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JOINT STIFFNESS OF MAIN TO SUB BEAM CONNECTIONS OF CABLE TRAY

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Abstract

The 2011 Tohoku-Oki Earthquake caused massive amounts of non-structural damage in practically all types of buildings. While few commercial, residential, office, and industrial buildings suffered structural damage, the functionality of many more facilities was disrupted, and significant economic losses were reported due mainly to non-structural damage. The seismic performance of cable trays in modern buildings has been highly valued. its large flexibility, long-span, low structural redundancy and complex geometry are directly related to the seismic characteristics. Owing to its structural characteristics of large flexibility, low redundancy and high load and the functional characteristics of line layout, the damage of cable tray will not only cause the interruption of building functions, but also cause casualties. In order to describe the joint behavior of the main and sub beam in detail, based on the hysteretic test, a semi analytical calculation method of the joint stiffness is proposed. The proposed model is validated by comparison with experimental results of shake table tests. The results show that the rigid joints used in the previous studies are far from meeting the actual calculation accuracy. The joint stiffness proposed in this paper coincides well with the experiment results in the modal analysis, which can be used as an effective calculation method of main and sub beam joint stiffness.

Keywords: cable tray system, joint stiffness, hysteretic test, shake table test

1. Introduction

As a kind of non-structural components, the cable tray system is vital component in modern buildings, which is used to support insulated electric cables for power distribution and communication, thus can be called the "lifeline". Cable trays are used as an alternative to open wiring or electrical conduit systems, and are commonly used for cable management in commercial and industrial construction. They are useful in situations where changes to a wiring system are anticipated, since new cables can be installed by laying them in the tray, instead of pulling them through a pipe. In the case of large-scale facilities, the maximum mass of the cable can reach 100 kg/m at most, and the fallen cables will threaten human lives. Its application in China is only ten years old. Non-structural components were extensively damaged during the 2008 Wenchuan Earthquake in China [1]. However, the collapse of numerous buildings that led to tremendous loss of life in the earthquake drew attention away from non-structural problems. It was not until the 2013 Lushan Earthquake in the same region that the non-structural damage emerged as a major concern to the building research community in China. According to the damage observed of non-structural components in Chile Earthquake in 2010 [2], the pounding of cable tray with the main structure, pipes, ducts, ceilings, fire sprinklers, and other suspended systems caused buckling of the trays, structural damage and cable falling. Similar damages were drawn in the recent Anchorage Earthquake in Alaska [3]. During the 2011 Tohoku-Oki Earthquake, although the city of Tokyo was so well prepared for major earthquakes in terms of structural safety, buildings did not suffer any structural damage, but the non-structural components caused widespread damage in practically all types of buildings. As reported by Masuzawa et al. [4], 68% damage to cables was caused by the failure of cable tray, which led to loss of functions, substantial economic losses, seriously affected the functional recovery after the earthquake, and even brought secondary disasters. This failure model violates the concept of resilient urban development advocated in recent years. Therefore, the use of cable tray in civil buildings and the study of seismic performance of suspension members have attracted major attention.

Cable trays are commonly suspended from the floor or ceiling using threaded rods, cold-formed steel struts or brace (if necessary), and the connectors between main and sub beam are welded or bolted (Fig. 1). Because of the great distresses of nuclear power accidents, the early studies of cable tray focused on the nuclear power plant. In order to avoid radioactive disaster, the possible accidents should be considered in the planning and design stage, it includes fire resistance behaviour [5] and electromagnetic interference among cables [6], etc. In terms of seismic design, its relatively flexibility, long-span, low structural redundancy and complex geometry are directly related to the seismic characteristics. When the cable trays are subjected to strong seismic excitation, structures will vibrate with large amplitudes, and even collapse. Seismic provisions for non-structural components were first introduced in the Chinese seismic design code for buildings and it was left unchanged until today [7]. In this code, an equivalent lateral force method was introduced, which only horizontal seismic force need to be taken into account, it is similar in most national codes [8], but different in specific definitions. Commonly, in actual engineering practice, although widely used in public and commercial buildings, the cable tray systems are rarely subjected to seismic design, and neither in their joints. In 1989 [9], a series of static, dynamic and shake table tests were carried out to study the seismic behaviour and set up analytical model for cable tray systems. Ito et al. [10] investigated influences of cable sliding motions on the seismic responses of cable trays. In the study of damage for cable tray systems. Takeshi et al. [11] utilised viscoelastic rubber damper to reduce acceleration and suppress deformation. Huang et al. [12] carried out shaking table test and obtained the change rule of damping ratio of cable tray with the increase of acceleration. Yang et al. [13] and Hu et al. [14] studied the seismic performance and reliability of cable tray in nuclear power plant using ANSYS. Above studies, the connection performance of joints was not involved, and the default connection mode was fixed. However, the structural behaviour of joints plays a key role in the seismic response of whole structures. Eder and Yanev [15] found that a support connection hardware was missing in 1985 Mexico earthquake, PGA=0.25g. Mcmenanmin [16] reported the fall of suspenders consisted mainly of the creep loosening of suspender bolts. Furthermore, Yao and Lu [17] discussed the continuous collapse caused by the failure of suspender connection. Additionally, Reigles et al. [18] concluded the main and sub beam bolt connections will loose, and even fail under monotonic loading

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and low cycle fatigue test, through a large number of tests. According to the hysteretic tests [19] and shaking table tests, the bolts between the assembly parts did not loose throughout the low cycle fatigue loading (Fig. 2), the connections of main and sub beams were in the semi-rigid state, which were between the complete rigid connection and the ideal hinged connection, and these joints were most fragile (Fig. 3), and the rest parts were not damaged. This phenomenon is similar to the brittle fracture of welded steel connections caused significant damage in the 1994 Northridge Earthquake in the U.S. and 1995 Kobe Earthquake in Japan. From above post-earthquake and experiment results, it is known that the study of connection behavior is of great significance to reflect the vibration characteristics for such structures.

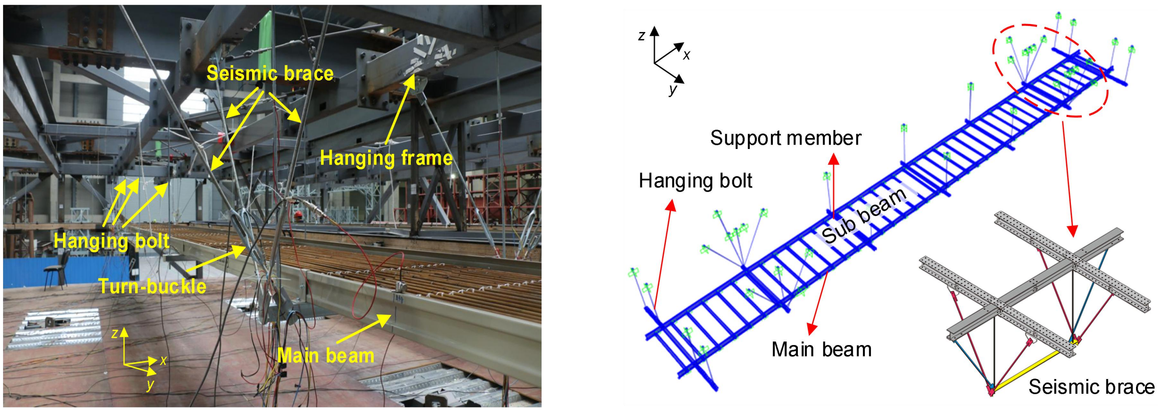

Fig. 1 – Cable tray system

Fig. 2 – Connection observation

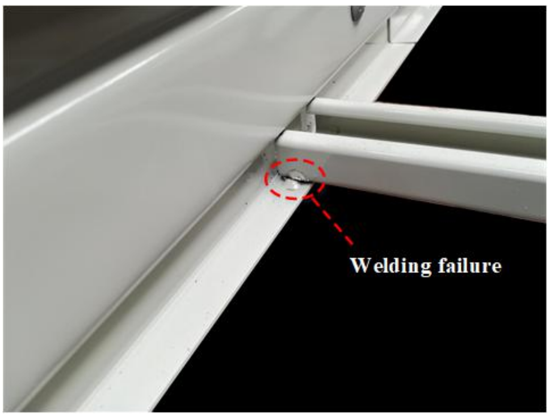

Fig. 3 – Detail of the connection after the failure

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In the following sections, based on phenomenological hysteretic model (moment rotation method), a rotational stiffness calculation method is presented for establishing the backbone curve. To develop and evaluate this calculation method, several comparisons are performed with simulations using rigid joint and experimental tests of shaking table. The results show that this method is appropriate for predicting the seismic response of cable tray systems.

2. Semi-analytical calculation method of joint stiffness

In order to avoid the randomness of the welding joint, the specimen with the 1.1 m main beam, 1m sub beam at 300 mm spacing are taken. The loading position is shown in Fig. 4, and the beam layout, arrangement of displacement sensors and detail dimensions as shown in Fig. 5 (Seki et al., 2016). The specimen is subjected to two cycles with interstory rotational angles of 1/400 (rad), 1/200 (rad), 1/100 (rad), 1/66 (rad), 1/50 (rad), 1/33 (rad), 1/22 (rad) until failure occurred.

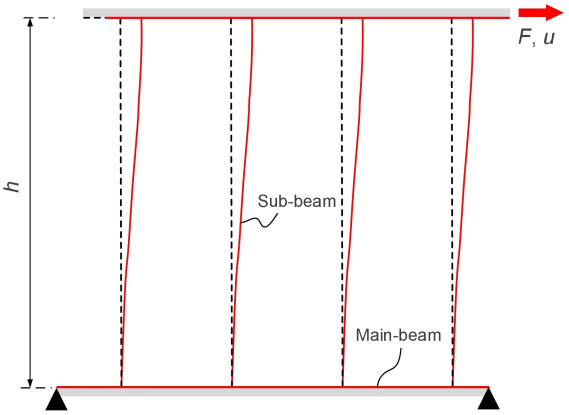

Fig. 4 – Loading diagram

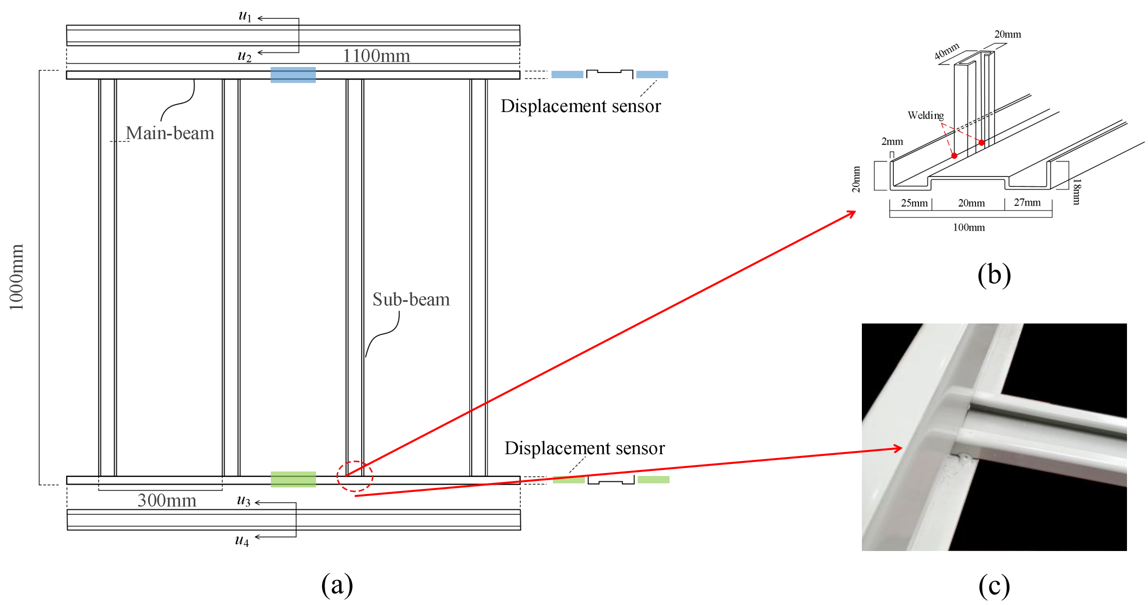

Fig. 5 – Specimen for hysteretic test: (a) beam layout and arrangement of displacement sensors, (b) detail dimensions, and (c) experimental connection

The behaviour of the joint is described by the moment-rotation curve; the moment M is given by:

$$M = \frac{F}{4} h \tag{1}$$

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$$u=\frac{u_1+u_2}{2}-\frac{u_3+u_4}{2} \tag{2}$$

$$\theta=\frac{u}{h} \tag{3}$$

Cyclic moment rotation curves are depicted in Fig. 6.

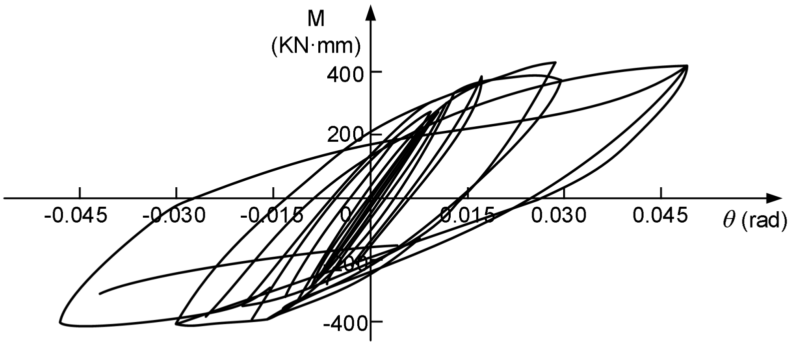

Fig. 6 – Cyclic moment rotation curves

In this test, the indirect measurement method is adopted for the joint rotational angles, and the main factors of the lateral deflection are caused by: a. elastoplastic deformations of the beam itself, b. the shear effects of beams should be taken into account when the transducers are placed near the connection, c. the connect deformation of the joint domain, d. the deformation of their each elemental part [20]. The former two are the actual moment rotation curve in need. Huang et al. [21] put forward the calculation method of semi analytical joint stiffness, Girão et al. [22] deducted the elastic deflection of the beam in the calculation of rotations, but the deformation of the elastic beam is based on the assumption of plane section, and there is error with the real deformation, so Timoshenko first proposed to rectify by introducing the shear coefficient. The shear coefficient is affected by such factors as section form, structural material, boundary condition and action load. At present, the mainstream calculation methods include Timoshenko method, Cowper method, Stephen Hutchinson method, trapezoid block algorithm, material mechanics method, finite element method and elastic mechanics method [23]. In order to facilitate the calculation, the calculation method based on the energy principle proposed by Hu [24] is adopted in this work. The shear coefficient is:

$$\alpha_s=A\iint\left(\frac{\tau}{Q}\right)^2\mathrm{d}A \tag{4}$$

where A denotes the cross-sectional area, τ is the shear stress, Q is the shear force. When a_s=0, the beam is a traditional Bernoulli-Euler beam disregarding the effect of shear effect.

2.1 Semi analytical calculation method of joint stiffness

According to the geometric and connection characteristics of test specimens, which can be simplified as a single rod calculation model of one end semi-rigid fixed support and one end semi-rigid movable support, as shown in Fig. 7. According to the force method, the bending moment diagram $\overline{M}$ (Fig. 7b) of the basic structure under the action of unit force X_1=1 and the bending moment diagram M under the action of load (Fig. 7c), calculate δ_{11} and Δ_1:

$$\delta_{11}=\sum\int\frac{\overline{M}_1\overline{M}_1}{EI}\mathrm{d}x+\sum\frac{\overline{M}_1M_1}{k_\theta}=\frac{l}{EI}+\frac{2}{k_\theta} \tag{5}$$

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$$\Delta_1 = \sum\int \frac{\overline{M}_1 M}{EI}\mathrm{d}x + \frac{\alpha_s Fx}{GA} + \sum \frac{\overline{M}_1 M}{k_\theta} = \frac{-Fl^2}{2EI} + \frac{\alpha_s Fx}{GA} - \frac{Fl}{k_\theta} \tag{6}$$

where EI is the flexural rigidity, k_θ is the rotational stiffness. According to force method equation:

$$\delta_{11}X_1 + \Delta_1 = 0 \tag{7}$$

Unknown force is:

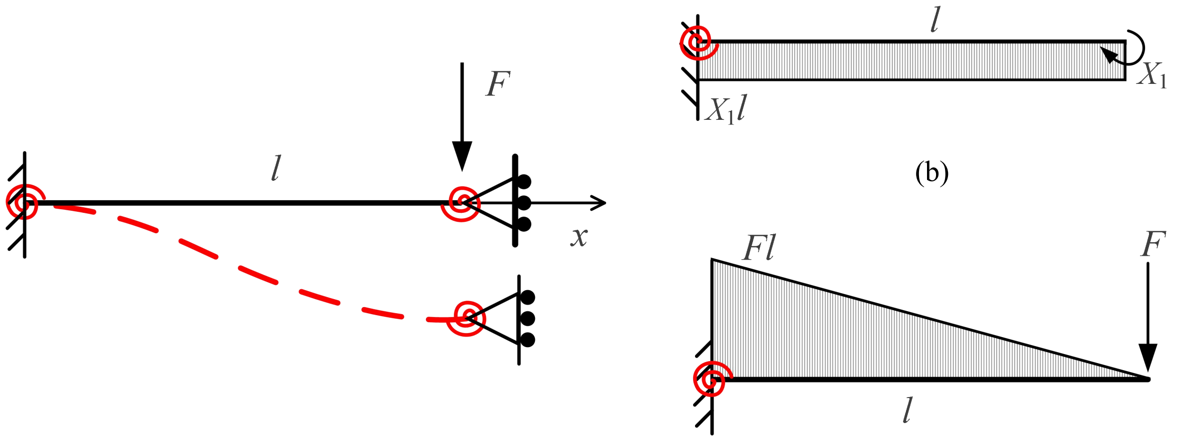
$$X_1(F) = \frac{\frac{Fl^2}{2EI} - \frac{\alpha_s Fl}{GA} + \frac{Fl}{k_\theta}}{\frac{l}{EI} + \frac{2}{k_\theta}} \tag{8}$$

The lateral deflection is:

$$\begin{aligned} u &= \sum\int \frac{\overline{M}M}{EI}\mathrm{d}x + \sum \frac{\overline{M}M}{k_\theta} \\ &= \int_0^l \frac{\left[Fx - X_1(F)\right]\left[x - X_1(1)\right]}{EI}\mathrm{d}x + \frac{\overline{M}(l)M(l)}{k_\theta} + \frac{\overline{M}(0)M(0)}{k_\theta} \\ &= \frac{1}{EI}\left[\frac{Fl^3}{3} - \frac{Fl^2 X_1(1)}{2} - \frac{X_1(F)l^2}{2} + X_1(F)X_1(1)l\right] + \frac{X_1(F)X_1(1)}{k_\theta} + \frac{\left[Fl - X_1(F)\right]\left[l - X_1(1)\right]}{k_\theta} \end{aligned} \tag{9}$$

The rotational stiffness of joints is:

$$k_\theta = \frac{X_1(F)X_1(1) + \left[Fl - X_1(F)\right]\left[l - X_1(1)\right]}{u - \frac{1}{EI}\left[\frac{Fl^3}{3} - \frac{Fl^2 X_1(1) + X_1(F)l^2}{2} + X_1(F)X_1(1)l\right]} \tag{10}$$



(b)

(a)

(c)

Fig. 7 – Calculation diagram

According to Eq. (10) and the measured load F and displacement u, the rotational stiffness k_θ (θ=y, k, t) can be obtained in sections, and the backbone curve of the joint can be established as shown in Fig. 8. The parameters of skeleton curve are presented in Table 1. The first branch of the strength envelope model elastic

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stiffness, second and third branches represent hardening stiffness. The forth bench represents strength degradation, such as from shear failure.

Table 1 – Model parameters

k_0 (KN·mm/rad)	θ_y (rad)	M_y (KN·mm)	θ_k (rad)	M_k (KN·mm)	θ_t (rad)	M_t (KN·mm)	k_t (KN·mm/rad)
19061.55	0.010	190.62	0.015	197.98	0.030	217.91	−698.02

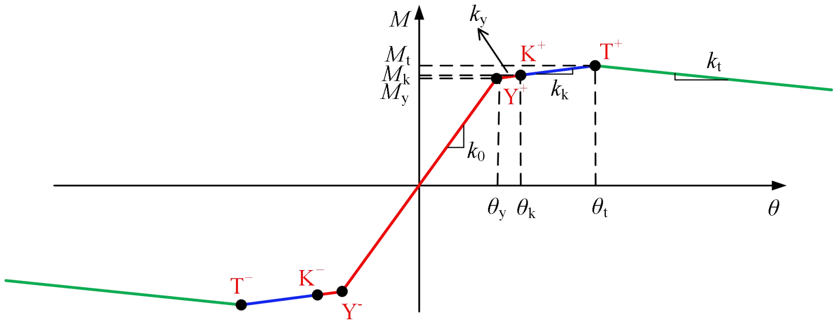

Fig. 8 – Backbone *M*-θ curve

3. Shark table tests

3.1 Test specimen

A full-scale model for cable tray system was studied. The lengths of main beams and threaded rods were 12 m and 1 m, respectively. The suspension rods and seismic bracing were placed at 2 m and 9.4 m distance, respectively. All cables were fastened by nylon band at intervals of 2m, in order to restrain the cable slide. The weight per meter of the cable was 97.2 kg/m^2. Other structural details are consistent with mentioned above. The specimen layout and dimensions of the cable tray are shown in Fig. 9, and displacement and acceleration sensors are installed in the middle of structure.

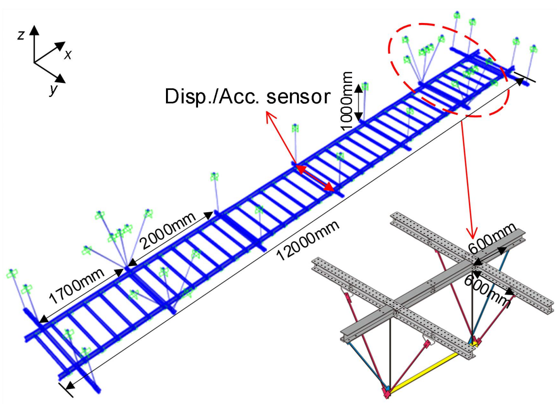

Fig. 9 – Specimen outline for shake table test

3.2 Modal analysis

The structural vibration mode is mainly related to the initial stiffness and load of the structure. Because the backbone curves of each hysteretic model are same, only the rigid joint and CT model are compared here. Through comparing the numerical and tests of vibration frequencies, it can be seen that the participation coefficient of the first three modes accounts for 0.95. According to Table 2, the semi-analytical stiffness

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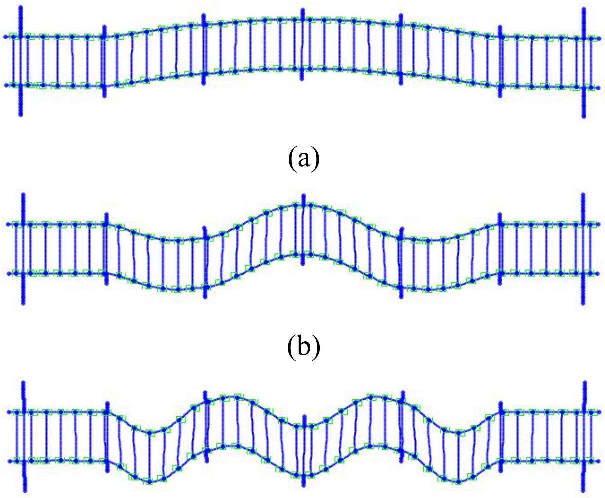
calculation method proposed in this paper can effectively calculate the joint stiffness. The first three modes are shown in Fig. 10.



(a)

(b)

(c)

Fig. 10 – Vibration mode shapes: (a) mode 1, (b) mode 2, and (c) mode 3

Table 2 – Modal analysis

Mode		**1**	**2**	**3**
Participating mass ratios		0.82	0.08	0.05
Frequency (Hz)	Test f_t	2.26	6.80	11.10
	Rigid joint f_f	3.14	8.83	14.89
	CT model f_c	2.37	6.36	11.14
$(f_t - f_f)/f_t \times 100$ (%)		−4.87	6.47	−0.36
$((f_t - f_c)/f_t \times 100$ (%)		−38.94	−29.85	−34.14

4. Conclusions

With the popularization of information technology, cable tray systems are widely employed in nuclear power plants and modern buildings. The post-earthquake damage to non-structural components seriously affects the normal use of building functions. Whether the main building or non-structural components. the yield, fracture and stiffness degradation of joints are all the important factors affecting the structural safety. For the cable tray systems, the failure phenomenon of the main and sub beam joints was found through experiment tests, but the mechanical behavior of the joints was not deeply studied, only adopting rigid joints. Combined with the results of this paper, it can be seen that the rigid connection is difficult to meet the computation accuracy. Therefore, a reasonable description of the mechanical behavior of joints is necessary for the subsequent numerical analysis.

The research work reported in this paper, combining with the hysteretic test of main and sub beam joints, a semi-analytical calculation method of joint stiffness is proposed. To check the effectiveness of calculation method, the vibration mode is discussed with results of shaking table test, and comparing with traditional

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rigid joint. Comparisons gave very satisfactory results. These comparisons confirmed that the numerical results using proposed stiffness are consistent with the experimental results.

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