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SEISMIC REHABILITATION OF REINFORCED CONCRETE WALLS USING FIBRE COMPOSITES

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SUMMARY

Reinforced concrete structural walls are an effective system for resisting lateral loads. Under earthquake loading, the wall is expected to exhibit a ductile behaviour and dissipate energy through the formation of a flexural plastic hinge in the bottom part of the wall. Premature shear failure of structural walls designed according to pre-seismic building codes was observed in several recent earthquakes. An experimental program to investigate potential rehabilitation schemes for the shear strength and ductility of the walls is undertaken. An innovative test setup that provides the possibility of controlling the ratio of the shear force to both bending moment and axial load is used. A control wall designed according to pre-seismic building codes was tested. The wall failed prematurely in shear reproducing failures observed following several recent earthquakes. Two different rehabilitation schemes using bi-directional fibre-reinforced sheets and through anchors were designed to improve the strength and ductility of the wall. Results showed the implemented rehabilitation schemes were successful in preventing the shear failure, enhancing the wall capacity, and significantly improving the ductility of the walls.

INTRODUCTION

Structural walls are known for their effectiveness in resisting lateral earthquake loads. However, failures in structural walls were reported in several recent earthquake reconnaissance reports for example Fintel [1], Saatcioglu et al. [2], and Sezen et al. [3]. Many of the failures can be attributed to poor shear detailing or lack of confinement of the walls. Walls with those deficiencies are in need of rehabilitation in order to have the required strength and ductility to sustain the expected earthquake loads.

There are several traditional techniques for rehabilitation of walls. One of the available techniques for rehabilitation of walls is concrete jacketing by pouring new concrete to increase the thickness and adding vertical, transverse, or diagonal reinforcement. Steel jacketing using external steel plates or rods has also been proposed by [4] and [5]. Jacketing is effective in increasing the strength and stiffness of the walls, however, it is labour intensive, time consuming, and disruptive to the occupancy of the building. The additional jacket thickness may also affect the function of the building especially in elevator cores.

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Jackets often require costly foundation modifications. In addition, increasing the wall stiffness may be undesirable since it will attract higher forces.

The use of advanced composite materials in rehabilitation of concrete beams and columns has gained wide acceptance in the construction industry. However, little research has been conducted on using fibre-reinforced polymers (FRP) in the rehabilitation of walls. Lombard [6] performed rehabilitation of shear walls using carbon fibre reinforced polymers, CFRP, externally bonded to the two faces of the wall to increase its flexural strength. Using uni-directional carbon fibres with the fibres aligned in the vertical direction increased the flexural strength and stiffness of the wall. Several cases of non-ductile modes of failure occurred such as loss of anchorage or tearing of the fibres. The significant increase in stiffness would mean a significant increase in seismic loads on the wall. Paterson and Mitchell [7] used headed bars combined with carbon fibre sheet to prevent lap splice failure in structural walls with deficient lap splice details. The rehabilitation schemes also involved the use of reinforced concrete collars, which is a form of jacketing. The tested specimens had a thickness to length ratio of l/4, which could be classified as a rectangular column rather than a wall. The schemes were successful in preventing the lap splice failure and reducing the shear distress in the walls. Antoniades et al. [8] tested squat structural walls up to failure and then repaired them using high strength mortar and lap-welding of fractured reinforcement. The walls were subsequently retrofitted using FRP jackets as well as adding FRP strips to the wall edges. It was reported that the FRP increased the strength of the repaired walls by approximately 30% with respect to the traditionally repaired walls. However, the energy dissipation capacity of the original walls could not be restored completely.

The available research conducted on the rehabilitation of walls using FRP is promising but there is a need for an effective rehabilitation scheme to prevent brittle shear failure and improve the ductility of structural walls. An experimental research program is undertaken with the objective of developing and testing rehabilitation schemes to improve the shear strength and ductility of structural walls using advanced composites.

EXPERIMENTAL PROGRAM

Modeling

To reduce the size of the required specimens, instead of modeling the whole wall, only the zone of the plastic hinge of the wall will be modeled. The effect of the top part of the wall which will mainly behave in an elastic manner on the plastic hinge will be represented by the moment, shear force, and axial load applied to the hinge zone as shown in Fig. 1. In this experimental program, three walls were tested. One wall represented the control CW test and two walls RW1 and RW2 were rehabilitated and tested.

Design of Specimens

In order to model an older code design that requires rehabilitation to improve the ductility and shear strength, the wall reinforcement was designed to comply with the 1963 ACI [9] and the CSA [10] code provisions. The wall dimensions and reinforcement details are shown in Fig. 2. In order to transfer the axial load, bending moment, and shear force uniformly to the top part of the wall specimen a heavy reinforced concrete loading beam was used at the top of the wall.

Material Properties

The average concrete compressive strength at the time of testing was 38 MPa. The average yield strength for the vertical steel bars was 470 MPa and the average yield strength for the transverse reinforcement was 600 MPa. Two types of CFRP sheets were used: Tyfo BCC Composite and Tyfo SCH-35 Composite. The Tyfo BCC is a bi-directional 0.864 mm thick carbon fabric where the primary fibres are oriented in the $\pm 45^{o}$ direction. The Tyfo SCH-35 Composite is a uni-directional 0.89 mm thick carbon fabric. The tensile modulus in the direction of the fibres as was supplied by the manufacturer is 65 GPa for Tyfo BCC and 78 GPa for Tyfo SCH-35 and the ultimate tensile strength is 717 MPa for Tyfo BCC and 991 MPa for Tyfo SCH-35 [11].

Test Setup and Loading

Three hydraulic actuators were setup as illustrated in Fig. 3. The two vertical actuators, one on each side of the test specimen as shown in Fig. 4, were used to produce an axial compression and a moment while the horizontal actuator imposed a shear force and a moment.

The shear (V) to moment (M) ratio was chosen so that the ratio M/VL would remain constant. The value of M/VL was selected to be 2.25. From the seismic analysis of multi-storey reinforced concrete buildings with wall-frame interaction, this relatively low value of M/VL is shown to be realistic. The actuators were controlled so that the M/VL ratio would be held constant at 2.25. This was achieved by controlling the vertical actuators to have a constant axial compression force and an additional force, which is directly proportional to the force in the horizontal actuator. The forces in the two vertical actuators F_{V1} and F_{V2} at any time are a function of the horizontal force F_H, as given by the following relationship:

$$F_{V\,1,2} = -170 \pm 0.1\ F_H \qquad (1)$$

The load was cycled under force control till just before first yielding of the reinforcement. After yield, multiples of the yield displacement were imposed on the wall incrementally in displacement control mode until failure.

Instrumentation

The data acquisition system consisted of an analog to digital board with a maximum capacity of 72 channels, a microcomputer, and data-acquisition software. Twenty-six strain gauges were attached to the horizontal and vertical reinforcement steel of each specimen. Vertical, horizontal, and diagonal strain gauges were attached to the FRP sheets for specimens RW1 and RW2. Lateral displacements of the wall, relative rotation of the two end blocks, and shear deformation in the wall were measured using linear voltage displacement transformers (LVDTs). In total, 21 LVDTs were used in each specimen as shown in Fig. 5. The displacement of the top of the wall relative to the base was calculated by subtracting the displacement at the bottom P9 from the displacement at the top P2. The vertical displacements of LVDTs P18 and P19 were used to calculate rotation. The shear deformation (γ) was measured using the relative displacements along the diagonals of the wall from the readings of LVDTs P20 and P21.

More details about the test setup, modeling, design of specimens, and loading are available elsewhere [12].

REHABILITATION SCHEMES

Wall RW1

The control wall was deficient in shear and ductility. The rehabilitation scheme had to include both a shear strengthening scheme and a ductility improvement scheme. The shear rehabilitation involved wrapping the wall with two layers of Tyfo BCC fabrics with fibres woven at $\pm 45^o$ using the procedure recommended by the manufacturer. Each layer had an overlap length of 150 mm.

The ductility improvement scheme involved the confinement of two end column elements of the wall. The confinement of the end columns included two components: unidirectional fibres (Tyfo SCH-35) wrapped around the edge elements of the wall in the form of a U-shaped partial hoop, and FRP anchors through the wall that acted as the fourth side that closed the U-shaped hoops. The U-shaped sheet covered approximately 300 mm on both sides of the wall and consisted of three layers of unidirectional carbon fibres. The FRP anchors were each made by wrapping a 110 mm wide unidirectional Tyfo SCH-35 sheet to create a bundle of uni-axial fibres equivalent to the number of fibres in one layer of the U-shaped sheets. The anchors were soaked in the Tyfo S Epoxy and then inserted into holes drilled through the wall as seen in Fig. 6. The length of the anchor was 220 mm, which meant that they protruded 50 mm out of the wall on both sides. At both ends the fibres were splayed radially over the first layer of fabric and covered with the remaining two layers thereby providing the required anchorage length for the anchor.

Wall RW2

In this wall, the shear strengthening scheme was identical to that of wall RW1 with one small but significant difference. Two through holes were drilled at the top and two at the bottom in the web of the wall as shown in Fig. 6. Four high strength steel bolts were inserted through the holes. Circular washer plates of 60 mm diameter and 8 mm thick were used on the outside of all the fibre layers. The objective of those bolts is to improve end anchorage for the fibres and prevent delamination from starting at the top and at the bottom regions of the wall.

The ductility enhancement procedure consisted of two components similar to the first rehabilitation scheme. The first component was U-shaped FRP sheets attached to the two end elements of the wall similar to the first wall RW1. The second component is steel anchors used to close the confinement hoop and generate two confined columns on both sides of the wall. Nine holes were drilled on each side at a spacing of 110 mm as seen in Fig. 6. High strength 16 mm-diameter threaded rods were inserted through the holes. The holes were spaced so that they comply with the spacing requirements for steel confinement hoops in the CSA [13] code provisions. The anchors were made of Grade 5, ASTM A193-87 threaded rod of 16 mm nominal diameter. Circular washer plates, 60 mm in diameter and 8 mm thickness, were used. All the anchors were tightened to a torque of 250 N.m.

RESULTS

Control Wall CW

The wall only sustained three cycles of loading. The first cracks that developed were tension cracks at the bottom of the wall and at mid-height at a load of 30 kN and drift of 0.1%. At the load of 300 kN and drift of 1.59% two diagonal shear cracks, which were about 100 mm apart, were clearly developed in both directions. During the next cycle at the load of 363 kN a sudden and complete shear failure of the wall occurred. The two diagonal cracks joined to form one large diagonal crack in the wall and the wall failed as seen in Fig. 7. Crushing of concrete was observed at the toe of the wall on the compression side and the vertical steel buckled although it did not yield. The failure mode of the wall was very similar to that reported in post earthquake reconnaissance reports [1, 2, and 3].

The relation between the lateral load and drift is shown in Fig. 8. There is no significant loss of stiffness after the first cracking and before the final failure. The maximum lateral drift that the wall reached before failure was 2.5%. The horizontal steel bars yielded at failure while the outermost vertical bars were close to yield just before failure but they did not yield. The strains in the horizontal reinforcement bars were always in tension whether pushing or pulling. The maximum recorded strains were from horizontal bars at mid-height of the wall, which yielded at failure allowing the crack to open up.

Rehabilitated Wall RW1

The wall sustained six complete full loading cycles. There were two cycles under force control at a horizontal load level of ± 200 kN and ± 300 kN and no deterioration in the load resistance was observed. The yield load in both directions was approximately 400 kN and the yield displacement was about 15 mm, this is equivalent to drift of 1.4%. A horizontal flexural crack was visible on both ends at the joint between the wall and the base. The two subsequent cycles at ductility levels of 1.5 and 2 were enough to cause delamination of the FRP that extended to almost one-third the height of the wall in the mid section of the wall. No debonding or delamination of the CFRP was observed in the confined end elements. The horizontal load sustained by the wall increased due to strain hardening of the longitudinal steel and reached 515 kN at the displacement ductility level of 2.

In the cycle at a displacement ductility level of 3, the diagonal fibres completely debonded in all the mid-section of the wall during the push half of the cycle. The concrete at the two end zones near the bottom was crushed. However, the wall sustained the maximum load of 515 kN due to the confining effect of the uni-directional fibres and the anchors of the end elements of the wall. Failure of the confining fibres at the bottom of the wall was triggered by failure of the bottom FRP anchors. The load started to drop slowly with increasing the displacement due to concrete crushing because the confinement of the concrete at the bottom was deteriorating. A final cycle that aimed at reaching a displacement ductility of 4 was not completed in the push direction because there was extensive out of plane movement at the base of the wall due to buckling of steel after the concrete crushed as shown in Fig. 9.

The hysteretic loops of the horizontal load at the top with drift are plotted in Fig. 10. The loops indicate that the wall maintained almost all of its maximum strength up to a displacement ductility of 3, which corresponds to lateral drift of approximately 3%.

Readings from strain gauges on steel bars show that the vertical bars sustained large strains while the transverse reinforcement did not yield. The outermost vertical bar yielded at the bottom of the wall at a horizontal load of approximately 400 kN, the strain at failure reached almost 1.3%. The bars at the top section of the wall reached yield at a horizontal load of 485 kN. Strains in the horizontal bars were almost half of those recorded in the case of the control wall.

The strain gauges attached to the FRP sheets recorded high readings indicating that the sheets were fully utilized. The readings measured by two diagonal strain gauges at mid-height of the wall were as high as 0.004 mm/mm at maximum load. The strains in the horizontal fibres, providing confinement to the concrete, were as high as 0.0025 mm/mm before failure.

Rehabilitated Wall RW2

The wall sustained nine cycles of loading. Yield was observed at a load of 430 kN and the yield displacement was 15 mm which represents a drift of 1.4%. At ductility level of 2, debonding of the fibres started at the top and the bottom ends of the wall but was controlled by the steel anchors. For the cycle at 3 times the yield displacement, the load reached 571 kN in the pull direction. At the end of the seventh cycle at 4 times the yield displacement, the load dropped significantly to 250 kN. This was attributed to yield of one of the confining steel anchors and failure of the confining fibres at the bottom of one of the end elements. In the eighth cycle at a ductility level of 5, the wall was shortened by almost 100 mm due to crushing of the concrete at the base. The last cycle at ductility level 6, was not completed because of out of plane movement at the bottom of the wall due to steel buckling.

The hysteretic loops of the horizontal load with top drift of the wall are plotted in Fig. 11. The wall maintained almost full maximum strength up to a displacement ductility of 4, which corresponded to a lateral drift of about 5%. The two subsequent cycles at a level of ductility of 5 and 6 showed deterioration in the strength to 225 kN with noticeable pinching and reduction in stiffness, which resulted in less energy dissipation. The vertical bars yielded at a horizontal load of approximately 430 kN, and the strain at failure reached almost 2%. The strain readings at the top of the wall show that the longitudinal bars at the top reached strains as high as 0.8% at a horizontal load of approximately 470 kN. Strains in the transverse steel bars were almost half of those recorded in the case of the control wall at the same level of load.

Readings from the strain gauges installed on the bolts at 110 mm above the base of the wall indicated that those bolts yielded at a horizontal load of 490 kN. The bolts of the second row at 220 mm above the base did not yield. The strain gauges attached to the FRP sheets recorded high readings that exceed the expected values based on simple design equations from the code. The readings in the two diagonal strain gauges at mid-height of the wall were as high as 0.0045 mm/mm at ultimate load. In the last two cycles after the wall passed its capacity and was failing, the strains in the diagonal fibres were as high as 0.008 mm/mm. The horizontal fibres at the bottom of the wall were subjected to high tensile strains that reached 0.0065 mm/mm at maximum load just before onset of failure.

ANALYSIS

Analysis of the behaviour of the walls was performed using Response 2000 program [14]. The program uses the Modified Compression Field Theory developed by Vecchio and Collins [15] to model the behaviour of prismatic reinforced concrete members. The properties of the materials were input based on the results of the tests. A pushover analysis was performed using the same moment to shear ratio and the same axial load that was used in the test. For the rehabilitated wall, the FRP was modeled as transverse reinforcement with equivalent stiffness and strength to the FRP. The confining effect of the concrete could not be modeled directly by the program. Therefore, the difference in behaviour between the two rehabilitated specimens can not be directly modeled by the program.

The load deformation results for the control and the rehabilitated walls were added to the experimental lateral load drift plots in figures 8,10, and 11. The estimate for the ultimate load was 383 kN for the control wall which failed prematurely in shear. The ultimate load sustained by the rehabilitated walls was estimated at 546 kN by the program. Both estimates are within 5% of the recorded test values. The estimates for displacement based on no rotation at the base were much smaller than the measured values. This can be attributed to the rotation at the base due to the yield penetration depth into the foundation of the wall. Taking this into consideration, reasonable estimate of the deflection was obtained.

Analysis of the concrete section suggests that the strain in the concrete in compression for the control wall at failure was less than 0.15%. For the rehabilitated walls, the moments sustained by the walls correspond to strains as high as 1%. This shows the effect of confinement using FRP in improving the behaviour of concrete in compression.

The curvature along the height of the wall was calculated for the control wall and for the rehabilitated wall and is shown in Fig. 12. The program predicted correctly that for the original wall the curvature is distributed almost evenly along the height. Whereas for the rehabilitated wall the curvature was concentrated at the base of the wall which indicates the development of a plastic hinge at the base of the wall.

DISCUSSION

The envelopes of the horizontal load with lateral drift in all cycles for the three specimens are shown in Fig. 13. The rehabilitated walls sustained on average 50% more load and 60% more lateral drift than the control wall. The improvement in displacement ductility was also significant. The control wall failed before even reaching the yield displacement while the two rehabilitated walls achieved displacement ductilities of 3 and 4 before the failure started. The second rehabilitated wall RW2 had a residual strength of about half the maximum capacity even after reaching a displacement ductility of 6. The test was ended when the actuator ran out of stroke.

The energy dissipated in each cycle was calculated and plotted against lateral drift for the three specimens in Fig. 14. The energy dissipated by the first rehabilitated wall is clearly more than that dissipated by the control wall due to the additional strength and ductility. The energy dissipated by the second rehabilitated wall is significantly higher than the energy dissipated by the first rehabilitated wall because the wall had substantial residual strength even after failure, which helped to dissipate more energy.

The maximum strain in longitudinal steel at failure was measured at 0.0027 mm/mm for the control wall while it reached 0.015 mm/mm for the first rehabilitated wall RW1 and 0.02 mm/mm for the second rehabilitated wall RW2. These high strain measurements indicate that unlike the original wall, the longitudinal steel was fully utilized for the case of the rehabilitated walls. The results from the section analysis also indicate that the concrete section was also fully utilized with the strain in concrete for the rehabilitated walls reaching almost seven times its value for the control wall.

The strain in the horizontal bars is much lower in the rehabilitated wall than in the original wall. This is attributed to the contribution of the fibres in arresting the diagonal shear cracks. The recorded strain in the diagonal fibres was close to 0.004 mm/mm at maximum load for both specimens RW1 and RW2, which is the maximum allowable fibre strain according to the code.

The strain in the horizontal fibres was 0.0025 mm/mm at failure of the rehabilitated wall RW1, which means that the capacity of the fibres was not fully utilized because the FRP anchors failed. For the second rehabilitated wall RW2, the strain reached 0.0065 mm/mm at failure of the wall, which indicates that the fibres were successfully stressed past the allowable design value of 0.004 and that the fibres were fully utilized. This superior behaviour for the second wall could be attributed to the fact the steel anchors did not fail in shear, as was the case for the FRP anchors. Some of the steel anchors were stressed to yield.

CONCLUSIONS

An innovative test setup enabled the laboratory simulation of observed failures in walls and testing of the plastic hinge region of the wall. The proposed rehabilitation schemes for structural walls under lateral load using CFRP sheets and carbon or steel anchors proved to be effective in enhancing the shear strength and ductility for structural walls. The shear mode of failure was eliminated because the carbon fibres aligned at 45 degrees were effective in arresting the diagonal shear cracks. The increased confinement of the end elements delayed the concrete crushing in compression up to very high compression strains and allowed the full utilization of the longitudinal steel in tension due to the increased strength of concrete.

The steel anchors through the wall provided better confinement than the FRP anchors because the FRP anchors failed in shear while the steel anchors sustained a higher load before yielding in tension. Having additional steel anchors in the web of the wall at the top and bottom regions proved effective in improving the end anchorage for the FRP sheets in the mid-wall zone.

Analysis of the walls was successful in estimating the strength of the control and rehabilitated walls within a 5% range. The mode of failure was also successfully predicted by the analysis. The experimental results and analysis showed that the rehabilitation schemes were effective in eliminating the premature shear failure mode and improving the ductility of the wall. Both the experimental results and the analysis showed that a plastic hinge was developed at the base of the wall for the rehabilitated specimens which is effective in dissipation of energy.

REFERENCES

1. Fintel, M. “Performance of buildings with shear walls in earthquakes of the last thirty years.” PCI Journal 1995; 40(3): 62-80.
2. Saatcioglu, M., Gardner, N.J., and Ghobarah, A. “1999 Turkey Earthquake, Performance of RC Structures”, Concrete International 2001;23(3): 47-56.

3. Sezen, H., Whittaker, A.S., Elwood, K.J., Mosalam, K.M. “Performance of reinforced concrete buildings during the August 17, 1999 Kocaeli, Turkey earthquake, and seismic design and construction practice in Turkey”, Engineering Structures 2003; 25:103-114.
4. Elnashai, A.S. and Pinho, R. “Repair and strengthening of RC Walls Using Selective Techniques”, Report No. 97-1 Engineering Seismology and Earthquake Engineering, Civil engineering Department, Imperial College, London, UK, 1997.
5. Taghdi, M. "Retrofitting Low-Rise Shear Walls with Steel Plates”, Ph.D. Thesis, University of Ottawa, Ottawa, Canada, 1998.
6. Lombard J. C. “Seismic Strengthening and Repair of Reinforced Concrete Shear Walls Using Externally Bonded Carbon Fiber Tow Sheets”, Master of Engineering Thesis, Carleton University, Ottawa, Canada, 1999.
7. Paterson, J. and Mitchell, D. “Seismic Retrofit of Shear Walls with Headed Bars and Carbon Fiber Wrap”. Journal of Struct. Eng., ASCE 2003; 129(5): 606-614.
8. Antoniades, K., Salonikios, T., and Kappos, A. “Cyclic Tests on Seismically Damaged Reinforced Concrete Walls Strengthened Using Fiber-Reinforced Polymer Reinforcement”, ACI Structural Journal 2003; 100(4): 510-518.
9. ACI Committee 318, “Building Code Requirements for Structural Concrete”, American Concrete Institute, Detroit, Michigan 1963.
10. CSA, “Code for the Design of Concrete Structures for Buildings”, CAN3-A23.3-M77, Canadian Standards Association, Rexdale, Ontario, Canada, 1977.
11. Fyfe (2002). Product technical specification, http://www.fyfeco.com/data_sheet.htm (August 2002).
12. Khalil, A. and Ghobarah, A. “Scale Model Testing of Structural Walls”, Proceedings of the Response of Structures to Extreme Loading Conference, Toronto, Canada, Paper No. O93, Elsevier Science, Oxford, UK, 2003.
13. CSA, “Design of Concrete Structures”, Standard A23.3-94, Canadian Standards Association, Rexdale, Ontario, Canada, 1994.
14. Bentz, E.C., “Sectional Analysis of Reinforced Concrete”, Ph.D. Thesis, Department of Civil Engineering, University of Toronto, 2000.
15. Vecchio, F.J. and Collins, M.P., “The Modified Compression Field Theory for Reinforced Concrete Elements Subjected to Shear”, ACI Journal 1986, 83(2): 219-231.

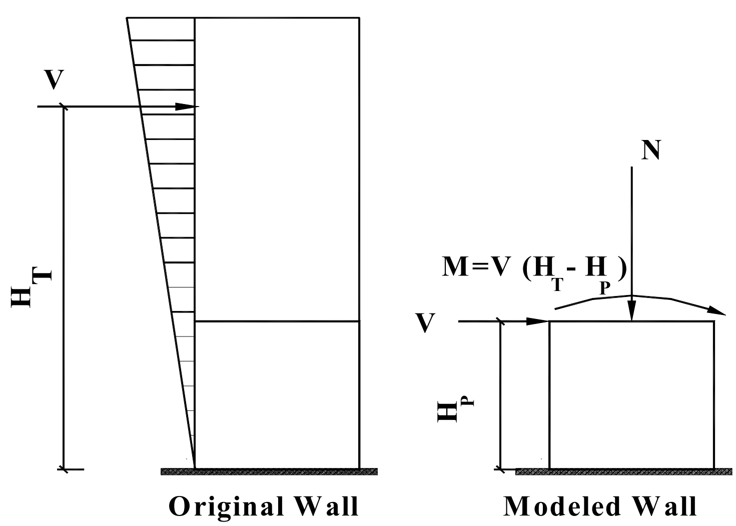

Fig. 1 Forces and moments on the tested part of the wall

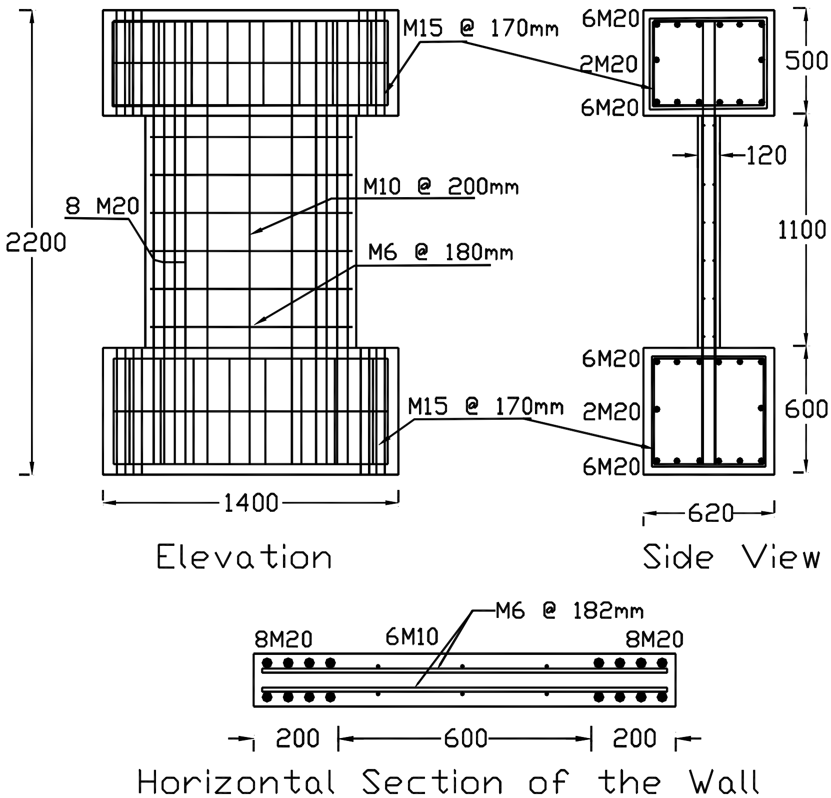

Fig. 2 Specimen dimensions and reinforcement details

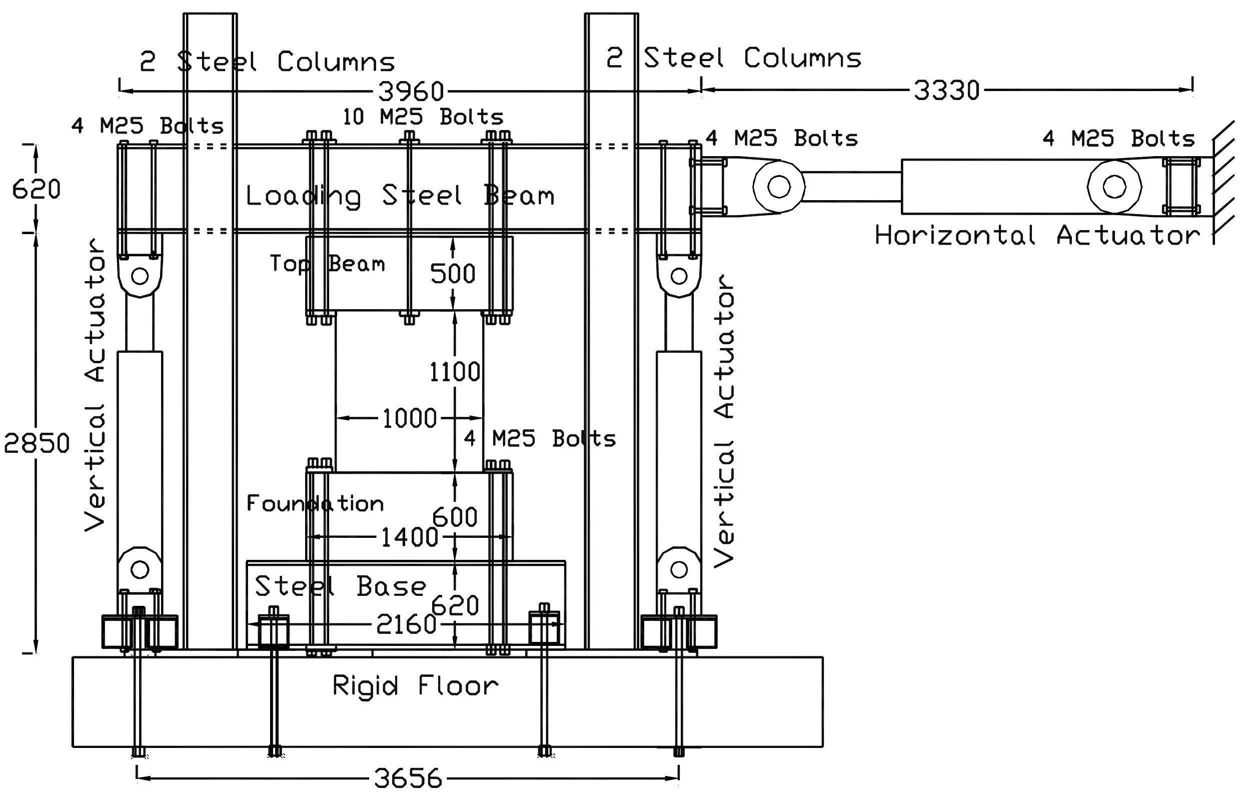

Fig. 3 Schematic of test setup

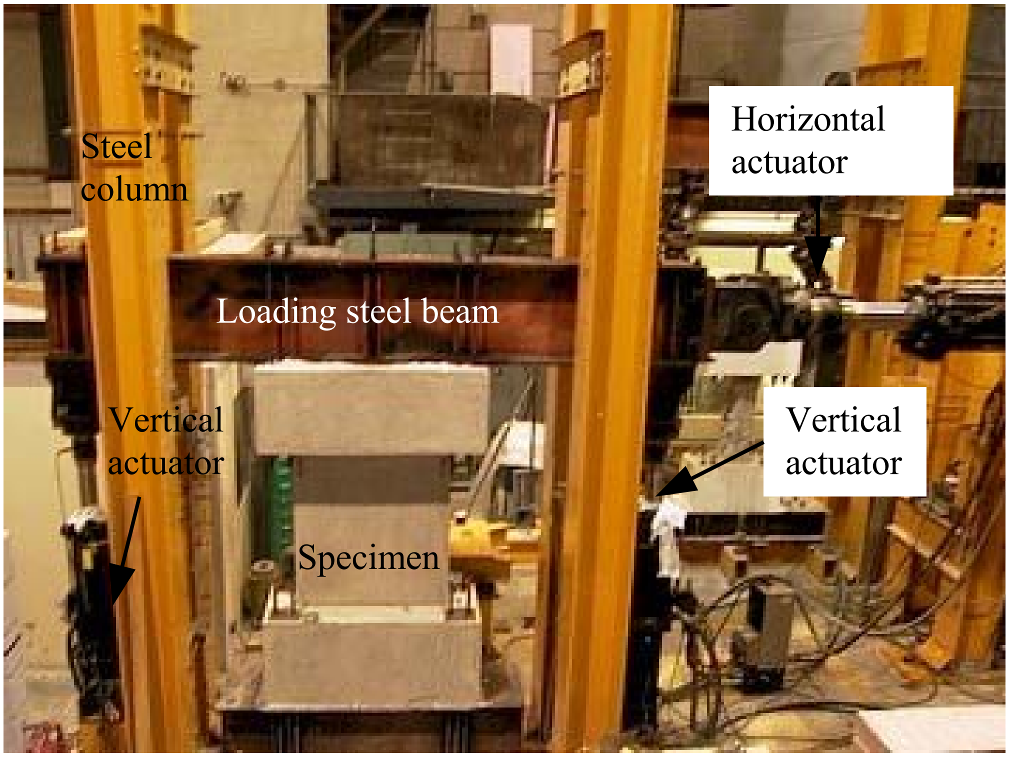

Fig. 4 Test setup

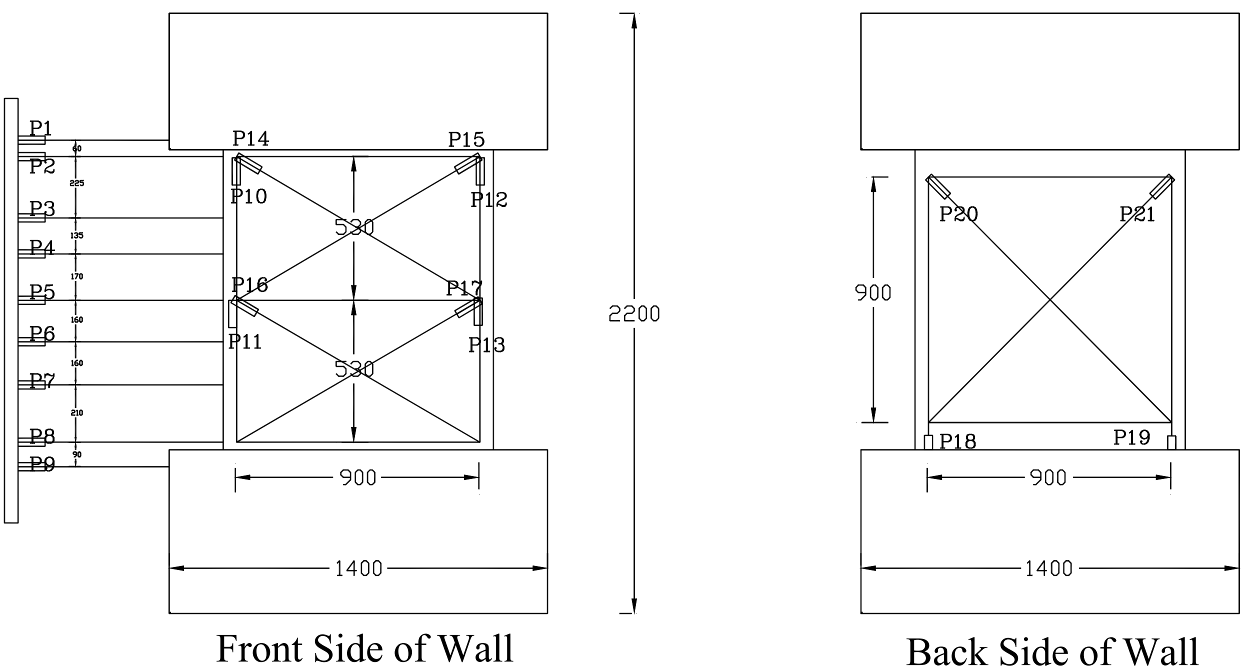

Fig. 5 LVDT locations

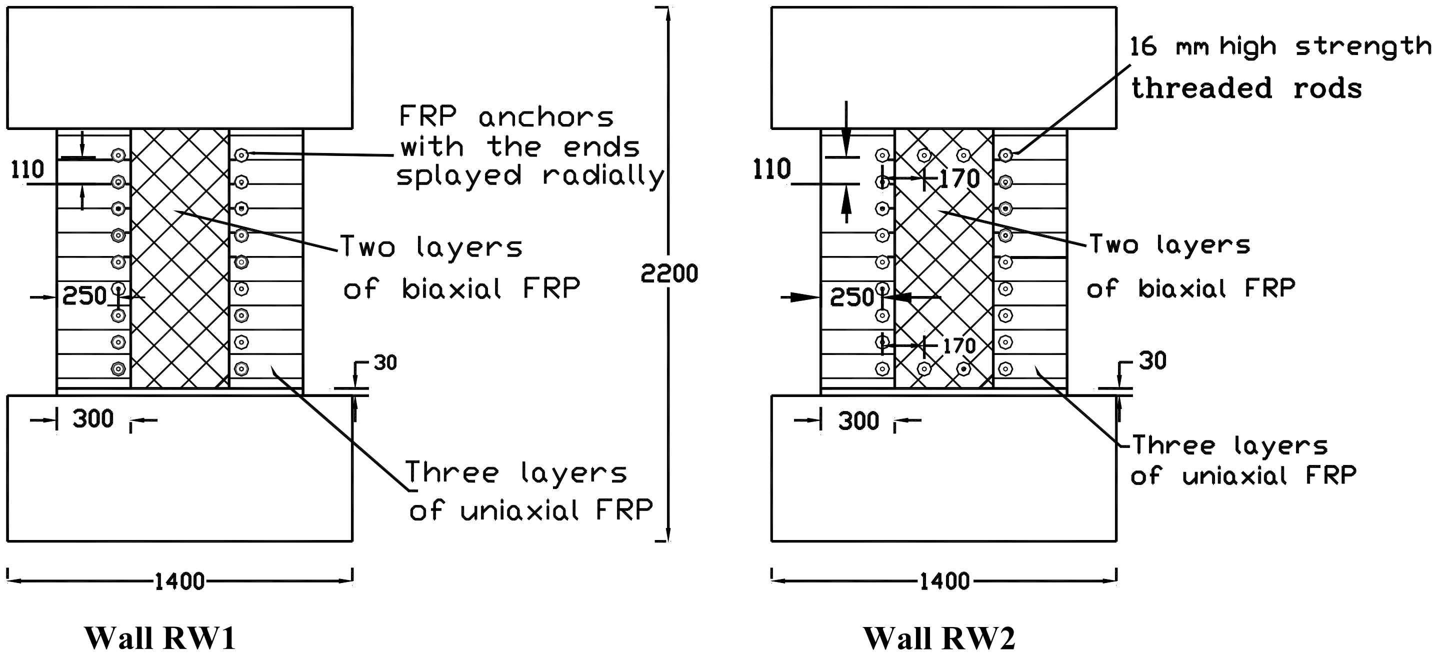

Fig. 6 Rehabilitation Schemes for RW1 and RW2

Fig. 7 Failure mode for the control wall CW

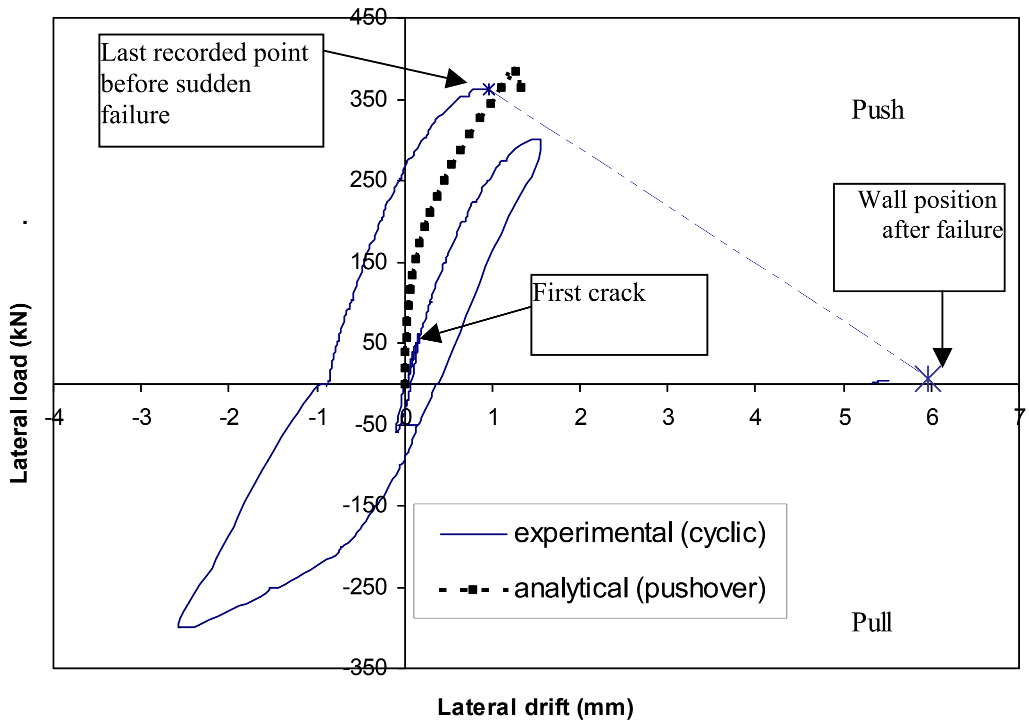

Fig. 8 Top drift of the control wall CW

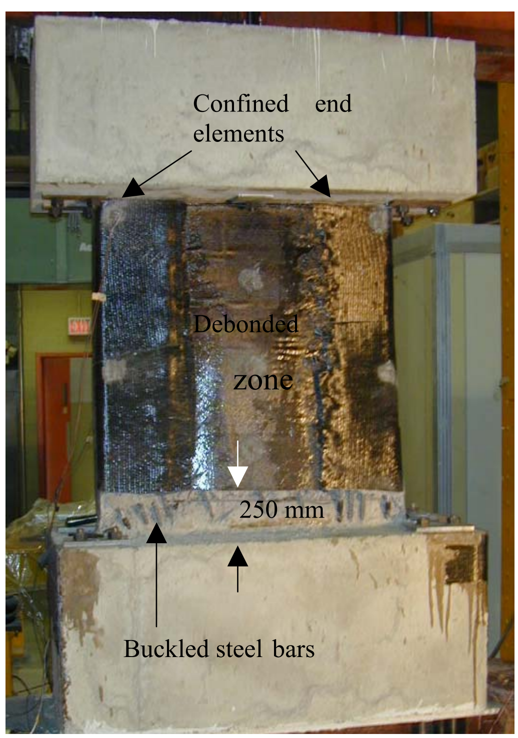

Fig. 9 Failure mode for the rehabilitated wall RW1

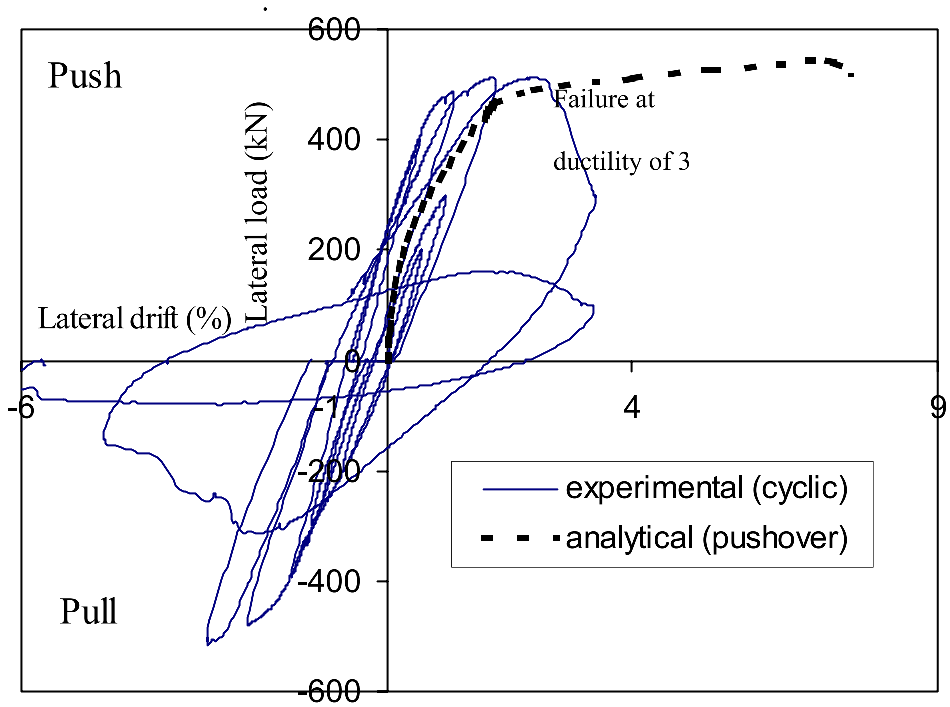

Fig. 10 Top drift of wall RW1

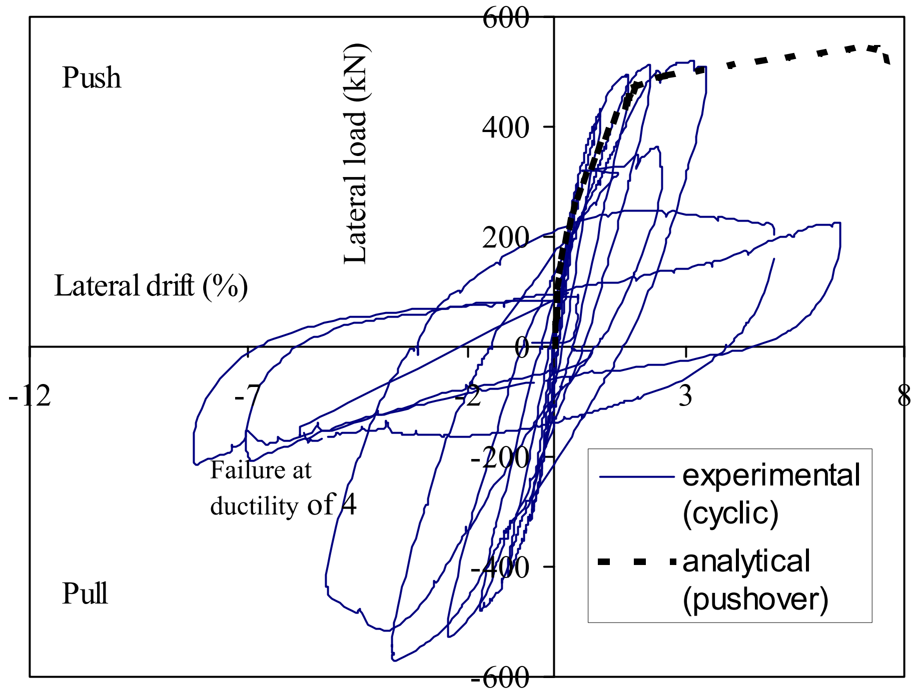

Fig. 11 Top drift of wall RW2

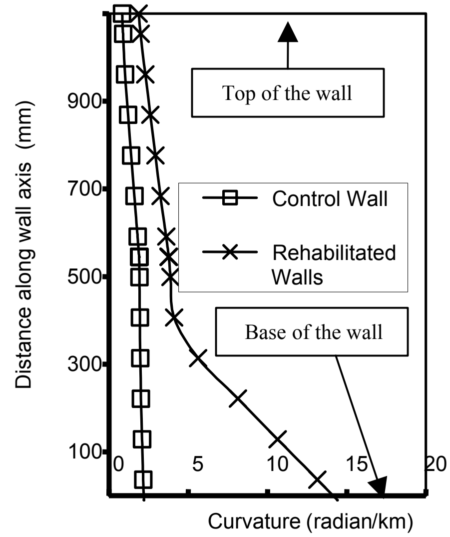

Fig. 12 Curvature distribution along the wall height

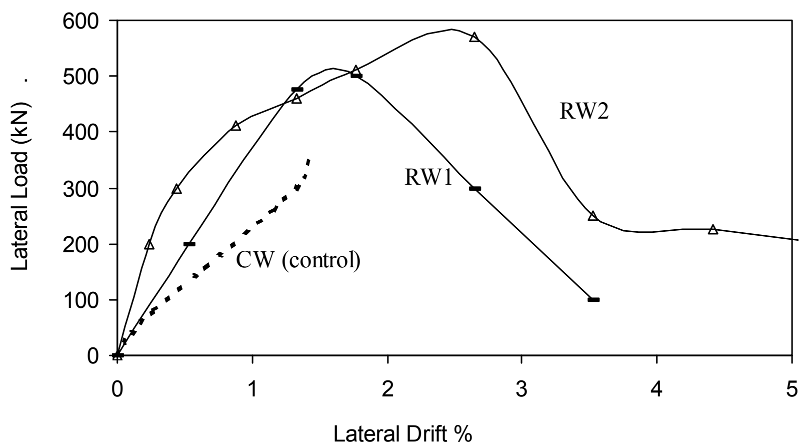

Fig. 13 Envelop for lateral drift based on the experimental cycles

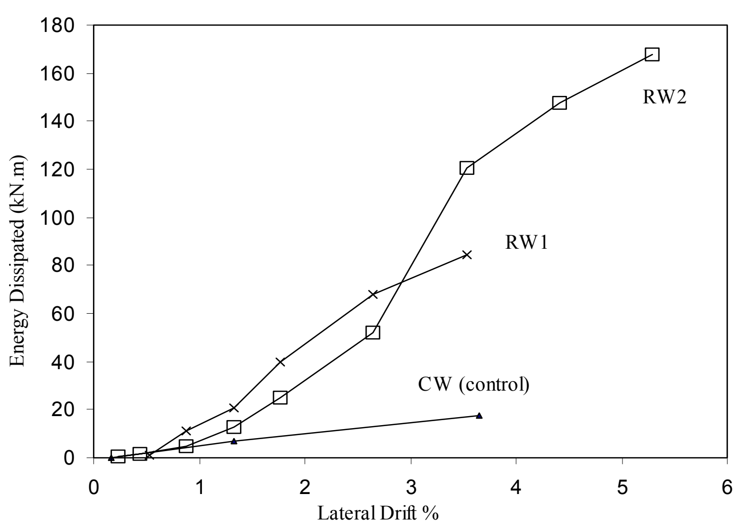

Fig. 14 Envelop for energy dissipation