

# DAMAGE MITIGATION USING A PASSIVE ISOLATION SYSTEM

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# SUMMARY

A one-story, lightly-reinforced concrete masonry unit building was constructed on the KSU shake table and subjected to dynamic testing to evaluate its performance when supported on the SDBIS. The model structure was subjected to uniaxial earthquake ground motions from the July 21, 1952 Kern County earthquake, Station 1095 <u>Taft</u> Lincoln School record (peak ground acceleration (PGA) = 0.15 g), the May 19, 1940 Imperial Valley earthquake, Station 117 <u>El Centro</u> Array #9 record (PGA = 0.33 g), the February 9, 1971 San Fernando earthquake, Station 279 <u>Pacoima Dam</u> record (PGA = 1.20 g), and the January 17, 1994 <u>Northridge</u> earthquake, Station 24436 Tarzana Cedar Hill record (PGA = 1.77g) earthquake ground motions. The Northridge displacement record was reduced to 77 percent to accommodate the displacement limitations of the shaking table, resulting in a PGA of 1.17g for the Northridge runs. The earthquake acceleration records were obtained from the displacement time histories for each record.

The structure was instrumented with electrical resistance wire strain gages and seismic accelerometers to measure the response of the structure during the simulated earthquakes. A yardstick was used to approximate maximum building displacements. The isolated structure was subjected to over 200 full-scale earthquakes and no damage was detected.

After completing tests on the isolated structure, the SDBIS system was de-activated and the non-isolated structure was tested to allow comparison of results. The non-isolated structure was subjected to reduced ground motions to ensure the shaking table and equipment would not be damaged. The El Centro displacement record was reduced to 50 percent and the Northridge displacement record was reduced to 19 percent for this phase of testing. Wall strains and base accelerations were recorded and compared to the results from the testing on the isolated structure.

# INTRODUCTION

Masonry is an inexpensive, durable, and readily-available building material, making it a popular construction choice. As a result, lightly-reinforced and unreinforced masonry buildings, both new and existing, are prevalent around the world. These buildings are vulnerable to earthquakes in many areas of the world and often sustain significant damage during seismic events, putting life and property at risk.

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Building codes have addressed this risk by imposing more stringent design and construction requirements on new masonry buildings constructed in seismically active areas. In addition, there has been a significant effort in the United States and other industrialized countries to upgrade existing masonry buildings to improve performance during seismic events. Most of these code provisions and upgrade strategies have been aimed at preventing building collapse, not damage to the structure or its contents.

Seismic isolation provides a means to go beyond collapse prevention and provide damage protection for the structure and its contents. Base isolation reduces the accelerations, and thus the forces, transferred into a building superstructure by decoupling the building from the ground. Kansas State University's (KSU) Stiffness Decoupler for the Base Isolation of Buildings (SDBIS) is a passive sliding base isolation system with elements that provide damping, a centering force, and resistance to overturning. Dynamic shake-table tests were performed on a full-scale masonry building supported by the SDBIS at KSU to confirm its effectiveness in preventing damage.

## **BUILDING MODEL**

A full-scale, one-story, lightweight cmu structure reinforced to comply with the minimum seismic zone 2 (PGA=0.15g) reinforcing required by the 1994 edition of the United States Uniform Building Code was used for the testing. The model structure was constructed of 20.32 cm (8 in) cmu and had plan dimensions of 2.44 m (8'-0") by 3.25 m (10'-8"). The structure was 4.47 m (14'-8") high to represent more slender walls common in one-story commercial applications. The plan dimensions were limited by the shaking table size. The structure was constructed with one standard window opening in the north wall and one standard door opening in the south wall. The openings were located in the walls parallel to the applied ground motions and were placed in opposite walls to minimize torsion. A plan view of the structure is shown in Figure 1, and Figure 2 shows the completed walls before the application of the roof diaphragm (Wipplinger[5]). The steel frame seen in Figure 2 was part of a previous testing program left in place for future testing and was not attached to the cmu building in any way. Clearances were left around the cmu structure to ensure it would not interact with the steel frame during testing. The steel frame added weight to the system, which was included in the design of the SDBIS bearings, but did not affect the results reported here.



Figure 1: Plan of Masonry Model Structure Wipplinger [6], by permission of ASCE



Figure 2: Masonry Model Structure

Certified masons constructed the masonry walls using lightweight Grade N cmu with a minimum specified compressive stress of 13.1 MPa (1900 psi) and Type S mortar. Grout with a minimum specified compressive stress of 13.79 MPa (2000 psi) was placed in the reinforced cells an in lintels and bond beams. Number 4, Grade 60 deformed vertical reinforcing bars were placed at wall corners, adjacent to openings, and at a maximum spacing of 1.22 m (4'-0") on center. Dowels were welded to the steel base of the shake table and lapped with the vertical reinforcing bars to simulate real construction as closely as possible. Bond beams were constructed at the base and top of the walls and were reinforced with two number 4 bars. Standard number 9 k-web joint reinforcing was placed every other course (0.405 m, or 1'-4" on center) to provide horizontal reinforcing.

Since the researchers did not interfere with the masons during the construction process, the construction should be considered un-inspected. As is common for un-inspected masonry, code violations occurred during the construction process. Vertical reinforcing bars were placed after the wall had been constructed higher than the termination point of the dowels and were shoved into the cells after the grout had been placed. No positioners were used to hold reinforcing in the center of the cell. Grouting procedures did not conform to either low-lift or high-lift procedures as outlined by the code. In addition, joint reinforcing was not lapped at the corners or at splices. These oversights make the test building representative of the typical construction found outside areas with stringent code requirements and code enforcement.

The building model roof structure was constructed of standard 2x6 wood joists supporting a  $\frac{3}{4}$ " thick blocked plywood diaphragm. Standard plywood diaphragm nailing was used and the diaphragm was bolted to the masonry wall with  $\frac{1}{2}$ " diameter bolts cast in the bond beam. This was intended to represent a common construction type and provides a flexible diaphragm for the structure.

The cmu building model weighed approximately 93.41 kN (21 kips) and was supported on the steel shaking table, which is in turn supported on the SDBIS system. The approximate period of the masonry structure was 0.1 seconds. Tests were not performed to determine the damping ratio for the model structure, but since the structure did not include finishes and remained elastic, the damping ratio will be quite low. A damping ratio less than 3 percent of critical (Wakabayashi [1], Irvine [2], Chopra [3]) can be assumed for the masonry model structure.

#### **SDBIS SYSTEM COMPONENTS**

The SDBIS system is composed of friction-type sliding bearings, interior supports, exterior supports, flexible coupling elements, optional loose cable bracing, and optional gapped bearing walls. The three elements used in this research are discussed below. A schematic representation of the system is shown in Figure 3.

# **Sliding Bearings**

The friction-type sliding bearings are designed using a filled or unfilled polytetrafluoroethylene (PTFE or Teflon) bearing pad recessed and attached into a steel bearing shoe. The Teflon bearing slides on a stainless steel bearing plate sized to accommodate the maximum system displacement. The stainless steel had a #8 mirror finish as defined by the American Society of Testing and Materials (ASTM) and was coated with graphite before testing began. The stainless steel plate was attached to the shaking table under the base of the structure. The bearing used in the testing program is shown in Figure 4.



**Figure 3:** Schematic Drawing of Stiffness Decoupler for the Base Isolation of Structures (Hu, United States Patent Numbers 5386671 and 5660007, Wipplinger [6], with permission)

The dynamic coefficient of friction for Teflon varies with the velocity of the motion, giving lower values for low velocities, and increases when the bearing pressure on the Teflon falls below 20.68 MPa (3000 psi). The system is designed to develop enough static friction to resist wind loads and small earthquakes without relative motion, but provides a dynamic friction coefficient small enough to isolate the structure. Once the system is activated, the bearing pads provide damping and energy dissipation through friction. The overall damping ratio for the SDBIS system varies, but may be conservatively taken as 20 percent of critical (Hu [4], Wipplinger [5]).



Figure 4: Bearing Pad Under Structure Wipplinger [6], by permission of ASCE

The model structure was supported on four 1.27 cm (0.5 in) thick, 4.44 cm (1.75 in) diameter Teflon bearing pads, resulting in a bearing stress of 23.65 MPa (3.43 ksi). The pads were placed under the base of the structure at each corner, between the structure and the shaking table, bearing on the stainless steel bearing plates attached to the shaking table as shown in Figure 4 (Wipplinger [5]).

#### **Supports**

Supporting elements below the isolation system support the gravity loads from the structure under static and dynamic loading conditions. The support system must also resist additional loads caused by any overturning forces generated during a seismic event. In a real structure, the supporting elements may be a series of concrete columns, piles, bearing walls, or foundation walls. The concrete masonry unit (cmu) model structure was supported on a steel base, which was in turn supported on the SDBIS and then on the shaking table.

#### **Flexural Coupling Elements**



Figure 5: Steel Rod Flexural Element

Flexural elements composed of concrete-filled steel pipes for large structures, or steel rods for small structures, are designed to span between the non-isolated base and the isolated structure, and perform several important functions. These flexural elements act as springs to provide a restoring, or re-centering, force to the structure through their flexural stiffness. As the structure displaces relative to the base, the flexural elements begin to resist the motion and apply an opposing force to the isolated structure. The effective stiffness is adjusted by varying the size of the pipe or rod and by adjusting the end conditions. The flexural element is provided with a free travel distance to allow the design displacement to occur before additional axial stiffness is mobilized. Once the design displacement has been exceeded by an extreme event, a nut and bolt assembly gradually brings the axial stiffness of the rod into play, limiting the maximum displacement of the system under severe loadings. This allows the system to be tuned to a maximum displacement.

The flexural elements also provide resistance to overturning forces through the connections to the base and to the structure. Any tensile uplift forces that begin to develop in the superstructure are transferred into the flexural elements, which transfer the forces into the foundation system.

The flexural elements used for the testing program were composed of 2.54 cm(1in) diameter rods spanning from the shaking table to the first level of the steel frame surrounding the masonry structure (see Figure 5). The rods had fixed ends with a

free travel distance of 0.5 inch. Using the equation  $T = 2\pi \sqrt{M/K}$ , where *M* is the mass of the isolated structure and *K* is the stiffness of the SDBIS (Hu [4], Wipplinger [5]), the isolated structure was determined to have a period of 6.6 seconds.

#### **DYNAMIC TESTING PROGRAM**

Dynamic tests were performed on a 2.90 m (9.5 ft) by 4.42 m (14.5 ft) shaking table. The table was constructed of structural steel and recessed into a 1.83 meter (6 ft) deep concrete pit with a 30.48 cm (12 in) deep reinforced concrete mat. The east, west and south sides of the pit are surrounded by 45.72 cm (18 in) thick reinforced concrete reaction walls. The north wall is a removable retaining wall that separates the structural testing pit from an adjacent pavement testing pit.

Four 88.96 kN (20 kip) capacity steel wheels support the shaking table, which can support a maximum model weight of 289.1 kN (65 kips). The table can accommodate uniaxial shaking with a maximum displacement magnitude of 20.32 cm (8 in) backward or forward, for a total stroke of 40.64 cm (16 in).

Full-scale El Centro and Pacoima Dam earthquake ground motions can be simulated with ninety percent accuracy. The table and data acquisition system specifications are outlined in previous publications by Hu [4] [6] and Wipplinger [5].

The response accelerations of the table and the isolated cmu structure during dynamic tests were measured with seismic accelerometers. The output data (time-histories) were recorded to computer files using a sample rate of 200 scans per second. One accelerometer was attached to the side of the shaking table below the isolation system and one accelerometer was attached to the base of the structure above the isolation system. Additional testing was done with an accelerometer placed at the top of the structure and with accelerometers placed at mid-height at the ends of the shear walls. These results are reported elsewhere (Wipplinger [5]).

In addition to the accelerometers, strain gages were placed at selected locations on the masonry shear walls (parallel to motion) to determine strain levels in the masonry during testing. Strain gages were located at mid-wall and at opening corners. The gages were placed on both faces of the wall when possible. Since the joints are most often where cracks originate, the strain gages were placed in the mortar joints. This caused the gages to be curved and induced an initial strain in the gages. The strain gages were balanced as much as possible and initial values were subtracted out of the strain response records. Strain gage locations are shown in Figure 6, Wipplinger [5]. The strain output data (time-histories) were recorded to computer files. Background noise levels in the multi-channel data acquisition system were too high to record the low levels of strain present in the isolated structure, so standard strain boxes were used to record the data. This resulted in recording only two strain gages at a time, requiring numerous test runs to acquire all the acceleration and strain time-histories. The Each set of tests was performed three times to verify reliability. This resulted in subjecting the isolated structure to over 200 full-scale earthquakes.

After completing tests on the isolated structure, the isolation system was bypassed by welding the steel base to the shaking table platform. Tests were then performed on the non-isolated structure using reduced ground motions from the El Centro and Northridge earthquakes. Base accelerations and wall strain time histories were recorded for these tests as well, and were compared to the data for the isolated structure.

#### **TEST RESULTS**

#### **Acceleration Results for the Isolated Structure**

The dynamic tests performed verify that the SDBIS system is very effective in reducing the accelerations transmitted into the structure and reducing the strain levels experienced in the masonry walls. A summary of the base acceleration results can be seen in the graph shown in Figure 7, Wipplinger [5]. The input motion peak accelerations for the Taft, El Centro, 77% Northridge and Pacoima Dam earthquakes are shown along the horizontal axis of the graph, and the peak accelerations at the base of the isolated structure are shown along the vertical axis. The graph indicates that the SDBIS system is more effective at isolating the structure from strong ground motions than from weak or moderate ground motions. This is the expected and desired result. Two sets of results are shown in the figure. The upper curve represents tests performed in September 1999 and the lower curve represents tests performed in June 2000. The September results were obtained after about 5 months of testing had worn the graphite coating off the stainless steel bearing plates, resulting in a slightly higher transmission of accelerations into the structure. The June 2000 tests were performed after new bearing plates, giving slightly higher reductions than the September results. In addition, the free travel distance for the flexural rods was

increased in November 1999 to help reduce the response of the isolated structure. The two sets of results shown in Figure 7 are reasonably consistent and show the same general trend.



Figure 6: Strain Gage Locations on Masonry Walls



Figure 7: Average Acceleration Reductions using SDBIS Wipplinger [6], by permission of ASCE

Representative time-history acceleration results for each of the individual earthquakes are shown in Figures 8, 9, 10, and 11, Wipplinger [5,6]. The top graph in each figure shows the acceleration record at the base of the isolated masonry structure. The lower graph in each figure shows the acceleration record for the shaking table as produced by the actuator. These figures again confirm the effectiveness of the SDBIS system in reducing the accelerations transmitted into the structure. In addition, the isolated response does not have the sharp acceleration peaks present in the input ground motions.



Figure 8: Acceleration Records for Isolated Structure with Taft Ground Motion Wipplinger [6], by permission of ASCE

Figure 9: Acceleration Records for Isolated Structure with El Centro Ground Motion Wipplinger [6], by permission of ASCE



Figure 10: Acceleration Records for Isolated Structure with Northridge Ground Motion Wipplinger [6], by permission of ASCE

Figure 11: Acceleration Records for Isolated Structure with Pacoima Dam Ground Motion Wipplinger [6], by permission of ASCE

Acceleration reductions across the SDBIS system averaged 34.5 percent for the Taft record, 64.2 percent for the El Centro record, 76.5 percent for the reduced Northridge record, and 77.8 percent for the Pacoima Dam record (Wipplinger [5]). Initial tests produced larger acceleration reductions due to the fresh coating of graphite applied to the stainless steel bearing plate. The graphite coating was not reapplied after each test. Even with the degradation of the graphite coating, the system remained effective in reducing accelerations transmitted into the structure for over 200 dynamic runs. The degradation in the system's performance was minor and verifies the system remains effective with little or no maintenance.

#### Strain Results for the Isolated Structure

Strain results were consistent with the acceleration results. The structure remained elastic and did not experience any stress cracking during testing. The highest strain readings occurred at corners adjacent to the openings, as expected. Some unusually high strain readings at the inside door corner prompted investigation in this area. The investigation revealed mortar was missing in the joint directly behind the strain gage, resulting in the higher readings. The strain levels in the masonry shear walls were all under 300 micro-strain, the typical cracking value for masonry. Visual inspection confirmed no stress cracks had formed in the masonry walls during testing.

Representative time-history strain responses recorded at the window corner and at mid-wall are shown in Figures 12 and 13. The strain time-histories compare well with the acceleration time-histories. Strong peaks in the strain response of the masonry walls have also been eliminated by the SDBIS system.



# Figure 12: Strain at Window Corner in Isolated Structure for Northridge Ground Motion

# Figure 13: Strain at Mid-Wall in Isolated Structure for Northridge Ground Motion

Strain gages were also placed on the flexural rods to determine maximum relative displacements between the shaking table and the isolated structure. The displacement that occurred over the length of the rod was calculated using the principles of mechanics. The top of the rod was attached to the isolated structure, while the bottom was attached to the shaking table below the isolation plane. The maximum calculated displacements agree with observations during testing and are summarized in Table 1.

Earthquake	Input PGA	Relative Displacement	PGA/0.4g
Taft	0.15g	16 mm (0.62 in.)	0.375
El Centro	0.33g	43 mm (1.7 in.)	0.825
77% Northridge	1.11g	254 mm (10 in.)	2.775
Pacoima Dam	1.20g	295 mm (11.6 in.)	3.000

 Table 1: Maximum Relative Displacements

# **Results for Non-Isolated Structure**

In order to prevent damage to the table or testing facility, the non-isolated structure was tested using only 50 percent of the El Centro ground motions and 19 percent of the Northridge ground motions. Even with the reduction in input ground motions, the peak accelerations for the non-isolated structure were higher than the peak accelerations produced by the full-scale ground motions in the isolated structure. The accelerations in the non-isolated structure essentially matched the input ground accelerations generated by the shaking table, with some slight amplification for the Northridge ground motion. Visual observations confirmed that the structure shaking for the non-isolated tests was much more severe than for the isolated structure supported on the SDBIS system. This comparison further verifies that the SDBIS system is very effective in reducing the ground accelerations transferred into the structure. Representative time-history acceleration results for the non-isolated structure are shown in Figures 14 and 15.

The strain results for the non-isolated structure follow the same trend as noted for the acceleration results. Higher strains were recorded for the non-isolated structure subjected to significantly reduced ground motions than were recorded for the isolated structure subjected to full-scale ground motions. Representative time-history strain results are shown in Figures 16 and 17. The peak strain in the wall was

recorded as approximately 376 micro-strain. The walls remained elastic throughout the testing and did not develop any stress cracks. The severity of the motion for the non-isolated structure to the reduced ground motions was remarkable and would likely have caused stress cracks had the input motion been stronger.

In addition to higher peak values, the time-histories for both the accelerations and strains show that the strong peaks in the ground motion are transmitted directly into the structure and that the response of the stiff structure looked very similar to the input ground motions, with some small amplification.



Figure 14: Acceleration Results for Non-Isolated Structure with 19% Northridge Ground Motion





Figure 16: Strain at Window Corner in Non-Isolated Structure for 19% Northridge Ground Motion



## **CONCLUDING REMARKS**

All elements of the SDBIS system performed well, remained elastic throughout the testing, and were effective in limiting the accelerations transferred into the masonry structure during a simulated seismic event. The reductions in accelerations for strong ground motions were significant and consistent throughout the 15-month masonry testing program. The SDBIS compares favorably to systems currently in use. It uses passive energy dissipation through sliding friction, combined with flexural elements to effectively limit displacements, prevent overturning, and provide a restoring force. The SDBIS system is constructed of readily available, easily maintained materials, making it an economical alternative for protecting new and existing masonry buildings from damage caused by moderate and strong ground motions.

The masonry structure did not sustain any damage or experience any instability during the testing. The flexural elements were able limit displacements and restore the system back to near the neutral position after every run. The masonry structure survived over 200 simulated full-scale earthquakes without sustaining any damage and with negligible net displacements.

The maximum displacements experienced by the isolated structure were less than the maximum design displacement calculation as prescribed by the 2000 International Building Code, section 16.23.2.2, for a base isolation system and a design earthquake of 0.4g. The maximum calculated displacement using the code equation for 0.4g and the test model properties is 33.5 cm (13.2 inches), while the maximum observed displacement of the structure supported on the SDBIS system was only 29.5 cm (11.6 inches) for 1.2g. This implies that the SDBIS can withstand strong earthquakes three times the magnitude of the standard design PGA of 0.4g and still not exceed the maximum design displacement. The restoring elements limit the maximum displacements to reasonable levels, limiting damage in the structure and at the isolation plane.

With the current movement toward performance-based design, it will become more difficult to design conventional masonry buildings that are economical and easy to construct for seismically active areas. The SDBIS provides a solution by decoupling the structure from the ground, thereby reducing the transmitted accelerations and limiting damage to the structure and its contents from moderate and strong

ground shaking. The SDBIS reduced strong shaking in the test structure enough to allow a wine glass half-full of water to remain in place through the Pacoima Dam and Northridge earthquake ground motions, without spilling the water.

The SDBIS system also provides an attractive solution for retrofitting existing masonry buildings, many of which are historic in nature. The system can be installed below the first level of the building, leaving the superstructure above the base unaltered.

Applying the SDBIS system to new and existing masonry buildings will likely present project-specific challenges. Different design configuration and foundation options will need to be investigated for each unique structure to determine the optimal arrangement for the system. In the test structure, the flexural rods spanned from the shaking table to the first level of the steel frame structure. In a real building, these elements could span from the foundation to the first floor level, or from a non-isolated basement level to the isolated first floor. Local stresses at the connection of the flexural element to the masonry superstructure will need to be carefully designed to ensure brittle behaviors do not develop. These details should be approached with more than the standard practice of care.

The completed testing program provides verification that the system works well, but may not provide enough detailed information to allow engineers to use the SDBIS for real projects. The inter-related relationships between the damping ratio, natural period, relative displacement and design earthquake acceleration are important and complex and are still under investigation. The authors hope to report the results of ongoing work in the near future.

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