A New Roller Bearing Isolation System for Highway Bridges

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ABSTRACT
One of the major challenges in seismic isolation design for highway bridges is to maintain the base shear at an adequately low level with acceptable bearing displacement. This paper describes a new isolation system made of steel cylinders rolling on sloped surfaces. This system has two distinct features: 1) the acceleration of the superstructure is maintained at a relatively low and constant level; and 2) the bearing displacement is kept to a level that is smaller than currently available in today's isolation bearings.

KEY WORDS
Seismic isolation, low and constant base shear, reduced bearing displacement

PRESENT DAY ISOLATION SYSTEMS
Present day isolation systems have been largely developed for buildings. These systems and their design methods have been subsequently modified for bridge applications (see Constantinou et al 1996; Naeim and Kelly, 1999, for instance). Several basic issues relevant to seismic isolation design for highway bridges are first reviewed in this paper. They represent the motivation for the development of a new seismic isolation system.

Design Window
The basic idea of using an isolation system is to shift the fundamental period from a relatively smaller value to a higher value to reduce the seismic force (see Den Hartog, 1956). In figure 1(a), the mean spectral values of the pseudo acceleration are plotted for 99 earthquake records, which are all scaled to 0.4 g amplitude. The dotted line is a typical design response spectrum with a damping ratio equal to 5%. In order to reduce the acceleration of the superstructure down to 0.3 g, the period must be longer than 1.5 seconds. However, in so doing, there will still be a large bearing displacement. Figure 1(b) shows that, the longer the period, the larger the displacement will be. Taking the same example of the curve with 5% damping ratio, we can plot the spectral displacement denoted by dotted line in figure 1 (b). If we need to limit the displacement with 0.35 m, then it is seen from figure 1(b) that the period cannot be longer than 3.5 sec. It is also seen that, the shorter the period, the smaller the displacement will be. Combining both (a) and (b) in figure 1, we can establish a design window marked by the vertical dotted lines, with which a design satisfies both acceleration and displacement requirement.

It is noted that, in figure 1, only the mean spectral value is used. In many cases, we often use the

mean value plus one standard deviation, which will further narrow the design window. The fact is, the narrower the window is, the more difficult it is to select proper bearings. If the design window is too narrow, then we may not achieve the desired bridge performances by using seismic isolation.

From figure 1, we can also realize that seismic isolation by using the design response spectrum employs mainly two parameters, the period and the damping coefficient, which are directly related to the lateral stiffness of isolation bearings. Thus, the design spectrum can be denoted by S_A (T, ξ), where T is the period and is ξ the damping ratio. Denote M and K as the mass of the superstructure and the lateral stiffness respectively, the period of the isolation system, T, is given

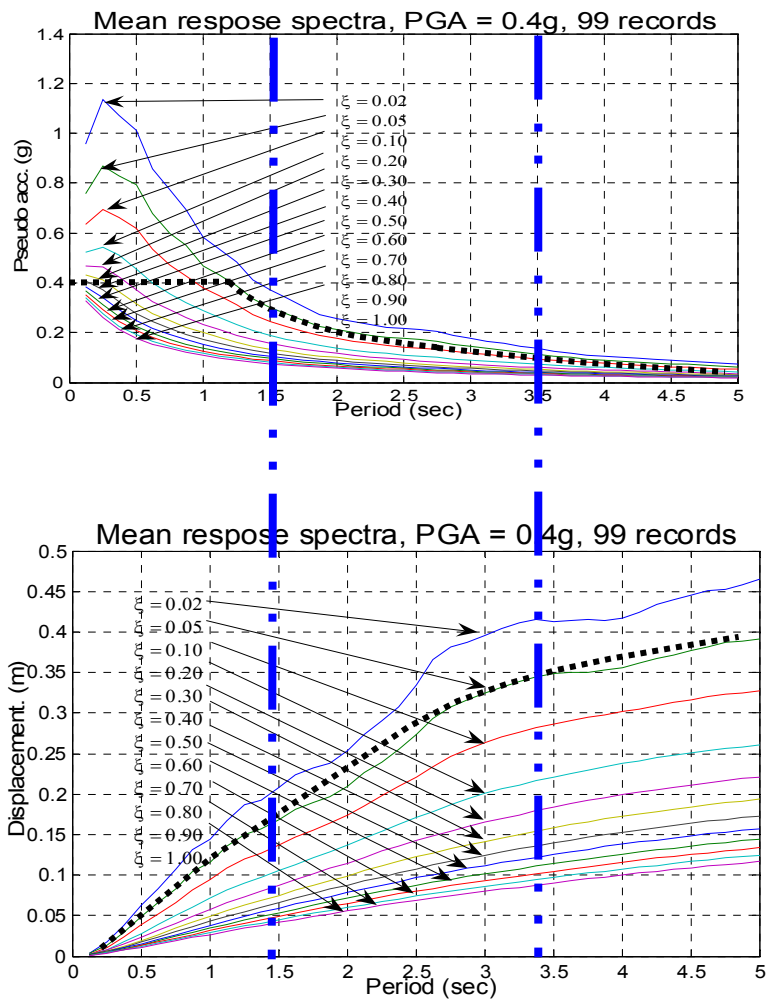


Figure 1. Design window

by

$$T = 2\pi\sqrt{\frac{M}{K}} \qquad (1)$$

Denote the energy dissipated by the bearing system in one sinusoidal cycle as E_d and the maximum bearing displacement as $D_{A,}$, respectively. The damping ratio, ξ, can be calculated

from

$$\xi = \frac{E_d}{4\pi\left(\frac{K D_A^2}{2}\right)} = \frac{E_d}{2\pi K D_A^2} \qquad (2)$$

From figure 1, it is seen that by increasing the damping ratio, a wider design window may result. In current design practice for various damping, the damping coefficient B is used to modify the response. B is defined as the ratio of the response spectral value $S_A(T, \xi)$ with damping 5% and with a different damping ratio ξ. That is,

$$B = \frac{S_A(T, 0.05)}{S_A(T, \xi)} \qquad (3)$$

In the first column of table 1, the value of B vs. the damping ratio is listed (AASHTO 1999 and NEHRP 2000).

Table 1. Numerical Damping Coefficients

	AASHTO	T=0.625	T=1.00	T=2.00	T=4.00
ξ = 2 %	0.8	*0.7918*	0.8049	0.8267	0.8533
ξ = 10 %	1.2	1.2711	1.2383	1.2051	*1.1794*
ξ = 20 %	1.5	1.6990	1.6378	1.5316	*1.4962*
ξ = 30 %	1.8	2.0867	2.0020	1.8224	*1.7806*
ξ = 40 %	2.1	2.4441	2.3580	2.1074	*2.0571*
ξ = 50 %	2.4	2.7842	2.7072	*2.3905*	*2.3247*
ξ = 60 %	2.7	3.1206	3.0488	*2.6685*	*2.5774*
ξ = 70 %	3.0	3.4537	3.3791	*2.9415*	*2.8255*
ξ = 80 %	3.3	3.7810	3.7081	*3.2110*	*3.0692*
ξ = 90 %	3.6	4.0939	4.0326	*3.4812*	*3.3104*
ξ = 100 %	4.0	4.3951	4.3504	*3.7482*	*3.5488*

By using damping coefficients, the design of an isolation system with a damping ratio different from 5% can be achieved. For example, the base shear V can be expressed as

$$V_\xi = \frac{V_{0.05}}{B} \qquad (4)$$

where the subscript ξ and 0.05 represent the cases with a damping ratio ξ and 0.05, respectively. The numerical damping coefficient B, used by many design codes, is solely determined by the damping ratio. However, by using the aforementioned 99 earthquake records, B is also found to be a function of the period. When the period is greater than 1.5 seconds, the values of B calculated based on equation (3) are actually smaller than those provided in the design guidelines. Also shown in table 1, the calculated values of B are given under different periods. Approximately speaking, when the period is higher than 1.7 seconds, and/or lower then 0.3 seconds, using the values listed in the first column of table 1 may result in unsafe design.

Nonlinear Bearing Behavior

Although isolation design is based on the above-mentioned linear response spectra, most

isolation bearings behave nonlinearly. Their nonlinear behavior is typically modeled by using a bi-linear parallelogram shown in figure 2.

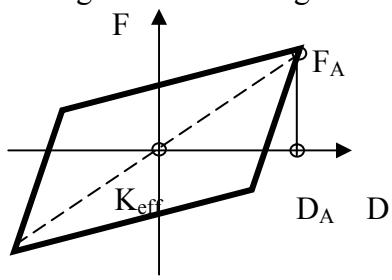


Figure 2. Bi-linear relationship

The slope of the diagonal line of the parallelogram is defined as the equivalent stiffness of the isolation system, that is,

$$K_{eff} = F_A/D_A \qquad (5)$$

The period and the damping ratio are calculated from equation (1) and (2) by substituting K_{eff} for K. Now, let us examine the limitations of this approach. It is well known that the period of a linear system is determined by equating the maximum potential energy, $\frac{1}{2}KD_A^2$, which is obtained at the maximum displacement D_A, to the maximum kinetic energy, $\frac{1}{2}MV_A^2$, which corresponds to the maximum velocity V_A.
That is, let

$$\frac{K\,D_A^2}{2} = \frac{M\,V_A^2}{2} \qquad (6)$$

If a system has a sinusoidal vibration, then $V_A = 2\pi/T\ D_A$. From the above two equations, the period , T can be defined. By using the parallelogram and following the same procedure with the quantity K_{eff} , the equivalent period T_{eff} can be calculated. However, at the point of maximum displacement, the corresponding force F_A may involve a conservative force component F_C and a dissipative force component F_D. That is,

$$F_A = F_C + F_D \qquad (7a)$$

For a nonlinear isolation system, it is likely that $F_D \neq 0$ or

$$F_A > F_C \qquad (7b)$$

In this case, use F_A instead of the conservative force component F_C in equation (6) to determine the potential energy can result in overestimated K_{eff} from equation (5). The amount of overestimation depends upon the properties of the specific isolation bearings, which can be negligible or very large. Equations (1) and (2) show that overestimation of the stiffness results in underestimation of both the period and the damping, which would further affect the design window.

Safety Margin of Isolation Bearing Design for Bridges

As mentioned above, when designing a regular building, the design spectrum that is determined by using the statistical mean plus one standard deviation is often used. One of the reasons the maximum response is not used in building isolation design is because buildings typically have redundancy that provides a margin of safety against collapse. However, this design concept may

not be suitable for bridge isolation because there is typically no redundancy at the bridge bearing isolation. Chances of falling spans should be reduced by using perhaps the maximum response.

NEW CONCEPT OF CONSTANT BASE SHEAR ISOLATION

The above discussion briefly describes the disadvantages of using a design based on the response spectra shown in figure 1, which may limit the effectiveness of the isolation system and enlarge the bearing displacement. In order to address and overcome these difficulties, a new concept of constant acceleration (or constant base shear) by using rolling surfaces on constant slope is developed. The concept of a single roller bearing is shown in figure 3. The working mechanism is shown in figure 4(a) and its capability for tolerating large bearing displacement is shown in figure 4(b), where the displacement of the superstructure, H_D, can be twice as much as the bearing radius, H_B. That is, the bearing can tolerance large displacements within a relatively small dimension. A prototype bearing with a single roller is shown in figure 5(a).

Figures 3 and 4 show a single layer bearing. However, to account for two directional ground motions, two layers of rollers perpendicular to each other, are needed. A two-layer assembly is shown in figure 5(b). The total height of the bearing will depend on the vertical load. For example, a practical design for 1000 Ton (2000 Kips) vertical load will need to be approximately 30-33 cm (12 – 13") high.

The horizontal acceleration is mainly determined by the angle of the slope. That is, despite the magnitude of the ground acceleration, the acceleration of superstructure, a_s, can be written as

$$a_s = \alpha\, g \qquad (8)$$

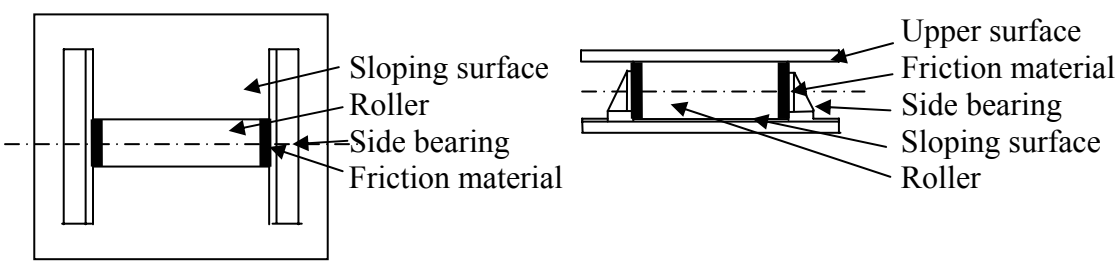


Figure 3. Assembly of a single layer bearing

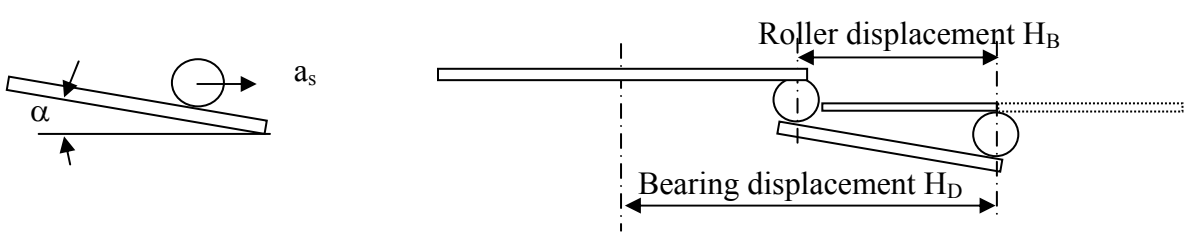


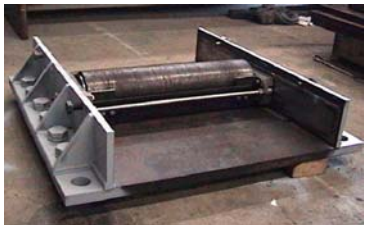
Figure 4(a). Conceptual assembly of side loading bearing

Figure 4(b). Bearing displacement

Since the angle of the slope is quite small, around 1.5^0 to 2.5^0, the acceleration contributed by the bearing can be as small as 0.03 -0.05 g.

In order to regulate the bearing displacement, the roller bearing must be used together with a larger damping mechanism, such as viscous or friction dampers. The friction damper is more

effective and less expensive, and can be built inside the housing of the bearing system. Figure 5(b) shows the friction material mounted at each end of the roller. The corresponding friction pair is the steel guide walls, shown in figure 5(a). It can be shown that when friction damper is used with the coefficient μ, the total acceleration α will be



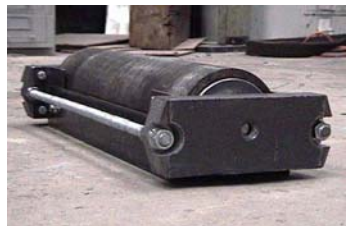


Figure 5(a). Single layer bearing assembly **(b). Double layer assembly**

$$\alpha = \mu + a_s \tag{9}$$

Roughly, when

$$\mu > a_s \tag{10}$$

The isolation bearing will not be able to return to its original position. When $\mu > 2a_s$ the system can be overdamped (see Liang and Lee, 1991). However, since the acceleration caused by the gravity is small, the total lateral acceleration is still smaller than 0.15g, even when the system is overdamped (i.e., damping about 0.05-0.10 g) Thus, since the acceleration caused by the rolling surface is only 0.03-0.05g, there is sufficient room to add more damping, which will help to regulate the bearing displacement more effectively.

From figure 4(a), it is seen that a ground acceleration with amplitude a_g will be reduced by a factor of α^2. For example, a ground motion with an acceleration of 2g will have a lateral acceleration of the superstructure of about 0.002g, if the angle of the slop is 2°. It is one order smaller than 0.03g, the acceleration resulting from the slope. Therefore, the component of acceleration contributed by the ground motion is negligible and the level of acceleration of the superstructure can be taken as a constant for all practical purposes.

SHAKING TABLE TESTS

In order to verify the performance of the roller bearing predicted by the theoretical analysis, several experimental sequences were carried out. First, a proof of the concept test was carried out at University at Buffalo. Second, a single roller bearing was tested at the University of Nevada at Reno. Third, the same bearings were tested at the University of California at San Diego under the test protocol of the Highway Innovative Technology Evaluation Center (HITEC). Fourth, a different set of roller bearings together with external friction dampers was tested at the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. Finally, a combined bearing and damper system was tested at the Institute of Engineering Mechanics (IEM) in China. These tests exhibited good agreement between the theory and the observed data.

DESIGN PROCEDURES FOR A ROLLER BEARING

Basic Design

Since the roller bearing guarantees a constant acceleration or base shear, new design procedures must be developed. Current procedures start the design process with specified bearing displacement, which is no longer necessary. The following design procedure is established for the roller bearings.

1. Determine the total allowed acceleration of the superstructure, a_T.

$$a_T = n\ \nu\ V/W \qquad (11)$$

Here, n is the total number of bearings; ν is the safety coefficient for unevenly distributed load, usually $\nu = 1.1 - 1.5$; V is the allowed base shear and W is the total weight of the superstructure. For example, a_T is taken to be 0.2g.

2. Determine the angle of the rolling slope α. As mentioned before, α is taken to be 0.035 -0.09 (2^0 to 5^0). The larger number used for rough construction and unstable sites.

3. Determine the allowed friction coefficient μ

$$\mu = a_s - \alpha \qquad (12)$$

4. Design the allowed bearing displacement according to table 2. This table was generated by using 99 earthquake records, which were all scaled to have 1.0 g peak ground acceleration (PGA). A similar table could be generated according to peak ground velocity, which would yield a smaller displacement. Larger values are used in this study for safety reasons. If the design PGA is equal to or less than 1 g, the following equations for the bearing displacement are used:

$$D_A = a_g\ D_T(\mu,\theta) \qquad (13)$$

where a_g is the PGA with unit to be g; D_T is the function of the friction coefficient μ, and the angle of the slope θ and D_T is listed in table 2. For example, if α is taken to be 0.07 (40), μ can be 0.20-0.07 = 0.13. Then the bearing displacement is 49.3 cm.

Table 2. Bearing displacement (cm)

	μ=.05	μ=.06	μ=.07	μ=.08	μ=.09	μ=.10	μ=.11	μ=.12	μ=0.13	μ=.14
$\theta=2^0$	89.0	84.0	79.3	74.9	70.7	66.7	62.7	58.5	55.2	51.6
$\theta=3^0$	90.0	84.1	78.9	73.9	69.3	64.7	60.4	56.3	52.5	49.0
$\theta=4^0$	89.4	83.1	77.4	71.8	66.7	61.6	57.4	53.3	49.5	46.0
$\theta=5^0$	87.7	80.7	74.4	68.8	63.4	58.7	54.3	50.2	46.5	43.0

5. If the selected slope angles and friction coefficient as well as the calculated acceleration and displacement satisfy the design parameters, then the design is completed. Otherwise, a different set of slope angle and the friction coefficient are chosen and the process repeats until the design is satisfactory.

Note that table 2 is only generated through 99 earthquake records. In specific site or applications, the displacement may be recalculated with a different set of earthquake records. For more important applications, a time history analysis may also be carried out.

Vertical Load

The vertical load per unit length P_{pul} can be determined by the following expression

$$P_{ul} \le \frac{2}{3}(9d\sigma_u{}^2 / E) = 6d\sigma_u{}^2 / E \qquad (14)$$

Where d is the diameter of rollers, σ_u is the nominal ultimate tensile strength of the material, and E is the modulus of elasticity of the material. For example, E = 30000 kips/in^2, $\sigma_u = 80$ kips/in^2, and the diameter of rollers d = 1.25 in, results in $P_{ul} \le 1.6$ kips/in.

SUMMARY

This paper briefly describes the motivation and highlights of the development of a new seismic isolation bearing for highway bridges.

- The new roller bearings have the advantage of fixed and very low superstructure acceleration because their lateral stiffness is near zero and the working period is near infinite.
- They have small bearing displacement, because they function in the overdamped range
- Tests conducted in several laboratories show good agreement with theoretical predictions.

The details of the development of this new isolation system are provided in a MCEER technical report (Lee et al) to be published by the end of the year 2005.

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