SEISMIC STRENGTHENING OF EXISTING TYPICAL JAPANESE WOOD HOMES USING GFRP SYSTEMS

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Abstract
It is estimated that out of existing 47 millions homes in Japan, approximately 11.5 million need urgent strengthening. These homes do not meet current earthquake resistant standards and would face severe damage in the event of a "Shindo" 7 quake as it is know on the Japan Meteorological Agency's seismic intensity scale.

Wooden houses in Japan are typically built by wood post-and-beam methods over a concrete strip foundation. Due to the constant mild seismic activity many of the existing homes have been further weakened with evidence of cracks in the foundation and in the wood-mortar walls. The currently available seismic strengthening systems involve a massive amount of intrusive work to the existing homes and are beyond the budget of an ordinary Japanese family.

This paper serves as a case study, from Concept to Implementation, into how an affordable minimally intrusive seismic strengthening system was developed to strengthen typical Japanese wooden houses for earthquakes resistance by using Glass Fibre Reinforced Polymers (GFRP) materials. It will highlight the concept for the concrete strip foundation and the concept for the wood-mortar wall strengthening with GFRP as well look into specific details of four years of research and development with the participation of Kyushu University in Japan, Nanyang Technological University in Singapore and Oita University.

Keywords: *Seismic Strengthening, Homes, Wood framed walls GFRP*

1. Introduction

A report by the Japanese Ministry of Transportation, Land and Infrastructure showed that approximately 41% of all Japanese homes need some sort of urgent seismic strengthening [1]. The strengthening systems currently available are far beyond the budget of an average Japanese household as it is required to change the structure of the house, which includes dismantling of walls and floors to add new structural members (such as wooden diagonal posts and bracings…etc.), then reassemble them and finally reapplying the architectural finishing.

This paper introduces a new method of strengthening typical Japanese wooden houses against seismic activity by using GFRP materials. This is achieved by reinforcing existing standardized cracked concrete strip foundations with GFRP to increase its flexural capacity as well as by increasing the shear capacity of the standardized wood framed mortar walls by applying a diagonal GFRP bracing system. The system can be installed directly onto the external walls of the house right over the architectural finish and is able to dramatically enhance foundation's flexural capacity as well as the wall's shear capacity.

The goal of this program was to test and develop and reliable, cost effective method to retrofit typical Japanese homes using minimally invasive methods as not to disrupt the lives of its occupants.

2. Concept for Strip Foundation Strengthening with GFRP

Fiber reinforced polymers (FRP) have been proven to enhance the flexural capacity of flexural members such as reinforced concrete beams and slabs by bonding the composite to the extreme tension surfaces of the member.

This practice of external bonding of FRP is an accepted practice with various technical reports; recommendations and codes of practice available worldwide. The concrete strip foundation of a typical Japanese home resembles and inverted T-beam, however due to site constraints, the accepted method of bonding FRP to the extreme tensile concrete surface of the strip foundation for enhancement of flexural capacity is not physically possible. In a majority of the homes only the external vertical surface of the strip foundation is accessible. With this major constraint in mind, a concept was put forth to bond GFRP to one side of the typical strip foundation as close to the tensile region as possible to enhance its flexural capacity as seen in Figure 1 in Appendix A.

2.1 Summary of Testing of GFRP strengthened Strip Foundation

In 2006 an in depth experimental study was conducted at the Graduate School of Civil Engineering, Kyushu University, Japan. Only the salient points of this study are pointed out in the sections that follow to determine the most effective scheme to strengthen the concrete strip foundation of a typical Japanese home.

2.1.1 Specimen dimensions and material details

Six full scale reinforced concrete T-shaped strip foundation specimens with dimensions and reinforcement typically found in the majority of homes were tested to investigate the possibility of increasing their flexural capacities by applying GFRP in various configurations based on the typical site constraints. All 3000mm length T-shaped specimens were typical full-scale cross-sections. All concrete specimens were cast with Grade 45MPa concrete, reinforced with 10mm diameter (fy= 364MPa) main bars and 13mm diameter at 100mm centers (fy=499MPa) single leg stirrups. Each specimen had a 100mm wide web by 420mm depth, with a 500mm wide flange of 120mm depth as shown in Figure 2 [2].

2.1.2 GFRP Material Selection

The uni-directional GFRP used in the research had composite values as follows: Ultimate tensile strength of 576 Pa, tensile modulus of 2.61 GPa, and elongation at break of 2.2% and a total composite thickness of 1.3mm per layer [3].

2.1.3 Strengthening Scheme

One control specimen (Type A) and five GFRP wrapped specimens with different wrapping configurations (Types B to F) were tested after pre-cracking and epoxy injecting the specimens. Pre Cracking and epoxy injection of the specimens was carried out to replicate the on-site condition of existing foundations prior to wrapping. The specimen wrapping configurations as seen in Figure 3, had GFRP running horizontally for the full length of the specimens as follows: Type B: 1-layer on one entire face, Type C: 2-layers (one full height and one partial height) on one face, Type D: 2-layers (one full height and one partial height) on both faces, Type E and F: 2-layers (both partial height) on one face with the introduction of 10mm diameter GFRP fiber anchors at 450 mm on center.

2.1.4 Test Setup and Loading

After the full curing of the composite, two-point static loading with a constant bending moment zone of 500 mm was applied gradually up to failure of the control specimen (Type A), the strengthened specimens without fibre anchors (Type B through Type D) and the one strengthened specimen with fibre anchors (Type E). The test setup is shown in Figure 4. The strains in the steel reinforcing bars, concrete and GFRP were measured while applying the load. The deflection of the specimens at mid-span were also recorded along with the corresponding load. For Specimen Type F, cyclic loading was introduced by controlling the deflection of the specimen, taking the deflection (δy) associated with yield load of identical specimen Type E as a base, which was tested earlier under static loading. The load increment of each further loading step was equal to 2δy and cyclic loading was applied three times in each loading step.

2.2 Test Results

The GFRP strengthening system was able to tremendously enhance the flexural capacity and ductility of the concrete strip foundation when compared the control specimen Type A. The comparison of the ultimate load capacities (Pu) of the specimens as well as the maximum strains developed in the GFRP can be seen in Table 1. Upon comparing the Pu of each GFRP strengthened specimen against the ultimate load of the control specimen (Pua), specimens Type B, Type C, Type D, Type E and Type F showed a 2.01, 2.46, 3.76, 3.44 and 3.11 times increase in ultimate load capacity respectively as shown in Figure 5.

A comparison of the ductility factor of specimen Type A (μa = 7.6) and the ductility factors of all other specimens is shown in Figure 6. The ductility factor is expressed as μ = δu / δy where δu is the deflection at mid- span at the time of ultimate load, and δy is the deflection at midspan at the time of yield loa [2]. The specimen with the maximum ductility factor equal to 22.3 was specimen Type E. This was 2.93 times higher than that of Type A. The ductility factor of cyclically loaded Type F was 16.6 and was 2.18 times higher than Type A.

It was observed that the primary failure mode of all the GFRP strengthened specimens was debonding and delamination of the composite from the concrete substrate. However the introduction of anchors in specimen Type E and Type F had a significant effect in delaying the onset of debonding and propagation of the delamination when compared to similar specimen Type C. The anchors allowed for more strain to be developed within the composite prior to debonding thus making the strengthening effect of the composite more efficient. This in turn contributed to the increase the ultimate load as well as the ductility of the anchored specimens. The strain values in the GFRP at Pu can be seen in Table 1in Appendix B.

The results of the load versus deflection relationship between specimens Type A to Type E can be seen in Figure 7. It was seen that Type C failed prematurely by delamination at a load of 70.2 kN with deflection of 17mm whereas Type E failed at a load of 98.1kN (39.7% higher than Type C) with deflection of 28.5mm. Cyclic loading seemed to have had an effect as Type F failed at a load of 88.7 kN and a corresponding deflection of 22 mm as seen in Figure 8. This failure load however was still 26.3% higher than that of specimen Type C which had no anchors.

At the conclusion of this study to test the concept of bonding GFRP to one side of the concrete strip foundation for flexural and ductility enhancement, it was determined that the most effective method to retrofit the typical strip foundation after taking into account the site constraints was to follow the strengthening scheme of specimen Type E and F. This resulted in an average flexural enhancement of 327.5% and an average increase in ductility of 255.5% over the un-strengthened strip foundation specimen.

3. Concept for Wood-Mortar Wall Strengthening with GFRP

The typical Japanese home is built on a concrete strip foundation using a wood post and beam construction method and finished with a 15mm cement mortar layer over the light wood framed walls as seen in the cross- section in Figure 9 [5]. This construction method results in an extremely poor resistance to lateral loads where even mild seismic activity results in cracking and spalling of the architectural mortar due to large deflection of the structure. To exasperate the problem further, most of these dwellings are topped off with heavy decorated ceramic of cement tiled roofs and in the event of a sizable earthquake these homes would suffer serious to catastrophic damage as these top heavy structures would deflect significantly until collapse. There are approximately ten million homes all over Japan that fit into this category [1]. In 2006 a strengthening promotion law came in to effect that stated that by 2015, 90% of Japanese wooden houses should be upgraded to handle any seismic activity up to a magnitude of 7 on the Japanese Richter Scale [5].

A unique concept was devised for strengthening Japanese wooden houses against seismic activity using GFRP materials. The concept was based on increasing the shear capacity of the typical wood-mortar framed walls by applying a diagonal GFRP bracing system. The system would be installed directly onto the external walls of the house right over the existing architectural finishes to enhance the wall shear capacity and the wall performance. The GFRP would then be covered over with a thin coat of architectural mortar making almost unnoticeable.

3.1 Testing of Wood-Mortar Walls Strengthened with GFRP

To investigate the effectiveness of this concept, a test program was initiated in September 2007 at the Nanyang Technological University in Singapore and followed with more in-depth testing at Oita University in Japan. Seven full scale standardized typical wooden wall specimens with mortar finishing were prepared and tested under in-plane cyclic loading up to failure following the instruction of the Japanese Building Standard Act, Enforcement order article 46, clauses 4 table 1-8 [6].

3.1.1 Wood Material Selection
All wood frame members were made of Japanese cedar, but only the upper beams were American pine with properties as shown in Table 2.

3.1.2 GFRP Material Selection
The GFRP composite system selected as the bracing system for six of the test walls was the same uni-directional high-strength GFRP with continuous E-glass fiber orientated parallel to longitudinal axis of the fabric that was used in the strip foundation strengthening. The same GFRP was used in order to try and keep the type of material uniform throughout the test program as well as under any future commercial environment as this would help keep costs down. Fiber anchors were once again incorporated to improve the end details and force transfer of the GFRP into the wood frame. For one specimen a custom, bi-directional GFRP with continuous E-glass oriented in the $\pm 45^{o}$ direction was used due to the specific configuration of this test wall specimen as it included a window opening. Details of the GFRP mechanical properties are listed in Table 3.

3.1.3 Specimen Details and GFRP Strengthening Scheme
The typical full scale timber wall specimen was composed of two horizontal beams (top beam and bottom sill) three vertical columns, two vertical internal studs and twenty-four lath boards as shown in Figure 10. The distance between two vertical columns is 910 mm, while the distance between the horizontal beams is equal to 2730 mm. All specimens have a total width of 2520 mm and height of 2857 mm.

The fabrication of the wooden wall section followed the same process as used in actual construction of a wooden house in Japan. The beams and columns were connected using a mortise and tenon joint with single dowel pin. The horizontal lath boards were attached to the wooden frame with nails. A waterproofing tar paper was then attached to the lath boards with staples at 300mm c/c both ways followed by a metal mesh (chicken wire mesh) stapled to the lath boards with staples 300mm c/c both ways over the waterproofing paper. A 15mm cement mortar finish was then applied to the surface of the wall over the steel mesh and waterproofing paper. The complete wall specimen configuration with lath boards and mortar finishing is shown in Figure 10.

In the experimental program, seven specimen types were tested. Specimen Type 1 (control specimen) was built to mimic a typical Japanese timber wall shown in Figure 10. Strengthened specimen Type 2 was similar to Type 1, but a one layer diagonal bracing of 300mm wide by 1.3mm thick GFRP was bonded over the mortar finishing and then fixed onto the four corners of the walls using introduced 300mm x 300mm plywood anchor plates (anchor boards) and fiber anchors as shown in Figure 11. To investigate the roll of the external finishing mortar, strengthened specimen Type 3 was prepared similar to Type 2 but without the 15mm finishing mortar as shown in Figure 12. Strengthened specimen Type 4 was similar to Type 2, but the GFRP diagonal sheets were extended and anchored into a reinforced concrete strip foundation to test the entire system as an assembly as seen in Figure 13. The role of plywood anchor plates was investigated in specimen Type 5 which was similar to Type 2 but with no anchorage plates as shown in Figure 14. This was done to determine if anchorage plates were really necessary in any future commercial environment as they involved more cost and labor. Specimen Type 6 (shown in Figure 15) followed the same strengthening system as Type 4 but represented a wall with a door-like opening. Finally specimen Type 7 used the +/- 45^{o} GFRP as this wooden wall specimen mimicked a wall with a window- like opening as shown in Figure 16. A summary of the details and GFRP strengthening scheme of all the wall specimens are listed in Table 4.

3.1.4 Test Setup

A hydraulic jack with 600 mm stroke was used to apply cyclic loading to the upper beam of the wall. The left end of the hydraulic jack was fixed to the steel frame which was used as a horizontal reaction wall. A load cell with 100kN capacity was connected to the end of the jack to measure the magnitude of the load. The wall specimens were fixed to a steel I-beam with three D16 anchor bolts and the steel I-beam was anchored to the ground as a rigid sill. The horizontal movement was restrained by the metal supports at both ends of the bottom of the wall. Four linear variant displacement transducers (LVDT) were employed to measure the displacements. The first one (H1) was used to measure the horizontal displacement at the top of the wall. The second (H2) was used to measure the bottom horizontal displacement. The last two (V3 and V4) were used to measure vertical displacements at the left and right column sides, respectively. The test setup can be seen in Figure 10.

3.1.5 Loading Procedure

To simulate the seismic loading conditions on real timber structures, cyclic loads with gradually increased amplitude are applied to the upper beam. Totally seven cyclic loading steps are applied. Three cycles push and pull are performed in each step. Loading steps is controlled by the observed Shear Transformation Angle (STA).

The details of the loading steps are shown in Figure 17. The vertical distance between the upper and lower horizontal LVDTs (H1 and H2) was equal to L= 2730 mm. The relative movement of the wall is $\delta 1$ = H1-H2. After the twenty-one cycles of cyclic loading, the specimens are loaded under static loading up to failure.

3.2 Structural Performance of the Wall

Based on the load versus STA data, the hysteretic performance envelope curve of the tested walls were drawn. The maximum load (Pmax), yield strength (Py), ultimate strength (Pu), allowable shear strength (Pa) and the ductility factor (μ) of the framed wall specimens were derived and calculated based on a bilinear model that follows the instruction of Japanese Building Standard [6]. Figure 18 shows details of the bilinear model. Line (I) is a straight line between 0.1Pmax and 0.4Pmax of the envelop curve. Line (II) is a straight line between 0.4Pmax and 0.9Pmax of the envelop curve. Line (III) is a line parallel to II and tangent to the envelop curve. The value of the yield strength (Py) can be determined by the intersection point of line (I) and line (III). The wall stiffness (K) can be obtained by dividing the value of yield strength (Py) by the value of the yield STA (Dy). The ultimate STA (Du) is equal to the value of the STA when the applied load is equal to 0.8Pmax. Line (V) is a line between the (0,0) point and the (Dy, Py) point. The ultimate strength of the wall

(Pu) is determined, so the area under the lines (V, VI and VII) is equal to the area under the envelop curve.
The Japanese Building Standard specifies that the value of the allowable shear strength (Pa) of framed wooden wall is taken as the smallest of the following values [6]:

1. Shear capacity when shear transformation angle is equal to 1/120 rad. (P120).
2. Yield capacity (Py).
3. 2/3rd the value of maximum load (Pmax).
4. The value of (0.2Pu) $\sqrt{(2\mu - 1)}$
   where: (μ=Du/Dy) is the ductility factor.

3.3 Summary of Experimental Results

All the strengthened specimens showed a remarkable increase in maximum load carrying capacity over the control wall specimen ranging from a 568% increase up to a 1036% increase depending on the type of specimen and GFRP wrapping configuration. A summary of the calculated test results can be found in Table 5. Note that the units for all the load values are based on a per meter running length of the wall (kN/m) as compared to the raw data in the hysteresis curves that show the loads in Kilo Newtons (kN). The Following five subsections compare and summarise the salient differences between the various types of wall specimens tested.

3.3.1 Type 1 vs. Type 2
The hysteretic performance of specimens Type 1 (control specimen mortar wall no GFRP) and Type 2 (with mortar wall and GFRP anchored in top wood beam and bottom wooden sill) are compared by the curves shown in Figure 11. For the control specimen without GFRP, the load capacity didn’t increase after the third load step of the cyclic loading. After the fifth step, the load dropped sharply. For specimen Type 2, the loads kept increasing throughout all the loading steps. When the applied load reached 33kN, initial failure in the wood sill ground beam occurred and the load carrying capacity of the wall started to drop gradually as the STA values increased. The allowable shear strength (Pa) of specimens Type 1 and Type 2 were 1.59kN/m and 8.63 kN/m respectively representing a 543% increase in Pa of Type 2 over Type 1. The wall stiffness also increased by
261% from 1.70 MN/rad to 4.44 MN/rad

3.3.2 Type 2 vs. Type 3
Figure 12 shows a comparison between the behaviour of specimen Type 2 (with mortar wall and GFRP anchored in top wood beam and bottom wooden sill by anchorage plates) and Type 3 (wall with no mortar but with GFRP anchored in top wood beam and bottom wooden sill with anchorage plates) under cyclic loading. Bonding the GFRP sheets directly over the mortar surface in wall specimen Type 2 seemed to change the role of the wall mortar from an architectural finish to a structural element by increasing the stiffness of the wall. The increased stiffness of the wall was 2.9 times over that of specimen Type 3 which had no mortar. The failure of Type 3 was due to horizontal longitudinal cracking in the wood sill ground beam. The Pa of Type 3 was 5.58 kN/m as compared to 8.63 kN/m for Type 2.

3.3.3 Type 2 vs. Type 4
Figure 13 shows a comparison between the behaviour of specimen Type 2 (with mortar wall and GFRP anchored in top wooden beam and bottom wooden sill by anchorage plates) and Type 4 (with mortar wall on concrete strip foundation assembly and GFRP anchored in the top wooden beam with anchorage plates and extended and anchored into the concrete strip foundation) under cyclic loading. The value of Pa for Type 4 was 1.72 times more than the value of the GFRP

strengthened specimen Type 2. The ductility factor and the initial stiffness were 1.93 and 1.4 times greater than that of Type 2. By extending the GFRP into the concrete strip foundation as in specimen Type 4, failure of the wood sill ground beam that sits on the foundation was averted as that seemed to be the weakest point in the system thus far. Specimen Type 4 also showed a remarkable increase in Pa of 938% over the control wall specimen Type 1 and a maximum load (Pmax) carrying capacity increase of 1036% over the control.

3.3.4 Type 2 vs. Type 5

Figure 14 shows a comparison between Type 2 (with mortar wall and GFRP anchored in the top wooden beam and bottom wooden sill by anchorage plates) and Type 5 (with mortar wall and GFRP anchored directly in the top wooden beam and bottom wooden sill without anchorage plates). Even though the Pmax of Type 5 was smaller than Type 2 by 11.4%, the Pa of Type 5 (8.14 kN/m) represented only a 7.8% drop over the Pa of Type 2 however this was still an 512% increase in Pa over the control specimen. What could be inferred with this result was that one could do away with the wooden anchor plates at each of the 4 corners of the wall specimens and directly insert to fibre anchor into the wooden posts and beams at the GFRP strip termination ends. This would be a cost and time saving under a commercial environment.

3.3.5 Type 6 and Type 7

The allowable shear carrying capacity for specimen Type 6 (mortar wall with a door like opening) after strengthening with GFRP is equal to 9kN/m, while the Pa for Type 7 (mortar wall with a window like opening) is equal to 3.8kN/m. Since there was no direct control to compare these type specimens, one can’t directly compare the increase in Pa. However, common sense dictates that control specimen Type 1 without a door or window opening would actually have a higher Pa than if there were control specimens of Type 6 and Type 7 with a door and window opening respectively. In using this assumption, the Pa of Type 6 and Type 7 would show a significant increase over their controls.

4. Conclusion

A three year study at three different universities was undertaken to systematically test, develop, patent and obtain performance based approvals for a method to strengthen wooden homes in Japan using GFRP.

Testing of standardized typical concrete strip foundations that are found in the majority of old homes strengthened with strips of GFRP bonded and anchored to only one side of the specimen showed a great increase in moment capacity and ductility over the un-strengthened control specimen. An average flexural enhancement of 327.5% and an average increase in ductility of 255.5% over the un-strengthened strip foundation specimen was obtained.

Testing of standardized typical wood-mortar walls that are found in the majority of wood construction homes in Japan for enhancement of shear capacity by bonding and anchoring a diagonal GFRP bracing system on the external mortar surface of the wall showed a significant increase in the allowable shear capacity of the strengthened wall specimens over the un-strengthened control wall. The increase in allowable wall shear capacity on the wood-mortar wall specimens ranged from 512% to 543% over the un-strengthened control wall specimen. When the diagonal GFRP bracing system was extended further down onto the concrete strip foundation a 938% enhancement of allowable shear capacity was observed.

At the conclusion of the wall study, it was clear that the existing weak wood-mortar walls of a typical Japanese home could be strengthen substantially with the use of GFRP strips applied in a cross brace pattern to the external surface of the walls and anchored into the already strengthened concrete strip foundation that was tested in the first phase of this research and development project. This minimally invasive, light weight system could be installed over the external walls of Japanese houses without disturbing the daily life of the residents as a majority of the work could be executed from the outside of the home.

This development program has resulted in a new, unique, effective and simple method to retrofit the concrete strip foundations and wood-mortar walls of a typical Japanese wood construction home for seismic resistance using of-the-shelf, readily available GFRP materials so as to meet the strengthening requirement as prescribed by the Japanese government. The system has subsequently been patented and approved by the local authorities and been on the market in Japan since March 2009 and has been installed on a number of homes throughout Japan.

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Appendix A – Figures

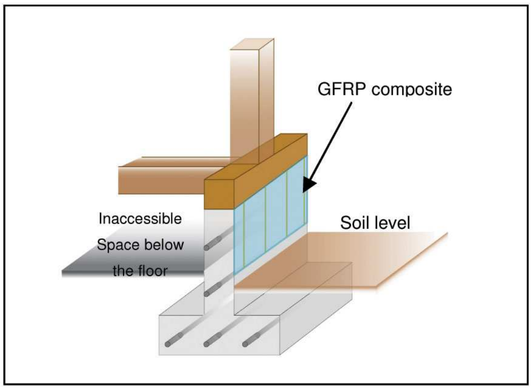


Figure 1: Proposed bonding of GFRP to External Face of strip Foundation

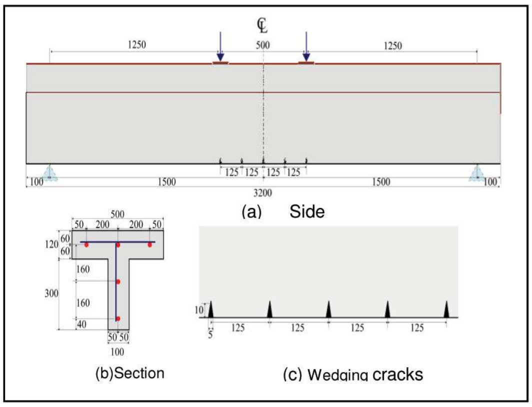


Figure 2: Specimen Dimensions and Details

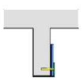
Type	Section	Description
A		Control Specimen – No GFRP
B		One layer of GFRP (3200mm x 300mm) bonded to one side of the web.
C		Two layers of GFRP: First layer (3200mm x300 mm) and second layer (3200mm x 150mm) bonded to one side of the web.
D		Two layers of GFRP: First layer (3200mm x 300mm) and second layer (3200mm x 150mm) bonded to both sides of the web.
E & F		Two layers of GFRP: Both layers (3200mm x 200mm) bonded to one side of the web. Six GFRP fibre anchors installed at 450mm c/c over the 2 GFRP layers.

Figure 3: GFRP Wrapping Scheme of Specimens Type A to Type F

Figure 4: Test Rig Setup of Strip Foundation

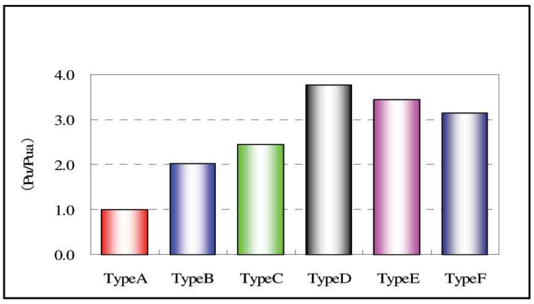


Figure 5: Ultimate load Comparison of Strip Foundation Specimens

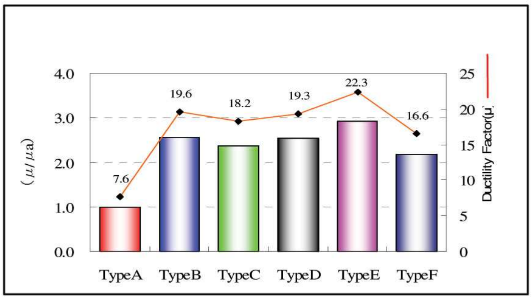


Figure 6: Comparison of Strip Foundation Specimen Ductility

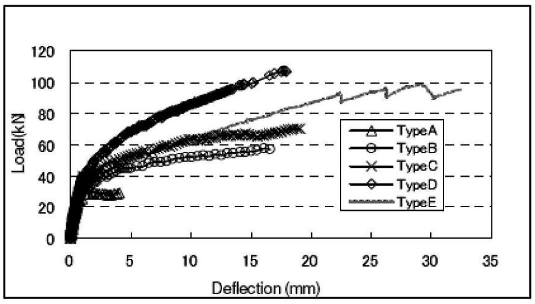


Figure 7: Load-Deflection Relationship of Specimens Type A to Type E

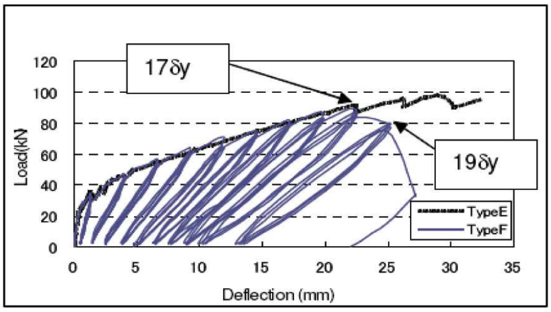


Figure 8: Load-Deflection Relationship of Specimens Type E and Type F

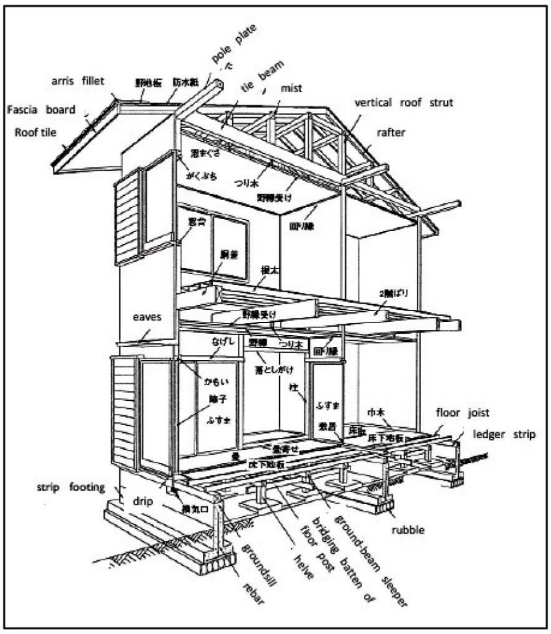


Figure 9: Cross Section of a Typical Japanese Home with Standardized Concrete Foundation and External Walls [5]

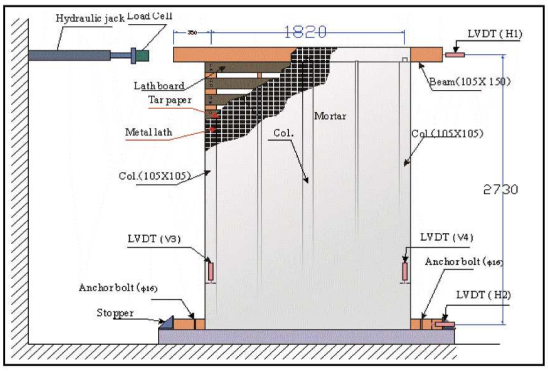


Figure 10: Test Setup and Cut Away Details of Typical Wall Specimen Type 1 (Control)

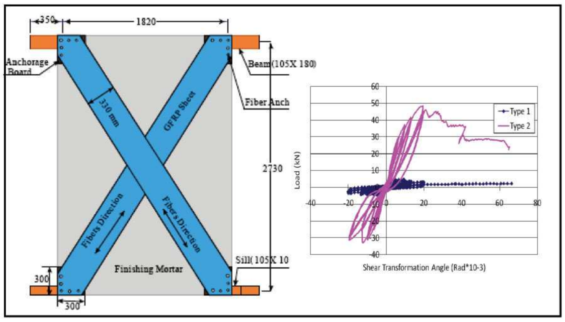


Figure 11: Wall Type 2 – Layout and Results

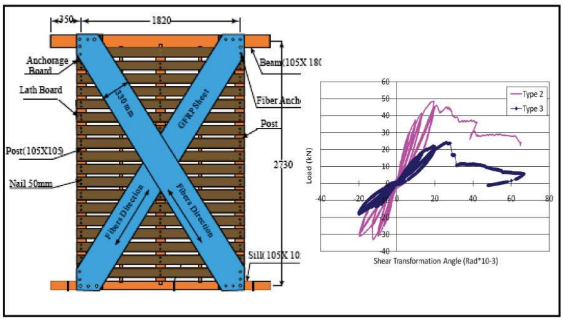


Figure 12: Wall Type 3 – Layout and Results

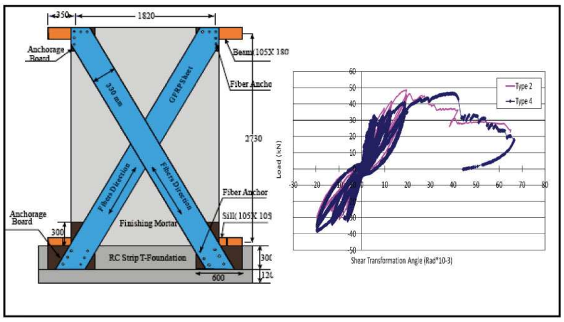


Figure 13: Wall Type 4 – Layout and Results

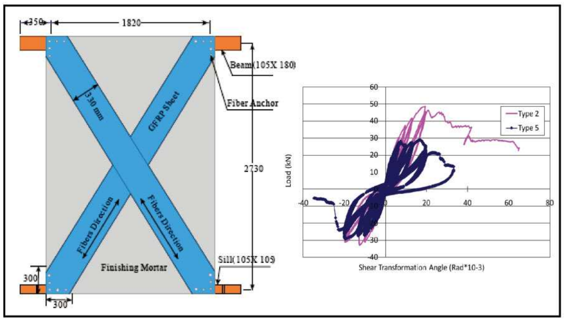


Figure 14: Wall Type 5 – Layout and Results

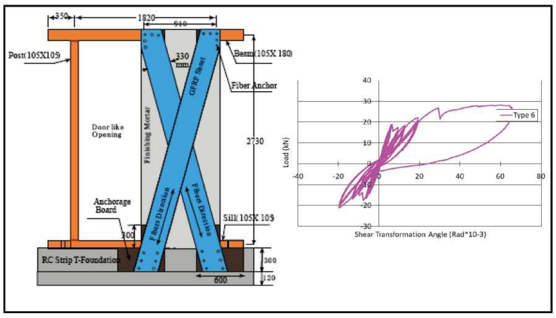


Figure 15: Wall Type 6 – Layout and Results

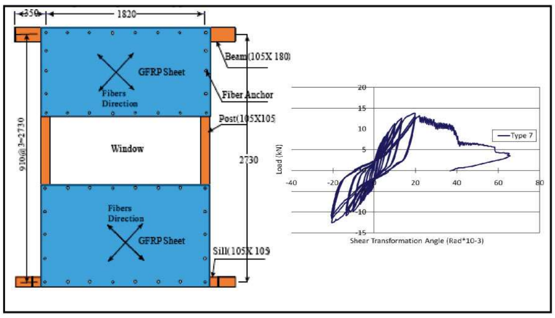


Figure 16: Wall Type 7 – Layout and Results

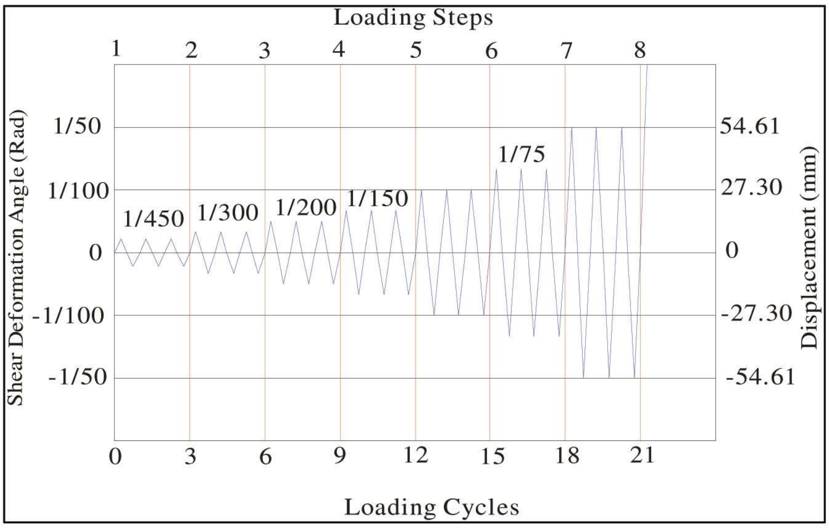


Figure 17: Load Step Procedure for Wall Tests

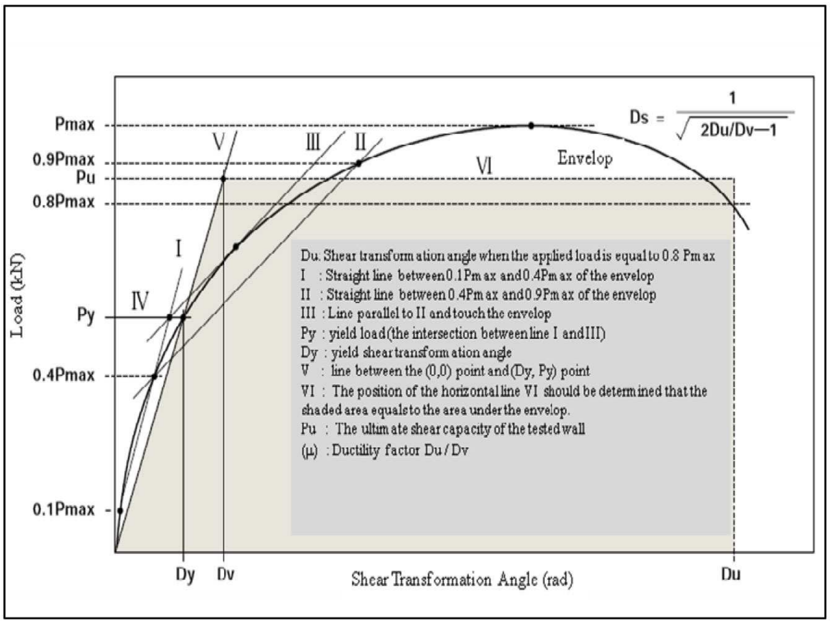


Figure 18: Details of Calculating Yield Strength, Ultimate Strength, Allowable Shear Strength and Ductility Factor of the Framed Wall Specimens using the Bilinear Model [6]

Appendix B: Tables

Table 1: Summarized Experimental Results for Concrete Strip Foundation

Property / Specimen Type	A	B	C	D	E	F
Crack Load, Pcr (kN)	23.0	20.5	23.5	23.7	20.5	20.5
Yield Load, Psy (kN)	20.6	29.2	36.0	36.6	28.3	28.3
Ultimate Load, Pu (kN)	28.5	57.3	70.2	107.3	98.1	88.7
Pu / Pua	1.0	2.01	2.46	3.76	3.44	3.11
GFRP Strain at Pu (%)	---	0.8	0.5	1.2	1.35	1.2

Table 2: Wooden Members Mechanical Properties

Members	Wood Type	E-Modulus (GPa)	Comp. Strength (MPa)	Tensile Strength (MPa)
Post	J. Cedar	6.86	17.7	22.2
Sill-Ground	J. Cedar	6.86	17.7	22.2
Beam	A. Pine	9.80	22.2	28.2
Lath Boards	J. Cedar	6.86	17.7	22.2

Table 3: GFRP Mechanical Properties [2], [3], [4]

GFRP	E-Modulus (GPa)	Ultimate Tensile Strength (MPa)	Ultimate Strain	Laminate Thickness per Layer (mm)
Unidirectional GFRP	26.1	575	2.2%	1.300
+/- 45^o GFRP	18.6	279	1.5%	0.864
Fiber Anchor	26.1	575	2.2%	10 mm Diameter

Table 4: Wall Specimen Details

Specimen Type	With Cement Mortar	Anchorage Plate	With RC Foundation	GFRP
Type 1	Yes	No	No	No
Type 2	Yes	Yes	No	Unidirectional
Type 3	No	Yes	No	Unidirectional
Type 4	Yes	Yes	Yes	Unidirectional
Type 5	Yes	No	No	Unidirectional
Type 6	Yes	Yes	Yes	Unidirectional
Type 7	Yes	No	No	+/- 45^o Bidirectional

Table 5: Wall Experimental Testing Results

Specimen Type	Wall Stiffness, K (MN/rad)	Incr. in K vs. Ctrl	Pmax*	Incr. in Pmax vs. Ctrl	Py*	Incr. in Py vs. Ctrl	2/3Pmax*	0.2Pu√(2μ-1)*	Pa*	Incr. in Pa vs. Ctrl
Type 1 (Ctrl)	1.70	---	2.47	---	2.20	---	1.65	1.59	1.59	---
Type 2	4.44	261%	18.13	734%	15.38	699%	10.99	8.63	8.63	543%
Type 3	1.53	-10%	14.04	568%	9.14	415%	8.03	5.58	5.58	351%
Type 4	6.17	363%	25.59	1036%	15.99	727%	17.10	14.91	14.91	938%
Type 5	6.73	395%	16.06	650%	12.93	588%	8.26	8.14	8.14	512%
Type 6	3.22	---	15.53	---	9.01	---	10.02	10.98	9.01	---
Type 7	1.85	---	7.64	---	5.63	---	3.77	4.18	3.77	---

* Units in kN/m