The 14th World Conference on Earthquake Engineering
October 12-17, 2008, Beijing, China

EXPERIMENTAL AND ANALYTICAL INVESTIGATIONS OF SOLID WOODEN WALL PANEL ELEMENTS SUBJECTED TO LATERAL LOADS

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ABSTRACT :

Within the bilateral project realized by the Institute of Earthquake Engineering and Engineering Seismology IZIIS, Skopje and Faculty of Civil and Geodetic Engineering, University of Ljubljana, Slovenia various configurations of cross-laminated timber wall elements, have been tested under combined constant vertical loads and cyclic horizontal loads subjected to different boundary conditions.
This paper presents the comparative numerical study of solid wooden wall panel elements' behaviour, preformed using the obtained quasi-static tests' results.
Based on the tests results special constitutive relations for connections between the panel elements and the foundation have been developed and integrated into numerical model of the whole system based on finite element method.
The results obtained from pushover, quasi-static and dynamic analyses have proved the applicability of the proposed numerical constitutive relationships in practice for prediction of the seismic behaviour of this type of massive wooden panels.

KEYWORDS: Wooden wall panels, racking strength, FEM analysis

1. INTRODUCTION

Cross laminated wooden structures are becoming very popular product in Europe. Wooden structures have always had a good reputation when subjected to seismic events, mainly due to the wood's high strength to weight ratio and its enhanced strength under short term loading. On the other hand cross lamination method has many advantages. It minimizes swelling and shrinkage in the board plane, considerably increases static strength and shape retention properties and enables load transfer across the entire plane of panel. Meaning it gives a material with high stability and good overall mechanical properties regardless of the wood quality. The connections of wooden structures are normally more ductile than the timber parts themselves which results in an overall ductile behaviour of wooden buildings and their good seismic performance. However seismic response of wooden structure is complex issue and still a lot of uncertainties exist and need to be clarified and quantified.
The objective of this study was to obtain performance characteristics of solid wooden wall panel elements subjected to lateral loads via experimental testing and analytical modelling.

2. EXPERIMENTAL WORK

At the laboratory of the Faculty of Geodetic and Civil Engineering in Ljubljana, Slovenia, various configurations of cross-laminated timber wall elements, product of Austrian company KLH Massivholz Gmbh have been tested under combined constant vertical loads and cyclic horizontal loads and different boundary conditions.

Figure 1-2 shows the two studied types of tested specimen. Type 1 is a solid KLH panel dimensions b/h/d = 320/272/9.4 cm, while Type 2 represent fenestrated wall element with same dimensions having window opening of 140/120 cm and door opening of 90/215 cm. BMF corner connectors with rib of height of 105cm, placed on every 100cm, and Wurth annularly nail 4,0/40mm with length of 40mm were used to fix the specimens to reinforced concrete beam representing the foundation (Fig. 3).

Figure.1 Type 1-fenestrated panel

Figure.2 Type 2-solid panel

Figure 3. BMF anchor

Tested specimens were loaded with constant vertical load of 15kN/m' and cyclic horizontal load following EN 12512 standard and SAC protocol and exposed to two different cases of boundary conditions: shear and rocking mechanism. Testing was preformed using the test set-up constructed at the UL FGG laboratory and specially designed for racking testing of panels under different boundary conditions and vertical loads.

3. ANALYTICAL WORK

Generally, the racking strength of the tested massive wooden wall panels depends on the material properties of the panel, the panel geometry, the level of the vertical load, and mostly on the nonlinear constitutive laws of the anchors in normal and tangential directions. The numerical simulation of the racking behaviour has been performed taking into account all of these parameters.

3.1 Constitutive Relationships for Anchors

The connection has been modelled by standard link element, with zero length and consisting of two points, each having two degrees of freedom, translations in horizontal and vertical direction (Fig. 4).

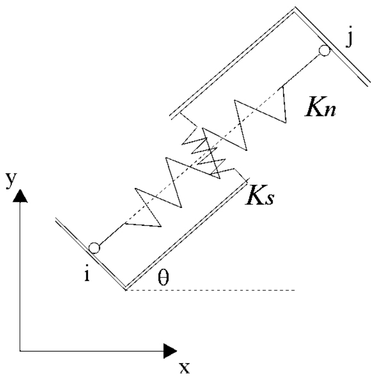

Figure 4. Formulation of the link element for connections (anchors)

Parameter K_n represents the tangent stiffness in axial direction; K_s is tangent stiffness in tangential direction. θ represents the angle of the slope of the element towards global x-axis

Figure 5-8 give schematic presentation of the constitutive relations describing the behaviour of the anchors and the contact between the wooden panel and the foundation in tangential and normal direction.

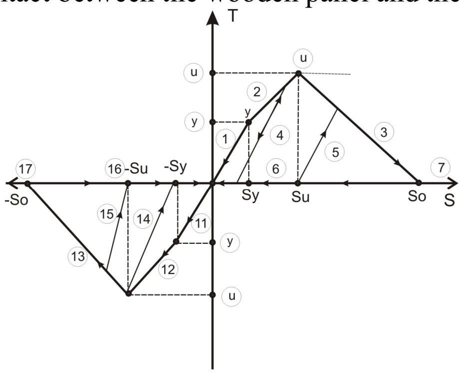

Figure 5. Constitutive relation T-s for BMF anchors

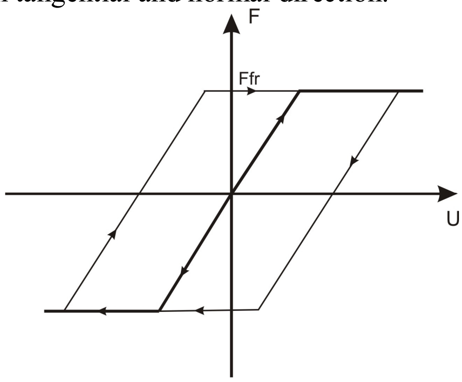

Figure 6. Simulation of the panel-foundation contact (tangential direction)

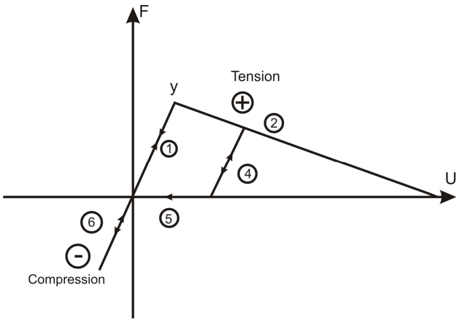

Figure 7. Constitutive relation N-s for BMF anchors

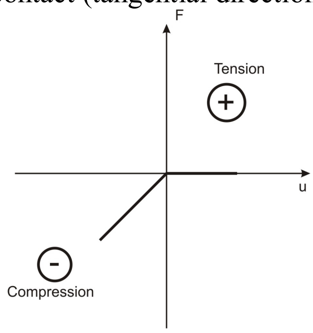

Figure 8. Simulation of the panel-foundation contact (normal direction)

In tangential direction, for shear force-slip modelling a bi-linear envelope including descending branch has been proposed in both positive and negative direction (Fig.5) and in normal direction the working diagram in tension has been approximated with elastic-softening plastic diagram (Fig.7). The contact between the panel and the foundation in tangential direction (Fig.6) has been treated as elastic - perfect plastic, demonstrating linear behaviour up to certain force value above which follows the straight line parallel to the displacement axis, while in normal direction (Fig.8) has been simulated assigning extremely high stiffness when loaded in pressure and zero stiffness when loaded in tension.

3.2 Constitutive Relations for the Timber Material

In this analysis the timber material is treated as a linear-elastic transversely isotropic material. The basic constitutive relations are:

$$\boldsymbol{\sigma} = \mathbf{D}\boldsymbol{\varepsilon} \tag{3.2.1}$$

Where D is the elastic constitutive matrix and for transversely isotropic case it can be found (with $E_1/E_2=n$ and $G_2/E_2=m$) as follows:

$$\mathbf{D} = \frac{E_2}{1 - n\nu_2^2} \begin{bmatrix} n & n\nu_2 & 0 \\ n\nu_2 & 1 & 0 \\ 0 & 0 & m\left(1 - n\nu_2^2\right) \end{bmatrix} \tag{3.2.2}$$

Where $\boldsymbol{\sigma} = \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix}$ are in-plane stresses, and $\boldsymbol{\varepsilon} = \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \\ \varepsilon_z \end{Bmatrix}$ are strains.

The proposed computational constitutive relationships have been implemented into the general-purpose software package FELISA/3M (Finite Element Inelastic Structural Analysis Program).

3.3 Model Formulation

According to the finite element modeling, the system can be treated as a continuum (wooden panel itself) with discontinuities (links). It is actually two-dimensional model (Fig.9 and 10) that comprises few types of non-linear finite elements.

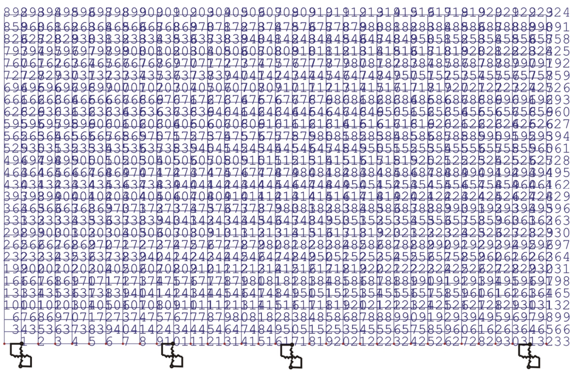

Figure 9. FE model for solid panel

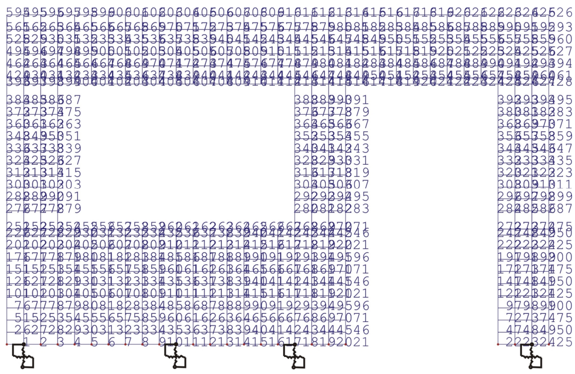

Figure 10. FE model for fenestrated panel

The tested wooden panels within this investigation have been modeled using isoparametric finite elements with four nodal points, with 2 x 2 Gausian integration scheme. For modeling of the connection between the wooden panel and the foundation non-linear link elements were applied. Standard boundary conditions have been applied: free translation without rotation was allowed for all nodal points except for the bottom nodal points of the link elements for which the translations and the rotation have been restrained.
The material model parameters were identified according to the results from the previous experimental tests carried out in the UL FGG laboratory (see approximation in Fig. 11 a and b).

For both of the models vertical load of 15kN/m’ was applied. Pushover, quasi-static and dynamic analyses have been run. For the pushover analysis displacements at both upper end nodal points of the panel in positive and negative direction have been applied. For the quasi-static analysis protocol followed during experimental testing was used and for the dynamic analysis El-centro acceleration history record was applied.

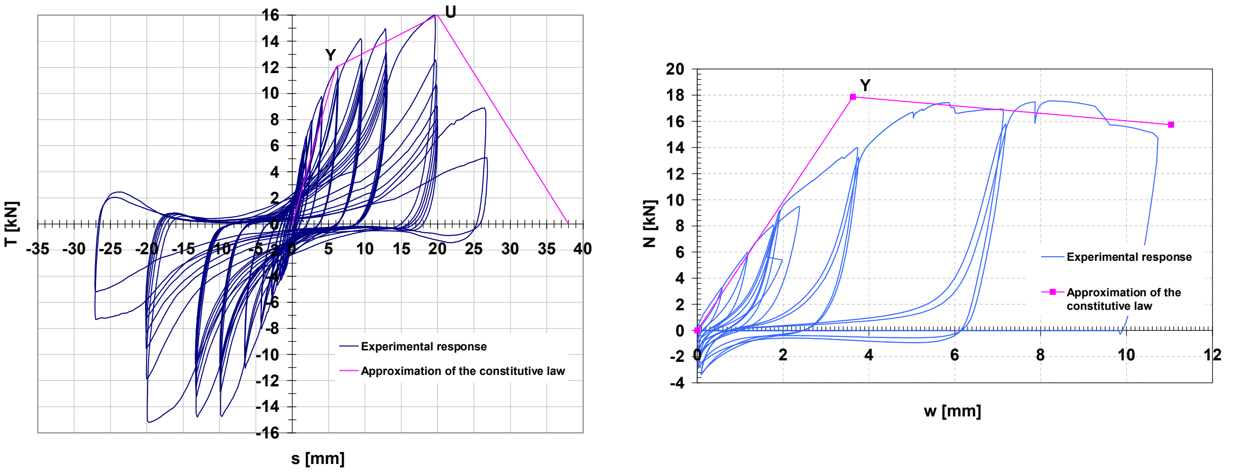

Figure 11. Approximation of the constitutive laws of BMF anchors
(a) in tangential direction by linear law (b) in normal direction by linear-softening plastic law

4. COMPARED RESULTS AND DISCUSSION

Selected results obtained from the preformed analyses are given bellow (Fig.12-13) and compared to the experimentally obtained ones (force-displacement diagrams for the displacement measured at the top of the panel element).

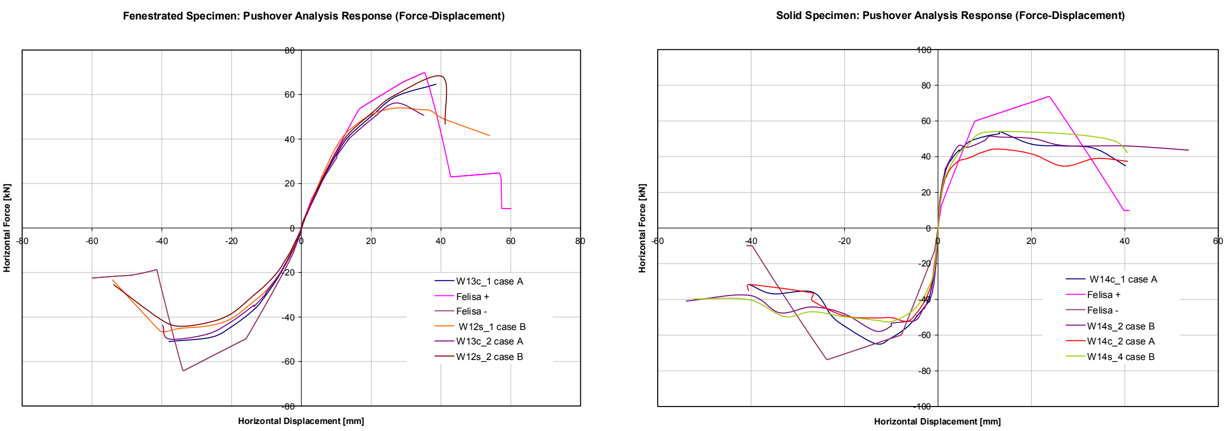

Figure 12. Comparison between finite element and experimental responses (pushover analysis)

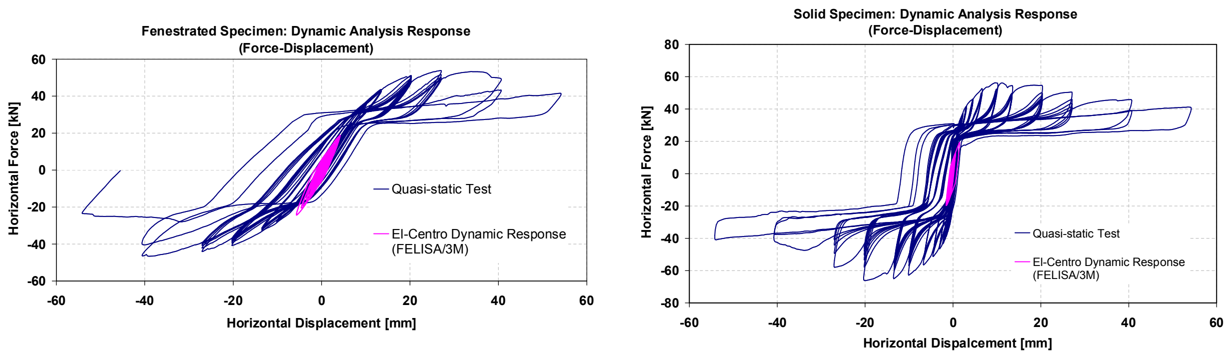

Figure 13. Comparison between finite element and experimental responses (dynamic analysis)

Generally the presented results show that the analytical model of the panels quite realistically simulates the response under static and quasi-static loads. The fitting between experimental and numerical results is very close making the analytical model applicable in practice. As presented the solid wall panel type exhibited higher discrepancy between the numerical and experimental results.

The numerical simulation showed that the main mode of behaviour of the investigated wall panels was between pure rocking and combined shear and rocking one. Especially, in anchors, shear slip capacity was first exhausted, however with still linear behaviour on tension in normal pullout direction. This is to be expected, having in mind the height/width ratio of the panel is 2.72/3.2 = 0.85 < 1, which together with the stiffer pullout behaviour of anchors (17.5 kN for 5.5 mm) compared to the more flexible shear-slip behaviour (16 kN for 20 mm) leads to the conclusion that the slip deformation will be much grater than the pullout deformation; while for the fenestrated wall panel type, also bending behaviour has been observed due to the, practically, existing column on the one side of the panel resulting in much flexible response, compared to the solid specimen.

The performed dynamic analyses using real seismic excitation in horizontal direction have shown that the assumed constitutive models can realistically simulate the dynamic response of these types of panels. Experimental results for dynamic response in this research were not available, but the numerically obtained dynamic hysteretic response in terms of base shear force (reaction) and the displacement on the panel top using El Centro acceleration history record were compared to the quasi-static p-Δ tests results, demonstrating the similar trend of response. For the original maximum peak acceleration of 0.34g, the panel wall behaviour has been almost in linear range, which can be explained by the good energy absorption of the anchor acting as a base isolation device.

5. CONCLUSION

Based on the obtained experimental results and usingFELISA/3M computer package finite element analyses of timber panels have been performed. Special numerical constitutive relations, describing the connections behaviour in normal and tangential direction, have been used. Having the obtained hysteretic responses approximate numerical models were defined taking into consideration the hardening as well as the softening behaviour. Pushover, quasi-static, and dynamic analysis have been run.
Provided results have shown good correlation with the experimental results considering the behaviour of the connections as well as the panel as a whole.
It is believed that felisa/3m has met the objectives of this study and that the developed models can be successfully applied in practice for prediction of structural behaviour of this type of structures.

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