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ROCKING CONFINED MASONRY WALLS WITH HYSTERETIC ENERGY DISSIPATORS AND SHAKE-TABLE VALIDATION

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SUMMARY

Rocking has been found to limit seismically induced inertial forces in structures while essentially eliminating structural damage and residual displacements. Rocking in structural wall systems can be achieved through appropriate detailing of connections. Rocking systems are material independent and can be applied to different technologies. For example, the system described in this paper is applied to confined-masonry walls (CM) for use chiefly in seismically prone countries of limited technology. CM walls are jointed at the base and are allowed to rock there.

The dynamic response of rocking wall systems can be effectively controlled with supplemental damping. In the proposed system mild steel energy dissipation devices connect the rocking walls and the foundation. These devices are designed to yield in flexure during rocking of the wall and to prevent any sliding at the wall-foundation interface.

A proof-of-concept rocking wall system was designed and tested on the shake-table of the Department of Civil Engineering at the University of Canterbury, New Zealand. The system was designed following the direct displacement-based approach proposed by Priestley (2000) and was built to 4/10 scale. The system was subjected to a variety of input ground motions, representing different levels of seismic demand. The performance of the system was excellent and highly predictable with no damage being observed anywhere in the CM wall.

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INTRODUCTION

Masonry is one of the oldest construction materials, and its use in the form of confined masonry as an aseismic system is well established within a life-safe context (Schultz, 1994). In the last few years, however, its use in earthquake prone regions is being questioned or at least severely restricted. A new seismic design paradigm with a more comprehensive definition of the seismic performance of the buildings sets the limiting criteria. In this new design paradigm, the single life-safety objective is not longer the sole objective of a seismic design. The reason behind the development of multi-objective design philosophies, namely performance-based designs, stems from the excessive economic and social impact of damage in buildings in moderate size earthquakes. Unfortunately, masonry buildings stand prominently as one of the most prone-to-damage structural systems.

The justification for the development of performance-based seismic design (PBSD) methodologies seems obvious in developed countries. In these countries the cost of damage to building non-structural systems, contents and downtime causes losses of billions of dollars per event of moderate magnitude. However, in less developed regions of the world, where economic loses do not reach such higher values in absolute terms, the social and economic impact is equally traumatic, if not worse. For example, in the last two major earthquakes that occurred in Peru, the 1996 Nazca earthquake (Bariola and Kuroiwa, 1996) and the 2001 Arequipa earthquake (Bariola, 2001) the number of casualties was minor, but the socio-economic impact was vast. Hundreds of school buildings (many of them masonry structures) were damaged and needed significant repair, and thousands of children could not attend classes for several weeks.

When a large number of CM walls form the lateral force resistance in a building, the system can provide satisfactory seismic performance because often there is enough reserve of strength that assures a nominally elastic response. Such response is not expected when the lateral force resistance is provided by few CM walls. In this case damage to the CM walls can occur due to expected inelastic response.

As part of this research, a number of alternatives were assessed for use in masonry buildings with a reduced number of walls. One such alternative was CM rocking walls. Rocking walls have desirable features and have the potential to produce a reliable aseismic system suitable for use under PBSD methodology.

SEISMIC RESPONSE OF ROCKING WALL SYSTEMS

It has been more than forty years ago that it was noticed that rocking might help structures survive significant earthquakes (Housner, 1963). Even though this experience led to the formulation of a number of non-linear elastic strategies to produce improved aseismic structures, rocking itself was not significantly developed as an aseismic system. Early examples where rocking concepts were used explicitly in seismic design are reported by Sharpe and Skinner (1983) and Cormack (1988). A renewed interest in rocking is growing with the development of PBSD methodologies. Rocking behavior is subject of a number of research programs (Mander and Cheng, 1997; Priestley et al., 1999; Rahmann and Restrepo, 2000; Kurama et al, 2000; Holden et al., 2002; Toranzo, 2002), which suggests that soon the technique would be mature enough to be more widely applied.

The most important features of the rocking process can be observed in the static lateral loading and unloading of a rocking wall. This process is depicted in Fig. 1. As it will be discussed next, the process features positive characteristics, from an aseismic viewpoint, but it also involves aspects that may be negative for an aseismic structure. Both are reviewed in some detail in the following paragraphs.

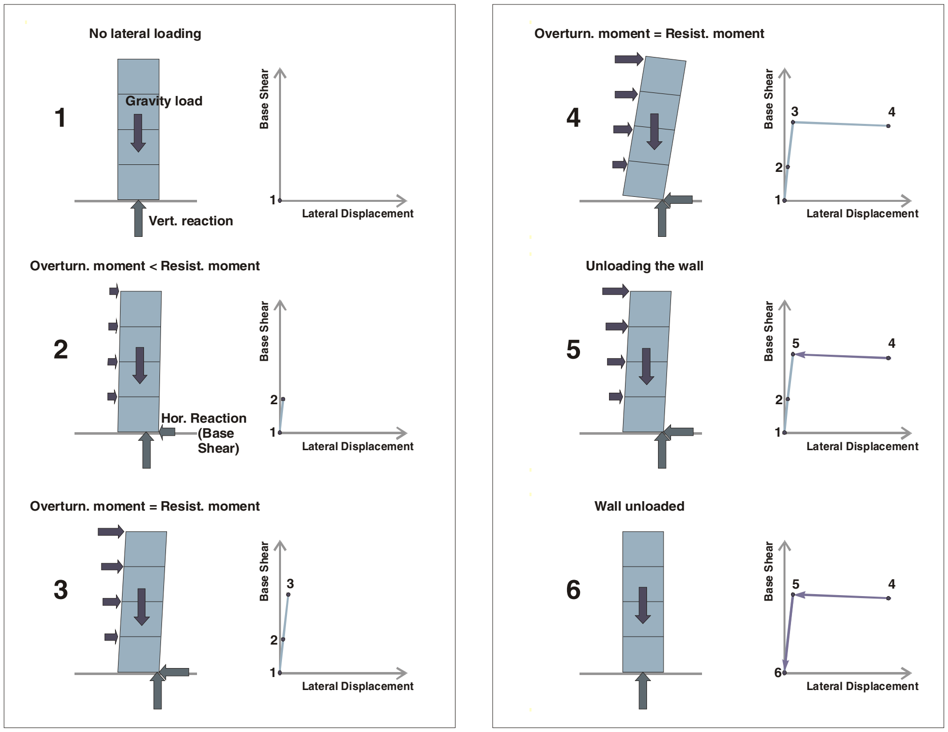


Figure 1. Static lateral loading of a rocking wall

Positive Aseismic Features of Rocking Wall Systems

- After rocking occurs, the base shear is almost independent from the applied displacement. In the static context depicted in Fig. 1, the base shear can also be reliably predicted and closely controlled with the manipulation of the wall's geometry and the vertical forces that the wall transfers. In a dynamic context, however, one must also account for impact actions and the effect of high modes of oscillation.
- The rocking wall can sustain large lateral displacements without damage. After rocking is triggered the lateral displacement of the wall occurs as a result of rigid body rotation.
- If overturning is prevented, the intrinsic re-centering mechanism of the system results in the lack of residual displacements over all the range of displacement demands. Re-centering, however, can be compromised by the formation of plastic hinges in the framing members of the rocking walls. This requires careful detailing of the system.
- Quasi-linear pattern of lateral displacement. The benefit is twofold. First, the deformation is almost evenly distributed along the height of the structure and not concentrated in a particular level (usually ground level). Secondly, the modelling of the structure, as a SDOF system, is more reliable as the displacement profile is more certain.
- Dual (stiff/flexible) behaviour of the system. Large lateral displacements are only expected passed point 3 in Fig. 1. Before this point, the wall will respond with a large stiffness. There is the possibility, therefore, to take advantage of the high initial stiffness of the wall to face the seismic demands at the serviceability level. Rocking, hence, may be triggered at a greater seismic demand, where large displacement capacity can be ensured without structural damage.

Negative Aseismic Features of a Rocking Wall

- Low energy dissipation capacity of the system. The main mechanism of dissipation of energy is through radiation of energy to the ground through impact. For practical dimensions of the walls, this only amounts to about 4% of equivalent viscous damping (EVD) for large displacements, and to about 2% for small displacements (Toranzo, 2002).
- Impact actions can be large. The impact of the wall against the foundation may cause large impact forces to the wall. This can crush the concrete in the impacting region. Impact also induces high vertical and horizontal floor accelerations to the rest of the structure, which can cause damage to non-structural components and building contents.
- Seismic response is difficult to predict. Large variability has been observed in the response of rocking walls to seismic-type oscillations at the base (Chik-Sing et al., 1980). Evaluation of response spectra show that, again, this is mainly due to the lack of an efficient energy-dissipation mechanism of the system: the variability of spectral curves is larger in systems with low energy dissipation capacity than in systems with larger energy dissipation capacity.

OPTIMIZATION OF ROCKING WALLS AS AN ALTERNATIVE ASEISMIC SYSTEM

The list of the negative aspects of rocking wall systems shows that most of them are related to the low energy dissipation capacity. Past research has shown that the use of mild steel bars designed to yield axially, connecting the rocking wall with the foundation, can provide a significant source of hysteretic energy dissipation capacity (Rahmann and Restrepo, 2000; Holden et al., 2002). From the lateral force-displacement pattern observed in quasi-static tests, Rahmann and Restrepo (2000) report that these hysteretic dissipators could provide EVD of up to 14%. This type of device, however, cannot be easily replaced after a design event. A new hysteretic dissipation device, designed to yield in flexure, is proposed and presented later in this paper.

Fig. 2 shows the role of hysteretic energy dissipation devices in the response of a rocking wall. Note how their incorporation in the system creates a hysteretic loop in the cycle loading of the structure, which is not observed in the case depicted in Fig. 1. The shape and total area enclosed by the hysteretic loop can be controlled manipulating the characteristics of the energy dissipators, so that a specific amount of energy can be dissipated per cycle. If such a source of energy dissipation is provided to the rocking wall, then a number of methodologies can be adapted for its seismic design. The capacity of the devices needs to be limited to less than the minimum gravity load in the wall, to ensure the re-centering capacity of the system.

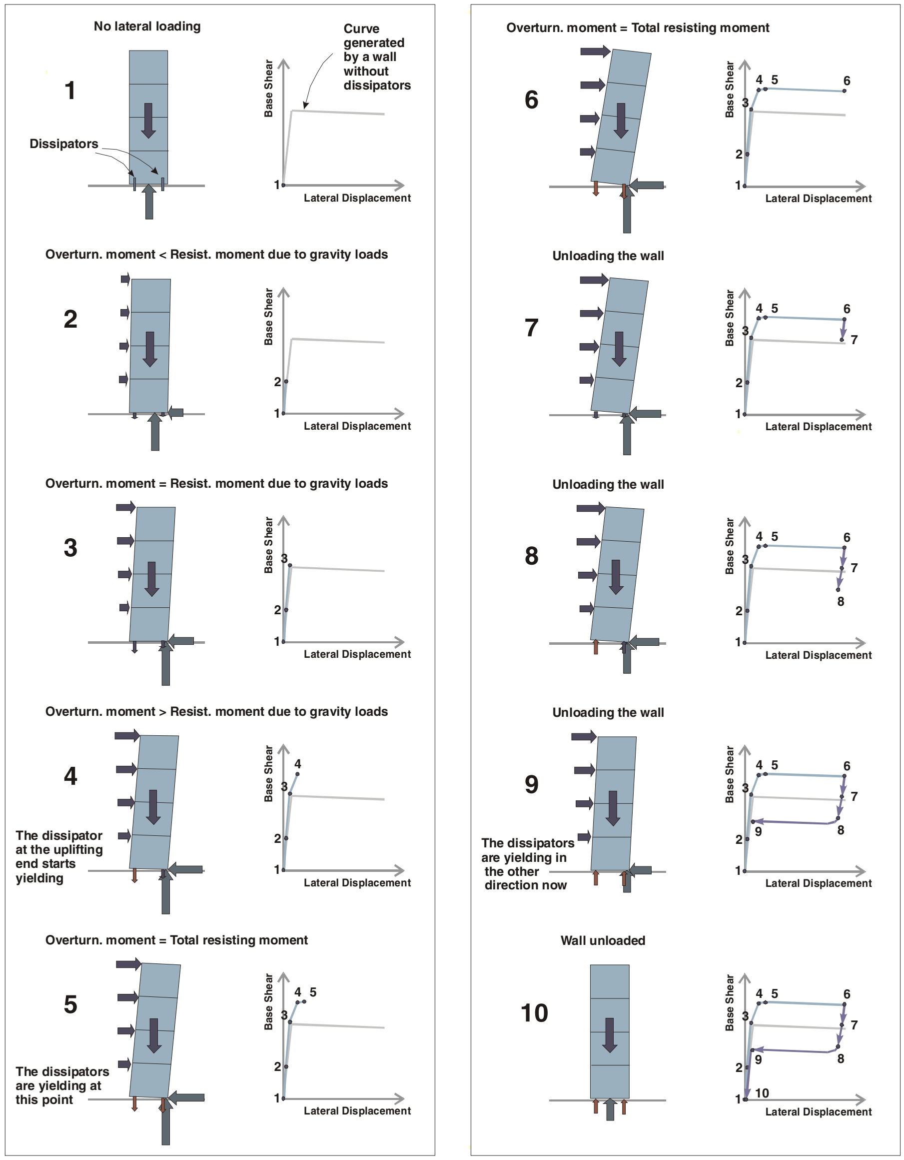


Figure 2. Static lateral loading of a rocking wall with dissipators

DISPLACEMENT-BASED SEISMIC DESIGN OF ROCKING WALLS

Toranzo (2002) proposed an iterative method for the seismic design of rocking structures adapted from the direct displacement-based design method proposed by Priestley (2000). In the adapted method, the multi-degree-of-freedom system is reduced to a single-degree-of-freedom (SDOF) substitute system. Then, a maximum lateral displacement is defined based on the permissible drift for the building. An EVD ratio, typically of the order of 15%, is assumed for the system. Then the required effective period of the system is drawn from the appropriate design spectrum (for the EVD and lateral displacement chosen). This allows calculating the effective shear of the system and then the required capacity of the dissipators. Having defined the capacity of the dissipators, the total energy dissipation capacity of the system is calculated and converted into EVD, and compared with the EVD assumed initially. The process is repeated until convergence is reached.

ADAPTATION OF CONFINED MASONRY WALLS TO ROCK AT THEIR BASE

Earlier research has shown that rocking reinforced concrete walls can be built connected to a foundation beam incorporating axial hysteretic energy dissipation devices (Rahmann and Restrepo, 2000; Holden and Restrepo, 2002). Adapting confined masonry walls to rock at their base is also possible with proper detailing. The procedure is simplified if the dissipators are attached externally as it was decided here.

Toranzo (2002) proposed the use of low-cost hysteretic dissipation devices designed to yield in flexure. The devices are connected at the wall toes and also in the foundation beam (Fig. 3). They are externally attached to the wall, so that the inspection and eventual replacement of the devices after an earthquake can take place. Sets of two devices are placed at each side of the wall. Each set act as a cantilever with one end fixed to the foundation and the other connected pinned to the wall toe. The expected mechanism is shown in Fig. 3. This arrangement will cause the dissipators to transfer a vertical load, F_d, to the end of the wall, equivalent to the one depicted in Fig. 2. Another important feature of this design is that the flexural dissipators avoid lateral sliding of the wall, without having to rely on the friction developed at the wall's base.

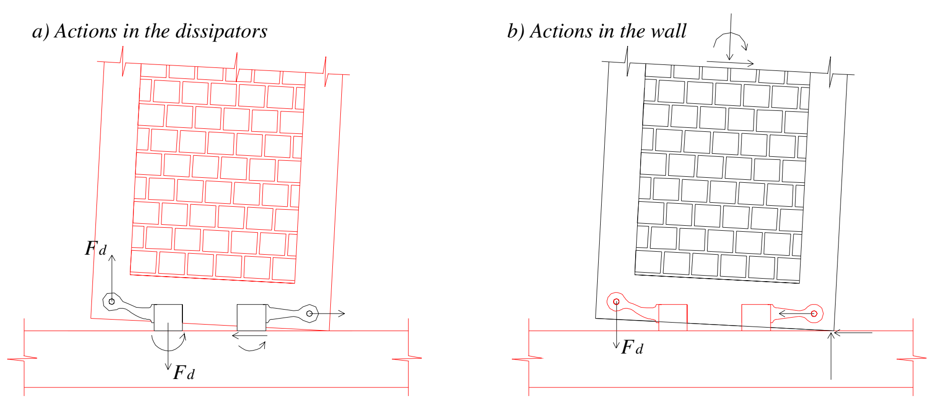


Figure 3. Actions in flexural dissipators and wall when the rocking wall uplifts

EXPERIMENTAL VALIDATION

As part of this research, a prototype building was designed according to the proposed design methodology by Toranzo (2002). The characteristics of the prototype building are shown in Fig. 4. This building does not represent any building in particular but was defined to reproduce the seismic demand on a three-storey school building in Peru. The plan of the building was chosen to ease the modelling and testing of a reduced scale model.

A 4/10 scale model of a typical frame-wall of the prototype building (hatched section in Fig. 4) was constructed and dynamically tested in the shake table of the Department of Civil Engineering of the University of Canterbury, New Zealand. To make the model dimensionally equivalent, additional mass had to be added. The time in the time-history ground-motions used for the dynamic tests also had to be modified to produce an equivalent seismic demand.

The New Zealand Standard (NZS4203, 1992) was used to define the seismic demand for the prototype building. The New Zealand Standard provides design spectra for different seismicity levels and different soil conditions. It also gives guidelines to produce design spectra for return periods other than the basic

475 years. With this information it was possible to define earthquakes whose response spectra matches the seismic demand for different return periods and therefore different levels of seismic demand.

The detailing of the reinforcement in the slabs was varied. In some slabs yield lines were allowed to develop at the column and wall faces whereas in other slabs were detailed with a deep grove to essentially hinge the slab and prevent the development of yield lines. The base of the columns was detailed as "pinned" so that only shear could be transmitted to the foundation.

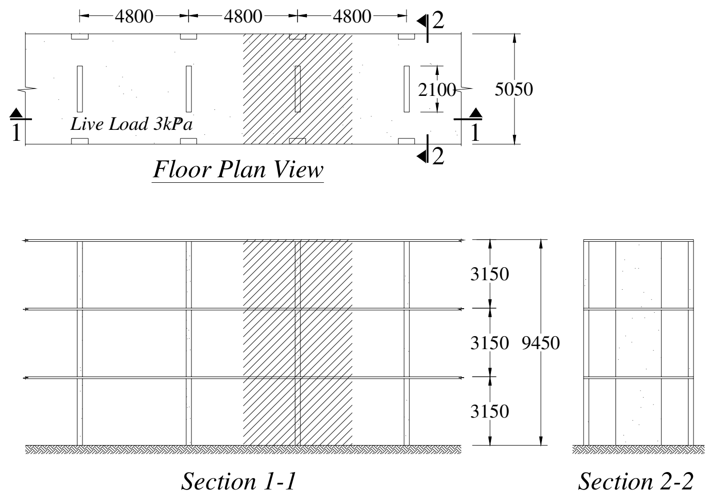
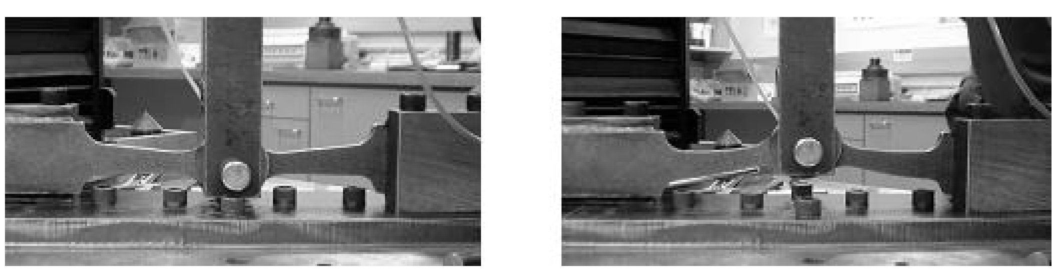


Figure 4. **Prototype building showing section (hatched) designed and modelled by Toranzo (2002).**

Three types of dissipators were designed and tested before the dynamic tests. The ideal dissipators should have been perfectly rigid-plastic. That is why in the design of the dissipators, it was tried to get the largest initial stiffness (but with a reliable response). A large initial stiffness would mean "fat" hysteretic loops and, therefore, larger energy dissipation capacity for the overall system. Another important point was to prevent the dissipators from exceeding the design force too much as it might stiffen the system and probably attract larger inertial forces. The dissipators were tested for cantilever action (Fig. 5), as they were supposed to work in the rocking wall (Fig. 2).



(a) vertical displacement = 0 mm *(b) vertical displacement = 25 mm*

Figure 5. Testing of energy dissipation devices incorporated into the rocking wall system

The results of the static cyclic test on the 3rd set of dissipators (the ones finally used in the tests) are presented in Fig. 6. The required capacity of the dissipators was 12 kN and a deformation capacity of at least 16mm. Note that beyond 16mm the dissipators provide an excessive vertical force due to their excessive curvature. Finally, the dissipators were connected to the base of the wall and the foundation as it is shown in Fig. 7.

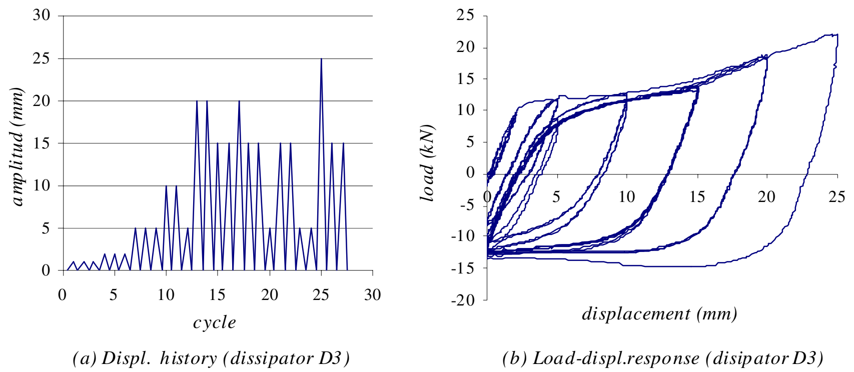


Figure 6. Cyclic testing of dissipator D3

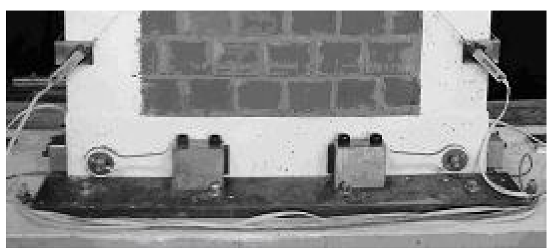


Figure 7. Connection of dissipators to foundation and rocking wall

RESULTS FROM THE DYNAMIC TESTS

The system was subjected to sixty dynamic tests, of which twenty-two were input ground motions whose response spectra matched the design spectra for four different seismic demand levels. The CM wall performed very well. No cracks developed in the masonry infill and rocking took place as expected. The results showed that the system was able to meet the required performance in terms of drift (Fig. 8). Larger than expected base shear forces, however, were found due to the development of yield lines in the slabs. Slabs build with deep grooves, intended to avoid plastic hinges in those areas, did not show any signs of plasticity.

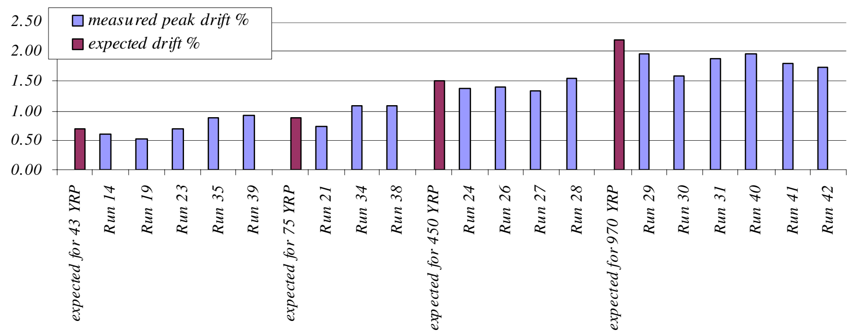


Figure 8. Observed peak drift compared with expected drift

Runs were also conducted with the rocking wall without dissipators and with new sets of dissipators. The shape of the force-displacement pattern of the responses is compared in Fig. 9 for the cases when the wall is with and without dissipators. The wall without dissipators is not capable of dissipating any significant hysteretic energy, which is the case in the wall with dissipators.

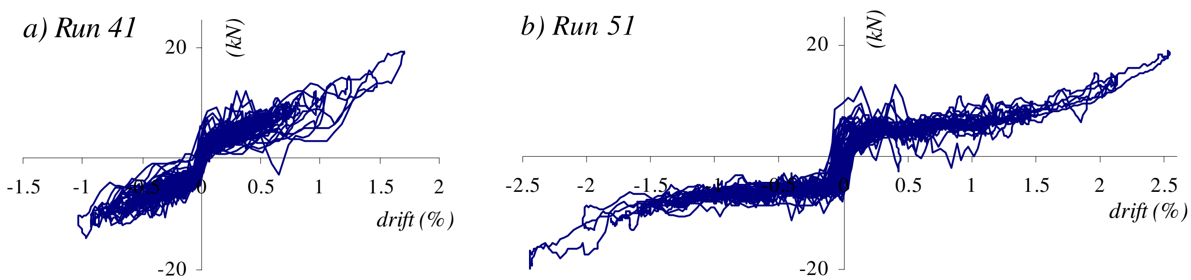


Figure 9. Comparing the force-displacement pattern observed in (a) the wall with dissipators and (b) the wall without dissipators

Levels of damping were assessed observing the amplitude's decaying shape of the free oscillation of the system. The results are presented graphically in Fig. 10. The specimen with the rocking wall with and without dissipators was set into free oscillation and its amplitude's decaying shape was observed. The EVD observed in the wall without dissipators did not exceed 4%, even for large drifts. On the other hand, the EVD observed when the wall had the dissipators reached 14% for around 2% drift. The EVD in the wall with dissipators diminished for smaller drifts. This behavior was expected, because the initial flexibility of the dissipators delays yielding in the dissipators until the wall is displaced more than about 0.5% drift; and the system, therefore, does not dissipate any significant hysteretic energy below that drift.

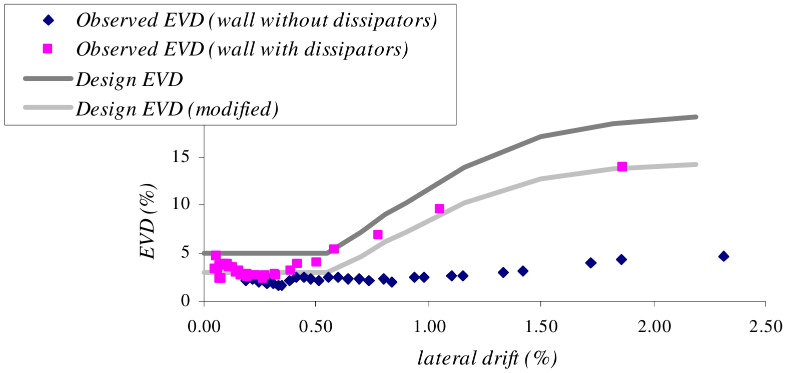


Figure 10. Equivalent viscous damping (EVD) of the system

Upon completion of the test program no damage was observed in the masonry infills and the system maintained its design strength, stiffness and energy dissipation capacity. The only observed (and expected) damage was in the yield lines of some slabs.

CONCLUSIONS

- Masonry structures are being limited in their application because of the extensive damage observed in masonry buildings in moderate to strong earthquakes.
- Rocking walls with hysteretic dissipators at the base have been proposed as an aseismic system capable to meet performance-based demands. Hysteretic dissipators at the base of a rocking wall give the structure a reliable energy dissipation mechanism that can be controlled to provide the adequate performance to the system.
- Confined masonry buildings with low density of walls can be adapted to have rocking walls with hysteretic dissipators instead of conventional fixed-base walls.
- Experimental results from a 4/10 scale model of a rocking CM wall system, showed that the system is capable of meeting prescribed design drifts. Energy dissipation capacity equivalent up to 14% of equivalent viscous damping has been observed in the system with hysteretic dissipators.
- After 60 dynamic runs, many of them equivalent to moderate and strong earthquakes, the masonry in the system specimen was undamaged and the structure maintained its design strength, stiffness and energy dissipation capacity.

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REFERENCES

1. Bariola, J. (2001). Earthquake in Arequipa, Peru June 23, 2001, EERI Special Earthquake Report, http://www.eeri.org/earthquakes/Reconn/Nazca/Nazca1.html
2. Bariola, J. and Kuroiwa, J. (1996). The Nazca, Peru, Earthquake of November 12, 1996, EERI Special Report, Jan. 1997, http://www.eeri.org/earthquakes/Reconn/Nazca/Nazca1.html

3. Chik-Sing, Y., Chopra, A., Penzien, J. 1980. Rocking response of rigid bodies to earthquakes. Earthquake Engineering and Structural Dynamics. Vol. 8, pp. 565-587.

4. Cormack, L. G. (1988). The design and construction of the major bridges on the Mangaweka rail deviation, The Institution of Professional Engineers of New Zealand, Transactions, Vol. 15, No. 1.

5. Holden, T., Restrepo, J.I. and Mander, J.B. (2002). Seismic performance of precast reinforced and prestressed concrete walls, Journal of Structural Engineering, ASCE, In Press.

6. Housner G.W. (1963). The behavior of inverted pendulum structures during earthquakes, Bulletin of the Seismological Society of America, Vol. 53, No. 2, pp. 403-417.

7. Kurama, Y., Sause, R., Pessiki, S. and Lu, L.W. (1999). Lateral load behavior and seismic design of unbounded post-tensioned precast concrete walls, ACI Structral Journal, Vol. 96, No 4, July-August 1999, pp. 622-632.

8. Mander, J. B. and Cheng, C. T. (1997). Seismic resistance of bridge piers based on damage avoidance design, Technical Report NCEER-97-0014.

9. NZS New Zealand Standards 4203 (1992). Code of practice for general structural design, and design loadings for buildings.

10. Priestley. M.J.N. (2000). Performance Based Seismic Design, Bulletin of the New Zealand Society for Earthquake Engineering. Vol. 33, No. 3, pp. 325-346.

11. Priestley, M.J.N., Sritharan, S., Conley, J.R., and Pampanin, S. (1999). Preliminary Results and Conclusions From the PRESS Five-Story Precast Concrete Test Building, PCI Journal, Vol. 44, No. 6 pp. 42-67.

12. Rahmann, A. and Restrepo, J. (2000). Earthquake resistant precast concrete buildings: seismic performance of cantilever walls using unbonded tendons, Research Report No. 2000-5, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

13. Sharpe, R. D. and Skinner, R. I. (1983). The seismic design of an industrial chimney with rocking base, Bulletin of the New Zealand Society for Earthquake Enginnering, Vol. 16, No. 2.

14. Shultz, A. E. (1994). Performance of masonry structures during extreme lateral loading events, Masonry in the Americas, American Concrete Institute, Paper SP 147-4.

15. Toranzo, L. A. (2002). The use of rocking walls in confined masonry structures: a performance-based approach. Ph.D. Thesis, University of Canterbury, Christchurch, New Zealand.