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AN INNOVATIVE FOUNDATION SYSTEM FOR TIMBER BUILDINGS: STEEL WELDEDMESH GABION BOXES INFILLED WITH AGGREGATES

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ABSTRACT: An adequate foundation system for timber buildings has to guarantee both load bearing capacity and durability of the wooden elements, providing a proper and efficient separation layer between the ground and the upper structure. Commonly, practitioners use reinforced concrete slab or beams as structural foundation system. Usually concrete foundation is casted directly into the ground providing an adequate structural solution but, at the same time, this traditional technique is in contrast with other essential demands of timber constructions as prefabrication, lightness and low environmental impact. This paper presents an innovative prefabricated foundation system using steel welded mesh gabion boxes filled with stone aggregates developed to be removed at the end of life of timber structure. The results of the experimental tests, conducted to characterize the mechanical behaviour of the foundation system, are presented in this paper.

KEYWORDS: removable foundation system, timber buildings, prefabrication, experimental tests

1 – INTRODUCTION

The rapidly increase of timber buildings demand observed in the last decades and the further growth expected in next years [1], mainly due to the mechanical/physical properties, good heat preservation, renewability and lightness of wood raw material, encouraged companies and designers to develop new wooden-based construction products and specific technologies, oriented in particular to environmental sustainability [2] and Design for Disassembly (DfD) [3]. The novelty of timber as a construction material for the whole buildings (not just for the roof) and lacks of knowledge has led designers and companies to some issues, mainly related to durability [4]. Nowadays the role of designers has become crucial to ensure an adequate service-life of the structures [5]. Several examples of structural details are reported in handbooks (e.g. Canadian CLT Handbook [6]) in order to avoid issues related to an excessive amount of moisture which led to a decreasing of mechanical properties of timber and connections [7]. Engineering wooden-systems with high level of prefabrication, especially Cross Laminated Timber (CLT) and Light Frame Timber (LFT), reduce costs and time spent building new structures, Researchers and companies proposed improvements of these techniques towards environmental sustainability that have led to develop mortar-wood or only-wood systems ([8],[9]) in order to avoid using glue in engineering wood. Despite this environmental-oriented trend, some structural details seem at odds with the aim of having a sustainable structure: as an example, the traditional foundation system for timber buildings consist of a thick reinforced concrete slab (often over-designed) casted directly into the ground, characterized by a huge environmental impact in terms of carbon emissions and non-reversible (non-removable) solution. Moreover, in order to protect the base of the timber upper-structure from water directly contact and capillary raise, an additional concrete curb element (150-350mm high) is often built above the concrete slab. Concrete foundations offer a very high stiffness compared to the timber upper-structure, allowing to disregard the differential

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settlements of foundation (low values) and simplifies the design phase. Conversely, the concrete curbs solution is affected by some technical issues, observed on construction site, in particular related to the not accurate casting and consequently non alignment of the wall, levelling phase of the timber structure and connections anchoring. Moreover, removing the reinforcement concrete slab at the end of service-life of the building is difficult. Several companies proposed different systems in order to solve some of the main issues of the concrete curbs solution i.e. through aluminium/steel curbs ([10],[11],[12]) or prefabricated concrete curbs [13] post-installed on site above the concrete slab. The proposed system, named Ledro Steel Foundation (LSF), is based on steel welded mesh gabion boxes infilled with stone aggregates typically use for retaining walls [14]. The proposed foundation system was developed for low-rise timber buildings and it is aimed to provide a load resisting (static and seismic), removable and environmentally sustainable alternative system to traditional reinforced concrete slab foundation. Several issues observed in timber building due to the traditional foundation solution can be mitigated or solved adopting the LSF system. This paper presents the outcomes of an experimental campaign conducted at the Mechanical Testing Laboratory (LPM) of National Research Council of Italy – Institute for BioEconomy (San Michele all'Adige, Italy).

2 – LSF SYSTEM DESCRIPTION

The LSF system is based on a welded mesh gabion box filled with stone aggregates, the dimensions are: 1000mm width (w) x 2000mm length (l) x 500mm height (h). The box is closed at the top by a precast reinforced concrete C45/50 element 500mm (w) x 1500mm (l) x 150mm (h); the top welded mesh is a part of reinforcement and it is fixed to the other panels via metal C-rings connectors, as shown in Fig. 1 (left). Four steel rebars (d=12mm) were applied to the gabion box in order to anchor the precast concrete element (RC beam) to the bottom welded mesh panel through M12 nut and steel plate (at the top) and a hook (at the bottom). At the centre of each precast concrete element there is a hole with a diameter equal to 100mm. A 4.8 class M20 threaded rod (named "hinge connection" because it is able to transfer only lateral and axial forces), welded to a steel plate, was positioned according to Fig. 1. The portion of the rod which protrudes from the hole is used to level (nut and locknut) and anchor the capping beam (the element that connect the different welded mesh gabion boxes on the same alignment). Designer can choose to use different material capping beam: steel, reinforced concrete or timber. Moreover, the hinge connection enables adjusting the in-plane position up to ±2cm. Then, the hole is filled with gap-filling mortar, in order to create a discrete support. All the capping beams have to be connected as showed in Fig. 2 in order to create a continuous alignment of the walls of timber structures. The orthogonal beams have to be connected as reported in Fig. 3. When all the welded mesh gabion boxes are connected one to each other by the capping beams, the upper timber structure can be fixed by metal plates to the LSF system using nails/screws for timber elements while bolts and chemical/mechanical anchors for steel and reinforced concrete, respectively. The durability of the LSF system steel elements (under the ground) was tested to guarantee at least 50 years of service life [15]. Metallurgica Ledrense Cooperative Company applied for an European Assessment Document (EAD) to certificate LSF system as a construction product.

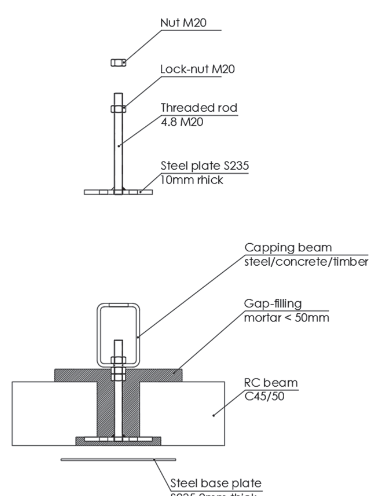

Figure 1. The welded mesh gabion box filled with aggregates (left) and the hinge connection (right)

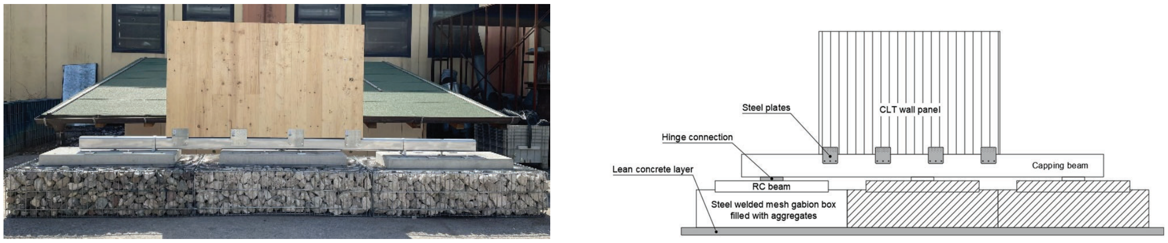

Figure 2. Single alignment of the LSF system (left) and its schematization (right)

Figure 3. Example of connection between orthogonal capping beams (left) and LSF system (right)

3 – EXPERIMENTAL SETUP

Two different typologies of test were performed at Mechanical Testing Laboratory (LPM) of National Research Council of Italy – Institute for BioEconomy (San Michele all'Adige, Italy): test on hinge connection and on complete LSF system.

3.1 HINGE CONNECTION SETUP

Nine monotonic tests were carried out on the hinge connection (Fig. 4), according to the indication reported in EN26891:1991 [16], in order to investigate its mechanical behaviour. The displacement rate was not greater than 0,2 mm/s and the maximum load was reached in 300 ± 120 s in all tests. In the hinge connection system tests, only the central part of precast reinforced concrete element (at hinge position) was tested. The dimension of specimens is 500mm (w) x 500mm (l) x 150mm (h). The thickness of gap-filling mortar layer is equal to 40mm. Three different configurations were tested (three test for each configuration): compression, tensile and shear (Fig. 4). Four LVTDs were applied to the specimens according to Fig. 4 and Table 1.

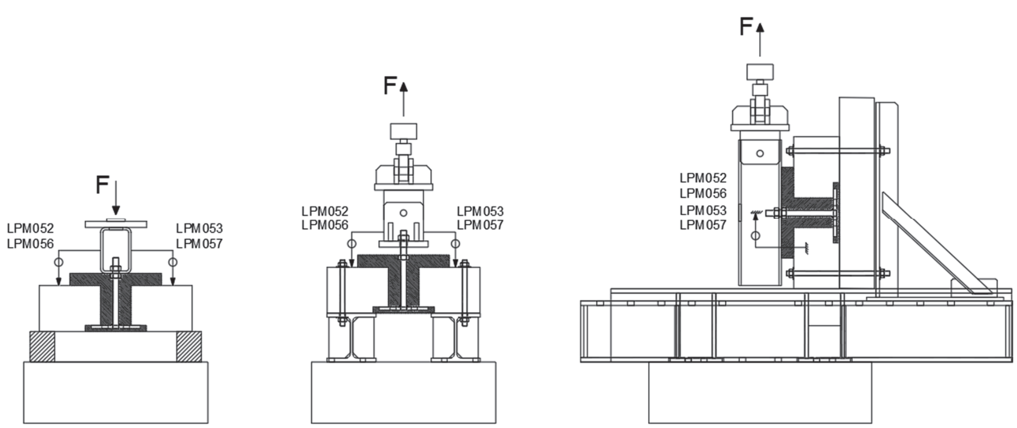

Figure 4. Hinge connection test setup: compression (left) tensile (centre) and shear (right)

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Table 1: Transducers applied to hinge connection specimens

ID	Measure	Range
LPM052	Vertical displacement South-East	±50mm
LPM053	Vertical displacement South-West	±50mm
LPM056	Vertical displacement North-East	±50mm
LPM057	Vertical displacement North-West	±50mm

In compression and shear configurations a S355 rectangular cross-section 100mm (w) x 150mm (h) and 10mm thick was connected to the specimen in order to replicate the behaviour of the capping beam, while in tensile configuration the load was applied directly to the threaded rod. The specimens in tensile and shear configuration were fixed to the testing machine through four passing through thread rods inserted in the holes of the RC beam, while in compression tests the specimen was simply supported by two steel elements.

3.2 LSF SYSTEM SETUP

Three monotonic tests were carried out on the LSF system specimens (Fig. 5), according to some indications reported in EN26891:1991 [16] and EN594:2011 [17], in order to characterized its mechanical behaviour. The configuration of LSF system is reported in Fig. 5. Tests were carried out controlling the horizontal displacement of the top of the panel with actuator speed not greater than 0,2 mm/s. The maximum load was reached in 300 ± 120 s in all tests and the tests were stopped at 100 mm of horizontal top displacement. The system is made up of a 5-layer (100mm thick) CLT wall panel [18], a capping beam 100mm (w) x 150mm (h) and 10mm thick rectangular cross-section S355, three aligned LSF elements laid on a lean concrete layer. Four steel plates (3mm thick) with thirty 5x60mm screws [19] were used in steel plate-to-timber connection while two 8.8 M12 bolts were adopted in steel plate-to-capping beam connection for each steel plate. The capping beam was levelled and then was fixed to the LSF element through a gap-filling mortar casting around the threaded rod protruding from the RC beam (Fig. 2) of each LSF element. The specimen was placed in the steel frame test apparatus of LPM according to Fig. 5. A monotonic horizontal load was applied through the ±400 kN hydraulic jack at the upper corner of the CLT wall panel. No vertical load was applied during the test. The LSF element number 1, according to Fig. 5, was restrained through a 100mm (w) x 150mm (h) and 10mm thick rectangular cross-section S355 that does not allow horizontal displacements at the base corner of the element. The lateral restrain was placed in order to avoid simultaneous sliding of the three LSF elements. In LSF system tests nine LVDTs were applied to the specimen according to Fig. 5 in order to measure horizontal and vertical displacements of each selected element (Table 2).

Table 2: Transducers applied to LSF system specimens

ID	Measure	Range
LPM052	Horizontal displacement at the base of LSF element number 1	±50mm
LPM053	Horizontal displacement at the top of LSF element number 1	±50mm
LPM054	Relative horizontal displacement between capping beam and RC beam	±100mm
LPM055	Vertical displacement of LSF element number 2	±100mm
LPM056	Vertical displacement of capping beam	±50mm
LPM057	Horizontal displacement at the top of CLT wall panel	±50mm
LPM058	Vertical displacement of LSF element number 3	±100mm
LFV030	Relative horizontal displacement between the base of CLT wall panel and capping beam	±100mm
LFV033	Vertical displacement (uplift) of CLT wall panel	±100mm

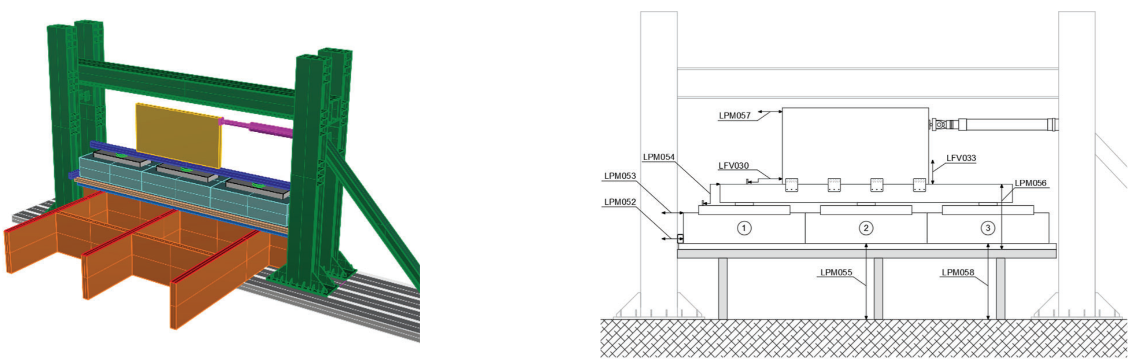

Figure 5. LSF system setup (left) and transducers (right)

4 – RESULTS

Results are reported for the two typologies of test setup: hinge connection and LSF system.

4.1 HINGE CONNECTION RESULTS

Three tests for each configuration were carried out in compression (C), tensile (T) and shear (V). The ID number of the tested specimens, mode of failure, stiffness k (according to EN26891:1991 [16]), maximum load at 15mm of displacement F_{15}, maximum load F_{max} and corresponding displacement δ_{max} (obtained from the mean values of the four LVDTs in Table 1) were reported in Table 3.

Table 3: Hinge connection tested specimens' mechanical properties

ID	Conf.	MoF	k	F_{15}	F_{max}	δ_{max}
	[-]	*[-]*	*kN/ mm*	*kN*	*kN*	*mm*
LPM 001	C	No failure	-	-	225,9	0,9
LPM 002	C	No failure	-	-	227,9	0,6
LPM 003	C	No failure	-	-	227,2	1,3
LPM 004	T	Bending	41,7	-	69,8	5,6
LPM 005	T	Bending	53,2	-	88,4	6,1
LPM 006	T	Bending	40,5	-	91,7	7,6
LPM 007	V	Displ. >15mm	13,0	79,8	83,0	18,5
LPM 008	V	Displ. >15mm	15,7	78,4	95,0	28,1
LPM 009	V	Displ. >15mm	8,6	80,0	94,1	21,6

Specimens tested in compression were not failed during the experimental campaign. The tests were stopped just before the limit load of the testing machine (250kN). After testing no cracks were observed in the specimens (Fig. 6). The expected failure mode was the punching of the RC beam. Bending failure occurs in specimens tested in tensile configuration, large cracks formation was observed in RC beam (Fig. 6). In all tests, tensile load was transferred by the hinge connection without relative sliding between the gap-filling mortar and the RC beam and was greater than the expected self-weight of the LSF element equal to 18,82 kN (see Table 3). The shear tests were interrupted due to large deformations (>15mm); no cracks were observed either in RC beam and gap-filling mortar layer (Fig. 6). According to EN26891:1991 [16], the stiffness was calculated taking into account the maximum load reached during the test before 15mm of displacement.

4.2 LSF SYSTEM RESULTS

The results obtained from experimental campaign are reported in Table 4 and 5 in terms of failure mode, stiffness k (according to some indications of EN594:2011 [17]), maximum load F_{max} and corresponding displacements $\delta_{LPM057,max}$ and $\delta_{LFV030,max}$ (at the top and the base of CLT wall panel, respectively), $\delta_{LFV033,max}$ (uplift of CLT wall panel), $\delta_{LPM054,max}$ (at capping beam), $\delta_{LPM053,max}$ (at the top of LSF element 1), $\delta_{LPM052,max}$ (at the base of LSF element 1). An initial sliding at the base (measured by LPM052) was registered for all tested specimens. The sliding occurred due to: 1) non-perfect contact between the corner of LSF element 1 and lateral restrain (rectangular cross-section) and 2) the local deformation of the welded mesh gabion box and aggregates at the contact point.

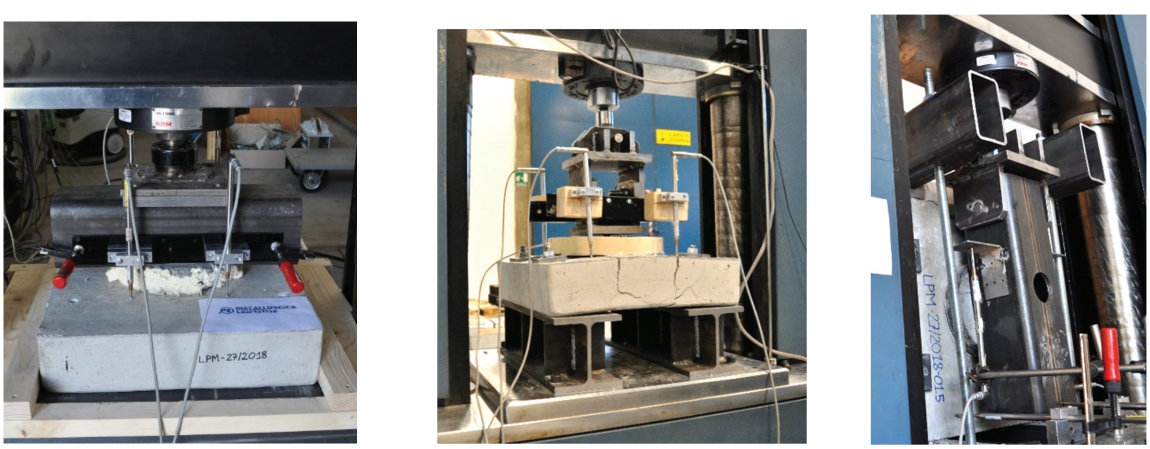

Figure 6. Hinge connection at the end of the test: compression (left) tensile (centre) and shear (right)

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At the end of the test all the specimens reached 100mm of displacement ($\delta_{LPM057,max}$) at the top of CLT wall panel according to [16] and showed the same failure mode related to a rigid body rotation mechanism (Fig. 7): LSF elements 2 and 3 were not in contact with the lean concrete layer (uplift) and the LSF system rotated around the lateral restrain. All the shear force was transferred from the upper structure to the lateral restrain through the hinge of LSF element 1.

Table 4: LSF system tested specimens' - results (mechanical properties)

ID	MoF	k	F_{max}	δ_{max}	$\delta_{LPM057,max}$
	[-]	*kN/mm*	*kN*	*mm*	*mm*
LPM 010	Rigid body rotation	1,7	88,6	0,9	100,0
LPM 011	Rigid body rotation	1,9	86,4	0,6	99,8
LPM 012	Rigid body rotation and hinge failure	2,6	84,7	1,3	100,0

Table 5: LSF system tested specimens' - results (displacements)

ID	$\delta_{LFV030,max}$	$\delta_{LFV033,max}$	$\delta_{LPM054,max}$	$\delta_{LPM053,max}$	$\delta_{LPM052,max}$
	[-]	*mm*	*mm*	*mm*	*mm*
LPM 010	Rigid body rotation	4,2	6,5	74,9	68,4
LPM 011	No failure	4,7	7,4	67,9	45,5
LPM 012	No failure	3,8	8,0	63,0	38,1

In specimen LPM-012, at the end of the test, a local failure of LSF element 1 hinge was observed: the gap-filling mortar results cracked (Fig.7). The values of base shear in full-scale tests are similar to the resistances obtained at connection level tests. In LSF system tests the failure mode involved the entire specimen: when the over-turning horizontal force applied to the CLT wall panel reached the maximum value (F_{max}) and, consequently, equilibrate the corresponding stabilizing moment due to the self-weight of the elements, the specimen started to rotate without load increasing.

The maximum horizontal load $F_{max,CM}$ can be estimated via calculation method (CM) through the rotational equilibrium around lateral restrain according to (1):

$$F_{max,CM} \cdot h = \sum_{i=1}^{3} b_i \cdot W + b_{CP} \cdot W_{CP} + b_{CLT} \cdot W_{CLT} \qquad (1)$$

where h is the distance (vertical direction) from the lean concrete layer to the point where the horizontal load is applied, W and b_i are the mean self-weight (18,82 kN) and the distance between the centre of mass of each LSF element and the lateral restrain, respectively, while W_{CP} and b_{CP} are the self-weight (1,70 kN) and the distance between the centre of mass of capping beam and the lateral restrain. W_{CLT} and b_{CLT} are the self-weight (1,32 kN) taking into account the mean density reported in [18]) and the distance between the centre of mass of CLT wall panel and the lateral restrain. A comparison between the experimental maximum lateral load F_{max} and values obtained via calculation method $F_{max,CM}$ is reported in Table 6 in terms of discrepancy ε.

Table 6: Comparison between experimental and analytical values

ID	h	$F_{max,CM}$	$F_{max,}$	ε
	mm	*kN*	*kN*	*[%]*
LPM 010	1945	91,7	88,6	3,5
LPM 011			86,4	6,2
LPM 012			84,7	8,3

Figure 7. LSF system failure modes: hinge connection failure (left) and rigid body rotation (right)

5 – CONCLUSION

The paper presents an innovative foundation system consisting of welded mesh gabion boxes intended to guarantee the durability of timber structures and a fast, safe and simple assembling on-site. The LSF system is completely removable at the end of life of the upper timber structure and represents an alternative to traditional reinforced concrete slab foundation. Moreover, it can mitigate the risks of water capillary rise and excessive amount of moisture in timber elements at the base/foundation. The experimental tests showed that the proposed connection system (“hinge connection”), between the upper structure and the foundation, is able to transfer compression, tensile and shear loads, highlight the potential of the LSF system also in seismic prone areas. The three full-scale tests ensure the mechanical properties of the entire system and confirm that it can be considered a valuable foundation system for low-rise building. In future research project, the mechanical behaviour of a single LSF element will be characterized in order to provide the characteristic (resistance) and mean (stiffness) values to designer for different combination of axial and lateral loads.

6 – ACKNOWLEDGMENTS

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