IDC Cabinet Antiseismic Explicit Nonlinear Time History Analysis

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Abstract. In order to accurately solve the problem of large deformation and high stress of the IDC cabinet structure caused by seismic load, in this paper, the seismic test of IDC cabinet physical prototype was completed, and the dynamic response of IDC cabinet model was studied by Radioss time-history analysis. Firstly, the theory of nonlinear time-history analysis and the method of seismic test are introduced. Finally, the specific method of Radioss time-history analysis is introduced. The comparison of simulation and twst results verify the accuracy of key parameter setting, including connection unit, contact unit and hourglass control, providing reference for future seismic design of IDC cabinet products.

Keywords: IDC Cabinet, Seismic, Test, Time-History Analysis

1. Overview
Under the action of high intensity seismic, the structure will enter the elastic-plastic stage, and the structural stiffness will change [1]. The elastic static method is simple, but ignoring the important characteristics of structure such as dynamic characteristics and non-rigidity [2]. The response spectrum method can consider the dynamic characteristics of structures and the relationship between them and the seismic action, but it cannot give the whole process of the seismic response of structures, or the internal forces and deformation states of each component entering the elastic-plastic deformation stage. In order to study and calculate the elastoplastic deformation of IDC cabinet structure, it is necessary to carry out the nonlinear analysis [3]. Nonlinear time-history analysis is a method which can directly calculate the seismic response of structures by inputing seismic waves. It can describe the state and failure process of the structure under the earthquake [4]. It can calculate the internal force and deformation state of the structure at each moment during the whole process of earthquake response. It has the characteristics of the whole process simulation and is a relatively
reliable method. The application of this method can greatly improve the accuracy of communication equipments finite element analysis, which is of great significance.

2. Theory of nonlinear time history analysis

Time history analysis method is also called step by step integration method in mathematics. This method directly integrates the dynamic equation according to the selected seismic wave [5]. By using the method of step by step integration, the response of displacement, velocity and acceleration of the structure at each moment during the seismic process is calculated, so that the changes of internal forces from the elastic to inelastic phases of the structure under the action of high intensity seismic, as well as the whole process of cracking, damage and collapse of the structure can be observed[6]. The central difference method is adopted to discretize the dynamic equation in the time domain and transform it into the difference scheme with respect to time.

For an actual structure, after discretization by finite element method, the dynamic equation is:

$$\left[ M \right]\ddot{u} + \left[ C \right]\dot{u} + \left[ K \right]u = F(t)$$

(2.1)

\{\ddot{u}\}, \{\dot{u}\}, \{u\} and \{F(t)\} represent acceleration, velocity, displacement and the external force vectors respectively, which are all time-dependent. \[ M \], \[ C \] and \[ K \] are stiffness matrix, damping matrix and mass matrix respectively.

Assuming that when $t = 0$, the displacement, velocity and acceleration are known as $u_0$, $\dot{u}_0$ and $\ddot{u}_0$ respectively. And then divide the time interval into equal $n$ parts, that is $\Delta t = \frac{T}{n}$. The integral format we want to establish is from the known solutions at time $0$, $\Delta t$, $2\Delta t$, ..., $t$ to calculate the solution of the next time step.

In the central difference method, the velocity and acceleration vectors are discretized by the central difference.

$$\{\ddot{u}\}_i = \frac{1}{2\Delta t} \left( \{u\}_{i+\Delta} - \{u\}_{i-\Delta} \right)$$

(2.2)

$$\{\dot{u}\}_i = \frac{1}{\Delta t} \left( \{u\}_{i+\Delta} - 2\{u\}_i + \{u\}_{i-\Delta} \right)$$

(2.3)

The two equations above express the velocity and acceleration at time $t$ in terms of the displacement at the adjacent time. The dynamic equation at time $t$ is as follow:

$$\left[ M \right]\ddot{u}_i + \left[ C \right]\dot{u}_i + \left[ K \right]u_i = F_i$$

(2.4)

Substitute equations (3.2) and (3.3) into equation (3.4) to obtain the following:

$$\left( \frac{1}{\Delta t^2} \left[ M \right] + \frac{1}{2\Delta t} \left[ C \right] \right)\{u\}_{i+\Delta} = \{F\}_i - \left( \frac{1}{\Delta t^2} \left[ M \right] - \frac{2}{\Delta t} \left[ C \right] \right)\{u\}_i - \left( \frac{1}{\Delta t^2} \left[ M \right] - \frac{1}{2\Delta t} \left[ C \right] \right)\{u\}_{i-\Delta}$$

(2.5)
Thus, the above equation is reduced to a set of algebraic equations represented by displacements at adjacent moments. So we can solve \( \{u\}_{t+\Delta t} \).

From the perspective of computational stability, the disadvantage of the central difference method is that it is conditionally stable, that is, when the time step \( \Delta t \) is too large, the integral is unstable. So, the limit on step size is as follow:

\[
\Delta t \leq \Delta t_{cr} = \frac{l_e}{c}
\]  \hspace{1cm} (2.6)

The \( \Delta t_{cr} \) is critical time step, \( c \) is the speed of sound in the material, \( l_e \) is the element characteristic side length. Therefore when \( l_e \) is small, the \( \Delta t \) has to be small. So the calculation is very huge.

3. Selecting seismic wave
The selection of seismic wave in this paper is in accordance with the stipulations of non-power equipment under 8 seismic level defined in *YD5083-2005 Specification for Seismic Test of Telecommunications Equipment*. The value of the ground acceleration \( a_1 \) is 0.2g, the value of equipment importance coefficient \( k_1 \) is 1.1, the value of floor acceleration magnification \( k_2 \) is 3.0. According to equation 3.1, the value of floor input acceleration \( a_H \) is 0.66g, the response spectrum of the floor is shown in Fig.1. Synthetic seismic wave is generated by random phase method according to floor response spectrum. The peak acceleration of seismic wave is 0.66g. The duration of the seismic wave is 30s, and the duration of strong seismic is 20s, as shown in Fig. 2. The horizontal axis of the curve is time, and the unit is second. The vertical axis is the seismic acceleration, and the unit is g.

\[
a_H = a_1 \times k_1 \times k_2
\]  \hspace{1cm} (3.1)

![Figure 1. The response spectrum of the floor](image-url)
Figure 2. Random phase method according to floor response spectrum

4. Seismic test
The test was carried out on a high performance triaxial and six degrees of freedom shake table. The test object was IDC cabinet, with a counterweight of 600kg. After the seismic wave is loaded, the sensor signals of the physical prototype can be collected, including accelerations and displacements. After the test, LMS software was used for FFT processing of the acquisition signal, and the Seismic Signal software was used for baseline calibration and filtering processing of the measured seismic wave acceleration signal installed on the shake table, which was used as the input of simulation analysis.

4.1. Dynamic characteristic test
The test object was installed on the shake table, and the dynamic characteristic parameters such as natural frequency and damping ratio of the structure were measured. By adjusting the model, the difference between the frequency analysis results and the test results can be controlled within 10% to ensure that the analysis model can reflect the real performance of the equipment.

4.2. Seismic test
The shake table is used to conduct the seismic test on the test object, and the relative displacement data of the structure are collected. Compared with the simulation analysis results, the magnitude error between the test results and the calculation results is analyzed, and the model is constantly revised and optimized.

The configuration and installation of the equipment see Fig.3.

The cabinet is consisted of uprights, beams, top plate, bottom plate, inner uprights and trays. The frame of the cabinet is welded, and the inner uprights and trays are connected to the frame by bolts.

Figure 3. The configuration and installation of the equipment
5. Explicit nonlinear time history analysis

5.1. Finite element modeling
Hyper Mesh software was used to establish the finite element model of IDC cabinet model, including mesh division, element selection, material selection, section assignment and connection element setting, etc. The overall structure of the cabinet frame is shell element, which is divided into 220,000 units by 2D grid. Rigid units are used for welding and bolting, constraining 6 degrees of freedom. The weight block was simulated by Mass unit and Rbody unit, and the contact relationship between the weight block and the tray was simulated by contact unit. The analytical model see Fig.4.

![Figure 4. The analytical model](image)

The cabinet is made of steel and simulated by the /MAT/LAW2 constitutive model in Radioss. Relevant parameters of the model are shown in table 1.

| No. | Parameter | Item                      | Numerical value          |
|-----|-----------|---------------------------|--------------------------|
| 1   | [Rho ,Initial] | Density                  | 7.85e6 Kg/mm³            |
| 2   | [E]       | Modulus of elasticity     | 210 GPa                  |
| 3   | [nu]      | Poisson's ratio           | 0.33                     |
| 4   | [a]       | Yield strength            | 0.206 GPa                |
| 5   | [b]       | Hardening parameter       | 0.450 GPa                |
| 6   | [n]       | Hardening exponent        | 0.5                      |

5.2. The control of hourglass energy
Explicit analysis uses reduced integration. Reduced integration means that the number of nodes in a unit is less than the actual numbers during the calculating. This operation speeds up the computation, but results in a zero-energy pattern of cells, known as hourglass. The principle of hourglass is shown in Fig. 5. A two-dimensional element of a plane deforms under load, having only one integral node (red node) at the center of the element. If the deformation or strain at the four vertices of the element is
observed, it must be greater than 0. That will generate stress. Furthermore, strain energy will accumulate in the element. If we only observe the integral node, we can notice that the normal stress and the shear stress are both zero. Hourglass can lead to distortion and even divergence, so the hourglass model must be controlled. Correction methods include penalty function method and physical stability method.

(1) Penalty function method.

An hourglass viscous damping is added to provide additional bracing forces against the hourglass deformation of the unit.

(2) Physical stability method

The internal energy of the unit is corrected directly by the analytical method. Physical stabilization is the ultimate solution if you want to eliminate the hourglass problem at all, but it can lead to a multiplicative increase in computing time.

In this paper, by analyzing whether the stress amplitude of each component conforms to the actual stress level, the hourglass energy suppression method of each component is obtained. The components connected with the counterweight block are controlled by the physical stability method, and in Radioss the element type is set as ishell24. The penalty function method is used to solve the hourglass problem in other components, and the element type is set as ishell3.

5.3. Load and solve

First, we apply the gravitational acceleration \(g=9810\text{m/s}^2\) to the whole structure. Then set up the SPCs at four bottom ground mounting bolts, releasing the translational degree of freedom of the seismic wave loading direction. Then the seismic wave data measured on the shake table after data processing is loaded, and the task is submitted for solution.

6. Results analysis and comparison

6.1. Comparison between the modal analysis result and the dynamic characteristic test result

The dynamic characteristic curve of the structure obtained after FFT treatment by LMS is shown in Fig.6, the first-order mode of the structure calculated by modal analysis is shown in Fig.7, and the comparison of the first-order frequencies is shown in Table 2.
Figure 6. The dynamic characteristic curve of the structure obtained after FFT treatment by LMS

Figure 7. The first-order mode of the structure calculated by modal analysis

Table 2. The comparison of the first-order frequencies

| Working condition                  | Cabinet with a counterweight of 600kg |
|-----------------------------------|----------------------------------------|
| Modal order                       | First order frequency                  |
| Frequency results                 | Dynamic characteristic test            | Modal analysis       | Difference |
| 4.297                             | 4.555                                  | 6%                    |

The results show that the difference between the analysis results and the test results is 6%, which is far less than the general requirement for modal analysis (20%). The calculation model can reflect the real performance of the structure, and can be used for subsequent nonlinear time-history analysis to ensure the accuracy of the analysis results.

6.2. Comparison between the nonlinear time history analysis result and the seismic test result

The relative displacement curve between the top and bottom of the equipment during the seismic test obtained by the laser displacement sensor is shown in Fig. 8, and the relative displacement curve between the top and bottom of the equipment obtained from the nonlinear time-history analysis is shown in Fig. 9. The comparison between the two is shown in Table 3.
Figure 8. The relative displacement curve between the top and bottom of the equipment

Figure 9. The relative displacement curve between the top and bottom of the equipment obtained from the nonlinear time-history analysis

Table 3. The comparison

| Working condition | Cabinet with a counterweight of 600kg, seismic level 8 |
|-------------------|-------------------------------------------------------|
| Relative displacement peak value (mm) | Test result | Analysis result | Difference |
|                   | 35          | 32            | 8%           |

From the results of the curves, the waveform, trends and peak value of the two curves basically agree, indicating the accuracy of the method used.
7. Conclusions

(1) The difference between the frequency simulation results and the relative displacement simulation results and the test results is within 10%, and the simulation results can reflect the structure's stress characteristics.

(2) The finite element model of IDC cabinet established in this paper takes into account the influence of material plasticity and contact between structures, and the analysis results are reasonable and accurate. The modeling method can be extended to the nonlinear time-history analysis of other structures.

(3) The method proposed in this paper provides a powerful tool for studying the reaction mechanism of IDC cabinet structure under high intensity seismic. However, from the perspective of engineering application, this method has a high requirement on computer memory and time resources, which requires further research on accelerated computing.

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