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DESIGN OF A SEISMIC SIMULATION FRAME FOR TESTING OF MASONRY STRUCTURES

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Abstract

Simulating seismic loads on masonry structures generally requires the use of a seismic simulation table also known as a “Shake Table”. These shake tables tend to be cost prohibitive and, due to the limited size of most systems in use, restrictive on the size of test specimens. The center of the shake table is often unused as the masonry walls are generally built along the perimeter of the Shake Table. Thus, a seismic simulation frame (Shake Frame) for experiments on masonry structures was designed to test masonry walls subjected to 300 kN dynamic loads with one degree of freedom. The final design of the shake frame was a rectangular braced frame 3 m x 2 m, constructed of W360 x 162 beams with HSS89 x 89 x 9.5 bracing, on a prefabricated Thomson Accumax™ Linear guide profile rail system built on a C200 x 15 channel sections base frame.

Key Words

Shake Table, Shake Frame, Seismic Load Simulation, Masonry

1 Introduction

The destruction and devastation caused by earthquakes is evident world wide. However, the most significant losses, in terms of loss of lives and the number of displaced people, is due to seismically inadequate low-cost masonry housing. It seems that in recent years, the largest losses occur in developing regions. In these regions, the cost of decent housing is generally beyond the reach of a large proportion of the population, leaving low cost housing the only alternative available. Recent examples include the Maharashtra (India) earthquake on September 30, 1993, and the Bhuj (Gujurat, India) earthquake on January 26, 2001. These two earthquakes caused 8000

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and 20,000 fatalities respectively, and destroyed or damaged 1.5 million homes with the cost of damages estimated to be in the order of US$5 billion. The majority of the earthquake-damaged dwellings in both incidences were of non-engineered stone masonry construction [Nikolic-Brzev et al. 1998]. In particular, non-engineered masonry construction constituted 95% of the building stock in the Gujurat region. The collapse of these masonry buildings was the main reason for the huge loss of life during this event [EERI 2001]. To increase the earthquake resistance of low cost masonry housing, methods of improving seismic load resistance, are being researched at the University of Calgary. This type of research requires a testing apparatus that can simulate seismic loads on masonry structures. Another area of research that has generated much interest in the masonry industry is in the use of FRP materials as connectors. Inexpensive, but effective corrosion free connectors are sorely needed and the investigation of glass fibre reinforced polymer ties at the University of Calgary has demonstrated there is great potential in this area. However, this area also requires the ability to simulate seismic loads on masonry structures since many areas in Canada are seismic regions. This type of testing generally requires the use of a seismic simulation table also known as a "Shake Table". Typically, the masonry structure is built on the shake table, which is connected to actuators that load the table with a known dynamic load, of a known load function and frequency. These shake tables tend to be either heavy and large in order to test full scale specimens without deflection of the table, or small and limited to the use of scaled models. As a result, the large tables often require powerful actuators to simulate the lateral ground motions that induce inertia of the structure caused by seismic activity and are cost prohibitive. On the other hand, the small shake tables are restrictive on the size of test specimens. Thus, a happy medium is needed. When studying the dynamic behaviour of masonry, the center of the shake table is often unused as the masonry walls are generally built along the perimeter of the shake table. For this reason, a seismic simulation frame (Shake Frame) for experiments on reinforced and unreinforced masonry structures was designed and will be constructed at the University of Calgary to test masonry walls subjected to seismic forces with one degree of freedom. This paper explores the design of a shake frame that is able support up to four two-story masonry wall specimens for testing of both the static and dynamic performance of these masonry specimens.

2 Shake Frame Design

2.1 Design Considerations

The original idea for the shake frame was to create a relatively low-cost testing apparatus that could be easily dismantled yet was able to test the dynamic response of masonry wall specimens up to two stories high and with lengths that varied from two to three meters. These specimens were envisioned to be built on the frame and could be connected as corner-connected walls or simply stand-alone single walls. The shake frame considered would be used for one degree of freedom dynamic testing, which required the frame to translate horizontally in one direction only, with as little friction as possible. Horizontal translation in one direction was deemed adequate as most structures have a large reserve of strength in the vertical direction. As a result, vertical accelerations produced by seismic activity were deemed less important than their horizontal counterparts. Consequently, it was determined that initially enough could be learned from only one horizontal component of earthquake motion.

The vertical load considered in the design of the shake frame was that of 4 two-story walls with a total length of 10 meters and a live load for a roof or floor that could be modeled in the lab by attaching a large concrete beam to the top of the walls or by applying vertical loads with an actuator. These loads were considered to be uniformly distributed and the result was a 51 kN/m dead load and a 4 kN/m live load. After applying the appropriate safety factors the result was a final factored load of 60 kN/m. In order to simulate seismic loads, three characteristics of earthquakes had to be mimicked. The first characteristic considered was displacement. For the most severe earthquakes displacement was estimated to be 250 mm. Secondly, typical ground velocities in past earthquakes were recorded between 0.200 and 0.35 m/s. Finally, typical peak ground accelerations during recorded earthquakes ranged from 0.3g to 0.6g. Based on these characteristics and the estimated mass of the unfactored 4 two story masonry walls, two actuators with a maximum load 150 kN each were selected to provide the lateral seismic force. This load would be applied directly to the frame at a load frequency of up to 4 Hz, and displacement of up to 125 mm in each direction (250 mm stroke). No safety factor was applied to the 300 kN dynamic load supplied by the two actuators as this load can be monitored and controlled.

The last design consideration was that the shake frame would experience negligible deformations during testing. This was necessary because, accurate measurement of specimen deformations and forces requires the frame to have negligible deformations.

2.1.1 Finite Element Analysis

The shake frame was modeled to scale using SAP2000 Non-Linear for the purpose of determining the design parameters of shear, bending, axial forces, and reactions at the supports. SAP2000 was used because of its ability to model dynamic loads and the fact that Canadian steel codes and sections were incorporated into the software. Two models were created using this software. Both models were subjected to worst-case scenarios when obtaining the parameters required for the design. The worst-case considered was when one actuator failed while the other continued to apply force. The result was a couple causing the shake frame to torque. In this event two scenarios were considered. The first scenario assumed the cross bracing was present. This event generated the worst axial load that the bracing would experience. In this event the two cross braces experienced a 104.6 kN load each, one in tension and the other in compression. The second scenario assumed no cross bracing was present. In this event only the moment connections resisted the couple generated by the single actuator and the support. This event produced an end moment in the beams of 75 kN-m. The largest forces were produced in the first model where the factored loads were much higher than the self-weight loads calculated by SAP2000.

The first shake frame model used only frame elements and was simplified by assuming a superimposed dead load of 60 kN/m acted on all four of the beams. This superimposed dead load accounted for the weight of the specimens on the frame. The dynamic load produced by the actuators was also applied to the frame. However, this load is discussed in more detail later. This model was used to determine the design parameters of shear, bending, member axial forces, and support reactions. In this model the bolted connections at the corners were modeled as rigid moment connections. The bolted connections for the cross bracing were modeled as shear connections by a feature in SAP2000 that allowed for the offset of member length by the gusset plate length and releasing of the frame from moments about the 2 principle axes. Under the worst-case event the top left corner was pinned while the right side actuator applied a 150 kN force. The results are tabulated in Table 1 below.
The second model was constructed of shell and frame elements. Here the masonry was modeled using shell elements and the shake frame was modeled using frame

elements and parameters that were identical to the frame created in the first model. Thus, the only difference between the models was the removal of the 60 kN/m superimposed dead load and the application of the masonry shell elements. These masonry shell elements were given a modulus of elasticity of 8.5 GPa and a density of 2400 kg/m^3. The live load from a floor or roof was accounted for by the addition of another meter of height to the wall. As a result, the height of the wall modeled in SAP2000 was seven meters instead of six. This model performed its calculations taking into consideration the self weight of the wall and frame. This was used to examine shear development, moments, accelerations and deflections in the proposed walls due to a dynamic load. This model was also used to illustrate shear development in the corners of the walls due to inertia, and to determine the fundamental period of the frame with actual brick walls rather than with an idealized uniform distributed load. Again, the dynamic load was produced by the actuators.

The dynamic load applied by each actuator was modeled as a 150kN load, with a sinusoidal load function and a frequency of 4 Hz (period of 0.25s). This function was governed by $P(t) = P_o \sin(\varpi t)$, where P_o was 150 kN, and $\varpi = 0.25s/2\pi$ yielding: $P(t) = P_o \sin(25.13274t)$. The load was applied to the bottom right and left corners (referring to Figure 3) of the shake frame. The results of the SAP2000 analysis are discussed below.

2.1.2 SAP 2000 Results

The results provided by SAP 2000 are tabulated in Table 1 and Table 2 below. Table 1 contains the loads produced by the worst case scenarios, where as Table 2 contains the deflections, fundamental periods, and accelerations of the worst-case scenarios as compared with simplified models (see Section 2.1.3).

Table 1: SAP 2000 model Loads

	Max Moment (kN-m)	Max Moment In-Plane (kN-m)	Max Shear (kN)	Max Shear In-Plane (kN)	Support Reactions (kN)	Max Compress Force in the bracing (kN)	Max Tensile Force in the bracing (kN)
Model 1	-39.5	-75	95.8	86.9	158.8	104.6	85.41
Model 2	-9.68	-75	18.9	86.9	82.7	104.6	85.41

Table 2: Deflections, deformations, and Fundamental Frequency

	Largest deflection y-direction (mm)	Largest deflection z-direction (mm)	Max Acceleration y-direction (m/s^2)	Fundamental Period (s)
SAP Model	2.597	0.22841	30.66	0.047
Simplified Model	8.889	N/A	27.97	0.047

2.1.3 Simplified Models

Some simplified calculations were used as estimates to verify the results of the finite element analysis. The first model was used to estimate the load on the bearing supports of the shake frame due to the dynamic lateral load. The model assumes that the maximum lateral force applied to the wall would be 0.6g. The self weight of the wall was neglected in the calculation as it would oppose the overturning moment and reduce the load on the bearings. The simplified model is illustrated in Figure 1.

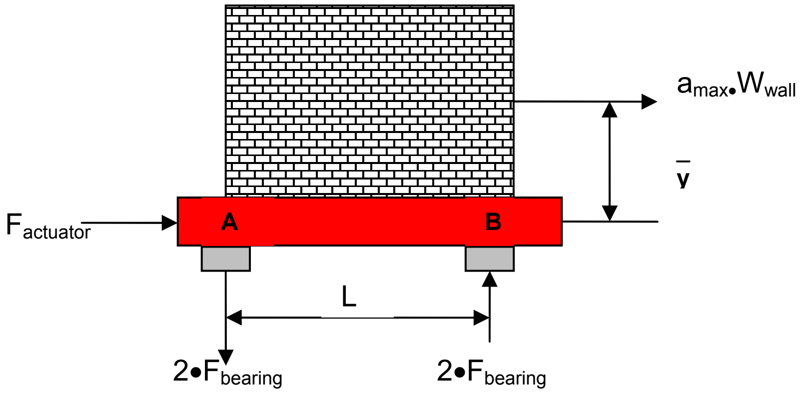

Figure 1 Simplified Shake Frame Model 1

Summing the moments around the bearings at “B” yields:

$$F_{bearing} = \frac{W_{wall} \cdot a_{\max} \cdot \bar{y}}{2L}$$

$$\Rightarrow F_{bearing} = \frac{33{,}600\ kg \cdot (0.6 \cdot 9.81\ m/s^2) \cdot 3.182\ m}{2(2\ m)}$$

$$\Rightarrow F_{bearing} = 157.3\ kN$$

The second simplified model shown in Figure 2 was used to estimate the natural period and acceleration response of the wall specimen in its dominant mode of vibration. This model assumed the wall was a 6 meter high cantilever, 3 meters long, 200 mm thick, has a total mass of 35,470 kg and is subjected to a ground acceleration of 0.6g at a frequency of 4 Hz. The total mass was increased to 35,470 kg because this model included the 1870 kg mass of the shake frame. The wall was assumed to have a modulus of elasticity of 8.5 GPa (that of structural masonry with a compressive strength of 10 MPa) and a moment of inertia of 0.45 m^4. Using equations resulting from the dynamic analysis of distributed parameter systems, the following results were obtained:

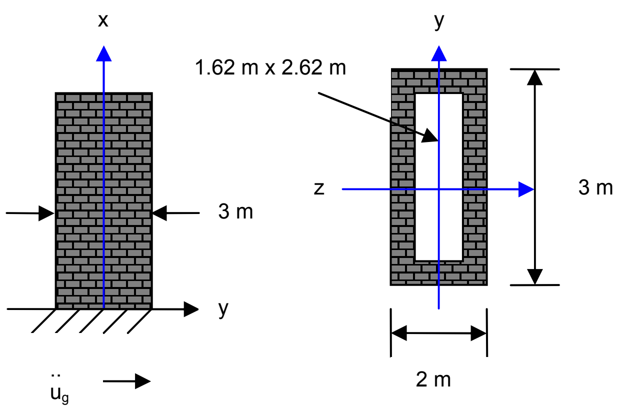

Figure 2 Simplified Shake Frame Model 2

$$E = 8.5\ GPa$$

$$I_z = \frac{(2\ m)\cdot(3\ m)^3}{12} - \frac{(1.62\ m)\cdot(2.62\ m)^3}{12} = 2.0721\ m$$

$$m = \frac{35{,}470\ kg}{7\ m} = 5067.1\ kg/m$$

$$\xi = 0.03$$

$$\ddot{u}_g = -5.7\sin(\varpi t) = -5.7\sin(\varpi t)$$

$$p(x,t) = -m\ddot{u}_g$$

The governing equation of motion of this distributed parameter system is:

$$\Rightarrow EI_z \frac{\partial^4 y(x,t)}{\partial x^4} + c\frac{\partial y(x,t)}{\partial t} + m\frac{\partial^2 y(x,t)}{\partial t^2} = p(x,t)$$

This equation must be solved by the Separation of Variables technique (Fourier's Method) and is found in most structural dynamics text books. Without derivation the solution is:

$$y(x,t) = \phi(x)\cdot q(t)$$

where:

$$\phi(x) = K\left[\cosh(\frac{1.875x}{7m}) - \cos(\frac{1.875x}{7m}) - \frac{\cosh(1.875)+\cos(1.875)}{\sinh(1.875)+\sin(1.875)}\cdot\left(\sinh(\frac{1.875x}{7m}) - \sin(\frac{1.875x}{7m})\right)\right]$$

where :

$$K = 12$$

$$q(t) = C1\sin(\omega x) + C2\cos(\omega x) + C3\sin(\varpi x) + C4\cos(\varpi x)$$

$$\begin{aligned} &where: \\ &C1 = -0.0000668 \\ &C2 = 0.000004161 \\ &C3 = 0.000356 \\ &C4 = -0.00000416 \end{aligned}$$

$$\varpi = 25.1327 \ \ rad/s$$

$$\omega = \frac{3.516}{L^2}\sqrt{\frac{EI_x}{m}} = \frac{3.516}{(7m)^2}\sqrt{\frac{8.5x10^9 Pa \cdot 2.0721 m^4}{5067.1 kg/m}}$$

$$\Rightarrow \omega = 133.8 \ \ rad/s$$

$$\Rightarrow T = \underline{\underline{0.047 \ \ s}}.$$

The 0.047s fundamental period is identical to that obtained in the SAP2000 model suggesting that the mode shape of the distributed parameter simplified model will also be very similar to the SAP2000 model. To verify this assumption, the maximum deflection and acceleration of the distributed parameter model were calculated at a height of 7 m and compared to the SAP2000 model results at the same height. The results are found in Table 2. The comparison demonstrated that results produced by SAP2000 were very close to the estimated values derived from the simplified models. and could be trusted. However, for the purpose of design, the loads generated by the SAP2000 model were tripled to ensure negligible deformations of the shake frame during testing. These values are found in Table 3.

Table 3: Design Loads

Beam		**Bracing**		**Corner Joints**		**Bracing Joints**	
M_f	118.5 kN-m	**C_f**	313.8 kN	**V_f**	670.8 kN	**C_f**	313.8 kN
V_f	287.4 kN	**T_f**	313.8 kN	**$(V_f)_{in\text{-}plane}$**	225	**T_f**	313.8 kN
$(Vf)_{in\text{-}plane}$	225 kN			**C_f**	450 kN		
C_f	450 kN			**T_f**	450 kN		
T_f	450 kN			**B_f**	450 kN		
$(M_f)_{in\text{-}plane}$	225 kN-m			**$(M_f)_{in\text{-}plane}$**	225 kN-m		

3 Final Design

The beams selected for the shake frame were four W360 x 162 steel I-sections. Two of these W360 x162 sections are 3 meters in length center to center and the other two beams are 2 meters in length center to center. These I -beams will be bolted together at the corner connections to form a rectangular shake frame as can be seen in Figure 3 below. The cross-bracing was designed using HSS 89 x 89 x 9.5 square tubing. Two HSS sections 1.7 m in length each and one HSS section 3.4 m in length will be used to construct the bracing. Four rectangular 150 mm x 150 mm x 10 mm gusset plates with two drilled holes will be welded to the ends of each tube and will be used to bolt the HSS sections to the frame. The other ends will be connected at a center joint by fillet welding square plates 300 mm x 300 mm x 10 mm to the top and bottom of HSS sections at their intersection forming a fixed joint. The fixed joint will form an “X” at the center of the shake frame as can be seen in Figure 3.

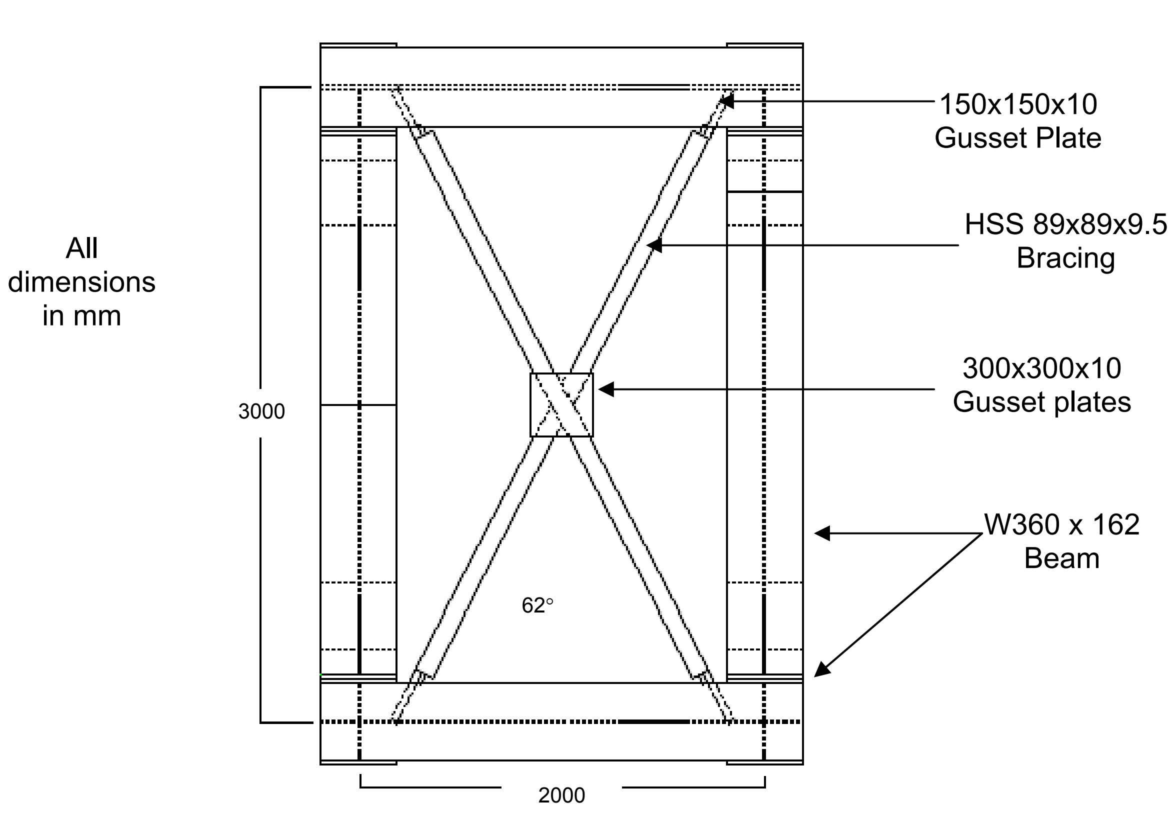

Figure 3 Shake Frame Plan View

The design of the connections assumed that the corner connections connecting the W360 x 162 beams would act as moment connections and the cross-bracing connections would act as shear connections. To connect the I-beams, 371 x 364 x 19 mm plates with eight drilled bolt holes each, spaced 90 mm on center and 47 mm from each plate edge will be welded to the ends of the I-beams. These plates will be fillet welded along the flange and web and butt welded between the plate and flanges with a weld throat thickness of 5 mm. The plate thickness of 19 mm was governed by tensile tear out of the bolts caused by the tensile stress generated by the 112.5 kN-m in-plane factored moment. For bolting the two plates together at the four corner connections, eight AST325 M30 bolts per connection will be used. The number and type of bolts was determined by estimating the amount of bending, shear and tension that the bolts would experience as a result of the moment connection. To connect the cross-bracing four 334 x 185 x 12.5 mm gusset plates will be welded 200 mm from each of the four corner connections along the 2-meter beams. The gusset plates will be fillet welded at an angle of 62° from the web (Figure 3) in order to align with the corners. Two bolt holes will be drilled into these gusset plates and are used to bolt the HSS 89 x 89 x 9.5 bracing to the frame corners. Two AST325 M25 bolts per connection will be used for connecting the bracing to the four 334 x 200 x 10 mm gusset plates (see Figure 3). Web stiffeners with the dimensions of the W360 x 162 beam's web, will be welded to the 2-meter I-beams to transfer load directly to the web of the 3-meter beams. Eight web stiffeners will also be welded onto the 3-meter I-beam, two on each side of the beam 25mm apart and directly above each of the four supports. These were designed to prevent web crippling directly above the supports.

The frame will be mounted on a pre-fabricated linear guide system called an AccuMax™ Profile Rail System. The rail system consists of two 3-meter long rails with 2 supports per rail. This rail system was selected to avoid having to align the supports. The AccuMax™ Profile Rail System was selected because it could support a static load of 191 kN. This value far exceeded the 158.8 kN per support produced by the weight of the shake frame and specimens calculated by SAP2000 and the 157.3 kN per support estimated by the simplified calculation. The 2 supports per rail will be set at 2 meters apart which will allow for translation of 500 mm either back or forth without derailing. The 500 mm allowance is well above the 250 mm stroke of the actuators. These rails will be attached by the supports to the 3 m long W360 x 162 beams 0.5 meters on center from the end of the beams. These supports are bolted into the beam with M20 bolts. The rail system will then be bolted to 25 mm thick plates that will be welded to the top of two back to back C200 x 15 channel sections per side. Two more 25 mm thick plates will be welded to the channels running along the 2 m beam creating a rectangular bottom frame that is used to provide a level sliding surface when bolted to the laboratory floor. This entire system is used to provide near frictionless support to the frame, while ensuring only linear translation.

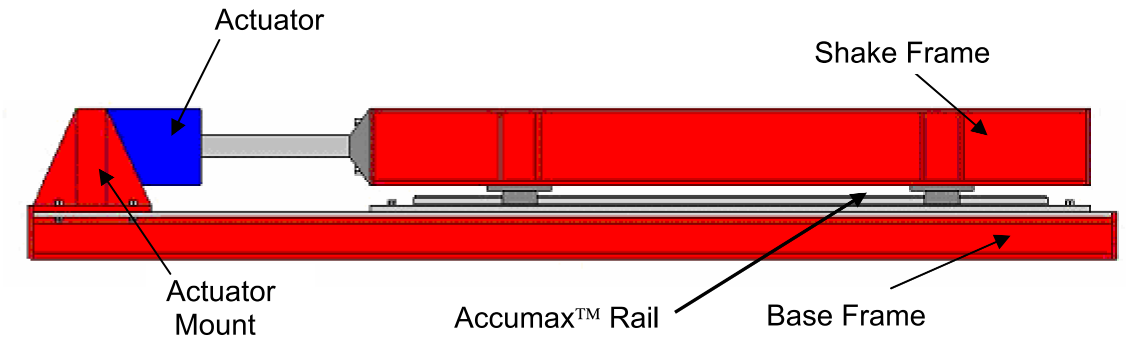

Figure 4 Shake Frame Profile

The ends of the two actuators will be bolted to plates welded on each end of the 2 meter long I-beams of the frame. The actuators will also be mounted on two actuator mounts (see Figure 4) that are bolted to the base frame to ensure the actuators apply the lateral force at the center of the frame cross section. The base frame will be bolted to the laboratory floor to prevent sliding. The design of the base frame and of the actuator mounts are not discussed in detail this paper.

Two other designs for frictionless translation in one horizontal direction were considered. The first design considered the use of Teflon™ bearing pads between the shake frame and the base frame. However, the Teflon™ pads would have to run the length of the frame which would be expensive. Furthermore, the pads would wear quickly due to the mass of the system. The second design considered using ball bearings as the supports. However, the steel ball bearings would be difficult to align and could be dangerous as they would not be able to provide the restraining force at the support preventing the overturning moment created by the dynamic load. Thus, the prefabricated rail system described above was deemed to be the most effective system for the least cost and least difficulty in construction.

4 Conclusions

The final design of the shake frame satisfies the initial vision of providing a low-cost dynamic testing apparatus for masonry wall specimens. Although minor changes may be required during actual construction of the frame in the summer of 2004, no prohibitive cost increases, or major alterations to the design are expected. Furthermore, a second horizontal direction may be able to be added in the future by rotating the table on the supports so that the resultant of the lateral load would be applied at an angle to the masonry wall specimens producing x and y lateral load components. The final design is estimated to cost C$10,000 which is a fraction of the C$2,000,000 cost of a six degree of freedom shake table of similar size. This makes the shake frame a slightly less versatile but far more affordable piece of laboratory equipment.

Acknowledgements

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References

Dawson, Robert V, 1968, Design of masonry, MSc. thesis, University of Calgary, Calgary, Canada.

Ghali, A., Neville, A.M., Brown, T.G. Stuctural Analysis A Unified Classical and Matrix Approach, 5th edition, SPON, London, 2003, pg 629-653

Chopra, Anil K., Dynamics of Structures Theory and Applications to Earthquake Engineering 2nd Edition, Prentice Hall, New Jersey, 2001 pg 629-657.

Nikolic-Brzev 1998: Nikolic-Brzev, S., and S.S. Momin, "Post-earthquake Housing Rehabilitation in Maharashtra", The Indian Concrete Journal, pp. 603-611, 1998.

EERI 2001: Earthquake Engineering Research Institute, "Preliminary Observations on the Origin and Effects of the January 26, 2001 Bhuj (Gujarat, India) Earthquake", http://www.eeri.org/earthquakes/earthquakes.html, 16 p., 2001.