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MULTI–SCALE MODELLING OF TIMBER–FRAME STRUCTURES UNDER SEISMIC LOADS

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Abstract. This paper introduces a versatile hysteretic constitutive law, developed for various joints with steel fasteners commonly used in timber structures (nails, screws, staples, 3D connectors of bracket type, punched plates). Compared to previous models available in literature, the proposed one improves numerical robustness and represents a step forward by taking into account the damaging process of joints with metal fasteners. Experimental tests carried out on joints are used for calibration purpose, and quasi–static and dynamic tests performed on shear walls allow validating the proposed Finite Element model. Finally, the development of a computationally efficient simplified FE model of timber–frame structures for shear walls is described, with emphasis on its validation and its use at the scale of a complete structure.

1 INTRODUCTION

The study presented in this paper is motivated by two facts. Firstly, timber–frame construction is becoming a common building system in Europe and they present many qualities, one of which being their good earthquake resistance due to the excellent strength-to-density ratio of timber and to the ductility of joints with metal fasteners, respectively leading to limited inertia forces and providing good energy dissipation. Secondly, the recent European code for design of earthquake resistant buildings is accompanied in some countries by a new seismic hazard map. In France, based on this revised map, earthquake resistance calculations are now mandatory in a much larger part of the territory. Therefore, the seismic behaviour of timber–frame structures has to be studied, in order to better understand their global and local behaviours. This study focuses on shear walls, as they contribute the most to the energy dissipation of structures.

Because nonlinear dissipative phenomena in timber–frame structures are mainly concentrated into joints, simplified force–displacement models for joints can be derived from refined analytical or FE models [3, 4, 11] or by fitting results of tests performed on joints. The proposed approach is based on a multi–scale concept, as proposed previously by different authors [13, 7, 14]. Such an approach requires a behaviour law to represent the force–displacement evolution at each scale. Numerous constitutive laws have been developed over the years, from the nonlinear laws for monotonic loads (*e.g.* [8]) to hysteretic models of various complexities [2, 6, 15, 14, 12]. A new model, developed by [10], can be considered as an improvement of Richard and Yasumura models [13, 15] and fulfils the following needs:

- The Richard's behaviour law showed that for some sets of parameters (*e.g.* for metal punched plate), an exponential function does not provide a strict analytical continuity at one end of the branch leading to numerical issues [10]. This issue is shared by all models using the exponential functions.
- The law should model asymmetric behaviour, such as those of punched metal plates for roof trusses and 3D connectors of bracket type. As far as we know, all aforementioned behaviour laws would require new developments to meet this need.
- In the aim of reliability analysis of structures, it is convenient to develop a robust model defined by means of physical parameters such as displacement, forces and stiffnesses whose variabilities can be identified. While most of the models already meet this condition, the BWBN model [14] does not.

It is important to notice that the hysteretic behaviour of nailed wood joints governs the response of many wood systems when subjected to lateral loadings, force–displacement backbone and hysteresis curves of shear walls and joints are then similar in shape. Thus, a common feature to all the previous force–displacement models is that they can be used to describe the constitutive behaviour of joints as well as the global shear wall response to lateral forces.

In this paper, a new hysteretic constitutive behaviour law for joints and timber–frame structures is proposed and its application to the modelling of OSB and Particleboard sheathed shear walls is presented. Quasi-static experimental tests on metal fasteners (nails, 3D connectors of bracket type and punched plates) are performed to calibrate their hysteretic constitutive behaviour. 14 quasi-static and 12 dynamic tests on several configurations of shear walls are carried out to validate the numerical model for shear walls. This important number of tests limits the effects of the experimental variability. Once the refined FE model is fully validated thank to experimental quasi–static and dynamic tests, a simplified FE model of shear wall is developed in order to reduce calculation costs. This simplified model is defined by a very limited number of degrees of freedom (DOF). It is calibrated on quasi–static force–displacement evolutions and used to perform dynamic calculations. Yet, the calibration is rarely validated by comparing the dynamic behaviour of the refined and simplified FE models, or by using experimental dynamic tests on a single shear wall. This validation step is emphasized in this paper. Finally, an illustration of the whole process is illustrated by the modelling of a 3D timber–frame structure.

2 FORCE–DISPLACEMENT HYSTERETIC MODEL

The one dimensional constitutive model is shown in Figure 1. The branches of the force–displacement model are grouped into two distinct categories and numbered from (0) to (5). A first group formed by branches (0) to (3) describes the behaviour under monotonic loading. The initial linear branch (0) ranges from the zero displacement up to the yield displacement d_y. The corresponding elastic stiffness is K_0. This branch is followed by branch (1), which models the non-linear phenomena in the joint up to the force peak at (d_1,F_1). After the force peak, branches (2) and (3) model up to the ultimate displacement d_u at force F_u associated to the collapse of the joint. F_u is generally chosen null to ensure a correct continuity of forces and prevent numerical issues. Therefore, 9 parameters describe the force–displacement behaviour under monotonic loading. Branch (1) is defined using a rational quadratic Bézier curve and provides a strict analytical continuity of forces.

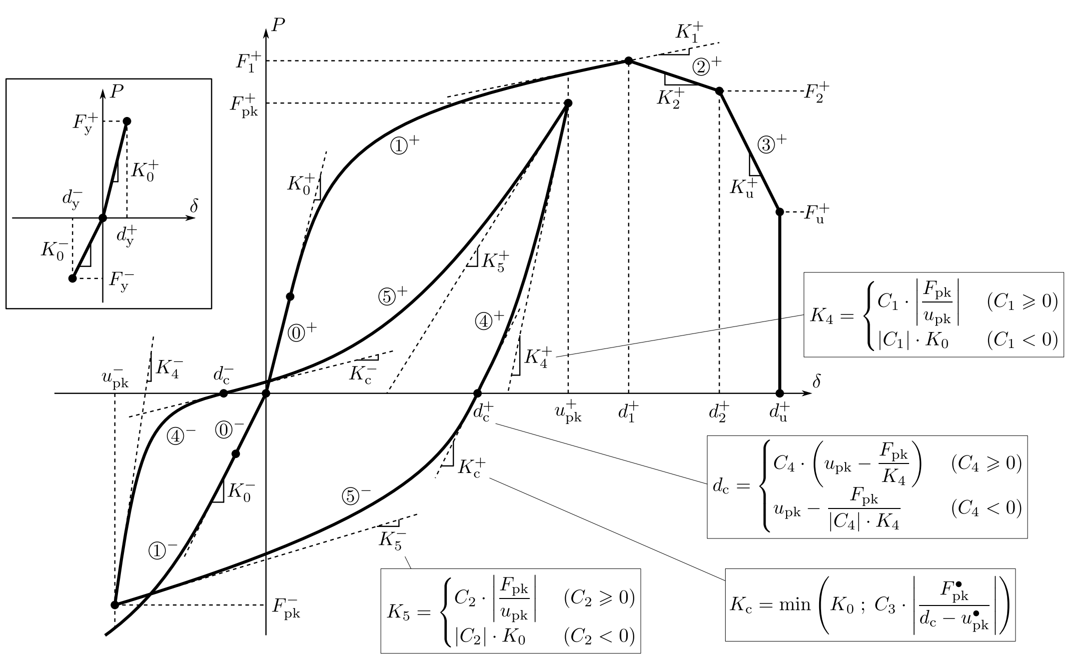


Figure 1: Proposed force–displacement model ([10])

A second group of branches describes the hysteresis loops which are typically observed when the joint undergoes a reversed loading. Starting from a previously reached loop peak (u_{pk},F_{pk}), branch (4) models the nonlinear elastic unloading down to a null force. A residual displacement $d_c \neq 0$ is commonly observed due to prior plastic deformations. The unloading stiffness K_4 is either: a) proportional to the elastic stiffness K_0 of the joint; or b) proportional to the secant stiffness F_{pk}/u_{pk} in order to model a stiffness decrease with displacements of increasing amplitude. Following this unloading, loading in the opposite direction is modelled with branch (5). The stiffness at d_c between branches (4) and (5) is denoted K_c and is used as a tangent for both branches for the sake of continuity. Branch (5) eventually reaches the previous loop peak ($u^{\bullet}_{pk}$,$F^{\bullet}_{pk}$) in the opposite loading direction. Like the unloading stiffness K_4, the reloading stiffness K_5 is proportional to the elastic stiffness K_0 or to the secant stiffness F_{pk}/u_{pk}. A second set of 4 control parameters $C_{i=1,...,4}$ governs the shape of the hysteresis loops. This allows modelling several mechanical behaviours, in particular, the thickness of the pinching area can be adjusted. Parameters C_1 and C_2 control the unloading stiffness K_4 and reloading stiffness K_5 respectively. Parameter C_3 controls the tangent stiffness K_c at location (d_c,0). Finally, parameter C_4 controls the value of the residual displacement d_c after the non-linear elastic unloading. These 4 control parameters $C_{i=1,...,4}$ depend mainly on the phenomena involved, and therefore on the configuration of the modelled system. They are constant for a given configuration.

Finally, a third set of 3 parameters controls the damage process of the model. The word *damage* refers here to the decrease of strength under cyclic. It is based on the hypothesis that the hysteresis loops are bound by the backbone curve which models the force–displacement evolution of the joint under monotonic loading. During the first loading, the peak (u_{pk},F_{pk}) is located on the backbone curve. The damage process defines the evolution of the ratio (1-D) between the "non-damaged load" F_{mono} and the "damaged load" F_{pk}. The scalar damage indicator D ranges from 0 to 1, where $D = 0$ corresponds to a non-damaged mechanical system and $D = 1$ corresponds to a fully collapsed mechanical system. D is increased of ΔD at each change of the force sign ((4) to (5) in Figure 1). To ensure the damage stabilization after a few cycles of constant amplitude as experimentally observed, an upper limit D_∞ for the displacement d_{max} is defined, using a power law (eq 1). A power term $B_r > 1$ ensures that the damage remains moderate before the force peak and becomes heavy after the peak.

$$D_\infty = B_c \, (d_{max}/d_1)^{B_r} \tag{1}$$

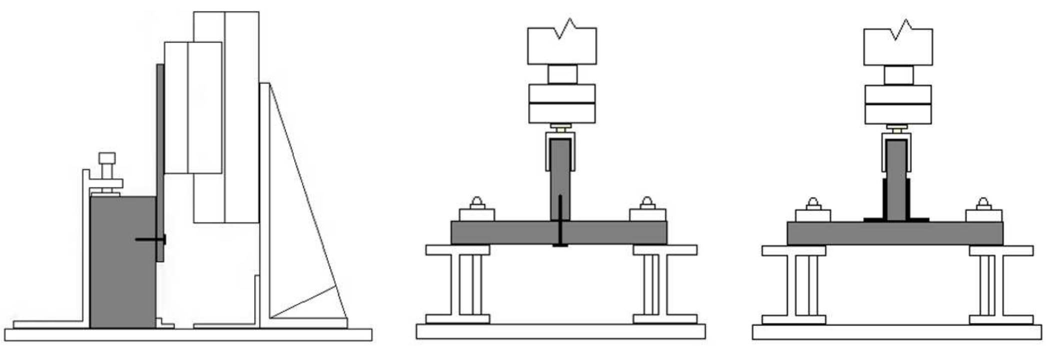
SCALE 1: JOINTS WITH METAL FASTENERS

In this section, the experimental tests carried out on joints with metal fasteners are presented. Then, the calibration process of parameters of the constitutive model of joints is described.

Experimental tests

Experimental tests on joints with metal fasteners (scale #1) are achieved to provide input data for the numerical model of shear wall (scale #2). There are three different steel joints in a shear wall: Panel-to-Frame (P2F) joint (Figure 2.a) made with nails, Frame-to-Frame Nail (F2F Nail, Figure 2.b) between the top / sill plate and the studs and Frame-to-Frame Bracket (F2F Bracket, Figure 2.c) at both ends of the shear wall, which have to be strengthened in order to prevent uplift of the exterior studs.

Three configurations are considered in this study: OSB 9 and 12 mm with 2.1x45 mm nails and Particleboard 16 mm with 2.5x50 mm nails, respectively named OSB9, OSB12 and P16. Figure 2.a shows the principle of these tests which consists in a shear test, first under a monotonic loading, then under a reversed–cyclic loading. F2F Nail joints were not tested and the results of tests achieved by [13] are used. These tests (Figure 2.b) consist in a cyclic pull–out load on a joint. The same tests are performed on F2F joints made of three–dimensional connectors of brackets type only (Figure 2.c). For each configuration, tests were repeated 2 times for monotonic loading and 5 times for reversed cyclic loading.

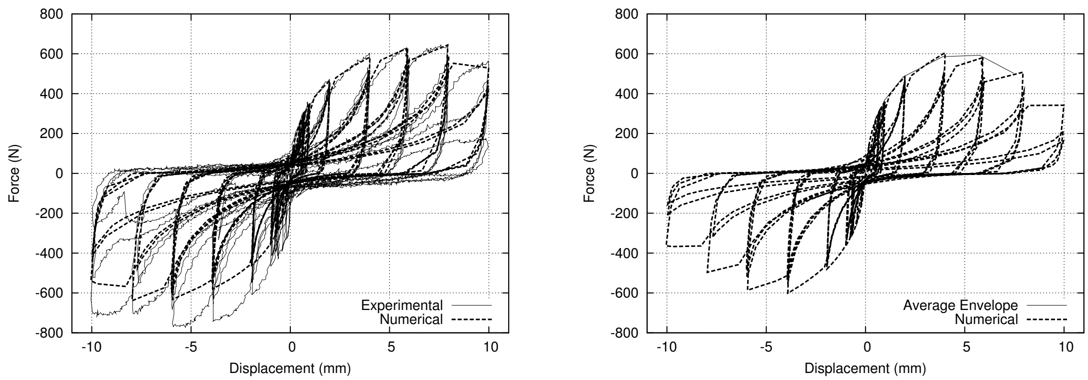


(a) Panel-To-Frame (b) Frame-To-Frame Nail (c) Frame-To-Frame 3D connector

Figure 2: Experimental tests on metal fastened joints

Calibration of the force–displacement model of joints

The results of the tests carried out on P2F nails are used to calibrate the constitutive model. Two levels of calibration are distinguished. The first level is a direct calibration which consists in reproducing one particular test. The second level is an average calibration which consists in calibrating the parameters to reproduce the average behaviour of several experiments. Figure 3.a presents a direct calibration of parameters for a P2F nail joint. It is obtained by calculating the backbone curve parameters from a single test under monotonic loading and by calibrating the pinching and damage parameters by successive simulations. Using the direct calibration as a starting point, backbone curve parameters are re–calibrated so that the simulation now reproduces the average envelope curve of all available cyclic tests. Figure 3.b presents the average envelope curve and the calibrated model. This process provides joint models to be used in the subsequent shear wall modelling.



(a) Direct calibration (b) Average calibration

Figure 3: Calibration of the hysteretic model for a 2.1 mm × 45 mm P2F nail in a 9 mm OSB panel

For F2F Nail connections, the compression (contact between the two timber elements) is linear and the stiffness is calculated according to material characteristics and the dimensions of the section in contact. The pull–out behaviour is bilinear and parameters are estimated from tests carried out by [13]. The shear behaviour is linear and symmetrical. For F2F bracket connections, experimental tests were carried out on E5® 3D connectors in shear and pull-out. That was not the case for joints made with AH 3D connectors, for which the behaviour is estimated based on the connector's dimensions and material properties and set as bilinear. Shear behaviour is linear and symmetric. The E5® pull–out behaviour parameters are calibrated with the same method as P2F connections. Note that for all joints (P2F and F2F), there is no rotational stiffness implemented.

SCALE 2: TIMBER–FRAME SHEAR WALL

In this section, the experimental tests carried out on shear walls are presented. Then, the refined FE model of shear wall is described and its prediction are confronted to experimental results.

Experimental tests

Figure 4 describes the studied shear wall. The frame is made of C24 timber and wood–based panels are nailed to the frame. In this study, OSB panels and particleboards are used. The spacing between two P2F joints along the perimeter of the panel is set to 150 mm (s_{ext}) and 300 mm (s_{int}) along intermediates studs. F2F joints are made by means of 3.1 mm × 90 mm nails. At both ends, 3D connectors of bracket type are added to prevent the uplift of the exterior studs (E5® standard or AH2950/2® reinforced connectors). Anchorage to the test machine is performed with bolts. For quasi–static tests, one push–over and two reversed–cyclic tests are achieved for each configuration tested.

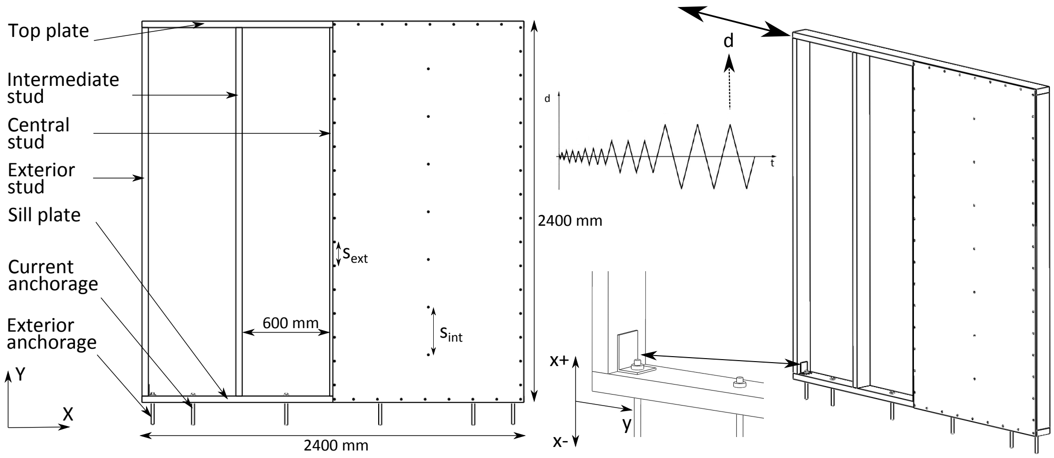


Figure 4: Shear wall description and principle of reversed cyclic tests

Dynamic tests are performed on the unidirectional shake table of the FCBA Technological Institute at Bordeaux, France. The dead load (1500 or 2000 kg) is directly attached on the top of the shear wall and the out–of–plane instability is limited by means of a frictionless guiding system quite similar to the one described in [5]. For each configuration of specimen and each mass (1500 or 2000 kg), three seismic tests were achieved with three different accelerograms. Two are natural signals (l'Aquila 2009 (GX066) and El Salvador 2001 (Zacatecoluca)) and the third is modified so that its frequency content correspond to the design spectra of the Eurocode 8. Tests are performed by repeating one accelerogram with increasing scaling ratios of the PGA.

Numerical modelling

The finite element modelling of shear walls is carried out by means of beam, plate and two–node spring–like finite elements. The constitutive behaviour presented previously is implemented into the free software Code_Aster[1]. Euler beam elements model the frame and their material properties correspond to a C24 timber. Four–node plate elements model the panels, and their material properties correspond to OSB or Particleboard panels. Each two–node spring–like element models

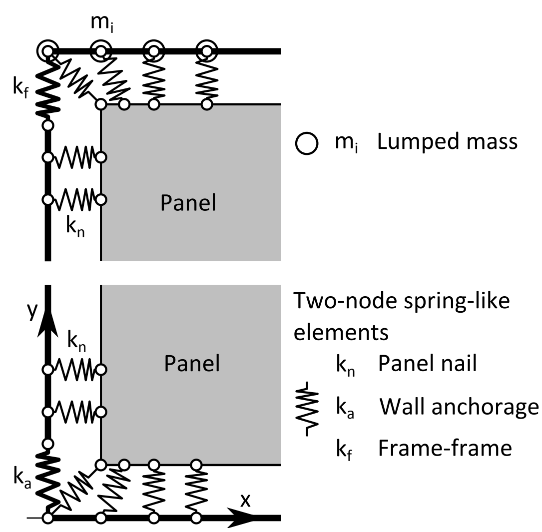


Figure 5: Refined FE model of shear wall

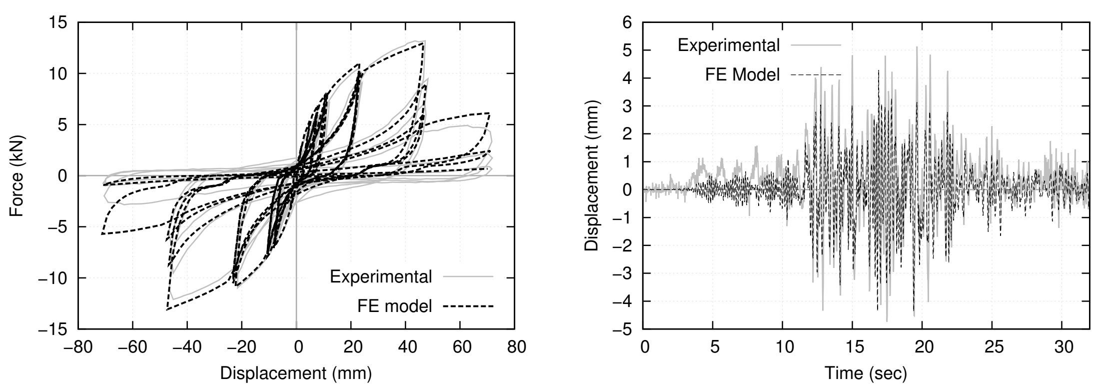
[1] All documentation is available at www.code-aster.org

a metal fastener joint whose properties are given by the previous calibration.

The sill plate is considered as embedded, since insignificant displacements are recorded in tests. For quasi–static loading, the top plate undergoes an imposed displacement. As for the tests, lumped masses are added to the frame plate in order to account for the roof weight and / or upper story (1500 or 2000 kg per shear wall). The damping matrix is build with the Rayleigh method ($C = \alpha K + \beta M$), such that α end β are equally distributed on the two first vibration modes. Experimental results showed that the global damping ratio (ξ_{glob}) is between 6 and 9 % for maximal relative displacements not exceeding 1 mm. Assuming that the energy dissipation is concentrated in the metal fasteners, a viscous damping ratio (ξ_{visc}) is identified such that the addition of viscous damping dissipation and hysteretic dissipation for all the joints allows to correctly predict the free vibration of the wall for a maximal relative displacement inferior to 1 mm. The identification led to $\xi_{visc} = 5$ %. This damping ratio is thus taken into account for each joint and for all calculations.

FE model validation

The model predictions are compared to experimental results obtained from 14 quasi–static tests and 12 dynamic ones. For quasi–static loading, the results show that the peak forces predicted by the numerical model are in good agreement with the experimental results (the average error is around 5% with a maximum of 11%). One example of predictions is presented in Figure 6.a (full results are in [1]), it can be seen that the model predictions are in good agreement with the experimental behaviour, as pinching and peak forces of the hysteresis loops are in accordance with the experimental data.



(a) Quasi–static loading (b) Dynamic loading

Figure 6: Comparisons of test results and numerical predictions

In order to complete the validation of the refined FE model, dynamic calculations are carried out and results are compared to the experimental dynamic tests. Figure 6.b presents experimental and predicted displacement–time evolutions for a P16 specimen with a mass of 1500 kg (PGA = 0.33 g). The FE model is generally in good agreement with the experimental results. Average error in peak displacement only exceed 10 % for long simulated time (more than 60 sec).

SCALE 3: STRUCTURE

The modelling of a whole 3D timber–frame structure at the scale of elementary components (joints, framing, sheathing...) may lead to important computational costs. Thus, a macro–scale finite element is proposed to model shear walls.

Simplified FE model development and calibration

The simplified FE model is composed of a frame of bars ensuring the hypothesis of a parallelogram–like deformation of the wall, therefore modelling only the shearing behaviour (*i.e.* in and out-of-plane bending and overturning effects are not taken into account). This hypothesis is based on previous studies [9], experimental observations and refined FE model results. The in–plane horizontal degree of freedom of the simplified element is modelled by the previously described constitutive hysteretic behaviour law, whose parameters are identified thanks to the results of the refined FE model under quasi–static loadings. The roof load, which is predominant, is modelled with two lumped masses $m/2$. The calibration process is similar to the direct calibration of the joints.

Simplified FE model validation

The macro–element model is calibrated for quasi–static loading and then used for dynamic calculations. In order to assess the accuracy of its behaviour under dynamic loading, the simplified FE model predictions and the experimental results under dynamic loading are now compared, those predictions can also be compared to those of the refined FE model. The dynamic calculations carried on both the refined and the simplified models showed that the computation time ratio is about 12. Results show that while the simplified and the refined FE models match perfectly under quasi–static loading, it is less the case under dynamic loading. The simplified FE model is based on the idealization of the refined FE model behaviour, which is not perfect but shows nevertheless a good agreement. When compared to the experimental results, the errors due to the simplified FE model predictions are slightly greater than the ones obtained with the refined FE model.

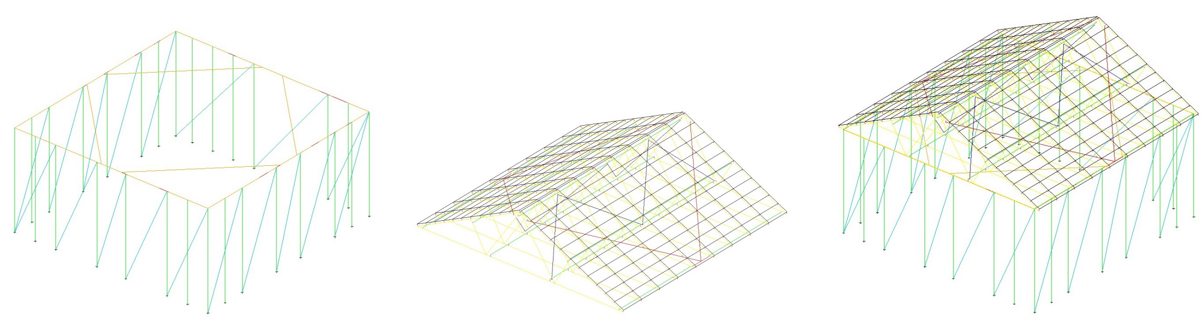
Use of the simplified FE model

The simplified FE model is developed in order to build accurate and computation time efficient models of buildings. A whole 3D building model is obtained by connecting different simplified FE models. Openings can be modelled either by a simplified FE model that includes adjacent shear walls, or by a specific simplified FE model limited to the opening area. The latter is proposed in this study for the following reasons:

- Calculations showed that the maximal force is proportional to the length of the wall. Thank to this property, the number of necessary refined FE models to calibrate the simplified ones is reduced to one. This property is only valid for blind shear walls (without openings).

- The opening area can be modelled by a specific simplified element, which necessitates a refined FE model for its calibration. Yet, due to the opening and the relatively small dimensions of the simplified element, the refined FE model is relatively light and therefore quickly developed and calculated.

A single story house (6×6 m) model is built according to the aforementioned method. Shear walls are modelled by means of the simplified elements, and are linked one to another by ball–and–socket connections. All the top nodes of the simplified elements are also attached to an horizontal diaphragm made of four beams placed in diagonal (Figure 7.a). A refined model of roof has been developed, following the same modelling principles than the refined models of shear walls (Figure 7.b). It is attached to the simplified shear walls by spring–like elements modelling 3D connectors of bracket type. The development of a simplified element of roof is considered irrelevant, as the refined one does not require an important computational time compared to refined models of shear walls. Figure 7.c shows the model of the single story house. It has been developed in order to design an experimental test of such a structure on the shake table of the CEA at Saclay, France.



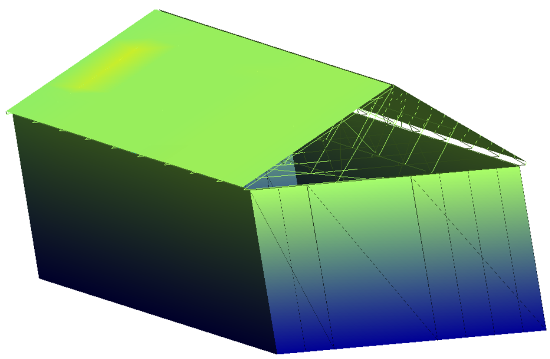
(a) Shear walls and horizontal diaphragm

(b) Refined model of roof

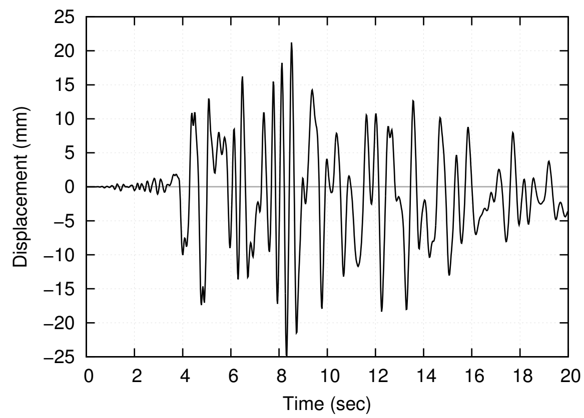
(c) House model

Figure 7: Mesh of the structure

The design of the seismic test is ongoing work. Dynamic calculations are carried out by using two–directional earthquake signals (North–South and East–West) in order to predict the behaviour of the structure. Figure 8.a displays an example of the house deformations, and Figure 8.b shows the displacement–time evolution of one of the wall.



(a) Deformation of the house during a dynamic calculation



(b) Displacement–time evolution of a wall

Figure 8: Dynamic calculation for the two–directional Kobe earthquake signal

CONCLUSION

This paper is dedicated to the development of a versatile hysteretic constitutive behaviour for timber joints made of metal fasteners. The main features of the model are its ability to accurately describe the hysteretic behaviour, notably by considering the damage effects (strength reduction), and its numerical robustness. Application of this constitutive behaviour law is then described for the multi–scale modelling of a timber–frame structure in the scope of an analysis under dynamic loading. Experimental results of tests performed on timber joints with metal fasteners are used for the identification of the model parameters. Using these calibrated models of joints, a FE model of shear wall is developed. The FE model predictions are compared to the experimental results of 14 quasi–static tests and 12 dynamic tests for validation. Those comparisons show that the FE model accurately predicts the experimental behaviour of different configuration of shear walls.

In order to reduce the calculation time for dynamic simulations, a simplified FE model is developed. The calibration of this element is based on the refined FE model results under quasi-static loadings. The results obtained with the two models for dynamic calculations are compared. The results are reasonably close and the simplified model provides a significant gain in calculation time. The discretization of a wall into several simplified elements is based on the property of proportionality between the maximal force of a full shear wall and its length. Finally, the simplified model is used to build a 6×6 m single story house. This FE model of a complete structure is used to design a future seismic test, which is currently ongoing research.

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