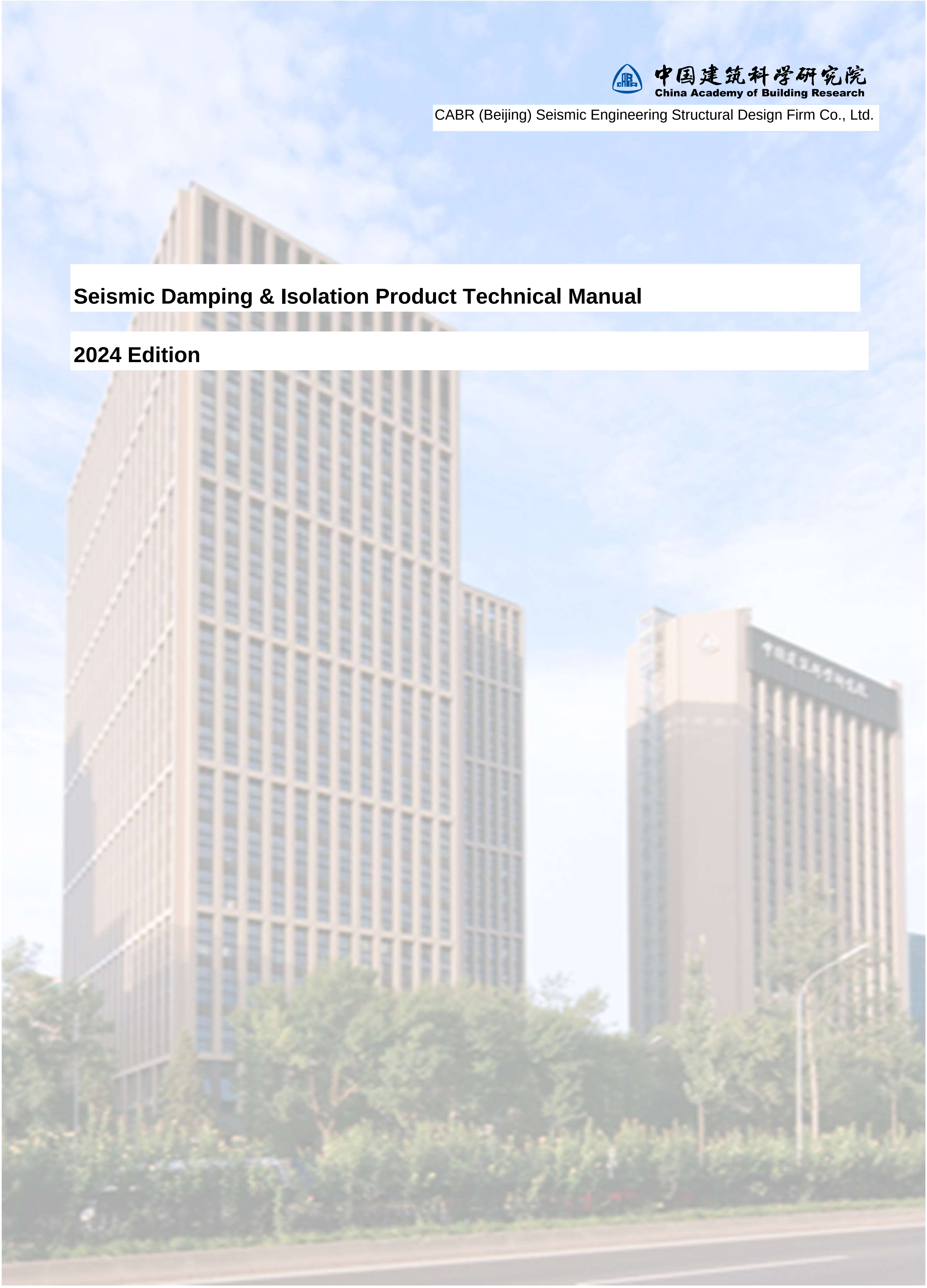
中国建筑科学研究院
China Academy of Building Research

CABR (Beijing) Seismic Engineering Structural Design Firm Co., Ltd.

Seismic Damping & Isolation Product Technical Manual

2024 Edition



Contents

Part 1 General

1 Overview

1.1 Concept of energy-dissipation damping

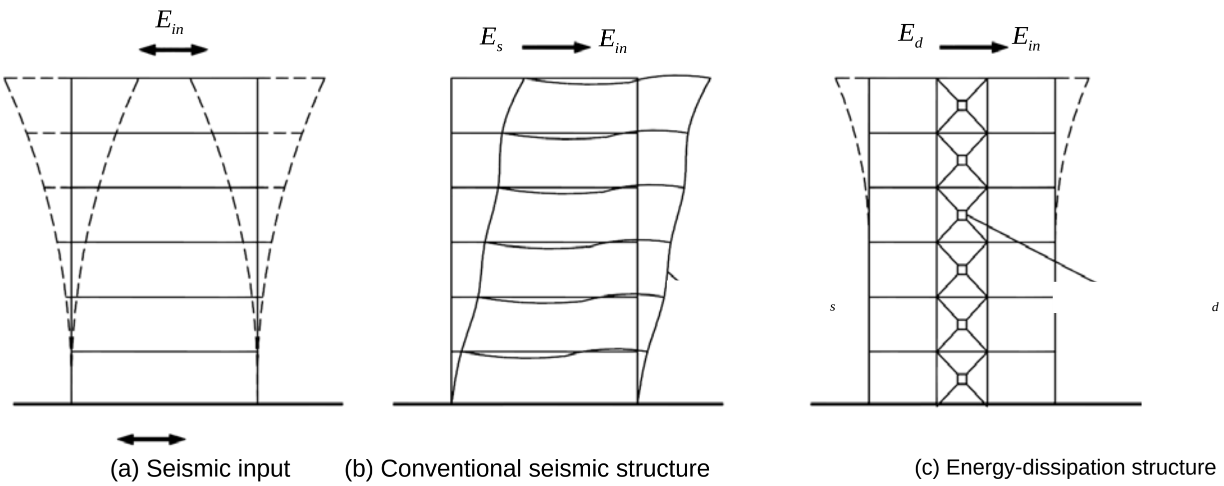
Structural energy-dissipation damping mainly refers to installing energy-dissipation devices at certain parts of a structure - such as inter-story gaps, joint connections or connection seams - or configuring secondary members such as braces, connectors or non-load-bearing shear walls as energy-dissipating members. When an earthquake strikes, these devices or members provide large damping through friction, plastic deformation, viscous fluid flow and so on, dissipating the energy input by the ground motion and reducing the seismic response of the main structure, thereby protecting its safety.

Unlike the traditional approach of resisting earthquakes by enlarging member sections, energy-dissipation damping reduces the seismic response of the main structure by having energy-dissipation members absorb and dissipate seismic energy - another powerful means of seismic protection. Among these methods, the passive approach of adding dampers to increase structural damping is favoured in the industry for its clear force transmission, easy installation and maintenance, low cost and wide applicability.

1.2 Principle of energy-dissipation damping

The essence of structural energy-dissipation damping is to install dampers in the structure. During an earthquake the input energy is absorbed first by the dampers, dissipating much of the seismic energy and effectively attenuating the structural response. The dampers provide supplementary damping during the earthquake.

The principle of energy-dissipation damping can be described from an energy standpoint, as shown in Figure 1.2.1.



(a) Seismic input (b) Conventional seismic structure (c) Energy-dissipation structure

Figure 1.2.1 Schematic of the energy-dissipation damping principle

The energy equation of the structure at any instant during an earthquake is:

Conventional structure: $$E_{in} = E_v + E_c + E_k + E_h \quad (1.1.1)$$

Energy-dissipation structure: $$E_{in}^{'} = E_v^{'} + E_c^{'} + E_k^{'} + E_h^{'} + E_d \quad (1.1.2)$$

$E_{in}^{'}$ —— energy input to the conventional / energy-dissipation system during the earthquake;

$E_{v}^{'}$ —— kinetic energy of the conventional / energy-dissipation system;

$E_{c}^{'}$ —— viscous-damping dissipation of the conventional / energy-dissipation system;

$E_{k}^{'}$ —— elastic strain energy of the conventional / energy-dissipation system;

$E_{h}^{'}$ —— hysteretic dissipation of the conventional / energy-dissipation system;

E_{d} —— energy dissipated or absorbed by the dampers.

In energy equations 1.1.1 and 1.1.2, Ev and Ev', Ek and Ek' only undergo energy conversion and do not dissipate energy, while Ec and Ec' account for only a small share of the total energy (about 5%) and are negligible. Thus in a conventional seismic structure, seismic energy is mainly dissipated by Eh - i.e. by the elastic-plastic deformation of structural members, which themselves suffer damage or even failure; the more energy a member dissipates, the more severely it is damaged. In an energy-dissipation system, the dampers placed in the structure dissipate energy before the main structure enters elastic-plastic deformation, greatly reducing the seismic response and effectively protecting the main structure.

1.3 Advantages of energy-dissipation damping

Compared with a conventional structural system, an energy-dissipation system offers the following advantages:

1) Safety

A conventional seismic system essentially uses the main structure (beams, columns, walls) as energy-dissipating members. Under conventional seismic design, structural members are allowed to suffer varying degrees of damage during an earthquake. Because earthquakes are unpredictable, the extent of this damage is hard to control - especially under strong earthquakes exceeding the fortification intensity, where structural safety is even harder to ensure.

Because an energy-dissipation structure is fitted with non-load-bearing dampers that have large energy-dissipation capacity, these dissipate energy first under a strong earthquake, absorbing the input seismic energy, attenuating the structural response and protecting the main structure from damage - thereby ensuring safety under strong earthquakes.

Shaking-table tests of energy-dissipation structures in China and abroad show that, compared with conventional seismic structures, the seismic response is reduced by 40-60%.

2) Economy

A conventional seismic system resists earthquakes by "brute force" - strengthening the structure, enlarging member sections and adding reinforcement to raise seismic performance, which markedly increases cost.

An energy-dissipation system reduces seismic response through "flexible energy dissipation", allowing smaller member sections and less reinforcement while actually improving seismic performance. Engineering experience shows that, compared with a conventional system, an energy-dissipation system can save 5%-10% of construction cost; for the retrofit of existing structures the savings are even greater.

2 Introduction to dampers

A damper (also called an energy dissipator) works essentially by dissipating energy. By working principle, dampers are classified as displacement-dependent and velocity-dependent. Displacement-dependent dampers mainly include metal-yielding dampers and friction dampers; velocity-dependent dampers mainly include viscoelastic dampers and viscous dampers.

2.1 Displacement-dependent dampers

2.1.1 Metal dampers

Metal dampers absorb and dissipate energy through the plastic yielding of metal. Their hysteresis reflects the hysteretic behaviour of the metal, with a hysteresis loop that is generally rectangular or a parallelogram with some post-yield stiffness. The common restoring-force model is the bilinear model (see Figure 2.1.1).

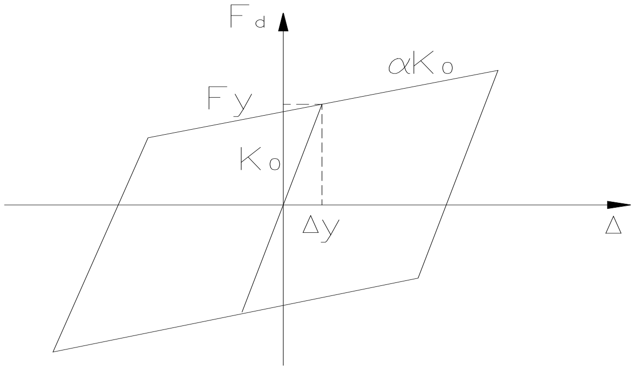


Figure 2.1.1 Bilinear model - damping-force vs. displacement skeleton curve

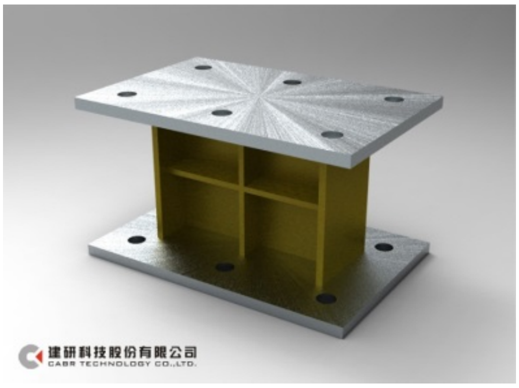
$$K_s = \begin{cases} K_0, F_d = K_0 x & \text{before yield} \\ K_1, F_d = F_y + K_1(x - x_y) & \text{after yield} \end{cases}$$

K_0 —— pre-yield stiffness of the metal;

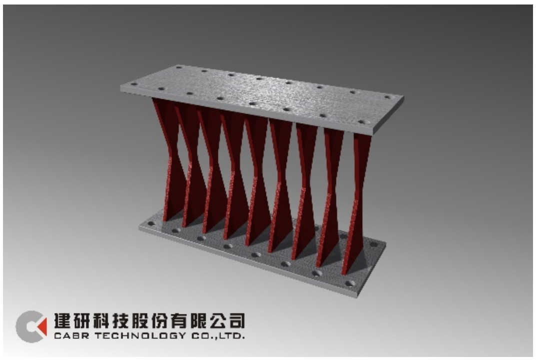
K_1 —— post-yield stiffness of the metal;

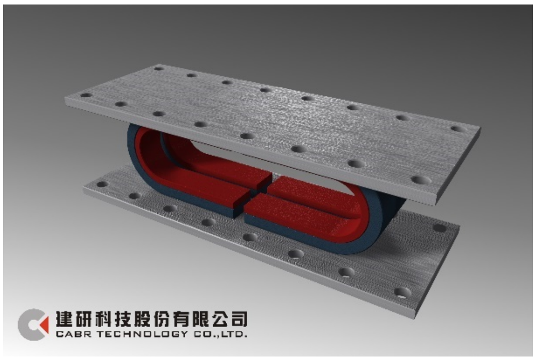
x —— relative displacement between the two ends of the metal damper.

There are four main types of metal damper: buckling-restrained brace, shear-type damper, bending-type damper and U-type damper.



(1) Buckling-restrained brace (2) Shear-type damper

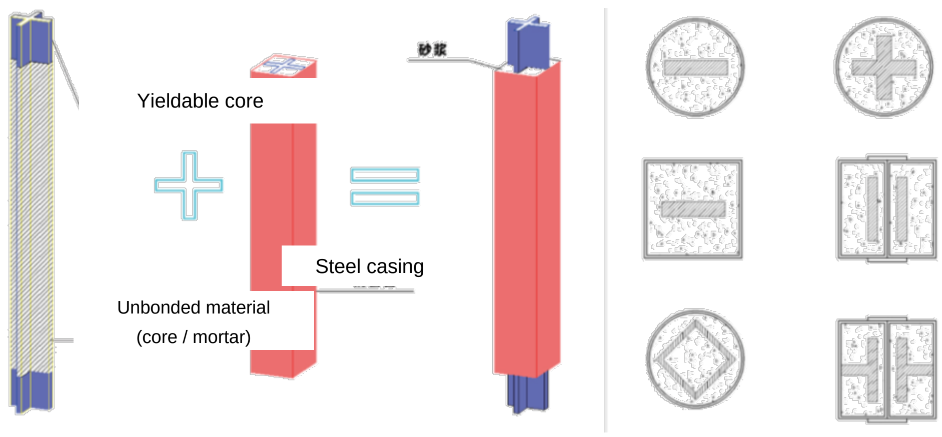




(3) Bending-type damper (4) U-type damper

Figure 2.1.2 Metal dampers

A buckling-restrained brace consists of a core, an unbonded filler and a restraining outer casing (Figure 2.1.3). Only the core plate connects to other members and carries all the load; the casing and unbonded filler merely restrain the core against compressive buckling, so the core can yield in both tension and compression - giving excellent hysteretic performance (Figure 2.1.4).



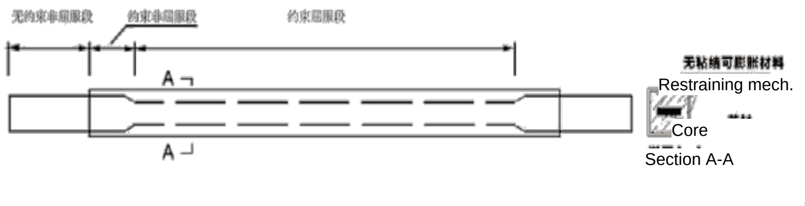


Figure 2.1.3　Construction of a buckling-restrained brace

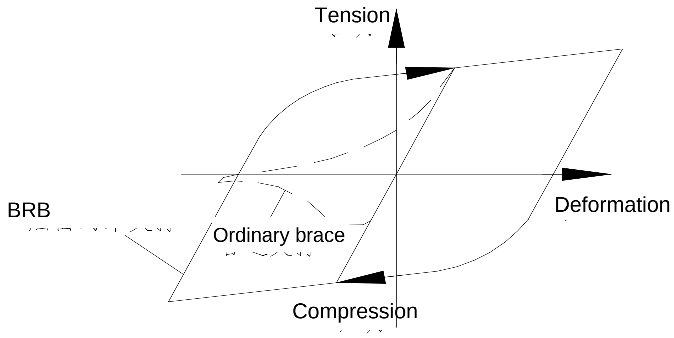


Figure 2.1.4　Performance comparison: buckling-restrained brace vs. conventional concentric brace

A shear-type damper dissipates energy through shear deformation of its web; to prevent buckling instability of the web under shear, anti-buckling detailing is provided so the material yields and dissipates energy.

A bending-type damper is formed of several identically shaped steel plates and dissipates energy through the lateral bending yield of the plates.

2.1.2　Friction dampers

Friction dampers emerged in the late 1970s. With technical development they now take many forms and the slip force is no longer fixed. Common friction dampers include the ordinary friction damper, the Pall friction damper, the centripetal variable-friction damper and the T-shaped core-plate friction damper.

A conventional friction damper has two states - non-slip and slip - and its hysteresis loop is rectangular under cyclic loading. Under the Coulomb assumption, when the relative sliding velocity of the friction contact bodies is small, the total sliding friction depends only on the contact-surface condition and the applied normal force. The friction can therefore be computed with a rigid-plastic model (damping-force vs. displacement skeleton curve, Figure 2.1.5):

$$\begin{cases} K_s = K_0 = \infty & \text{before slip} \\ K_s = K_1 = 0, F_d = -\mu N \operatorname{sgn}(x) & \text{after slip} \end{cases}$$

where:

F_d ——output force of the damper;

N ——normal pressure;

μ ——friction coefficient;

x ——relative displacement between the two ends of the damper.

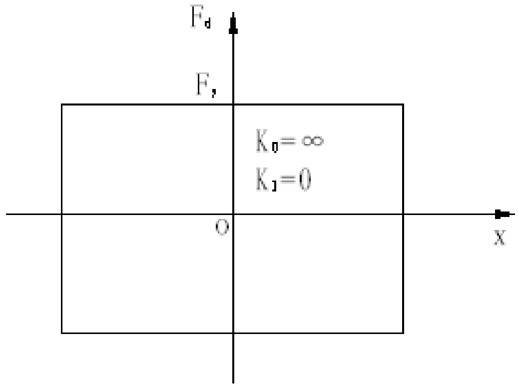


Figure 2.1.5 Rigid-plastic damping-force vs. displacement skeleton curve

The core components of an ordinary friction damper are shown in Figure 2.1.6. Energy is dissipated by the friction between a central steel plate with a slotted hole and the upper and lower copper pads; adjusting the bolt tightening force changes the sliding friction. The sliding friction is proportional to the bolt force; moreover, the difference between the maximum static and the sliding friction at the steel-copper interface is small and the sliding friction decays little, ensuring stable operation of the friction-dissipation system.

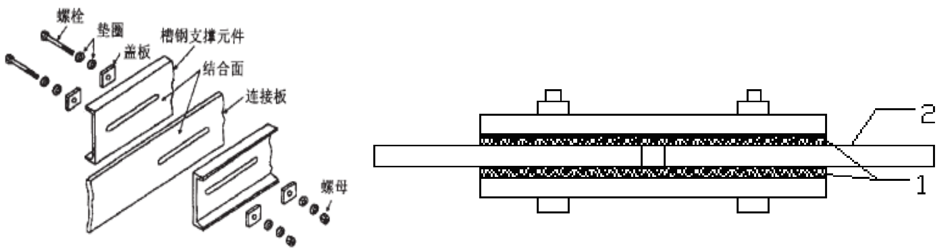


Figure 2.1.6 Construction of an ordinary friction damper

2.2 Velocity-dependent dampers

Velocity-dependent dampers mainly include viscous dampers and viscoelastic dampers.

2.2.1 Viscous dampers

The stress-strain relationship of a viscous damper depends greatly on its construction and varies with different researchers' methods. It is generally agreed that the damping force of a viscous damper is governed mainly by velocity. The common model is the Maxwell model, which represents the damper as an elastic element in series with a damping element (Figure 2.2.1).

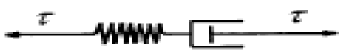


Figure 2.2.1 Maxwell model

The output force of a viscous damper is calculated as:

$$F_d = c_d V^m$$

F_d ——damping force of the viscous damper;

V ——velocity of the piston;

c_d ——a constant related to the cylinder diameter, piston diameter, guide-rod diameter, fluid viscosity, etc.;

m ——velocity exponent - 1 for a linear viscous damper, usually 0.2-1 for a nonlinear viscous damper.

By construction, viscous dampers can be classified as cylinder-type viscous dampers (Figure 2.2.2), viscous damping walls (Figure 2.2.3), three-directional viscous dampers (Figure 2.2.4) and combined dampers. Common viscous fluids are methyl silicone oil, silicone gel and hydraulic oil. The research and production technology of cylinder-type dampers is relatively mature and they are widely used in engineering.

2.2.

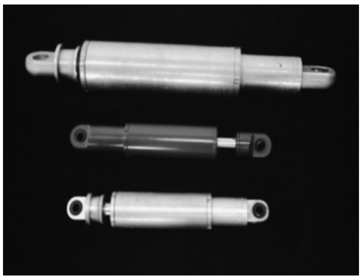


Fig. 2.2.2 Cylinder-type viscous damper

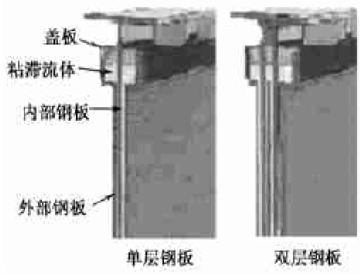


Fig. 2.2.3 Viscous damping wall

Fig. 2.2.4 Three-directional viscous damper

The construction of a cylinder-type viscous damper largely follows that of a mechanical hydraulic cylinder, and its cylinder and piston dimensions have been standardized into series. By construction it can be a single-rod fluid damper, a double-rod fluid damper or a clearance-type fluid damper. A cylinder-type damper generally comprises a cylinder, guide rod, piston, damping orifice and viscous damping fluid (Figure 2.2.5). The piston reciprocates inside the fluid-filled cylinder; the orifices in the piston act as damping holes. When the piston moves relative to the cylinder wall, the pressure difference forces the fluid through the orifices, generating the damping force.

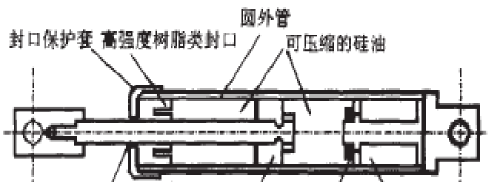


Figure 2.2.5 Construction of a cylinder-type viscous damper

Viscous dampers have been studied systematically and produced industrially in the USA, France and Japan. Corresponding research in China started later but has developed rapidly.

2.2.2 Viscoelastic dampers

A viscoelastic damper dissipates seismic energy through the hysteretic behaviour of a viscoelastic material. Its main advantage is that it provides both damping and stiffness simultaneously, with no obvious working-range limit, dissipating energy under both major and minor earthquakes. The damping force is usually proportional to (or a power of) velocity, and proportional or nearly proportional to displacement.

A viscoelastic damper (Figure 2.2.6) usually consists of a viscoelastic material and restraining materials such as steel plates, with the viscoelastic material bonded to the steel by vulcanization. The restraining materials raise the elastic modulus and deformation capacity and reduce temperature effects. The viscoelastic material is generally a copolymer or glassy substance such as methyl silicone oil, silicone gel or hydraulic oil. By load behaviour, dampers are classed as tension-compression type and shear type, with the shear type being more common.

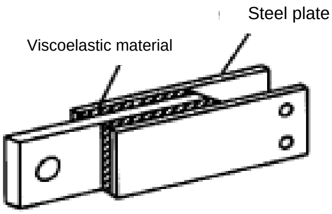


Figure 2.2.6   Viscoelastic damper

2.3   Damper testing requirements

1. For displacement-type dampers, the sample size shall be no less than 3% of the same type and specification in the same project; when the quantity is small, 3% of the total of that type may be sampled, but not fewer than 2. With a 100% pass rate, that batch may be used in the main structure. Tested dampers may not be used in the main structure.

2. For viscous dampers, the sample size shall be no less than 20% of the same type and specification in the same project, and not fewer than 2. With a 100% pass rate, that batch may be used in the main structure; after passing, a damper may be used in the main structure if it has no damage and still meets normal-use mechanical-performance requirements.

3. For viscoelastic dampers, the sample size shall be no less than 3% of the same type and specification in the same project; when the quantity is small, 3% of the total of that type may be sampled, but not fewer than 2. With a 100% pass rate, that batch may be used in the main structure. Tested dampers may not be used in the main structure.

4. If the product pass rate is below 100%, double the sample size from the same batch. If the doubled sample reaches 100%, the batch may be used in the main structure; if the doubled sample still does not reach 100%, that batch may not be used in the main structure.

3   Calculation of additional damping ratio

Under frequent earthquakes the main body of an energy-dissipation structure remains elastic, and the hysteretic dissipation of the dampers provides additional damping. The Code for Seismic Design of Buildings (GB 50011-2010) gives response-spectrum curves for different damping ratios, from which the seismic response of the energy-dissipation structure can be computed using the high-damping spectrum. Determining the additional damping ratio of the dampers is

There are three main methods for calculating the additional damping ratio of dampers, to be selected according to circumstances.

3.1 Code method

China's seismic code and the US NEHRP seismic design provisions (FEMA 450, ATC-33) give an additional-damping-ratio method based on deformation energy (the "code method"). The code-method formula for the additional damping ratio is:

3.1 Code method

$$\xi_a = \sum_j W_{cj} / (4\pi W_s)$$

W_{cj}——energy dissipated by damper j in one full cycle at the expected displacement;

W_s——total strain energy of the energy-dissipation structure at the expected displacement.

Studies show that when the additional damping ratio is small, the code method has very little error compared with exact methods such as the complex-mode method, meeting engineering-design accuracy. In practice, however, the code method has two difficulties:

First, iterating the expected displacement of the energy-dissipation structure. The formula requires the structural deformation, which is itself affected by the additional damping ratio; multiple iterations are usually needed to determine both the deformation and the additional damping ratio.

Second, estimating the actual damper deformation. In ordinary frame structures the damper deformation is proportional to the structural deformation and can be estimated from the story drift. In frame-shear-wall (flexure-shear) structures, as the number of stories increases, the proportion of non-working story drift in the total story drift grows. This non-working story drift, caused by the overall flexural deformation of the lower stories, produces upper-story drift without deforming the dampers. Hence the actual damper deformation in flexure-shear structures cannot be easily derived from the story drift.

3.2 Seismic response reduction coefficient method

By analogy with isolation design, the seismic story shears of the energy-dissipation and the non-energy-dissipation structures are calculated separately, and the maximum of the story-shear ratios is taken as the damping coefficient. After obtaining the reduced seismic-influence coefficient, the total damping ratio of the energy-dissipation structure is back-calculated from the code spectrum using the fundamental period. However, because the spectrum reduced by the damping coefficient differs greatly from that reduced by the additional damping ratio, structural analysis using the damping ratio back-calculated this way may have large deviations.

3.3 Free-vibration decay method

Treating the structure as an equivalent single-degree-of-freedom system, free-vibration decay theory relates the amplitudes of m adjacent periods to the damping ratio per Eq. (3.3.1); when the damping ratio is small, Eq. (3.3.3) may be used approximately (the error is 2% at a damping ratio of 0.2).

$$\xi = \frac{\delta_m}{2\pi} \qquad (3.3.1)$$

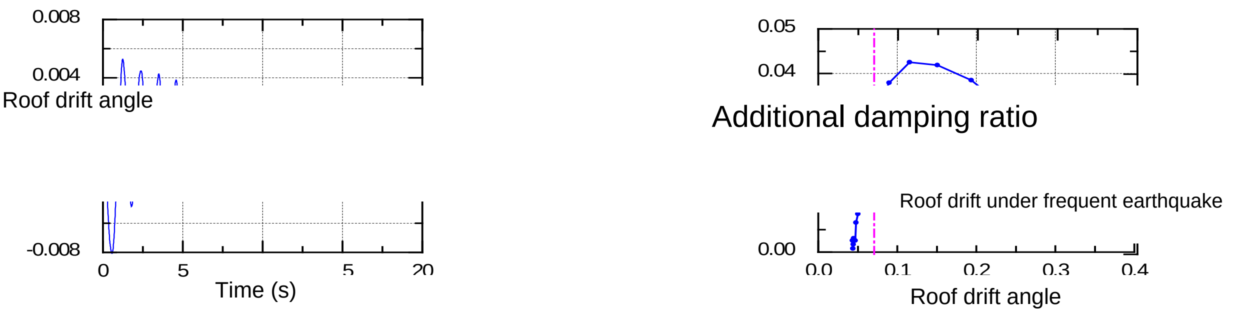
$$\delta_m = \ln(s_n / s_{n+m}) \qquad (3.3.2)$$

$$_D = \omega\sqrt{1-\xi^2} \approx \omega$$

$$\xi = \frac{\delta_m}{2\pi} \quad \frac{\delta_m}{2\pi\ m} \qquad (3.3.3)$$

where m is the number of periods between the two amplitudes, delta is the logarithmic decrement, and omega and omega_D are the natural frequencies of the undamped and damped vibration respectively.

Set the inherent damping ratio of the energy-dissipation structure to 0, apply an instantaneous excitation to make it vibrate freely, and use nonlinear analysis to obtain the free-vibration decay time-history of the roof displacement, as shown in Figure 3.3.1. Substituting the amplitude of each cycle into Eq. (3.3.3) gives the relationship between the additional damping ratio and the roof amplitude, as shown in Figure 3.3.1. From the structural target deformation, the damping at that amplitude - i.e. the additional damping ratio of the dampers - is determined.



a) Roof-displacement time-history　　b) Additional damping ratio vs. roof displacement

Figure 3.3.1　Schematic of the free-vibration decay calculation

Part 2 Buckling-Restrained Braces

1 Basic principles

Brace members are widely used as lateral-force-resisting members in structures. Their main lateral-resisting mechanism converts the frame's lateral resistance into overall tension-compression of the columns at the two ends of the brace, greatly increasing the lateral stiffness.

Under cyclic earthquake action, brace members carry alternating tension-compression loads of roughly equal magnitude. Because an ordinary brace generally has a not-very-small slenderness ratio, it has a stability problem in compression, and its compressive capacity is markedly lower than its tensile capacity. For a brace with a slenderness ratio of about 100, the compressive capacity is only half the tensile capacity. Hence, when designing ordinary steel braces, compression usually controls the section design. Meeting the compressive capacity requires a large section, which needlessly raises the structural stiffness and thus increases the seismic force.

A buckling-restrained brace (BRB/UBB) consists of a core, an unbonded filler and a restraining outer casing (Figure 1.1.1). Only the core plate connects to other members and carries all the load; the casing and unbonded filler merely restrain the core against compressive buckling, so the core yields in both tension and compression, giving excellent hysteretic performance (Figure 1.1.3). The BRB thus avoids the marked tension-compression capacity gap of an ordinary brace while providing the energy-dissipation capacity of a metal damper, acting as a "fuse" that protects the main structure.

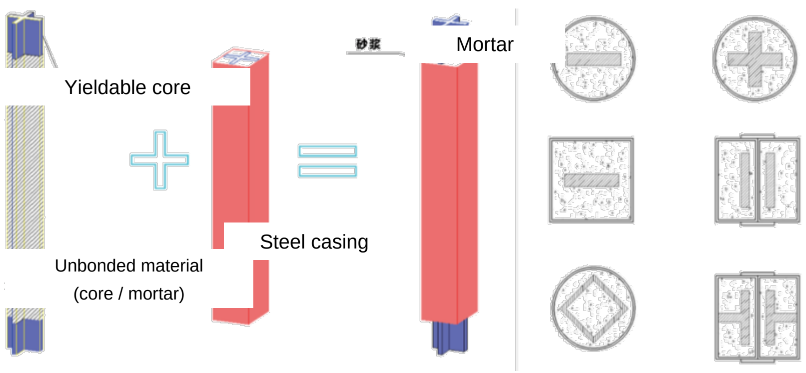


Figure 1.1.1 Working principle of the buckling-restrained brace

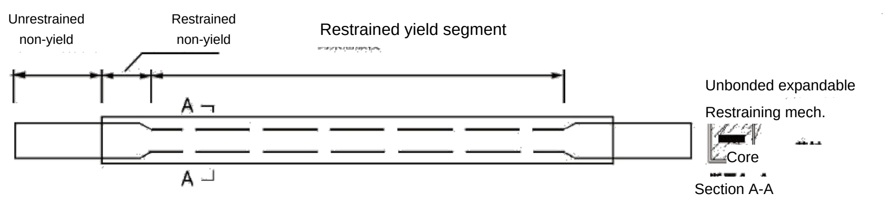


Figure 1.1.2 Construction of the buckling-restrained brace

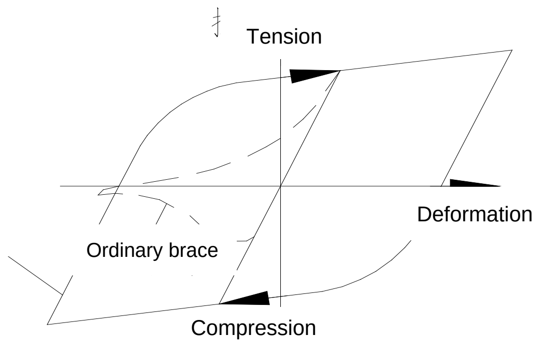


Figure 1.1.3 Performance comparison: buckling-restrained brace vs. conventional concentric brace

Because the buckling-restrained brace has no compressive stability problem, its member capacity under wind and minor earthquakes is 2-10 times that of an ordinary brace, and the longer the member the greater the increase. For the same capacity, its section can be greatly reduced compared with an ordinary brace, so the structure's lateral stiffness softens, its period lengthens accordingly, and the seismic action decreases.

A comparison of the seismic performance of a BRB frame and an ordinary braced frame is given in Table 1.1.1.

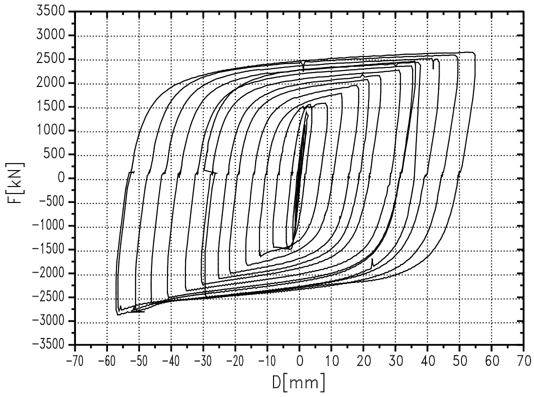
Table 1.1.1 Seismic performance comparison: BRB frame vs. ordinary braced frame

State	Conventional braced frame		BRB frame	
	Main structure	Ordinary brace	Main structure	BRB
Minor quake	Elastic	Elastic	Elastic	Elastic
Moderate quake	Elastic/plastic	Elastic/buckling	Elastic	Plastic (E.D.)
Major quake	Plastic	Buckling	Elastic/plastic	Plastic (E.D.)
After mod./major quake	Remove damaged parts; affects use		Inspect/replace BRBs; use unaffected	

2 Product performance and acceptance criteria

2.1 Features of the JY-SD buckling-restrained brace

The JY-SD buckling-restrained brace embodies many years of CABR research on energy-dissipation damping. Developed through extensive testing, it is an energy-dissipating member with proprietary intellectual property whose performance reaches the advanced international level for similar products.



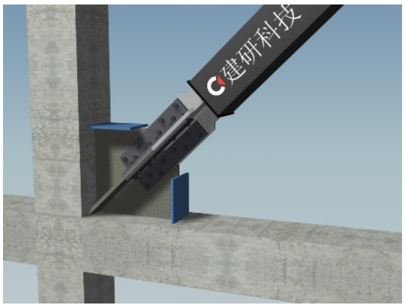
JY-SD buckling-restrained brace   Hysteretic behaviour of the JY-SD buckling-restrained brace

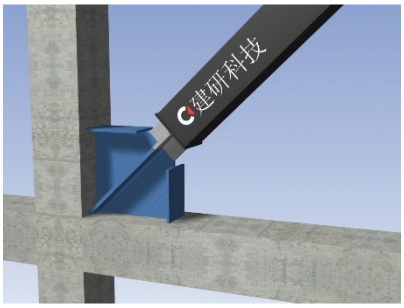
Its main features are:

1) Stable hysteretic behaviour and large deformation capacity, up to 2%-5%;

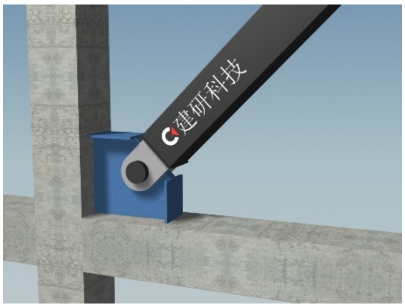
2) Meets all requirements for displacement-dependent dampers in the Code for Seismic Design of Buildings (GB 50011-2010);

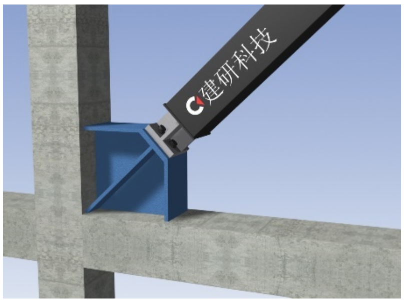
3) Several connection types are available: bolted, pin, flange and welded.





Bolted connection   Welded connection





Pin connection   Flange connection

Figure 2.1.1 Connection types of the JY-SD buckling-restrained brace

2.2 Product testing and acceptance criteria

Within a single project, buckling-restrained braces shall be sampled and tested by category according to the brace's construction form, restrained-yield-segment material and yield capacity.

Braces with the same construction form and restrained-yield-segment material whose yield capacities lie within 50%-150% of one another are grouped into the same category.

The sample size for each category is 3% and not fewer than 2 braces; the pass rate shall be 100%, and tested braces may not be used in the main structure.

The test shall apply 3 cycles of tension-compression deformation at each of 1/300, 1/200, 1/150 and 1/100 of the brace length in turn. The hysteresis curves obtained shall be stable and full with positive incremental stiffness; the capacity in the 3rd cycle of the final deformation level shall be no less than 85% of the maximum capacity experienced, and the maximum capacity experienced shall be no more than 1.1 times the calculated ultimate capacity of the brace. When used as a damper, after 30 further cycles at the maximum design displacement amplitude, the error and decay of the brace's main design indices shall not exceed 15%, and there shall be no obvious low-cycle fatigue.

3 Design method

3.1 Overall design method

The overall design of a BRB structure can be summarised as two-stage design with three capacity checks. The two stages are the elastic-design stage and the elastic-plastic-design stage; the three capacity checks are the design-capacity, yield-capacity and ultimate-capacity checks.

In the elastic-design stage, the design of a BRB-frame system is basically the same as that of an ordinary braced-frame system, but differs in member checking and connection design. In member checking, a BRB needs no stability check, only a strength check, whereas the stability check is the most important check for an ordinary steel brace. To ensure the BRB's energy-dissipation capacity, its connection capacity shall be no less than 1.1 times the brace's ultimate capacity;

In the plastic-design stage, if dynamic time-history analysis is used, the BRB hysteresis can be modelled with a simple bilinear model (Figure 3.1.1), whereas an ordinary steel brace requires a tension-compression-asymmetric hysteresis model (Figure 3.1.2).

3.1.

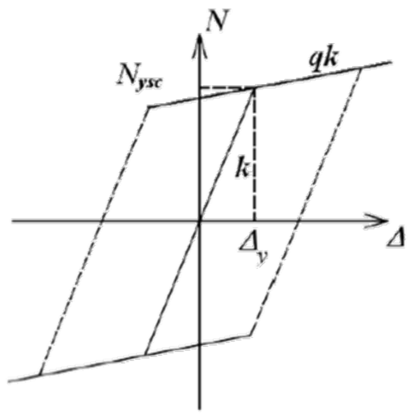


Figure 3.1.1 BRB hysteresis model

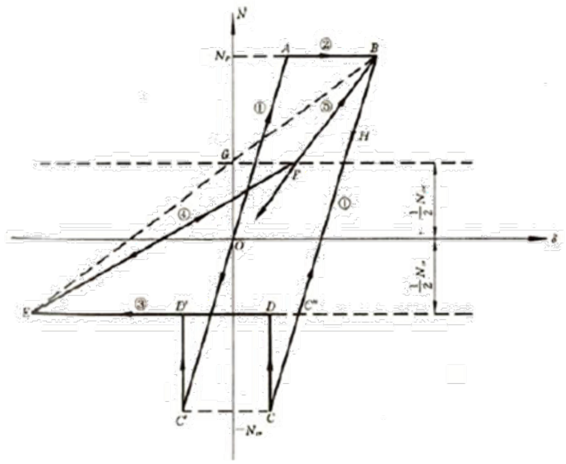


Figure 3.1.2 Ordinary steel-brace hysteresis model

Thus, in design terms, the BRB is simpler than an ordinary brace, avoiding the stability check and the tension-compression-asymmetric hysteresis model.

3.2 Equivalent cross-sectional area of the brace

A buckling-restrained brace consists of a restrained yield segment, a restrained non-yield segment and an unrestrained non-yield segment. Its elastic stiffness is directly related to the lengths and cross-sectional areas of these three parts; its equivalent linear stiffness is the series stiffness of the three springs (Figure 3.2.1).

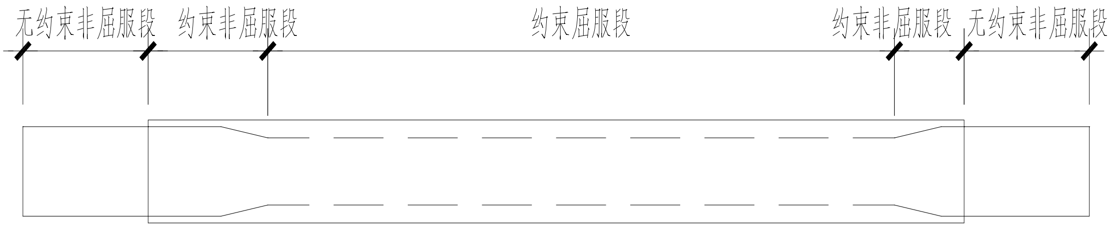


Figure 3.2.1 Schematic of the BRB core

Assuming the three segments have linear spring stiffnesses k1, k2, k3 respectively, the equivalent spring stiffness of the BRB is:

$$\frac{1}{k}=\frac{1}{k_1}+\frac{1}{k_2}+\frac{1}{k_3} \qquad (3.2.1)$$

From this the equivalent cross-sectional area Ae of the BRB can be calculated:

$$A_e=\frac{kl}{E} \qquad (3.2.2)$$

From the BRB yield capacity and the member length, the user can look up the equivalent cross-sectional area in the product specification table in the appendix.

3.3 Brace capacity

A buckling-restrained brace has three capacities - design, yield and ultimate - applicable to different situations in structural design. The design, yield and ultimate capacities of the JY-SD BRB series are given in the appendix and may be selected directly by the designer.

3.3.1 Design capacity

The design capacity is the elastic capacity, used for static-load and minor-earthquake analysis. Generally, first estimate a brace yield capacity and choose the core-plate material, then look up an equivalent cross-sectional area Ae in the appendix product table, and define a two-force member of section Ae for the overall elastic analysis. For checking the brace member itself, compare the maximum enveloped axial force with the brace design capacity (from the appendix table); if it is smaller, the brace passes the elastic check. The brace design capacity is calculated as:

$$N_b = fA \qquad (3.3.1)$$

A —— cross-sectional area of the steel in the restrained yield segment;

f —— design strength of the core-plate steel.

3.3.2 Yield capacity

The yield capacity is used in the elastic-plastic analysis of the structure; it is the axial force at which the brace first yields, calculated as:

$$N_{\mathrm{by}} = R_{\mathrm{y}} f_{\mathrm{y}} A \qquad (3.3.2)$$

N_{by} —— yield capacity of the buckling-restrained brace;

R_{y} —— overstrength factor of the core-plate steel: 1.25 for Q235, 1.1 for Q345, Q390 and Q420 take 1.05; when measured data are available they shall govern, and the measured value shall not exceed the above by more than 15%.

3.3.3 Ultimate capacity

Under earthquake action the BRB core undergoes strain-hardening as it yields in tension and compression. Taking strain-hardening into account, the brace's maximum capacity is the ultimate capacity, calculated as:

$$N_{bu} = \xi \omega N_{by} \qquad (3.3.3)$$

—— compressive- and tensile-strength adjustment factors of the brace; omega is determined from Table 3.3.3-1, and xi may be taken as 1.03-1.2 depending on conditions;

N_{by} —— yield capacity of the buckling-restrained brace.

The ultimate capacity is used for the connection and joint design of the buckling-restrained brace.

Table 3.3.3-1 Strain-hardening adjustment factors of core-plate steel

Material grade	
Q160	2.0
Q225	1.5
Q235	1.5

3.4 Brace design requirements

3.4.1 Capacity requirement under wind load

Under the combination of wind or minor earthquake with other static loads, the maximum tension-compression axial design force N of the BRB shall satisfy:

$$N \le N_b \tag{3.4.1}$$

N ——axial design force of the buckling-restrained brace;

N_b ——design capacity of the buckling-restrained brace.

3.4.2 Flexural-stiffness requirement of the restraining casing

Under first-stage frequent earthquakes and second-stage rare earthquakes, the BRB member shall not buckle globally. The ratio of the member's global elastic buckling load to the yield capacity of the restrained yield segment shall satisfy:

$$\frac{\pi^2 E_0 I_0}{l^2} / (\xi \omega R_y A f_y) \ge 1.2 \tag{3.4.2}$$

$_0$ —— elastic modulus of the buckling-restraining unit and its moment of inertia about the section centroid;

l ——effective length of the buckling-restrained brace member;

R_y —— overstrength factor of the restrained-yield-segment steel; when fy is determined by test, Ry = 1.0;

—— compressive- and tensile-strength adjustment factors of the brace;

$_y$ —— cross-sectional area and characteristic yield strength of the restrained yield segment.

The JY-SD BRB product series already satisfies the global-stability requirement by design; if a JY-SD BRB product is used, the above check is unnecessary.

3.4.3 Width-to-thickness-ratio requirement of plate elements

Under first-stage frequent and second-stage rare earthquakes, the plate elements of the BRB member shall not buckle locally. The width-to-thickness ratio of the unrestrained non-yield segment (connection segment) shall satisfy:

$$\frac{b}{t} \le 15\sqrt{\frac{235}{f_y}} \tag{3.4.3}$$

outstanding width and thickness of the unrestrained-non-yield-segment steel.

3.5 Brace connection design requirements

3.5.1 Bolted connection

A bolted connection is shown schematically in Figure 3.5.1.

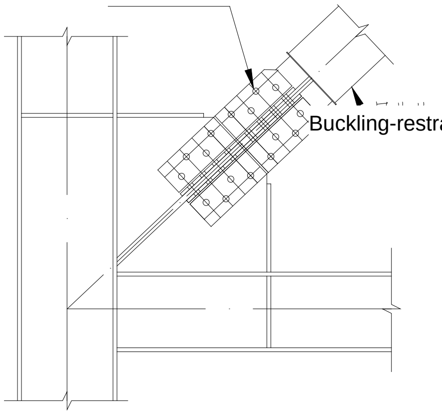


Figure 3.5.1 Typical bolted connection

To ensure that the connection to the BRB does not slip under a rare earthquake, the number n of high-strength friction-grip bolts can be determined by:

$$n \ge \frac{1.2N_{bu}}{0.9n_f \mu P} \tag{3.5.1}$$

n_f —— number of load-transferring friction surfaces;

μ —— slip-resistance factor of the friction surfaces;

P —— pretension of each high-strength bolt;

N_{bu} —— ultimate capacity of the buckling-restrained brace.

3.5.2 Welded connection

For a high-capacity BRB, a bolted connection would require many bolts and hence a long connection segment; a welded connection may be used instead. A welded connection is shown schematically in Figure 3.5.2.

3.5.

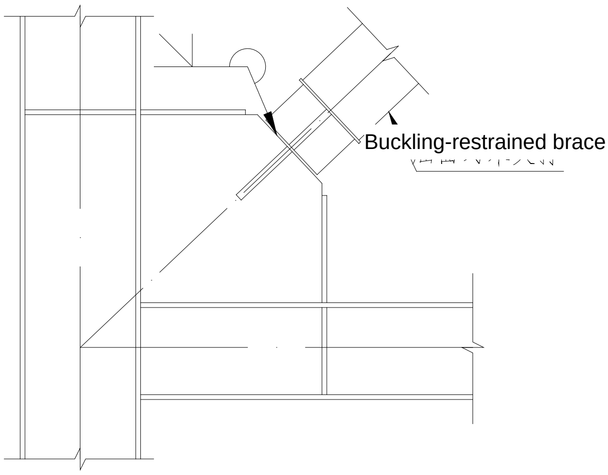


Figure 3.5.2 Typical welded connection

Fillet or butt welds may be used; the capacity of the welded connection shall satisfy:

$$N_f \geq 1.2N_{bu} \tag{3.5.2}$$

3.5.3 Pin connection

A typical pin connection is shown in Figure 3.5.3.

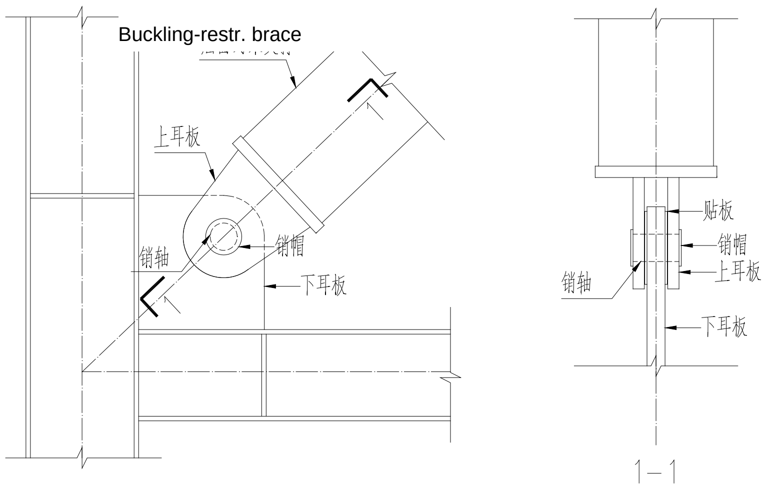


Figure 3.5.3 Typical pin connection

1. Pin calculation

First perform the pin shear calculation to determine the pin diameter.

The total shear on the pin is Vb = Nmax, where Nmax is the maximum axial force of the BRB.

Pin diameter: where nv is the number of shear planes, fbv is the design shear strength of the pin;

if the pin is made of quenched-and-tempered No. 45 steel, fbv = 250 MPa.

2. Gusset-plate (lug) design

For a tension lug, shear, local bearing and tension design are required; for a compression lug, local bearing and compression design are required;

(1) Lug shear design

$t_1 \geq V_b \times 1.5 / (f_v \times n_{v耳} \times 耳板抗剪长度)$ n

$n \quad = 2$

(2) Lug local-bearing design

$t_2 \geq V_b / (f_c^b \times D)$ where D is the pin diameter and fbc is the design bearing strength of the bolt, determined from Table 3.4.1-4 of the Code for Design of Steel Structures according to the lug material.

(3) Lug tension design

$t_3 \geq V_b / (f \times 耳板抗拉长度)$

(4) Lug compression design

$t_4 \geq V_b / (f \times 耳板承压宽度)$

$t_{耳板} = max(t_1, t_2, t_3, t_4)$

3. Cover-plate-to-lug connection calculation

When the lug thickness from the shear and local-bearing checks is large, it can be addressed by attaching cover plates to the sides of the lug; each cover plate carries shear,

$$V_t = V_b \times \frac{nt_t}{(t_{耳板} + nt_t)}$$

where tt is the single cover-plate thickness and n is the number of cover plates (assuming equal design capacities for cover plate and lug).

The cover plate and lug are usually joined by an all-round fille $h_e \geq \frac{V_t / n}{lf_f^w}$ effective leg size

where l is the weld perimeter and ffw is the design shear strength of the fillet weld. The weld size shall also meet the code's detailing requirements.

4 Design procedure

For wind load and frequent-earthquake action, the brace is designed elastically by essentially the same method as an ordinary braced-frame system; the specific software implementation is as follows:

1) Build the analysis model of the BRB structure by the ordinary modelling method, inputting all design loads (dead, live, wind and seismic).

2) Define the buckling-restrained braces. The method is similar to defining ordinary steel braces, except that the brace is defined by the BRB's equivalent cross-sectional area - the steel-brace section area need only equal the BRB equivalent area. When choosing the section type, a square section is recommended for convenient calculation of the equivalent area.

3) Place the buckling-restrained braces. Following the designer's intent, place the BRBs using each software's method for placing ordinary steel braces.

4) Compute the seismic-action effects by the ordinary method and perform the structural design. At this point the BRB design result generally fails the code requirements, mainly because of member slenderness. However, the BRB's key feature is that its tension and compression behaviour are essentially identical with no compressive buckling, so for the BRB one can ignore the software design result and instead extract the maximum internal force of the brace under each load combination and compare it with the BRB design capacity; if the maximum internal force under every combination is less than the design capacity, the requirement is met. Otherwise a larger product should be selected and re-checked until the requirement is met.

The next section uses a worked example to illustrate BRB modelling and design in the popular design software, then briefly introduces the elastic-plastic analysis of BRBs.

4.1 Overview of the worked example

This project is a residential building with a concrete frame system, 1 basement storey and 6 above-ground storeys. The basement storey height is 2.6 m and the above-ground storey height is 2.9 m; the seismic fortification intensity is 8 degrees, the site class is III, and the seismic
2.6 group is Group 1.

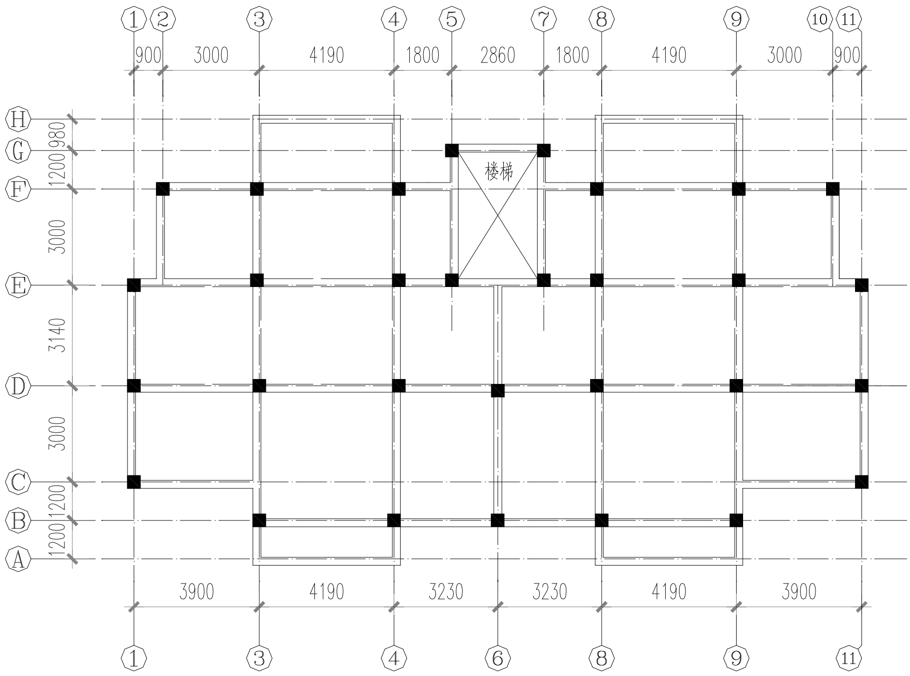


Figure 4.1.1 Structural plan of storeys 2-6

Based on the structural characteristics, buckling-restrained braces are to be placed on storeys 1-5 at the plan positions shown below: single diagonal braces at C-D and D-E of axis 1, C-D and D-E of axis 11, and 4-6 and 6-8 of axis B; chevron (inverted-V) braces at 3-4 and 8-9 of axis F.

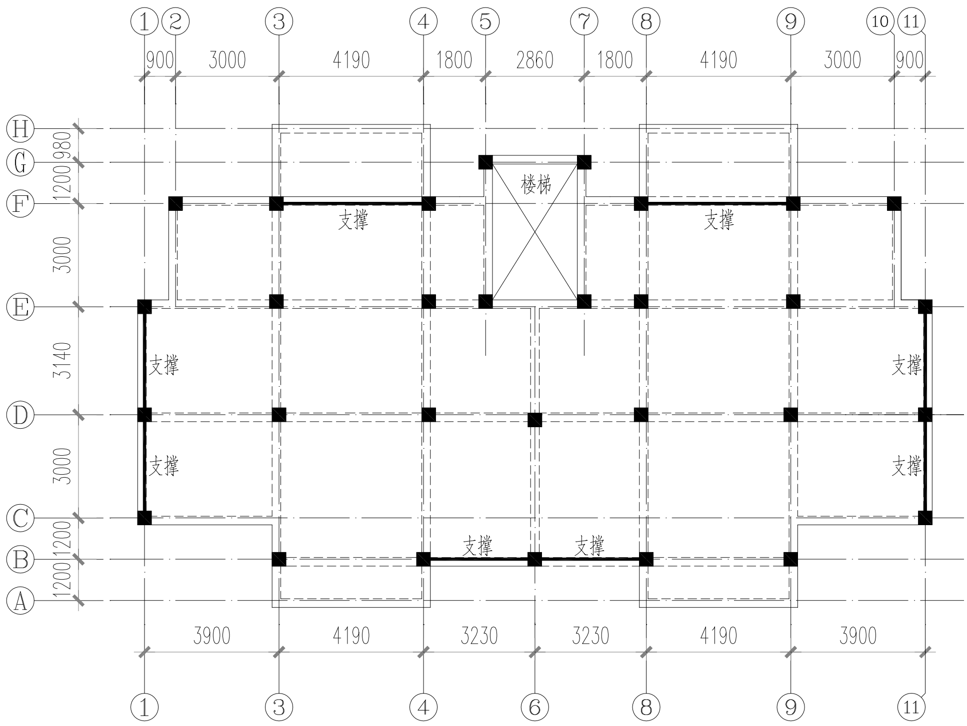


Figure 4.1.2 Layout of the buckling-restrained braces

4.2 BRB design procedure in PKPM

First model the beams, slabs and columns by the ordinary method, and add the floor dead and live loads, partition loads on beams, etc. The structural model without braces is shown below.

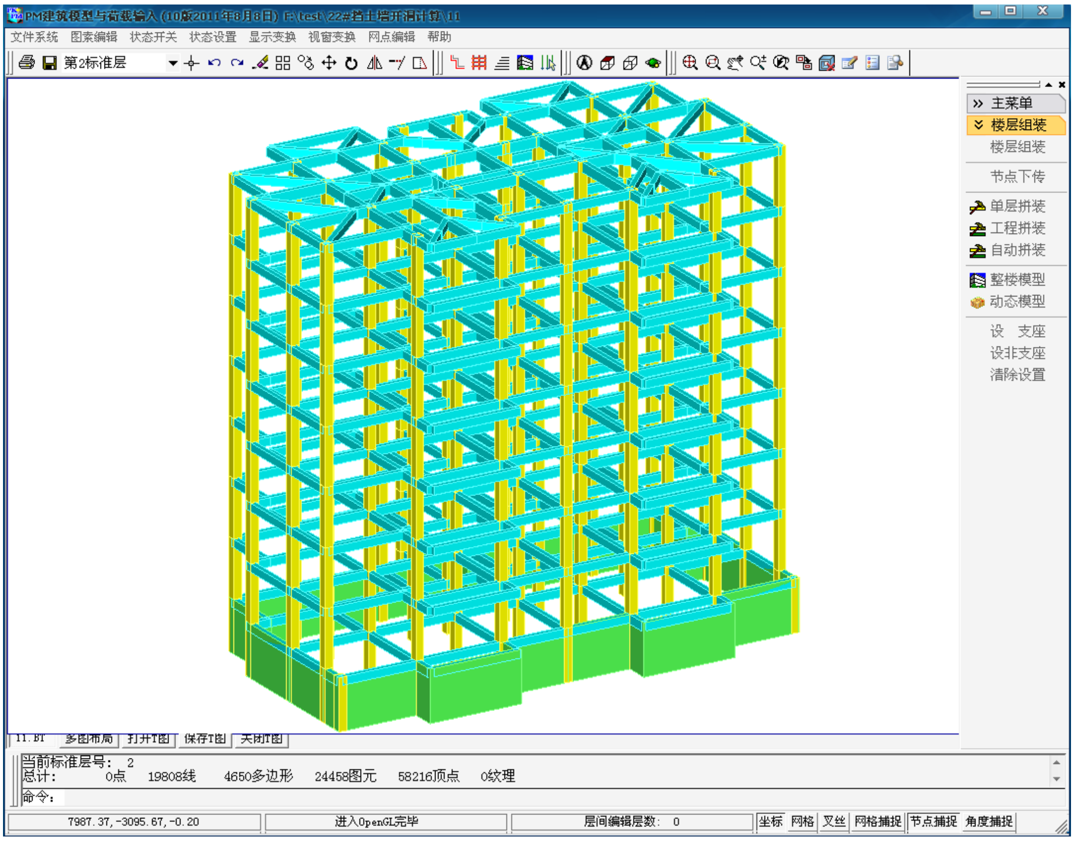


Figure 4.2.1 PKPM structural model without buckling-restrained braces

4.2.1 Defining the buckling-restrained braces

Defining a BRB is similar to defining an ordinary steel brace, except that the equivalent cross-sectional area is used.

Open the [Floor Definition] menu and click [Diagonal Brace Layout] to open the brace dialog. Click [New]; as a rectangular section does not support steel material, choose a box section as the equivalent section, enter the section-parameter dialog (Figure 4.2.2), and input the box-section dimensions and material class. Define the second and third standard brace sections likewise. The defined brace sections are shown in Figure 4.2.3.

Figure 4.2.2 Standard diagonal-brace parameter-input dialog

Figure 4.2.3 Diagonal-brace section list

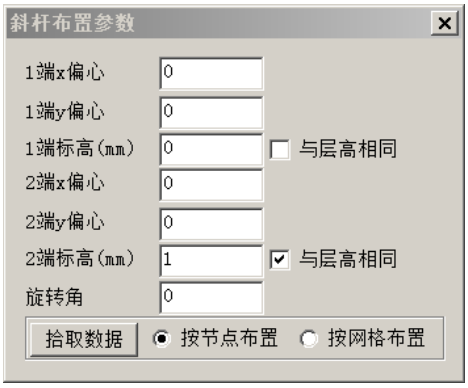


Figure 4.2.4 Diagonal-brace layout-parameter dialog

4.2.2 Placing the buckling-restrained braces

Diagonal braces can be placed in two ways: by node and by grid. The nodes at the two ends of a brace must exist not only on the standard storey where it is placed; the standard storey receiving the other end must also mark the node there.

After clicking the layout button, the dialog in Figure 4.2.4 appears; set end point 2 equal to the storey height, then click the lower and upper points of the brace in turn. The model after placing the BRBs is shown in Figure 4.2.5.

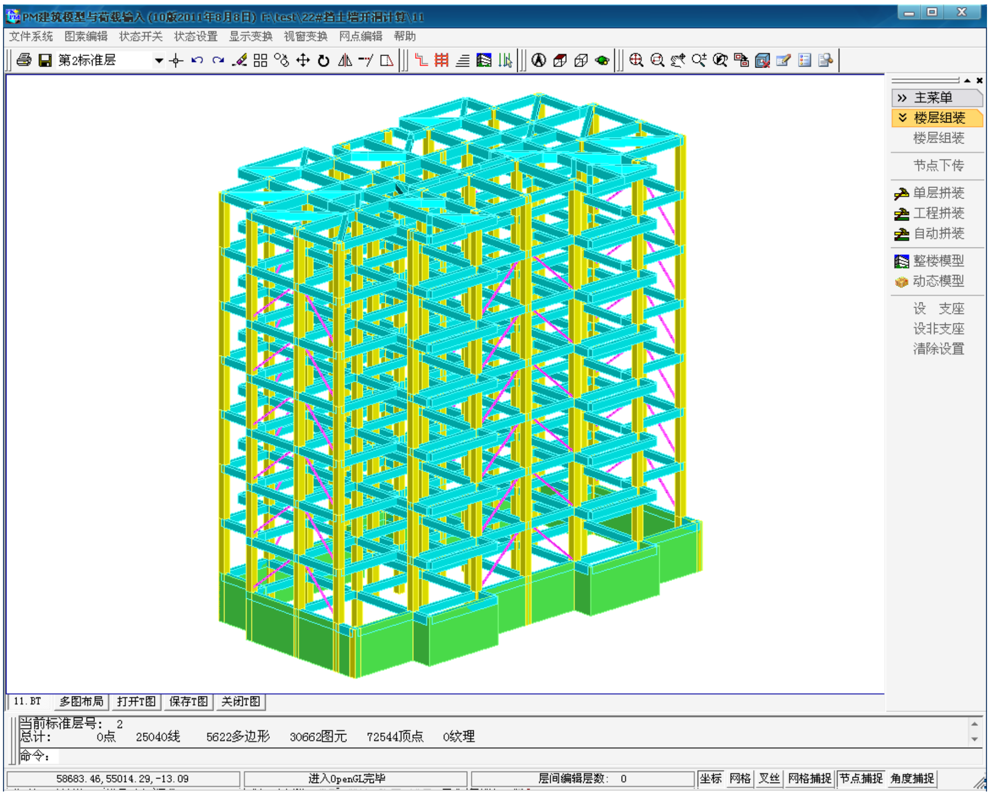


Figure 4.2.5 Structural model after placing the buckling-restrained braces

4.2.3 SATWE calculation settings

The supplementary analysis and design parameters of SATWE are defined as in conventional design. Click SATWE [Supplementary Definition of

Analysis and Design Parameters] and define the parameters as shown below.

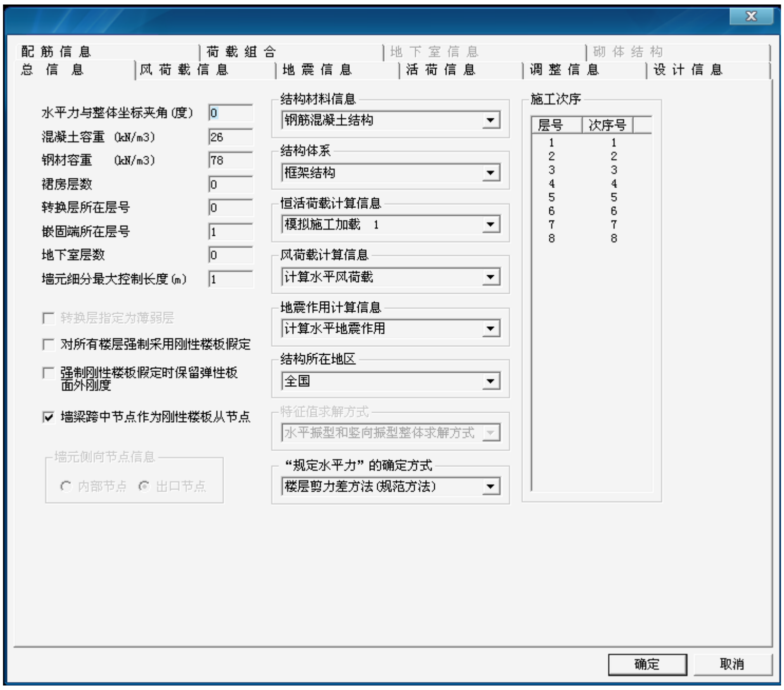


Figure 4.2.6 General-information definition

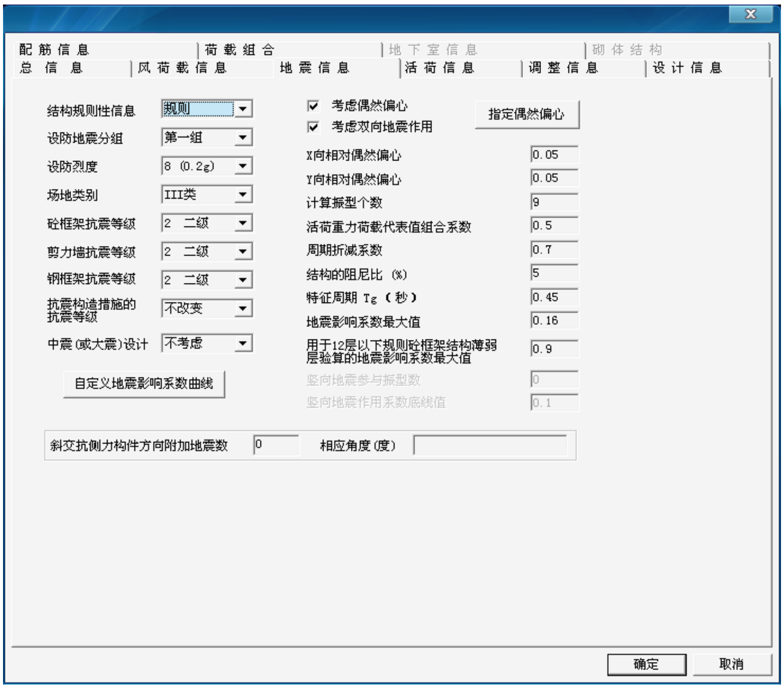


Figure 4.2.7 Seismic-information definition

After defining the BRBs, open [Supplementary Definition of Special Members] and check that both ends of each brace are pinned; if not, change them to pinned at both ends.

Then run [Generate SATWE Data File and Data Check]; once the data file is formed, perform the internal-force and reinforcement calculations. The subsequent process is the same as ordinary structural design.

Note that the buckling-restrained braces are usually shown in red, failing the code requirements. However, since a BRB needs no stability check, the SATWE result can be ignored; simply check the BRB capacity directly as required in the next section.

4.2.4 Method for checking buckling-restrained brace members

The BRB member is checked in three steps:

1) Extract the maximum internal force of the brace under each load combination;

2) From each brace's equivalent cross-sectional area, the intended core-plate steel grade and the estimated actual length, look up the design capacity of each BRB in the appendix tables of this manual;

3) Compare each BRB's maximum internal force with its design capacity; if the maximum force is less than the design capacity, the requirement is met, otherwise select a higher-capacity BRB and re-analyse.

4.3 BRB design procedure in ETABS

Build the structural model without braces by the ordinary ETABS modelling method, as shown below.

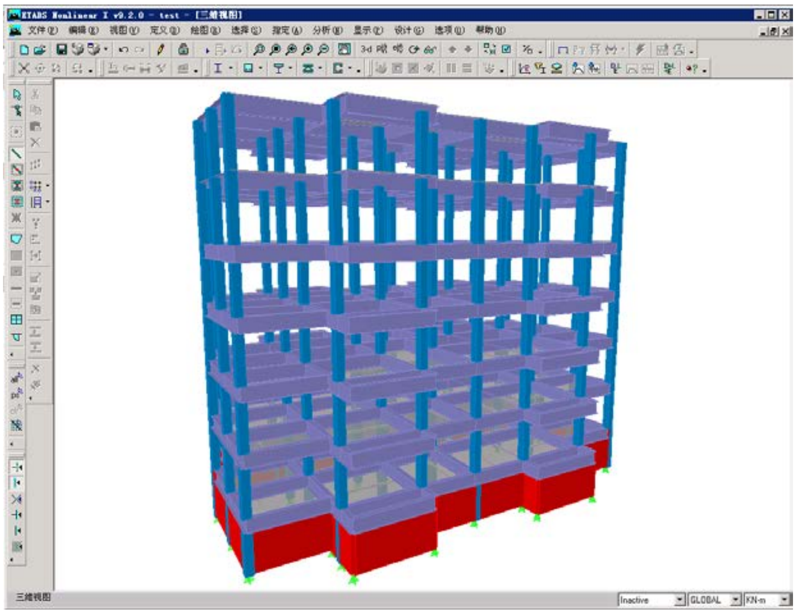


Figure 4.3.1 ETABS structural model without buckling-restrained braces

4.3.1 Placing the buckling-restrained braces

First define the BRB section. In ETABS, open the [Define] menu, click [Frame Sections], and choose Add Rectangular. In the dialog (Figure 4.3.2), define the height and width of the rectangular section so that its area equals the BRB equivalent cross-sectional area.

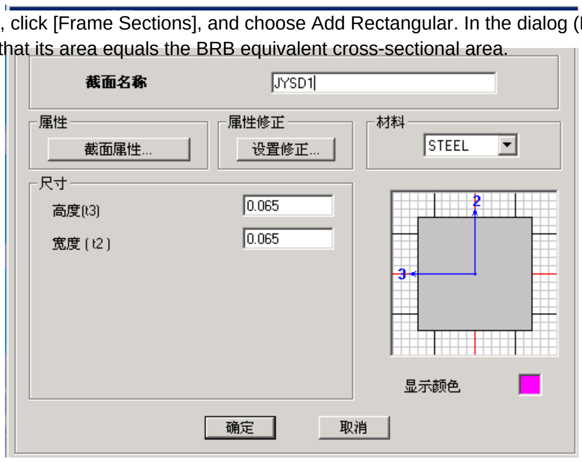


Figure 4.3.2 Section-definition dialog

After defining the BRB section, click [View] -> [Set Elevation View], select axis 1 in the dialog and click OK, then place BRBs at C-D and D-E as in Figure 4.3.3.

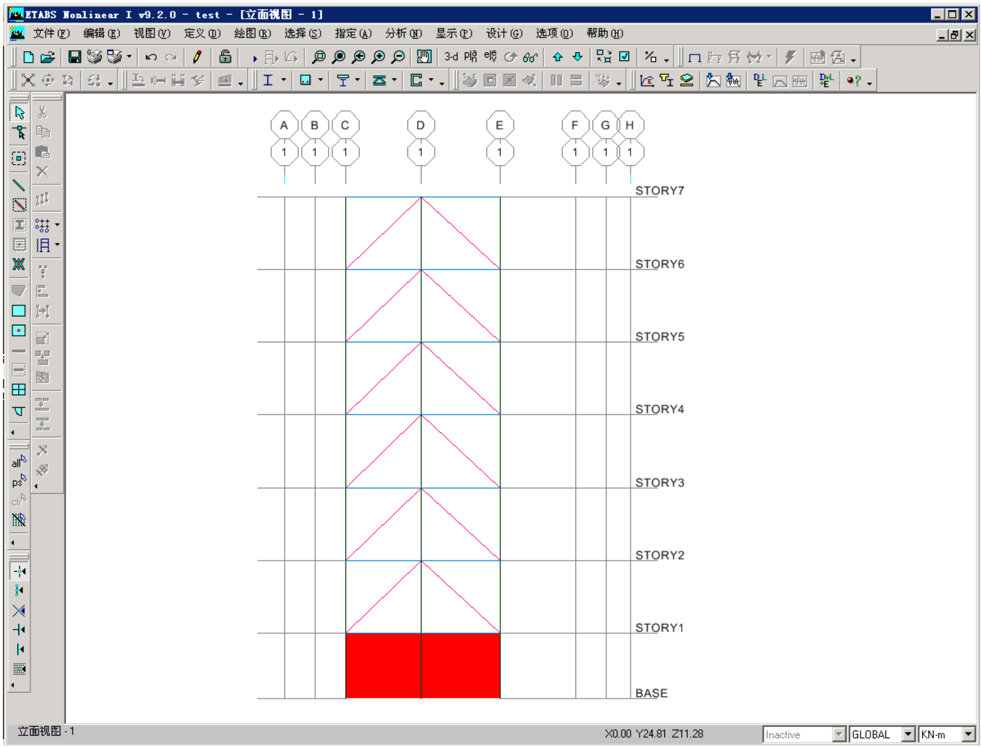


Figure 4.3.3 Brace layout on axis 1

Place the BRBs on axes 11, B and F in the same way; the overall model afterwards is shown in Figure 4.3.4.

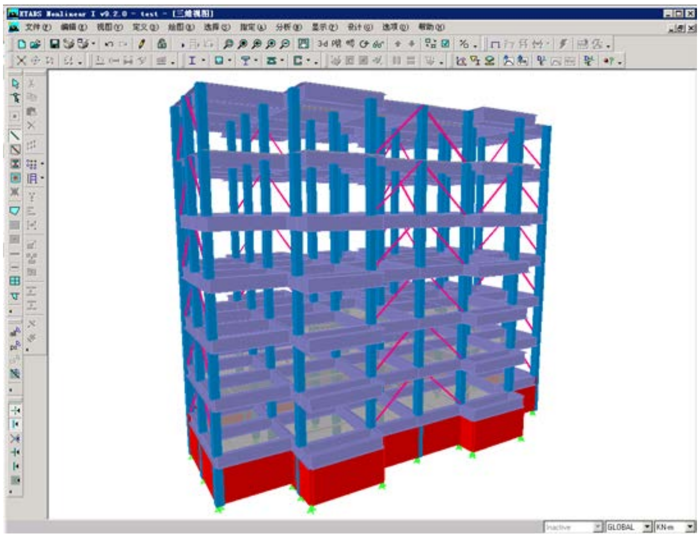


Figure 4.3.4 ETABS structural model after placing the buckling-restrained braces

4.3.2 Defining the seismic analysis case

Click [Define Response-Spectrum Function]; in the dialog (Figure 4.3.5), select "Chinese2002 Spectrum" in the function-type drop-down and click [Add Function]. In the dialog that appears, set the maximum influence coefficient, seismic intensity, site characteristic period, period-reduction factor and damping ratio per the code.

Figure 4.3.5 Response-spectrum-function definition dialog

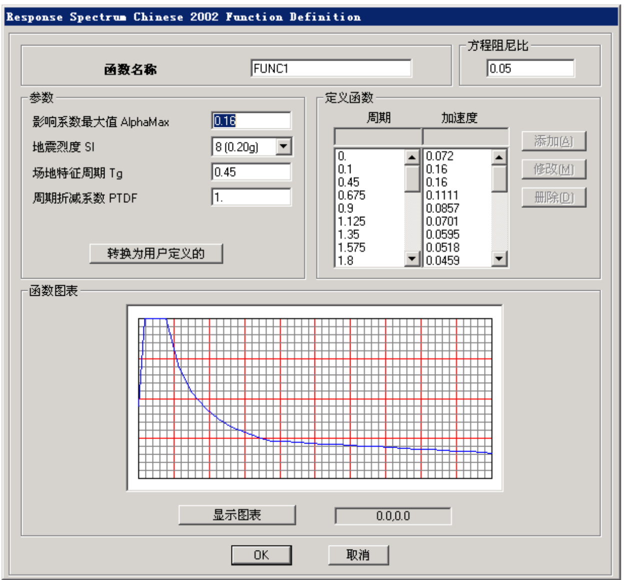


Figure 4.3.6 Defined response-spectrum function

After defining the response-spectrum function, click [Define] -> [Define Response-Spectrum Case]; in the dialog (Figure 4.3.7) click [Add New Spectrum], and set the parameters as shown in Figure 4.3.8.

4.3.

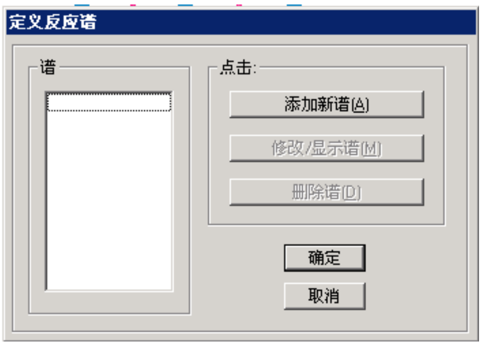


Figure 4.3.7 Response-spectrum-case definition dialog

4.3.3 Brace-checking method

Run the analysis for all load cases on the model above and design the members.

As with PKPM, ETABS considers member stability when designing the BRB, so its design result is unreasonable. Extract the maximum internal force of each BRB under all load combinations in ETABS and check it by the method of Section 4.2.4.

Figure 4.3.8 Response-spectrum-case data dialog

4.4 BRB design procedure in MIDAS

Build the structural model without braces by the ordinary MIDAS modelling method, as in Figure 4.4.1.

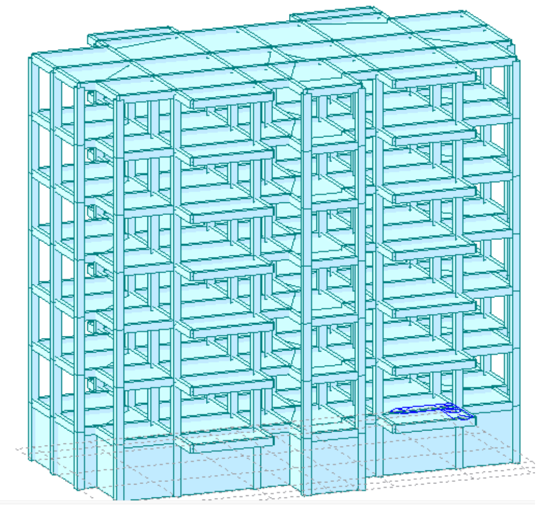


Figure 4.4.1 MIDAS structural model without buckling-restrained braces

4.4.1 Placing the buckling-restrained braces

First define the BRB section. In MIDAS, open the [Define] menu, click [Frame Section], and choose Add Rectangular. In the dialog, define the height and width of the rectangular section so that its area equals the BRB equivalent cross-sectional area.

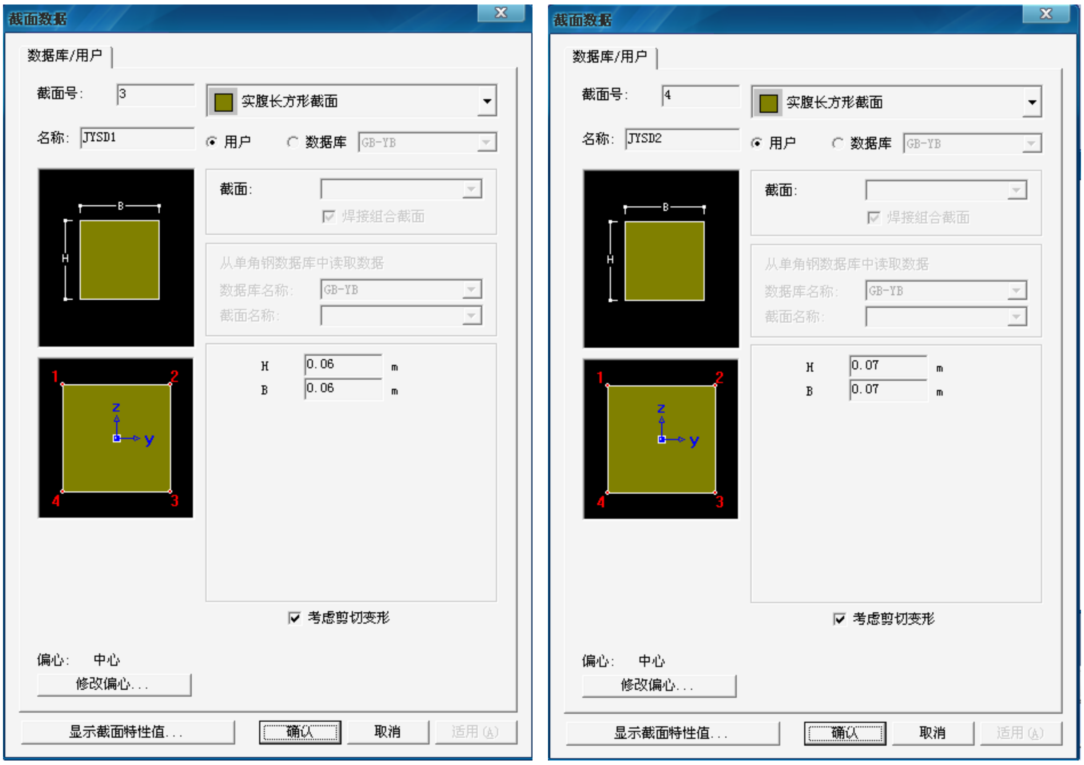


Figure 4.4.2 BRB section-definition dialog

After defining the BRB section, select and activate the frame planes of axes 1, 11, B and F in turn and place the BRBs at the specified positions. The BRB layouts on the axes are shown in Figures 4.4.3-4.4.5.

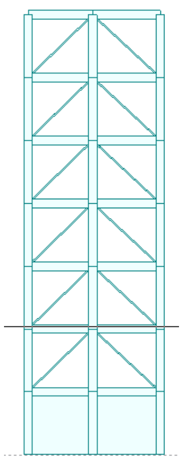


Figure 4.4.3 BRB layout on axes 1 and 11

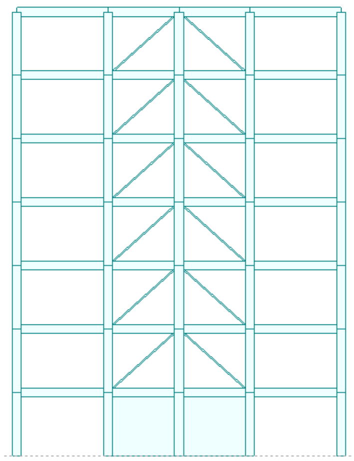


Figure 4.4.4 BRB layout on axis B

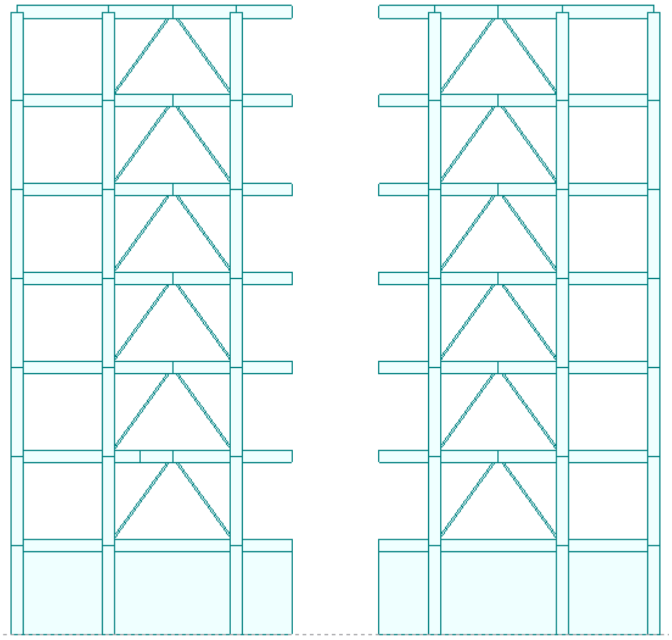


Figure 4.4.5　BRB layout on axis F

The overall model after placing the BRBs is shown in Figure 4.4.6.

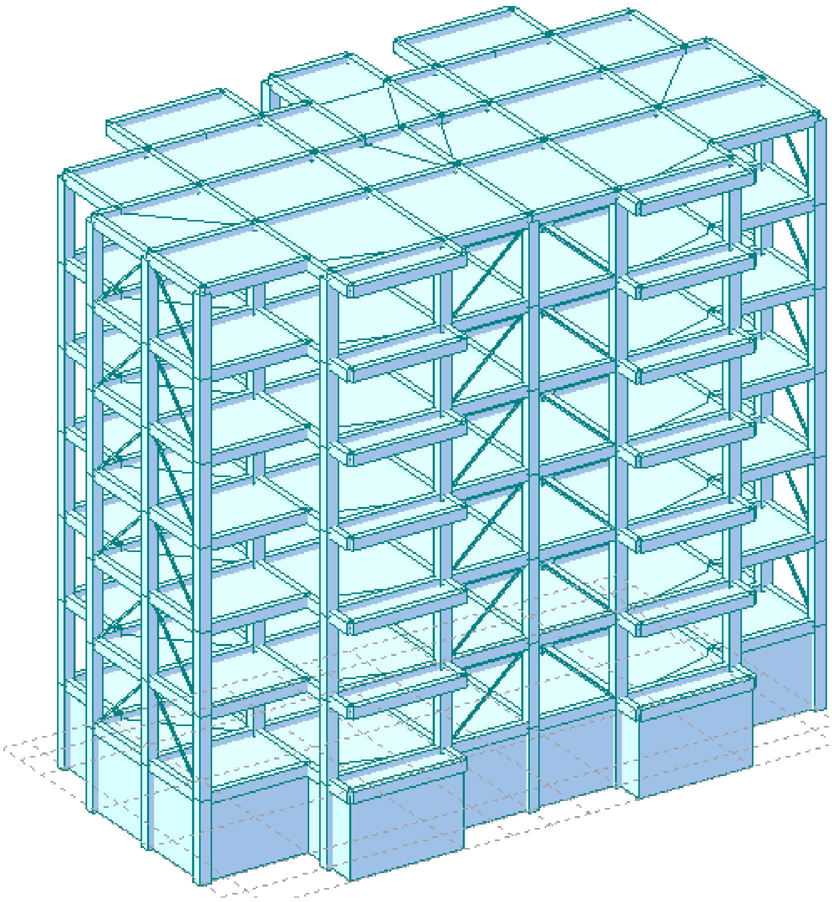


Figure 4.4.6　MIDAS structural model after placing the buckling-restrained braces

4.4.2 Defining the seismic analysis case

Click [Load] -> [Response-Spectrum Analysis Data] -> [Response-Spectrum Function] and click Add; in the dialog (Figure 4.4.7) click the Design-Spectrum button. In the dialog (Figure 4.4.8), set the seismic group, fortification intensity, site class, damping ratio and maximum period per the code, then click OK to build the response-spectrum function in Figure 4.4.9.

Figure 4.4.7 Response-spectrum-function definition dialog

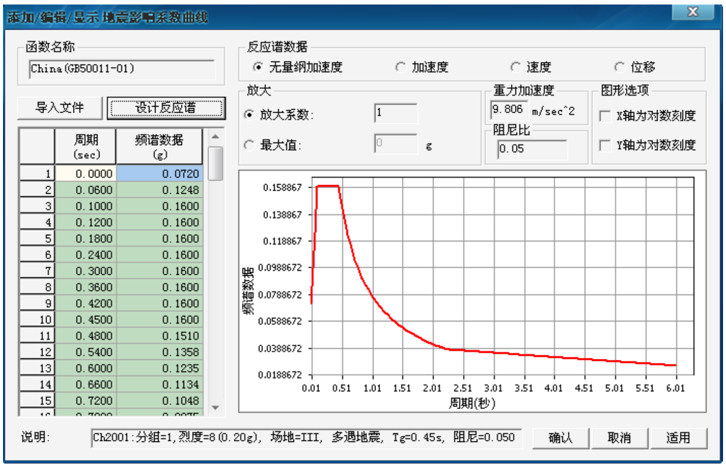


Figure 4.4.8 Design-response-spectrum dial Figure 4.4.9 Defined response-spectrum curve

After defining the response-spectrum function, define the response-spectrum case: click [Load] -> [Response-Spectrum Analysis Data] -> [Response-Spectrum Load Case] and define the case in the dialog (Figure 4.4.10).

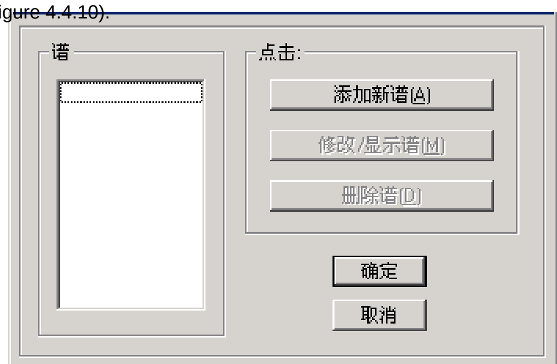


Figure 4.4.10 Response-spectrum-load-case definition dialog

4.4.3 BRB-checking method

Run the analysis for all load cases on the model above and design the members.

Extract the maximum combined internal force of each BRB under all load cases in the MIDAS model and compare it with the BRB capacity; if the maximum combined force is less than the BRB capacity, the BRB meets the code, otherwise keep adjusting and re-analysing until it does.

4.5 Plastic analysis method for buckling-restrained braces

4.5.1 Static elastic-plastic (pushover) analysis

The plastic hinges of frames, shear walls, etc. in a BRB structure are set up as in conventional pushover analysis; the difference lies in how the BRB is modelled, illustrated below with ETABS.

A BRB can be modelled with the axial-force hinge among the frame hinges. The program's default axial hinge accounts for compressive buckling, whereas a BRB yields in both tension and compression.

First select the axial-force hinge (Figure 4.5.1). When defining the BRB, the default frame-hinge property cannot be used; its behaviour must be customised, as in

Figure 4.5.2 Plastic-hinge definition dialog

Figure 4.5.2 - remove the default option. Click [Modify] to enter the modify dialog (Figure 4.5.3) and input the positive and negative yield forces and yield displacement of the BRB.

After defining the frame hinge, assign it to the BRB with the hinge located at 0.5 times the member length, as in Figure 4.5.4.

4.5.

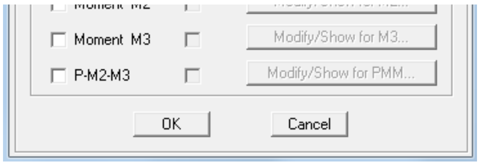
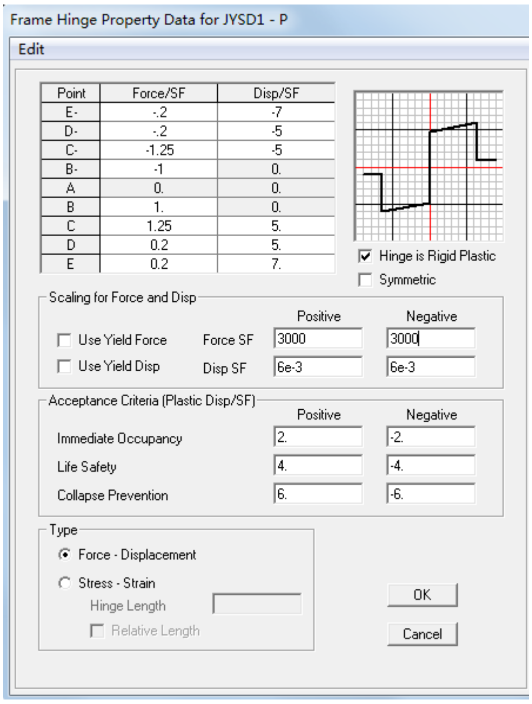


Figure 4.5.3 Creating a plastic hinge (axial-force hinge selected)



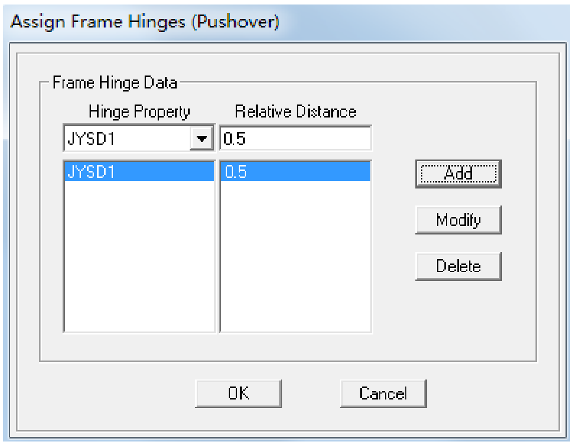


Figure 4.5.4 Adding a plastic hinge to a member

Figure 4.5.1 Input plastic-hinge properties

4.5.2 Dynamic elastic-plastic analysis

For dynamic elastic-plastic analysis of a BRB structure, general-purpose finite-element software such as ABAQUS, SAP2000 or ANSYS is recommended, or structural-analysis software such as EPDA or MIDAS. The BRB hysteresis can be modelled with a simple bilinear model or with a Bouc-Wen plastic-element model. The bilinear model is simpler, as shown in Figure 4.5.5.

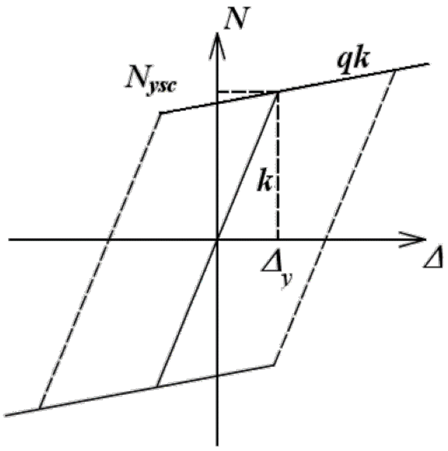


Figure 4.5.5 Bilinear hysteresis model

The Bouc-Wen model has a simple expression and is built into SAP2000, ETABS, MIDAS, etc. as the Wen plastic-element model. Its nonlinear force-deformation relationship is:

$$f = rkd + (1-r)F_y z \tag{4.5.1}$$

where k is the elastic spring constant, Fy is the yield force, r is the specified ratio of post-yield stiffness to elastic stiffness k, and z is an internal hysteresis variable bounded by |z| <= 1, with the yield surface represented by |z| = 1.

$$|z| \le 1 \qquad\qquad |z| = 1$$

$$\dot{z} = \frac{k}{F_y}\begin{Bmatrix} \dot{d}(1-|z|^n) \\ \dot{d} \end{Bmatrix} \begin{matrix} (\dot{d}\cdot z > 0) \\ (\dot{d}\cdot z \le 0) \end{matrix} \tag{4.5.2}$$

In the program, simply set Fy, k and n. A typical Wen-model hysteresis loop is shown in Figure 4.5.6.

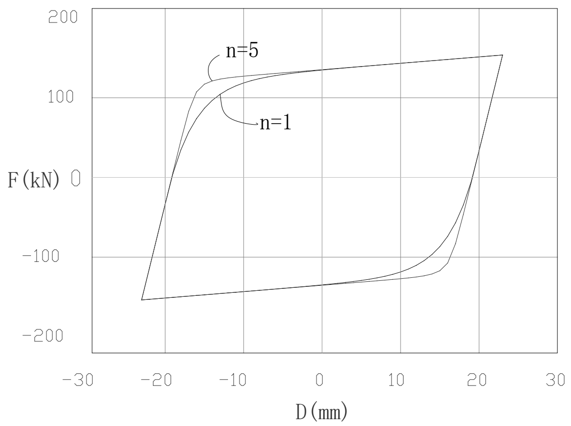


Figure 4.5.6 Typical Wen-model hysteresis curve

5 Engineering application modes

Based on years of design and engineering practice, buckling-restrained braces can address roughly the following structural problems.

I. Increasing lateral stiffness and reducing inter-storey drift

In frame structures in high-intensity zones of 8 degrees and above, meeting the code's inter-storey-drift limits often forces very large beam and column sections, conflicting with architectural needs and sometimes making design very difficult. Traditionally there are two solutions:

1) Adding shear walls to the frame. Functional requirements often limit the position and number of walls, and detailing means the walls cannot be very thin, so their stiffness is very large relative to the frame. Most of the seismic shear is then carried by the walls, making their shear-compression ratio excessive and reinforcement difficult. The only fix is more walls, which often conflicts with building function, so this approach often fails.

2) Adding ordinary steel braces. This solves the weak-lateral-stiffness problem well, but steel braces are designed for minor earthquakes, so their stability is only guaranteed under minor-earthquake forces; once seismic force exceeds that, stability is not guaranteed. Hence Appendix G of GB 50011 requires, for steel-braced concrete frames, that reinforcement be the envelope of a pure-frame model and a steel-braced-frame model, so steel braces do not help save steel.

Because, unlike an ordinary steel brace, a BRB has no loss-of-capacity-from-instability problem, it solves both the stiffness and the reinforcement problems. BRBs are therefore widely used in concrete frames, and the BRB-frame system is an excellent choice for hospitals, schools, banks, data centres and sports/cultural centres of 8 degrees and above.

Case study 1

The Phase I factory of Beijing Tianlide Electromechanical Equipment Co., Ltd., located in Chaoyang District, Beijing, has a total floor area of 39,220 m^2. The east, north and west towers have 5 above-ground storeys (roof elevation 23.2 m) and a shared 2-storey basement, used mainly as factory and ancillary offices. The towers use a reinforced-concrete frame; as the pure frame could not meet the code's inter-storey-drift and torsional-displacement-ratio limits, 48 BRBs of 3000 kN yield capacity were installed on storeys 1-2, markedly increasing stiffness while reducing frame reinforcement.

Figure 4.5.1 Phase I factory of Beijing Tianlide Electromechanical Equipment Co., Ltd.

Case study 2

A data-centre project in Chaoyang District, Beijing, has 7 above-ground and 2 basement storeys. In an 8-degree 0.2g zone, the pure-frame model gave an inter-storey drift of 1/370, far exceeding the code limit. To reduce beam/column sections and reinforcement, a frame + BRB system was adopted, using 105 BRBs in total.

Figure 4.5.2 A data-centre project

II. Reducing the torsional displacement ratio

In a tall building with a podium, architectural and fire requirements usually place the tower to one side of the podium. The tower is generally a frame-shear-wall structure, while the podium is usually a frame or a lightly-walled frame-shear-wall structure; the podium thus has severe stiffness/centre-of-mass eccentricity, making the torsional displacement ratio exceed the code limit.

Case study

Tianjin Modern City, in Heping District, Tianjin, has 48 above-ground and 5 basement storeys, a maximum building height of 209.0 m and a structural height of 202.4 m, with a 10-storey podium of maximum structural height 56.1 m. The hotel tower and podium are integral with no seismic joint. Because the tower sits to one side (left box below), the podium's stiffness centre shifts left and the torsional displacement ratio exceeds 1.6. A BRB frame was added on the right (right box), reducing the ratio below 1.4 to meet the code; 20 BRBs of 5500 kN yield capacity were installed in the podium.

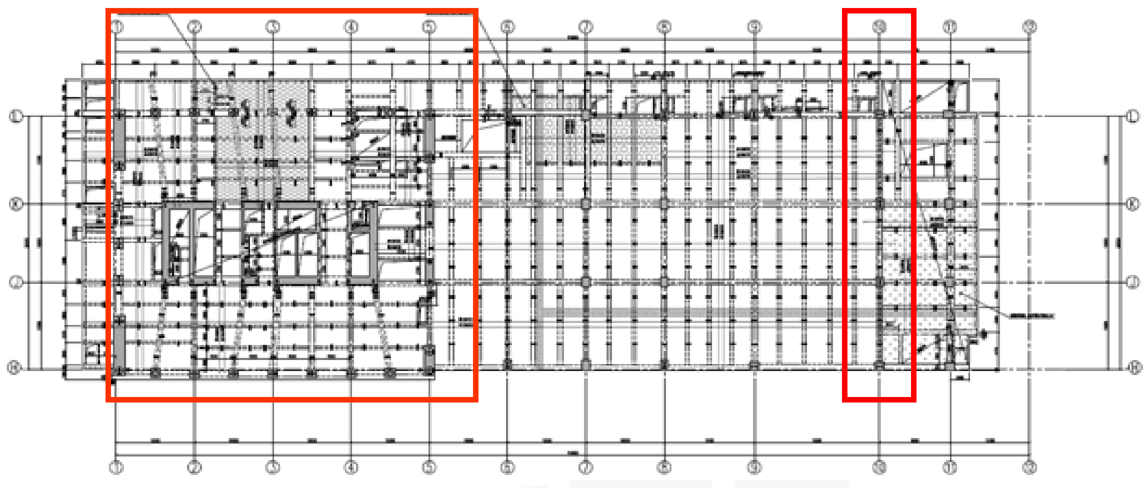


Figure 4.5.3 Layout of the podium of Tianjin Modern City

Figure 4.5.4 Rendering and installation site of Tianjin Modern City

III. Tuning strengthened-storey stiffness - BRB as a fuse for outrigger trusses

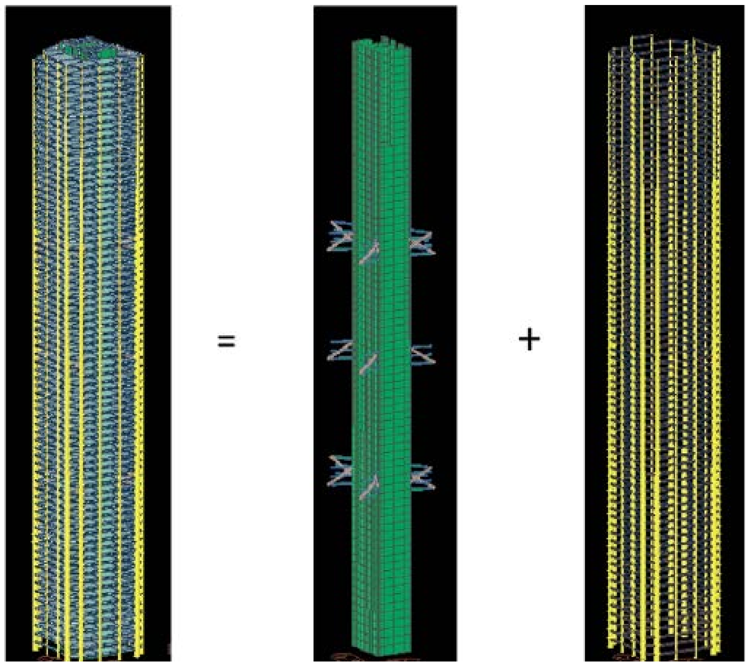
To raise overall lateral stiffness, reduce wind-induced horizontal displacement and absorb more of the overturning moment, super-tall buildings often place stiff outrigger trusses between the core and the perimeter frame at certain storeys, e.g. the Shanghai Jinmao Tower (421 m, 3 outrigger steel trusses) and Taipei 101 (508 m, 10 outrigger steel trusses). Such a system maximises material efficiency and gives more façade-design freedom. However, although outrigger trusses reduce wind-induced displacement, under earthquakes their strengthened storey can undergo abrupt stiffness change and sharp internal-force increase, making adjacent storeys weak - very unfavourable seismically. As the key member transferring axial force to the perimeter columns, the truss's stiffness, capacity and stability must be ensured. Ordinary steel-brace diagonals need large sections for in-plane and out-of-plane stability; using BRBs avoids compressive instability and keeps section and stiffness easy to control, forming a 'limited-stiffness' strengthened storey that reduces abrupt stiffness/internal-force change and yields a ductile mechanism under rare earthquakes.

BRBs have been used as super-tall-building outrigger-truss diagonals in many projects, e.g. Beijing Yintai Centre (249.9 m, 6450 kN), Baoji International Finance Centre (203 m,

30000 kN), Tianjin Modern City Zone B (300 m, 13000 kN), Changsha Shimao (350 m, 35000 kN), Yiw

Case study

The Changsha Shimao Plaza project, developed by Changsha Shimao Investment Co., Ltd., is a super-tall commercial-office complex in the busiest central area of Changsha, set to become a landmark. It comprises an office tower, a commercial podium and a basement, with a total floor area of about 222,000 m². The office tower has 75 above-ground storeys (~155,000 m²), a structural height of 332.5 m at the main roof and 349.8 m at the roof frame. The podium has 5 above-ground storeys (~22,000 m²) and the basement 4 storeys (~45,000 m²). To raise overall lateral stiffness, the outrigger-truss diagonals form a large share of the strengthened storey's stiffness. To prevent uneven overall stiffness from compressive instability of some diagonals under unfavourable combined loads (concrete creep plus major earthquake), BRBs were used as diagonals: 8 of 35000 kN, 4 of 25000 kN and 8 of 21000 kN.



Lateral-force system Core + BRB Perimeter frame

Figure 4.5.5 Lateral-force system of the Changsha Shimao Plaza project

Figure 4.5.6 Rendering and installation site of the Changsha Shimao Plaza project

IV. Reducing member sections

BRB design shows that, for the same capacity, a BRB's outer section can be smaller than an ordinary brace's; and for the same section size, a BRB's capacity can reach about twice that of an ordinary brace. BRBs are therefore often used where member section size is limited.

The Xinjiang Grand Theatre uses a long-span spatial grid-arch system of two outer shells and one inner shell. The outer shell spans up to 55.5 m and the inner shell up to 102.5 m (below). As the inner shell's longitudinal lateral-force system consists of two edge trusses, their root braces carry large forces under longitudinal earthquake. Ordinary steel braces would need large sections, and as this is a spatial truss with many members meeting at nodes, oversized sections would make the nodes hard to detail. To minimise the brace sections, BRBs replaced ordinary steel braces, solving the node problem while giving an attractive appearance.

Figure 4.5.5 Rendering of the Xinjiang Grand Theatre

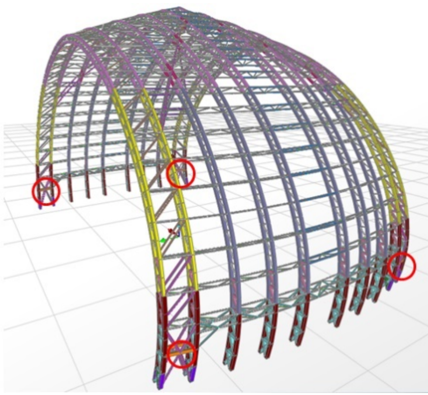


Figure 4.5.6 Inner-shell structure

V. Application in frame-strengthening (retrofit) projects

Using BRBs in frame strengthening is very common. The advantage: once BRBs are added, the BRB frame carries most of the seismic force, greatly reducing forces on other frame members, so strengthening can be limited to the frame beams and columns connected to the BRBs, greatly cutting the workload and saving time. This is especially advantageous in school retrofits, where the schedule is tight and work often must finish in the two-month summer break.

Case study 1

The kindergarten of Beijing Normal University has a reinforced-concrete frame main teaching building. To meet the state's raised seismic-fortification level for schools, seismic strengthening was required; after analysis, 64 BRBs of 100 t yield capacity were used, greatly reducing the strengthening workload.

Figure 4.5.7 Beijing Normal University kindergarten after strengthening

Case study 2

The strengthening and renovation of the Military Museum Exhibition Building comprises seismic strengthening (33,000 m²) and new extension (115,000 m²). 28 JY BRBs were installed on storeys 6-7, mitigating the abrupt stiffness change caused by the large set-back of those storeys, reducing the strengthening quantity, shortening the schedule and yielding good social and economic benefits.

Figure 4.5.8 Strengthening site of the Military Museum Exhibition Building

Part 3 Shear-, Bending- and U-Type Dampers

1 Basic principles

A shear-type damper dissipates energy through shear plastic deformation of its web. To let the web reach shear yield without out-of-plane buckling instability, anti-buckling stiffeners or out-of-plane anti-buckling clamp plates are provided on the web.

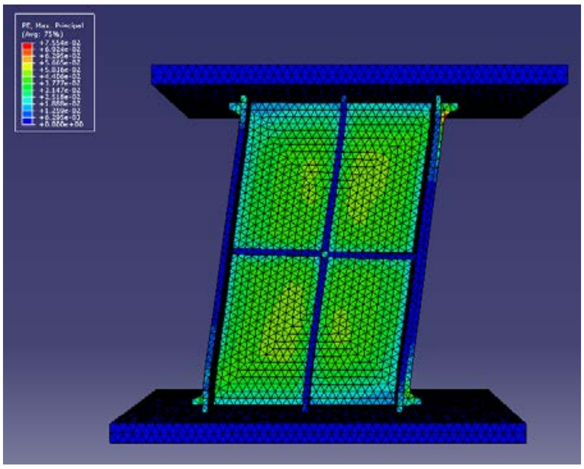


Figure 1.0.1 Shear-type damper

A bending-type damper dissipates energy through out-of-plane bending plastic deformation of steel plates (below). Its maximum energy-dissipation amplitude exceeds that of a shear-type damper, but because the plate's out-of-plane stiffness is weak, the added stiffness it provides is relatively small.

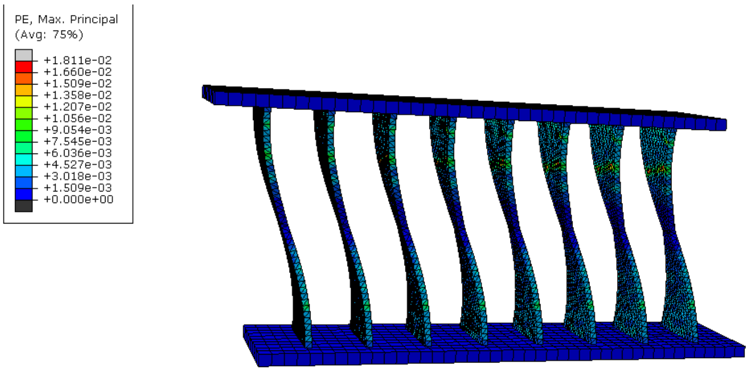


Figure 1.0.2 Bending-type damper

A U-type damper dissipates energy by the rolling of a restrained arc-section steel plate (below). As deformation increases, the point of maximum plastic-deformation fatigue in the steel moves along, so the plastic deformation is evenly distributed and the material is used efficiently; it suits environments with high hysteretic-energy-dissipation requirements.

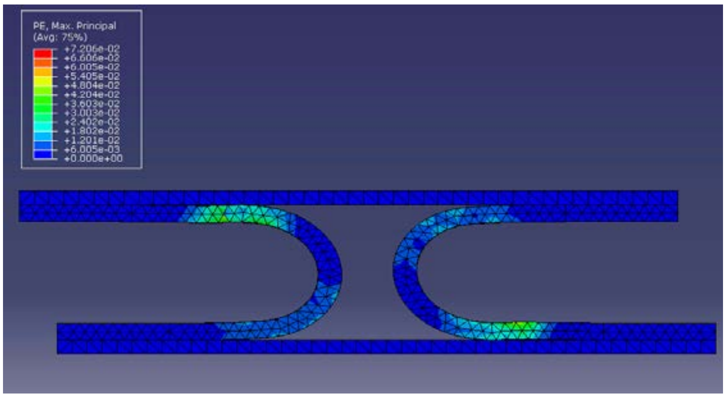


Figure 1.0.3 U-type damper

Typical hysteresis curves are shown below.

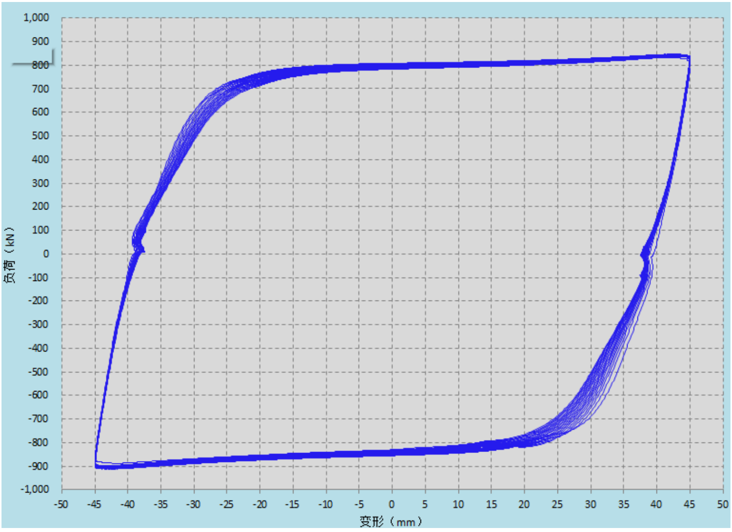


Figure 1.0.4 Hysteresis curve of a shear-type damper

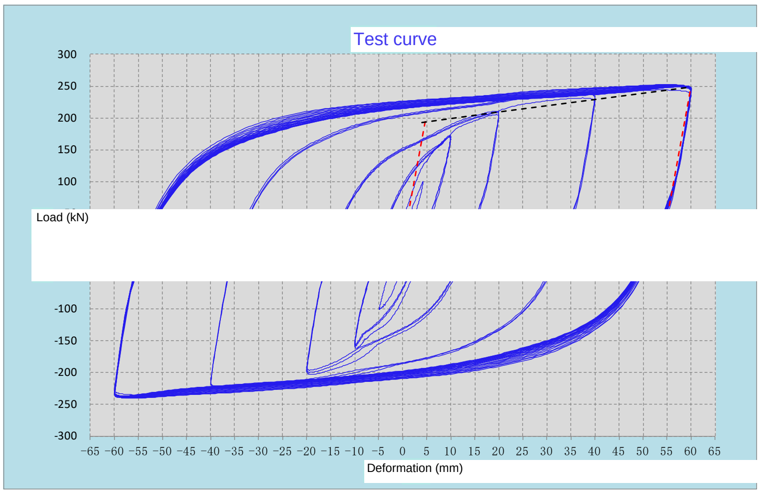
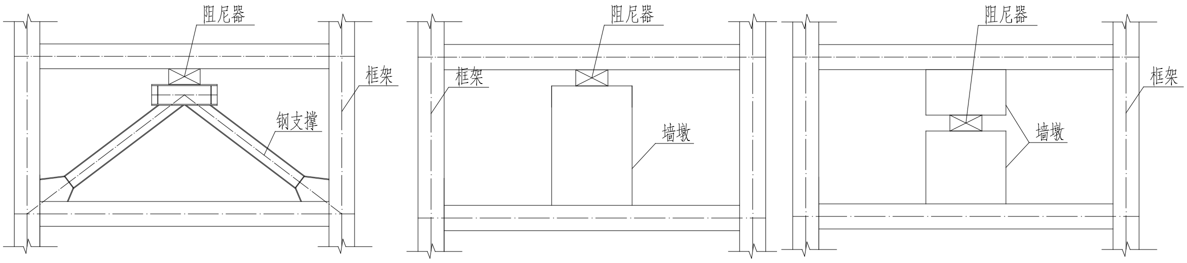


Figure 1.0.5 Hysteresis curve of a U-type damper

To achieve good energy-dissipation capacity, the web of shear-type dampers and the bending steel plates of bending-type and U-type dampers generally use low-yield-point steel, commonly Q100LY, Q160LY, Q225LY, etc.; Q235 steel may also be used.

2 Computational simulation

A damper can be applied in a frame in three ways: brace + damper, lower wall-pier + damper, and upper-and-lower wall-piers + damper.



Brace + damper   Lower wall-pier + damper   Upper-and-lower wall-piers + damper

In structural-software analysis, the stiffness of the above damper sub-structure can be made equivalent to a single wall panel or a chevron brace to simulate the added stiffness it provides.

A shear-type damper can also be placed in the coupling beam of a shear-wall structure, forming an energy-dissipating coupling beam. This scheme usually cuts the coupling beam at mid-span and inserts a shear-type damper, so the coupling-beam deformation under earthquake concentrates at the damper; the post-yield plasticity of the damper dissipates large amounts of energy, protecting the wall limbs and the concrete part of the coupling beam from major damage (below).

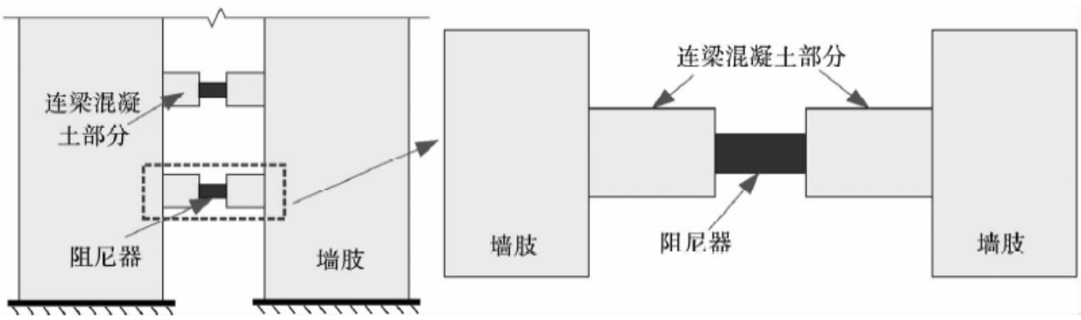


Figure 2.0.1 Schematic of the shear-wall energy-dissipation-damping scheme

The additional damping ratio provided by the damper can be determined by the method in Part 1, Chapter 3.

3 Design procedure

A shear-type damper usually yields and dissipates energy at a relatively small displacement, providing both stiffness and additional damping under frequent earthquakes. In software, an equivalent elastic wall segment can model the series shear stiffness of the damper and the pier. The additional damping ratio provided by the damper is calculated by the method in Section 5.2, and its energy-dissipation effect is modelled by raising the structural damping ratio.

3.1 Implementation in PKPM

3.1.1 Original structural model

Complete the structural model by the ordinary method, adding floor dead and live loads, partition loads on beams, etc. The model without dampers is shown in Figure 3.1.1.

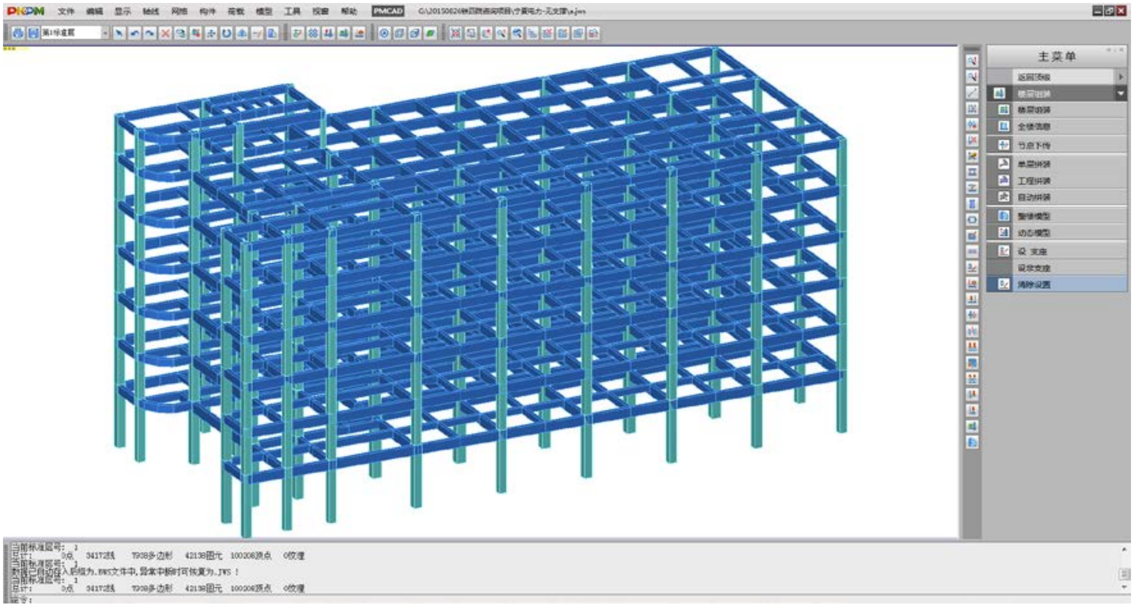


Figure 3.1.1 PKPM structural model without dampers

3.1.2 Defining the equivalent shear wall-pier

Use an ordinary concrete wall to model the series shear stiffness of the damper and the pier.

Open the [Floor Definition] menu and click [Wall Layout] to open the wall dialog. Click [New], and input the section width and material class (Figure 3.1.2). The defined wall parameters are shown in Figure 3.1.3.

Open the [Axis Input] menu, click [Node], and add nodes at the start and end of the wall. Open the [Floor Definition] menu, click [Wall Layout], select the equivalent wall, click place, and place the walls in turn at the newly added nodes. The model after placing the walls is shown in Figure 3.1.5.

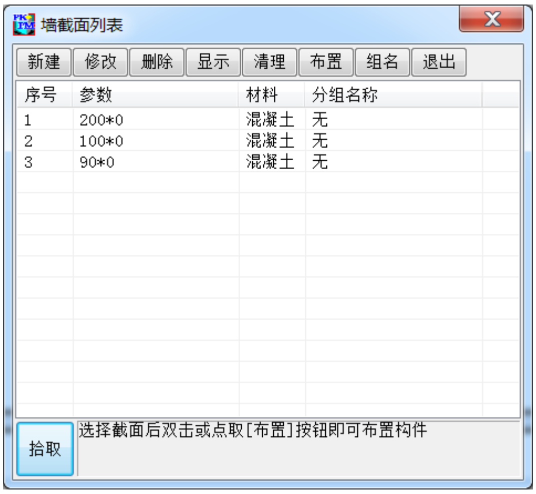


Figure 3.1.2 New equivalent-wall parameter definition Figure 3.1.3 Wall parameter list

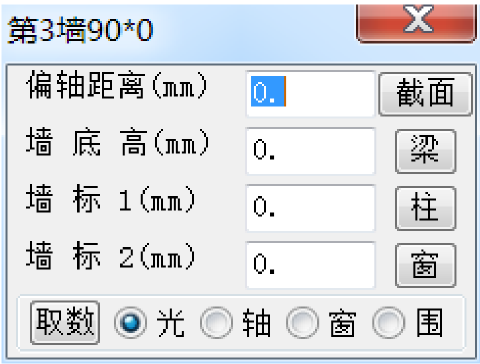


Figure 3.1.4 Wall-layout dialog

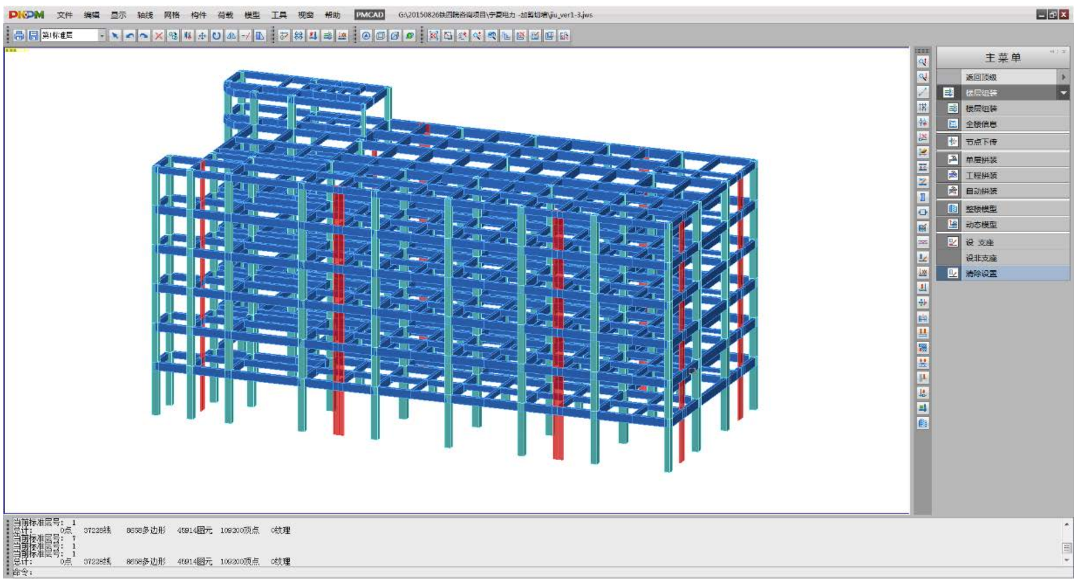


Figure 3.1.5 PKPM structural model after placing the walls

3.1.3 Calculating the additional damping ratio

The additional damping ratio of the damper under frequent earthquakes shall be calculated from the damper's actual nonlinear behaviour. The method is given in Part 1, Chapter 3; the code method or the free-vibration decay method is recommended. With the code method, the energy dissipated by the damper in one hysteresis cycle is taken as the area enclosed by one hysteresis loop. With the free-vibration decay method, the upper and lower piers are modelled as built, and the damper is modelled with a nonlinear link element of the Wen-model type.

After calculating the damper's additional damping ratio, input the sum of the original structural damping ratio and the additional damping ratio as the equivalent structure's damping ratio.

3.1.4 SATWE calculation settings

The supplementary SATWE analysis and design parameters are defined as in conventional design, only changing the damping ratio to the energy-dissipation-structure damping ratio that includes the additional damping. Click SATWE [Supplementary Definition of Analysis and Design Parameters] and define the parameters per Figures 3.1.6 and 3.1.7 (the additional damping ratio computed for this model is 2.5%, so the equivalent model's damping ratio is set to 7.5%).

3.1.

After setting, run [Generate SATWE Data File and Data Check]; once the data file is formed, perform the internal-force and reinforcement calculations and review the results of members other than the damper wall-piers. The damper wall-piers shall be designed for 1.2 times the damper's ultimate load.

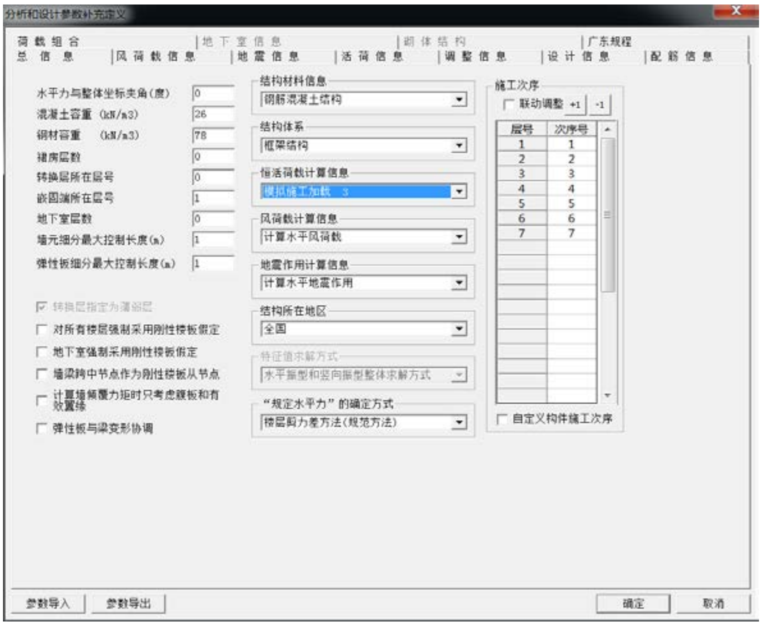


Figure 3.1.6 General-information definition

Figure 3.1.7 Seismic-information definition

3.2 Implementation in ETABS

3.2.1 Original structural model

Complete the structural model by the ordinary method; the model without damper walls is shown in Figure 3.2.1.

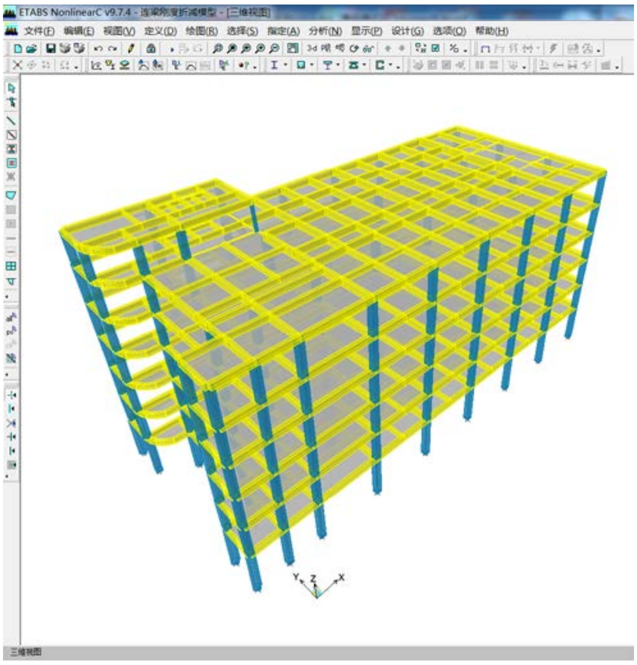


Figure 3.2.1 ETABS structural model without damper walls

3.2.2 Defining the equivalent shear wall-pier

Click ETABS [Define] > [Wall/Slab Section] to open the section dialog and choose Add New Wall. Set the wall material and thickness; the defined parameters are shown in Figure 3.2.2.

Figure 3.2.2 Equivalent-wall definition

Click ETABS [View] > [Set Elevation View], select the elevation where the damper wall is to be set, and click OK. Add points at the corresponding positions on the elevation and place the equivalent wall as in Figure 3.2.3.

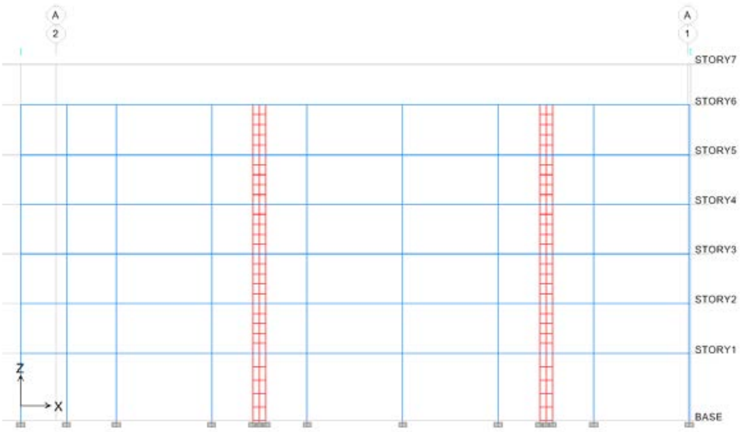


Figure 3.2.3 Elevation layout of the equivalent wall

Place the other walls in the same way; the overall model afterwards is shown in Figure 3.2.4.

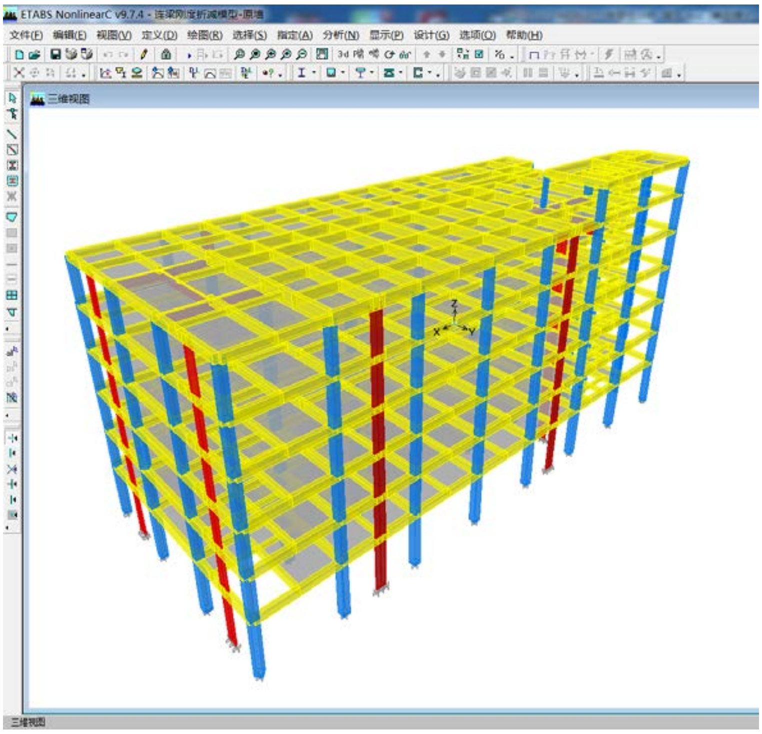


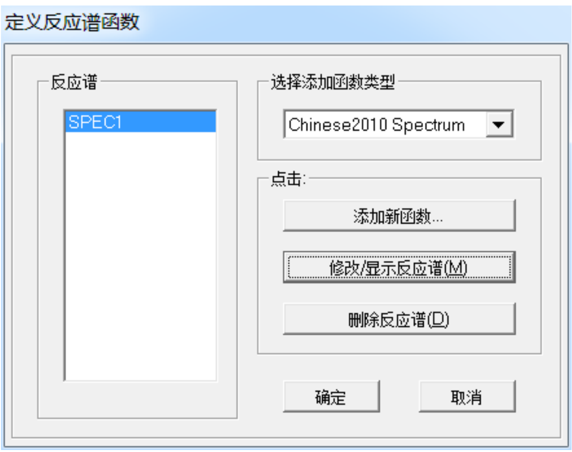
Figure 3.2.4 ETABS structural model after placing the walls

3.2.3 Calculating the additional damping ratio

Calculate the additional damping ratio of the damper wall as above, and input the sum of the original structural damping ratio and the additional damping ratio as the equivalent structure's damping ratio.

3.2.4 Response-spectrum analysis case

Click ETABS [Define] > [Response-Spectrum Function] to open the dialog; in the function-type list choose "Chinese2010 Spectrum" and click Add New Function (Figure 3.2.5). This model takes a period-reduction factor of 0.8 and a structural damping ratio of 0.075 (Figure 3.2.6).



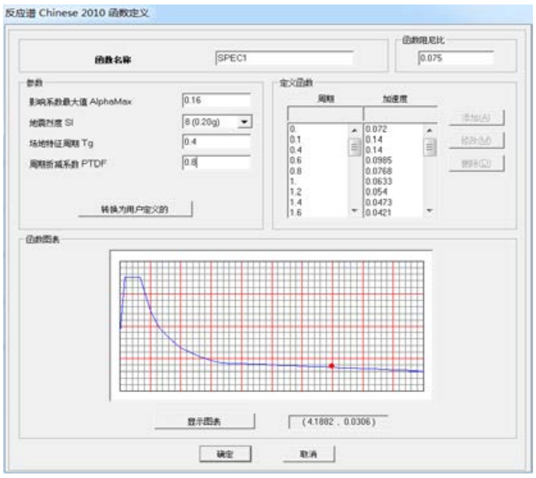


Figure 3.2.5 Adding a response-spectrum function Figure 3.2.6 Defining the response-spectrum function

Click ETABS [Define] > [Response-Spectrum Case] to open the dialog (Figure 3.2.7) and click Add Response-Spectrum Case. Set the X-direction load case Ex per Figure 3.2.8, and set the Y-direction load case Ey.

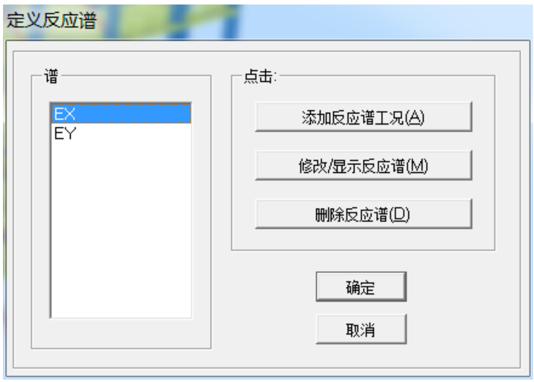


Figure 3.2.7 Adding a response-spectrum case

Figure 3.2.8 Defining the response-spectrum case

Build the ETABS model by the above steps and perform the analysis and member design for all load cases. Again, the damper wall's capacity design need not be reviewed; design the wall-piers by the same method as in PKPM.

3.3 Implementation in MIDAS

3.3.1 Original structural model

Complete the structural model by the ordinary method; the model without dampers is shown in Figure 3.3.1.

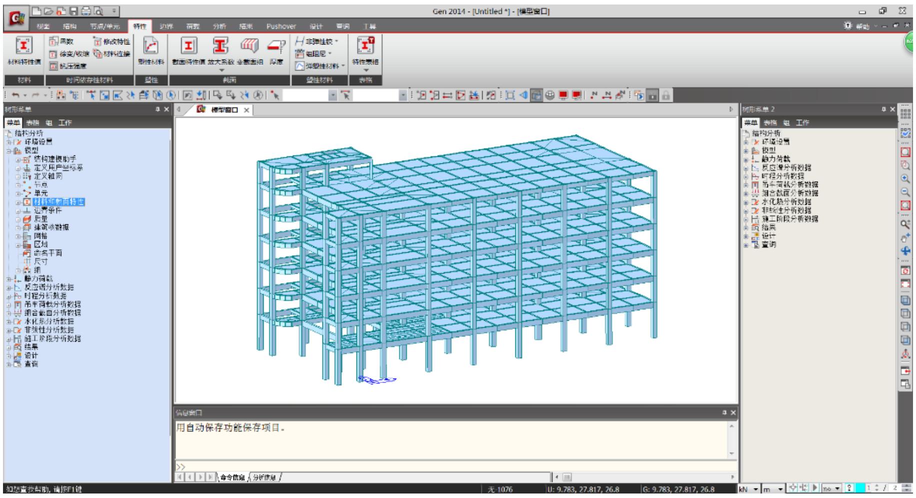


Figure 3.3.1 MIDAS structural model without dampers

3.3.2 Defining the equivalent shear wall-pier

Click MIDAS [Properties] > [Section Properties], choose the [Thickness] option, and click [Add] to open the dialog (Figure 3.3.2). Set the thickness of the equivalent shear wall-pier.

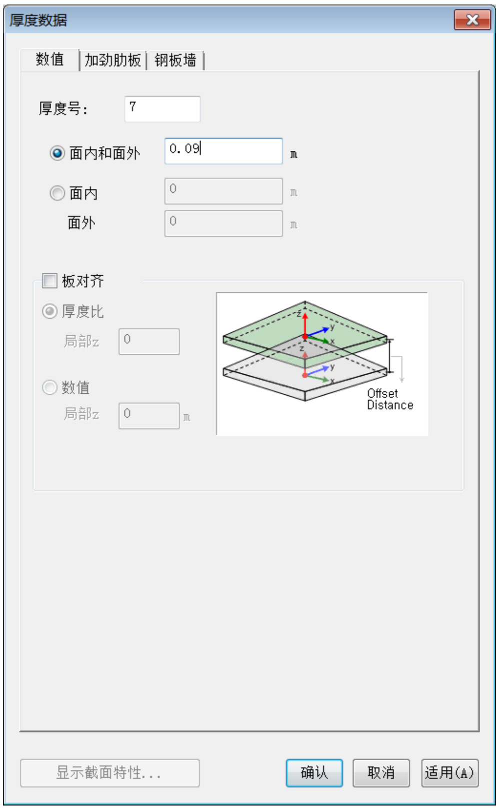


Figure 3.3.2　Equivalent-shear-wall-pier section-definition dialog

Select the elevation where the damper wall is located and place the equivalent shear wall-pier on it as in Figure 3.3.3.

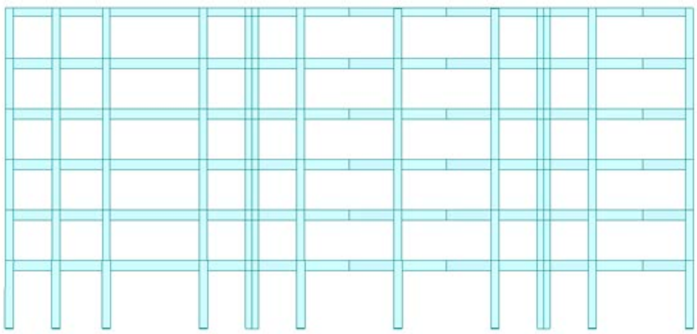


Figure 3.3.3　Elevation layout of the equivalent shear wall-pier

The overall model after placing the equivalent wall-piers is shown in Figure 3.3.4.

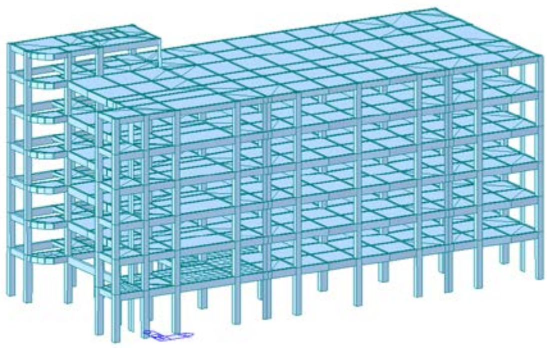


Figure 3.3.4 MIDAS structural model after placing the equivalent shear wall-piers

3.3.3 Response-spectrum analysis case

Click MIDAS [Load], select the seismic-action load type, then click [Response-Spectrum Function] to open the dialog, and click Add to open the seismic-influence-coefficient curve dialog. Click the Design-Spectrum button to open the Generate Design Spectrum dialog (Figure 3.3.5). This model takes a period-reduction factor of 0.8 and a structural damping ratio of 0.075. The defined function is shown in Figure 3.3.6.

0.07

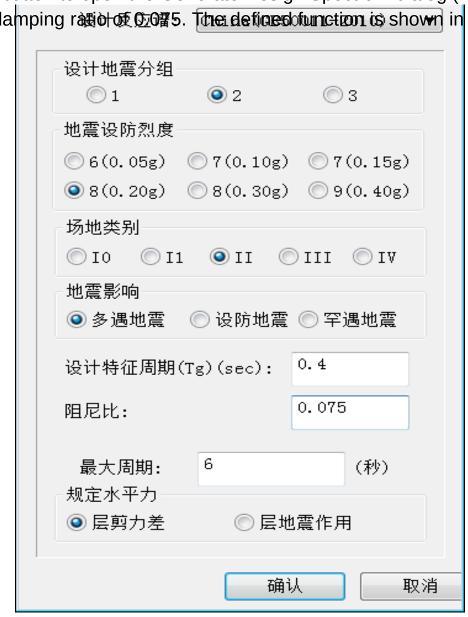


Figure 3.3.5 Defining the code response-spectrum function

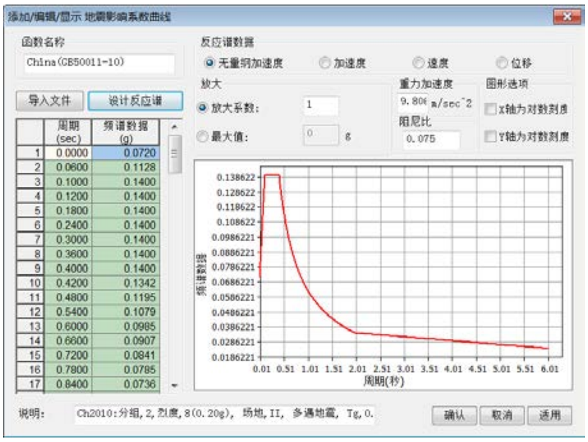


Figure 3.3.6 The defined response-spectrum function

Click MIDAS [Load] > [Response Spectrum] to open the response-spectrum-case dialog. Set the Y-direction load case Ey per Figure 3.3.7, and set the X-direction load case Ex. Set the damping ratio of each structural mode to 0.075.

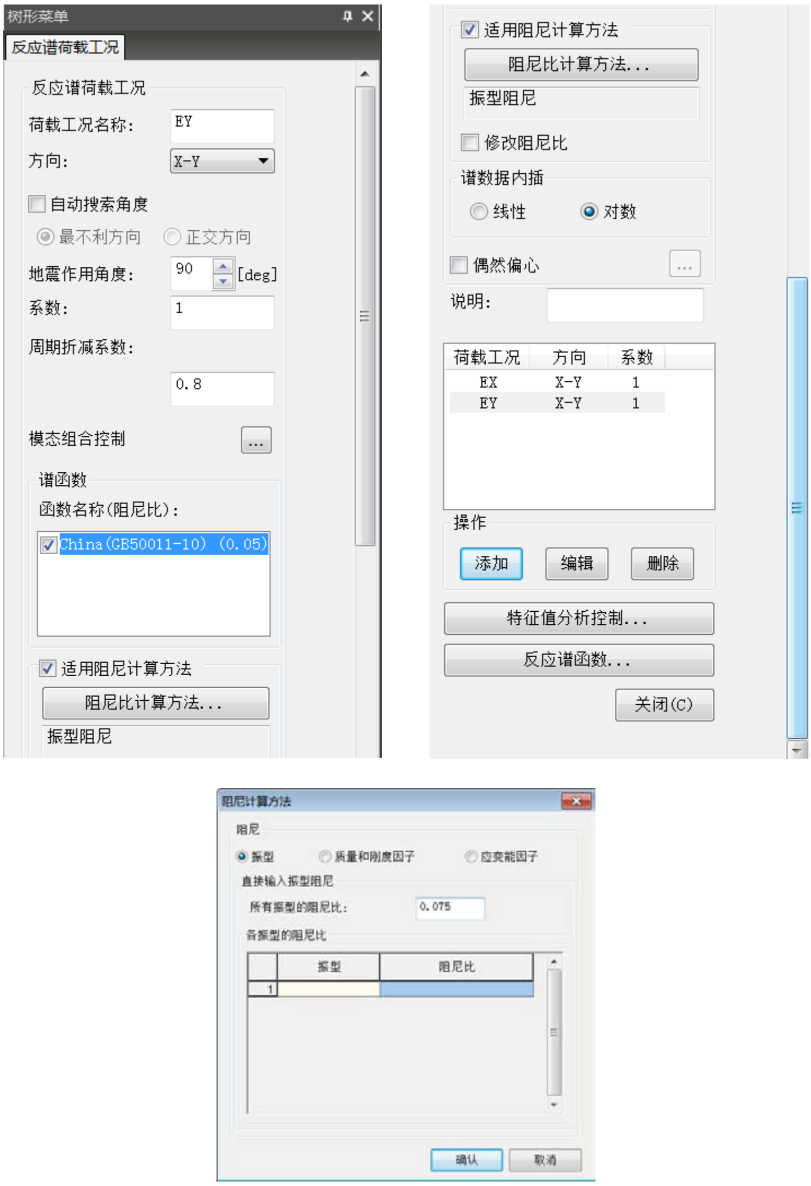


Figure 3.3.7 Response-spectrum case definition

Build the MIDAS model by the above steps and perform the analysis and member design for all load cases. Again, design the upper and lower piers of the damper by the same method as in the PKPM procedure.

Part 4 Friction Dampers

1 Basic principles

A friction damper mainly comprises a friction-pair system and a pressure-applying system. The friction pair generally uses two different materials to prevent cold welding of identical materials under long-term normal pressure and to keep the friction coefficient stable. The pressure-applying system is realised by bolt pretension.

The simplest friction damper has a three-layer construction: a central steel plate and an upper and a lower friction-material layer. It dissipates energy mainly through the friction of the central steel plate against the two friction layers; the central plate connects to the friction material by bolts, and the friction force is adjusted by tightening or loosening the bolts. The greater the bolt tightening force, the greater the friction force generated by the damper's motion.

In operation, a friction damper has two states - non-slip and slip - and its hysteresis loop is rectangular under cyclic loading. Under the Coulomb assumption, when the relative sliding velocity of the friction contact bodies is small, the total sliding friction depends only on the contact-surface condition and the force applied on the contact surface. The friction can therefore be computed with a rigid-plastic model (damping-force vs. displacement skeleton curve, Figure 1.0.1):

$$\begin{cases} K_s = K_0 = \infty & \text{before slip} \\ K_s = K_1 = 0, F_d = -\mu N \operatorname{sgn}(x) & \text{after slip} \end{cases}$$

where:

Fd—— output force of the damper;

N—— normal pressure, which varies with deformation;

μ —— friction coefficient;

x—— relative displacement between the two ends of the damper.

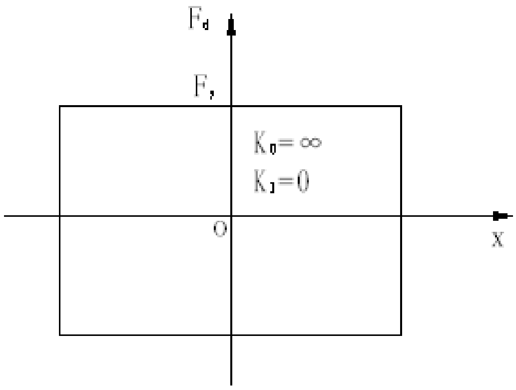


Figure 1.0.1 Rigid-plastic damping-force vs. displacement skeleton curve

The stability of a friction damper's performance depends on the stability of the friction coefficient of the friction-pair system and the constancy of the normal-pressure system. The JY-FD friction damper is a patented product of the China Academy of Building Research, incorporating a series of research innovations - a special friction material, an anti-relaxation construction for the normal-pressure system, and a friction-pair pre-running-in technology before delivery - ensuring accurate sliding friction and stable normal pressure over the whole service life of the structure.

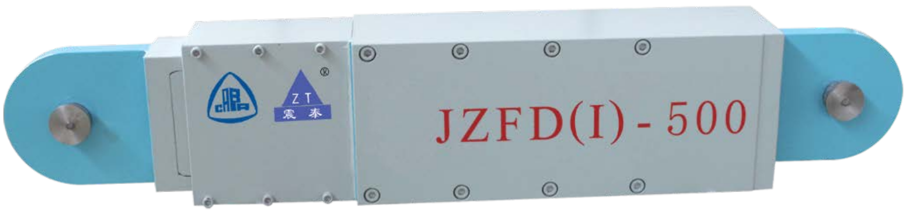


Figure 1.0.2 JY-FD friction damper

A typical hysteresis curve of the CABR friction damper is shown below.

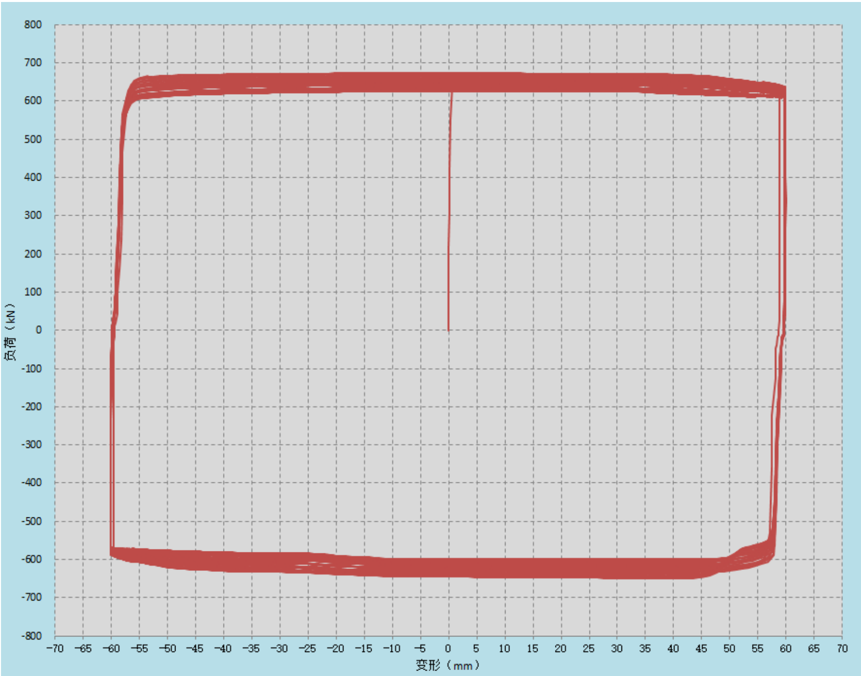


Figure 1.0.3 Hysteresis curve of the CABR friction damper

2 Design method

2.1 Sliding friction force

A friction damper can slip even under a minor earthquake, dissipating seismic energy and providing additional damping to the structure.

The sliding friction force of a friction damper is calculated as:

$$F_d = \mu N$$

F_d ——

μ —— sliding friction coefficient of the friction damper;

N —— design normal pressure of the friction damper.

2.2 Slip-initiation displacement

A sliding friction damper is generally treated by a rigid-plastic model; the slip-initiation displacement can be regarded as very small and neglected.

2.3 Ultimate displacement

The ultimate displacement of a friction damper is the maximum design displacement it can reach at this stable friction force. The ultimate displacement must be greater than 1.2 times the damper's displacement under a major earthquake.

2.4 Equivalent stiffness

The equivalent stiffness of a friction damper depends on the sliding friction force and the displacement. As the slip-initiation displacement is very small and negligible, its equivalent (secant) stiffness is:

$$k = \frac{F_d}{u_x}$$

F_d ——

F_d —— sliding friction force of the friction damper;

u_x — sliding displacement of the friction damper.

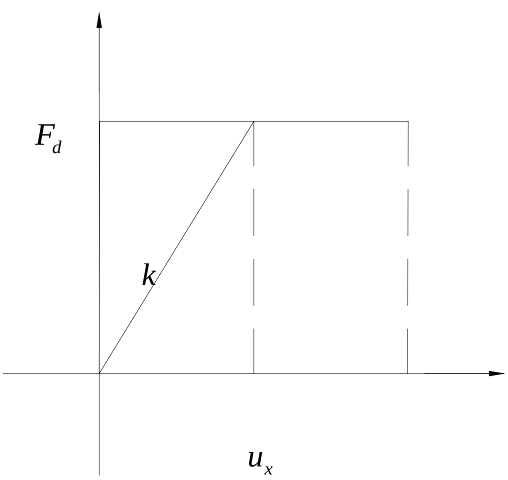


Figure 2.4.1 Equivalent stiffness of the friction damper

In PKPM, since the friction-damper displacement is unknown before analysis, the displacement of the structure without dampers can be used as the initial value to compute the damper's equivalent stiffness; perform the minor-earthquake analysis, correct the equivalent stiffness from the computed damper displacement, and re-analyse until the damper displacements of two successive runs differ sufficiently little.

2.5 Friction-damper connection design requirements

When designing a friction-damper connection, the connection force is taken as 1.2 times the damper's sliding friction force. The design methods for the various connection types follow Part 1, Section 5.5, with Nbu replaced by Fd.

3 Application example

3.1 Project overview

A data centre with a plan of 57.6 m x 84.0 m and 5 above-ground storeys, with an additional floor dead load of 6.0 kN/m2 and a floor live load of 10.0-14.0 kN/m2, uses a concrete frame with a 7.2 m x 8.4 m column grid; the seismic fortification intensity is 8 degrees. As the pure frame could not meet the code's inter-storey-drift limit, damping technology was adopted.

ETABS was used to analyse the energy-dissipation structure under frequent earthquakes considering the nonlinear behaviour of the friction dampers. The friction dampers were modelled with nonlinear link elements.

3.2 Original structural model

Complete the structural model by the ordinary method; the model without friction dampers is shown in Figure 3.2.1.

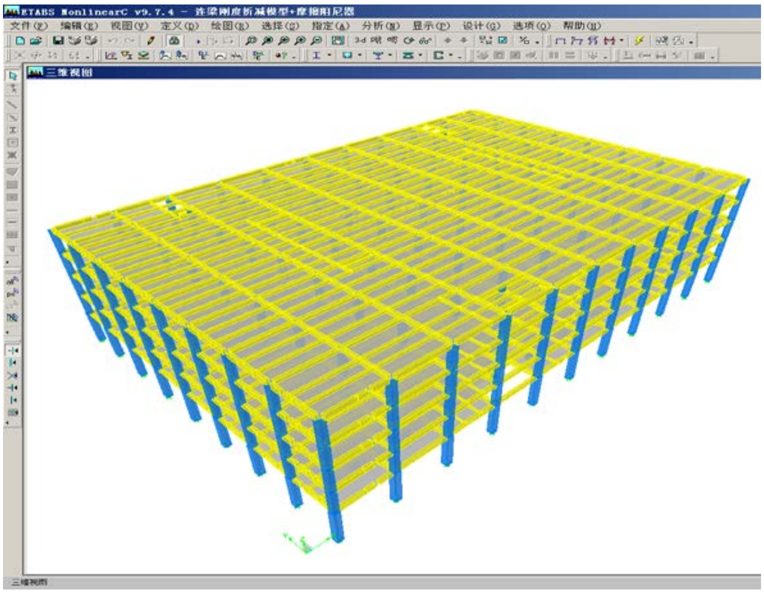


Figure 3.2.1 ETABS structural model without friction dampers

3.3 Adding friction dampers and supports

Click ETABS [Define] > [Link Properties] to open the link-properties dialog, choose link type Plastic1, and tick the U1 direction and its nonlinear behaviour (Figure 3.3.1). The defined friction-damper parameters are shown in Figure 3.3.2.

Figure 3.3.1 Nonlinear link properties

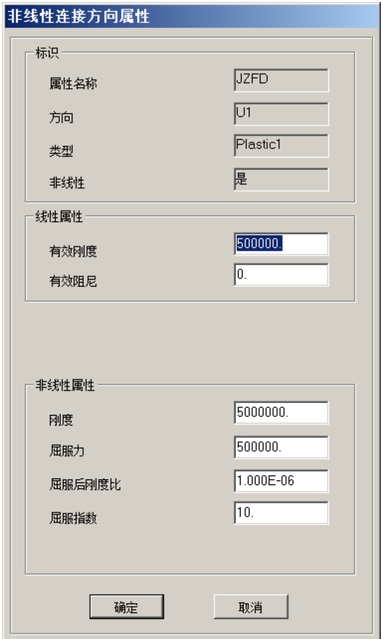


Figure 3.3.2 Friction-damper parameter definition

Click ETABS [View] > [Set Elevation View], and select the elevation where the friction damper is to be set. Add ordinary steel chevron-brace supports and friction dampers at the corresponding positions on the elevation, as in Figure 3.3.3.

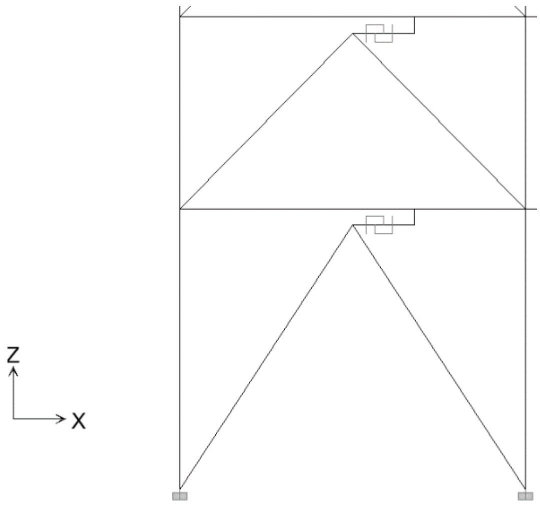


Figure 3.3.3 Elevation layout of the friction dampers

Place the other friction dampers in the same way; the overall model afterwards is shown in Figure 3.3.4.

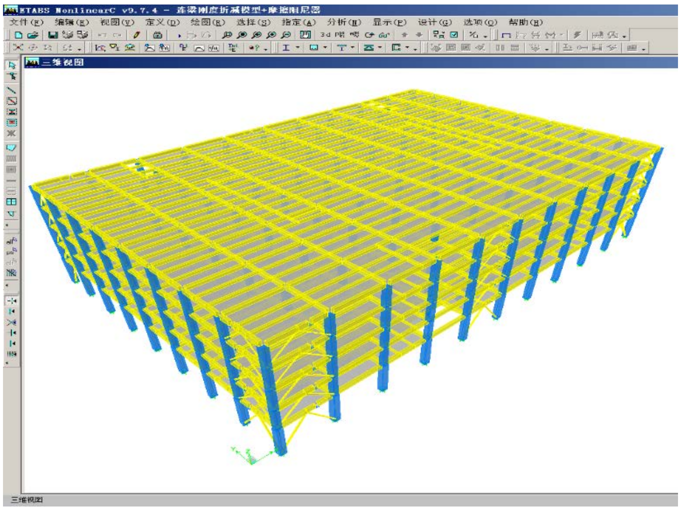


Figure 3.3.4 ETABS structural model after placing the friction dampers

3.4 Comparison of structural response under frequent earthquakes

Under frequent earthquakes, the seismic responses of the structure without dampers (uncontrolled) and with friction dampers (friction) are shown in Tables 3.4.1-3.4.2 and Figures 3.4.1-3.4.2. After adding the friction dampers, both the inter-storey drift and the column shears under frequent earthquakes are greatly reduced.

Table 3.4.1 Seismic-response comparison, X direction

Storey	Inter-storey drift			Storey shear (kN)			Sum of column shears (kN)		
	Unc.	Fric.	Ratio	Unc.	Fric.	Ratio	Unc.	Fric.	Ratio
5	1/801	1/927	0. 86	12406	11740	0. 95	12406	7836	0. 63
4	1/515	1/648	0. 79	18142	21468	1. 18	18142	13991	0. 77
3	1/395	1/591	0. 67	24159	29136	1. 21	24159	16564	0. 69
2	1/393	1/706	0. 56	30095	30373	1. 01	30095	18289	0. 61
1	1/578	1/1183	0. 49	34440	31478	0. 91	34440	21669	0. 63

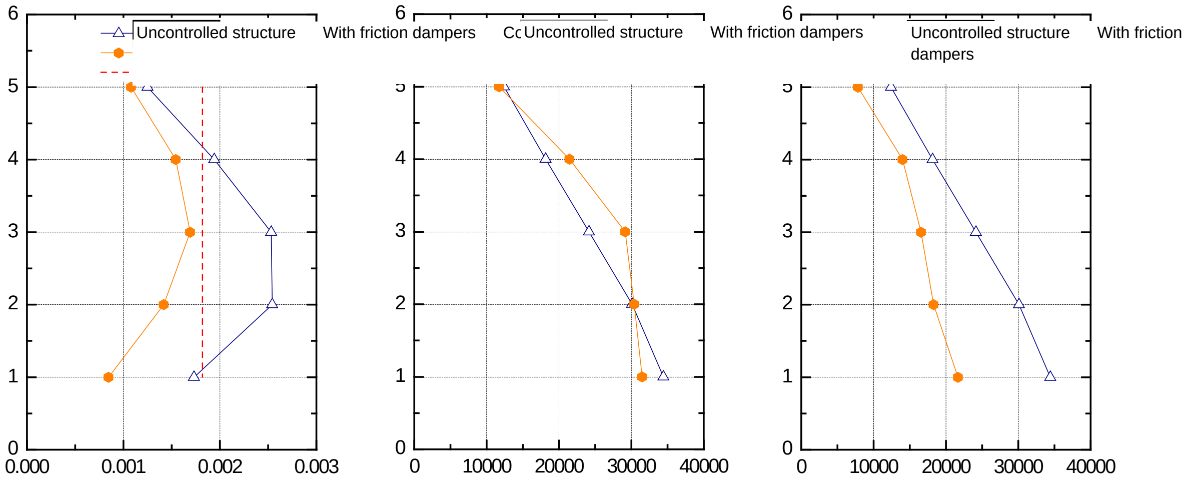
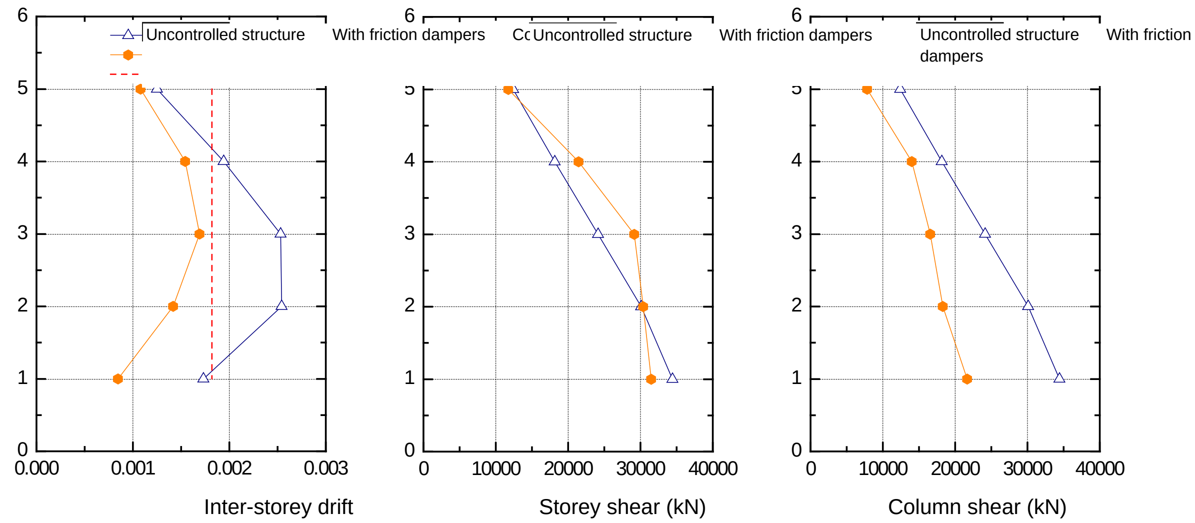


Figure 3.4.1 Seismic-response comparison, X direction

Table 3.4.2 Seismic-response comparison, Y direction

				kN)			kN)		
5	1/802	1/853	0.94	12496	12258	0.98	12496	9120	0.73
4	1/525	1/584	0.90	18161	20892	1.15	18161	14978	0.82
3	1/401	1/574	0.70	24237	27381	1.13	24237	17285	0.71
2	1/403	1/664	0.61	30135	31218	1.04	30135	20481	0.68
1	1/593	1/1005	0.59	34585	32115	0.93	34585	23941	0.69

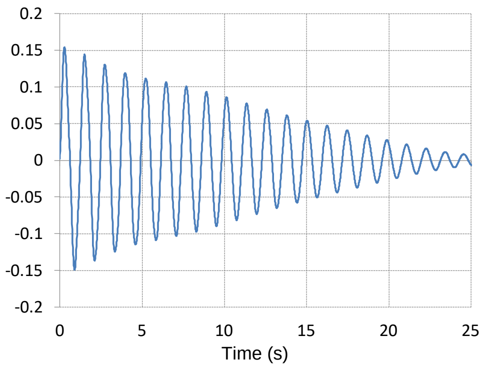


Storey Unc. Fric. Ratio Unc. Fric. Ratio Unc. Fric. Ratio

Figure 3.4.2 Seismic-response comparison, Y direction

3.5 Calculating the additional damping ratio

The structure's additional damping ratio was calculated by the free-vibration decay method. The results are shown in Figures 3.5.1-3.5.4: the additional damping ratio is 3.7% in the X direction and 3.6% in the Y direction.



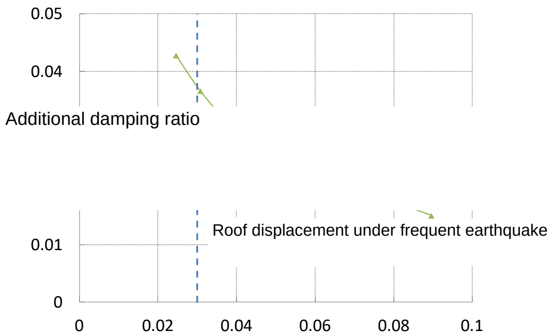


Figure 3.5.1 Free-vibration decay, X direction

Figure 3.5.2 Additional damping ratio, X direction

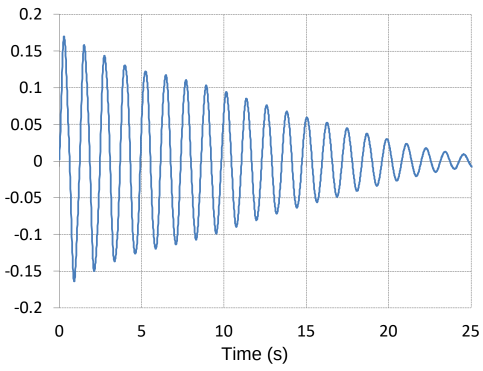


Figure 3.5.3 Free-vibration decay, Y direction

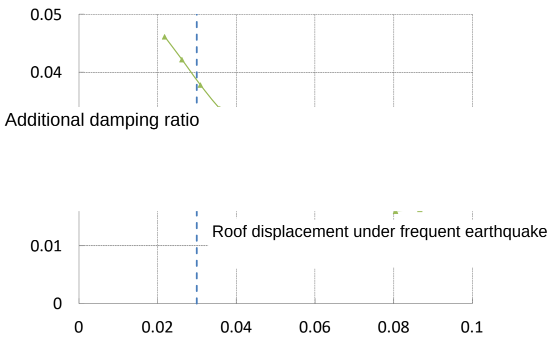


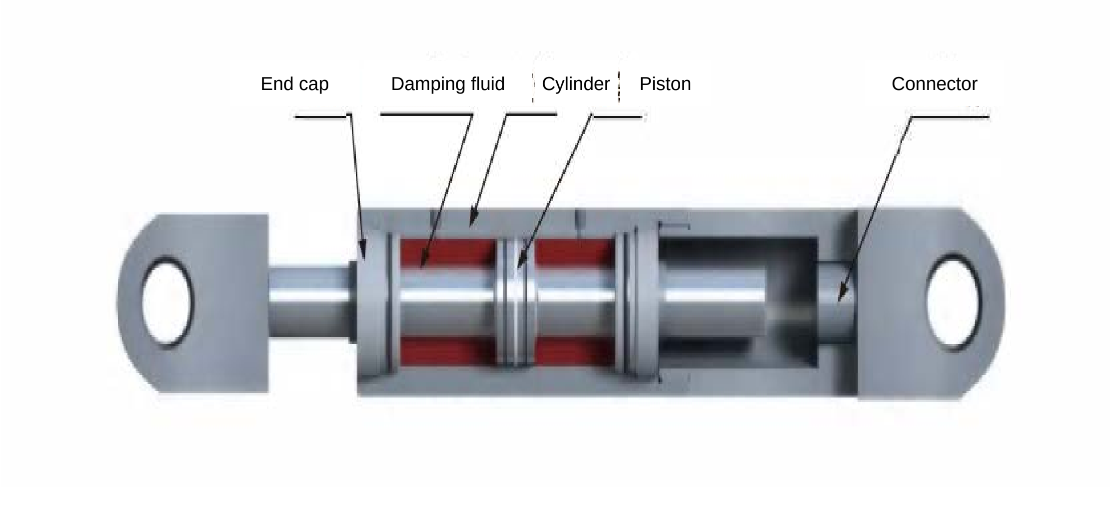
Figure 3.5.4 Additional damping ratio, Y direction

Part 5 Viscous Dampers

1 Basic principles

A viscous fluid damper is a cylinder structure filled with a damping medium. The reciprocating motion of the piston drives the internal medium to flow, producing a damping effect and converting kinetic energy into heat - an energy-dissipating seismic-control device. Viscous fluid dampers are mainly used in schools, hospitals, stadiums and tall buildings in earthquake- and typhoon-prone areas.

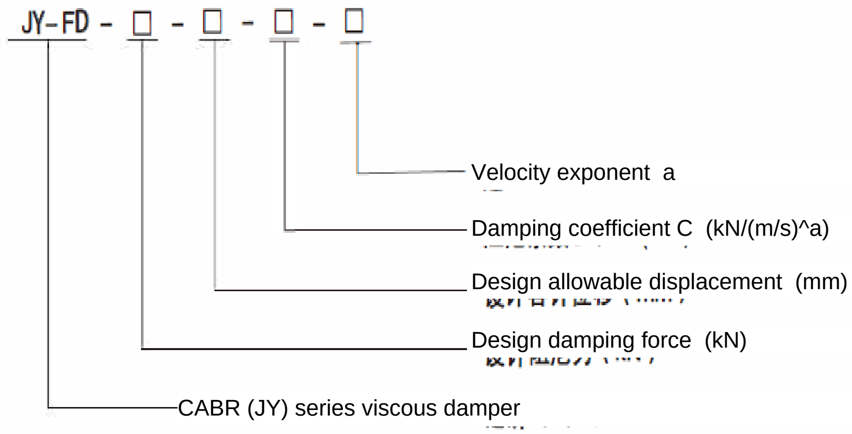
2 Working principle



A viscous fluid damper mainly consists of a piston, cylinder, end caps, damping medium and connectors. The piston divides the cylinder in two; as the piston reciprocates, the damping medium flows rapidly between the two chambers, with intense friction between the medium's molecules and between the medium and the piston, and large throttling damping as the medium passes through the piston orifices - the resultant of these actions is the damping force. This force converts seismic kinetic energy into heat through the piston's reciprocating motion in the medium, gradually slowing the piston and achieving energy dissipation.

Because a viscous fluid damper is a stiffness-free, velocity-dependent damper, it does not change the structure's inherent dynamic characteristics during operation, providing only additional damping. Its damping-force vs. displacement hysteresis loop is full and nearly rectangular, giving it stable dynamic characteristics and strong energy-dissipation capacity.

3 Damper models



Example: JY-FD-500kN-e50-1000-0.3 denotes a viscous fluid damper with design damping force 500 kN, design allowable displacement +-50 mm, damping coefficient 1000 kN(m/s)^0.3 and velocity exponent 0.3.

4 Hysteretic performance

The output force of a viscous damper vs. the relative velocity of the piston can be expressed as:

$$F = C . V^a$$

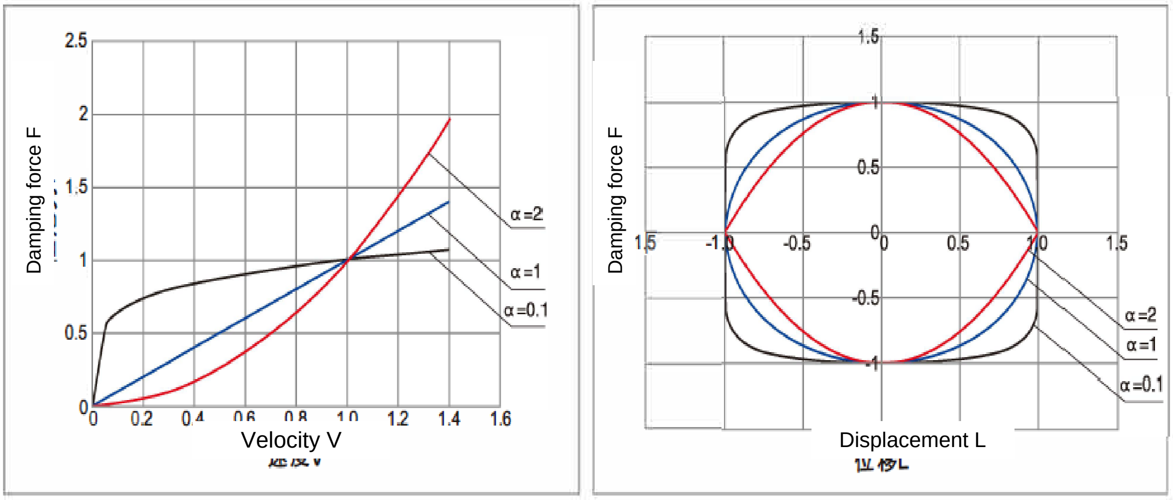
where: F - output damping force of the damper (kN);

C - damping coefficient (kN/(m/s)^a);

V - relative velocity between the damper piston and the cylinder (m/s);

a - velocity exponent;

The damping coefficient C and the velocity exponent a are important damper parameters; the velocity exponent determines the damper's energy-dissipation capacity. The damping-force vs. velocity and damping-force vs. displacement curves for different exponents are shown below.



Viscous damper performance curves

From the damping-force vs. velocity curve: at a=0.1 the damping force increases little as velocity rises; a=2 is the opposite, and a=1 lies between. From the damping-force vs. displacement curve: at a=0.1 the hysteresis loop is largest; a=2 is the opposite, and a=1 lies between. Therefore, a velocity exponent a of 0.3-0.8 is usually chosen when the structure needs energy dissipation, and a=2 when it needs load distribution/transfer.

5 Product testing

Damper products shall strictly meet all requirements of the design drawings; their mechanical performance shall meet the requirements of JG/T 209-2012 Energy-Dissipation Dampers for Buildings.

Viscous damper testing

The inspection includes:

1) Appearance test: check whether the damper's geometric dimensions match the design and meet the allowable tolerances; check for oil leakage, paint peeling, casing damage, etc.

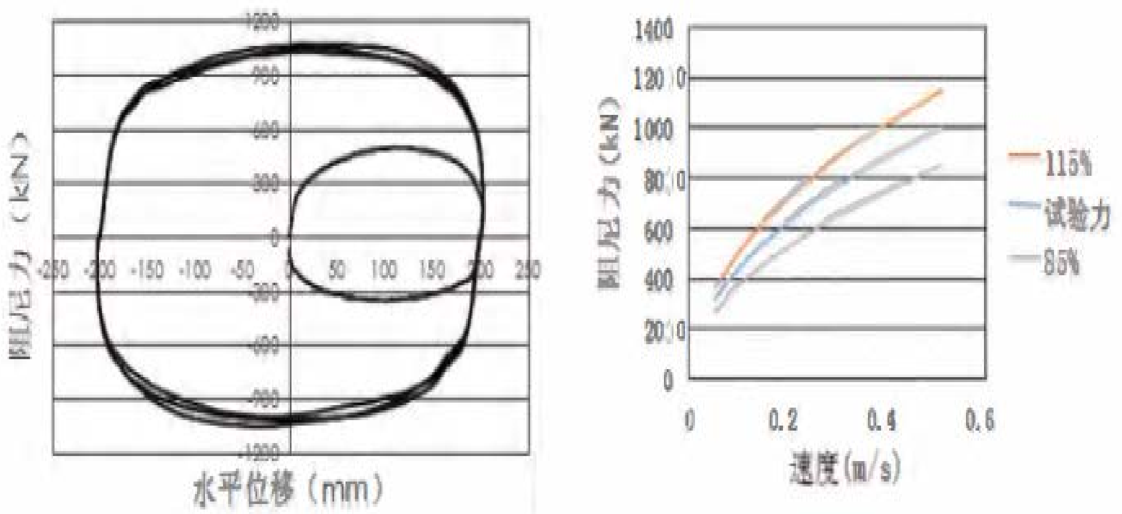
2) Ultimate-displacement test: use a static loading test, moving the damper slowly at constant speed, and record the ultimate displacement of its extension-compression motion.

3) Mechanical-performance test: every damper produced is tested on a dynamic energy-dissipation testing machine using the sinusoidal-excitation method, running 5 continuous cycles as required by the design, giving the following parameters and curves:

Hysteresis curve of displacement vs. damping force

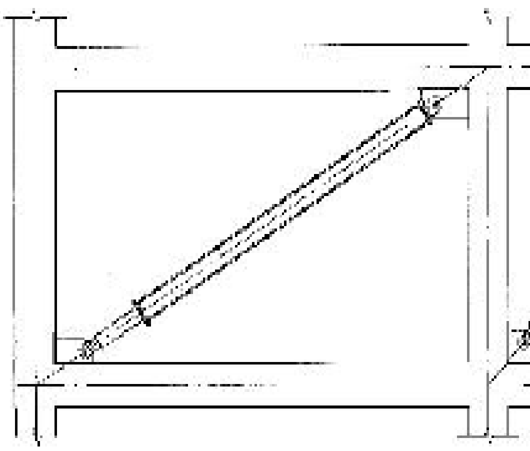
Comparison of damping force with the theoretical curve under different conditions (within 15% error)

Maximum damping force and displacement in tension and compression

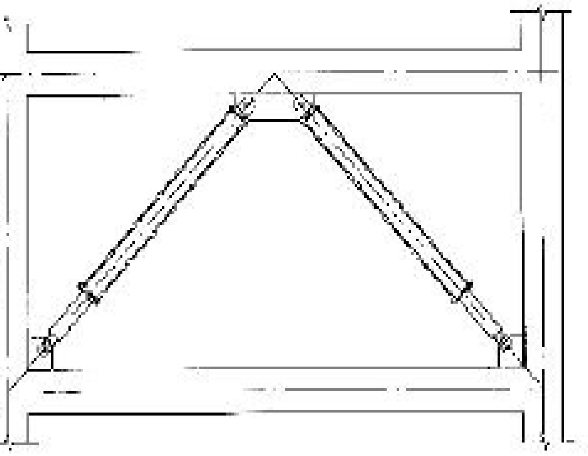


产品试验曲线

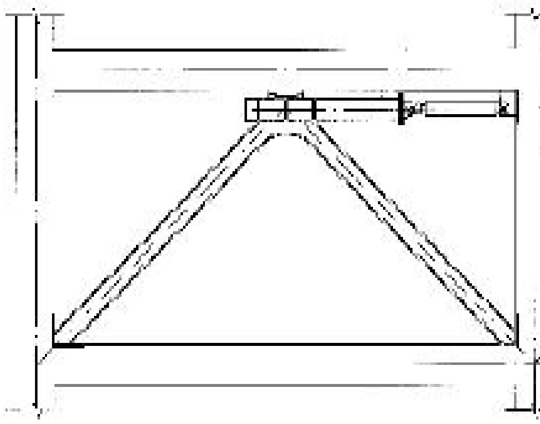
6 Common installation configurations



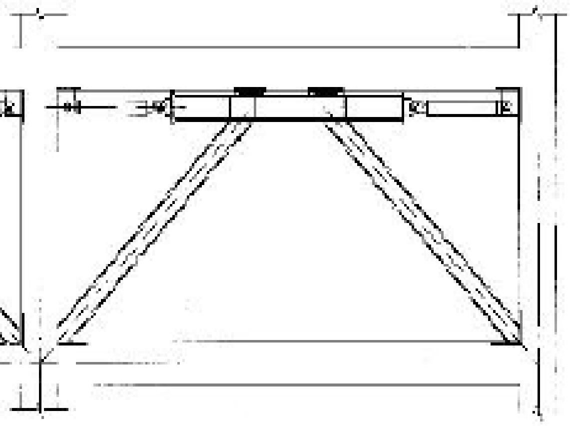
Single-diagonal type



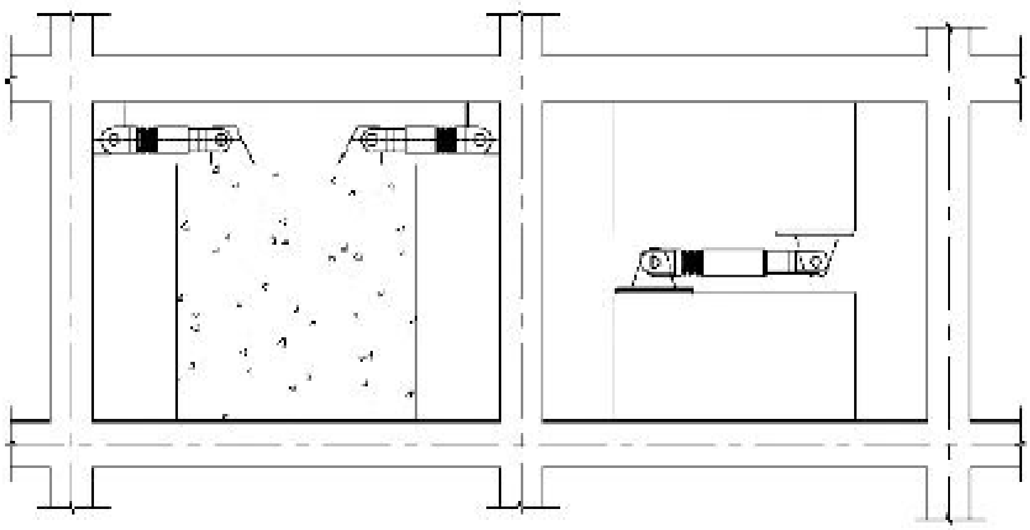
Double-diagonal type



Chevron type



Portal-brace type



Concrete-pier type

7 Maintenance and repair

A building viscous damper has a service life of at least 30 years and is maintenance-free. However, depending on the application and emergencies, the following maintenance should still be carried out: an initial inspection 1 year after installation; periodic inspections at 3, 5 and 10 years after completion and every 10 years thereafter; and a prompt inspection after a strong earthquake, strong wind, fire or other building disaster.

Inspection items:

Whether the damping medium leaks.

Whether the unpainted parts of the body surface are clean.

Periodically apply anti-rust paint to steel parts.

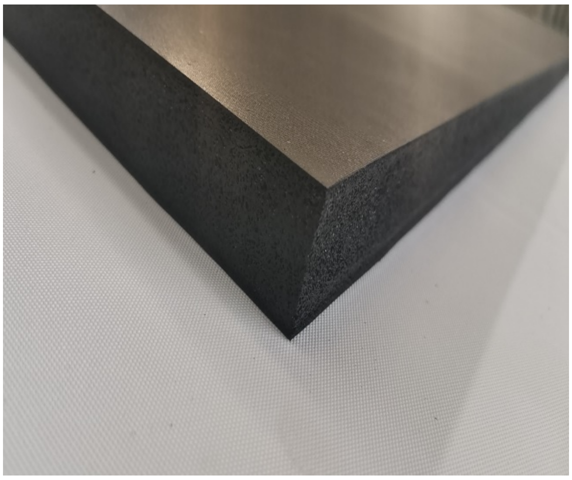
Add lubricating grease at the pin connections and promptly clean off dirt.

Use a torque wrench to check all connection bolts for looseness and tighten them.

Measure and record the damper centre-to-centre dimension; correct promptly if it exceeds the allowable range.

Whether there is a gap at the damper-to-building connection.

Part 6 EBP Elastic Vibration-Damping Pads



1 Product introduction

The development of modern metropolises generally leads with transport systems. As the backbone of urban transport, the metro carries large volumes at high speed and high frequency and has grown rapidly in many cities. Convenient as it is, the metro's foremost pollution to nearby buildings is noise and vibration.

The traditional way to reduce vibration is to stay far from the source, but with little usable above-ground space left in cities, more and more building boundaries are breaking into the 50 m safety range from the metro. Developers want to fully exploit the densely-trafficked areas around metro lines and the space above stations to maximise land use - but the vibration and noise when trains pass then become their foremost problem.

The traditional isolation trench works well but is extremely hard to maintain afterwards.

Through on-site noise-source analysis, and drawing on the principle of railway vibration-damping pads (floating the track base to block vibration transmission into the structure), it was found that laying an elastic vibration-damping pad layer under the foundation slab can effectively isolate the transmission of metro vibration and noise into the building.

Building on railway vibration-damping pads, we successfully developed the EBP (elastic bearing plate) series of elastic vibration-damping pads for building-foundation isolation, made of micro-cellular foamed rubber.

(elastic bearing plate

2 Product features

In the floating-treatment isolation method, the damping pad must be buried deep under the foundation and is not easily replaced, so the product must meet higher requirements:

1) Excellent elasticity under high load (0.1-0.5 MPa, i.e. 10-50 t/m2), with internal damping and marked vibration improvement;

2) Resistant to water and grease; maintenance-free;

3) Stable material properties; physical indices remain stable after one million fatigue cycles;

4) Ageing-resistant, with a service life over 50 years;

5) Meets environmental standards with no secondary pollution;

6) The material can be freely cut and drilled, making construction convenient with minimal impact on other trades.

In sum, thanks to its convenient construction, superior vibration-damping performance and economical cost, the EBP elastic vibration-damping pad is well suited to vibration and noise reduction for buildings along metro lines.

3 Application scope

The product series offers different thicknesses for different loads and a wide isolation-frequency range, overcoming the single-application limitation of current isolation products so it can be used in every aspect of 'building vibration and noise reduction'. It is suitable for:

Building-foundation isolation

Structural side-wall vibration damping

Floor-slab isolation

Track-bed and ballast isolation

Industrial plants, equipment and other vibration/noise-reduction applications

4 Specifications, models and performance parameters

Model	Load (MPa)	Ult. load (MPa)	Base size (mm)	Thickness (mm)
JY-EBP-005	0.005	0.02	600×12000	20
JY-EBP-010	0.10	0.2	600×12000	25
JY-EBP -150	0.15	0.3	600×12000	25
JY-EBP -200	0.20	0.4	600×12000	25
JY-EBP -250	0.25	0.5	600×12000	25
JY-EBP -300	0.30	0.6	600×12000	12.5
JY-EBP -400	0.40	0.8	600×12000	12.5
JY-EBP -500	0.50	1.0	600×12000	12.5

Beyond the load, size and thickness ranges in the table, products can be customised.

Product size range:

Length: unlimited;

Width <= 600 mm;

Thickness <= 25 mm;

JY-EBP-005 Performance Parameters

No.	Parameter	Unit	Value
1	Load range	MPa	0.05-0.1
2	Ultimate load	MPa	0．2
3	Thickness	mm	20．0±1
4	Size	mm	600×12000
5	Elastic modulus	N/mm^2	0.20-0.40
6	Dyn./static stiffness ratio	----	≤2．0
7	Hot-air ageing	Static stiffness change %	≤15%
8	Liquid-resistance change	Vol./mass/stiffness change %	Volume <=5%
			Static stiff. <=15%
			Dyn. stiff. <=30%
9	Tensile strength	MPa	≥1．5Mpa
10	Elongation at break	%	≥150%
11	Permanent compression set	% of initial compression	≤10%
12	Rebound	Rebound value	≥40
13	Ozone resistance	Cracking	No cracks

JY-EBP-100 Performance Parameters

No.	Parameter	Unit	Value
1	Load range	MPa	0.08-0.12
2	Ultimate load	MPa	0.2
3	Thickness	mm	25.0±1
4	Size	mm	600×12000
5	Elastic modulus	N/mm^2	1.50-1.80
6	Dyn./static stiffness ratio	----	≤1.60
7	Hot-air ageing	Static stiffness change %	≤15%
8	Liquid-resistance change	Vol./mass/stiffness change %	Volume <=5%
			Static stiff. <=15%
			Dyn. stiff. <=30%
9	Tensile strength	MPa	≥1.5Mpa
10	Elongation at break	%	≥150%
11	Permanent compression set	% of initial compression	≤10%
12	Rebound	Rebound value	≥40
13	Ozone resistance	Cracking	No cracks

JY-EBP-150 Performance Parameters

No.	Parameter	Unit	Value
1	Load range	MPa	0.12-0.18
2	Ultimate load	MPa	≤0.3
3	Thickness	mm	25.0±1
4	Size	mm	600×12000
5	Elastic modulus	N/mm^2	1.80-2.10
6	Dyn./static stiffness ratio	----	≤1.60
7	Hot-air ageing	Static stiffness change %	≤15%
8	Liquid-resistance change	Vol./mass/stiffness change %	Volume <=5%
			Static stiff. <=15%
			Dyn. stiff. <=30%
9	Tensile strength	MPa	≥1.5Mpa
10	Elongation at break	%	≥150%
11	Permanent compression set	% of initial compression	≤10%
12	Rebound	Rebound value	≥40
13	Ozone resistance	Cracking	No cracks

JY-EBP-200 Performance Parameters

No.	Parameter	Unit	Value
1	Load range	MPa	0.16-0.24
2	Ultimate load	MPa	≤0.4
3	Thickness	mm	25.0±1
4	Size	mm	600×12000
5	Elastic modulus	N/mm^2	2.10-2.40
6	Dyn./static stiffness ratio	----	≤1.65
7	Hot-air ageing	Static stiffness change %	≤15%
8	Liquid-resistance change	Vol./mass/stiffness change %	Volume <=5%
			Static stiff. <=15%
			Dyn. stiff. <=30%
9	Tensile strength	MPa	≥1.5Mpa
10	Elongation at break	%	≥150%
11	Permanent compression set	% of initial compression	≤10%
12	Rebound	Rebound value	≥40
13	Ozone resistance	Cracking	No cracks

JY-EBP-250 Performance Parameters

No.	Parameter	Unit	Value
1	Load range	MPa	0.2-0.3
2	Ultimate load	MPa	≤0.5
3	Thickness	mm	25.0±1
4	Size	mm	600×12000
5	Elastic modulus	N/mm^2	2.30-2.60
6	Dyn./static stiffness ratio	----	≤1.60
7	Hot-air ageing	Static stiffness change %	≤15%
8	Liquid-resistance change	Vol./mass/stiffness change %	Volume <=5%
			Static stiff. <=15%
			Dyn. stiff. <=30%
9	Tensile strength	MPa	≥1.5Mpa
10	Elongation at break	%	≥150%
11	Permanent compression set	% of initial compression	≤10%
12	Rebound	Rebound value	≥45
13	Ozone resistance	Cracking	No cracks

JY-EBP-300 Performance Parameters

No.	Parameter	Unit	Value
1	Load range	MPa	0.24-0.36
2	Ultimate load	MPa	≤0.6
3	Thickness	mm	25.0±0.5
4	Size	mm	600×12000
5	Elastic modulus	N/mm^2	2.20-2.50
6	Dyn./static stiffness ratio	---	≤1.60
7	Hot-air ageing	Static stiffness change %	≤15%
8	Liquid-resistance change	Vol./mass/stiffness change %	Volume <=5%
			Static stiff. <=15%
			Dyn. stiff. <=30%
9	Tensile strength	MPa	≥1.5Mpa
10	Elongation at break	%	≥150%
11	Permanent compression set	% of initial compression	≤10%
12	Rebound	Rebound value	≥50
13	Ozone resistance	Cracking	No cracks

JY-EBP-400 Performance Parameters

No.	Parameter	Unit	Value
1	Load range	MPa	0.32-0.48
2	Ultimate load	MPa	≤0.8
3	Thickness	mm	25.0±0.5
4	Size	mm	600×12000
5	Elastic modulus	N/mm^2	2.30-2.60
6	Dyn./static stiffness ratio	----	≤1.60
7	Hot-air ageing	Static stiffness change %	≤15%
8	Liquid-resistance change	Vol./mass/stiffness change %	Volume <=5%
			Static stiff. <=15%
			Dyn. stiff. <=30%
9	Tensile strength	MPa	≥2.5Mpa
10	Elongation at break	%	≥150%
11	Permanent compression set	% of initial compression	≤10%
12	Rebound	Rebound value	≥50
13	Ozone resistance	Cracking	No cracks

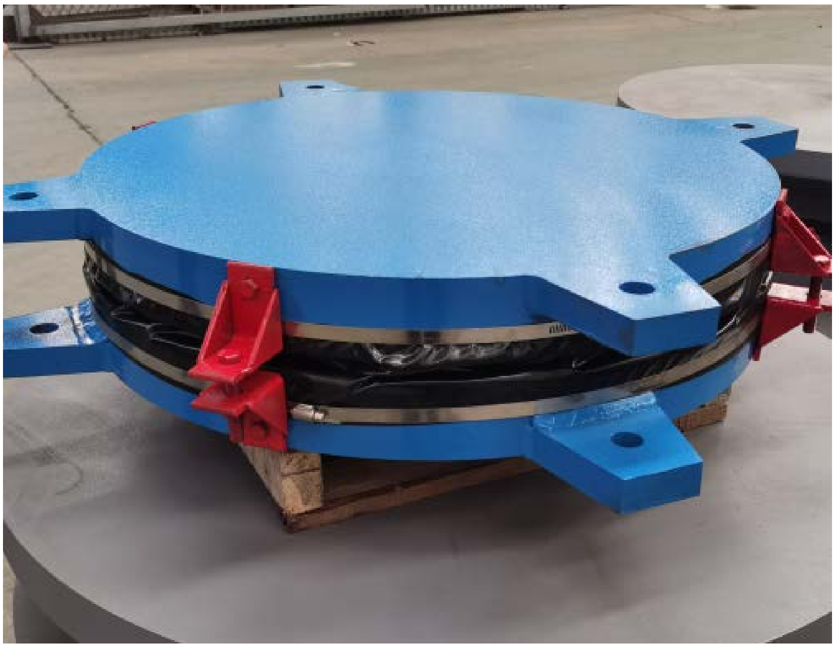
JY-EBP-500 Performance Parameters

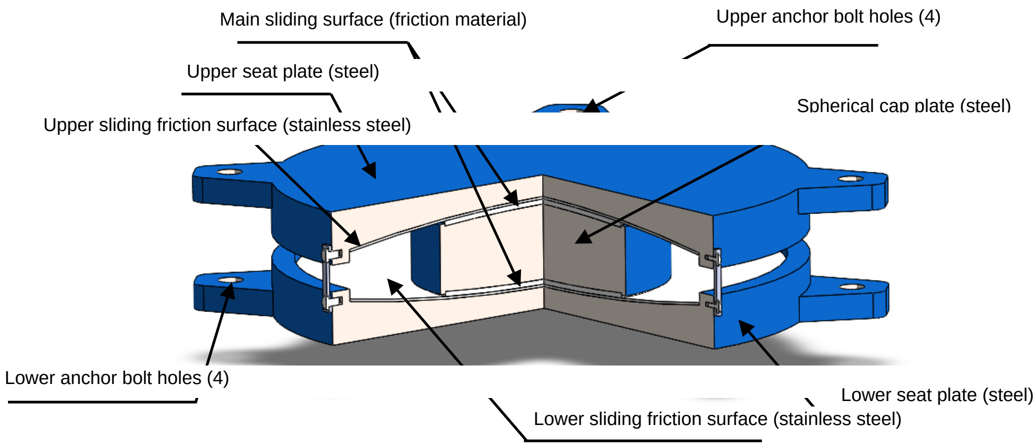
No.	Parameter	Unit	Value
1	Load range	MPa	0.4-0.6
2	Ultimate load	MPa	≤1.0
3	Thickness	mm	25.0±0.5
4	Size	mm	600×12000
5	Elastic modulus	N/mm^2	2.60-2.90
6	Dyn./static stiffness ratio	----	≤1.60
7	Hot-air ageing	Static stiffness change %	≤15%
8	Liquid-resistance change	Vol./mass/stiffness change %	Volume <=5% Static stiff. <=15% Dyn. stiff. <=30%
9	Tensile strength	MPa	≥2.5Mpa
10	Elongation at break	%	≥150%
11	Permanent compression set	% of initial compression	≤10%
12	Rebound	Rebound value	≥55
13	Ozone resistance	Cracking	No cracks

Part 7 Friction-Pendulum Isolation Bearings

4.1 Basic construction and composition

The friction pendulum bearing (FPB) is a seismic-isolation device designed on the single-pendulum principle, mainly comprising the following core parts: 1. Upper and lower connection plates - connect respectively to the superstructure (e.g. building or bridge) and the foundation, providing support and fixing. 2. Spherical-cap slider (double-spherical slider) - the core moving part, usually of metal or composite, achieving swinging and friction energy dissipation through spherical sliding. 3. Friction-material layer - covers the sliding surface, commonly PTFE, ultra-high-molecular-weight polyethylene (UHMWPE) or special metal alloy, providing stable friction performance. 4. Sealing and protection - such as rubber seal rings and stainless-steel protective layers, for dust- and corrosion-proofing and longer service life. 5. Auxiliary devices - some improved bearings add disc-spring mechanisms, viscous dampers, etc. to enhance vertical energy dissipation.

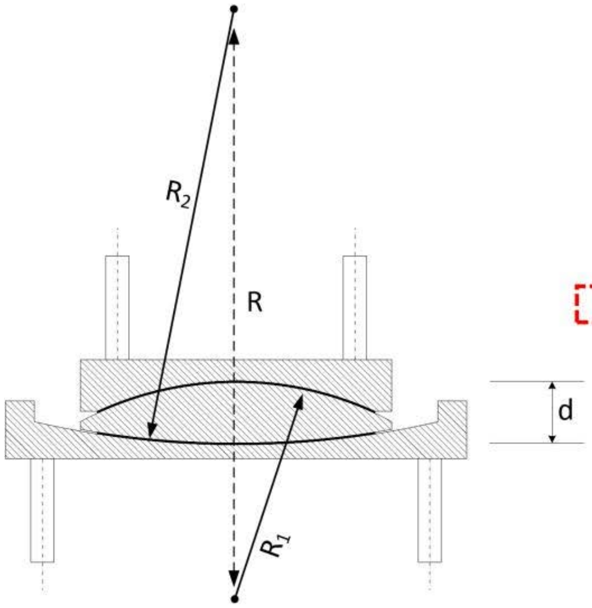



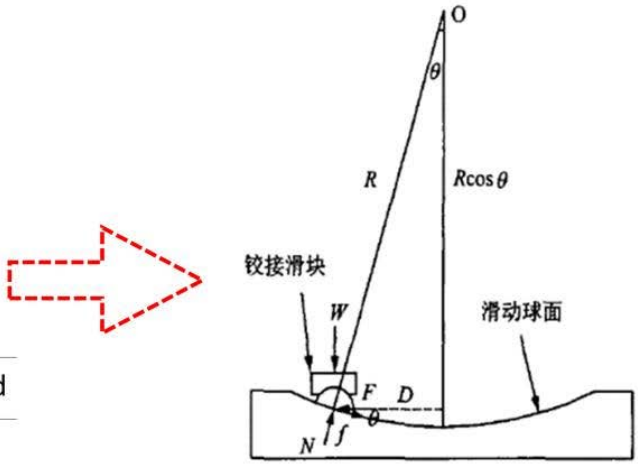


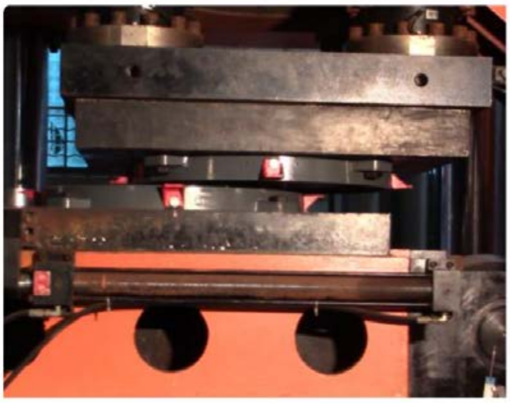
4.2 Isolation principle of the friction-pendulum bearing

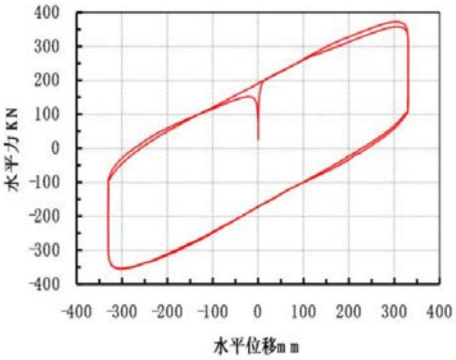
The working principle of the FPB is based on two core mechanisms: friction energy dissipation and period elongation by swinging. 1. Friction energy dissipation: when the ground shakes, the building is subjected to horizontal seismic forces. These are transmitted through the connectors to the pendulum, making it slide. During sliding, friction between the pendulum and the friction material converts the seismic energy into frictional heat. This conversion reduces the earthquake's effect on the building, achieving the damping effect.

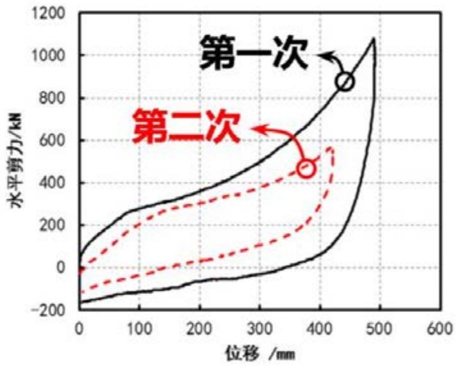
2. Period elongation by swinging: besides friction energy dissipation, another important mechanism of the FPB is to lengthen the structure's natural period by swinging. When seismic forces act on the building, the pendulum swings following the seismic waves. Because the pendulum's mass is relatively large and its motion path long, its natural period is usually greater than the building's. This period elongation lets the building better adapt to the frequency changes of seismic waves, reducing earthquake damage.









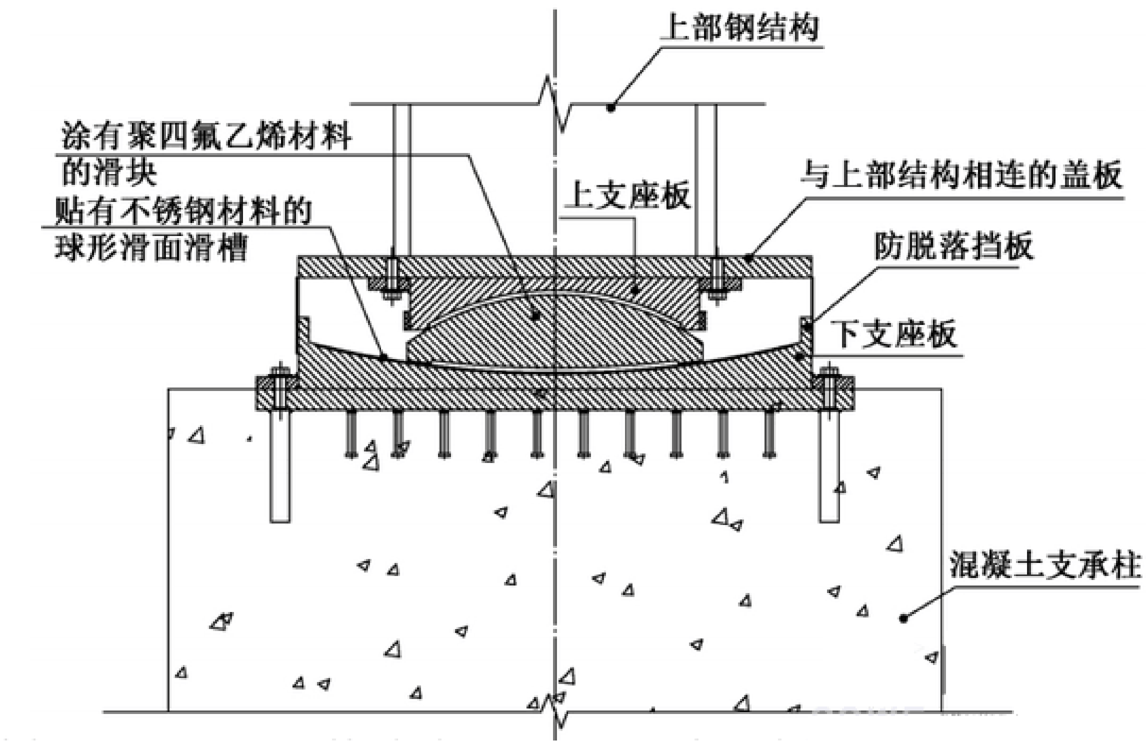


4.3 Features of the friction-pendulum isolation bearing

1. The FPB provides isolation similar to a rubber isolation bearing, with higher vertical load capacity and greater horizontal deformation capacity. 2. Its natural period is stable; the sliding surfaces are made of special metal and polymer wear-resistant materials, combining a low friction coefficient with high damping. 3. It has good self-centring capability, with its mass centre coinciding with its stiffness centre, eliminating torsion caused by eccentricity between the two. 4. Good durability and high-temperature resistance; its mechanical performance is little affected by ambient temperature.

4.4 Applications and advantages of the friction-pendulum isolation bearing

Friction-pendulum isolation bearings are widely used in tall buildings, bridges and other structures. Their advantages are mainly: 1. Significant damping effect - through the combined action of friction energy dissipation and period elongation, the FPB markedly reduces the earthquake's effect on the building; the lateral forces and horizontal vibration on the building are effectively reduced, protecting structural safety. 2. Good structural stability - because the FPB works by spherical-contact friction sliding, its friction interface is highly adaptive; even under large deformation and displacement during an earthquake, it maintains stable friction performance, ensuring structural stability. 3. Easy construction - compared with traditional damping methods, installing and commissioning the FPB is relatively simple, needing no large-scale structural modification; the bearings are simply installed between the building and the foundation, reducing the impact on the structure as well as construction difficulty and cost. 4. Easy maintenance - the FPB relies mainly on the friction material and the pendulum's motion, so only periodic replacement and inspection of the friction material is needed, which is convenient and effectively extends the bearing's service life.



Appendix 1 Sample Damper Design Specifications

Buckling-Restrained Brace Design Specification

I. Design basis

1. Code for Design of Steel Structures (GB 50017-2003);
2. Code for Welding of Steel Structures (GB 50661-2011);
3. Code for Seismic Design of Buildings (GB 50011-2010);
4. Code for Acceptance of Construction Quality of Steel Structures (GB 50205-2001);
5. Technical Specification for High-Strength Bolt Connections of Steel Structures (JGJ 82-2011);
3. Code for Seismic Design of Buildings (GB 50011-2010);
7. Energy-Dissipation Dampers for Buildings (JG/T 209-2012);
8. Technical Specification for Seismic Energy Dissipation of Buildings (JGJ 297-2013).

II. Technical requirements

1. Buckling-restrained braces may not be replaced by ordinary steel braces and shall be supplied by a specialist manufacturer. The manufacturer shall provide the MOHURD recommended-product certificate for the BRB product, which shall have proprietary intellectual property and product patents.
2. The bidding manufacturer shall provide a BRB test report whose design yield capacity meets this project's testing requirements (the report shall bear CNAS, CAL and CMA stamps). The manufacturer shall also provide an experimental research report on a concrete-frame BRB structure using its own products, to prove the product performance and damping effect.
3. This drawing is a layout of the BRBs and connectors; the brace sizes and their connection joints are schematic only. The BRBs (including connection joints) shall be detailed, fabricated and installed by a qualified party (manufacturer) and may only be installed after confirmation by the designer.
4. These BRBs are dampers; BRB sample test pieces shall be tested at the National Quality Supervision and Inspection Centre for Building Engineering (with BRB-specific testing qualification). The brace shall show stable, repeatable hysteretic behaviour, with stable, full hysteresis curves under 3 tension-compression cycles at each of 1/300, 1/200, 1/150 and 1/100 of the brace length; after 30 cycles at the 1/80 brace design displacement, the error and decay of the main design indices shall not exceed 15%, with no obvious low-cycle fatigue. The test report must bear CMA, CAL and CNAS stamps.
5. The BRB shall have good environmental properties - weather- and corrosion-resistant. Its fire rating is Grade 2 with a 2-hour fire-resistance limit. The normal maintenance service life is 50 years.
6. All gusset-plate material connected to the BRBs is Q345B.
7. Site welds are Grade 2.
8. The allowable member-length deviation is 3 mm.
9. The BRB manufacturer shall provide a construction manual and engineering-application proof, specifying the installation method, storage and maintenance conditions, etc.

10. The BRB manufacturer must provide more than 6 engineering examples using its own BRB products.

11. BRB installation shall strictly follow the relevant provisions of current national fabrication, installation and construction-acceptance codes. Matters not covered in the drawing, or changes to design requirements, may only be constructed after the designer's consent.

U-Type Damper Design Specification

I. Design basis

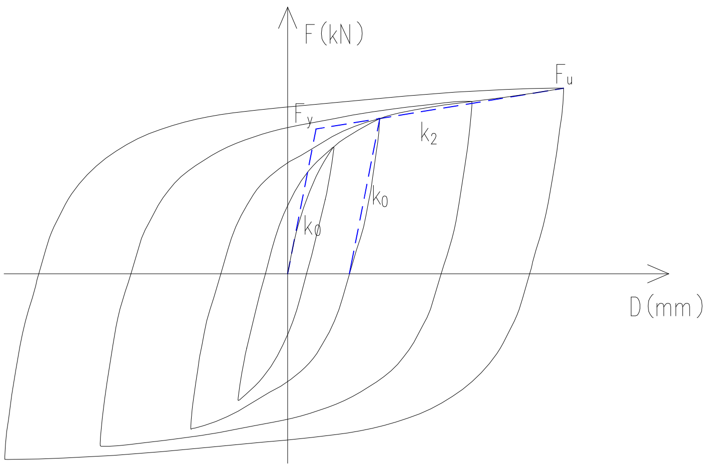
1. Code for Design of Steel Structures (GB 50017-2003);
2. Code for Welding of Steel Structures (GB 50661-2011);
3. Code for Seismic Design of Buildings (GB 50011-2010);
4. Code for Acceptance of Construction Quality of Steel Structures (GB 50205-2001);
5. Technical Specification for High-Strength Bolt Connections of Steel Structures (JGJ 82-2011);
3. Code for Seismic Design of Buildings (GB 50011-2010);
7. Energy-Dissipation Dampers for Buildings (JG/T 209-2012);
8. Technical Specification for Seismic Energy Dissipation of Buildings (JGJ 297-2013).

II. Technical requirements

1. The USD-I high-performance damper used in this project shall be supplied by a specialist manufacturer with the corresponding qualification for energy-dissipation products and the R&D and testing capability for related products.
2. When bidding, the damper manufacturer shall provide a damper inspection report (bearing CNAS, CAL and CMA stamps). The tested damper parameters shall meet this project's requirements; under 30 cycles at 60 mm displacement and 3 cycles at 72 mm, the hysteresis curve shall be full and stable, meeting JG/T 209-2012, with no fracture or failure of the damper.

3. The parameters of the USD-I high-performance damper are listed below:

型号	初始刚度 k_0	屈服力 Fy	屈服后二次刚度 k_2	极限力 Fu	消能器外观尺寸(宽度BX高度H)
USD-I型高性能消能器	44kN/mm	200kN	1kN/mm	250kN	200x290mm



4. This drawing is a layout of the USD-I damper connectors, showing the damper dimensions and connection details schematically. Connection joints not specified shall be detailed by a qualified party (manufacturer) and may only be installed after confirmation by the designer.

5. The USD-I high-performance damper shall be tested at the National Quality Supervision and Inspection Centre for Building Engineering (with engineering-testing qualification).

The damper sampling quantity shall be no less than 3% of the same specification in this project and not fewer than 2; the test report must bear CMA, CAL and CNAS stamps. The test content and requirements are:

1) Under 3 reciprocating cycles at each of 10 mm, 20 mm and 40 mm deformation, the damper shall have a stable, full hysteresis curve and positive incremental hardening stiffness.

2) Under 30 cycles at 60 mm deformation, the error and decay of the main design indices shall not exceed 15%, with no obvious low-cycle fatigue.

3) Under 3 cycles at 72 mm deformation, the ultimate capacity shall not drop by more than 15%, with no member failure.

6. The damper shall have good environmental properties - weather- and corrosion-resistant. Its fire rating is Grade 2 with a 2-hour fire-resistance limit. The normal maintenance service life is 50 years.

7. All gusset-plate and embedded-part material connected to the damper is Q345B.

8. Site welds are Grade 2.

9. The allowable joint-part deviation is 3 mm.

10. The USD-I damper manufacturer shall provide a construction manual specifying the installation method, storage and maintenance conditions, etc.

11. The damper manufacturer must provide more than 3 engineering examples using its own products and have the relevant R&D and production capability.

12. USD-I damper installation shall strictly follow the relevant provisions of current national fabrication, installation and construction-acceptance codes. Matters not covered in the drawing, or changes to design requirements, may only be constructed after the designer's consent.

Friction Damper Design Specification

I. Design basis

1. Code for Design of Steel Structures (GB 50017-2003);
2. Code for Welding of Steel Structures (GB 50661-2011);
3. Code for Seismic Design of Buildings (GB 50011-2010);
4. Code for Acceptance of Construction Quality of Steel Structures (GB 50205-2001);
5. Technical Specification for High-Strength Bolt Connections of Steel Structures (JGJ 82-2011);
3. Code for Seismic Design of Buildings (GB 50011-2010);
7. Energy-Dissipation Dampers for Buildings (JG/T 209-2012);
8. Technical Specification for Seismic Energy Dissipation of Buildings (JGJ 297-2013).

II. Technical requirements

1. The friction-damper manufacturer shall provide at least one friction-damper test report from a previous project, bearing a CMA stamp.

2. This drawing is a layout of the friction dampers and connectors; the damper sizes and their connection joints are schematic only. The friction dampers (including connection joints) shall be detailed, fabricated and installed by a qualified party (manufacturer) and may only be installed after confirmation by the designer.

4. Friction-damper sample test pieces shall be tested at the National Quality Supervision and Inspection Centre for Building Engineering (with damper-specific testing qualification). The friction damper shall show stable, repeatable hysteretic behaviour. The measured slip-force deviation of each product shall not exceed 15% of the design value. Under 30 continuous cycles at the design displacement, the max/min damping-force deviation of any cycle shall not exceed 15% of the average max/min damping force of all cycles.

5. The manufacturer must provide a friction-damper ageing test report: the deviation between two tests of the same damper at least 180 days apart shall not exceed 15%; the report must bear a CMA stamp.

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6. The friction-damper manufacturer shall provide a construction manual and engineering-application proof, specifying the installation method, storage and maintenance conditions, etc.

7. The manufacturer must provide more than 1 engineering example using its own friction-damper products.

8. Friction-damper installation shall strictly follow the relevant provisions of current national fabrication, installation and construction-acceptance codes. Matters not covered in the drawing, or changes to design requirements, may only be constructed after the designer's consent.

Q235 CABR BRB Product Technical Parameter Reference Table

Q235 Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions (mm)		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area mm^2	No. of HS bolts (one end)	Equiv. area mm^2
2-3	90	100	187.5	120	120	10	567	4M16	481
3-5				120	120	10	515	4M16	467
5-8				140	140	10	482	4M16	456
8-12				180	180	10	468	4M16	451
2-3	180	200	375	130	130	12	1193	8M16	963
3-5				130	130	12	1060	8M16	936
5-8				160	160	12	981	8M16	912
8-12				200	200	12	946	8M16	904
2-3	270	300	562.5	190	190	12	1765	8M16	1447

Q235 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b (kN)$	Yield capacity $N_{by} (kN)$	Ultimate capacity $N_{bu} (kN)$	Outer dimensions		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area	No. of HS bolts (one end)	Equiv. area mm^2
3-5				190	190	12	1579	8M16	1406
5-8				190	190	12	1466	8M16	1370
8-12				250	250	10	1414	8M16	1357
2-3				150	150	16	2592	12M20	1927
3-5	360	400	750	150	150	16	2219	12M20	1873
5-8				200	200	14	2011	12M20	1826
8-12				250	250	12	1920	12M20	1809
2-3				180	180	18	3224	12M20	2409
3-5	450	500	937.5	180	180	18	2766	12M20	2341
5-8				220	220	16	2509	12M20	2282

Q235 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area	No. of HS bolts (one end)	Equiv. area mm⁻
8-12				280	280	14	2398	12M20	2261
2-3	540	600	1125	180	180	20	3851	12M20	2890
3-5				180	180	20	3311	12M20	2809
5-8				250	250	16	3002	12M20	2740
8-12				300	300	14	2871	12M20	2714
2-3	630	700	1312.5	200	200	20	4457	12M20	3374
3-5				200	200	20	3846	12M20	3279
5-8				250	250	18	3499	12M20	3196
8-12				350	350	14	3342	12M20	3167
2-3	720	800	1500	200	200	22	5316	12M24	3855

Q235 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions b(mm)	Outer dimensions h(mm)	Gusset plate thickness (mm)	High-strength bolt friction connection: Equiv. area	High-strength bolt friction connection: No. of HS bolts (one end)	Welded/pin connection: Equiv. area mm^2
3-5				200	200	22	4497	12M24	3747
5-8				250	250	20	4049	12M24	3652
8-12				350	350	18	3856	12M24	3618
2-3	810	900	1687.5	200	200	25	5977	12M24	4335
3-5				200	200	25	5058	12M24	4213
5-8				280	280	22	4552	12M24	4108
8-12				350	350	20	4336	12M24	4069
2-3	900	1000	1875	200	200	30	6662	12M24	4810
3-5				200	200	30	5627	12M24	4676
5-8				280	280	25	5058	12M24	4563

Q235 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions b(mm)	Outer dimensions h(mm)	Gusset plate thickness (mm)	High-strength bolt friction connection: Equiv. area	High-strength bolt friction connection: No. of HS bolts (one end)	Welded/pin connection: Equiv. area mm^2
8-12				350	350	20	4809	12M24	4522
2-3	1080	1200	2250	220	220	30	7891	12M27	5777
3-5				220	220	30	6708	12M27	5616
5-8				300	300	25	6042	12M27	5477
8-12				380	380	20	5750	12M27	5428
2-3	1260	1400	2625	220	220	35	9597	12M27	6735
3-5				220	220	35	7997	12M27	6548
5-8				300	300	30	7139	12M27	6388
8-12				400	400	25	6767	12M27	6330
2-3	1440	1600	3000	250	250	35	11700	16M27	7702

Q235 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area	No. of HS bolts (one end)	Equiv. area mm^{2}
3-5				250	250	35	9449	16M27	7487
5-8				350	350	30	8310	16M27	7302
8-12				400	400	25	7824	16M27	7236
2-3	1620	1800	3375	250	250	40	12895	16M27	8545
3-5				250	250	40	10492	16M27	8334
5-8				350	350	35	9254	16M27	8136
8-12				400	400	30	8717	16M27	8059
2-3	1800	2000	3750	250	250	40	14948	16M30	9476
3-5				250	250	40	11904	16M30	9246
5-8				350	350	35	10394	16M30	9031

Q235 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions b(mm)	Outer dimensions h(mm)	Gusset plate thickness (mm)	High-strength bolt friction connection: Equiv. area	High-strength bolt friction connection: No. of HS bolts (one end)	Welded/pin connection: Equiv. area mm^2
8-12				400	400	30	9745	16M30	8947
2-3	2250	2500	4687.5	300	300	50	20778	20M30	11845
3-5				300	300	50	15666	20M30	11558
5-8				400	400	40	13288	20M30	11279
8-12				450	450	30	12279	20M30	11167
2-3	2700	3000	5625	300	300	55	22831	22M30	14192
3-5				300	300	55	18012	22M30	13854
5-8				400	400	40	15565	22M30	13515
8-12				450	450	40	14655	22M30	13410
2-3	3150	3500	6562.5	300	300	60	29509	26M30	16539

Q235 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b (kN)$	Yield capacity $N_{by} (kN)$	Ultimate capacity $N_{bu} (kN)$	Outer dimensions b(mm)	Outer dimensions h(mm)	Gusset plate thickness (mm)	High-strength bolt friction connection: Equiv. area	High-strength bolt friction connection: No. of HS bolts (one end)	Welded/pin connection: Equiv. area mm^2
3-5				300	300	60	22068	26M30	16149
5-8				400	400	40	18517	26M30	15749
8-12				500	500	35	17222	26M30	15617
2-3	3600	4000	7500	350	350	60	33037	30M30	18862
3-5				350	350	60	24967	30M30	18426
5-8				450	450	50	21247	30M30	18011
8-12				500	500	40	19679	30M30	17848
2-3	4050	4500	8437.5	350	350	60	36448	32M30	21183
3-5				350	350	60	27818	32M30	20702
5-8				450	450	50	23763	32M30	20245

Q235 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions b(mm)	Outer dimensions h(mm)	Gusset plate thickness (mm)	High-strength bolt friction connection: Equiv. area	High-strength bolt friction connection: No. of HS bolts (one end)	Welded/pin connection: Equiv. area mm^2
8-12				550	550	40	22043	32M30	20067
2-3	4500	5000	9375	350	350	60	40997	36M30	23503
3-5				350	350	60	31081	36M30	22977
5-8				450	450	50	26495	36M30	22478
8-12				550	550	40	24556	36M30	22285
2-3	5400	6000	11250	400	400	60	48945	44M30	28139
3-5				400	400	60	37189	44M30	27524
5-8				450	450	60	31987	44M30	26974
8-12				600	600	50	29636	44M30	26747
2-3	6300	7000	13125	400	400	60	55685	50M30	32773

Q235 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions b(mm)	Outer dimensions h(mm)	Gusset plate thickness (mm)	High-strength bolt friction connection: Equiv. area	High-strength bolt friction connection: No. of HS bolts (one end)	Welded/pin connection: Equiv. area mm^2
3-5				400	400	60	42854	50M30	32070
5-8				500	500	60	37067	50M30	31440
8-12				600	600	50	34408	50M30	31182
2-3	7200	8000	15000	400	400	60	68935	58M30	37405
3-5				400	400	60	50840	58M30	36614
5-8				500	500	60	43199	58M30	35904
8-12				600	600	50	39729	58M30	35617
2-3	8100	9000	16875	450	450	60	83987	64M30	42035
3-5				450	450	60	59275	64M30	41157
5-8				500	500	60	49501	64M30	40369

Q235 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area	No. of HS bolts (one end)	Equiv. area mm^2
8-12				650	650	60	45497	64M30	40080
2-3	9000	10000	18750	500	500	60	90153	72M30	46665
3-5				500	500	60	64844	72M30	45699
5-8				550	550	60	54556	72M30	44832
8-12				650	650	60	50301	72M30	44514

Q160 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions b(mm)	Outer dimensions h(mm)	Gusset plate thickness (mm)	High-strength bolt friction connection: Equiv. area	High-strength bolt friction connection: No. of HS bolts (one end)	Welded/pin connection: Equiv. area mm^2
2-3	90	100	187.5	120	120	12	830	4M16	706
3-5				120	120	12	755	4M16	687
5-8				140	140	10	708	4M16	670
8-12				180	180	10	687	4M16	664
2-3	180	200	375	130	130	18	1746	8M16	1411
3-5				130	130	18	1554	8M16	1372
5-8				160	160	16	1439	8M16	1339
8-12				200	200	12	1387	8M16	1328
2-3	270	300	562.5	190	190	18	2588	8M16	2122
3-5				190	190	18	2316	8M16	2062

Q160 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area	No. of HS bolts (one end)	Equiv. area mm^2
5-8				190	190	18	2151	8M16	2010
8-12				250	250	12	2072	8M16	1993
2-3	360	400	750	150	150	25	3800	12M20	2823
3-5				150	150	25	3254	12M20	2745
5-8				200	200	20	2949	12M20	2680
8-12				250	250	20	2820	12M20	2655
2-3	450	500	937.5	180	180	25	4709	12M20	3533
3-5				180	180	25	4049	12M20	3435
5-8				220	220	20	3674	12M20	3352
8-12				280	280	20	3518	12M20	3320

Q160 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area	No. of HS bolts (one end)	Equiv. area mm^2
2-3	540	600	1125	200	200	30	5639	12M20	4235
3-5				200	200	30	4853	12M20	4119
5-8				250	250	25	4408	12M20	4020
8-12				300	300	20	4213	12M20	3985
2-3	630	700	1312.5	220	220	30	6529	12M20	4946
3-5				220	220	30	5640	12M20	4809
5-8				250	250	25	5129	12M20	4691
8-12				350	350	20	4904	12M20	4650
2-3	720	800	1500	220	220	30	7740	12M24	5656
3-5				220	220	30	6573	12M24	5498

Q160 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions b(mm)	Outer dimensions h(mm)	Gusset plate thickness (mm)	High-strength bolt friction connection: Equiv. area	High-strength bolt friction connection: No. of HS bolts (one end)	Welded/pin connection: Equiv. area mm^2
5-8				280	280	25	5919	12M24	5363
8-12				350	350	20	5633	12M24	5314
2-3	810	900	1687.5	220	220	40	8771	12M24	6344
3-5				220	220	40	7420	12M24	6172
5-8				280	280	30	6666	12M24	6030
8-12				350	350	25	6347	12M24	5977
2-3	900	1000	1875	250	250	40	9679	12M24	7055
3-5				250	250	40	8216	12M24	6862
5-8				280	280	30	7386	12M24	6702
8-12				350	350	30	7059	12M24	6638

Q160 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b (kN)$	Yield capacity $N_{by} (kN)$	Ultimate capacity $N_{bu} (kN)$	Outer dimensions b(mm)	Outer dimensions h(mm)	Gusset plate thickness (mm)	High-strength bolt friction connection: Equiv. area	High-strength bolt friction connection: No. of HS bolts (one end)	Welded/pin connection: Equiv. area mm²
2-3	1080	1200	2250	220	220	30	11166	12M27	8498
3-5				220	220	30	9659	12M27	8258
5-8				300	300	20	8672	12M27	8052
8-12				380	380	20	8358	12M27	7974
2-3	1260	1400	2625	250	250	30	13310	12M27	9919
3-5				250	250	30	11401	12M27	9637
5-8				300	300	20	10150	12M27	9395
8-12				400	400	20	9771	12M27	9304
2-3	1440	1600	3000	250	250	30	16263	16M27	11339
3-5				250	250	30	13498	16M27	11017

Q160 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area	No. of HS bolts (one end)	Equiv. area mm^2
5-8				350	350	20	11845	16M27	10738
8-12				400	400	20	11314	16M27	10634
2-3	1620	1800	3375	250	250	35	17727	16M27	12444
3-5				250	250	35	14868	16M27	12162
5-8				350	350	25	13153	16M27	11881
8-12				400	400	20	12467	16M27	11780
2-3	1800	2000	3750	250	250	35	20378	16M30	13807
3-5				250	250	35	16810	16M30	13498
5-8				350	350	25	14729	16M30	13193
8-12				400	400	20	13905	16M30	13083

Q160 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area	No. of HS bolts (one end)	Equiv. area mm^2
2-3	2250	2500	4687.5	300	300	40	27235	20M30	17239
3-5				300	300	40	21733	20M30	16858
5-8				400	400	30	18730	20M30	16487
8-12				450	450	30	17737	20M30	16367
2-3	2700	3000	5625	300	300	40	30485	22M30	20642
3-5				300	300	40	25165	22M30	20196
5-8				400	400	30	22125	22M30	19766
8-12				450	450	30	21074	22M30	19624
2-3	3150	3500	6562.5	300	300	50	39299	26M30	24101
3-5				300	300	50	30877	26M30	23576

Q160 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_b(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area	No. of HS bolts (one end)	Equiv. area mm^2
5-8				400	400	40	26595	26M30	23076
8-12				500	500	30	24720	26M30	22881
2-3	3600	4000	7500	350	350	50	43633	30M30	27504
3-5				350	350	50	34776	30M30	26914
5-8				450	450	40	30109	30M30	26354
8-12				500	500	30	28044	30M30	26137
2-3	4050	4500	8437.5	350	350	50	47747	32M30	30905
3-5				350	350	50	38575	32M30	30251
5-8				450	450	40	33563	32M30	29632
8-12				550	550	30	31322	32M30	29393

Q160 CABR BRB Product Technical Parameter Reference Table

Brace length	Design capacity $N_{b}(kN)$	Yield capacity $N_{by}(kN)$	Ultimate capacity $N_{bu}(kN)$	Outer dimensions		Gusset plate thickness (mm)	High-strength bolt friction connection		Welded/pin connection
				b(mm)	h(mm)		Equiv. area	No. of HS bolts (one end)	Equiv. area mm^2
2-3	4500	5000	9375	350	350	60	56233	36M30	34365
3-5				350	350	60	44128	36M30	33632
5-8				450	450	50	38107	36M30	32943
8-12				550	550	40	35510	36M30	32679

CABR Shear-Type Damper Parameter Reference Table

Damper model	Energy-dissipating steel	Yield capacity $N_{by}(kN)$	Yield displacement	Outer dimensions (mm)		
				L(mm)	**b(mm)**	**h(mm)**
JY-SSD-100	Q100LY	100	0.38	340	200	500
JY-SSD-150	Q100LY	150	0.36	445	200	500
JY-SSD-200	Q100LY	200	0.36	465	200	500
JY-SSD-250	Q100LY	250	0.35	555	200	500
JY-SSD-300	Q100LY	300	0.36	555	200	500
JY-SSD-350	Q100LY	350	0.35	625	200	500
JY-SSD-400	Q100LY	400	0.37	555	200	500
JY-SSD-450	Q100LY	450	0.36	605	200	500
JY-SSD-500	Q100LY	500	0.36	600	200	500
JY-SSD-550	Q100LY	550	0.36	650	200	500
JY-SSD-600	Q100LY	600	0.35	660	200	500
JY-SSD-650	Q100LY	650	0.36	590	200	500
JY-SSD-700	Q100LY	700	0.35	625	200	500
JY-SSD-750	Q100LY	750	0.35	660	200	500
JY-SSD-800	Q100LY	800	0.36	600	200	500
JY-SSD-850	Q100LY	850	0.36	630	200	500
JY-SSD-900	Q100LY	900	0.35	660	200	500
JY-SSD-950	Q100LY	950	0.35	690	200	500
JY-SSD-1000	Q100LY	1000	0.35	720	200	500

CABR Shear-Type Damper Parameter Reference Table

Damper model	Energy-dissipating steel	Yield capacity $N_{by}(kN)$	Yield displacement	Outer dimensions (mm)		
				L(mm)	**b(mm)**	**h(mm)**
JY-SSD-100	Q160LY	100	0.66	255	200	500
JY-SSD-150	Q160LY	150	0.61	325	200	500
JY-SSD-200	Q160LY	200	0.59	390	200	500
JY-SSD-250	Q160LY	250	0.57	460	200	500
JY-SSD-300	Q160LY	300	0.59	445	200	500
JY-SSD-350	Q160LY	350	0.57	500	200	500
JY-SSD-400	Q160LY	400	0.59	480	200	500
JY-SSD-450	Q160LY	450	0.58	525	200	500
JY-SSD-500	Q160LY	500	0.61	460	200	500
JY-SSD-550	Q160LY	550	0.60	490	200	500
JY-SSD-600	Q160LY	600	0.59	525	200	500
JY-SSD-650	Q160LY	650	0.55	580	200	500
JY-SSD-700	Q160LY	700	0.55	615	200	500
JY-SSD-750	Q160LY	750	0.55	650	200	500
JY-SSD-800	Q160LY	800	0.56	575	200	500
JY-SSD-850	Q160LY	850	0.56	600	200	500
JY-SSD-900	Q160LY	900	0.56	625	200	500
JY-SSD-950	Q160LY	950	0.55	655	200	500
JY-SSD-1000	Q160LY	1000	0.57	575	200	500

CABR Shear-Type Damper Parameter Reference Table

Damper model	Energy-dissipating steel	Yield capacity $N_{by}(kN)$	Yield displacement	Outer dimensions (mm)		
				L(mm)	b(mm)	h(mm)
JY-SSD-100	Q235	100	1.16	225	200	500
JY-SSD-150	Q235	150	1.08	275	200	500
JY-SSD-200	Q235	200	1.14	275	200	500
JY-SSD-250	Q235	250	1.10	310	200	500
JY-SSD-300	Q235	300	1.06	350	200	500
JY-SSD-350	Q235	350	1.04	390	200	500
JY-SSD-400	Q235	400	1.08	365	200	500
JY-SSD-450	Q235	450	1.06	395	200	500
JY-SSD-500	Q235	500	1.04	425	200	500
JY-SSD-550	Q235	550	1.03	460	200	500
JY-SSD-600	Q235	600	1.07	425	200	500
JY-SSD-650	Q235	650	1.05	455	200	500
JY-SSD-700	Q235	700	1.04	480	200	500
JY-SSD-750	Q235	750	0.97	525	200	500
JY-SSD-800	Q235	800	0.97	550	200	500
JY-SSD-850	Q235	850	1.01	465	200	500
JY-SSD-900	Q235	900	1.00	485	200	500
JY-SSD-950	Q235	950	0.99	505	200	500
JY-SSD-1000	Q235	1000	0.99	525	200	500

CABR Damper Parameter Reference Table

		$N_{by}\,(kN)$			(mm)		
					L(mm)	**b(mm)**	**h(mm)**
JY-HDD-100	Q225	100	72	60	400	200	290
JY- HDD -150	Q225	150	72	60	400	300	290
JY- HDD -200	Q225	200	72	60	800	200	290
JY- HDD -300	Q225	300	72	60	800	300	320
JY- HDD -400	Q225	400	72	60	1500	200	320
JY- HDD -500	Q225	500	72	60	1200	200	290

JZ-F

					b(mm)	h(mm)
JZFD-100	100	90	70	1300	250	350
JZFD-200	200	90	70	1300	250	350
JZFD-300	300	90	70	1300	250	350
JZFD-400	400	90	70	1300	250	350
JZFD-500	500	90	70	1500	250	350
JZFD-600	600	90	70	1500	250	350
JZFD-700	700	90	70	1500	250	350
JZFD-800	800	90	70	1500	250	350
JZFD-800	900	90	70	1500	250	350
JZFD-800	1000	90	70	1500	250	350

Appendix 3 Friction-Pendulum Isolation Bearing Model Tables

Design vertical capacity /kl	Model	Vertical stiffness (kN/mm)	Dyn. friction coef. lower (slo	Dyn. friction coef. upper (fast)	Equiv. radius of curvature /mm	Design horizontal displ. /mm
kN					mm	mm
2000	FPS-Ⅰ-2000- 100	1100~1700	0.01~0.06	0.02~0.08	2000/2500/3000	±100
	FPS-Ⅰ-2000- 200	1100~1700	0.01~0.06	0.02~0.08	2000/2500/3000	±200
	FPS-Ⅰ-2000- 300	1100~1700	0.01~0.06	0.02~0.08	2000/2500/3000	±300
3000	FPS-Ⅰ-3000- 100	1700~2500	0.01~0.06	0.02~0.08	2000/2500/3000	±100
	FPS-Ⅰ-3000- 200	1700~2500	0.01~0.06	0.02~0.08	2000/2500/3000	±200
	FPS-Ⅰ-3000- 300	1700~2500	0.01~0.06	0.02~0.08	2000/2500/3000	±300
4000	FPS-Ⅰ-4000- 100	2200~3400	0.01~0.06	0.02~0.08	2000/2500/3000	±100
	FPS-Ⅰ-4000- 200	2200~3400	0.01~0.06	0.02~0.08	2000/2500/3000	±200
	FPS-Ⅰ-4000- 300	2200~3400	0.01~0.06	0.02~0.08	2000/2500/3000	±300
5000	FPS-Ⅰ-5000- 100	2700~4200	0.01~0.06	0.02~0.08	2000/2500/3000	±100
	FPS-Ⅰ-5000- 200	2700~4200	0.01~0.06	0.02~0.08	2000/2500/3000	±200
	FPS-Ⅰ-5000- 300	2700~4200	0.01~0.06	0.02~0.08	2200/2500/3000	±300
6000	FPS-Ⅰ-6000- 100	3300~5000	0.01~0.06	0.02~0.08	2000/2500/3000	±100
	FPS-Ⅰ-6000-200	3300~5000	0.01~0.06	0.02~0.08	2000/2500/3000	±200

Friction-Pendulum Isolation Bearing Model and Performance Parameter Table

Design vertical capacity /kl kN	Model	Vertical stiffness (kN/mm)	Dyn. friction coef. lower (slo	Dyn. friction coef. upper (fast)	Equiv. radius of curvature /mm	Design horizontal displ. /mm mm
	FPS-Ⅰ-6000-300	3300~5000	0.01~0.06	0.02~0.08	2000/2500/3000	±300
7000	FPS-Ⅰ-7000-100	3800~5800	0.01~0.06	0.02~0.08	2000/2500/3000	±100
	FPS-Ⅰ-7000-200	3800~5800	0.01~0.06	0.02~0.08	2000/2500/3000	±200
	FPS-Ⅰ-7000-300	3800~5800	0.01~0.06	0.02~0.08	2000/2500/3000	±300
8000	FPS-Ⅰ-8000-100	4500~6500	0.01~0.06	0.02~0.08	2500/3000/4000	±100
	FPS-Ⅰ-8000-200	4500~6500	0.01~0.06	0.02~0.08	2500/3000/4000	±200
	FPS-Ⅰ-8000-300	4500~6500	0.01~0.06	0.02~0.08	2500/3000/4000	±300
9000	FPS-Ⅰ-9000-100	4700~7500	0.01~0.06	0.02~0.08	2500/3000/4000	±100
	FPS-Ⅰ-9000-200	4700~7500	0.01~0.06	0.02~0.08	2500/3000/4000	±200
	FPS-Ⅰ-9000-300	4700~7500	0.01~0.06	0.02~0.08	2500/3000/4000	±300
10000	FPS-Ⅰ-10000-100	5200~8000	0.01~0.06	0.02~0.08	2500/3000/4000	±100
	FPS-Ⅰ-10000-200	5200~8000	0.01~0.06	0.02~0.08	2500/3000/4000	±200
	FPS-Ⅰ-10000- 300	5200~8000	0.01~0.06	0.02~0.08	2500/3000/4000	±300

Friction-Pendulum Isolation Bearing Model and Performance Parameter Table

Design vertical capacity /kN	Model	Vertical stiffness (kN/mm)	Dyn. friction coef. lower (slow)	Dyn. friction coef. upper (fast)	Equiv. radius of curvature /mm	Design horizontal displ. /mm
2000	FPS-Ⅱ-2000-100	1100~1700	0.01~0.06	0.02~0.08	3000/4000/4500	±100
	FPS-Ⅱ-2000-200	1100~1700	0.01~0.06	0.02~0.08	3000/4000/4500	±200
	FPS-Ⅱ-2000-300	1100~1700	0.01~0.06	0.02~0.08	3000/4000/4500	±300
	FPS-Ⅱ-2000-400	1100~1700	0.01~0.06	0.02~0.08	3000/4000/4500	±400
3000	FPS-Ⅱ-3000-100	1700~2500	0.01~0.06	0.02~0.08	3000/4000/4500	±100
	FPS-Ⅱ-3000-200	1700~2500	0.01~0.06	0.02~0.08	3000/4000/4500	±200
	FPS-Ⅱ-3000-300	1700~2500	0.01~0.06	0.02~0.08	3000/4000/4500	±300
	FPS-Ⅱ-3000-400	1700~2500	0.01~0.06	0.02~0.08	3000/4000/4500	±400
4000	FPS-Ⅱ-4000-100	2200~3400	0.01~0.06	0.02~0.08	3000/4000/4500	±100
	FPS-Ⅱ-4000-200	2200~3400	0.01~0.06	0.02~0.08	3000/4000/4500	±200
	FPS-Ⅱ-4000-300	2200~3400	0.01~0.06	0.02~0.08	3000/4000/4500	±300
	FPS-Ⅱ-4000-400	2200~3400	0.01~0.06	0.02~0.08	3000/4000/4500	±400
5000	FPS-Ⅱ-5000-100	2700~4200	0.01~0.06	0.02~0.08	3000/4000/4500	±100
	FPS-Ⅱ-5000-200	2700~4200	0.01~0.06	0.02~0.08	3000/4000/4500	±200
	FPS-Ⅱ-5000-300	2700~4200	0.01~0.06	0.02~0.08	3000/4000/4500	±300

Friction-Pendulum Isolation Bearing Model and Performance Parameter Table

Design vertical capacity /kI	Model	Vertical stiffness (kN/mm)	Dyn. friction coef. lower (slov	Dyn. friction coef. upper (fast)	Equiv. radius of curvature /mm mm	Design horizontal displ. /mm
	FPS-Ⅱ-5000-400	2700~4200	0.01~0.06	0.02~0.08	3000/4000/4500	±400
6000	FPS-Ⅱ-6000-100	3300~5000	0.01~0.06	0.02~0.08	3000/4000/4500	±100
	FPS-Ⅱ-6000-200	3300~5000	0.01~0.06	0.02~0.08	3000/4000/4500	±200
	FPS-Ⅱ-6000-300	3300~5000	0.01~0.06	0.02~0.08	3000/4000/4500	±300
	FPS-Ⅱ-6000-400	3300~5000	0.01~0.06	0.02~0.08	3000/4000/4500	±400
7000	FPS-Ⅱ-7000-100	3800~5800	0.01~0.06	0.02~0.08	3000/4000/4500	±100
	FPS-Ⅱ-7000-200	3800~5800	0.01~0.06	0.02~0.08	3000/4000/4500	±200
	FPS-Ⅱ-7000-300	3800~5800	0.01~0.06	0.02~0.08	3000/4000/4500	±300
	FPS-Ⅱ-7000-400	3800~5800	0.01~0.06	0.02~0.08	3000/4000/4500	±400
8000	FPS-Ⅱ-8000-100	4500~6500	0.01~0.06	0.02~0.08	3000/4000/4500	±100
	FPS-Ⅱ-8000-200	4500~6500	0.01~0.06	0.02~0.08	3000/4000/4500	±200
	FPS-Ⅱ-8000-300	4500~6500	0.01~0.06	0.02~0.08	3000/4000/4500	±300
	FPS-Ⅱ-8000-400	4500~6500	0.01~0.06	0.02~0.08	3000/4000/4500	±400
9000	FPS-Ⅱ-9000-100	4700~7500	0.01~0.06	0.02~0.08	3000/4000/4500	±100
	FPS-Ⅱ-9000-200	4700~7500	0.01~0.06	0.02~0.08	3000/4000/4500	±200
	FPS-Ⅱ-9000-300	4700~7500	0.01~0.06	0.02~0.08	3000/4000/4500	±300

Friction-Pendulum Isolation Bearing Model and Performance Parameter Table

Design vertical capacity /kl	Model	Vertical stiffness (kN/mm)	Dyn. friction coef. lower (slov	Dyn. friction coef. upper (fast)	Equiv. radius of curvature /mm	Design horizontal displ. /mm
	FPS-Ⅱ-9000-400	4700~7500	0.01~0.06	0.02~0.08	3000/4000/4500	±400
10000	FPS-Ⅱ-10000-100	5200~8000	0.01~0.06	0.02~0.08	3000/4000/4500	±100
	FPS-Ⅱ-10000-200	5200~8000	0.01~0.06	0.02~0.08	3000/4000/4500	±200
	FPS-Ⅱ-10000-300	5200~8000	0.01~0.06	0.02~0.08	3000/4000/4500	±300
	FPS-Ⅱ-10000-400	5200~8000	0.01~0.06	0.02~0.08	3000/4000/4500	±400

Friction-Pendulum Isolation Bearing Model and Performance Param

Design vertical capacity /kl	Model	Vertical stiffness (kN/mm)	Dyn. friction coef. lower (slo\	Dyn. friction coef. upper (fast)	Equiv. radius (




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