened floor is largest because many plastic hinges appear below or on it. It suggests that the first strengthened floor could be the weak part of the structure and need to be paid more attention during design.

(4) The Maximum inter-story drift of the structure is 1/88 among NSPA and NDA, which indicates that the structural system is reasonable and valid under severe ground motions.

(5) As the Table1 showed, the maximum base shear of the NSPA-2 is more close to the NDA-average; however the displacements of the NSPA-1 and the NDA-average have less difference. According to the Fig8-9, it is clear that the NSPA-1 makes a nearly perfect match
with the NDA-average in the inter-story drift analysis. Although both the NSPA-1 and NSPA-2 underestimate the shear force compared with the NDA-average, the differences between them are acceptable.

(6) In addition, the results of the NSPA-1 and NSPA-2 are quite different from each other because of the different lateral force patterns. An improved lateral pattern could be investigated to make the NSPA better match the response from the NDA.

5. CONCLUSION

It is found that the higher structure shows good displacement performance during nonlinear analysis; furthermore, the nonlinear static pushover analysis and dynamic analysis are all suitable for the seismic study of this kind of structure system because the differences between their results are acceptable and reasonable. However, in order to make better recognition of this structure’s seismic behaviors, there is still a lot of research work to do. As the Krawinkler said, observation of behavior during the nonlinear analysis is more important than a rigorous quantitative evaluation based on maximum force or deformation values. To understand the seismic behavior of the structure under severe earthquakes more accurately is the main goal of the nonlinear analysis (Krawinkler, 2006).

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