RESPONSE AND SENSITIVITY ANALYSIS OF THE FRAME-CORE-TUBE SUPER HIGH-RISE STRUCTURE BASED ON THE IMPROVED LAYERED SHELL MODEL

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ABSTRACT

The frame-core-tube structure occupies a large proportion of modern super high-rise buildings because of its high space utilization and flexible structure feature, and the numerical method is more popularly employed to study the mechanism of the structure under the strong earthquake disaster. As the main anti-lateral force component of the frame-core-tube structure, the numerical model of the shear wall has a great impact on the response of the super high-rise structure under earthquake. The improved layered shell model is applied to conduct the elastic-plastic time history analysis of the super high-rise structure under earthquake, and the results calculated are compared with the OpenSees program and SAP2000 software. The influence of different layered shell models on the global and local response of a height of 258m frame-core-tube super high-rise building is studied. It’s found that material model has great effect on the two aspects, and shell element has little influence on overall structural responses but in-negligible impact on the shear walls. The coefficient formula of bending and shearing stiffness is applied to describe the change of structural performance quantitatively, and the sensitivity of the frame column, the coupling beam, the shear wall and the strengthened story to the structural damage is studied. From the results, it is found that frame columns, coupling beams and strengthened stories are more sensitive to the development of the global structural damage. And when the structural stiffness coefficient of components adjusted is smaller than the initial model, the evolution of the structural damage is slow. Otherwise the evolution of the structural damage is rapid.

Keywords: frame core-tube structure; earthquake; sensitivity analysis; multiple-layered shell element

1. INTRODUCTION

The frame-core-tube structure occupies a large proportion of modern super high-rise buildings because of its high space utilization and flexible structure feature. However, due to complex shape and various types of elements, these structures under seismic action have more complicated dynamic characteristics. Therefore, it’s necessary to employ the numerical means to study the mechanism of the structure under the strong earthquake disaster. The software for elastic-plastic time history analysis can be divided into commercial and open source software. Generally, commercial software has great advantages in pre-post processing and user experience, and its kernel uses common, stable algorithms and elements. But program scalability has many limitations, e.g. many off the shelf software can only be redeveloped on the interfaces provided. Focusing on the development of algorithms, elements and materials, open source software integrates many of the latest scientific researches. However, its development is restricted by the poor usability and the openness of some core codes. In this paper, the DUT proposed by Fu et al. (2015), a self-developed elastic-plastic analysis platform, is used to conduct the case study. The advantage, high computational efficiency of the platform, makes it suitable for elastic-plastic analysis of super high-rise structures. The accuracy of the numerical

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simulation results depends on the reasonable selection of each component model of the structure, and the research on the beam-column element is relatively mature, but the shear wall model is not yet perfect. The shear wall is the main anti-lateral component of super high-rise structure and its mechanical behavior is complicated under earthquake. Therefore, the shear wall model has a great influence on the calculation of nonlinear analysis of the super high-rise structure. At present, the layered shell model is widely used in the simulation of shear walls and its calculation precision depends on the reasonable shell model and constitutive model of concrete material. For shell elements, based on the MITC4 four-node shell element proposed by Dvorkin and Bathe (1984) in OpenSees, a layered shell model is developed by Lu et al. (2015). The feasibility of this shell model is verified through the elastic-plastic analysis of shear wall members and the super-tall buildings. However, the MITC4 shell element has poor simulation ability for large deformation and damage of shear wall under strong earthquake, and it has the problem of shear locking. Based on the theory of flat-panel shell, Wang et al. (2016) proposed a new type of quadrilateral flat shell element DKGQ. And combined with updated Lagrangian formula proposed by Belytschko et al. (2013), the shell element NLDKGQ suitable for geometrically nonlinearity is further proposed. The shell model has a good calculation accuracy and can effectively avoid the problem of shear locking. Because of using the quadrilateral thin plate element DKQ proposed by Batoz and Tahar (1982), DKGQ and NLDKGQ has quite high accuracy in simulating thin wall bending, but poor simulation ability for out of plane performance of shear wall with large thickness to span ratio. For the constitute of concrete material, the most representative two-dimensional materials are the SMM model proposed by Hsu et al. (2002), the MCFT model proposed by Vecchio et al. (1986) and the PlaneStressUserMaterial model proposed by Lu et al. (2015) which are based on damage mechanics and smeared crack model and integrated into OpenSees. However, the combination of SMM model and MCFT model with layered shell elements is not quite perfect, and the SMM and MCFT model may have the problem of numerical stability in the calculation process of shear wall in super high-rise buildings. The core codes of PlaneStressUserMaterial model have not been public. Therefore, by improving and refining the existing layered shell model at the level of element and material, an improved layered shell model developed by He and Sun (2017) is integrated into the DUT program. The rationality and reliability of the improved layered shell model have been verified through the numerical simulation of the slab members, shear wall members and frame shear structures. In recent years, the elastic-plastic time-history analysis of super high-rise buildings is carried out by many researchers. Lu et al. (2011) established the finite element model of Shanghai Tower, based on the MSC.MARC analysis platform. Modeling methods and failure criteria for huge-columns and other various types of components are discussed, the basic dynamic characteristics of the structure are analyzed, and the failure mode and collapse process of the structure under earthquake are predicted. The ABAQUS and VEPTA programs are applied by Huang et al. (2011) to conduct the elastic-plastic time-history analysis of Ping An International Finance Centre, in order to test the seismic performance of the structure under large earthquake. In the analysis, the elastic-plastic behavior of beams, columns and shear walls is simulated accurately at the stress-strain level, the geometric nonlinearity of the structure is considered and the explicit integration algorithm is adopted.

However, there are not many studies about the influence of various components on the seismic performance of super high-rise structures. In this paper, based on the DUT program and the improved nonlinear shear wall model, the elastic-plastic time-history analysis of a height of 258m frame-core-tube super high-rise building is analyzed. In the analysis, the influence of different types of shear wall elements on the dynamic responses of the super high-rise building is studied, then, in view of the 258m structure, the damage sensitivity of components with different types to the frame-core-tube super high-rise building is researched.

2. THE DUT ANALYSIS PLATFORM

2.1 The architecture of DUT program

The DUT analysis platform is a finite element program specially designed for dynamic nonlinear time-history analysis of large-scale building structures, aiming at efficiently simulating the earthquake catastrophic behavior of large-scale structures, which could perform the high accuracy and efficiency
in the real computing process. The computational accuracy is ensured by considering the geometrical
and material nonlinearity of the fiber beam column element and the layered shell element. The
computational efficiency is ensured by element parallel processing strategy (PSTP), CPU/GPU
parallel matrix solver and accelerated iterative algorithm INC and so on. The complex system and
diversity of the super high-rise buildings leads to the problem for the analysis program including
multi-type variable and large storage, which could be solved through the object-oriented language
with the character of the easy to pack and further development. Therefore, the object-oriented language
C++ is used to develop the DUT program. According to the finite element analysis process, DUT
abstracts out the element classes, system classes, analysis classes, node classes, load classes and so on.
The basic architecture of DUT program is described in Figure1.

![Diagram](attachment://image.png)

Figure 1. The basic architecture of DUT program

2.2 The layered shell model

The accurate simulation of mechanical properties of shear walls is the key for the numerical analysis
of frame-core-tube structures. A reasonable layered shell model is an important way to understand the
mechanism of internal force redistribution in one component (such as coupling beams and limbs) and
among different components, and it is a necessary condition to accurately capture the global structure
damage and local damage localization. The improved layered shell element based on TMQ plate element, enhances the calculation accuracy and efficiency of the shell element out of plane. The element is named ShellGL and ShellGNL in the DUT program, corresponding to the shell DKGQ and the geometric nonlinear shell NLDKGQ. Using the improved layered shell element, four pieces of the most representative shear wall components are simulated, including ordinary one glyph shear wall SW1-1, short-limb shear wall SW2-1 analyzed by Zhang (2007), shear wall with edge member of column SW2 analyzed by Gong et al. (2006) and coupled shear wall CW-3 analyzed by Chen and Lv (2003). The comparison between the experimental and simulated hysteresis curves is shown in Figure 2. It can be seen from the figure that the simulation results of ShellGNL are consistent with the experimental results, and the calculation results are reliable, which can be used for the analysis of the frame-core-tube super high-rise building. With the Los Angeles wall-mounted architecture as a prototype, the University of California, San
Diego and the Portland Cement Association jointly designed a seven-story reinforced concrete shear
wall structure, and the shake table testing on the structure is conducted by Waugh and SriSritharan
(2010). The improved layered shell model is applied to carry out the time history analysis of the
structure under the seismic wave EQ1, and the results are compared with the test results shown in
Figure 3. Because the numerical model is calculated under the ideal state differing from the actual
structure, so there is a certain deviation in the partial displacement response. But the phase points of
the response are basically the same, which proves that the model can describe the structural
mechanical behavior accurately.

![Graphs of Load-displacement curves for various types of shear wall components](image1)

(c) shear wall with edge member of column  
(d) coupled shear wall

Figure 2. Load-displacement curves of various types of shear wall components

![Graph of the comparison of vertex displacement time history curves](image2)

Figure 3. The comparison of vertex displacement time history curves

3. THE EFFECT OF LAYERED SHELL ON THE RESPONSE OF THE SUPER HIGH-RISE STRUCTURE

This section will study the influence of different shear wall models on the global and local responses of the super high-rise structure. The dynamic nonlinear time-history analysis of a height of 258m frame-core-tube structure under earthquake is conducted by DUT, OpenSees and SAP2000 respectively.

3.1 The introduction of a 258m frame-core-tube structure

The structure with the height of 258m has 61 stories and an oval shape with a long side width of 69.4m and a short side width of 41.7m, its ratio of height to width is 6.2. The building model is shown in Figure 4. The structure consists of the core tube, the outer frame and the strengthened story with outrigger trusses and girdle trusses. For the core tube, the bottom 10 stories and strengthened stories
adopt the steel reinforced concrete shear wall, and the rest of stories adopt ordinary reinforced concrete shear wall, in which the thickness of the wall gradually decreases from 800mm to 300mm from the bottom to the top. For the outer frame, the frame column is made of concrete-filled steel tubular. The section size decreases along the height of the structure with the maximum diameter of 1.6m, the minimum diameter of 0.8m and the steel ratio is 5%-10%. The floor system is composed of section steel beam and reinforced concrete floor, in which both ends of the steel beam connecting core tube and huge-columns are hinged, and the rest are rigid and pinned. The strengthened floors are set in the 16th floor, 31st floor and 46th floor, using the herringbone strutting composed of square steel tubes.

![Figure 4. The building model of S1 structure](image)

### 3.2 Applicability comparison of different layered shell models

The analysis is conducted by DUT, OpenSees and SAP2000 respectively, and the layered shell models of shear wall used in the DUT and OpenSees program are shown in Table 1.

<table>
<thead>
<tr>
<th>Program</th>
<th>Name</th>
<th>Type</th>
<th>Material type</th>
</tr>
</thead>
<tbody>
<tr>
<td>DUT</td>
<td>DUT_SM</td>
<td>ShellGNL</td>
<td>MCFT2S</td>
</tr>
<tr>
<td></td>
<td>DUT_NM</td>
<td>NLDKGQ</td>
<td>MCFT2S</td>
</tr>
<tr>
<td></td>
<td>DUT_MM</td>
<td>MITC4</td>
<td>MCFT2S</td>
</tr>
<tr>
<td>OpenSees</td>
<td>OpenSees_NP</td>
<td>NLDKGQ</td>
<td>PlaneStressUserMaterial</td>
</tr>
<tr>
<td></td>
<td>OpenSees_MP</td>
<td>MITC4</td>
<td>PlaneStressUserMaterial</td>
</tr>
</tbody>
</table>

Static analysis of structure under gravity is done firstly, and then the dynamic analysis under seismic wave RH2TG055 which could implement the acceleration in X and Y direction at the same time. The PGA in main direction is set to 35gal and 400gal respectively, and the peak scale factor of Y:X is 1:0.85. The curves of maximum story drift ratio and vertex time history displacement are shown in Figure 5 and Figure 6.

It can be drawn from the comparison results that there is a large deviation for the results of the OpenSees_NP model probably caused by data transmission problem in OpenSees platform, which will not be discussed here. For other shell models, when the PGA is 35gal, the calculated results are close because the structure is still in elastic state. The location of the maximum story drift ratio appears at 57th floor and the maximum difference is only 9.1%. When the PGA comes to 400gal, the structure is in the elastic-plastic state, and the distribution of structural response calculated by each model is
basically the same. However, there is a difference in the peak point value and the floor position. The structural response calculated by SAP2000 is relatively largest and the response calculated by OpenSees_MP is relatively minimal. Using the same material model, the difference between DUT_SM considering geometric large deformation and elastic shell model DUT_MM is about 4.9%. With the same type of shell element and different type of material, the difference between DUT_MM and OpenSees_MP is about 11.2%. When the type of shell element and material are both different, the difference between DUT_SM and OpenSees_MP is about 16.67%. However, the calculation results of DUT_SM and DUT_NM which consider different out-of-plane characteristics basically coincide.

According to the analysis of the 258m structure, it can be seen that different types of shell element and material make the difference for the global structural response, in which the influence of the material model is larger, and the out-of-plane characteristic of the shell model has almost no effect on the global response of the structure. In order to analyze the influence of different layered shell models in the response of local shear wall members, the shear wall in the middle of the bottom floor is selected for comparison. In Figure 7, the displacement and load curves of the selected shear wall of different layered shell models under different levels of PGA are drawn to describe the state of the shear wall under earthquake.

It shows that, when the PGA is 35gal, the wall cracks but does not reach the yield stage, and the results of different models are basically the same. When the PGA is 400gal, there are significant differences between calculation results of different shell models, except for the coincidence of DUT_SM and DUT_NM. Using the DUT_SM and DUT_NM, the bearing capacity of the component reaches the maximum bearing capacity of 6582.13kN when the displacement is 6.71mm. The bearing capacity drops sharply after the peak point due to relatively large axial compression ratio. The internal force calculated by DUT_MM model does not reach the maximum bearing capacity, and the curve is in the rising phase. The internal force calculated by OpenSees_MP model reaches a peak load of
6711.74kN at a displacement of 3.61mm. It can be seen from the analysis that, when studying the nonlinear behavior of shear walls at important positions in the super high-rise structure, there are great differences in describing the damage of the shear wall components for different layered shell models, which may cause huge security risks to the structure. So it is very important to choose a reasonable layered shell model to carry out the nonlinear analysis.

Figure 7. The shear wall load displacement curves of different layered shell models

4. SENSITIVITY ANALYSIS OF THE GLOBAL STRUCTURAL DAMAGE

Adjusting the key components of the structure not only affects the global response of the structure, but also causes the change of internal force distribution in structure, resulting in the change of the global damage performance of the structure. This section focuses on the impact of shear walls, frame columns, coupling beams and strengthened stories on the global structural damage evolution, and figures out the most sensitive component type. Due to the great influence of high order vibration mode on the super high-rise structure, the global damage formulas proposed by He et al. (2016) are adopted, in which the damage combination of high modes is considered. And He et al have verified the relationship between the damage model and the maximum node displacement of the structure in the paper. When the damage index of the structure is close to 1.0, the maximum displacement of the structure tends to diverge. There is also a corresponding relationship between the damage model and the modified Park-Ang (MPA) layer damage model, the damage model and the MPA model can get similar results used to assess the problem of structural collapse studied by Guo and He (2017).

\[
D = \sqrt{1 - \prod_{i=1}^{n} (1 - D_i^2)} 
\]

(1)

\[
D_n = 1 - \frac{T_n^2}{T_{red}^2} 
\]

(2)

Where, \(D\) is the damage index of the structure, \(D_n\) is the damage index of the \(n\)-th order mode of the structure, \(T_n\) and \(T_{red}\) are the values of \(n\)-th order period of the structure before and after earthquake. And \(T_{red}\) is calculated using the mass and stiffness matrix at the end of the earthquake.

4.1 Consistency verification of the seismic wave responses
Using the three selected seismic waves, Hector Mine, RH2TG055 and Imperial Valley, the earthquake time-history analysis of the structure is conducted respectively, and the damage index of the structure under different conditions is calculated with the above formula.

![Graphs](8)

Figure 8. The structural damage comparison curve of different member sections

The comparison of structural damage curves calculated for the three seismic waves under various conditions is shown in Figure 8, from which, it can be seen that the trend and sensitivity of calculated curves of the three waves are basically the same.

4.2 The calculation and analysis of structural sensitivity

Because adjusting the section of components causes the change of the global performance of bending and shearing resistance, the coefficient formula of bending and shearing stiffness proposed by Miranda and Taghavi (2005) is applied to describe the change of structural performance quantitatively, which is also helpful to study the reasons of differences in the damage sensitivity of components. The unidirectional coefficient formula of bending and shearing stiffness is:

$$\alpha_0 = H \left( \frac{GA}{EI_0} \right)^{1/2}$$

(3)

For this structure, the coefficient formula of bending and shearing stiffness combining the two directions is:
\[ \alpha = \sqrt{\alpha_x^2 + (0.85\alpha_y)^2} \]  

(4)

Where, \( GA_0 \) is the global shear stiffness, \( EI_0 \) is the global bending stiffness, and \( H \) is the height of the structure.

4.2.1 The frame column

The conditions of initial section, half the frame column section, double the frame column section and triple the frame column section are calculated respectively, the comparison of stiffness coefficient value and damage curve is shown in Figure 9. It can be seen that the stiffness coefficient value decreases with increasing the section of the frame column, the corresponding damage curve is obviously lower than the initial model, and the damage development becomes slow. That is, the structure is relatively sensitive to the change of the stiffness of the frame column. Although increasing the section of the frame column increases the stiffness of the structure and seismic force at the same time, the frame column shares more seismic force initially sustained by the wall, so that the failure of the wall is slowed down. In addition, the method is in line with the concept of strong column and weak beam, enhancing the ductility of the structure.

<table>
<thead>
<tr>
<th>structure type</th>
<th>( \alpha_x )</th>
<th>( \alpha_y )</th>
<th>( \alpha )</th>
</tr>
</thead>
<tbody>
<tr>
<td>initial model</td>
<td>0.652</td>
<td>0.91</td>
<td>1.066</td>
</tr>
<tr>
<td>half the frame column section</td>
<td>1.017</td>
<td>1.183</td>
<td>1.465</td>
</tr>
<tr>
<td>double the frame column section</td>
<td>0.56</td>
<td>0.709</td>
<td>0.854</td>
</tr>
<tr>
<td>triple the frame column section</td>
<td>0.554</td>
<td>0.628</td>
<td>0.785</td>
</tr>
</tbody>
</table>

Figure 9. The structural damage comparison curve of different frame column sections

4.2.2 The shear wall

The conditions of initial section, double the shear wall section and triple the shear wall section are calculated respectively, the comparison of stiffness coefficient value and damage curve is shown in Figure 10. It can be seen that the stiffness coefficient value slightly decreases with increasing the section of the shear wall, the change of corresponding damage curve is not obvious, and the structure is not sensitive to the change of the stiffness of the shear wall. Because the seismic force born by the shear wall increases with the increase of the component resistance, so the internal force transmission of the structure does not change, and the damage curve is basically same. When the damage index \( D \) is greater than 0.6, the force born by the frame column increases rapidly because of the failure of the shear wall, which leads to the rapid development of the structural damage.

<table>
<thead>
<tr>
<th>structure type</th>
<th>( \alpha_x )</th>
<th>( \alpha_y )</th>
<th>( \alpha )</th>
</tr>
</thead>
<tbody>
<tr>
<td>initial model</td>
<td>0.652</td>
<td>0.91</td>
<td>1.066</td>
</tr>
<tr>
<td>double the shear wall section</td>
<td>0.654</td>
<td>0.923</td>
<td>1.078</td>
</tr>
<tr>
<td>triple the shear wall section</td>
<td>0.654</td>
<td>0.932</td>
<td>1.085</td>
</tr>
</tbody>
</table>

Figure 10. The structural damage comparison curve of different shear wall sections

4.2.3 The coupling beam

The conditions of initial section, half the coupling beam section, double the coupling beam section and triple the coupling beam section are calculated respectively, the comparison of stiffness coefficient
value and damage curve is shown in Figure 11. It can be seen that the stiffness coefficient value decreases with increasing the section of the coupling beam, the corresponding damage curve is obviously higher than the initial model. That is, the structure is relatively sensitive to the change of the stiffness of the coupling beam. Although the stiffness of the wall enhances with the increase of the section of the coupling beam, the wall limb at the junction occurs failure prematurely. At the same time, the more rigid coupling beam is not conducive to make full use of function of energy dissipation and shock absorption, so that the structure is more prone to brittle failure.

<table>
<thead>
<tr>
<th>structure type</th>
<th>$\alpha_x$</th>
<th>$\alpha_y$</th>
<th>$\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>initial model</td>
<td>0.652</td>
<td>0.91</td>
<td>1.066</td>
</tr>
<tr>
<td>half the coupling beam section</td>
<td>0.575</td>
<td>0.887</td>
<td>1.013</td>
</tr>
<tr>
<td>double the coupling beam section</td>
<td>0.791</td>
<td>1.019</td>
<td>1.221</td>
</tr>
<tr>
<td>triple the coupling beam section</td>
<td>0.897</td>
<td>1.116</td>
<td>1.352</td>
</tr>
</tbody>
</table>

Figure 11. The structural damage comparison curve of different coupling beam sections

### 4.2.4 The strengthened story

The conditions of initial model, double strengthened story member section, triple strengthened story member section, no strengthened stories, one strengthened story and five strengthened stories are calculated respectively, the comparison of stiffness coefficient value and damage curve is shown in Figure 12. It can be seen that the stiffness coefficient value increases obviously with increasing the section of the strengthened story member, but the corresponding damage curve is basically same. That is, the structure is not sensitive to the change of the section of the strengthened story member. The reason is similar to the shear wall, increasing the section of the member increases the stiffness of the structure, but the seismic force born by the member also increases, and this may lead to the appearance of weak floor because of the inhomogeneous distribution of structural floor stiffness. Decreasing the number of strengthened story reduces the value of stiffness coefficient and slows the development of structural damage. That is, the structure is relatively sensitive to the change of the number of strengthened story. The reason is that, the decrease of structural stiffness reduces the earthquake force, and makes the distribution of structural floor stiffness more homogeneous. Therefore, as far as the structure permits, there should be as few strengthened stories as possible.

<table>
<thead>
<tr>
<th>structure type</th>
<th>$\alpha_x$</th>
<th>$\alpha_y$</th>
<th>$\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>initial model (three strengthened stories)</td>
<td>0.652</td>
<td>0.91</td>
<td>1.066</td>
</tr>
<tr>
<td>double strengthened story member section</td>
<td>0.703</td>
<td>1.018</td>
<td>1.181</td>
</tr>
<tr>
<td>triple strengthened story member section</td>
<td>0.725</td>
<td>1.063</td>
<td>1.229</td>
</tr>
<tr>
<td>no strengthened story</td>
<td>0.556</td>
<td>0.733</td>
<td>0.873</td>
</tr>
<tr>
<td>one strengthened story</td>
<td>0.616</td>
<td>0.837</td>
<td>0.988</td>
</tr>
<tr>
<td>five strengthened stories</td>
<td>0.809</td>
<td>1.068</td>
<td>1.271</td>
</tr>
</tbody>
</table>

Figure 12. The structural damage comparison curve of different strengthened story
5. CONCLUSIONS

Based on the improved layered shell model, the elastic-plastic time-history analysis of a 258m height frame-core-tube super high-rise structure under earthquake is carried out. Two conclusions can be reached as follows.

(1) The material model of the shear wall element has a great influence on the global and local response. The global response of the structure is almost unaffected by the out-of-plane characteristic of the shell model. But there are significant differences in describing the damage of the shear wall components for different out-of-plane characteristics of layered shell models.

(2) The adjustment of frame columns, coupling beams and strengthened stories is relatively sensitive to the development of global structural damage. When the structural stiffness coefficient after adjustment is smaller than the initial model, the evolution of the structural damage is slow; otherwise the evolution of the structural damage is rapid.

6. ACKNOWLEDGMENTS

This research was financially supported by the National Natural Science Foundation of China (Grant No. 91315301 and 51261120376).

7. REFERENCES


Guo X, He Z (2017). Correlation between multi-mode global damage, MPA storey damage, and interstory drift of RC frame. 16th World Conference on Earthquake, 9-13 January, Santiago, Chile.


