SEISMIC DESIGN AND EXPERIMENTAL INVESTIGATION OF A NEW TIMBER HOLD-DOWN CONNECTION

A. Marchisella(1)*, M. Cervio(2), G. Muciaccia(3)

(1) Phd Candidate, Department of Civil and Environmental Engineering, Politecnico di Milano, Italy, angelo.marchisella@polimi.it
(2) Phd Candidate, Department of Civil and Environmental Engineering, Politecnico di Milano, Italy, marco.cervio@polimi.it
(3) PhD, Ass. Prof., Department of Civil and Environmental Engineering - Politecnico di Milano, giovanni.muciaccia@polimi.it
** Corresponding Author*

Abstract

Timber connections made with hold-down are usually adopted for timber wall to foundation connection at the edges of the wall to restraint the possible overturning moment (due to rocking) which can occur by applying a horizontal force to the panel (e.g. under seismic action). The main components of the connection are the steel plate fastened to the panel via threaded screws or nails and the anchor system to the concrete foundation. As such, the connection behavior under seismic loading is characterized by the contemporarily contribution of different resistant mechanisms, i.e. (i) laterally loaded timber screws or nails; (ii) axially loaded steel plate; (iii) axially loaded anchor in concrete. The former has been basically neglected in past investigations having anchored the hold-down with steel bolt directly to the strong floor. However, the design of the anchor should be considered as crucial when it comes to capacity design, especially in case of narrow foundations (edge failure) and post-installed anchors when the resistance is highly reduced with respect to steel bolt capacity. Moreover, the overall dissipative performance might be affected by the anchor's displacement. An innovative hold-down connection has been designed to promote steel failure of the axially loaded plate. The plate's geometry has been optimized such that capacity design rules according to Eurocode 8 can be applied. Additionally, a target displacement requirement is addressed, and the stretch length is defined accordingly. The whole connection is tested against cyclic loading and results show enhanced performances with respect to standard configuration. In particular, brittle mechanism such as concrete-cone failure of the anchors and splitting of timber are prevented. The paper discusses the experimental results for different type and size of the anchor, also the size of the optimized steel plate is considered as a parameter.

Keywords: Hold-Down; Steel-to-Timber Connection; Ductility;

1. Introduction

The timber connection made with hold-down is usually adopted for timber wall-to-foundation connection at the edges of the wall to restraint the possible overturning moment (due to rocking) which can occur by applying a horizontal force to the panel (e.g. seismic condition). The main components of the connection are: (i) steel plate; (ii) threaded screws or nails; (iii) anchors to the concrete foundation. Consequently, the connection behavior is characterized by the contemporarily contribution of different resistant mechanisms, i.e. (i) laterally loaded screws; (ii) axially loaded steel plate; (iii) axially loaded anchor.

1.1. Literature Review

In a brief survey conducted on the same topic, i.e. hold-down connection, the following relevant works differentiated for their application area can be of interest:

1. Experimental characterization of hold-down connection (at the level of the sub-assembly): [1][2][3][4]. It is worth mentioning that in none of the referred works, the problem the issue of axially loaded anchor in concrete it is not addressed since in every experimental campaign the hold-down is fastened to the strong floor or reactions steel frame and the dissipative behavior of the connection is evaluated accordingly.

2. Experimental characterization of hold-down connection (at the structural level of shear wall): [5][6][7][8]. Same considerations regarding the absence of concrete foundation applies also in these experimental campaigns.

3. Analytical Studies (at structural level): [9][10][11][12]. Particularly, in [9] an example of application for the design of a shear wall system is reported. The problem of strength hierarchy is markedly addressed, but the overstrength factor seems to be used in a wrong way when it comes the inequality between the mechanism involving the steel plate and the axially loaded anchor. Additionally, for the former case, the technology of cast-in anchor is addressed. Whereas, the presented paper intends to suggest the use of post-installed anchor with the suitable use of overstrength factors.

1.2. Research significance

In Figure 1 two different hold-down prototypes are shown after tensile test execution. In the first case, in the following named “as-built” (see (a)), the failure is characterized by the break of the flange’s section hosting the holes for the screws. In the second case, in the following named “optimized geometry” the failure concentrates in the reduced cross-section, with no evidence of damage both at screws and at the concrete side. The first case can be considered as “classical” hold-down configuration. The vertical flange is characterized by the rectangular cross-section having height (h) and thickness (t). The holes hosting the screws are characterized by the diameter (Φ_{holes}) and the base has a square plan having the dimension equal to (b). In most of the cases the base hosts an additional steel plate (named also “thick washer”) to stiffen the base behavior.

It is worth mentioning that in the as-built condition, the vertical flange failure is achieved only when the hold-down is fastened to the strong floor, as constantly adopted in experimental campaigns mentioned in the literature survey. Conversely, anchor’s failure was achieved when a post-installed anchor was used to fasten the hold-down to the concrete member. In this paper the performance of an optimized geometry, promoting the ductile behavior (steel failure), addressing the use of post-installed fasteners.

The plan of the paper is as it follows: (i) in the section titled “Experimental details and seismic design” the design of the hold-down (taking into account design values) is explained and the details of the experiments are outlined; (ii) the section “Experimental results” report the results obtained in both monotonic and cyclic tests; (iii) in the section “Discussion” comparison of the response for as-built and optimized geometry are given and an insight on the mechanical behavior with the optimized geometry is discussed; finally in the section “Conclusions” the work is summarized.

(a) (b)

Figure 1 - Hold-down after test execution: (a) Prototype As-Built; (b) Prototype Optimized

2. Experimental details and seismic design

2.1. Test matrix

In the following section details for the experimental campaign are given. The experimental campaign can be divided into two part, namely (i) as-built hold-down and (ii) optimized hold-down. In both the cases, different types of anchor are tested (including the case of strong floor anchoring). The models of the hold-down differ for the presence, in the case of the optimized prototypes of a vertical flange with reduced cross-section. Such a type of geometry favors the development of ductile failure (yielding of the steel plate). Details of the tested prototypes with the assumed nomenclature for the tests are given in Table 1 and Table 2, respectively.

Table 1 – Hold-down prototypes label

Test Label : AA_BB	
AA	**BB**
ASB: As-Built	S: Static
OPT: Optimized geometry	C: Cyclic

Table 2 – Hold-down test matrix

	Test	ASB_S/C	ASB_S/C	ASB_S/C	OPT_S/C	OPT_S/C	OPT_S/C
Hold-Down		R620	R620	R620	SHD_620	SHD_540	SHD_440
t	(mm)	3.0	3.0	3.0	3	3	2.5
h	(mm)	620	620	620	620	540	440
b	(mm)	80	80	80	80	70	60
Φ_{holes}	(mm)	5.1	5.1	5.1	5.1	5.1	5.1
Steel		S355	S355	S355	S355	S355	S235
f_{yk}	(MPa)	355	355	355	355	355	235
f_{uk}	(MPa)	510	510	510	510	510	360
Net Area	(mm^2)	225	225	225	136	75	50
F_u	(kN)	114.7	79.8	79.8	69.3	38.3	18.0
Stretch Length	(mm)	-	-	-	50	70	70
Screws		5x40	5x40	5x40	5x40	5x40	5x40
Φ_{screw}	(mm)	5	5	5	5	5	5
L_{screw}	(mm)	40	40	40	40	40	40
n_{screw}	(-)	53	53	53	45	30	20
F_{screw}	(kN)	1.92	1.92	1.92	1.92	1.92	1.92
Anchor		Bolt	Sleeve	Bonded	Bonded	Sleeve	Sleeve
Threaded rod	(-)	M20(8.8)	M20(8.8)	M20(8.8)	M20 (8.8)	M24 (8.8)	M16 (8.8)
f_{yk}	(MPa)	640	640	640	640	640	640
h_{eff}	(mm)	-	125	160	250	150	100
Test Data							
F_{max}	(kN)	97.5/-	65.1/-	61.7/-	66.1/73.2	31.3/41.2	20.2/22.0
F_{max}^{Anch}	(kN)	121.4	66.1	62.5	123.3	49.1	-
v_{max}	(mm)	18	4	5	18	20	22
Failure	(-)	Y	CC	PO + CC	Y	Y	Y
k_t	(-)	-	-	-	1.41	1.25/1.10	-

NOTES. (F_{max}) Half of the maximum force measured by the load cell in series with the hydraulic jack; **(**F_{max}^{anch})Maximum force measured on the anchor via washer cell; (v_{max}) displacement at failure; (Failure) [Y] Failure of the vertical flange (at the critical section) after reaching the yielding point, [CC] Concrete-Cone failure, [PO + CC] Combined Concrete Cone failure and Pullout. (k_t) eccentricity factor.

2.2. Seismic Design of the prototypes

The optimized prototype is designed considering two assumptions on the overstrength hierarchy between the relevant failure mechanisms and promoting the steel failure of the vertical flange.

In the case of anchors, the occurrence of brittle mechanisms such as concrete break-out, must be prevented by the adoption of suitable overstrength factors, as reported in the following equations:

$$F_{rk,anchors} / \gamma_{Ma} > k_t \cdot \gamma_0 \cdot F_y / \gamma_{Mste} \quad (1)$$

being k_t the eccentricity factor (experimentally evaluated with value equal to 1.25, and better explained in the next section), γ_0 the overstrength factor for steel yielding mechanism, which assumes a value equal to 1.30 and (γ_{Mste}) the material safety factor which for steel is assumed equal to 1.05. The material safety factor for the anchor (γ_{Ma}) is product dependent and prescribed by the manufacturer. In this study, the value assumed for both the mechanical anchor (sleeve) and the bonded anchor is 1.5. Additionally, severe cracked condition for the concrete base material are assumed, resulting in the C2 seismic category, according to [13]. Based on the abovementioned inequality the net area to be designed for the vertical flange is calculated.

In the case of strength hierarchy between failure at the screws side and steel failure of the vertical flange, as it is markedly proven in the next section presenting experimental results, the overall capacity of the screws group is higher with respect the bare summation of the single screw capacity (against lateral load). The explanation for this is twofold: (i) the capacity of the group as whole can be enhanced by the presence of friction resistance mechanism between steel and timber promoted also, in large displacement, by the presence of a restrain effect due to the double configuration (as better discussed in the next section, it can be considered as negligible); (ii) the analytical evaluation of the lateral resistance of a single screws might be too much conservative with respect the redundancies offered by the group behavior. In the following equation, the strength hierarchy considering the screws' failure is reported:

$$\Psi_{eff} \cdot F_{rk,screws,\,tot}\, 1/\gamma_{M3} > \gamma_0 \cdot F_y / \gamma_M \quad (2)$$

Being Ψ_{eff} the effectiveness factor evaluated according to the Eurocode 5 [14] in case of group of screws laterally loaded ($\Psi_{eff} = n_{eff} / n_{tot} < 1.0$). The material safety factor adopted in timber-to-steel connection (γ_{M3}), usually it assumes a value equal to 1.30. The symbols presented in the right part of the inequality have been already discussed in the case of the equations (1).

It is worth noting that the prototypes are designed directly considering the design equations (or inequalities). Consequently, the connection (in its final configuration) is intended to fulfill the requirement of Eurocode 8 [15] in terms of strength hierarchy and capacity design at the level of design stage.

The authors are aware about the large margin achieved by such kind of force-based approach in terms of load-carrying-capacity. Nevertheless, aside the mere verifying the correctness of the design assumptions via testing, the interest in the final displacement capacity of the connection and the cyclic performance paved the way to perform the tests in the case of the optimized geometry. In this regard, all the prototypes with optimized geometry are designed having 50 to 70 mm as stretch length (zone of the vertical flange characterized by the reduced cross-section), in order to guarantee sufficient elongation during the yielding phase. Assuming a conventional level of deformation of steel at ULS equal to 40%, the resulting displacement target is equal to 20 mm, in the case of the 50 mm as stretch length. This value has been confirmed during the experimental phase, as discussed in the next section.

Three different size of the hold-down connection are designed, with different load-carrying capacity. Referring to the characteristic load-carrying capacity (considering the breaking of the steel flange as a dominant mechanism) the following values are achieved: 69.3 kN; 38.3 kN; 18.0 kN for hold-down having the height (h) equal to 620 mm, 540 mm, 440 mm, respectively. More details can be found in Table 2.

2.3. Test Configuration

In Figure 2 the test setup is shown. A timber post having dimensions 720x320x160 mm is connected to a concrete slab 1600x750x500 mm, using two hold down devices. For the timber post a glulam GL24 class is used. Concrete is C20/25 class. Anchors are installed in un-cracked configuration.

The test configuration having two hold-down, symmetrically installed, can be reliable in simulating the behavior of a shear panel under rocking movement for the following reasons: (i) the out-of-plane restraint degree is guaranteed; (ii) considering the low level of variability characterizing the steel yielding point and steel failure, a symmetric behavior during the test execution can be pursued; (iii) in case of failure at the anchorage to concrete such configuration provides enough safety to control the post-failure, preventing the occurrence of unpredicted shear loading the cylinder shank. Nevertheless, tests with single hold-down installed have been performed too.

The load is applied using a hydraulic cylinder of 30 ton capacity, with the load axis lying in the symmetry axis of the connections. The test is carried out in displacement control, i.e. the stroke of the cylinder is set as a control variable applying a rate of 0.08 mm/s in case of monotonic test. LVDTs measure the displacements of two point per sides, i.e. (i) the steel flange and (ii) the corresponding timber fiber as it is shown in Figure 3. During the execution of the test, the vertical flange undergoes in large displacement having a deformed shape which is characterized by the bending of the flange and consequently, the occurrence of a surface pressure loading the vertical flange sheet. Being the configuration symmetric, the occurring actions equally counter-balanced. However, this parasitical effect might affect the lateral-load-carrying capacity of the steel-to-timber connection in terms of enhanced resistance due to the activation of friction steel-to-timber. Comparing the results obtained in trial tests with single configuration, this effect should be considered negligible in the overall evaluation of the connection behavior.

The mechanical behavior of the connection is characterized by a kinematic variable (s, conventional slip) which does not coincide with the cylinder's stroke, but it follows the trend proportionally. Such behavior is obtained, providing that the load equipment provides enough stiffness during the loading path.

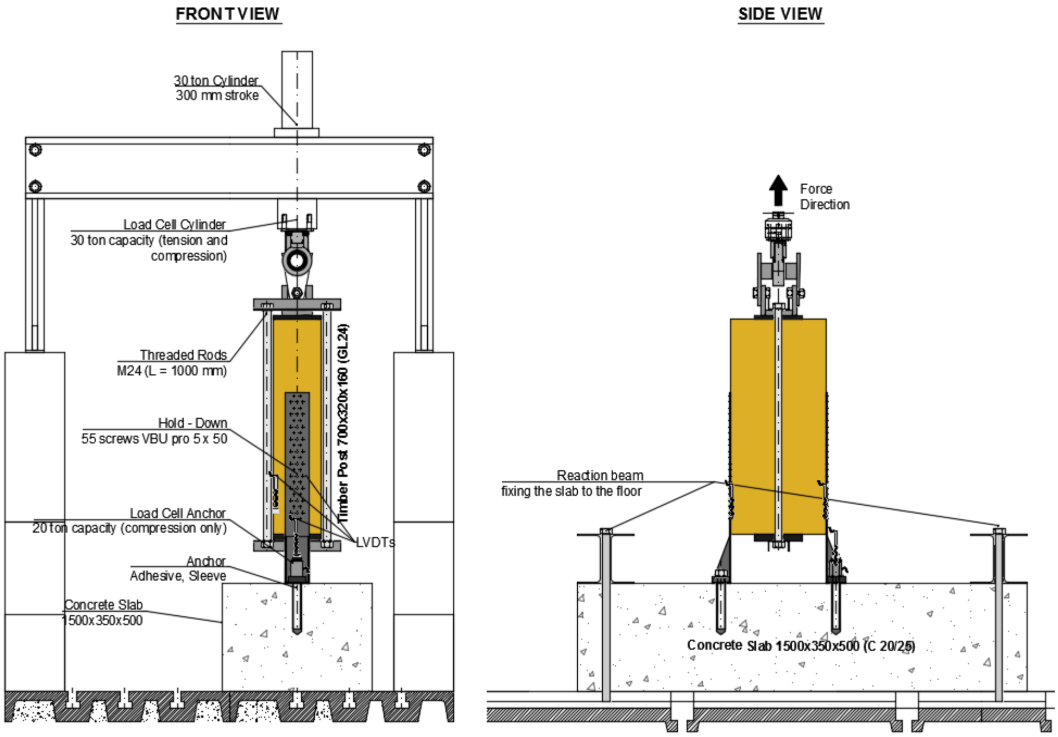
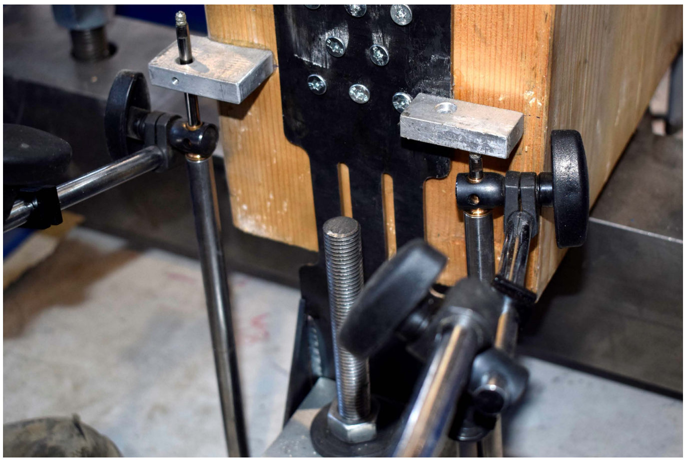


Figure 2 – Test Setup

(a)



(b)

Figure 3 – Test Setup photo: (a) frontal layout and (b) LVDT's position measuring conventional slip

2.4. Test Procedure

The test procedure, described in [16], applies to both the cases of as-built and optimized geometry. The mechanical behavior assessment under cyclic loads of steel-to-timber connection is performed following a conventional procedure. Specifically, a monotonic test is carried out and consequently the yielding point (F_y; V_y) is evaluated. A cyclic test is then performed used a step-wise increasing displacement protocols. The target displacements (steps) are multiple of the V_y (e.g. $0.75/1.00/1.50/2.00 \cdot V_y$ etc.) defined in the monotonic test. Three cycles are performed per each step. Since there is no precise specification in [16] for (i) the duration of the monotonic test and (ii) the frequency to be adopted during cyclic test, the following threshold supported by reference literature [17] [3] are defined: 30 minutes for test duration and 1 mm/s for the stroke rate.

The mechanical behavior of the connection is studied considering the definition of a conventional kinematic variable, i.e. the conventional slip. It is worth mentioning that a non-unique definition is accepted and in most of the cases [1] [3] [18] the relative displacement (slip) between the different connection's components is considered. In particular, the case of the connection made with hold-down the following slip sources are recognized: (i) screws' lateral displacement; (ii) steel elongation; (iii) anchor displacement. In the presented work, the following hypothesis are made:

1. The displacement of the steel flange and the corresponding timber fiber is measured, and the slip is evaluated. The anchor's displacement is not measured continuously. In the test with the optimized geometry, a dial gauge has been installed at the anchor's head.
2. The displacement of the steel flange is considered as the reference kinematic variable.
3. Considering the behavior of the hold-down connection, a tension-only constitutive law is recognized. In fact, the compression phase (due to rocking) is completely taken up by the contact between the timber panel and the foundation. For this reason, in this test campaign, the displacement protocol does not include the sign change.

3. Experimental Results

3.1. As-Built

3.1.1. Monotonic tests

In the as-built condition, different types of anchors are tested using the same hold-down. Results show that sleeve anchor and bonded anchor are not able to reach the steel failure. However, in case of anchoring the hold-down at the strong floor the yielding of the vertical flange is reached at the cross section with reduced area (due to the presence of holes). The load-displacement plot for the three different anchors types are presented in Figure 4. Ductile behavior is obtained only in case of anchor fastened to the strong floor (named “anchor bolt”). In the other two cases the behavior is characterized by an abrupt loss of strength after reaching the point of maximum load. This kind of behavior is rather typical in brittle mechanisms involving the formation of concrete cone [19]. In particular, the following mechanisms are observed: (i) in case of bonded anchor the combined Pullout and Concrete cone mechanism is achieved as can be inferred in Figure 6; (ii) in case of sleeve anchor (see Figure 7) concrete cone failure is obtained with the contemporarily presence of a splitting crack. In this cases, the non-symmetric behavior is experienced quite suddenly with respect the case of steel failure. The concrete cone develops at one side of the double configuration.

Only in the case of the fastening to the strong floor, the formation of plastic hinges to the screws were observed to the first row of screws as can be observed in Figure 5. Additionally, it is proven that the group of anchor has as higher load-carrying capacity with respect the estimated one, barely evaluated considering the summation of the capacity of a single fastener extended to the whole number of the screws, in this case the number is equal to 53.

3.1.2. Cyclic tests

For the sake of synthesis, here the results of cyclic tests are not reported. However, it can be stated that in terms of load-displacement behavior, the curve follows the envelop of the monotonic test. The mode of failure obtained, exactly replicate the ones obtained during the corresponding monotonic test.

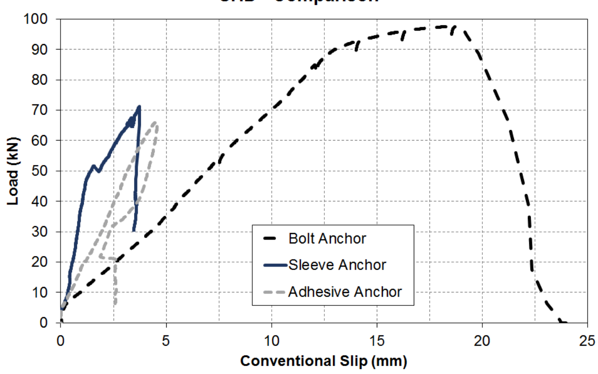


Figure 4 – Results for the monotonic tests in case of as-built geometry

Figure 5 – Mode of failure in case of As-built geometry with anchor fastened to the strong floor.


Figure 6 – Mode of failure in case of As-built geometry with bonded anchors.

Figure 7 – Mode of failure in case of As-built geometry with sleeve anchor anchors.

3.2. Optimized geometry

3.2.1. Monotonic test

The optimized prototypes are characterized by three different levels of the peak loads, i.e. 66 kN, 31 kN and 20 kN for the OPT-640, OPT-540, OPT-440, respectively. The load-displacement curves, differentiating the two sides of the test configurations, are reported in Figure 8. For all the cases, the load-displacement behavior shows (i) a well discernible yielding point and (ii) e pronounced plastic plateau with hardening. Notwithstanding the non-symmetric behavior that occurs once one side reaches first the failure, the yielding of the both sides has been observed in all the cases except for OPT-620.

In all the tests, the break-out of the steel flange (at least in one side of the tested double configuration) has been reached as shown in Figure 9. Moreover, both the hold-down (placed in different side of the test configuration) are characterized by a non-recovered deformation within the stretch length

At the end of the test there was neither the evidence of damage of the anchors to the concrete block nor the occurrence of plastic hinges of the screws. The abovementioned evidences confirm the reliability of the test configuration having two hold-down installed symmetrically.

It is worth mentioning that the difference is negligible, between the displacement measured in the steel flange (at the level of the first row of the screws) and the associated one in the timber. Additionally, the anchor head displacement has been measured for OPT-620 (during the cyclic test) via dial gauge showing a value of 0.7 mm at the point of the maximum load, when the steel displacement reaches a value almost equal to 15 mm.

The lever arm coefficient (k_t) is evaluated directly form experimental results. Referring to Table 2, the load is measured both (i) in series of the hydraulic actuator and (ii) at the anchorage level using a washer load cell. As expected, the value of the tensile force for the anchor overcomes the load applied (half of the load applied by the actuator) due to the lever arm effect. The k_t coefficient is evaluated by means of the following equation:

$$k_t = (F_{max}^{anch} - F_{torque}) / F_{max}^{Cylinder} \quad (3)$$

Where: (F_{max}^{anchor}) is the maximum force during the monotonic test measured by the washer cell; (F_{max}^{cyli}) is the actuator force [considering one half for the symmetry]; (F_{torque}) is the pre-load applied to the anchor.

The experimentally - based evaluation is made at the point of yielding of the steel flange which corresponds to the occurrence of the maximum load. Three different values (evaluated in case of OPT-620 and OPT-540) are considered and a final average is made. Finally, the value characterizing the connection is 1.25.

The value of k_t experimentally based should be more reliable with respect simplified analytical models. In those cases, the internal lever arm takes place between the anchor force and the compressive reaction given by the contact of the horizontal plate and the concrete base. The amount of the compressive force depends on how the contact is developed. It is commonly assumed that the distribution of stresses is linear. Notwithstanding the non-linear deformed shape has been observed during the execution of the test.

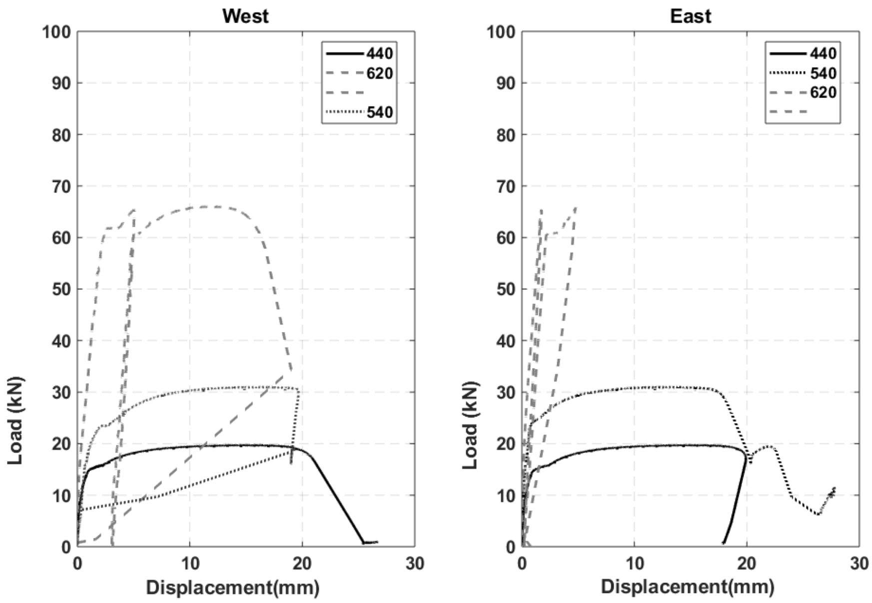


Figure 8 – Results for the monotonic tests in case of optimized geometry

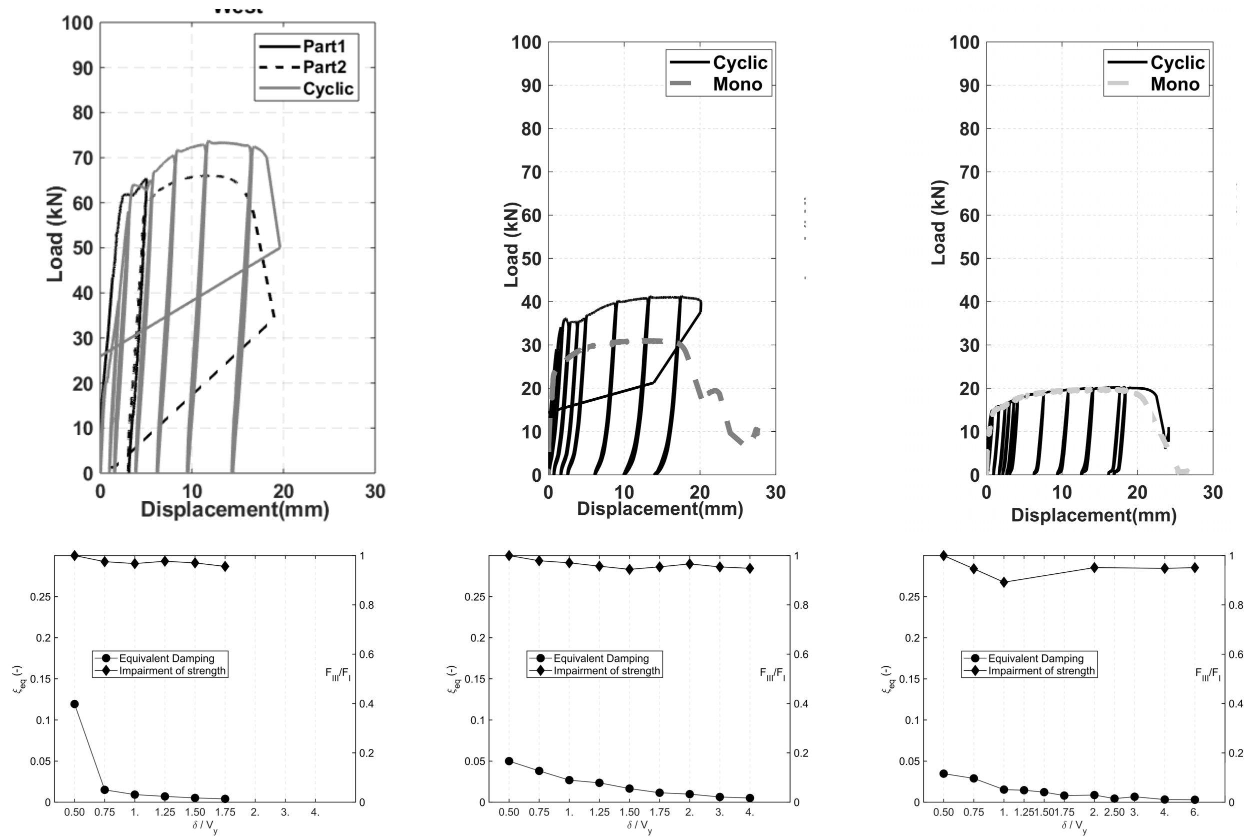
OPT – 620 C OPT – 540 C OPT – 440 C

Figure 9 – Mode of failure in the case of the optimized geometry for the different hold-down prototypes

3.2.2. Cyclic test

The results for cyclic tests are shown in Figure 10, for OPT-440, OPT-540 and OPT-620, respectively. For the OPT-440, at each level of the displacement, performing the cyclic protocol, the load level is comparable with the one obtained for the monotonic tests. However, the same cannot be stated for the OPT-540 and OPT-620 where higher values of the load are obtained. In all the tests, the failure mechanism confirms the one obtained in the case of the static, i.e. break-out of the steel flange.

The evaluation of the parameters for the interpretation of the cyclic behavior (impairment of strength and equivalent viscous damping), is carried out according to [16]. It is evident that the trend of strength reduction does not overcome the 10% when the ratio is calculated between the force reached at the third cycle of the target displacement (F_{III}) and (F_I) the one reached at the first cycle. The equivalent viscous damping assumes values less than 5% considering as relevant the cycles after the passing of the yielding point ($1.0 \cdot V_y$). In fact, the authors feel questionable the definition of the viscous damping for the previous cycles (as defined in [18]), in which, theoretically, the cycle are performed following linear-elastic paths.



OPT – 620 C OPT – 540 C OPT – 440 C

Figure 10 – Results for cyclic tests in case of optimized geometry. (Top) Load displacement curves. (Bottom) Parameters of cyclic behavior, i.e. impairment of strength [diamond mark] and equivalent hysteretic damping [round mark].

4. Discussion

4.1. Comparison between as-built and optimized geometry

Results of the monotonic tests in the case of the optimized geometry, clearly show that ductile steel failure is reached. The failure, as expected, concentrates in the reduced area region of the hold-down's vertical flange. The values of the peak loads are comparable with ones (F_{uk}) obtained through analytical evaluation as shown in Table 2. In case when the optimized geometry is adopted, the failure of the anchor has been prevented and there is no evidence of surface crack pattern for the concrete slab.

4.2. Mechanical behavior with the optimized geometry

The displacement has been measured at the level of the 1st row of the screws which might be the most loaded when the steel-to-timber connection behaves as “long-joint” [20]. Negligible differences between the displacement of the steel flange and the correspondent one measured on the timber have been observed. The implication of this is twofold: (i) the mechanism of load-transfer involving laterally loaded screws is stiff enough and does not contributes to the dissipative performance of the whole connection; (ii) the steel elongation within the “stretch length” (zone of the vertical flange characterized by the reduced cross-section) can be considered as the kinematic variable (slip) of the whole connection when applying the definition stated in [16]. Same considerations apply also when the anchor’s displacement is compared.

Cyclic performances have been evaluated according to [16]. For the same displacement level, the load decreases of the 5% at most when compared to the load level reached at the 1st cycle. This evidence, as expected, confirms the maintenance of the strength characteristic during cycles providing that (i) the number of cycles is not relevant to start fatigue issues and (ii) reverse of the load is prevented and no limitation should be adopted to contrast the Bauschinger effect [21].

5. Conclusions

A new type of hold-down connection with optimized cross-section of the vertical flange has been tested against monotonic and cyclic loading in comparison with the performance of classical hold-down configuration (as-built with regular cross-section of the vertical flange). The cross section has been designed in order to promote steel failure of the vertical flange. The optimized prototypes have been defined using design inequalities, thus considering the presence in the formulas of both (i) material partial safety factors and (ii) overstrength factors. Based on the experimental evidences, the following conclusions can be drawn:

1. In the case when the as-built geometry has been used, anchor failure has been reached resulting in the occurrence of brittle mechanisms, i.e. concrete cone failure and combined pullout and concrete cone. Steel failure has been reached in all the cases when the optimized geometry is adopted confirming the validity of the design assumptions.

2. The dissipative performance of the connection (in the optimized configuration) are characterized by the elongation of the steel which takes place within the stretch length. A target level of 15-20 mm of displacement at the ULS can be considered.

3. The slip occurring between the steel plate and the timber post has negligible effect in the performance of the connection. The same consideration can be inferred for the anchor’s displacement. This evidence is consequent to the application of the design rules. In fact, even if based on force inequalities, the overstrength factors (considering both steel-to-timber and steel-to-concrete) provides an associated margin to the displacement, providing that, below certain load threshold, the mechanical behavior remains in the elastic regime.

4. The behavior during cyclic test shows no significant strength degradation and values of the equivalent viscous damping comparable with past results which can be retrieve from literature.

6. Acknowledgments

The authors wish to extend their gratitude to Friulsider S.P.A. for the financial support. Special thanks should be conveyed to Daniele Spinelli from the Testing Lab. for Materials, Buildings and Civil Structures of the Politecnico di Milano in setting up the test apparatus.

7. References

[1] Sartori, T. and Tomasi, R. (2013): Experimental investigation on sheathing-to-framing connections in wood shear walls: *Eng. Struct.*: vol. 56: pp. 2197–2205.

[2] Liu, J. and Lam, F. (2019): Experimental test of coupling effect on CLT hold-down connections: *Eng. Struct.*: vol. 178, no. September 2018: pp. 586–602.

[3] Tomasi, R. and Sartori, T. (2013): Mechanical behaviour of connections between wood framed shear walls and foundations under monotonic and cyclic load: *Constr. Build. Mater.*: vol. 44: pp. 682–690.

[4] Pozza, L., Ferracuti, B., Massari, M., and Savoia, M. (2018): Axial – Shear interaction on CLT hold-down connections – Experimental investigation: *Eng. Struct.*: vol. 160, no. January 2017: pp. 95–110.

[5] Latour, M. and Rizzano, G. (2017): Seismic behavior of cross-laminated timber panel buildings equipped with traditional and innovative connectors: *Arch. Civ. Mech. Eng.*: vol. 17, no. 2: pp. 382–399.

[6] Gattesco, N. and Boem, I. (2016): Stress distribution among sheathing-to-frame nails of timber shear walls related to different base connections: Experimental tests and numerical modelling: *Constr. Build. Mater.*: vol. 122: pp. 149–162.

[7] Hashemi, A. and Quenneville, P. (2020): Large-scale testing of low damage rocking Cross Laminated Timber (CLT) wall panels with friction dampers: *Eng. Struct.*: vol. 206, no. January: p. 110166.

[8] Hashemi, A., Zarnani, P., and Quenneville, P. (2019): Development of Resilient Seismic Solutions for Timber Structures in New Zealand Using Innovative Connections: *Struct. Eng. Int.*: vol. 0, no. 0: pp. 1–8.

[9] Vassallo, D., Follesa, M., and Fragiacomo, M. (2018): Seismic design of a six-storey CLT building in Italy: *Eng. Struct.*: vol. 175, no. May 2017: pp. 322–338.

[10] Tamagnone, G., Rinaldin, G., and Fragiacomo, M. (2018): A novel method for non-linear design of CLT wall systems: *Eng. Struct.*: vol. 167: pp. 760–771.

[11] Lukacs, I., Björnfot, A., and Tomasi, R. (2019): Strength and stiffness of cross-laminated timber (CLT) shear walls: State-of-the-art of analytical approaches: *Eng. Struct.*: vol. 178, no. October 2018: pp. 136–147.

[12] Shahnewaz, M., Popovski, M., and Tannert, T. (2019): Resistance of Cross-Laminated Timber Shear Walls for Platform-Type Construction: *J. Struct. Eng. (United States)*: vol. 145, no. 12.

[13] EC2 (2019) , EN 1992-4 - Design Of Concrete Strucutres: Design Of Fastenings For Use In Concrete, vol. 250. .

[14] EC5 (2004) , EN 1995-1-1 Design of timber structures: General - Common Rules And Rules For Buildings, vol. BS EN 1995, no. 1. BSI.

[15] EC8 (2004) , Eurocode 8: Design of Structures for earthquake resistance, vol. 3, no. January 2011. .

[16] BritishStandard (2000) , BS EN 12512 Timber structures - Test methods - Joints made with punched metal plate, vol. 3, no. 1. .

[17] Gavric, I. (2011), Seismic Behavior of Cross-Laminated Timber Buildings, Università Degli Studi di Trieste.

[18] Jorissen, A. and Fragiacomo, M. (2011): General notes on ductility in timber structures: *Eng. Struct.*: vol. 33, no. 11: pp. 2987–2997.

[19] Eligehausen, R., Mallèe, R., and Silva, J. F. (2006), *Anchors In Concrete Structures*. Ernst&Shon.

[20] CEN (European Commitee for Standardization) (2005), *EN 1993-1-8 - Design of Steel Structures - Joints*. .

[21] Lemaitre, J. and Chaboche, J.-L. (1992), *Mechanics of solid materials*, vol. 19, no. 1. Cambridge: Cambridge University Press.

