Timber connections under seismic actions

STEP Lecture C17
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Objectives

To provide an understanding of the behaviour of joints of timber structures in seismic zones and the method of evaluating the performance in accordance with the Eurocode 8 format.

Prerequisite

C3 Joints with dowel-type fasteners - Theory

Summary

The behaviour of timber structures under earthquake actions is mainly determined by the behaviour of the connections under low cycle loadings. The different mechanisms for dissipating energy such as plastic deformations in wood and steel, friction between different parts and viscous damping, are quoted and evaluated. Cyclic performances of different kinds of connections are considered, referring to the available test data. Finally the method of evaluating test results is given according to Eurocode 8, Constructions in seismic zones, Part 1.3, chapter 5.

Introduction

The modern approach to the design of structures in seismic regions considers that buildings should resist the so-called "service" earthquake ("moderate" but "likely", i.e. with a peak ground acceleration having an average return period of 50 years) without limitations of use, serious deformations or significant damage. In addition buildings should resist the "ultimate" earthquake ("severe" but "accidental", i.e. with a return period of 250 years); in this case, there may be serious damage to the structural elements, but there must not be complete collapse. When subjected to a severe seismic motion, the structure "softens", increases its own period of oscillation, "dissipates" kinetic energy and, thanks also to the cyclic character of the input action, "has time" to invert its motion prior to develop deformations leading to collapse. The capacity of a structure to developing plastic deformations within its structural elements and to dissipate energy without breaking is an essential part of its capacity to resist a seismic input (Ceccotti, 1989). It is demonstrated that a structure with plastic and dissipative joints, if appropriately designed, can resist higher seismic motions than the same structure with rigid and non-dissipative joints. In principle this is true also for all kinds of structure. However especially referring here to the case of timber structures, it is necessary to take into account some further considerations.

Under alternating load, timber elements exhibit a generally linear elastic behaviour. Failure is brittle, primarily because of natural defects like knots, and there is little dissipation of energy in the wood, except maybe in zones with compression perpendicular to the grain. Glued joints also behave linearly elastically, and therefore contribute neither to the plastic behaviour of the structure nor to the energy dissipation. This means that timber structures composed of glued joints and of members assembled with perfect hinges, for example, should be regarded as non-dissipative, with no plastic performance whatsoever.

However, plasticity and capacity to dissipate energy can be achieved in the connections between the various structural elements if they are "semi-rigid" (as most mechanical ones are) instead of "rigid" (as glued ones are). Well designed joints with mechanical fasteners have, in general, a very pronounced plastic behaviour.

Structures may be classified into categories taking into account their plastic behaviour and their ability to dissipate energy (see STEP lecture D10 for more detailed discussion). This is a fundamental aspect to consider when designing for seismic loads as it allows a much more economic design, than if every part of the structure had to be kept in the elastic range of its behaviour. In terms of seismic design codes this is done by designing for the design earthquake load actions divided by a behaviour coefficient q which reflects the above inelastic behaviour and the global ductility of the structural system. The design earthquake to be considered is defined by taking into account the relevant seismic zone map (produced by the national authorities). In Eurocode 8, the coefficient q is called the "Action Reduction Factor" or "Behaviour Factor", and each structural category is characterised by a particular value of q. According to the type of timber structure, q ranges from 1 to 3. For perfectly elastic structures, obviously $q = 1$. But if a higher behaviour factor is assumed, then sufficient plasticity and energy dissipating ability in the joints must be guaranteed.

However, if design calculations carried out for static loads anticipate forces on the sections that are already higher than the ones expected in the case of seismic loads (even when assuming $q = 1$), then there is no advantage in requesting any particular ductility from the joints. This may be the situation with many large structures with heavy snow loads. In these cases it is not necessary to make tests or follow particular detailing rules other than the usual ones for static situations.

Apart from some particular cases, in general it is advantageous to consider an appropriate action reduction factor, but this implies the need to demonstrate that connections are sufficiently plastic and dissipative to match the foreseen q value. This can be achieved with dedicated tests or, in the case of very well known types of connection, by just following certain detailing rules as explained below.

Ductility

Mechanical joints in general exhibit a very plastic behaviour, provided that the usual requirements for end and edge distances are respected. This is due to the embedding behaviour of timber itself, coupled with the plasticity and the ability to dissipate energy of the steel elements (see STEP lecture C3). The load-slip diagram under monotonic static loading is characterised by an initially steep incline (see Figure 1a, I). Once the elastic limit of either the fastener material and/or the wood embedding stress has been passed, the slope of the load-slip curve decreases until a horizontal part of the diagram is reached, indicating the limiting resistance F_{max} of the joint (see Figure 1a, II). This is followed by a decreasing part (see Figure 1a, III) which indicates that the joint has failed due to for example the splitting of the wood or the breaking of the steel. (Of course this part can only easily be recorded if the test is made under displacement control).

A definition of ductility is given in Figure 1. A distinction is made between the case when the characteristics are approximately bi-linear and the case when they are completely non-linear.

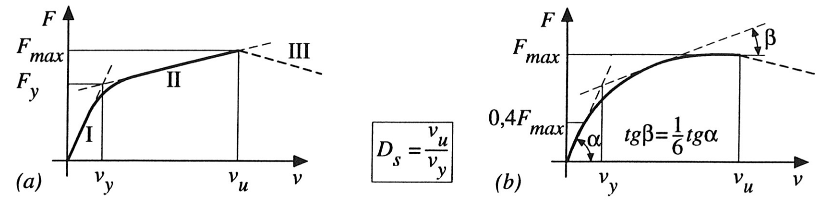

Figure 1 *Criteria for evaluation of static ductility: examples for different possible load-slip curves. (a) two different slopes are easily identified (b) the curve has a continuously changing curvature. D_s is the ductility, v_u the ultimate slip and v_y the yield slip.*

The idea in the second case is to take tan β equal 1/6 tan α. The factor 1/6 is a reasonable compromise between the different extreme curves. The uncertainties in the determination of v_y in order to determine the ratio v_u/v_y may be disregarded, bearing in mind the other uncertainties present.

The eventual descending part of the curve after the maximum load has been reached indicates that the joint is fractured but still resistant. Reference to a load 20% less than the maximum, if a larger ductility is to be considered, is usually permitted.

Cyclic behaviour and energy dissipation

The cases in Figure 1 refer only to monotonic loading, but some more complex phenomena will happen under seismic actions when cyclic loading with an inversion of the force is applied in a few seconds. Consider the case of simple regular cyclic loading applied, in a quasi-static manner, to a nailed joint as shown in Figure 2. On the first loading to a given level the wood fibres around the nail are compressed and crushed, leaving a cavity, in which the nail is unsupported during subsequent loading cycles within the displacement range. The subsequent residual strength in this range arises solely from the strength of the fastener acting as a cantilever over the length of the cavity. As the previous displacement is exceeded, the nails once more take up bearing against the wood fibres, and loading proceeds approximately along the parent curve as it would during monotonic loading (slight differences may be due to the pull-out effect of the head of the nail, as shown in Figure 2b, and to strain hardening of the steel during the alternating loading).

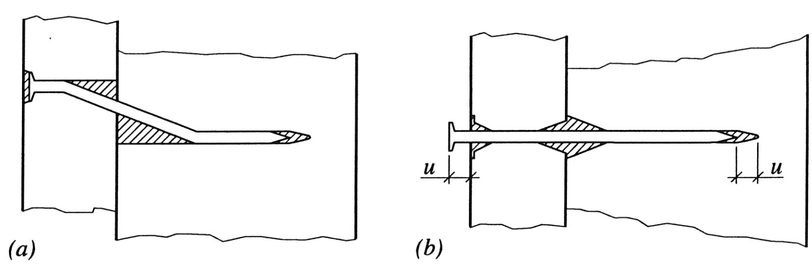

Figure 2 *Cavities in plywood and framing adjacent to the nail in load cycling.*

Typical loops in the load-slip diagram, whether for low, intermediate or high deformations, are quite narrow, or "pinched", as shown in Figure 3. They differ from the "fat" loops typical of mild steel, where the forces necessary to restore the plastic deformations to zero are similar to those causing the plastic deformations in the first place, (Figure 4c). The "pinching" of the nailed joint loops reflects the "cavity" phase of the deformation.

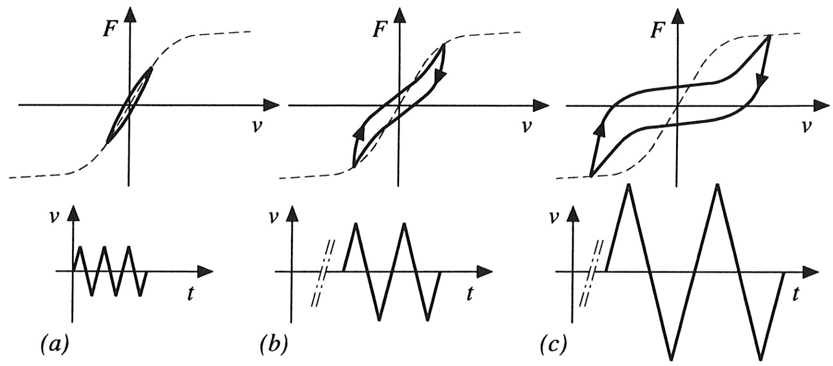

Figure 3 *Typical load-deformation loops for different load levels for dowelled joints.*

Figure 4a represents the shape of a well designed dowelled joint, where energy dissipation is due both to the embedding behaviour of the timber and the plastic behaviour of the steel. If the dowel is so rigid and resistant that it does not bend, and the dissipation of energy and ductility are those obtained from the embedding strength of the timber only, then the load-slip curve is as shown in Figure 4b.

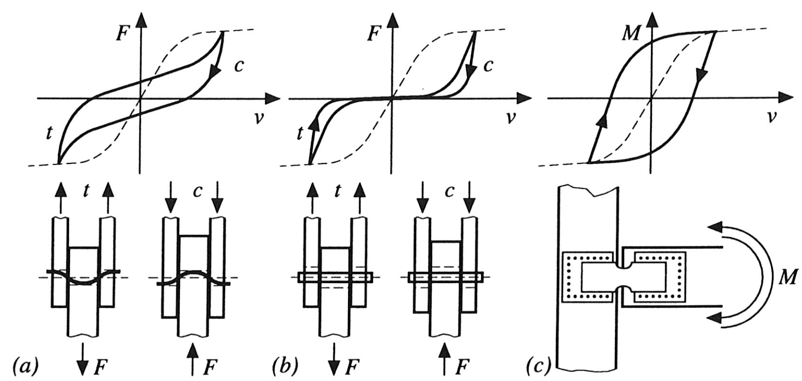

Figure 4 *Possibilities of dissipating energy according to different arrangements of joints (t is tension, c is compression).*

It has been seen that the envelope curve for cyclic loading is assumed to coincide with that for monotonic loading, i.e. it is independent of the loading history. The difference between the two is normally less than 10% unless there is some alteration in the configuration of the joint (e.g. a very pronounced pull-out effect on the fasteners) or some fracture due exclusively to low cycle fatigue, even though mechanical timber joints are generally not very sensitive to such effects.

There are exceptions to the above. An example is the many types of steel plates with integral punched nails (teeth). In these plates, failure under repetitive loading will often be caused by sudden tooth withdrawal or by brittle failure in the steel. Other examples are joints with light gauge steel straps and pre-perforated holes for nails. In these, the alternating load may cause pull out of the nails. Another case is that of timber framed walls with very brittle board materials where after cyclic loading, important pieces of material are damaged and the original strength is lost.

Therefore in order to have a harmonised basis for the evaluation of the cyclic behaviour of joints, a CEN Standard is under preparation, giving a simple method for testing joints, in a quasi-static mode, performed under displacement control. Figure 5 represents the specified cyclic history, with triple cycles of amplitudes that are multiples of the yield slip v_y.

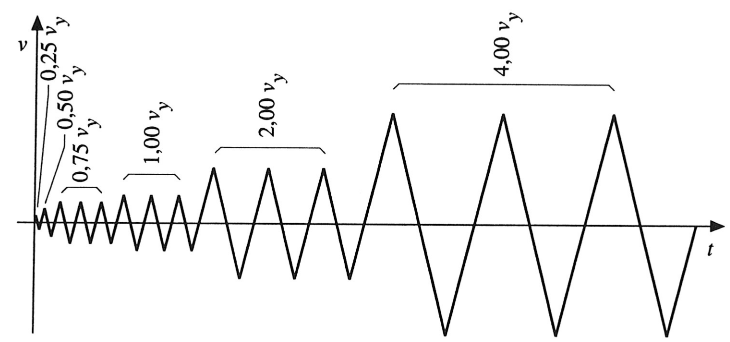

Figure 5 *Recommended procedure for cyclic loading tests.*

In Figure 6 ΔF represents the "impairment of the strength" at the same displacement level, between the envelope curve of the first loading and that of the third loading.

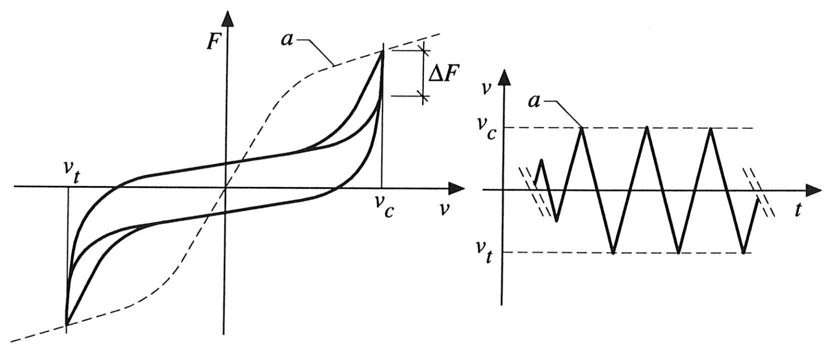

Figure 6 *Impairment of strength between the envelope curve corresponding to the first cycle, curve a, and the third cycle.*

In the inelastic range, the amount of dissipated energy per half cycle due to plastic deformation (hysteresis) is measured by the shaded area E_d in Figure 7. The ratio between the dissipated energy and the available potential energy E_p is called the "hysteresis equivalent viscous damping ratio" ν_{eq}. The hysteresis dissipated energy E_d increases with an increasing amplitude of the loops, whilst ν_{eq} remains more or less constant. Values of about 8-10% have been evaluated for well designed dowel type fastener joints and for plywood framed walls.

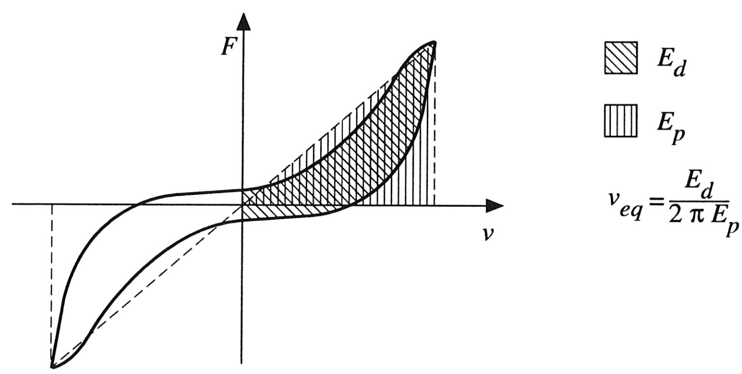

Figure 7 *Dissipation of energy by hysteresis.*

Of course, in the elastic range, the hysteresis damping is in principle zero (Figure 3a). However, in the elastic range, some energy can also be dissipated. In low amplitude dynamic vibrations with no secondary structures in place, only a "viscous" damping ratio of less than 1% can be measured. But the damping ratio due to friction in the junctions between different elements and through compression perpendicular to the grain can easily reach values of 4% and more. This is especially the case with many redundant elements and contact points, such as are found in dwellings. This explains why in the elastic range a value of 5% for the damping ratio is usually assumed.

Behaviour of different types of joint

As seen above, the cyclic performance of mechanical joints is characterised by good ductility, lack of sensitivity to repeated loads, and the ability to dissipate energy.

In order to avoid brittle failure due to premature splitting, the rules given in EC5 concerning end and edge distances should be followed; these distances have been given in order to ensure ductile behaviour, otherwise the Johansen theory and the derivations of the verification formulae given by EC5 would not be valid (see STEP lecture C3). At the present state of knowledge, there is no clear evidence that cyclic loading *per se* worsens the risk of splitting. However, the adoption of greater spacings between fasteners, and larger edge and end distances would contribute to increasing the splitting resistance and consequently the ductility of the joint.

Splitting can also be prevented by including in the connection area reinforcing materials with high tensile strength perpendicular to the grain, such as plywood or densified veneer wood. In this way not only is splitting better controlled, but also plasticisation of the steel fasteners is ensured, thus improving the yield performance of the timber joint in terms of ductility. Obviously the use of mild steel fasteners, which have a larger deformation capacity, will in general be more suitable for ductile and energy dissipating connections than hardened steel fasteners with low ductility. In order to improve the ability to dissipate energy, it is possible to take advantage in design of the slenderness of dowel-type fasteners. Slenderness is defined as the ratio between the thickness of the wood member and the diameter of the dowel-type fastener. Slender fasteners always tend to dissipate more energy, because plastic hinges will always appear in the steel whereas if stocky steel fasteners are used, they perform elastically without dissipation of energy in the steel. Moreover, splitting is better prevented if the thickness of the timber member is increased in relation to the diameter of the fasteners.

To avoid unacceptable loss of strength under cyclic loading, three general principles should be followed. These are to use details where elements cannot easily pull out, to avoid materials liable to brittle failure, and to try to use those materials which retain a consistent behaviour under repeated loading.

Now the cyclic behaviour of the most common types of mechanical joint will be examined (for "detailing rules" see STEP lecture D10).

Dowel-type fasteners

Nails, staples and screws

Apart from those made from hardened steel, nails, staples and screws show a distinct plastic behaviour when loaded in mechanical timber joints. The length of the shank should be increased if risk of pull-out is foreseen. Smooth nails are not recommended, in order to prevent this. If the slenderness ratio of the nail is greater than 8, good ductility will be ensured (Figure 8).

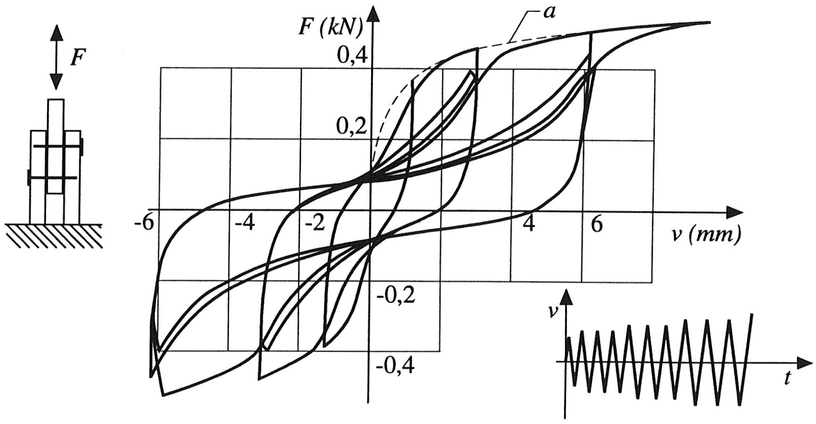

Figure 8 Typical cyclic behaviour of a nailed joint (nail slenderness 8,5).

In connections between plywood panels and timber elements, ductile behaviour can be obtained provided that the slenderness of the nails is higher than 4. Tests with nailed shear walls show large ductilities, and large energy dissipation capacities (Figure 9).

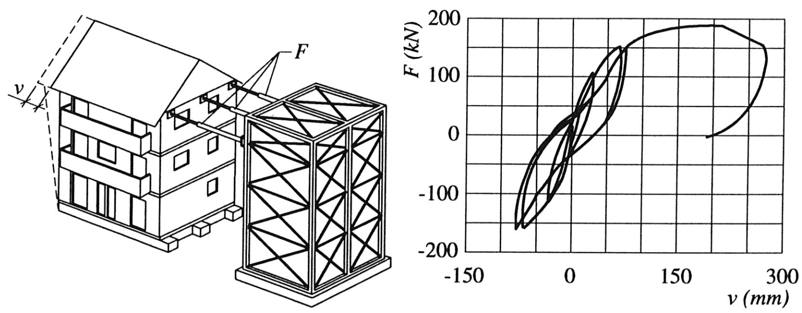

Figure 9 Ductile behaviour of a plywood sheathed timber house.

Dowels

Connections with slender steel dowels are able to yield in both the steel and the wood, thus allowing a large amount of energy dissipation. If the slenderness of dowel fasteners is higher than 8, the behaviour will certainly be of good ductility. Irrespective of other parameters, such a slenderness will ensure mode three failure (see STEP lectures C3 and D10). With stocky dowels and standard spacings, plasticity will depend upon the embedding behaviour of the wood alone. With less capacity for energy dissipation, tests are recommended for assessing the ductile performance of such joints.

Bolts

In bolted connections, oversized holes due to fabrication tolerances cause non-uniform load distribution. The consequent overloading of particular bolts may cause splitting of the wood under these bolts, preventing a redistribution of the load within the connection. In seismic regions, therefore, only precisely manufactured bolted joints, and preferably those using slender fasteners, are recommended. Large bolts (d >16 *mm*) have difficulty in deforming and hence in dissipating energy. It is recommended that they should only be used in combination with toothed ring connectors.

Surface fasteners (connectors)

Split ring and shear plate connectors
Because of the small plastic deformations which are possible, such types of fastener are less suitable for use in dissipative zones.

Toothed plate connectors
If well designed, toothed plate connector joints can exhibit good plastic behaviour. Attention must be paid to spacing rules, so that splitting does not occur.

Punched metal plates
Although load-displacement curves for joints with punched metal plates show a certain amount of plastic deformation, the possibility of a brittle failure of the plate and the potential pull-out effect under cyclic loading indicate that prototype tests are advisable if dissipative design is intended.

Seismic behaviour of mechanical joints

Until now, consideration has been given to the quasi-static evaluation of the cyclic behaviour of joints. But a different loading than that is imposed in a real earthquake. Obviously the influence of the loading rate cannot be taken into account by these types of cyclic test. On the other hand, the frequency content of the seismic input is also unknown.

It is emphasised, therefore, that cyclic tests seem actually to be sufficient to estimate with enough accuracy the seismic behaviour of joints. With the present state of knowledge, it is felt that the actual behaviour of joints is likely to be more stiff and resistant under "instantaneous" loading than under short-term loading of the same magnitude. It has not been shown that instantaneous loading of the velocity ratio induced by earthquakes has any significant influence in reducing ductility. Cyclic tests are considered sufficient since they provide, with enough accuracy, all of the parameters necessary to predict the behaviour of a structure in a real earthquake.

In fact if the "shape" of the cyclic behaviour of the joint is known (RILEM, 1994) a calculation programme for non-linear seismic analysis can be used and theoretical calculations performed in order to find the strength of the structure under a given earthquake i.e. the acceleration producing collapse (the problem of representing a particular earthquake is not considered here which, of course, presents the same difficulties for all materials and structures).

Another point to be emphasised is that under a real earthquake the cycles will be less "regular" than those in cyclic laboratory tests because the input will be random and irregular: so the number of entire cycles at the maximum displacement will in general be very small, whilst the smaller cycles will be more numerous. As an example, Figure 10 shows the Moment - Rotation history of a dowelled corner joint of a portal frame, under the El Centro earthquake. This is based on a numerical simulation, and the earthquake effect was amplified by a factor of 1,5.

Requirements of Eurocode 8

In EC8 "Constructions in seismic regions" structures are classified into categories according to the ability of their joints to be ductile and be capable of dissipating energy in the plastic field. It is, in any case, recommended that structures be designed to be sufficiently rigid to meet serviceability criteria. For structures designed to take profit from their ability to dissipate energy ($q > 1$) it is also recommended that the strength of the timber members should be higher than the strength of the connecting joints. This implies that plasticity in the joints will be achieved.

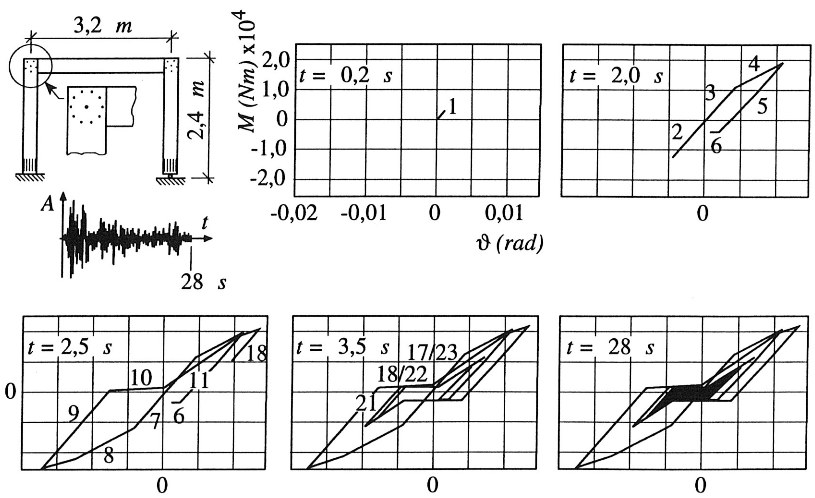

Figure 10 *The Moment - Rotation history of a portal frame corner under El Centro earthquake.*

The properties of dissipative joints under seismic actions are as a rule required to be demonstrated through testing, by means of agreed international standards. By such tests, it must be shown that the ductility is sufficient and that the joint properties are stable under cyclic loading at a reasonably high load/deformation level. To ensure sufficient ductility, it is required that the ductility obtained from cyclic tests should be greater than the assumed behaviour factor q multiplied a factor of 3. This value is reduced to 2 for panel structures, because of the highly positive effect in reducing inertia forces due to damping caused by friction, and due to compression perpendicular to the grain between parts. Such effects are believed to give a damping ratio more than the usual 5%.

In addition, it is stated that connections between elements must be able to deform plastically for at least three fully reversed cycles at the above ductility ratio without an impairment of their strength of worse than 20%. By complying with these conditions, the designer is allowed to calculate the strength and stiffness of the joint following the normal design rules of EC5.

Concluding summary

- For design purposes the seismic behaviour of mechanical timber joints can simply be related to quasi-static behaviour under cyclic loading.

- Ductility of joints and dissipation of energy are the most important features for dissipative design of structures to resist earthquakes.

- If the cyclic behaviour of joints is sufficiently stable, seismic design can be performed using normal EC5 design values for the strength and stiffness of mechanical joints.

References

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