Application of JY Series Energy Dispersion and Shock-Absorbing Technique in Super High-Rise Building

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SUMMARY:
Energy dispersion and shock-absorbing technique is one of effective methods for realizing structure performance design, metallic displacement type damper has relatively good elastic rigidity and energy dissipation performance, and it has extensive application in the field of energy dispersion and shock-absorbing technique. Super high-rise building usually needs better earthquake-resistant capability because of its height or complicated shape, component for "hard resistance" is adopted in traditional technique, by means of adopting energy dispersion and shock-absorbing technique, the earthquake resistant capability of structure can be guaranteed to be more effectively improved on the prerequisite of unchanged earthquake resistant capability level in frequently occurred earthquake, and the plastic damage mechanism and collapse resistant performance can be thoroughly changed under fortification intensity and rarely occurred earthquake.

Key words: *energy dispersion and shock-absorbing technique, metallic damper, super high-rise*

1. GENERAL

In the recent years, with the development of building toward vertical "super-rise", facade "unique" and planar "complex", higher requirements are put forward to the earthquake resistant capability of building. Accompanying the development of energy dissipation damper, the energy dispersion and shock-absorbing technique is incessantly applied in high-rise and super high-rise buildings.

2. APPLICATION OF BUCKLING RESTRAINT BRACE IN CANTILEVER TRUSS

In the frame-corewall structure of high-rise building, the spatial effect of corewall at central part and peripheral frame sometimes needs to be given scope to by setting cantilever truss between them, so as to improve the integral lateral rigidity and make the indexes of structural deformation (etc.) meet the design requirements. But is shall be pointed out: although the setting of reinforcing layer effectively reduces the horizontal displacement of structure under the effect of lateral load, it causes abrupt change in structural rigidity and internal force at the same time, that is particularly unfavorable for earthquake resistance of structure, therefore, "it is appropriate to adopt reinforcing layer with limited rigidity for frame-corewall structure under seismic action". It is a pity that cantilever truss component needs larger cross section and rigidity, since relatively high carrying capacity usually needs to be set for it in the design of cantilever truss structure, how to solve the above contradiction also becomes a problem of great attention by engineering design personnel. Combined with the result of dynamic elastic-plastic analysis to the cantilever truss in a super high-rise building structure, where two schemes of common support and buckling resistant brace are respectively adopted for the diagonal wed member, the path and advantage of adopting buckling resistant brace for realizing the cantilever reinforcing layer with limited rigidity is explored.

The main roofing structure of a super high-rise building is 222.7m high, it is totally 50 storeys above ground, and it belongs to high-rise building with super B grade limited height. The structure adopts section concrete frame-reinforced concrete corewall-cantilever truss reinforcing layer structure system. Since the rigidity of this engineering structure in Y direction is relatively good, therefore it is cantilever truss segment in the design of expansion restriction structure, the arm truss is arranged only along X direction of structure, it is respectively located No. 16, 28 and 40 storeys (building services storey or refuge storey), and circumferential waist truss is set at the outer frame of reinforcing layer position. The floor plane and the arrangement of buckling resistant brace are shown in Figure 2.1**Error! Reference source not found.**. The earthquake fortification intensity of structure is degree 7 (0.1g), the earthquake fortification category is category C, the designed earthquake group is group 1, the site classification is category II, and the characteristic period of site is 0.35s. Since remarkable non-linear degeneracy relationship can be seen in the axial force of member and the shortening of its member length in this engineering scheme, i.e.,. with the shortening in member length, phenomenon of continued reduction emerges in the carrying capacity, it presents typical compression buckling failure pattern. It is worthy to explain: both ends of the braced component are hinged connection in calculation, initial defect is not introduced into either of them, the reason for emergency of above compression buckling is mainly relatively big length of braced component, it is caused by the coupling effect between the inertia force of itself and axial force in the process of earthquake input, and that kind of coupling effect would increase with the increase in intensity of ground shaking [Code for load of architectural structure (2006 edition); Code for the aseismic design of building-2010)", Beijing: *China Building Industry Press*].

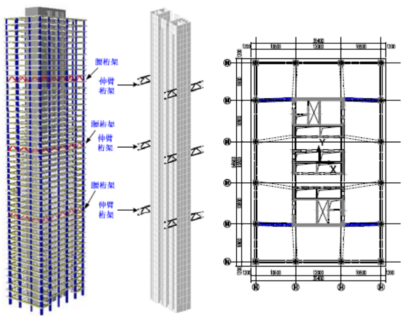
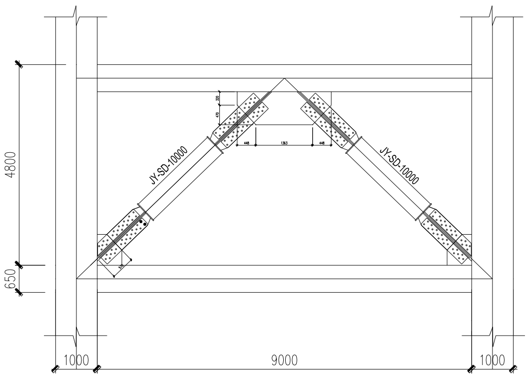


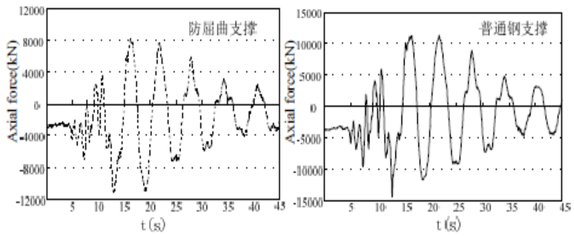
Figure 2.1. Structure plane and façade

Through setting the performance objective of no yielding in medium earthquake, dynamic elastic-plastic analysis under rarely occurred earthquake is performed to the integral structure under bidirectional input action, the response of two schemes of respectively adopting common steel support and buckling resistant brace for cantilever truss diagonal web member under action of rarely occurred earthquake and its influence on the integral structure are investigated. Figure 2.2 shows the deformation relationship curve between axial deformation and axial force under action of three groups of rarely occurred waves of degree 7 (respectively being artificial wave, US265/US266 wave and US397/US396 wave), bidirectional input (the peak acceleration in the principal and secondary directions is respectively

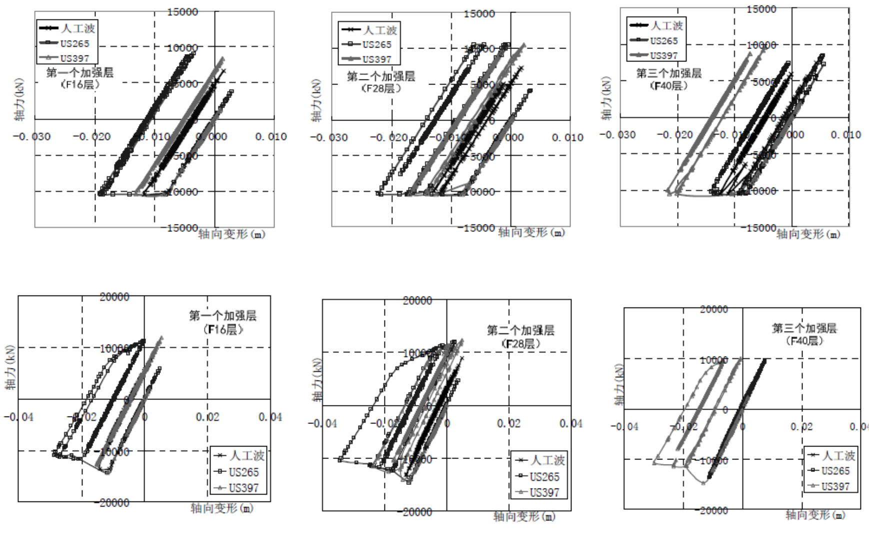
2.2m/s2 and 1.87m/s2) with X being the principal direction of input, for three reinforcing layer positions of structure, to which common steel support and buckling resistant brace are respectively adopted for typical diagonal web member. It can be seen that the response of diagonal web member of cantilever truss in both schemes has remarkable non-linear feature, among them: when the scheme of buckling resistant brace is adopted, the component enters the plastic energy dissipation stage and the area of hysteresis loop is relatively big, the action of energy dissipation is obvious, and carrying capacity of component does not drop after occurrence of compression yield; but in the scheme of common steel support, there is remarkable non-linear degeneracy relationship between the axial force of component and the shortening in its member length, with the shortening in the member length, the phenomenon of continued reduction emerges in the carrying capacity, i.e., it presents typical compression buckling failure pattern. It is worthy to explain that both ends of braced component is hinged connection in calculation, and initial defect is not introduced, the reason for emergency of above-mentioned compression buckling is mainly relatively big length of braced component, it is caused by the coupling action between the inertia force of itself and the axial force in the process of earthquake input , and that kind of coupling action would increase with the increase in intensity of ground shaking [Control of structural vibration – active, semi-active and intelligent control, Beijing, Science Press, 2003].



（a）Connected nodes



（b）Time travel curve of axial force of support

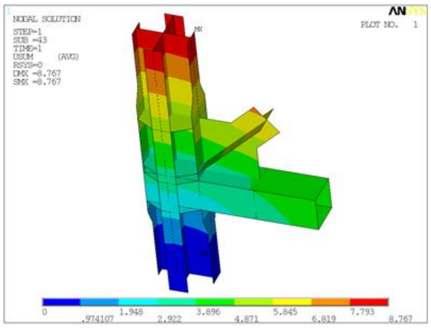


（c）Buckling Restrained Braced hysteresis curve

Figure 2.2. Interconnect structure and the hysteresis curve

Figure 2.2b**Error! Reference source not found.** is the time travel curve of axial force of member of buckling resistant brace component and common steel support at the first reinforcing layer position under the action of US265 wave (X is the principal direction of input). It can be seen that the buckling resistant brace reaches the yield carrying capacity for the first time at 13s, and the buckling resistant brace basically reaches compression yield status at ensuing 16.4s and 19.4s; After common steel support reaches compression yield carrying capacity at 12.9s for the first time, the compression carrying capacity gradually drops, the axial pressure reaches a relatively big value 11257kN at 16.6s, and it is only 11257kN/14663kN=77% of the yield carrying capacity of cross section (the compression stability reduction is not taken into consideration). It is shown by that phenomenon: once common steel support enters yield, it is very prone to compression buckling failure when it is compressed again, its carrying capacity remarkably drops, the emergence of original internal force could not continue to bear, meanwhile, the burden of other lateral force resistant component connected with it would also be increased.

It is worthy to mention: the yield carrying capacity of BUCKLING RESTRAINT BRACE adopted in cantilever truss is 10000KN, the support of such a large tonnage would certainly produce relatively great load on the main structure, and the connection mode is as follows:



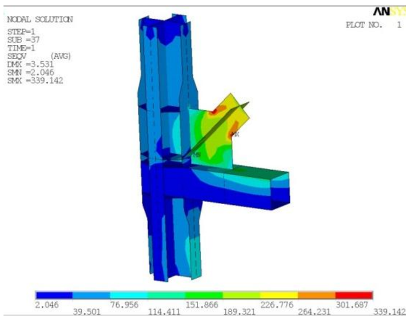


Figure 2.3 (left). Schematic diagram for the BUCKLING RESTRAINT BRACE node
Figure 2.4 (right). Finite element analysis of node of buckling restraint brace

3. APPLICATION OF BUCKLING RESTRAINT BRACE IN BOTTOM PART REINFORCING AREA

Similar to the circumstance in the reinforcing layer or cantilever truss, under earthquake action at the bottom area of high-rise and super high-rise building, relatively large tensile-compressional load action and overturning bending moment would be produced, unfavorable failure would be formed at the bottom area, therefore, bottom part reinforcing area needs to be set. For steel frame support system, the performance of support under large earthquake would certainly be ensured if common support is adopted, and its large deformation failure is generally not allowed to occur. That measure would increase its sectional rigidity by a big margin while guaranteeing different support stability, as a result, earthquake force would increase by a big margin, and that would cause stress concentration of peripheral components at the same time. Similarly, if common support is replaced by BUCKLING

RESTRAINT BRACE, the rigidity of bottom part can be effectively reduced in the integral structure while guaranteeing the lateral resisting rigidity under small earthquake, and it can also reduce the extent of rigidity change from the reinforcing area to the non reinforcing area at the bottom part.

A super high-rise building in degree 7 region, the structural pattern is steel structure, there are 70 storeys above ground, and it is 268m.

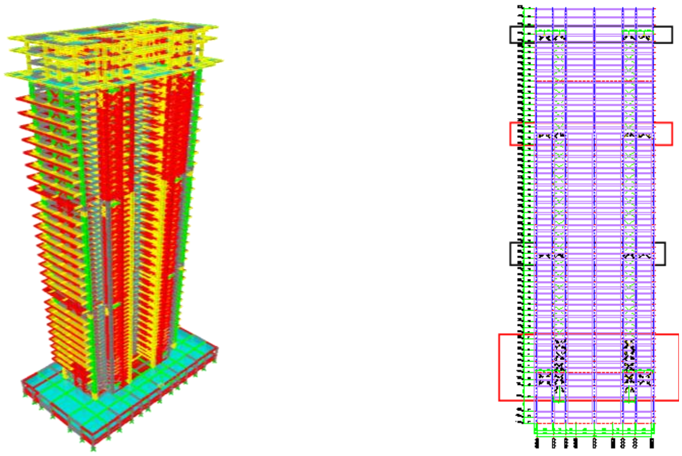
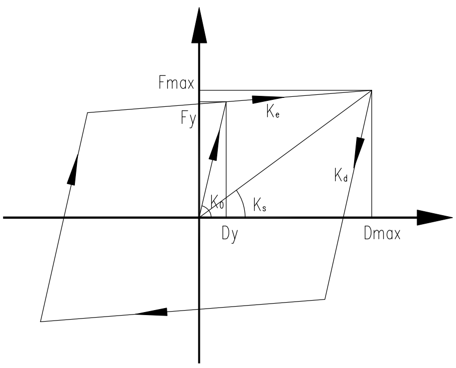
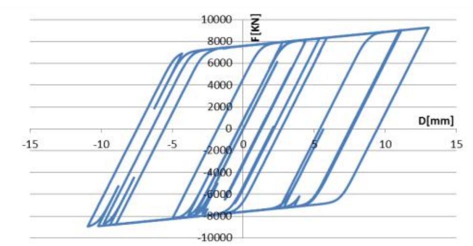


Figure 3.1 Façade of typical structure

Ideal elastic-plastic model is adopted for the BUCKLING RESTRAINT BRACE, i.e., double fold line model, where the elastic rigidity of BUCKLING RESTRAINT BRACE and the rigidity after yield are approximately equivalent to two segments of straight lines, the rigidity after yield is 5% of rigidity before yield as shown in the following diagram:





Figures 3.2 and 3.3 Ideal elastic-plastic model   Hysteresis curve of typical

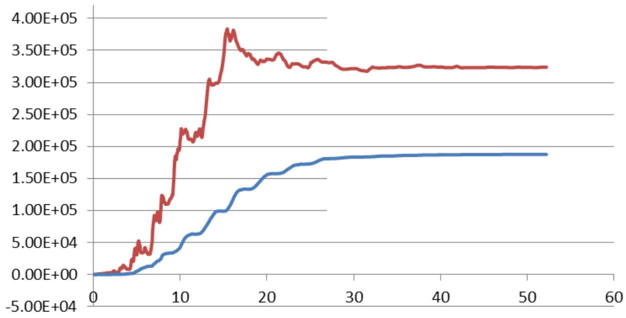


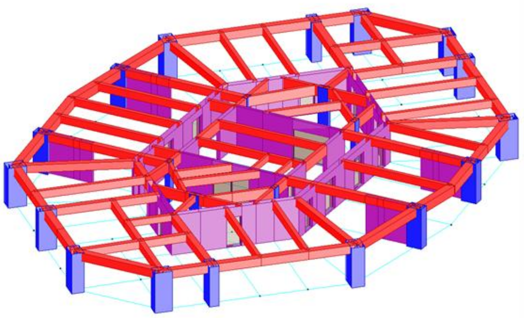
Figure 3.4 Proportion of energy dissipation of energy dissipation component

According to the above analysis model, the hysteresis curve of typical (the yield carrying capacity is 8000KN) support under the action of rarely occurred earthquake is shown in and **3.3**. If the energy dissipation component can relatively well dissipate earthquake energy, the damage extent of main stressed component (beam, column) under the action of large earthquake would be greatly decreased, the reflection of its macroscopic index can be measured by the proportion of earthquake energy dissipated by the energy dissipation component accounting for the earthquake energy of total earthquake input, through replacing the common support by BUCKLING RESTRAINT BRACE at the bottom reinforcing area and reinforcing layer in this project, the proportion of energy dissipation of energy dissipation component can reach 50% and over.

4. APPLICATION OF ENERGY DISSIPATION SHEAR WALL

In the region with high intensity, the requirement to the earthquake resistant capability of structure is increased with the increase of earthquake action by a big margin, and the corresponding cost is to increase the component size by a big margin, i.e., the mode of “hard resistance”, the energy dispersion and shock-absorbing technique can improve the damping ratio of structure to reduce the earthquake action, while not obviously changing the rigidity of structure, the external input is decreased by means of energy dissipation, so as to improve the seismic resistance capacity of structure. Metallic shear type damper is one pattern of metallic energy dissipation damper, since its sectional size is small and installation structure is flexible, it is mostly used high-rise and super high-rise buildings.

For a super high-rise building at degree 7 region, there are 45 storeys above ground (the main roofing structure is about 157.5m high), the plane of standard floor is about 33.6mx46.1m, and the planar shape is kept consistent from bottom to top. According to the earthquake fortification objective of structure, the structural performance needs to be improved, through comprehensive contrast, the self-weight of structure would be increased by about 8% while the structural size is increased if traditional scheme is adopted. It is shown by contrast analysis: if shock-absorbing technique is adopted, the damping ratio of structure provided under the circumstance of frequently occurred earthquake would be about 6.94%, meanwhile, the self-weight of structure can be reduced to some extent on the prerequisite of guaranteeing the same earthquake resistant capability.



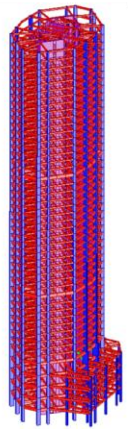


Figure 4.1. Plane of typical structure **Figure 4.2.** Façade of typical structure

According to the technical requirements, two aspects of problems mainly need to be solved in the original structure: decreasing the self-weight of structure and increasing the lateral resisting rigidity. It is shown through calculation: if it is arrangement mode of metallic damper + shear wall is adopted, i.e ., energy dissipation shear wall as shown in the following diagram:

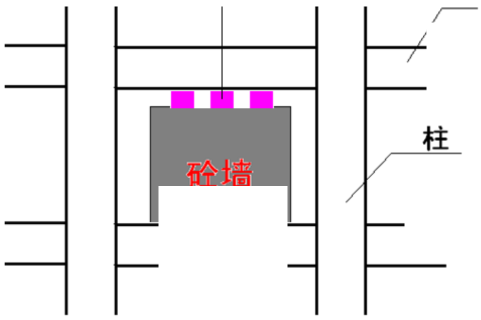


Figure 4.3. Energy consumption shear wall composition [Gao Jie, Xue Yantao et al, 2011]

Adopting the mode of energy dissipation shear wall can make the initial rigidity equivalent, i.e., elastic rigidity of shear wall + elastic rigidity of metallic damper = elastic rigidity of energy dissipation shear wall. The design in small earthquake stage can be realized in conventional software PKPM for the model after it is equivalent, when the damper enters yield stage, the energy dissipation characteristics can be embodied by changing the damping ratio of structure ["Manual for design and construction of passive shock-absorbing structure", China Building Industry Press, 2008].

The structural pattern is frame-corewall, the lateral resisting rigidity in the X direction tends to be weak because of the restriction in architectural function, and the main reason is that the connection of internal tube and external frame part is relatively weak. Therefore, it is designed to set four courses of damping walls in the X direction (the plane arrangement is shown in Figure 4.4).

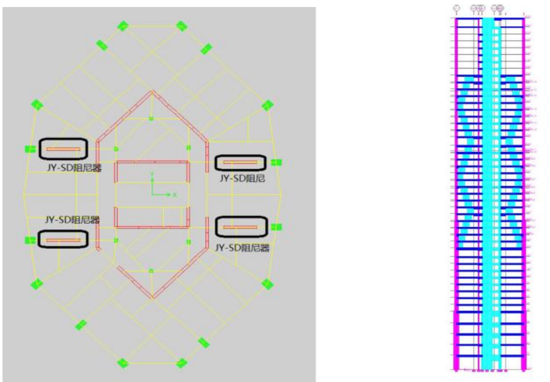


Figure 4.4. Plane and façade arrangement of damper

Setting damper in the X direction can provide lateral resisting rigidity for the structure, but because of the characteristics of damper itself, there is certain gap between it and wall and frame column at both ends to provide working space, but the internal tube could and the external wall could not be closely connected by that. If the damper is vertically arranged by staggering, it is vertically distributed from the edge of frame column and gradually approaches the internal tube horizontally, diagonal strut in diagonal direction would be formed in space, and the connection between internal tube and external column can be strengthened while reducing the internal force of framed girder connected with damper

(as shown in Figure 4.4 and 4.5) ["Manual for design and construction of passive shock-absorbing structure", China Building Industry Press, 2008].

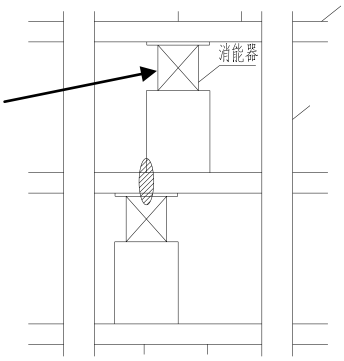


Figure 4.5. Vertical arrangement of damper

It is shown by calculation and analysis with the above scheme: the designed damper would yield under frequently occurred earthquake, and additional 2.94% damping ratio is provided, meanwhile, the earthquake force is decreased by a big margin, and the self-weight can be decreased by about 7%. The earthquake resistant capability of structure under frequently occurred earthquake and rarely occurred earthquake varies obviously.

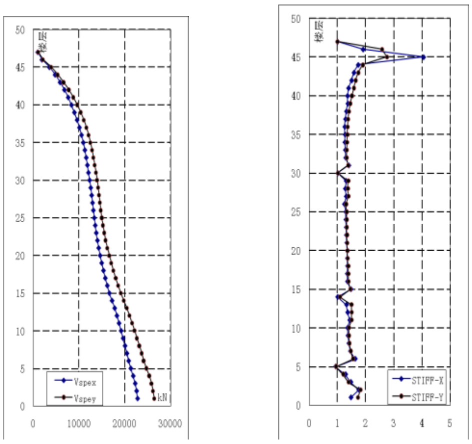


Figure 4.6. Storey shear and interlayer displacement angle distribution

5. ENERGY DISSIPATION LINTEL

The metallic displacement type damper is passive type, it can begin to work only when the structure has relative deformation, in order to give scope to the efficacy of metallic damper to the full, it shall be arranged at that part of the structure where interlayer displacement is maximum, but that usually could not be realized in the architectural structure because of the restriction in service function. The lintel in high-rise structure is usually the part with the greatest concentration of shear deformation, and the part for guiding plastic deformation concentration is usually designed at lintel. If the damper is set inside lintel (arrangement along the direction of vertical shear), multiple purpose can be answered, the efficacy of damper can be realized with high efficiency, higher additional damp can be provided for the main structure, and the damage of lintel or stressed component of main body can be avoided or alleviated.

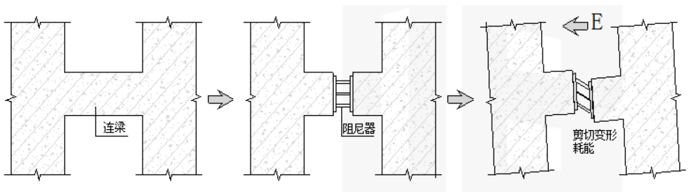
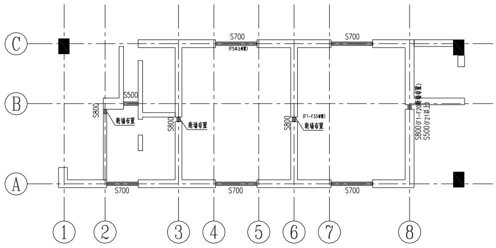


Figure 5.1. Composition of energy dissipation lintel

For a high-rise building with frame-shear wall structure in degree 7 region, there are 33 storeys above ground, the damping ratio of structure needs to be improved under frequently occurred earthquake for special functional requirements, if traditional arrangement mode is adopted, large quantities of dampers need to be set, corresponding economic character and architectural function are not allowable, therefore, the scheme of energy dissipation lintel is put forward, adopting that scheme can reduce the quantity of dampers by a big margin, and the architectural function and use would not be affected



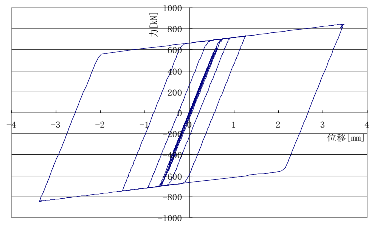


Figure 5.2. Arrangement plane of energy dissipation lintel

Figure 5.3. Hysteresis curve of energy dissipation lintel when ideal elastic-plastic model is adopted

Through setting at the ground floor and the part of middle and upper part where shear deformation of lintel is relatively great, by calculating additional 3% damping ratio of energy dissipation lintel under small earthquake, yield would not occur under the action of wind loading. Meanwhile, there is still certain safety reserve at the energy dissipation lintel under the action of rarely occurred earthquake.

The energy dissipation lintel (single) at typical floor of that structure is about 650KN. It is shown by contrast analysis: if damper is horizontally arranged along floor, the harmful displacement under frequently occurred earthquake would be about 1mm, the effective displacement can reach about 3mm when energy dissipation lintel is adopted, and the working efficiency can be improved by a big margin.

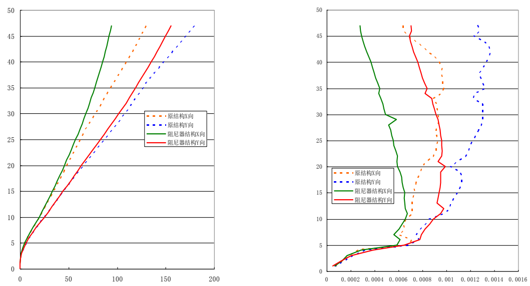


Figure 5.5. Comparison of floor displacement and interlayer displacement angle between traditional structure and energy dissipation structure

Compared with the traditional structure scheme and energy dissipation lintel scheme, the earthquake resistant capability of floor is obviously improved (the earthquake force can be reduced by the damping ratio provided in energy dissipation lintel scheme), and the floor displacement and interlayer displacement angle is obviously reduced.

6. BRIEF SUMMARY

It is shown by engineering practice and theoretical analysis: the earthquake resistant capability of structure can be effectively improved by the energy dispersion and shock-absorbing technique. Especially at the complicated structure in the region with high intensity, the comprehensive benefit is more remarkable, and the integral performance and working efficiency of structure can be improved by a big margin by adopting the new structure:

1） Abrupt change in vertical rigidity can be reduced by adopting BUCKLING RESTRAINT BRACE in cantilever truss, and the internal force load of component at upper and lower area of the reinforcing layer can be reduced;

2） By adopting BUCKLING RESTRAINT BRACE in the reinforcing area at the bottom part of super high-rise structure, no instability of the support can be guaranteed under rarely occurred earthquake, meanwhile, under the prerequisite of meeting the design requirements, the sectional size of support can be reduced by a big margin, and the unfavorable circumstance of internal force concentration at peripheral component caused by rigidity concentration can be reduced.

3） The energy dissipation shear wall can reduce the influence on architectural space to the greatest extent, additional damping ratio can be provided by adopting energy dissipation wall under frequently occurred earthquake, so as to reduce the earthquake action. Vertical staggered arrangement can effectively improve the connection of internal component, and reduce the shear force of connective component.

4） The working efficiency of damper can be maximally given scope to by the energy dissipation lintel, the efficacy of damper can be given scope to by utilizing the stress of lintel and its characteristics of deformation concentration, so as to protect the safety of main structure.

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