SEISMIC ISOLATION FOR SMALL REINFORCED CONCRETE STRUCTURES: A PRELIMINARY INVESTIGATION ON MATERIAL COST

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Abstract

The objective of this investigation is to show the effectiveness of a rubber bearing isolation design for a low-rise reinforced concrete structure in Algeria, Africa. The results of this investigation focus on a specific example. The method followed was to design two possible frames, A and B; this was done according to applicable codes for an office building. Frame A is a moment resisting frame, and the latter is an isolated frame. The total material required for both was then compared. The building has a two story, 8100 square-foot square plan, with four rows of four columns. A high damping rubber bearing was chosen as the isolating device. The procedure to design the isolation systems was basically the following (adhering to UBC-97 standards): 1) Determine and establish parameter-dependent factors, 2) select a type of bearing and estimate target values such as stiffness, damping ratio and bearing displacement, 3) relate required dimensions of the unit with target values from previous step, and 4) produce isolator detail. The design of frame B called for 16 rubber bearing units. The resulting weight ratios between frames A and B showed a total steel ratio of 3.3 and a total concrete ratio of 1.8.

Objectives

The objective of this investigation is to show the effectiveness of a specific rubber bearing isolation design. The building is a symmetrical, low-rise reinforced concrete structure in the region of Algeria, Africa. Algeria was chosen because in May, 2003, the country was struck by its worst earthquake in 23 years. More than 2,000 people were killed and over 9,000 injured. Many homes were destroyed in the initial quake and thousands more were damaged and made uninhabitable. This investigation explores a system that could protect similar structures, preventing disasters like this in the future. As mentioned before, it focuses on a specific example; but it paves the way for the consideration of base isolation by demonstrating how cost effective it can be.

Research approach

Two possible frames were designed, A and B. Frame A is a moment resisting frame and Frame B is a lighter frame with an isolated base. Frame A was subjected to an equivalent lateral static earthquake load, (explained later), and then designed according to the appropriate code. An isolation system was designed for Frame B, as a result it was designed to only carry gravity loads. The total required material for each major structural element was then compared for both structures. Ratios were found for the girders, columns and foundations.

Reinforced concrete was designed according to the American Concrete Institute's (1999) regulations, non-seismic design loads and base isolation designs conform to Uniform Building Code (1997) requirements, and seismic load determination was developed from Algerian seismic codes. Member forces were modeled and analyzed using RISA-2D (2004). It is important to note this software's sign convention: Positive moment refers to tension above and compression below the neutral axis of a member subject to bending. Moment envelopes and diagrams are presented with this convention.

As shown below, Figure 1, the plan considered was an 8100 ft^2 two story plan.

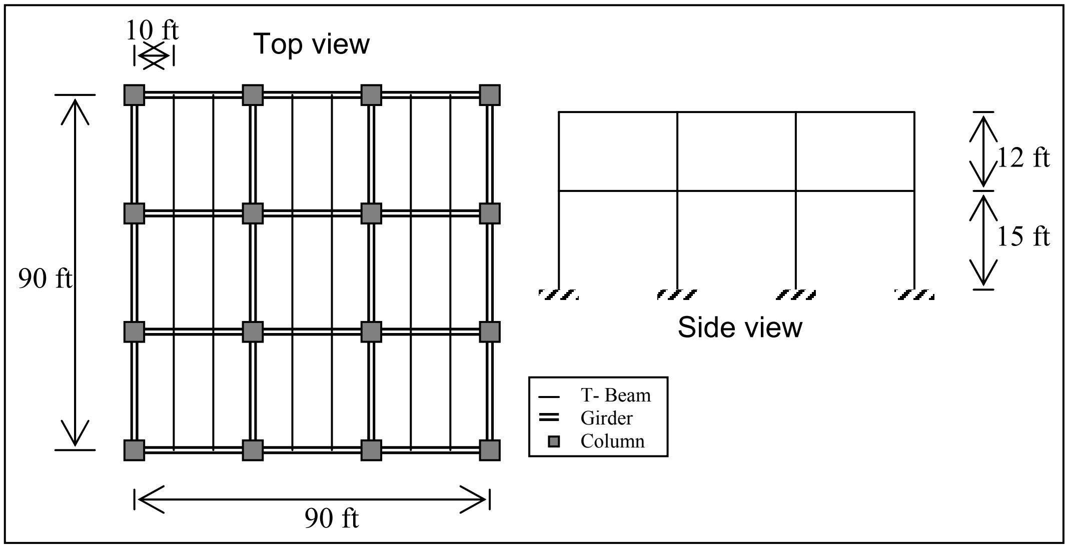

Figure 1. Structural plan

Parameters and assumptions

The parameters established for this project were selected to be representative of a typical situation in the region. The soil profile is a uniform stiff material that can support an allowable stress, q_{all} = 4,000 psi. The soil's unit weight, $\gamma_{soil} = 100\ ^{lb}/_{ft^3}$. The site is in a Zone III region, (from Algerian Seismic Codes, which agrees with UBC-97). All concrete was decided to have a compressive strength, f'_c = 3,000 psi. Steel used for all reinforcement has a strength, f_y = 60,000 psi. The design methods used in this investigation are considered standard practice for reinforced concrete design; and although some resulting designs are conservative, the same degree of conservatism was used for parallel designs (girders, columns, etc of Frame A and Frame B).

A floor system was designed, not to be considered for comparison, but rather to obtain more accurate dead weight values. This is because the floor system, a slab and T-beam section as described later, are assumed to only carry gravity loads; so they are the same sections in both frames. These gravity loads were defined as a live load of 50 psf for the first floor, and 20 psf for the roof. Dead loads were found to be 12 psf for both stories.

Algerian seismic codes allow the use of a static lateral force design method if three requirements are met: 1) Total structure height is less than 30 meters, 2) no soil liquefaction or resonance behaviors are predicted and 3) the structure is classified as regular (no more than a height difference of 25% per story, no eccentricity, symmetrical and consistent damping throughout plan). As shown in Figure 1, all three requirements are satisfied for this method.

Although a stiffer floor diaphragm needs to be designed for Frame B, this preliminary investigation only compares the cost of structural elements common to both frames. Consequently, it is omitted from this report.

Outcomes

The outcomes for Frame A and Frame B are discussed below. Detailed calculations are described in the Appendices for the design of the base isolation unit. Shear analyses are only highlighted in Appendix A-1 since minimum code requirements were sufficient for most structural elements. Thus it is assumed that including shear reinforcement in the analyses does not change the overall ratio of materials.

Frame A details

The procedure for determining a static lateral force resulted in the following loads:

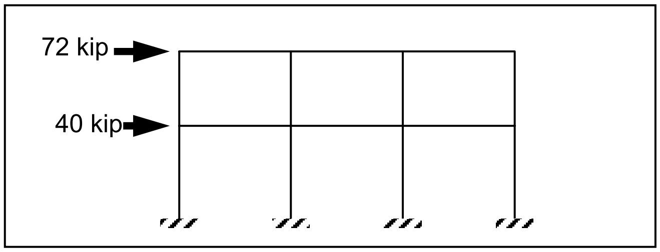

Figure 2. Earthquake loads

Identical slabs were used for the roof and first floor. They are simply supported, 6” thick, and reinforced with #3 bars spaced at 6½” on center. Temperature steel is also called for, so #3 bars at 12” are used. The beams used are considered to behave as T-beams:

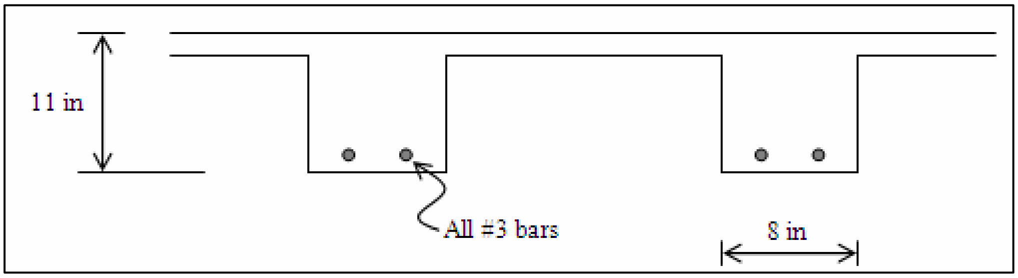

Figure 3. Floor system detail

Analyzing the system it is shown that the factored moment required, Mu = 28 k- ft, which is sufficiently carried by the design.

A computer generated moment envelope for the factored loads of the worst case girder (first floor, interior) is as follows:

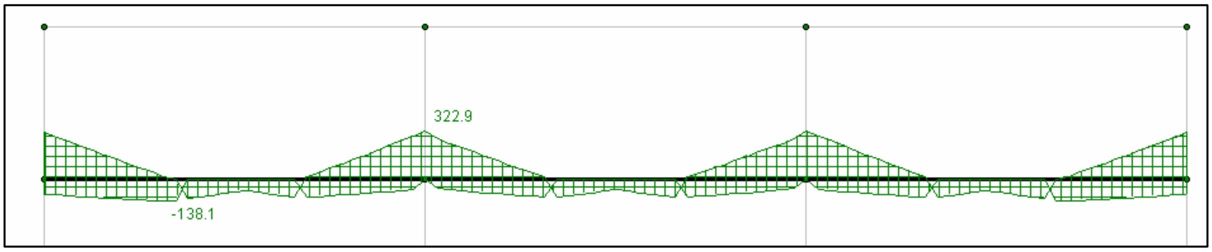

Figure 4. Moment diagram for interior girder, frame A.

The girder also has a worst axial load, P_u = 86 kip. With a max M_u = 323 k-ft, the beam-column was designed: A 16” by 27” cross section with three No. 10 bars on the bottom to carry the moment. To carry the axial load, the bars are continuous, and three No. 10 bars are placed on top to make the member symmetrical as required for a column.

The column’s moment envelope from RISA resulted in the following:

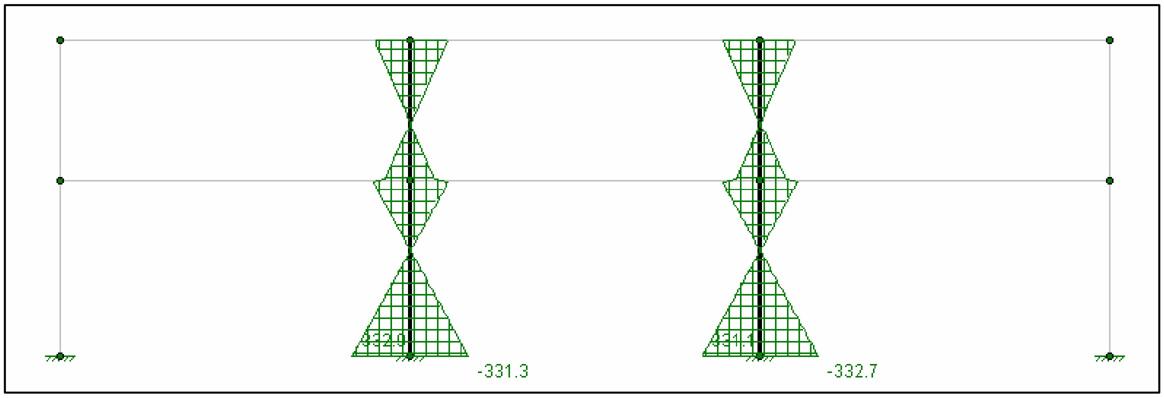

Figure 5. Moment envelope for interior columns, frame A.

Also, an axial requirement of 102 kips was reported. To accommodate this load, and the 333 k-ft of moment at the base, a 22” by 22” design was produced. Reinforcement was decided upon by following ACI interaction diagrams; so eight #10 bars in a square arrangement are necessary.

The footing for this frame has to withstand a P_u=144 kips and a M_u=333 k-ft. An 18” thick, 10’ by 10’ foundation was designed, and eight #11 bars are placed for reinforcement (4 per dimension).

Frame B

An isolated base is used for frame B. This allows the design to consider the earthquake loads negligible.

The purpose of base isolation is to increase the structure’s period, T, from T_A = 0.2 sec (frame A) to target effective periods of T_D = 2.5 sec and T_m = 2.7 sec. Firstly, an isolator that complies with UBC-97 standards needed to be selected. The three basic requirements specify that the system has to 1) Be stable 2) Provide increasing strength with increasing displacement and 3) Not degrade under cyclic loading. A high damping rubber bearing was selected as the isolating device. It is one of the most commonly used devices, so numerous studies have quantified its properties and behaviors. A soft rubber with a modulus of rigidity, G = 58 psi is assumed. Only one bearing design that accommodates the worst load (from an interior column) is used for all 16 locations to save on the cost of using multiple molds. Figure 6 depicts the selected isolator and its final design dimensions.

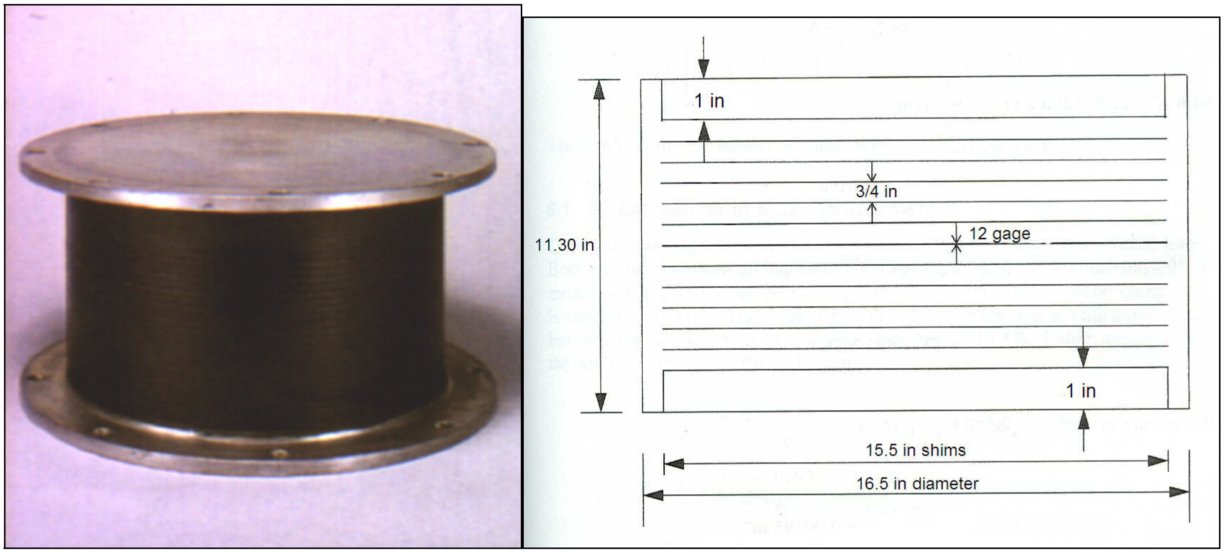

Figure 6. High damping rubber isolator and dimensions

As expected, the girder's moment diagram shows a much smaller worst-case value compared to frame A. Also, a negligible axial load results, and minimum axial load Code requirements are satisfied by the concrete cross section alone.

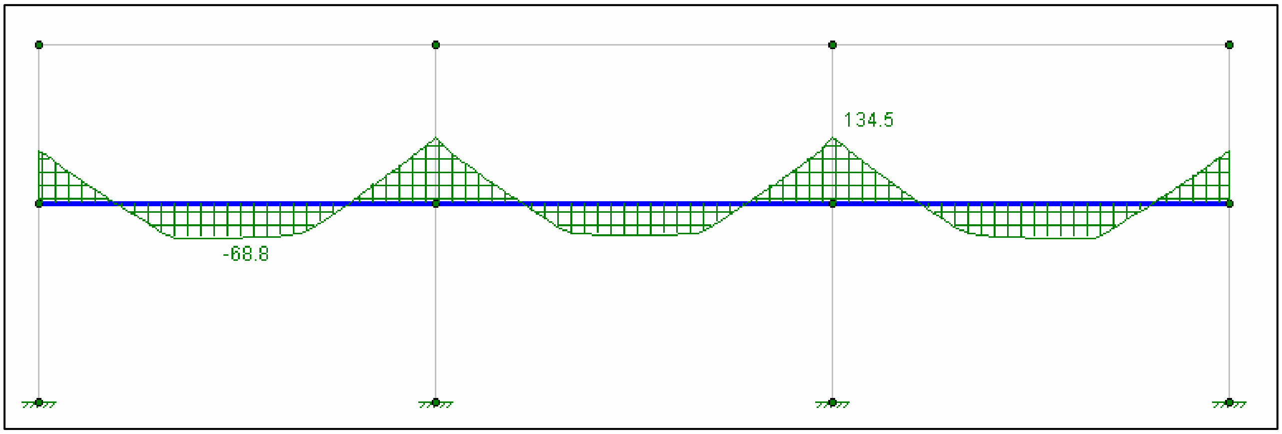

Figure 7. Girder moment diagram, frame B.

Figure 7 represents the moment diagram for an interior girder on the first floor, (worst case). The designed girder is a 12" by 21" face with reinforcement detailed below. The same member is used for all girders.

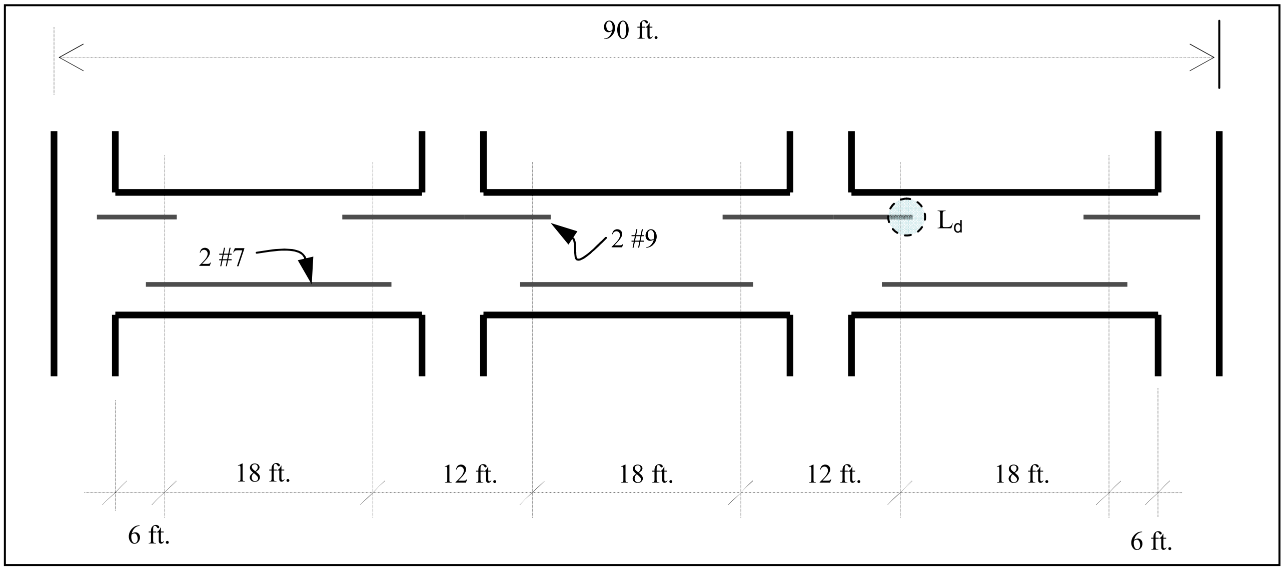

Figure 8. Girder reinforcement detail, frame B.

Development lengths, L_d, as shown in Figure 8, were not determined because it is assumed they do not contribute a significant change in the total amount of steel for the structure. They are depicted because they would have to be considered in a more finalized design, (as required by ACI).

The columns are assumed to only take axial load, and minimal moment capacity requirements are provided as recommended by ACI. For an interior column on the first floor, the P_u = 88 kips. An 18” by 18” column provides a much larger capacity than required, but it is a conservative design that will not be prone to buckling. For this column, only minimal reinforcement is required. Four #9 bars are placed at each corner with a minimum amount of concrete cover, the least amount of steel the Code suggests.

The foundations in this frame are only subjected to a gravity load, P_u = 125 kips. Using an 18” thick base, design calculations result in a 6’ by 6’ footing with 4 #11 bars each way.

When isolating a frame, the base-level diaphragm needs to be reinforced, to restrict independent movement of the columns. Minimal code provisions are required for this, so they are not considered in the comparison of materials.

Comparing frame A and frame B

As mentioned before, the research approach to compare costs is to get weight ratios of several elements, and also that of the entire structure. The table below presents results for the design weights, by elements, for each frame.

Table 1. Material take-off by element

	Fixed-Base Moment Resisting Frame (frame A)		**Isolated Base Frame (frame B)**	
Structural Element	**Concrete Weight,** kips	**Steel Weight,** kips	**Concrete Weight,** kips	**Steel Weight,** kips
Beams	295.4	2.3	295.4	2.3
Girders	636.6	37.1	375.7	7.5
Columns	212.1	18.6	144	5.9
Foundations	357.9	6.8	128.4	4.1
Total	**1502**	**64.8**	**943.5**	**19.7**

From Table 1, the following ratios were computed:

Table 2. A / B weight ratios of main structural elements

	Concrete ratio	**Steel ratio**
Girders	1.7	4.9
Columns	1.5	3.2
Foundations	2.3	1.8
Entire Structure	**1.6**	**3.3**

Conclusions

As expected, base isolation turns out to be the best design, economically speaking; this is due to the fact that the cost of the isolators will not surpass the need for 3.3 times more steel in the reinforced frame. The rubber bearings utilized are very common, so several manufacturers can offer competitive prices. Also, the same isolator is used in 16 footings, thus requiring just one cast or mold.

The results obtained are fairly typical, yet not all-inclusive. Design outcomes for other parameters can vary significantly. Factors that influence structural design are: importance of the building, architectural requirements, soil profile, height of the structure (more so than plan dimensions in some cases), regional issues (cost of labor, availability of specific materials, etc.), and many others.

Another interesting find, was that base isolation can be used to protect non-structural elements and equipment. In figure 8, a frame affected by a seismic loading is still standing, but the curtain walls are destroyed. If the frame had been isolated as a whole, the walls would have been subjected to a less destructive vibration, and probably still be standing in good shape.

Figure 8. An RC structure affected by 2003 Earthquake in Algeria

Base isolation can also be used to protect mechanical equipment or large fragile objects, as in a museum. For example, if it turns out that retrofitting a museum building is not cost-effective; objects that it houses can be fitted with smaller scale isolation systems to be protected in case of a seismic event.

References

Paz, Mario. *International Handbook of Earthquake Engineering : Codes, Programs and Examples.* October, 1994; Chapman and Hall. London, England.

McCormack, Jack. *Design of Reinforced Concrete.* 2001; John Wiley & sons, Inc. New York, New York.

Kelly, James M.; Naeim, Farzad. *Design of Seismic Isolated Structures: From Theory to Practice.* 1999; John Wiley & sons, Inc. New York, New York.

Appendices

Appendix A-1. Shear reinforcement sample calculation.

For girder in Frame B:

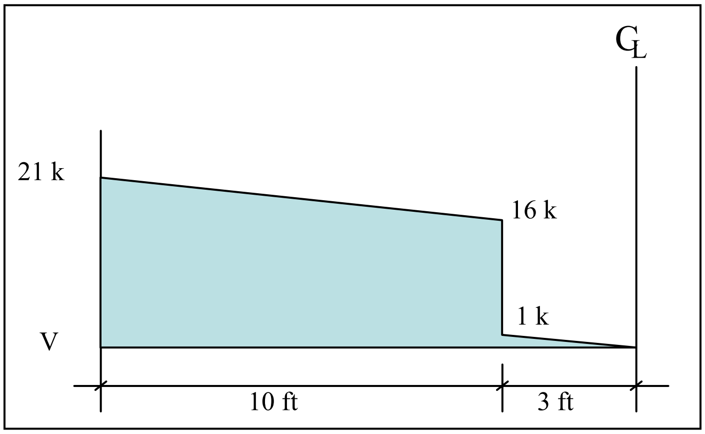

Figure A-1. Shear diagram for Frame B girder (worst case)

Width, b_w = 12”
Depth, d = 18”

$$\theta V_c = 2\sqrt{f'c}\,b_w d \longrightarrow \text{(ACI 11.3.1.1)}$$

$$\theta V_c = (.85)2\sqrt{3000}(12)(18) = 20.1 \ \textbf{kip}$$

$$Vu(@18") = -\frac{5}{120}(18) + 21 = 20.3 \ \textbf{kip}$$

$$Vu > \tfrac{1}{2}\phi Vc \quad , \quad \therefore \ \textbf{Reinforcement is required} \longrightarrow \text{(ACI 11.5.5.1)}$$

Use maximum spacing for #3 stirrups:

$$\text{s = smaller of -}\left[\frac{\dfrac{A_v f_y}{50 b_w} = \dfrac{2(0.11)(60000)}{50(12)} = 22"(ACI - 11.5.5.3)}{or\ d/2 = 9"(Vs = 0)(ACI - 11.5.4.1)}\right], \ \mathbf{s_{max} = 9"}$$

So all stirrups would be placed at 18” from support spaced at 9” for 10’-6”

Appendix A-2. Base isolation device design

From the 1997 Uniform Building Code,

$Z = 0.3$ → (Sec. 1653)

$S=S_D$ → (Table 16-J)

Assumed seismic source type => Type A → (Table 16-U)

Active fault is ∼ 10 km from site, so

$N_a = 1.0$ → (Table 16-5)

$N_v = 1.2$ → (Table 16-T)

$ZN_v = (0.3)(1.2) = 0.36$

Interpolating for M_m in Table A-16-D,

$M_m = 1.35$

$C_v = C_{vD} = 0.54$ → (Table 16-R)

$C_A = 0.36$ → (Table 16-Q)

For $M_m ZN_v = (1.35)(0.36) = 0.49 > 0.40$

$C_{VM} = 1.6 M_M ZN_V = 0.778$ → (Table A-16-G)

For $M_M ZN_a = 1.1(0.405) = 0.446$ → (Table A-16-F)

For an ordinary moment resisting frame, $R_I = 2.0$ (Table A-16-E)

A laminated rubber bearing with soft rubber is assumed to have 15% damping.

$B_D = B_M = 1.35$ → (Table A-16-C)

For preliminary design, the isolation system should provide effective isolated periods of :

$T_D = 2.5$ sec $T_M = 2.7$ sec

From Eq. 4.7 and Eq. 4.8 (Naeim, Kelly),

$$T_D = 2\pi\sqrt{\frac{W}{K_{D\,\min}\,g}} \qquad \text{and} \qquad T_M = 2\pi\sqrt{\frac{W}{K_{M\,\min}\,g}}$$

where,

W = weight of building
g = gravity

An approximated weight of W = 352 kips is assumed from beam, girder, columns and slab information.

Solving equations 4.7, 4.8 for K gives

$$\mathbf{K_{Dmin} = 5754.2\ {}^{lb}/_{in}} \qquad \text{and} \qquad \mathbf{K_{Mmin} = 4933.3\ {}^{lb}/_{in}}$$

Estimation of design displacement, D_D :

$$D_{D,M} = \frac{gC_{VD,M}T_{D,M}}{4\pi^2 B_{D,M}} \qquad \text{(Eq. 4.2, 4.3, Naeim)}$$

Substituting values,

$$\mathbf{D_D = 9.79\ in} \qquad \text{and} \qquad \mathbf{D_M = 15.23\ in}$$

The Code requires design for a 5% accidental eccentricity, e :

$$\mathbf{e = (.05)(90)12 + 3.28(2) = 93.36\ in}$$

$$D_{T_{D,M}} = D_{D,M}\left(1 + y\frac{12e}{b^2+d^2}\right) \xrightarrow{\hspace{4cm}} \text{(UBC 58-5, 58-6)}$$

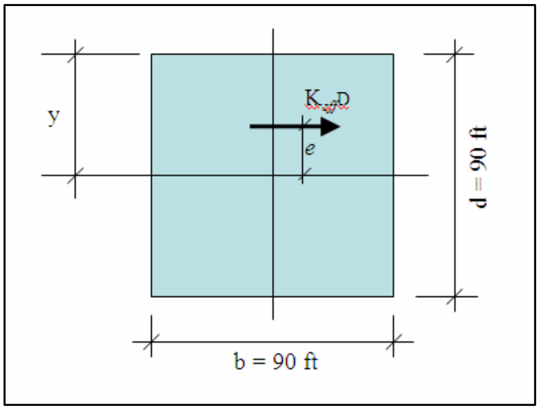

Figure A-2. Accidental eccentricity

$y = (90)(12)(0.5) = 540$ in

$b^2 + d^2 = \left[(90)(12)\right]^2(2) = 2332800\ \text{in}^2$

$12e = 12\ (93.36) = 1120.2$ in

$\mathbf{D_{TD} = 12.33\ and\ D_{TM} = 19.18\ in}$

Check total displacements against minimum values:

$$D'_{D,M} = \frac{D_{D,M}}{\sqrt{1+\left(T/T_{D,M} \right)^2}} \longrightarrow \text{(UBC 59-1,59-2)}$$

With T = 0.2 sec (From Frame A design)

$$\mathbf{(D'_D = 9.42\ in) < (D_{TD} = 12.33\ in)}$$

$$\mathbf{(D'_M = 14.70\ in) < (D_{TM} = 19.18\ in)}$$

Soft rubber bearing design:

Design load is worst axial load, P = 88 kips; this load will be used to design a single bearing for all footings to save on mold costs.

Rubber properties are G = 58 psi, $\beta = 0.08$, Max shear strain, $\gamma = 1.5$

Horizontal stiffness

$$K_H = P\left(\frac{2\pi}{T_D}\right)^2 \qquad \text{(metric)}$$

P = 88 kip = 39.82 metric tons

KH = 0.252 MN/m = 1.44 $^{k}/_{in}$

$\gamma = D / t_r$, where t_r =thickness of rubber

$$\mathbf{t_r = 8.2\ in}$$

$K_H = GA / t_r$ and solving for A with K_H and G gives a required area, **A = 203.6 in^2**

A trial bearing diameter is used, **$\phi = 16^1/_2$" , A = 213.8 in^2**

Bearing pressure, p = P/A = 412 psi

Actual horizontal stiffness, $K_{H_{act}} = 1.51\ ^{k}/_{in}$

Composite stiffness, $K_{H_{com}} = (16 units)(1.51\, k/in) = 24.16\ ^{k}/_{in}$

Horizontal period, T_H :

$K_{H_{com}}$ = (24.16 k/in)(1/5.72) = 4.22 MN/m

W = (352 k)(1/2.206) = 159.6 metric tons

Actual squared frequency , $w^2_H = (4.22 \times 10^6 \text{ N/m})(1/159550)$, so $w_H = 5.15\ {}^{\text{rad}}/_{\text{sec}}$

$$T_H = \frac{2\pi}{w_H} \qquad\qquad \therefore \quad \mathbf{T_H = 1.22\ sec}$$

Composite damping remains 8% since $\beta_{com} = \dfrac{w_D}{2\pi K_H D^2} = \dfrac{K_H \beta}{K_{H_{com}}} = 0.08$

From UBC-97 Table A-16-C for β = 8% ,

$$1.0 + 3/5(1.2\text{-}1.0) = \mathbf{B_D = 1.12}$$

$$D_D = \frac{g}{4\pi^2} \cdot \frac{C_{V_D} T}{B_D} \xrightarrow{\qquad\qquad\qquad\qquad\qquad} \text{(UBC 58-1)}$$

$$\mathbf{D_D = 5.73\ in}$$

From earlier calculations, D_T = 12.33, but it seems too large, so the real torsional stiffness, K_θ

$$K_\theta = \sum_{i=1}^{n} K_{H_i} \left(x_i^2 + y_i^2 \right)$$

where x and y are distances from isolator to plan centroid.

$$\mathbf{K_\theta = 72480\ k\text{-}ft = 869760\ k\text{-}in}$$

The applied torque comes from:

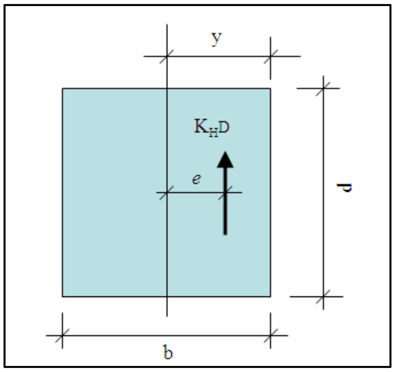

Figure A-3. Composite torque

$M = (K_H D)e$

So additional displacement is

$\theta y = (K_\theta D e / K_\theta)\ y = 4.64$ in

Total displacement , $\mathbf{D_T = 10.37\ in}$

$\gamma_{max} = D / t_r = 1.3 <$ (1.5 assumed)

Check minimum torsion allowance:

$D_{TDmin} = 1.1\ D \xrightarrow{\hspace{6cm}}$ (UBC 1658.3.5)

$1.1(5.73) = (6.3\ \text{in}) < (D_T = 10.37\ \text{in})$

Use $D_T = 10.37$ in

Bearing Dimensions

Set vertical frequency $f_v = 10$ Hz, then from $6S^2 \approx \frac{f^2{}_v}{f^2{}_H}$ and solving for S ,

$$S = \frac{1}{\sqrt{6}} \cdot \frac{10}{\left(1 / T_H \right)} = 4.98$$

For the vertical frequency, it is necessary to have a small strain shear modulus, $\gamma \approx 20\%$. The compound will then have the following properties:

$G_{0.2} = 101.5$ psi and K = 290 ksi (from common lab tests, pg. 99, Naeim)

Then $$E_c = \frac{6GS^2K}{6GS^2 + K}$$

where,

G = 101.5 psi = 0.7 MPa
K = 290 ksi = 2000 MPa
$S^2 = 4.98^2 = 24.8$

So,

$E_c = 14355.6$ psi and **$K_{vcom} = 5{,}988{,}736\ ^{lb}/_{in}$**

$w^2_v = 1049 \times 10^6\ ^N/_m (1 / 159.55 \times 10^3\ \text{kg}) = 6574.74\ \text{sec}^{-2}$

$w_v = 81.08\ ^{rad}/_{sec}$

$f_v = 12.91$ Hz (close enough to assumed 10 Hz)

So $S \approx 5.5$ is adequate

$S = \dfrac{\Phi}{4t}$, where t is the thickness of each layer. Solving for t, gives t = 0.75 in

The total thickness of rubber, $t_r = 8.2$ in, and $nt = t_r$, where n is the number of layers.

∴ Use 11 layers with a thickness of t = ¾ in (nt=total rubber thickness = 8 ¼ in)

Final details are shown in Figure 6.