

Seismic Analysis of Structural Enclosures: A Review

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ABSTRACT: For a structural enclosure, seismic testing consumes a lot of resources. The accuracy of obtaining seismic performance through theoretical calculation in the design stage has become the focus of attention. However, the choice of theoretical calculation methods affects the accuracy and timeliness of the structural enclosure's seismic performance analysis. This paper uses the principles of structural seismic analysis to determine the applicability of each theoretical calculation method, including elastic static method, response spectrum method, bottom shear method, nonlinear static method, time history analysis method, energy design method, and random vibration method, to solve the rationality of the selection of seismic performance calculation methods for structural enclosure. The main objective of this paper is to perform a literature review on works done on seismic analysis of structural enclosures.

I. INTRODUCTION

An earthquake is a sudden and transient motion of the earth's surface. According to geologists, the earth has suffered earthquakes for hundreds of millions of years, even before humans came into existence. In terms of the geological time scale, it is only recently (the middle of seventeenth century) that an earthquake has been viewed as a natural phenomenon driven by the processes of the earth as a planet. Thus, subsequent work especially in nineteenth century, led to tremendous progress on the instrumental side for the measurement of earthquake data. Seismological data from many earthquakes were collected and analysed to map and understand the phenomena of earthquakes. These data were even used to resolve the earth's internal structure to a remarkable degree, which in turn, helped towards the development of different theories to explain the causes of earthquakes. While the body of knowledge derived from the study of collected seismological data has helped in the rational design of structures to withstand earthquakes, it has also revealed the uncertain nature of future earthquakes for which such structures are to be designed. Therefore, probabilistic concepts in dealing with earthquakes and earthquake resistant designs have also emerged. Both seismologists and earthquake engineers use the seismological data for the understanding of an earthquake and its effects, but their aims are different. Seismologists focus their attention on the global issues of earthquakes and are more concerned with the geological aspects, including the prediction of earthquakes. Earthquake engineers, on the other hand, are concerned mainly with the local effects of earthquakes, which are capable of causing significant damage to structures. They transform seismological data into a form which is more appropriate for the prediction of damage to structures or, alternatively, the safe design of structures. However, there are many topics in seismology that are of immediate engineering interest, especially in the better understanding of seismological data and its use for seismic design of structures.

Seismic analysis is a subset of structural analysis and is the calculation of the response of a building (or nonbuilding) structure to earthquakes. It is part of the process of structural design or earthquake engineering in regions where earthquakes are prevalent. Earthquake has great threat and can cause damage for the enclosure system. Structural enclosures with poor seismic performance are easy to be damaged under the earthquake, which leads to the failure of the whole system. Therefore, the seismic performance of the enclosures must be considered to avoid the effects caused by an earthquake.

At present, the methods for analysing the seismic performance of structures mainly include theoretical calculation methods, experimental analysis methods and seismic damage statistics methods. However, as the requirements for the seismic performance of the structural enclosures is important, the theoretical calculation method is required to analyse the seismic performance of enclosures in the product design stage, and the selection of different

theoretical calculation methods affects the accuracy of seismic performance analysis of enclosures. This paper summarizes and analyses the research progress of structural seismic analysis theory, analyses its applicability from the principle of seismic analysis and provides a theoretical basis for the selection of seismic theory and methods required for the seismic analysis of the structural enclosures.

II. SEISMIC REQUIREMENTS FOR STRUCTURAL ENCLOSURES

Seismic loading is one of the basic concepts of earthquake engineering which means application of a seismic oscillation to a structure. It happens at contact surfaces of a structure either with the ground or with adjacent structures. Seismic loading depends, primarily on seismic hazard, geotechnical parameters of the site, and structure's natural frequency etc. There are horizontal and vertical components of earthquake ground motions, but the horizontal component of earthquake ground motion is the main cause of the damages caused. Therefore, only horizontal earthquake ground motion is considered in the seismic analysis.

Inertial forces arise from the move. It is sufficient to determine these forces and apply them on the structure. Thus, the dynamic load is transformed into a static load. It is difficult to predict the precise movement of subsoil and therefore it is difficult to determine the seismic forces precisely. But it is possible to apply formulas of a technical standard or employ the frequency spectrum of a real earthquake for the purpose of analysis.

A. Symbols and Notations

A_p : Dynamic Response Amplification factor

F_a : Amplification factor of site response acceleration spectrum

F_{ph} : Earthquake load (Horizontal direction)

F_{pv} : Earthquake load (Vertical direction)

$\frac{H_x}{H_n}$: Ratio of building height where equipment is placed to building height

I_p : Seismic Importance factor

N_a : Near seismic source factor

R_{pa} : Allowance seismic response reduction factor

R_p : Seismic response reduction factor

S_{ds} : Design horizontal response acceleration at short periods

S_s : Structural response acceleration parameter at short periods

W_p : System weight

B. Structural Classification

A rigid structure is one that has a primary natural frequency of at least 17 Hz, according to 2005 ASCE7 Section 11.2 of the code. A structure is categorised in accordance with this definition as:

Flexible Structure: Natural frequency < 17 Hz

Rigid Structure: Natural frequency \geq 17 Hz

C. Seismic Forces

The total design horizontal force F_{ph} , is independent of building structural properties and shall be determined based on IBC (International Building Code) 2009 and NEHRP (National Earthquake Hazard Reduction Program) in accordance with the appropriate formula stated in ASCE 7 section 13.3 as,

$$F_{ph} = 0.4 S_{ds} \frac{A_p \times I_p}{R_{pa}} \left[1 + 2 \frac{H_x}{H_n} \right] W_p$$

The total design vertical force F_{pv} , in addition to system weight is determined from ASCE 7 section 12.4.2.2, earthquake usually produce much higher vertical seismic forces which must be considered at 67 % of the designed horizontal force which is represented as,

$$F_{pv} = \frac{2}{3} F_{ph}$$

Load combinations for the analysis are defined in IBC 2009 section 1605 which represents the worst case basic loads for strength design according to the following formulas:

Horizontal load: Apply the value of F_{ph} along the horizontal direction.

Vertical load: Apply along the vertical direction according to the following values,

$1.2 W_p + F_{pv}$ for downward (direction same as gravity)

$0.9 W_p - F_{pv}$ for uplift (direction opposite to gravity)

D. Determining Parameters A_p , R_p , I_p , $\frac{H_x}{H_n}$ and S_{ds}

a. Dynamic Response Amplification Factor, A_p

This factor depends on structural rigidity and material ductility. ASCE 7 Chapter 13, Table 13.5 – 1 classifies material into the following three deformability categories: low, limited and high. Although the structure framing could be classified as high deformability element, a conservative approach of limited deformability classification is taken due to the connection in framing. According to this classification, in reference to ASCE 7 Table 13.5 – 1 this factor assumes the following values: 1.0 for rigid structure and 2.5 for a flexible structure.

b. Seismic Response Reduction Factor, R_p

This factor does not depend on structural rigidity but does depend on material ductility. With a material classification of limited deformability, in reference to ASCE 7 Table 13.5 – 1 this factor assumes the value of 2.5.

c. Seismic Importance Factor, I_p

This factor is assigned the values of either 1.0 or 1.5 the value is based on according to the harm potential a failure has to human life and environment. A value of 1.5 is assigned to components carrying Hazardous Potential Material (HPM).

d. Ratio of Building Height, $\frac{H_x}{H_n}$

Worst case for equipment is simulated for component placed at building top is with this ratio assigned a value of 1.0.

e. Horizontal Response Acceleration at Short Periods, S_{ds}

By considering the conservative worst case is and in reference to ASCE 7 Table 13.5 – 2 the value of horizontal response acceleration at short periods, S_{ds} factor is assigned the value of 1.024.

E. Seismic Hazard Analysis

A seismic hazard at a site is defined as a quantitative estimation of the most possible ground shaking at the site. It may be obtained either by a deterministic approach or by a probabilistic approach. The possible ground shaking may be represented by peak ground acceleration, peak ground velocity, or ordinates of the response spectrum. Whatever the approach or the representation of ground shaking are, the seismic hazard analysis requires the information about some important factors in the neighbourhood of the site. They include geologic evidence of earthquake sources, fault activity, fault rupture length, historical, and instrumental seismicity. Geologic evidence of earthquake sources is characterized by the evidence of ground movement of the two sides of a fault, disruption of ground motion, topographic changes, and so on. Fault activity is characterized by movements of ground at the fault region; it is the active fault that leads to future earthquakes. Past earthquake data regarding the relationship between the fault rupture length and the magnitude of earthquake provide valuable information for predicting the magnitude of future earthquakes and, thus, help in seismic hazard analysis. Historical records of earthquakes and instrumentally recorded ground motions at a site also provide valuable information on potential future earthquake sources in the vicinity of the site.

As mentioned above, seismic hazard analysis may be carried out using two approaches, namely, deterministic seismic hazard analysis and probabilistic seismic hazard analysis.

a. Deterministic Hazard Analysis

Deterministic hazard analysis (DSHA) is a simple procedure which provides a straightforward framework for the computation of ground motions to be used for the worst-case design. For specialty or special structures such as nuclear power plants, large dams, and so on, DSHA can be used to provide a safe design. DSHA involves many subjective decisions and does not provide any information on the likelihood of failure of the structure over a given period of time. Because of these reasons, its application is restricted when sufficient information is not available to carry out any probabilistic analysis. The DSHA is carried out using the following steps.

- i. Identification of all potential earthquake sources surrounding the site, including the source geometry.
- ii. Evaluation of source to site distance for each earthquake source. The distance is characterized by the shortest epicentral distance or hypocentre distance if the source is a line source.
- iii. Identification of the maximum (likely) earthquake expressed in terms of magnitude or any other parameter for ground shaking for each source.
- iv. Selection of the predictive relationship (or attenuation relationship) to find the seismic hazard caused at the site due to an earthquake occurring in any of the sources.
- v. Determination of the worst-case ground shaking parameter at the site.

b. Probabilistic Hazard Analysis

Probabilistic hazard analysis (PSHA) uses probabilistic concepts to predict the probability of occurrence of a certain level of ground shaking at a site by considering uncertainties in the size, location, rate of occurrence of earthquake, and the predictive relationship. The PSHA is carried out using the following steps.

- i. The first step is to identify and characterize the earthquake sources probabilistically. This involves assigning a probability of occurrence of an earthquake at a point within the source zone. Generally, a uniform probability distribution is assumed for each source zone, that is, it is assumed that the earthquake originating from each point within the source zone is equally likely. Secondly, the probability distribution of the source to site distance, considering all points in the source zone to be potential sources of an earthquake, is determined from the source geometry.
- ii. The second step is to characterize the seismicity of each source zone. The seismicity is specified by a recurrence relationship indicating the average rate at which an earthquake of a particular size will be exceeded.
- iii. In the third step, a predictive relationship is used to obtain a seismic parameter at the site for a given magnitude of earthquake and source to site distance for each source zone. Generally, the uncertainty is expressed by a log normal distribution by specifying a standard deviation for the seismic parameter and the predictive relationship is expressed for the mean value of the parameter.

III. ANALYTICAL METHODS

A. Equivalent Static Analysis

The principle of equivalent static analysis is to simplify the seismic dynamic response of the structure to static response, and conduct seismic analysis of the structure in the static mode of equivalent seismic load. It assumes that the vibration of each part of the structure is the same as the ground motion, and the inertial force generated under the action of ground horizontal acceleration is regarded as static force acting on the structure.

B. Response Spectrum Method

The response spectrum method is based on the dynamic relationship between structural dynamic characteristics and ground motion characteristics, and uses response spectrum to calculate the resonance effect generated by structural dynamic characteristics. It assumes that the earthquake is a stationary stochastic process, the structure is in the linear elastic response stage and the response can be superimposed, ignoring the interaction between the ground and the structure.

C. Bottom Shear Method

The bottom shear method can actually be regarded as a simplified mode decomposition response spectrum method. Its principle is to first calculate the total horizontal seismic action acting on the structure, that is, the shear force acting on the bottom of the structure, and then distribute the bottom shear force to each particle according to a certain rule, so as to obtain the horizontal seismic action of each particle, and then calculate the seismic response of the structure according to the static method.

D. Nonlinear Static Method

The nonlinear static method is a method that considers that the structure enters the elastic-plastic work stage under earthquake damage. Its representative is the capacity spectrum method combined with Pushover

analysis. The basic principle is to establish one capacity curve and another demand curve. The two curves are placed in the same coordinate system, and the intersection point of the two curves is the "target displacement point". Compared with the allowable displacement value, it can be judged whether or not the structure meets the seismic performance requirements.

E. Time History Analysis Method

Time history analysis method is a deterministic dynamic analysis method. It starts from selecting the appropriate seismic wave input (earthquake acceleration time history), and uses multi-node finite element dynamic calculation models to establish vibration differential equations for linear or nonlinear structures, then directly integrate and solve the equation step by step to calculate the acceleration, velocity and displacement response of each instantaneous structure in the earthquake process, so as to obtain the time history relationship of structural stress and strain.

F. Energy Design Method

During the earthquake, the structure is in the energy field, and there is continuous energy input, transformation and dissipation between the ground and the structure. The energy design method believes that the response of structure in earthquake is a process of dissipating seismic input energy, which is to consider solving the problems of energy input and dissipation calculation of different structures, that is, to determine the energy needed to be dissipated by the structure and the energy dissipation capacity of the structure itself.

G. Random Vibration Method

At present, the random vibration method mainly used in seismic response analysis is the frequency domain random vibration method, and there is also the time domain random vibration method which is not widely used. The frequency domain random vibration method is to obtain the statistical characteristics of the structure seismic response by establishing the relationship of the power spectrum function between the ground motion input and the structure seismic response output.

IV. NUMERICAL METHODS

Most of the numerical methods for seismic modelling belong to one of the following three categories: spectral methods, finite difference methods, or finite element methods.

A. Spectral Method

This method approximates the wave field using Fourier modes. They are very efficient for certain specific geometries, such as a layered earth with purely horizontal interfaces, but are usually not applicable to more generic geometries.

B. Finite Difference Method

This method approximates the wave field on a uniform grid. They can be applied to generic geological structures and are very easy to implement, which makes them currently the most popular choice for large-scale modelling and inverse problems. For complex topographies and sharp contrasts in the rock material, however, they require a very fine grid resolution to remain accurate.

C. Finite Element Method

This method can handle complex geological interfaces more efficiently by carefully subdividing the earth's geometry into simple geometries called elements. The wave field is then approximated on each element using simple interpolation functions, usually polynomials up to a certain degree. Finite elements methods are more difficult to implement compared to finite difference methods and require more computation time and memory to model simple rock structures. For complex geological structures, however, they can remain very accurate without having to refine the element mesh as long as the elements are well-aligned with the geological interfaces. Finite element methods can therefore significantly outperform finite difference methods in such cases.

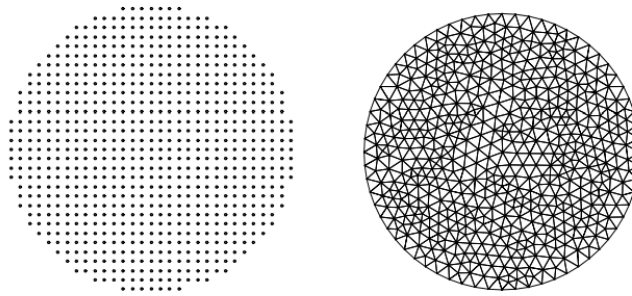


Fig. 1. Finite difference grid (left) and triangular element mesh (right) for a circular domain.

V. SEISMIC ANALYSIS METHODOLOGY

As the structural enclosure are vulnerable to seismic loads, hence it is important to investigate the structure's performance under seismic activity. The strength and stiffness of the structure subjected to seismic loads are analysed using the specifications prescribed by the International Building Code (IBC). The stepwise procedure to determine the results are as follows,

A. Develop a Finite Element Model

As all analyses in the study are dependent on this model, this is a crucial stage that will likely take more time to complete. To account for the mass attributes of all significant components, significant effort must be made. Each significant component's load path in relation to the main structure must be specified and accurately described. It is necessary to account for and assign each material property correctly. The system centre of mass ought to match the actual dimensions as closely as feasible. There must be tie down points in the model, and these points should be represented by "fixed" constraints.

B. Perform Static Analysis

The structure's stress status under normal load circumstances is reported by this analysis. The only loading condition that should be taken into account is gravity. The modal analysis is a precursor to this approach. With loading, the structural modal response modifies. Therefore, the stresses on the structure due to gravitational loading must be determined and input into the modal analysis.

C. Perform Modal Analysis

Modal analysis must be conducted on a pre-stressed frame. In this step, structural stiffness is determined. Typically, we seek to identify the structure's first three main modes. The value of the lowest modal frequency determines structural stiffness. If this frequency is more than or equal to 17 Hz, the structure is stiff or rigid.

D. Run Equivalent Seismic Analysis

This stage involves doing an analogous static analysis on the structure based on loading scenarios that take structural stiffness and potential material hazards into account. In comparison to a rigid system, a flexible system necessitates evaluation at higher loads.

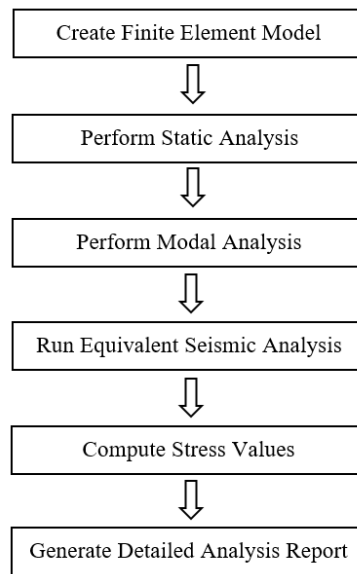


Fig. 2. Seismic Analysis Flowchart.

VI. CASE STUDIES

1. Seismic Analysis on Historical Bridge Using Photogrammetry and Finite Elements

a. Introduction

Seismic-resistant design and construction standards were developed for the design of conventional structures. However, large buildings do not have a specific design code. The seismic-resistant design standards, in the seismic part, consider that constructions should be designed generally for the demands in horizontal seismic component. The damping coefficient with respect to the critical one in these structures must be smaller than the used conventionally (5%). At the present time there are resistant seismic norms for the construction of bridges, the historical bridges by being patrimony of the nation must be maintained in good conditions, the use of techniques to evaluate the changes before seisms, in this will allow to evidence the necessity of reinforcements in specific zones of the structure. This can lead to aspects of analysis and design that are not taken into account in the case of conventional structures. Seisms with vertical acceleration records have presented acceleration values much higher than traditionally considered. According to the Northridge seism, vertical accelerations of up to 85% of gravity acceleration occurred on the soil. The spectral response of the seisms is related to the distance from the epicentre to the point where the accelerogram is recorded. Three fields can be established in seisms: near field, intermediate field and far field.

When a spectral analysis for vertical acceleration as a function of distance is added to these analyses, a detailed study of the response of structures can be made. Multi-physics simulation works where the behaviour of old structures is evaluated and modern construction systems, Through the use of FEA analysis with simulation tools is possible to evaluate the deformations and pressures exerted by fluids or acceleration variations over time.

b. Design Methodology

The photogrammetric adjustment of the images captured with the optical sensor was performed. The software used to make this adjustment is Agisoft Photoscan. This software searches for similar points by analysing all the captured images. The algorithm used to perform this photogrammetric adjustment is the SIFT function for machine vision, which also uses binary descriptors to facilitate the work of the processor and to match the key points of each image with the adjacent ones. With the alignment of the images, the points that match in their XYZ coordinates are calculated and with each point, an interpolation is made to form an irregular triangular mesh. Figure 3b, shows the adjustment of the images captured with the UAV figure 3. a-b, figure 3. c-d show the model obtained.

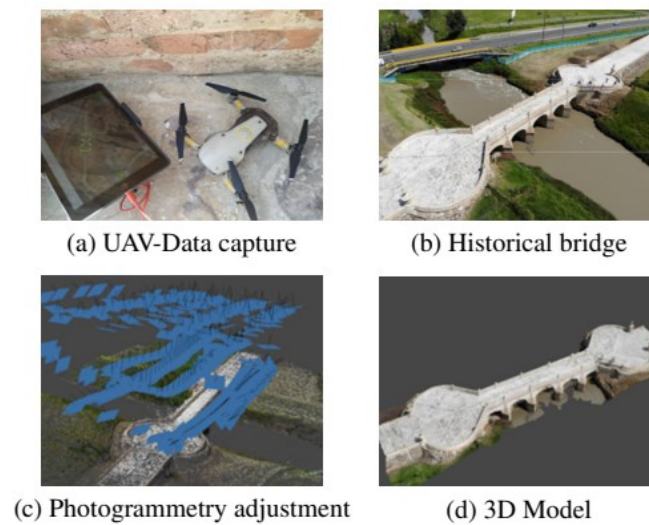


Fig. 3. Data Capture process.

With the study of the soil layers and the confirmation of the materials that make up the soil, it is possible to properly set the soil simulation parameters. The mesh was constructed by the ICEM mesher figure 4, was generated with automatic meshing options and fine mesh configuration, resulting in the mesh having 215954 nodes and 120410 elements.

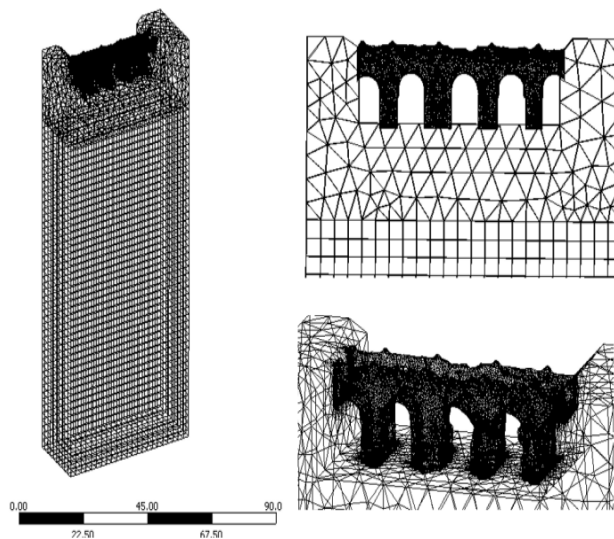


Fig. 4. 3D Mesh.

Studies on FE simulation have been carried out a large number of structures, showing the behaviour of deformations and stresses. Based on this, it was possible to obtain the waveform of the acceleration signal of an earthquake figure 5.

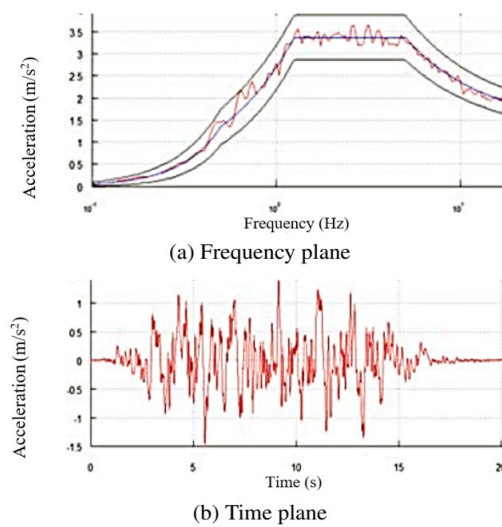


Fig. 5. Seismic signal.

c. Simulation setup

Using the acceleration signal in time, the simulation configuration transient structural is made, configuring 3 different materials; silicon and magnesium for the soil layers, and composite material concrete for the 3D model of the bridge, the simulation time is configured for 15 s, with a number of 100-time steps. This acceleration was applied in the +Z or normal direction to the base of the rectangular cube configured as a ground with 100 m depth.

d. Results

Evaluating the behaviour of the structure before a seism was done by the post processing of three variables such as deformation, the equivalent of elastic tension and the equivalent Von Mises stress.

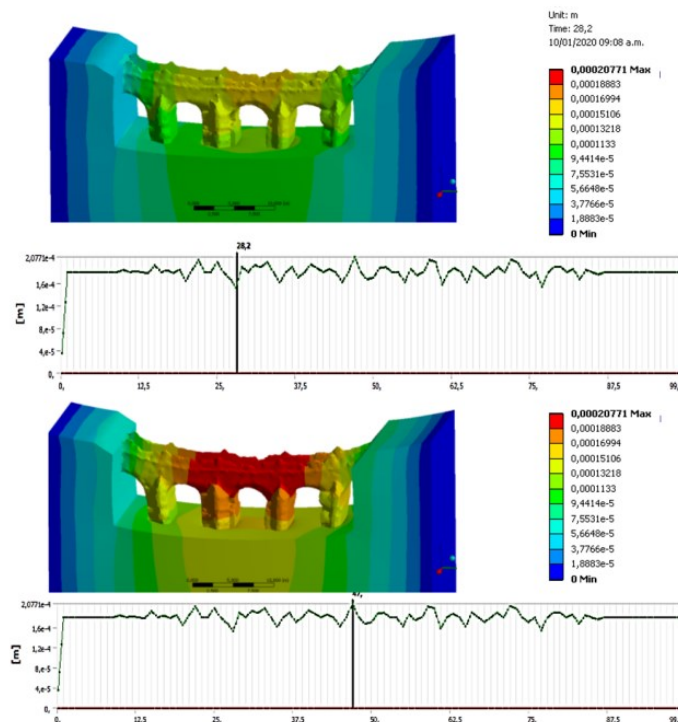


Fig. 6. Deformation at minimum and maximum seismic acceleration.

Figure 6, shows the deformations in the ground and the structure with a scale factor in the animation that allows appreciate the zones of maximum deformation; the simulation shows that the structure of the concrete bridge in front of vertical seism will present maximum deformations in the centre of the structure mainly in the central arch near to 0.2 mm.

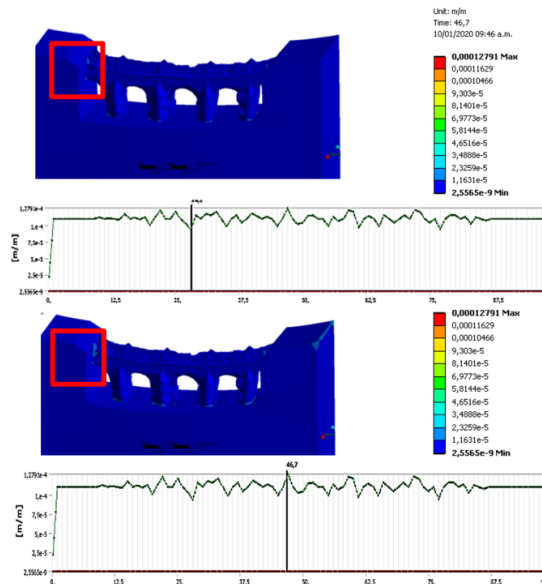


Fig. 7. Equivalent elastic strain at minimum and maximum seismic acceleration.

Figure 7 shows the results of simulated elastic stress equivalents. Maximum values are shown at the lateral support points of the structure and at the lower parts of the bridge piers, the values are shown near cracks in the 3D model with values close to 3.4×10^{-5} (red box). For negative peak values of simulated seismic accelerations, the equivalent elastic stress regions in the bridge are smaller than for positive peak of maximum accelerations.

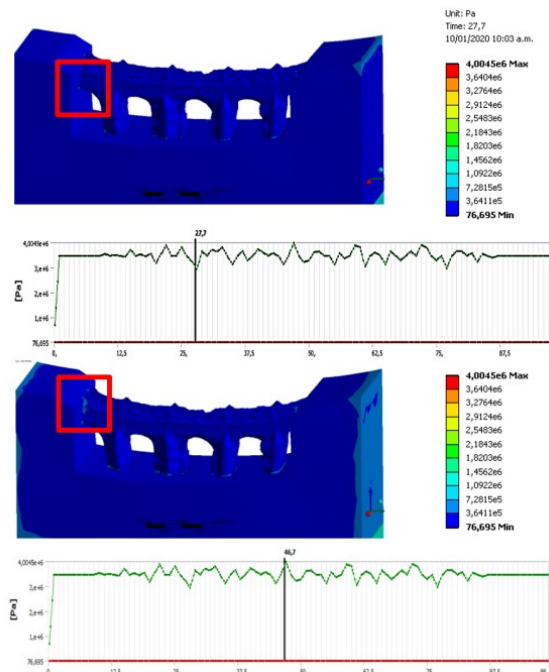


Fig. 8. Von Mises Stress equivalent at times of minimum and maximum seismic acceleration.

Figure 8, simulation results are observed for stress Von Mises equivalent, with their highest values in places similar to those obtained for the equivalent of elastic strain, for the colour scale they are observed in the 3D model with values close to 1.45×10^6 Pa, (red box), for negative peak values of simulated seismic accelerations the stress regions are smaller than for peak or maximum positive accelerations.

The structural simulation of the model in 3D by means of photogrammetry was possible once the solid and the mesh is achieved, by means of simulation tools based on finite elements, the deformations and efforts obtained allow to appreciate zones in which the structure can fail, because the simulation is made in transient it was possible to appreciate the deformations as a function of time which conserved the waveform of the acceleration configured in the base of the soil.

2. Seismic Performance of Enclosure Wall in Post-earthquake Temporary Prefabricated Light-Weight Steel Structure

a. Introduction

The prefabricated light-weight steel structure in which members were fabricated in factor and assembled onsite had many advantages, such as relatively better construction quality and faster construction speed. Therefore, the structure is often applied to the rural residential. Due to the light weight of the structure, the earthquake resistant behaviour is good and the structure is especially suitable for rural residential. In order to make use of the construction waste, prefabricated slender columns and prefabricated strip slabs were proposed and applied to enclosure walls, in the post-earthquake temporary prefabricated light-weight steel structure. Therefore, the difference between the post-earthquake temporary prefabricated light-weight steel structure and conventional light-weight steel structure is the enclosure walls. For the conventional light-weight steel structure, scholars mainly focus on the seismic performance of beam-to-column joints or end-plate joints.

In order to make full use of construction waste and apply the post-earthquake temporary prefabricated light-weight steel structure to rural residential, the seismic performance of the structure was investigated by the test method in this paper and the seismic performance of the enclosure walls was studied in particular.

b. Experimental Model

In order to investigate the seismic performance of prefabricated light-weight steel structure, a full-scale two-story experimental model was built (Figure 9). The full-scale two-story experimental model mainly included enclosure walls composed of prefabricated strip slabs and prefabricated slender columns, steel frame composed of steel H beams and steel H columns (Figure 10), precast reinforced concrete floor slab, roof board, etc. For the enclosure walls, prefabricated strip slabs with 180 mm width, 40 mm thickness, and different lengths were placed on the prefabricated slender columns with 100 mm width and 100 mm thickness, which were perpendicular to the prefabricated slender columns.

During the construction of the full-scale two-story experimental model, prefabricated slender columns of enclosure wall should be first connected with steel H beams of the steel frame. The bottom of the prefabricated slender column was embedded in the ground beam and the middle of pre-fabricated slender column, and the top of the prefabricated slender column was connected with steel H beams according to the U-shaped hoop by using bolts. For the steel frame, the hot-rolled H-shaped steel (HW 125 × 125 × 6.5 × 9.0 mm) was used as a column and beam. In the experimental model, precast reinforced concrete floor slabs with 60 mm thickness were connected with beams by welding, and fine aggregate concrete with 30 mm thickness was poured to the surface of precast reinforced concrete floor slabs. For the roof board with 60 mm thickness, it was placed on the roof truss. In order to simulate additional gravity load, the weight of 5.76 kN was placed on the surface of floor slab on the second floor (Figure 11(a)) and the weight of 1.73 kN was placed on the roof truss (Figure 11(b)). Total weight of the experimental model was about 117.6 kN.

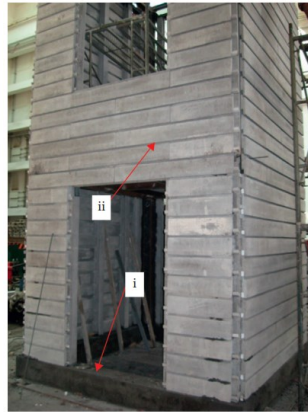


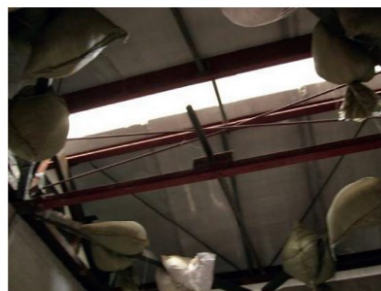
Fig. 9. Full-scale two-story experimental model (i) Ground beam (ii) Enclosure wall.



Fig. 10. The steel frame composed of steel H beams and steel H columns.



(a)



(b)

Fig. 11. Additional gravity load (a) On the floor slab of the second floor and (b) On the roof truss.

c. Testing System

In the test, the test system of the shake table simulating earthquake was used, mainly including shake table, electrohydraulic servo actuator, shake table control system, data acquisition system, etc. as shown in Figure 12. In addition, the parameters for the shake table used in the test are shown in Table 1. The test system of the shake table simulating earthquake can be used to study the response and failure mechanisms of the structure subjected to earthquake in the laboratory. In the test system of the shake table simulating earthquake, the shake table can only vibrate horizontally in the test. For the shake table control system, it can be divided into two parts, including simulation control and digital control. Simulation control can generate waveshape with different frequencies; besides, the recorded signal of earthquake with different intensities can also be input into the simulation control. For digital control, it can not only extend or shorten the period of input waveshape but also increase or decrease the maximum acceleration of seismic signal, reducing the distortion of the seismic signal. During the test, according to the computer control system, seismic signals with different maximum accelerations were input into the shake table control system which can control the vibration of the electrohydraulic servo actuator with different maximum accelerations. Then, the electrohydraulic servo actuator can make the shake table vibrate with different maximum accelerations. During the process of vibration of the experimental model, the signals from acceleration transducers and displacement transducers fixed on different locations of the experimental model can be recorded by the data acquisition system.

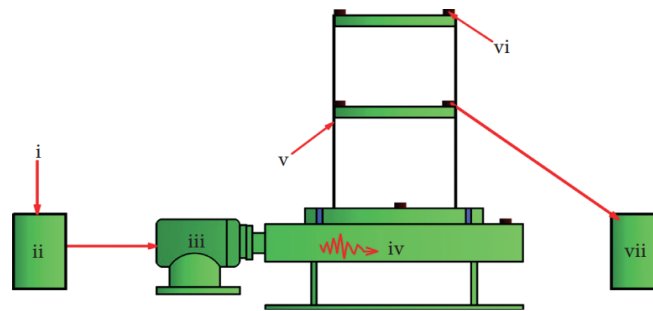


Fig. 12. Seismic simulation shake table system. (i) Seismic signal, (ii) shake table control system, (iii) electrohydraulic servo actuator, (iv) shake table, (v) experimental model, (vi) transducer, and (vii) data acquisition system.

Table 1. Parameters for the shake table.

Size (mm)	Dead weight (kN)	Maximum displacement (mm)	Maximum speed (mm/s)	Maximum acceleration (g)	Frequency range (Hz)	Direction of vibration	Vibration wave
3000 × 3000	58.8	±127	±60	±2.5	0.1~50	Horizontal	Seismic signal

d. Test Procedure

In order to obtain the seismic performance of the post-earthquake temporary prefabricated light-weight steel structure, two typical recorded seismic signals were used in the test, including the El-Centro wave and Kobe wave. For the El-Centro wave, it was widely used in the study of the seismic performance of the structure and the maximum acceleration in the east-west direction and maximum amplitude was 210.1 cm/s^2 and 7.3 m , respectively, For the Kobe wave, the maximum acceleration in the east-west direction and maximum amplitude were 818.02 cm/s^2 and 55.1 m , respectively. In the test, seismic signals with different maximum accelerations were input into the shake table control system which could control the vibration of the electrohydraulic servo actuator, and then the shake table could make the experimental model vibrate. In addition, in the process of the test, maximum acceleration of the seismic signal input into the shake table control system increased with the step of 0.1 cm/s^2 until severe cracks appeared on the surface of the enclosure wall. At the end of the shake table test with different loading steps, the test should be terminated and then we observed whether severe cracks occurred on the surface of enclosure wall or the experimental model was damaged. The detailed procedure for the test is shown in Table 2.

Table 2. Loading procedure for the experimental model.

Serial number	Seismic signal	Maximum acceleration (cm/s ²)	Seismic intensity
Carry out the resonance test and obtain the natural frequency of the experimental model not subjected to the seismic signal			
1	Kobe wave	0.1	V
2	El-Centro wave	0.1	
Observe cracks and carry out the resonance test			
3	Kobe wave	0.2	VII
4	El-Centro wave	0.2	V
Observe cracks and carry out the resonance test			
5	Kobe wave	0.3	VIII
6	El-Centro wave	0.3	VI
Observe cracks and carry out the resonance test			
7	Kobe wave	0.4	IX
8	El-Centro wave	0.4	VII
Observe cracks and carry out the resonance test			
9	El-Centro wave	0.5	VIII
Observe cracks and carry out the resonance test			
10	El-Centro wave	0.6	IX
Observe cracks and carry out the resonance test			

e. Results

When the Kobe wave with maximum acceleration of 0.4 cm/s^2 was input into the shake table control system, the experimental model shook very violently, large displacement occurring on top of the experimental model. In addition, the roof board fixed to the roof truss almost fell off the top of the experimental model. Many cracks appeared on the surface of the mortar filled to the corner of the enclosure wall. Besides, cracks also appeared on the surface of prefabricated strip slabs, but prefabricated strip slabs had no fracture. For the flexible connection between the steel frame and enclosure walls, they were not damaged at all.

When the El-Centro wave with maximum acceleration of 0.6 cm/s^2 was input into the shake table control system, shaking of the experimental model was also very violent and a large number of cracks appeared on the surface of mortar filled to the corner of the enclosure wall, but the mortar was not damaged. The roof board fixed to the roof truss fell off the top of the experimental model. In addition, large deformation of the steel frame did not occur and the enclosure walls were also not damaged (Figure 13(a)), a lot of cracks appearing on the surface of enclosure walls. In addition, some flexible connections between the steel frame and enclosure walls parallel to the direction of vibration were slightly damaged. Moreover, the prefabricated strip slabs of enclosure walls parallel to the direction of vibration were loosened relative to the mortar filled to the corner of the enclosure wall, as shown in Figure 13(b). Besides, cracks occurred on the connection of the ground beam and enclosure wall, as shown in Figure 13(c).

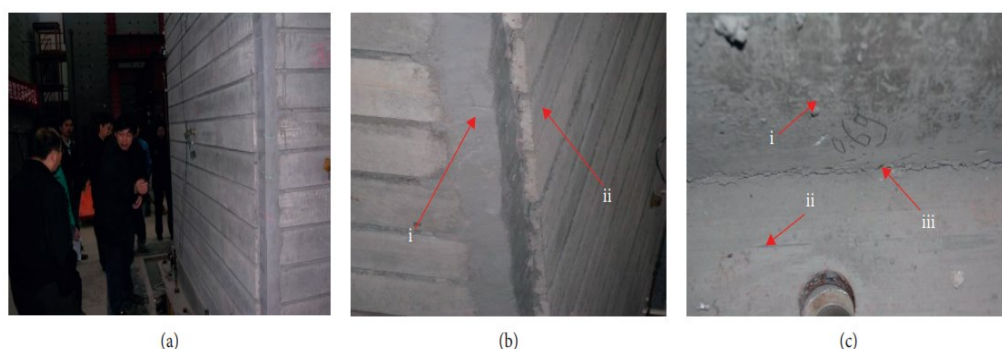


Fig. 13. Test phenomenon. (a) Enclosure walls subjected to earthquake (b) Loosened prefabricated strip slabs: (i) mortar and (ii) prefabricated strip slab (c) Cracks on the bottom of enclosure wall: (i) enclosure wall, (ii) ground beam, and (iii) crack.

The seismic performance of the post-earthquake temporary prefabricated light-weight steel structure was investigated by the test method. According to the inputting Kobe wave and El-Centro wave into the shake table control system, the full-scale two-story experimental model could vibrate and the following conclusions can be drawn:

- (1) The seismic performance of the post-earthquake temporary prefabricated light-weight steel structure was good, and the steel frame and enclosure walls composed of prefabricated slender columns and prefabricated strip slabs were not damaged when the experimental model was subjected to earthquake with maximum acceleration. In addition, only a lot of cracks appeared on the surface of enclosure walls.
- (2) The natural frequency of the experimental model decreased with increasing maximum acceleration of El-Centro wave, and it dropped sharply when maximum acceleration of the El-Centro wave was equal to 0.4 cm/s^2 .
- (3) When the maximum acceleration of seismic signals was less than 0.4 cm/s^2 , the collaborative performance of the steel frame and enclosure wall was good.

3. Seismic Qualification of Electrical Cabinet Using High-Fidelity Simulation under High Frequency Earthquakes

a. Introduction

An electric cabinet system, is the key facility to support electrical power distribution in power plants and it must be expected to remain functional during and after strong earthquakes. Over the past decade, most components have received less attention in comparison to the structural system regarding their behaviour and design, in spite of it being the cause of economic losses and mechanical damage during a seismic event. Such problems cause a loss of operation and a loss of electrical power in critical facilities, for example, due to overturning of the cabinet system, and component rocking and component sliding by inadequate restrain anchorage in the cabinet system during an earthquake. Consequently, most damage of electrical cabinets was related to structural damage in terms of sliding or overturning and also, the damage was associated with sensitivity due to the amplification of acceleration in the cabinet. The cabinet instrument is related to the fundamental frequency of the components and it is sensitive to the response of acceleration during and after an earthquake.

More specifically, the detailed study of this seismic analysis include the following subjects:

- Identification of the dynamic characteristics such as global and local modes of a single door cabinet on the shaking table test, by using sinusoidal sweep (2 octave/min, 1~50 Hz).
- Evaluation of the dynamic response such as acceleration time histories at the inside and outside of the cabinet, using measurement of accelerometers at the single door on the shaking table test due to ICC-ES AC156.
- Development of a 3D Finite Element (FE) model using the ABAQUS platform to validate the numerical model compared with the responses observed from the shaking table test.
- Analysis of the dynamic response of the single door cabinet system subjected to high and low frequency ground motions, using time history analyses of the 3D FE model.

b. Experimental Test of Electrical Cabinet

The electrical cabinet system is typically constrained by a partially fixed support or fully anchored system and the bracing system in the cabinet is very complicated, in order to mitigate the damage and maintain the functionality of the equipment. In addition, it is quite difficult to describe a complex behaviour and construct a finite element model of the system. Hence, it is necessary to perform a laboratory test such as a shaking table test to understand the complicated behaviour. A prototype of a single door cabinet was selected in this study, for the shaking table test. The width, depth, and height of the specimen were about 800, 800, and 2350 mm, respectively and also the weight of the cabinet was about 480 kg. The cabinet was anchored by eight M16 bolts as a support condition, as shown in Figure 14. The bolts connected onto two channels at the bottom in the specimen were integrated with a jig-plate and then, the jig was anchored onto a shanking table by M24 bolts at 8 positions.

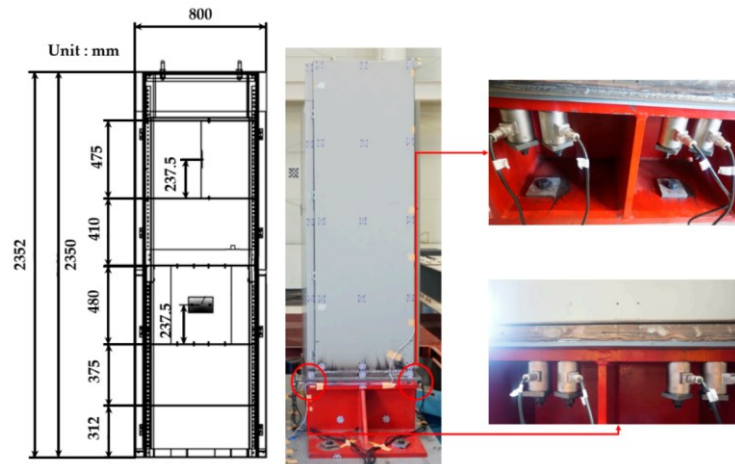


Fig. 14. The prototype cabinet system for an experimental test.

c. Shaking Table Test

A shaking table test of the cabinet was conducted at the Seismic Research and Test Centre, Pusan National University, in which a 6-degrees-of-freedom shaking table can be performed. Before time history tests, a resonance search test carried out for evaluation of the fundamental frequencies and dynamic characteristics of the electrical cabinets. The resonance test was carried out through uni-axial swept sine waves with 2 octave/min and amplitude of 0.07 g, in order to minimize the damage of the cabinet during the test. In addition, it was sequentially performed once within the range of 1 to 50 Hz, corresponding to each orthogonal principal axis, since it had to include the scope of frequency 1.3 to 33.3 Hz during the test. The detailed procedure for the test is shown in Figure 15.

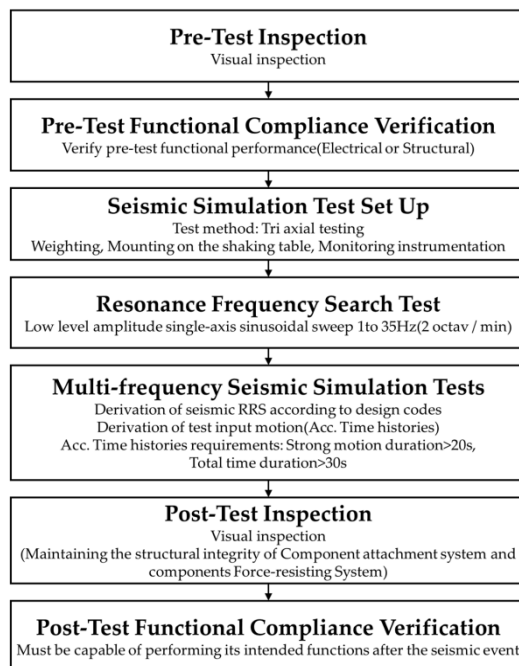


Fig. 15. Procedure of shaking table test for the cabinet system.

d. Finite Element Model of the Cabinet

To understand the more complex behaviour of the single door cabinet, it is necessary to perform a finite element model (FEM) of the electrical cabinet and it is important that the FEM must be reconciled with the data from the experimental test. Often, a simplified model of electrical cabinet system is applied using a lumped mass model with the link element but such a model, while capturing the significant deformation of electrical cabinet subjected to strong ground motions, can be unrealistic. Consequently, this study used a three-dimensional cabinet system to overcome such a weakness, using the ABAQUS platform [13]. In

addition, elastic modulus, density, and Poisson's ratio as the material properties in the cabinet were added appropriately. The three-dimensional FE model of the cabinet system using ABAQUS was demonstrated in Figure 16 and the boundary conditions at the base were implemented to be hinge supports. The shell element (S4R) for frames and panels was used in the FE model. The total number of elements and nodes was 117,655 and 128,198 respectively.

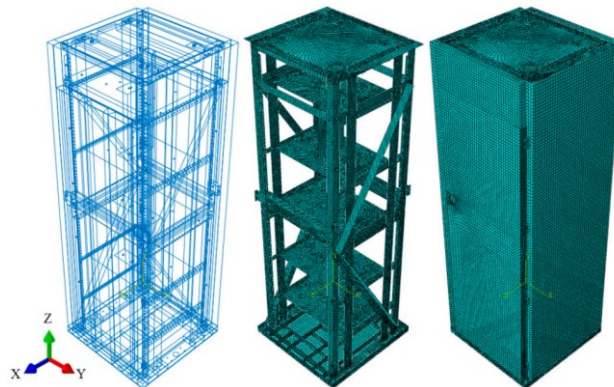


Fig. 16. Detailed FE model of the cabinet system.

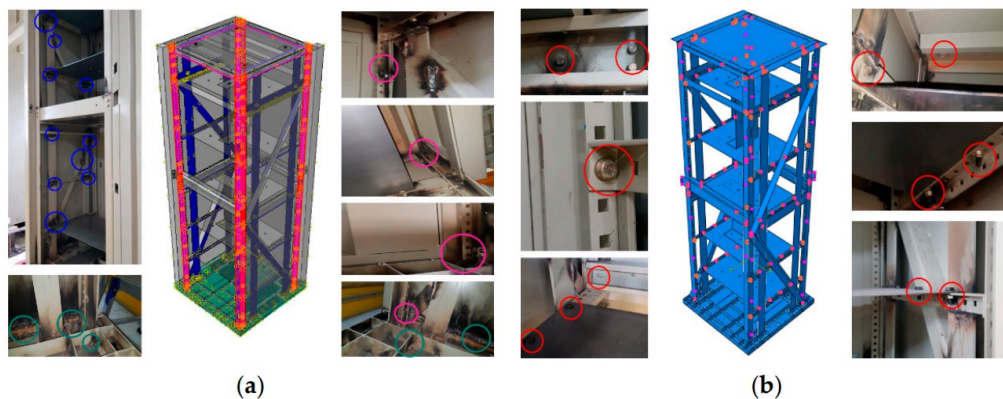


Fig. 17. Location of connection in FE model: (a) Tie, (b) Coupling.

e. Validation of FE Model

Based on the preliminary experimental test, a high-fidelity simulation model with a 5% damping ratio was developed using the shell element in the ABAQUS platform. It was interesting to find that the accurate mode shapes of the cabinet system were able to be demonstrated through the high-fidelity FE model. The fundamental frequencies were also obtained from the resonance test using sine sweep function during the shaking table test. Figure 18 depicted the results obtained from the resonance search tests, and there was no amplification up to 15 Hz among x, y, and z directions. Subsequently, the first amplification occurred at 16 Hz of all accelerometers, which can be determined as the primary global mode of the cabinet.

Furthermore, after the 1st mode, the second targeted global mode was decided at 24 Hz, as shown in Figure 18. In order to reconcile the results of the experimental test with the high-fidelity FE model, modal analysis, corresponding to the target fundamental frequencies, was performed in ABAQUS. The first global mode of the cabinet obtained from the numerical model is shown in Figure 19a, and it was investigated as 16.08 Hz. Besides, Figure 19b describes the second global mode (24.21 Hz) of the system from the modal analysis. It was revealed that the dominated modes were the first and second mode, since mass participation was over 90% at those modes.

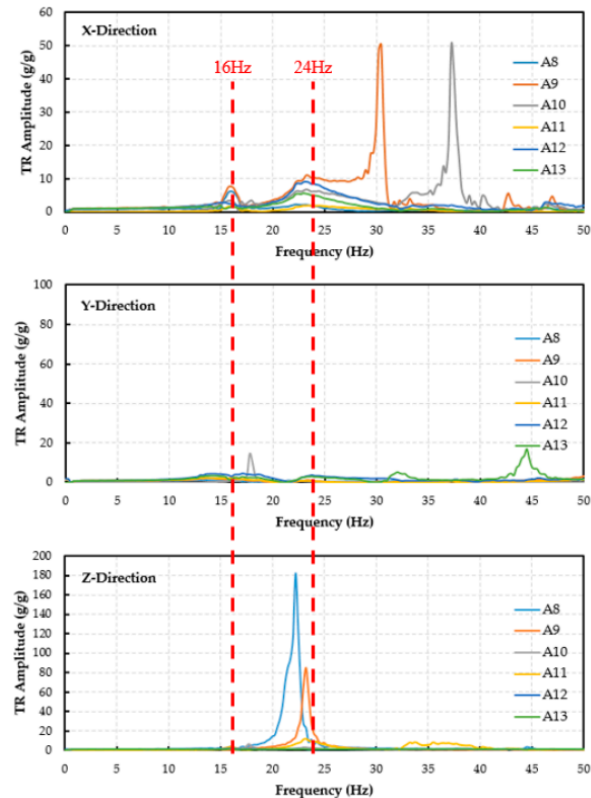


Fig. 18. Resonance data obtained from experimental tests.

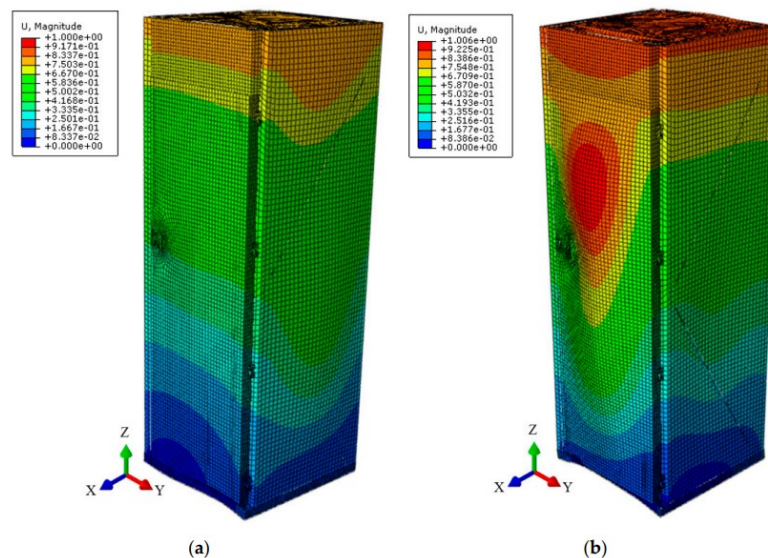


Fig. 19. Mode shapes of the cabinet using high-fidelity FE model: (a) 1st Global Mode Shape (16.08 Hz); (b) 2nd Global Mode Shape (24.21 Hz).

f. Results

Based on the high-fidelity FE model and the different ground motions, time history analyses were conducted in this study. As shown in Figure 20, it was noted that the stress concentration of the cabinet system occurred at the support conditions, more specifically, of the connection between the anchors and the bottom plates. It must be noted that it was quite possible to increase the stress of the cabinet because of the amplification between natural frequency and excitation frequency. In other words, the cabinet system was sensitive to high frequency ground motions.

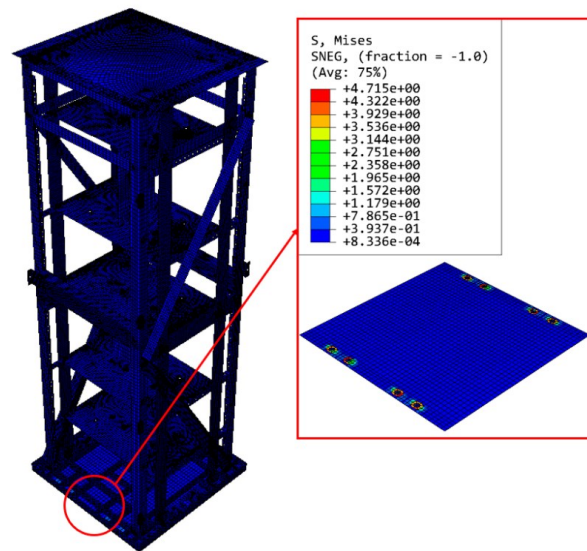


Fig. 20. The results of the cabinet using time history analysis.

In order to support sustainable energy distribution and operation, the cabinet system is one of the essential pieces of electrical equipment in nuclear and non-nuclear power plants. This study presented the seismic qualification of the cabinet system by the observation of the experimental test based on shaking table and the high-fidelity simulation model. The particular focus was on evaluating the dynamic properties of the electrical cabinet under the shaking table test.

VII. CONCLUSION

On the whole, the response spectrum method is the first choice for the seismic theoretical calculation method of the structural enclosures, and the time history analysis method can be used to supplement the seismic design of the structural enclosures with special irregularities or special requirements, and take the larger value between the average of the calculation result of multiple time history curves and the calculation result of the mode decomposition response spectrum method.

Secondly, the static method can be used for rigid components, the bottom shear method can be used for components meeting the application conditions of bottom shear method, and the nonlinear static method should be used for seismic calculation of components requiring elastic-plastic analysis.

However, as the seismic performance of structural enclosures becomes more complex, the existing theoretical calculation methods will not be able to meet the requirements of the development of seismic performance analysis. So, the following will be the future development direction of seismic design analysis theory for structural enclosures:

- (1) Considering the nonlinear characteristics of the structure under earthquake action, when using the time history analysis method, consider the changes in structure quality, stiffness and damping, or when using the response spectrum method, use the elastic-plastic response spectrum instead of the elastic response spectrum to analyse and calculate the structural response under earthquake.
- (2) Research on the correct and reasonable structural simplification method of electrical equipment will not only help reduce the amount of seismic theoretical calculations, but also help expand the application range of seismic theoretical calculation methods.
- (3) Due to the characteristics of large span, wide range of structural enclosures and the influence of the spatial variation characteristics of ground motion (travelling wave effect, coherence effect and local site effect), the seismic response obtained by using non-uniform seismic excitation instead of uniform seismic excitation as the input of vibration equation is more reasonable and closer to reality.

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