Paper No. 420. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

EXPERIMENTAL INVESTIGATION OF THE USE OF VISCOELASTIC DAMPERS TO REHABILITATE A REINFORCED CONCRETE FLAT SLAB STRUCTURE

JOHN R. HAYES, JR.[1], DOUGLAS A. FOUTCH[2], and PAMALEE A. BRADY[1]

1. U.S. Army Construction Engineering Research Laboratories, P.O. Box 9005, Champaign, IL 61826-9005, U.S.A.

2. Civil Engineering Department, University of Illinois at Urbana-Champaign, 205 N. Mathews, Urbana, IL 61801, U.S.A.

ABSTRACT

The U.S. Army Construction Engineering Research Laboratories (USACERL) and the University of Illinois at Urbana-Champaign (UIUC) constructed a one-third scale model of an existing reinforced concrete flat slab building. The model structure was fitted with viscoelastic dampers (VED) and then tested on the USACERL shaking table. This paper outlines the experimental program and reports preliminary observations on the test results.

KEYWORDS

Viscoelastic dampers; reinforced concrete flat slabs; lightly-reinforced concrete; seismic rehabilitation.

BACKGROUND

Older lightly-reinforced concrete (LRC) frame buildings have performed poorly in moderate to high intensity earthquakes. Such structures were designed using building codes emphasizing gravity loading, without providing the deformation ductility needed to withstand cyclic lateral motions. Shortcomings of LRC structures include inadequate column shear capacity, insufficient confinement of column longitudinal reinforcement, location of column reinforcement lap splices directly above floor levels, and insufficient column reinforcement lap splice length.

Moreover, flat slab-column structures in general provide poor seismic resistance. They lack sufficient concrete and reinforcement volumes in joint regions for adequate transfer of earthquake-induced shears and moments. Laboratory experiments and field experience have shown that this can lead to punching shear failures of slabs in column joint regions. Older slab-column structures also tend to have continuity through joint regions for only negative moment (top) reinforcement, whereas current practice is to provide continuity for both positive and negative moment reinforcement. In addition to strength and ductility inadequacies, the low stiffnesses of flat slab structures can lead to large earthquake-induced motions in buildings, which can damage critical equipment and architectural elements.

Traditional means of rehabilitating these structures can be costly and functionally disruptive. Supplemental passive "damping" devices, which dissipate energy without significant structural damage, may provide efficient alternatives to the conventional technologies. One supplemental energy dissipation technique involves adding VED to an LRC structure. The VED are mounted in diagonal braces that extend between columns in a structure. Earthquake-induced tensile and compressive forces that develop in the braces are carried in direct shear by the VED. The addition of the VED in the braces both stiffens the structure and enhances energy dissipation capacity. The increased damping reduces structural response to ground motion. While the use of supplemental VED is conceptually effective, most prior research has focused on their use in steel structures. There has been concern that VED have limitations that might preclude their use in reinforced concrete structures. Foremost is the possibility that, for VED to deform sufficiently to utilize their energy dissipation characteristics, the parent reinforced concrete structure will have deformed enough to undergo significant cracking, with associated strength and stiffness loss.

MODEL STRUCTURE

Using the case of the slab-column structure as an extreme example of the possible rehabilitation of LRC buildings using VED, this research assesses the efficacy of these devices for rehabilitation and will develop design-oriented models that can be used to analyze rehabilitation schemes. The project synthesizes information from three key areas: the response of LRC column elements to earthquake motions, and the means by which such elements might be rehabilitated; the response and analytical modeling of flat slab-column structures to combined gravity and earthquake loadings; and, the use and modeling of VED in rehabilitating existing structures.

Developing an accurate procedure to rehabilitate an existing LRC slab-column structure is complicated by a number of factors. First, most previous experimental research on LRC structures in general, and flat slab structures in particular, has focused on static and pseudo dynamic tests of individual structural elements, joints, or subassemblages, and most of that research has focused on improving the performance of new structures through improved detailing, not on analyzing or rehabilitating older structures. Only one previous research project (Moehle and Diebold, 1984) involved shaking table tests of a complete flat slab structural system model. There is therefore no strong consensus on the appropriate method for analytical modeling of flat slab-column structures. Key uncertainties are the slab width that effectively acts as a beam, and the shear and flexure transfer mechanisms between slabs and columns. Finally, little or no experimental or analytical research has been performed on slab-column structures that have been rehabilitated with VED.

The prototype for this study is a three wing complex of three story structures that was constructed by the U.S. Army near Tacoma, Washington. A section of one wing of the prototype was modeled in the study. The prototype wing is rectangular in plan, with dimensions of approximately 40 feet by 117 feet. The structural framing system is predominately an LRC slab-column moment frame system. Cast-in-place shear walls at the ends of the long dimension provide lateral force resistance for transverse ground motions, but there are no intermediate shear walls in that direction. In the longitudinal dimension, spandrel beams run the length of the building on both exterior walls, at the top of each story. The spandrels, which are cast monolithically with the floor slabs, support exterior wall and window systems, stiffen the framing system in the longitudinal direction, and aid in shear and moment transfer from the slab to the exterior columns. Columns are founded on individual spread footings.

As-built drawings indicate the prototype was constructed circa 1956. The drawings indicate the designers used the Pacific Coast Uniform Building Code (UBC) and the American Concrete Institute (ACI) Building Code. All analyses for the project are being based on the assumption that the 1955 UBC and 1951 ACI Code were used, because they were in use at the time of construction. The drawings state that the structure was designed to UBC Seismic Zone III provisions, with concrete designed for a 28-day compressive strength of 3,000 psi and reinforcing steel designed for a working stress of 20,000 psi. Review of the structural drawings showed that the characteristic weaknesses of LRC frames and flat slab systems are present in the prototype.

A transverse section near the center of the prototype was selected for modeling. This region was considered the most vulnerable area in the structure, because of its long distance from the transverse shear walls. Without high shear wall strength, and high slab diaphragm strength and stiffness, lateral load resistance will be limited. The model was constructed at one-third of full scale (Figures 1 and 2). Individual story heights are 3' 4" and the model length and width were 13' 4". Reinforcement details and material properties in the model were essentially the same as those in the prototype, to capture the performance limitations of the LRC system. Insofar as possible, reinforcement was scaled to one-third of the prototype. Deformed steel wire was used to model the reinforcement. The prototype reinforcement was assumed to have a nominal yield stress of 40,000 psi. The wire used in the model initially had a yield stress approaching 100,000 psi, so the wire used for longitudinal reinforcement was annealed to lower its yield strength to match the prototype. Transverse reinforcement was not annealed. The prototype's wide tie spacing, resulting in poor concrete core confinement, was believed to be more critical than tie strength. First-story columns in the model were cast over longitudinal reinforcement that extended out of a monolithic base girder. The girder increased the base fixity of the first-story columns in the model beyond that which would occur in the prototype, where column footings would permit some base rotation to occur.

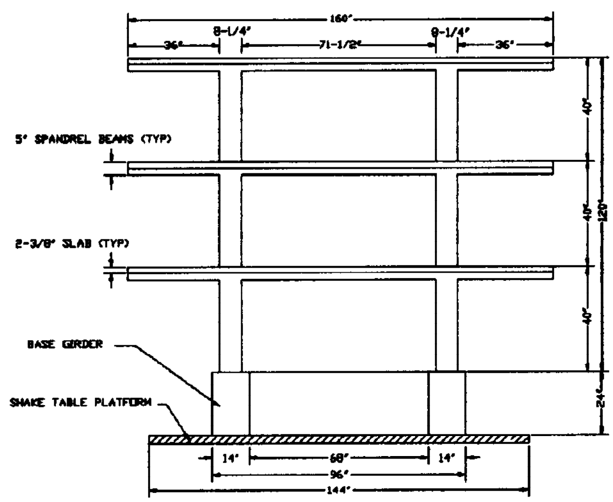

Fig. 1. East-West Elevation of Model Structure

To simulate the gravity load stresses in the model and to maintain its proper dynamic response characteristics, lead ingots were added. The ingots were placed on elastomeric pads on the floor slabs, to minimize their interaction with the slabs. Including the ingots, the floor load in the model was approximately 100 pounds per square foot (psf).

VED were added in the column lines of the test structure by using diagonal braces that contained the devices as links in the braces (Figure 2). The braces were attached to the columns by means of steel collars that were bolted in place on the columns. Researchers measured all column dimensions and fabricated the collars to provide a snug fit when they were bolted in place. As the collars were installed, a thin sand-cement mortar grout was troweled on the exposed column dimensions. The collars both transferred the forces that were transmitted by the damper braces, and confined the column concrete, increasing the shear and rotational capacities of the columns. To ensure that the collars did not slip along the column height, an all-thread bolt was placed through each column in the collar region, providing anchorage. Because of concern for damaging the model columns, the bolt was cast into the column during construction.

Pretest structural analyses indicated that the damper braces would significantly increase shear at the bases

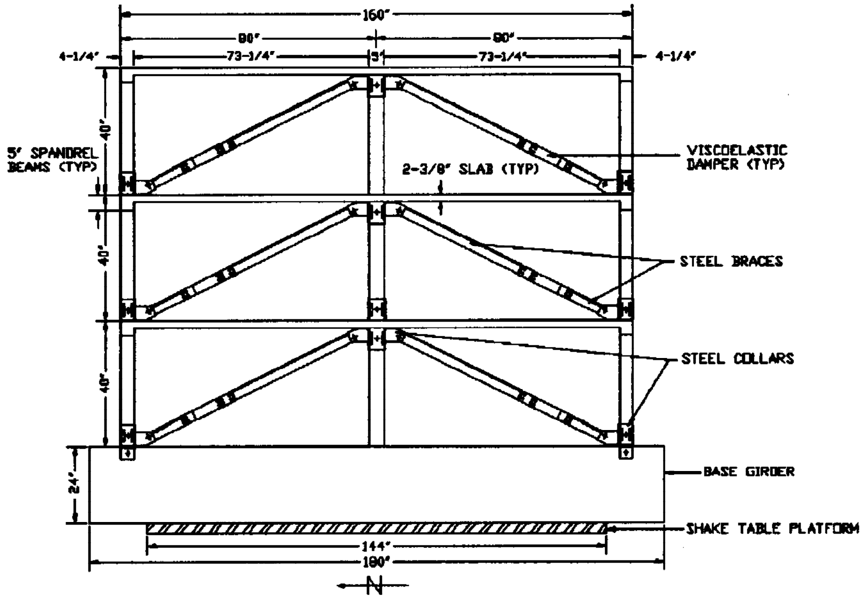

Fig. 2. North-South Elevation of Model Structure

of the first and second floor exterior columns. To minimize the likelihood of column shear failure, shear transfer capacity was increased at both levels. At the base girder, the collars were connected to the girder by means of drilling into the girder on each side of the column and epoxying an anchor bolt that was in turn attached to the collar. On the first floor level, researchers drilled and epoxied into the spandrel beam in a similar manner. The only other significant strengthening measure was to increase column flexural strength at the connection to the base girder by using No. 3 bars of Grade 60 steel, instead of the annealed wire reinforcement. Increasing the flexural capacities of these columns was assumed to be likely in any rehabilitation scheme.

EXPERIMENTAL PROGRAM

Following construction, the model was placed on the shaking table for earthquake simulation. Approximately 75 instrumentation channels recorded longitudinal and lateral accelerations and displacements of the shaking table, base girder, and each floor; damper displacements and forces; damper temperature change (in selected dampers); and reinforcement strains in selected column and slab locations.

Earthquake simulation was conducted using two different sets of VED that were designed and supplied by 3M Corporation. One set of VED was optimized to produce a total equivalent viscous damping ratio of approximately 20% in the structure, using an approach previously developed for steel structures. The second set of VED had half the volume of polymer material of the first. Representative samples of the VED were also tested in sinusoidal tests.

Before earthquake simulations, researchers determined the natural frequencies and damping properties of the undamaged structure in four different configurations: with no VED or braces installed (Figure 3), with solid steel braces installed, and with either large (Figure 4) or small VED installed. Methods of determining dynamic characteristics included using a simple "pullback" test; using the shaking table to subject the structure to white noise base motions (by examining acceleration-based transfer functions between floors of the model, the first three modes of response and the associated amounts of equivalent viscous damping were analyzed); and, after a white noise test, using the table to excite the structure sinusoidally at its measured fundamental frequency, followed by table shutdown and logarithmic decrement calculations. These procedures were used after each major subsequent earthquake simulation. After some trials with the pullback tests, researchers abandoned them; when VED were installed, the damping was so high that the structure would not oscillate.

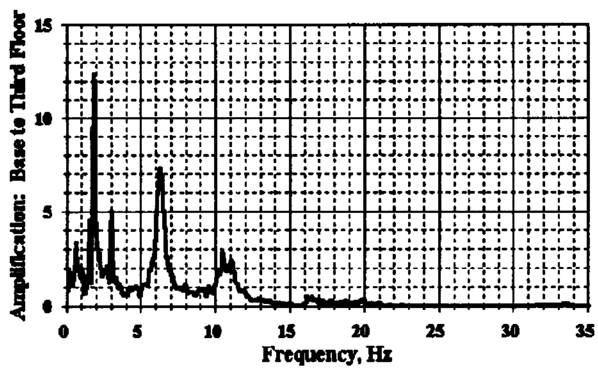

Fig. 3. Acceleration Transfer Function-No Dampers

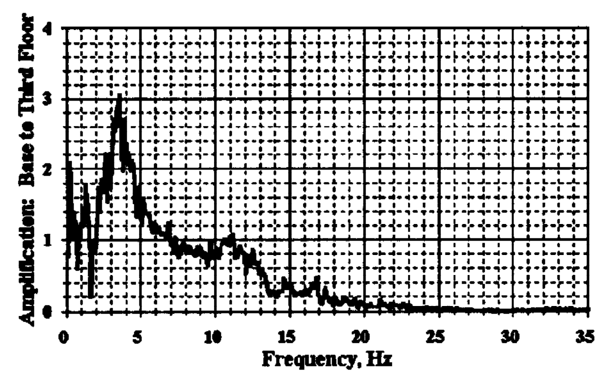

Fig. 4. Acceleration Transfer Function-Dampers

For the earthquake simulations with VED installed, which were conducted before any earthquake tests without VED were conducted, two characteristic earthquake records were used. The first was the El Centro site record from the May 18, 1940, Imperial Valley, CA, earthquake. The second was the Taft site record from the July 21, 1952, Kern County, CA, earthquake. Time scales for both records were compressed by a factor of $1/\sqrt{3}$. The peak motion amplitudes and frequency contents of the two records differ. Using the ratio of the spectrum intensities (Housner, 1959) of the two compressed records, Taft acceleration amplitudes were multiplied by 2.1 to equate the Taft record energy content with that of the El Centro record.

Researchers conducted a series of low level earthquake simulations with the small VED installed. The simulations were run with acceleration amplitudes scaled to 1%, 10%, and 25% of the El Centro acceleration record; and at 2.1%, 21%, and 52.5% of the Taft acceleration record. The structure was then tested using the same records with large VED installed. After each earthquake simulation, the modal characteristics were determined, and the model was checked visually for cracking or other deterioration.

At the conclusion of the low-level tests, little damage had occurred in the model. Further simulations were then conducted using the large VED only. The simulations were run at shaking table acceleration amplitudes scaled to 10%, 25%, 50%, 75%, 100%, 125%, 150%, 200%, and 250% of the El Centro acceleration record; and at acceleration amplitudes of 21%, 52.5%, 105%, 157.5%, 210%, 262.5%, 315%,and 420% of the Taft acceleration record. Because the higher level El Centro simulations included ground displacements exceeding the nominal horizontal motion capacity of the shaking table, the 200% and 250% El Centro records were run with high pass filters, to remove the low frequency, large displacements in the table motion.

All dampers were then removed and the braces were disconnected. The El Centro earthquake simulations were then repeated. The collars that had been used to attach the VED to the columns were left in place, so that the only test variable that was changed was the effect of the dampers and braces. The simulations repeated the 25% through 200% scaled El Centro records. Tests were concluded after the 200% El Centro simulation, when structural failure occurred.

PRELIMINARY OBSERVATIONS

Initial white noise tests of the structure without braces or VED (Figure 3) showed a fundamental frequency of approximately 1.8 hertz (Hz). With the large VED installed, the first mode frequency was approximately 3.5 Hz (Figure 4).

The model with VED appears to have sustained only minimal damage through the 150% El Centro earthquake simulation. The white noise tests following this simulation showed a fundamental frequency that was virtually unchanged from the initial conditions. Only minor additional hairline cracking was visible. During the 150% El Centro simulation, the largest interstory drift was measured as 0.45 inches (1.1%), in the second story. The total third floor drift was 1.17 inches (1.0%). The maximum base shear experienced during the test was 34.6 kips; the total model weight, neglecting the base girder, was 54.7 kips. Corresponding El Centro and Taft simulations were run sequentially, so, prior to the 150% El Centro test, the model had sustained the El Centro tests through the 100% level and the Taft tests through the 210% level.

Initial data review indicates that the model with VED began to sustain discernable damage in the 200% El Centro simulation. The white noise tests following this simulation showed a fundamental mode frequency of approximately 2.88 Hz, showing some reduction in stiffness. Visual checks of the model showed additional hairline cracking had occurred. Most of this cracking was confined to flexural cracking in the floor slabs and minor torsional cracking of the spandrel beams at their interfaces with the columns. During the 200% El Centro earthquake test, the largest interstory drift was measured as 0.88 inches (2.2%), in the second story. The total drift was 2.04 inches (1.7%). The maximum base shear was 44.2 kips.

For the earthquake tests without VED, only the El Centro record was used. An initial white noise test showed a first mode frequency of approximately 1.56 Hz, versus the original 1.8 Hz. The model had softened, indicating damage to the structure had accrued during the preceding earthquake simulations. For simulations through the 75% El Centro level, white noise tests and visual observations showed little added structural deterioration. After the 100% and 150% El Centro tests, the fundamental frequency decreased to approximately 1.38 Hz. Initial examination of roof displacements during the 150% El Centro test indicates that significant damage began occurring in the model early in the test. During the 200% El Centro test, the structure failed; it was not catastrophic, but further testing was deemed to be unsafe. Failure occurred when the concrete cover over the reinforcement that extended from the third floor columns into the third floor slab spalled off, substantially softening the joint regions, and the second and third floor spandrel beams failed in torsion at their interfaces with the columns.

A brief comparison of the 150% El Centro (peak input acceleration = 0.56 g) responses with and without VED shows higher base shear forces and lower displacements with VED. In addition to lower peak displacements, the sustained displacements later during the response are markedly lower in the tests with VED. Maximum interstory drift, which occurred in the second story, with the VED installed was 0.45 inches, versus 1.21 inches without. Maximum base shear with dampers installed was 34.6 kips, versus 17.0 kips without. Interpretation of all test data is continuing. Figures 5-9 provide representative plots of the 150% El Centro response.

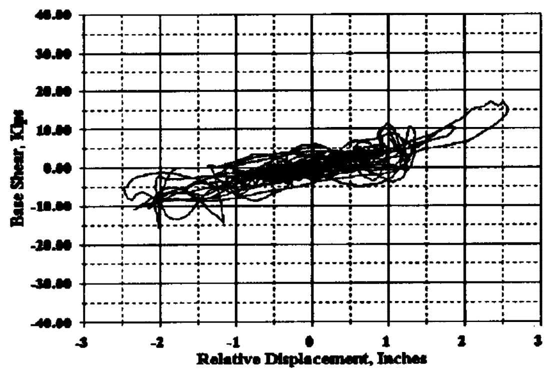

Fig. 5. Third Floor Relative Displacement Vs.. Base Shear, 150% El Centro Simulation, No Dampers

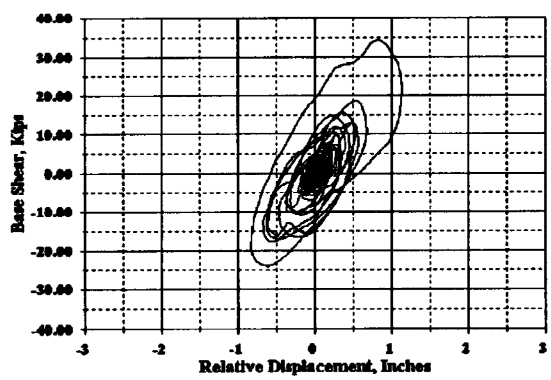

Fig. 6. Third Floor Relative Displacement Vs.. Base Shear, 150% El Centro Simulation, Dampers

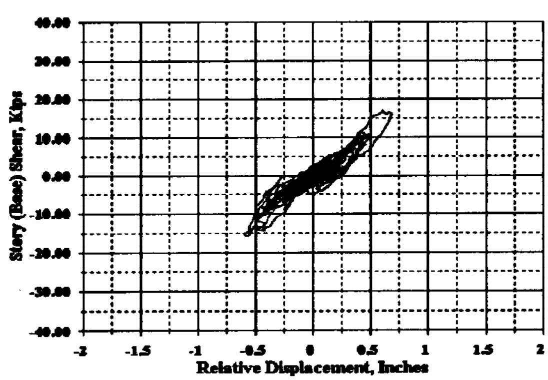

Fig. 7. First Floor Relative Displacement Vs.. Base Shear, 150% El Centro Simulation, No Dampers

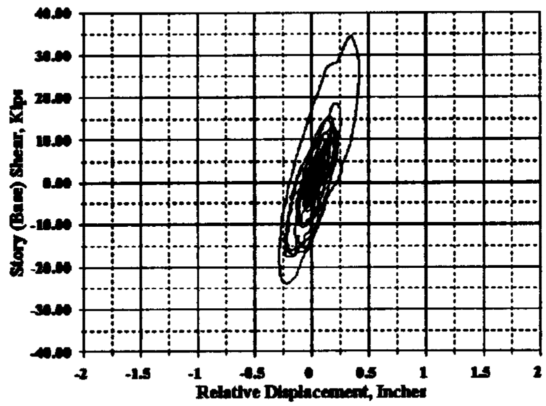

Fig. 8. First Floor Relative Displacement Vs.. Base Shear, 150% El Centro Simulation, Dampers

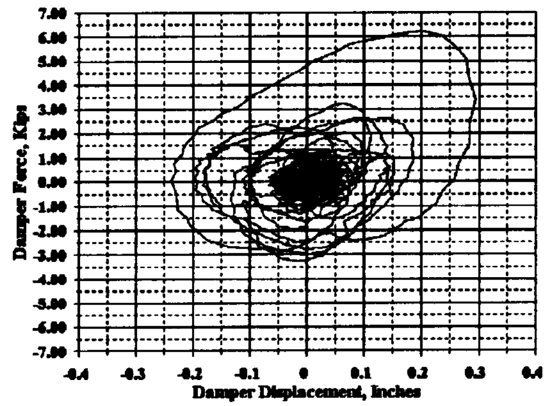

Fig. 9. First Floor NW Damper Displacement Vs.. Damper Force, 150% El Centro Simulation

ACKNOWLEDGMENTS

The authors gratefully acknowledge the support of the U.S. Army Corps of Engineers and the National Center for Earthquake Engineering Research; these organizations funded the project. In addition, 3M Corporation, Minneapolis, MN, furnished the damping devices and provided on-site assistance during the testing; and; Ivy Wire and Steel, Houston, TX, donated all wire reinforcement used in the project. UIUC graduate students Antonio Arthay, Elizabeth Dausman, Jon Sfura, and Brian Umbright provided assistance with model construction, testing, and data analysis.

REFERENCES

Moehle, JP, and Diebold, JW, "Experimental Study of the Seismic Response of a Two-Story Flat-Plate Structure," Earthquake Engineering Research Center, Report UCB/EERC-84/08, August 1984.

Housner, GW, "Behavior of Structures During Earthquakes," Journal of the Engineering Mechanics Division, ASCE, Vol. 85, No. EM4, October 1959, pp 109-129.