# OPTIMUM DESIGN OF RESILIENCE-FRICTION-SLIDE BASE ISOLATION SYSTEM FOR LOW COST BUILDINGS

XIYUAN ZHOU and MIAO HAN

Institute of Earthquake Engineering, China Academy of Building Research
Beijing 100013, China


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## ABSTRACT

This paper offers a resilience friction slide base isolation system consisted of recentering steel bar springs and friction slide bearings. This system is well adapted to low cost building such as multi-story-masonry building for its less expansive and high efficiency. And the principles of optimum design and engineering decision making have been discussed.

## KEYWORDS

Masonry structure, base isolation system, recentering spring, friction slide, optimum design

## INTRODUCTION

The low cost building in this paper usually means multi-story masonry building with brick bearing wall and R/C slab roof and floors. Because this kind of building costs lower than R/C frame and shear wall structure and also possesses good thermal isolation property, it is prevailing type of residential building in China. But masonry structure is brittle and highly vulnerable during earthquake. In order to enhance the strength of earthquake resistance of this kind of building following two main counter measures usually have been adopted. The first one is the limitation of building height, total number of stories and other stipulations related to dimension and materials of the structure and component. And other one is to strengthen the integrity of the whole building by means of R/C tie columns and ring beams. However the ductility of the strengthened masonry building or confined masonry structure is still lower than that of R/C structure or steel structure. So that it is inevitable for the masonry building suffering a certain damage during strong earthquake. The defense level in the current seismic design code is "cracking but non-collapse". Further improvement of earthquake resistance strength and ductility of masonry structures not only needs higher cost but also involves some technical complicacies, therefore seismic base isolation becomes a atractive approach[1]. For low cost building only the less expansive and high efficient base isolation measures are feasible. It is usually demanded that the total cost of base isolation building should be keep on the same level like the corresponding fix base building and without exceeding the acceptable payment level of the building owner[2]. This is also the target the paper is hunting for. In order to realize the target this paper provides a simple resilience friction slide base isolation system that is able to greatly enhance the seismic reliability of the masonry building and the principle and method of optimum design have been proposed.

## DEMONSTRATION BUILDING IN BRIEF

For sake of simplicity and intuition here we prefer to discuss the principles of the optimum design and the engineering decision for resilience friction slide base isolated building by means of a example of demonstration building. The project of optimum design is a 5-story brick building located in seismic area with intensity 8. This building is 42*m* long and 9*m* wide. The interstory height is 2.8*m* and the height of basement is 2.48*m*. The weight of the building is supported by brick walls. The allocations of the base isolation devices in plan and the structure details of base isolation layer have been shown in [2].

In this demonstration building 30*cm* × 30*cm* square teflon flat slabs are taken as friction sliding isolation bearings, and the recentering forces are provided by restoring element consisted of spring steel bars. Each restoring element includes two boxes up and down, and six of steel tube are fixed in the boxes (see Fig.2). The recentering steel bar is just inserted into the up and down tubes vertically[2]. The deformation capability of the steel bar has been tested in laboratory. And it is shown that the maximum lateral deformation of the spring steel bar with net height of 470*mm* beyonds 150*mm*.

Since the configuration of the building both in plan and elevation is regular and symmetric, the seismic response analysis of the base isolated building can follow multi-masses shear model shown in Fig.1.

In Fig.1 $k_1$ is total rigid coefficient of the resilience steel bars across isolation layer, $\mu$ is friction coefficient of sliding bearings. The masses and interstory rigidity of superstructure are listed below:

$$m_1 = 2.65 \times 10^5 kg\,,\ m_2 = \cdots = m_5 = 6.95 \times 10^5 kg\ ,\ m_6 = 3.95 \times 10^5 kg\,.$$
$$k_2 = k_3 = \cdots = k_6 = 6570\, kN/mm\,.$$

The fundamental period of the fixed base structure is 0.21*s*.

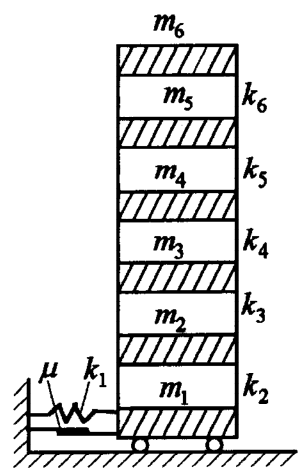



Fig.1 Multi-story shear modle for seismic response analysis

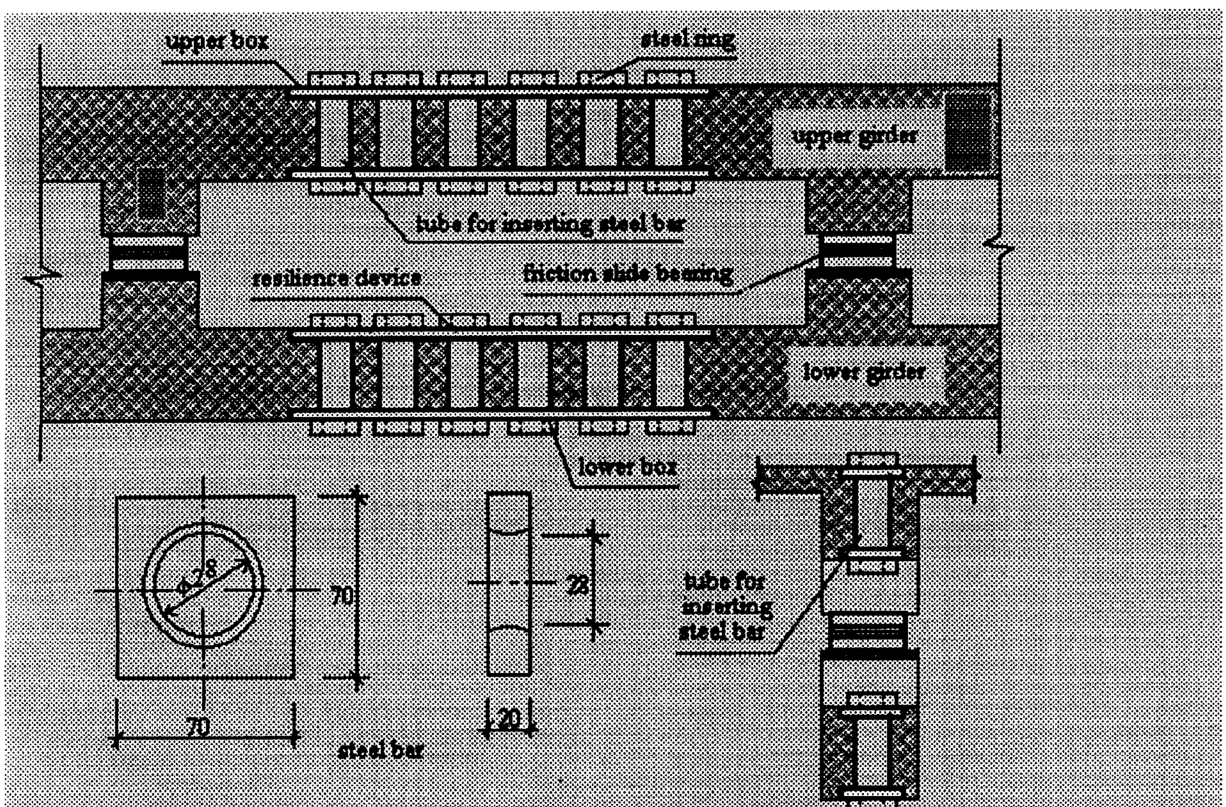



Fig.2 Friction slide bearings and steel bar box pre-buried in concrel

## MAIN VARIABLES FOR OPTIMUM DESIGN

Relative recentering spring rigidity $k_1/k_2$ and friction coefficient $\mu$ are basic parameters of resilience-friction-slide base isolation system. In order to optimize these parameters, time history analyses are carriedout for the base isolated building subject to N-S component of El Centro acceleration record on May 18, 1940 with peak accelerations scaled to 212gal, 425gal and 850gal. Equivalent base shear force and sliding drift of the friction slide bearings are adopted as criteria for evaluating the working quality of the base isolation system and seismic safety.

For analyzing the changing law of the evaluation criteria, we may firstly consider the influence of the friction coefficient $\mu$ and the relative recentering spring rigidity $k_1/k_2$ on the distribution of interstory shear force along height. Fig.3 shows the distribution of the peak interstory shear force along height for base isolated structure, with friction coefficient $\mu=0.13$, subject to earthquake excitation given by El Centro record with peak acceleration scaled to 425gal. The four curves shown in Fig.3 indicate the influence of the rigidity ratio $k_1/k_2$ on the distribution of shear force along height.

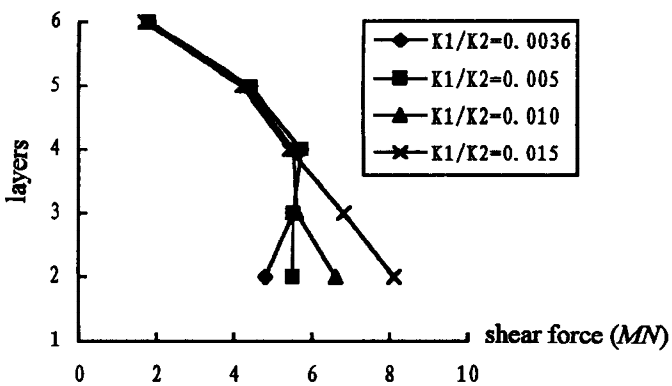



Fig.3 the interstory shear force distribution of base isolation structure along height

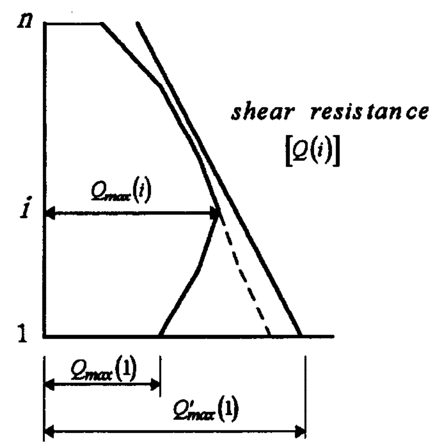



Fig.4 the changing laws of the interstory shear force and shear resistance

It can be seen from the curves illustrated in Fig.3 that the influence of the recentering spring rigidity on the shear force distribution is mainly on the lower part. If the rigidity of the recentering spring is small, the maximum interstory shear occurs at the middle to lower height of the structure. And also, the location of the maximum shear force is moving toward to the bottom of the structure as recentering spring coefficient increasing. For the multi-story masonry building with evenly distributed masses and interstory rigidity, the interstory shear resistance usually decreases as height of the floor increasing and the maximum value of the interstory shear resistance normally occurs at bottom story if the adopted shear strength of the mortar in masonry walls is all the same from bottom to top (Fig.4). If the distribution of the interstory shear resistance following the law shown in Fig.4, $k_1/k_2$ =0.01 or 0.015 are better choice in Fig.3.

In order to decrease the number of the referring variables in optimum design, we only select the maximum value of the interstory peak shear forces along height as referring variable which might occurs at bottom or middle to lower height of the building. Here, we use $i$ to indicate the ordinal number of the story, at which the maximum value occurs.

For the sake of simplicity, if the maximum value of the interstory shear forces $Q_{max}(i)$ occurs at $i$-level, we still only check the shear resistance capability at bottom:

$$Q'_{max}(1) \le [Q(1)] \tag{1}$$

and

$$Q'_{\max}(1) = Q'_{\max}(i)\frac{\sum_{j=1}^{n} G_j}{\sum_{j=i}^{n} G_j} \tag{2}$$

here $G_i$ is the gravity load of $i$-story level, [G(1)] is the allowed value.

## PRINCIPLE AND METHOD OF OPTIMUM DESIGN FOR BASE ISOLATION SYSTEM

For rigid structure such as masonry structure, the base shear force can be calculate according to following basic formula which has been stipulated in seismic design codes for buildings and civil engineering structures

$$V(1) = EPA \cdot \beta_{\max} \cdot C_{em} \cdot \sum_{j=1}^{n} G_j \tag{3}$$

where $EPA$ is equivalent or effective peak acceleration of design earthquake, $\beta_{\max}$ is maximum amplification factor of acceleration response spectra, $C_{em}$ is participation factor of masses. In China $EPA$ is scaled by design basic intensity which is comparable with MM intensity. The $EPA$ value for intensity 7, 8, 9 is 0.1g, 0.2g, and 0.4g respectively. The exceedance probability of design basic intensity in standard service period of 50-years is 0.1 that corresponds a return period of 475-years. Actually, checking of the earthquake resistance strength may be carried out under the level of design earthquake with return period of 50-years. In average the value of $EPA$ of corresponding to design basic earthquake with return period of 475-years is about 2.8 times of that of the design earthquake with return period 50-years. In consideration of this disparity in adoption of $EPA$ value the base shear force $V(1)$ calculated from eq.(3) should be reduced by a factor of 2.8 if earthquake resistance strength checking is carried out. The value of 2.8 is roughly equal to the ductility coefficient for the confined masonry structure. However the structures designed with base shear force of $V(1)/2.8$ will suffer a certain cracking or damaging under design basic earthquake.

As it is described above for the design earthquakes with intensity 7, 8, 9 and 10 we adopt $EPA$=0.105g, 0.212g, 0.425g and 0.85g respectively rather than 0.1g, 0.2g, 0.4g and 0.8g, so we use a reduce factor of 3.0 into $V(1)$ calculated by eq.(3) to replace 2.8.

In order to carry out option of optimization several sorts of friction coefficient $\mu$ and relative recentering spring rigidity $k_1/k_2$ have been selected and the non-linear dynamic response analysis has been done for the demonstration building shown in Fig.1 subject to input wave of El Centro record which peak acceleration has been scaled to required value. Being as criteria of optimum design of base isolation system the maximum drift of base sliding devices $\Delta_{\max}$ and base shear force $Q_{\max}(1)$ ( or $Q'_{\max}(1)$ ) in accordance of eq.3 have been chosen and calculated values of that are listed in table 1 and 2.

It can be seen from the results listed in table 1 and 2 that the maximum base shear force calculated following seismic design code is quite close to that from time history analysis. The results listed in table 1 and 2 also illustrated that the value of $Q_{\max}(1)$ is usually decreasing as friction coefficient $\mu$ decreasing. In view of requirement of reducing $Q_{\max}(1)$ it is sure that the friction coefficient $\mu$ is the smaller the better. But in the other hand the smaller the friction coefficient, the more the complex of the device of friction and the more the cost of the device. In consideration of the feasibility both in technology and economics the friction coefficient $\mu$ not less than 0.1 is suggested. The figures shown in table 1 indicate that the value of $\Delta_{\max}$ usually decreases as increasing $\mu$, but all the values of $\Delta_{\max}$ are quite small if $EPA$=0.212g. So in such a table 1 the calculated values of $\Delta_{max}(cm)$ and $Q_{max}(1)$ ( or $Q'_{max}(1)$ ) $(MN)$ for the system shown in fig.1 with *EPA*=0.212g

| | $\mu=0.1$ | | $\mu=0.13$ | | $\mu=0.16$ | | $\mu=0.2$ | |
|---|---|---|---|---|---|---|---|---|
| $k_1/k_2$ | $\Delta_{max}$ | $Q_{max}(1)$ | $\Delta_{max}$ | $Q_{max}(1)$ | $\Delta_{max}$ | 0 $Q_{max}(1)$ | $\Delta_{max}$ | $Q_{max}(1)$ |
| 0.0036 | 1.2 | 4.2 | 0.48 | 6.91 | 0.45 | 8.32 | | |
| 0.005 | 1.2 | 4.2 | 0.47 | 6.91 | 0.42 | 8.32 | 0.59 | 9.86 |
| 0.010 | 1.3 | 4.3 | 0.44 | 6.78 | 0.36 | 8.32 | 0.53 | 7.7 |
| 0.015 | 1.4 | 4.5 | 0.41 | 6.91 | 0.32 | 8.19 | 0.48 | 7.7 |
| 0.030 | 1.3 | 6.5 | 0.40 | 7.04 | 0.27 | 8.32 | 0.35 | 7.8 |
| Base shear force of corresponding fixed base structure | | | by seismic design code 12.9/3.0 $(MN)$ | | | by time history analysis 12.0 $(MN)$ | | |

table 2 the calculated values of $\Delta_{max}(cm)$ and $Q_{max}(1)$ ( or $Q'_{max}(1)$ ) $(MN)$ for the system shown in fig.1 with *EPA*=0.425g

| | $\mu=0.1$ | | $\mu=0.13$ | | $\mu=0.16$ | | $\mu=0.2$ | |
|---|---|---|---|---|---|---|---|---|
| $k_1/k_2$ | $\Delta_{max}$ | $Q_{max}(1)$ | $\Delta_{max}$ | $Q_{max}(1)$ | $\Delta_{max}$ | 0 $Q_{max}(1)$ | $\Delta_{max}$ | $Q_{max}(1)$ |
| 0.0036 | 6.5 | 6.67 | 4.7 | 7.8 | 3.5 | 9.72 | | |
| 0.005 | 4.7 | 6.53 | 4.5 | 7.04 | 3.4 | 9.72 | 2.4 | 10.9 |
| 0.010 | 4.6 | 6.0 | 4.2 | 6.6 | 3.4 | 8.3 | 2.6 | 8.6 |
| 0.015 | 5.4 | 8.1 | 4.5 | 8.1 | 3.9 | 8.7 | 2.7 | 8.9 |
| 0.030 | 6.4 | 15 | 5.3 | 14 | 4.1 | 13 | 2.6 | 11 |
| Base shear force of corresponding fixed base structure | | | by seismic design code 25.8/3.0 $(MN)$ | | | by time history analysis 24.0 $(MN)$ | | |

case the value of $\Delta_{max}$ is not a dominant factor, and speaking of the rigidity of the recentering spring, thesmaller the better for saving money. Sum up the considerations described above it seems a optimum choice if adopting $\mu=0.1$ and $k_1/k_2=0.0036$ for the base isolation structure subject to design basic intensity with *EPA*=0.225g. In this case as what indicated by table 1 $\Delta_{max}$=1.2*cm*, $Q_{max}(1)$= 4.2*MN*. It is interesting to point out that $Q_{max}(1)$= 4.2*MN* shown by table 1 is almost the same with that calculated value by formula (3) in seismic design code but reduced by a factor of 3.0. This result means the seismic action of the base isolated structures is only one-third that of corresponding fixed base structure. So that if the superstructure of the base isolated building has past through the earthquake resistance strength checking, no damage will occur even though no structural measures for increasing ductility are adopted. In contrast, if without base isolation measures the superstructures that have past through the same earthquake resistance strength checking only reach the state of "cracking but non-serious damage" even though the required structural measures for enhancing ductility from the seismic design code have been adopted. If the superstructure of the base isolated structure is designed according to intensity 7 including passing through the corresponding earthquake resistance strength checking and adoption of structural measures for increasing ductility, the damage degree will lower than that of the corresponding fixed base structure designed according to the requirements of against intensity 8. The superstructure of the demonstration building shown in Fig.1 is designed according to the basic requirement of earthquake resistance of intensity 8 in seismic code[2]. The base isolation measures further improve the seismic safety of the structure.

Due to the randomness and uncertainty of earthquake occurrence the event of earthquake with intensity greater than basic intensity indicated by seismic zoning map occasionally takes place. In order to protect the building in case of such kind of greater earthquake beyond anticipation occurs the current seismic design code for buildings in China implies the additional requirement of non-collapse once the building subject to earthquake with intensity greater one grade than design basic intensity. Following this additional requirement the time history analysis for the demonstration building subject to earthquake with intensity 9 (*EPA*=0.425g) has carried out and the calculated maximum sliding drift $\Delta_{max}$ and maximum base shear force $Q_{max}(1)$ are shown in table 2. It can be seen from table 2 that in this case the maximum base shear force only increases 50% compared with that for intensity 8 though the value of *EPA* is doubled if the parameters of the isolation layer keep unchanged, i.e. $k_1/k_2 = 0.0036$ and $\mu = 0.1$. If the earthquake resistance strength of the superstructure can comply the requirement against earthquake with intensity 8 only very limited ductility, which is easy available for well designed masonry structure, is sufficient for against earthquake with intensity 9. Nevertheless the maximum sliding drift is much more than that for intensity 8 (see the figures framed in a rectangle in right upper part of table 2). But even in such case the maximum sliding drift is still far less than the allowed value for example 14*cm*. These result means that for that demonstration building, if whose superstructure has designed according to intensity 8, no further danger but only undergoes a greater sliding drift during earthquake with intensity 9.

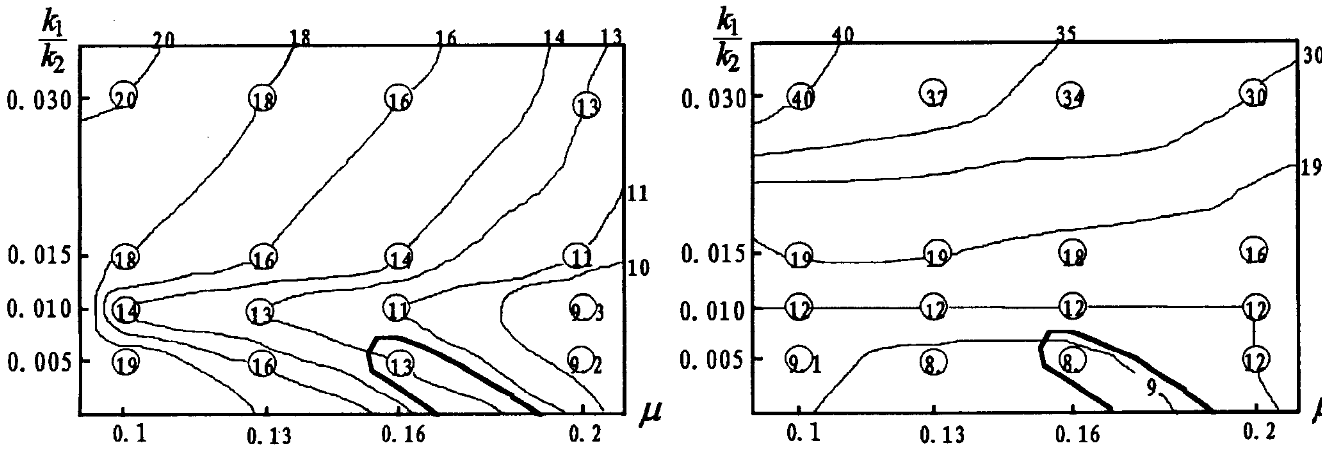
## THE PARAMETER OPTIMIZATION OF THE RESILIENCE FRICTION BASE ISOLATION SYSTEM AGAINST EARTHQUAKE WITH INTENSITY 9

Now suppose the demonstration building is located in seismic zone with intensity 9. In fact the maximum sliding drift $\Delta_{max}$ and maximum base shear force has been listed in table 2 for this case. From the results in table 2 the values of $\mu$ and $k_1/k_2$ framed in rectangle still is optimum option, and the corresponding sliding drift is still within the allowable range. But the shear resistance capability of masonry wall requires expansion of 50%, or *EPA*=0.33g as mentioned above. This result shows more effectiveness of the base isolation system if the design basic earthquake is intensity 9. In order to check the seismic behavior of the resilience friction base isolation system subject to the earthquake excitation of intensity 10 the time history analysis also has been done under input motion of El Centro record with *EPA* scaled to 0.85g. The calculated values of $\Delta_{max}$ and $Q_{max}(1)$ are listed in table 3.

table 3 the calculated values of $\Delta_{max}(cm)$ and $Q_{max}(1)$ (*MN*)
for the system shown in fig.1 with *EPA*=0.85g

| | $\mu = 0.1$ | | $\mu = 0.13$ | | $\mu = 0.16$ | | $\mu = 0.2$ | |
|---|---|---|---|---|---|---|---|---|
| $k_1/k_2$ | $\Delta_{max}$ | $Q_{max}(1)$ | $\Delta_{max}$ | $Q_{max}(1)$ | $\Delta_{max}$ | 0 $Q_{max}(1)$ | $\Delta_{max}$ | $Q_{max}(1)$ |
| 0.005 | 19 | 9.1 | 16 | 8.9 | 13 | 8.9 | 9.3 | 12 |
| 0.010 | 14 | 12 | 13 | 12 | 11 | 12 | 9.2 | 12 |
| 0.015 | 18 | 19 | 16 | 19 | 14 | 18 | 11 | 16 |
| 0.030 | 20 | 20 | 18 | 37 | 16 | 34 | 13 | 30 |
| Base shear force of corresponding fixed base structure | | | by seismic design code 51.6/3.0 (*MN*) | | | by time history analysis 48.0 (*MN*) | | |

It can be seen from the results in table 3 that the maximum sliding drift now become dominant criterion. The maximum sliding drift should be controlled less than 14*cm* since the dimension of the square flat slab of sliding is only 30*cm* × 30*cm* in the demonstration building. If taking $\Delta_{max} \leq 14cm$ as dominant criterion and requiring $Q_{max}(1)$ as less as possible, the values of $\mu$ and $k_1/k_2$ framed with thin thread would be good choices. The further option within the couples of $\mu$ and $k_1/k_2$ framed with thin thread may depend upon whether the main target is placed on reducing $\Delta_{max}$ or $Q_{max}(1)$. If you desire to enhance the function of the base isolation system and to reduce base shear force as much as possible then $k_1/k_2$=0.005 and $\mu = 0.16$ becomes better choice. On the contrary, if you prefer strictly controlling $\Delta_{max}$, $k_1/k_2$=0.005 and $\mu = 0.2$ is surely a more attractive choice. If adopting $\mu = 0.2$, taking $k_1/k_2$=0.005 is better than $k_1/k_2$=0.01 although their $\Delta_{max}$ and $Q_{max}(1)$ are almost the same because the small the $k_1/k_2$ the less the cost. The results shown in table 3 also can be reillustrated by contourlines in Fig.5. The area enclosed by thick thread is an option of parameters complying $\Delta_{max} \leq 14 cm$ and $Q_{max}(1) \leq 10 MN$.




(a) the contourlines of $\Delta_{max}$ $(cm)$ (b) the contourlines of $Q_{max}(1)$ $(MN)$

Fig.5 the contourlines of dominant variable in plan of $\mu$ and $k_1/k_2$

## OTHER FACTORS THAT SHOULD BE CONSIDERED IN PARAMETER OPTIMIZATION AND ENGINEERING DECISION

In above discussion and analysis related to optimization of base isolation system, what we have done is to search an optimum couple of $\mu$ and $k_1/k_2$ at given *EPA*. And, the uncertainty of *EPA* value and the total cost level of base isolation devices and structure have also been incorporated into optimum analysis of the base isolation system. Besides, there are other factors that needs being considered:

*1. The shape of story shear force distribution along superstructure.* We have supposed that the interstory shear resistance of the superstructure decreases as the height increasing and the checking of earthquake resistance strength can be carried out following the method of base shear force provided by seismic design code. In this case the base shear force is a dominant variable in determination of the seismic action of superstructure. If the shear resistance of the superstructure can be allocated according needs, the real shape of the interstory shear force distribution along height should be taken into consideration in parameter optimization and engineering decision for the base isolated building and then to develop the reasonable method for checking earthquake resistance strength of superstructure.

*2. The uncertainty of friction coefficient.* The friction coefficient in resilience friction slide base isolation system is a key parameter but it is hard to control. Due to the variance of the material property and inaccuracy of installation and construction work as well as uneven sinking and tilt of the foundation the real value of the friction coefficient usually greater than the designed one. For example the design value of friction coefficient from laboratory test is 0.1 for the sliding device in demonstration building shown in Fig.1 but the real value tested in field is 0.13. Hence it is necessary to consider the possible changing of friction coefficient with time and even motion directions.

*3. The randomness of the input ground motion.* As having been described above the well known El Centro record has chosen input acceleration excitation. Generally speaking the peculiarity of input wave should be suitable for the regional seismic environment and the site condition. And in addition the uncertainty of the frequency contents of the input wave also should be taken into account. For the demonstration building we have chosen several input waves to do time history analysis. The results indicate that the influence of the input wave upon the base sliding drift is more significant than upon the interstory shear force induced in superstructure. Due to that the demonstration building is located on stiff soil layers and the frequency contents contained in design ground acceleration excitation likely incline to high frequency side and then taking El Centro record as input wave would be a conservative option.

*4. The influence of the uncertainty involved in rigidity characteristics of the recentering spring.* The uncertainty of the rigidity coefficient of the recentering spring is smaller than that of $\mu$ value, and therefore influence of the uncertainty involved in rigidity characteristics can be ignored. Nevertheless the influence of the rigidity parameters of the recentering springs upon interstory shear forces and base sliding drift should be taken into consideration if adoption of recentering springs with elasto-plastic hysteric loop characteristics.

## CONCLUDING REMARKS

The general principles for parameter optimization of the base isolation system and structure design should include:

1. Both the base sliding drift and the maximum interstory shear force are controlled within allowed values simultaneously.

2. In order to consider the influences of the uncertainties involved both in the parameters of the base isolation system and the spectral characteristics of the ground motion the base sliding drift and the maximum interstory shear force should not exceed the allowed values even though both the parameters of the base isolation system and ground motion spectral characteristics take unfavorable values.

3. The seismic safety of base isolation building should be better than that of corresponding fixed base building.

4. In the case of complying above prerequisites the additional cost put on earthquake protection should be as less as possible and to realize a safe and economic design.

## REFERENCES


1. Xi Y. Zhou *et al.* (1994), Application of laminated-rubber bearings in masonry structure apartments, International workshop on use of rubber based bearing for earthquake protection of building, Shantou, China, May 17-19.

2. Shui Y. Fan *et al* (1994), A demonstration building of masonry structure and its earthquake behaviors analysis, in proceedings of 3rd state wide conference on masonry structure, Chendou, China, October 8-14, (in Chinese).