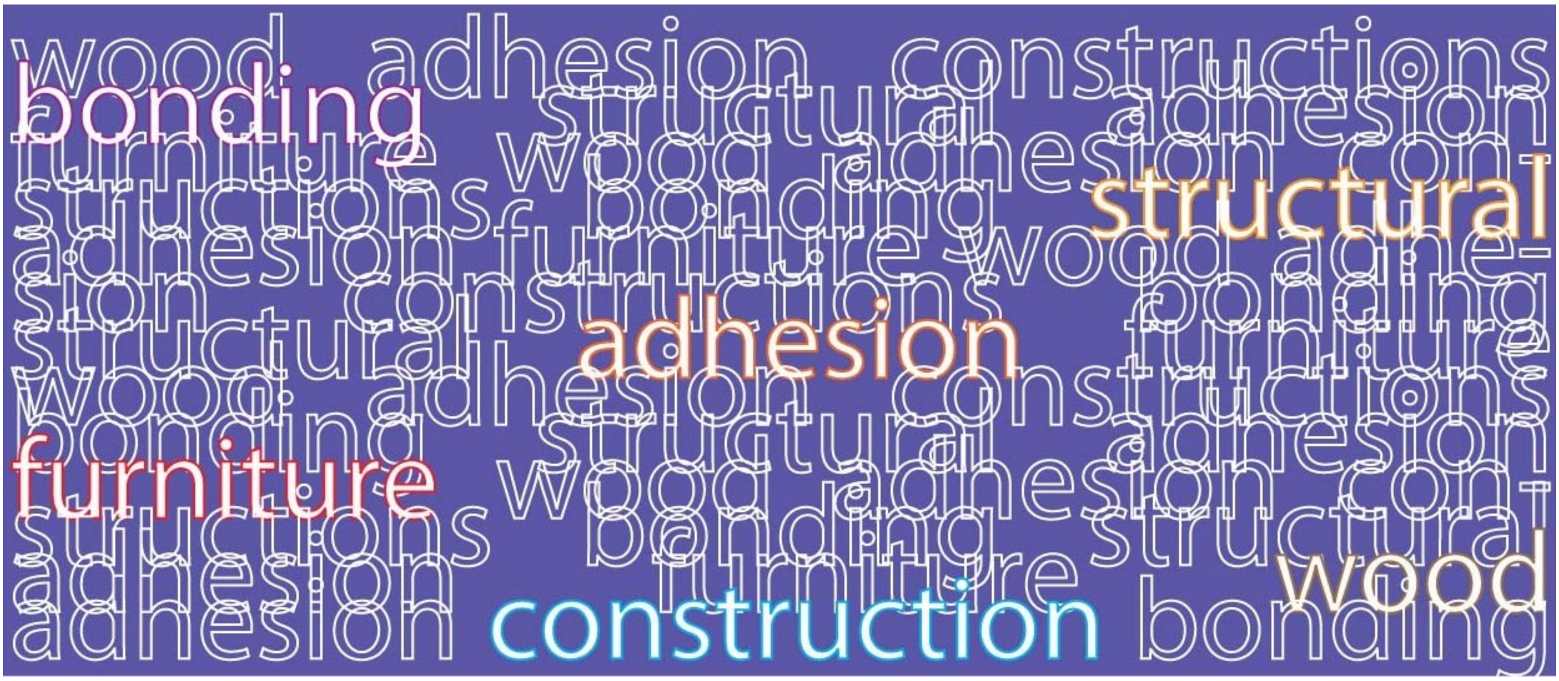
Practical Solutions for Furniture and Structural Bonding



International Workshop

Larnaka - Cyprus, 22-23 March 2007

Edited by

Dr. Georgios Ntalos & Dr. George Mantanis

COST Action E34 *"Bonding of Timber"*

in cooperation with

Dept. of Wood & Furniture Design & Technology & TTR Centre

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Scientific Committee

The main objective of the COST Action E34 is the improvement in bonding timber for a broad range of applications including a higher common technical, economical and environmental standard. In particular, in the 4th International Workshop held within this Action, the topics are focused on the *Practical Solutions for Furniture and Structural Bonding*, and as such, new technologies and practical industrial projects are presented and discussed. The contributions to this workshop cover therefore the whole range from practical expertice to scientific studies.

The contributions to this workshop have been evaluated and approved by a Scientific Committee composed of the following members:

- Manfred Dunky, Dynea Austria GmbH, Austria
- Helena Cruz, LNEC, Portugal
- Björn Källander, Sweden
- George Ntalos, TEI Larissa, Greece
- Martin Ohlmeyer, BFH, Hamburg, Germany
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Opening of the COST E34 - 4th Workshop
"Practical Solutions for Furniture & Structural Bonding"

George Ntalos, George Mantanis and Michalis Sokratous
Dept. of Wood & Furniture Design & Technology (TEI Larissa) and TTR Centre (Cyprus)

Dear colleagues:

It is our great pleasure to welcome you all to this excellent place (Larnaka, Cyprus), and we are almost ready to kick-off this COST E34 workshop!

It is great to see many colleagues coming all the way to present their recent works on topics relating to *"Practical solutions for furniture and structural bonding".* In this workshop additionally some state-of-the-art techniques and recent technical and research data will be presented on the areas of furniture and wooden constructions as well as on structural rehabilitation of timber structures on-site.

At the same time, we wiil hopefully give the opportunity to local technical people coming from the furniture manufacturing or civil engineering sector to experience and learn about new methods and techniques.

We would like to thank much all the contributors to this technical workshop and of course to the COST Office. We are positive that this workshop will not only provide a state-of-the-art, but also it will function as an inspiration for wood-working enterprises and technical people initialising a more widespread practical implementation of the presented technologies, techniques and applications.

Thus, we hereby declare that the COST E34 "4th Workshop" is open!

"Furniture"

Chaired by Dr. **Martin Ohlmeyer**

Wood bonding in the furniture industry and the effect of changing wood supply

Frihart, C.R., Wiedenhoeft, A.C., Jakes, J.E.[1]

Abstract: *Wood is a complex and heterogeneous material, exhibiting variation in its structure and properties at all size scales. For furniture manufacturing, both macro- and microscopic variations in wood structure affect its bondability with various adhesives and the longevity of those bonds. For example, the relative proportion of earlywood and latewood affect mechanical and rheological properties of wood and dimensional stability and is an important macroscopic feature. Especially in some hardwoods, microscopic characteristics such as vessel size and their distribution influence minimum thickness of the veneer or adhesive formulation to minimize bleed through. At a larger size scale, the presence of juvenile wood or reaction wood in a piece of core stock affect mechanical and physical properties of the wood, thus potentially changing the expected efficacy of bonding and durability of these bonds. The substitution of plantation-grown wood for old-growth wood complicates the performance of these bonds by decreasing uniformity of wood properties. In summary, variations in chemical composition and micro- and macroscopic wood structure play important roles in bonding wood. Understanding these factors is the first step toward achieving good service life for furniture and structural wood applications.*

Introduction

The widespread availability, favorable economics, and aesthetic appeal of wood have led to many uses in our homes and in businesses of bonded wood products from lumber, veneers, flakes, fibers, and particles. Different product types and assembly conditions necessitate many types of wood-bonding adhesives. The fact that an adhesive for structural uses needs very different properties than one for furniture assembly makes it difficult to draw general conclusions about essential adhesive properties. However, two aspects are desired for all wood adhesives: the bond should be stronger than the wood, and the bond should adjust to dimensional changes of wood as humidity and temperature change. A better understanding of wood structure, adhesive properties, and adhesive interaction with wood can help manufactures make better wood products, including furniture.

Wood is an unusual substrate in many respects. For a structural material, wood can shrink and swell repeatedly with changes in moisture content while losing only a small portion of its intrinsic structural integrity. Wood is porous, so adhesives and bonding conditions need to be controlled to obtain sufficient but not excessive penetration. Material properties can vary widely between the many wood species and even within a species, depending upon the quality of the material and the way in which it was processed. Even material properties of pieces of wood from the same tree can vary greatly depending on the relative amounts of juvenile, reaction, and mature wood present. The small joint sizes used in furniture can make these bonds very sensitive to changes in material properties of the wood used and its moisture level. Given that most of the questions about bonding problems or bond failures that we receive at the Forest Products Laboratory are related to furniture, we felt that more detailed knowledge about wood and wood bonds would be helpful to the furniture industry. Thus, this paper discusses wood structure, adhesives, adhesive–wood interactions, and wood availability issues.

Wood Anatomy and Properties

Wood is a biological composite having coordinated domains of various component cells in distinct sizes, shapes, and configurations. One can describe wood at a variety of scales, from single chemical bonds all the way to functioning of wood in a living tree. For this paper, we consider wood structure at two interrelated levels: (1) cells and cell assemblages that define the microscopic structure of wood and (2) larger scales of analysis that are evident to the unaided eye. Both scales of analysis are critical for understanding bond durability in wood–adhesive interactions

Cell walls and cell types in wood

Cells in wood are composed of two domains, the cell wall and the cell lumen (pl. lumina) (Figure 1B,C). The cell wall is the actual substance of wood, and the lumen is the air space internal to the cell wall. This air space or void volume is critically important in wood properties because it affects physical properties such as density

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and adhesive interactions with the wood. The lumen of a cell is as much a part of the cell as is the cell wall itself.

Although all cells in all wood species have both cell walls and lumina, not all cells in wood are the same; characteristic cells are found in hardwoods (wood from broadleaved trees such as maple and beech) and characteristic cells are found in softwoods (wood from cone-bearing trees such as pine, fir, and cedar). The relative proportions of cells in each wood species combine to define its wood structure and consequently its properties.

Softwoods contain two main kinds of cells: tracheids and parenchyma cells. Tracheids are long, thin cells that make up over 90% of the volume of most softwoods. They are oriented along the grain of the wood and vary in wall thickness, cell length, and other microscopic features but are otherwise similar across all softwoods. In the tree they function in long-distance water transport in sapwood and mechanical strength, which makes them the critical cell type in softwood–adhesive interactions. Tracheids have either thin or thick walls, depending on their position in the growth ring (see below). Parenchyma cells are roughly brick-shaped cells that play only a small role in either mechanical strength of wood or interaction with wood adhesives. The resin in the resin canals can cause surface appearance and bonding problems.

Hardwoods contain three main cell types: parenchyma cells (virtually identical to those in softwoods and less important to wood bonding than other cell types), vessel elements, and fibers. Vessel elements are the defining cell type of hardwoods, are specialized for long-distance water transport, and are oriented along the grain of the wood. They are barrel-shaped, large-lumined, and thin-walled, with little mechanical strength. Fibers, also oriented along the grain of the wood, are thick-walled, long spindly cells, much like tracheids in their overall shape but narrower and shorter; they are specialized for providing mechanical strength. Knowing the functions of these cell types in the living tree will prove relevant to understanding wood–adhesive interactions in the context of wood permeability and bond strength.

Gross wood structure

Although the component cells of softwoods and hardwoods differ in some respects, the overall organization of cells in both kinds of wood is similar. Specifically, wood has two cell systems: an axial system and a radial system. The axial system is the sum of all the cells running along the grain of the wood; indeed, the grain of the wood is the axial system, and it functions largely in water transport and in providing mechanical strength to the tree. The radial system runs at a 90-degree angle to the axial system, from the center of the tree out toward the bark, and functions primarily to provide living cells with necessary chemicals, including water.

The axial system, however, is critically important, and is further subdivided into functional units that are important to the tree, to the wood user, and to adhesive interactions. As most people know, trees in the temperate world lay down a certain amount of wood each growing season—a growth ring or growth increment. That growth increment is, at least in some species, clearly divided into two different domains, earlywood and latewood (Figure 1D,E). The earlywood is the first-formed wood of the growth increment and when distinct from latewood is characterized by generally thinner cell walls and larger lumina. This results in the earlywood having lower density and higher permeability. Conversely, the latewood is generally characterized by cells with thicker cell walls and narrower lumina. In addition to wall thickness, hardwoods can show appreciable variation in the proportions of cell types (vessels and fibers) in each domain of the growth ring. An important macroscopic property of wood that is derived from growth rings and growth rate is the number of rings per centimeter, which is a measurement of how quickly the tree grew. In the case of softwoods and many hardwoods, slower growth generally indicates higher density and more desirable wood properties. For ring-porous hardwoods such as oak, elm, and ash, slower growth actually results in lower density and generally less desirable wood properties because of relatively fewer fiber cells and more vessel elements.

The two systems give rise to three planes of section, or three directions of observation, in wood. They are the transverse, radial, and tangential (Fig. 1A) planes. Radial and tangential are also used to describe directions of dimensional change across a board. Tangential change is change in the direction along the growth rings, across the board, and radial change is at 90 degrees to tangential change.

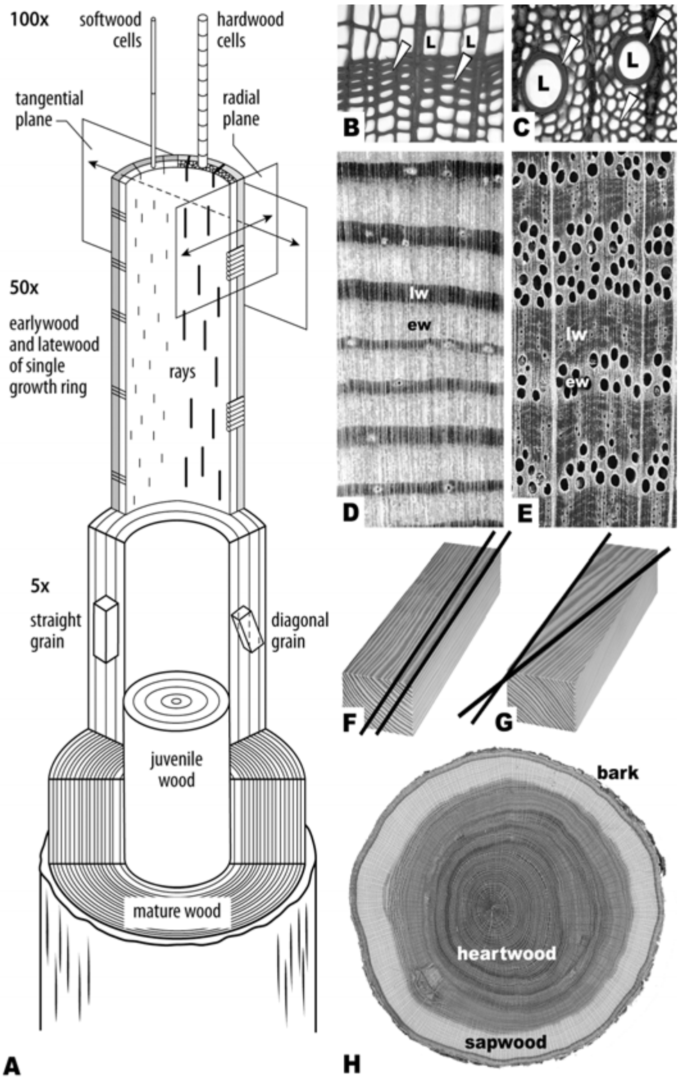


Fig. 1. *A, cut-away illustration of a tree at various magnifications intended to correspond roughly with the images at right; top, a softwood cell and several hardwood cells illustrated to give a sense of scale between the two; one tier lower, a single growth ring of a softwood (left) and a hardwood (right) and an indication of the radial and tangential planes; next tier illustrates many growth rings together and how one might produce a straight-grained rather than a diagonal-grained board; lowest tier illustrates the relative position of juvenile and mature wood in the tree. B and C, light microscopic views of the lumina (L) and cell walls (arrowheads) of a softwood (B) and a hardwood (C). D and E, hand-lens views of growth rings, each composed of earlywood (ew) and latewood (lw) in a softwood (D) and a hardwood (E); F, a straight-grained board; note that the line along the edge of the board is parallel to the line along the grain of the board. G, a diagonal-grained board; note that the two lines are markedly not parallel; this board has a slope of about 1 in 7. H, gross anatomy of a tree trunk, showing bark, sapwood, and heartwood.*

Zones in the tree

If we step away from the microscopic structure of wood and instead think about the standing or freshly cut tree (e.g., Fig 1A), we can consider several large-scale features. Specifically, the typically dark-colored, extractive-rich heartwood is distinct from the light-colored sapwood (Figure 1H). The heartwood represents the older wood in the tree and often is the wood of commercial importance for most furniture applications because it is the color-bearing wood. The sapwood, found directly beneath the bark, is conversely the younger wood of the tree, lacks the color of the heartwood, and is sometimes removed during wood processing. However, some species have mainly sapwood. Historically, when people were harvesting mature, naturally grown trees, heartwood and sapwood were the only distinctions of importance to make in lumber.

Now, as we harvest ever-increasing volumes of fast-grown plantation material, the difference between juvenile and normal wood is another distinction of concern.

Juvenile wood is the collection of the first 5 to 20+ growth rings, depending on species, found at the center of the tree. This means that some of the heartwood is in fact juvenile wood. Juvenile wood is structurally and chemically different from mature wood and as such it has different physical and mechanical properties. For example, Kretschmann (1997) concluded that shear strength parallel to the grain of solid wood decreased as the percentage of juvenile wood in the shear specimen increased in loblolly pine. For the most part, compared with normal wood, juvenile wood has undesirable traits and is best avoided, if possible. In many ways like juvenile wood, reaction wood (wood that is formed by leaning trees) also has chemical and structural differences compared with normal wood and behaves differently, as well. Juvenile wood is discussed below in some detail with regard to moisture relations in wood and how it affects the properties of an adhesive–wood bond, but much of what is said applies to reaction wood, as well.

Moisture relations in wood

Wood is fairly unique as a material, in that it undergoes dramatic changes in dimension with changes in moisture content (MC) of the board. The MC is defined as the weight of water in a board as a percentage of the dry weight of the board. Moisture in wood can either be chemically/physically adsorbed into the cell walls (bound water) or be liquid water in the lumina of the cells (free water). When a board has adsorbed all the water that can be physically bound, it is said to be at fiber saturation point (FSP), and any additional water will be held as free water. Between the FSP and the oven-dry state is where the gain or loss of water causes dimensional change in wood. Environmental conditions of temperature and relative humidity determine the MC of wood. The MC under a given set of conditions is referred to as the equilibrium moisture content (EMC), at which point the moisture taken up by the wood is equal to the moisture lost.

In the case of normal wood, changes in MC between 0 percent and FSP give rise to radial and tangential strain; generally, tangential strain is roughly twice the radial strain, and strain in the longitudinal direction is negligible. In juvenile wood, however, longitudinal strain can approach that of radial strain, resulting in massive changes in board length and influencing stress on bonded joints.

Wood Adhesives

Adhesive interaction with wood

This complex nature of wood makes it likely that adhesive interactions with wood will be complex. In addition, the wide variety of adhesives used in wood bonding and the different joint types further increase complexity. Both wood and adhesive play important roles in controlling the bond formation process and ultimate performance of the assembly.

A key issue in wood bonding is proper control of penetration so that it is sufficient to develop a good adhesive–wood interaction but not so excessive that it leads to an adhesive-starved joint. Penetration into wood can involve either flowing into the lumina and cracks or migrating into the cell wall. To aid in distinguishing these two fundamentally different processes in this paper, the former will be referred to as penetration and the latter as diffusion. Penetration into lumina is controlled by grain angle, density, wood species, and wood surface preparation. Grain angle is very important: In bonding to the edge or face of wood pieces, adhesive penetration is limited if the surface is exactly parallel to the grain. However, being exactly parallel to grain is unlikely (see Figure 1G, for an extreme example); thus, adhesive can flow into many open lumina, leading to deeper penetration than when parallel to grain. This flow into lumina away from the surface can provide stronger bonds through mechanical interlocks, but it also removes adhesive from the bondline. If too much adhesive flows into the wood, over-penetration occurs and insufficient adhesive remains at the bondline (i.e., a "starved bond"). Excessive flow into lumina can be a large problem for butt, scarf, and finger joints. Excessive flow can cause bleed-through on veneers, especially if they have large vessel elements (Christiansen and Knaebe 2004). The ability of the adhesive to penetrate into wood is species dependent and is generally greater for earlywood than for latewood, especially in softwoods and for vessel elements in hardwoods. For example, adhesives more readily penetrate into a pine board, such as loblolly pine (*Pinus taeda*), than they do into a hard maple board, such as sugar maple (*Acer saccharum*), because of the larger median cell lumina of the earlywood cells in pine. Penetration of heartwood is generally more difficult than it is for sapwood because heartwood can have aspirated pits and higher extractives, decreasing its porosity. Many adhesive studies are done on sapwood; thus, bonding of a wood species can be more difficult than the literature indicates if the wood surface is heartwood.

For penetration to take place, the adhesive needs to wet (intimately cover) the wood surface. Thus, freshly prepared surfaces from mechanical planing or hand sanding are better for bonding because the adhesive better wets the surface (River et al. 1991). On the other hand, abrasive planing often crushes surface cells, with poor bond strength resulting from a mechanically weak boundary layer. Some wood species, such as teak (*Tectonia grandis*), are hard to bond because they have oily extractives that limit the ability of the adhesive to come into contact with the wood and therefore provide a chemically weak boundary layer. Solvent-wiping the surfaces of oily wood improves bond strength. In contrast, a wood without such oils, such as Afrormosia (*Afrormosia elata*, known by some as "poor man's teak") is more easily bonded. Thus, correct identification and understanding of the wood to be bonded can reduce bonding problems.

Adhesive properties also greatly influence their interaction with wood. Key factors include the ability of the adhesive to wet the surface and penetrate/diffuse into the wood. Although most wood adhesives are water-borne and wet surfaces poorly, the few that are not, such as polymeric diphenylmethane diisocyanate and epoxies, can wet wood surfaces well, especially those that are not freshly prepared. As for ability to penetrate/diffuse into the wood, most adhesives will fill cell lumina at and near the surface, creating a mechanical interlock. Additionally some actually penetrate into cell walls, creating micro-sized mechanical interlocks; those that penetrate may also alter the cell walls' swelling capacity and modify their mechanical strength.

Another important factor is their ability to fill gaps between the surfaces. Those adhesives that are cured by moisture, including isocyanates and polyurethanes, generally prefer tight fitting joints because the curing process generates gas bubbles that can weaken the joint of thicker adhesive bondlines. The formaldehyde-type adhesives using phenol, resorcinol, urea, melamine, and combinations of these chemicals can tolerate somewhat thicker bondlines. Poly(vinyl acetates) are generally used in tight bondlines because shrinkage is an issue in thicker bondlines. For gap-filling ability and lower clamping pressures, epoxies are preferred.

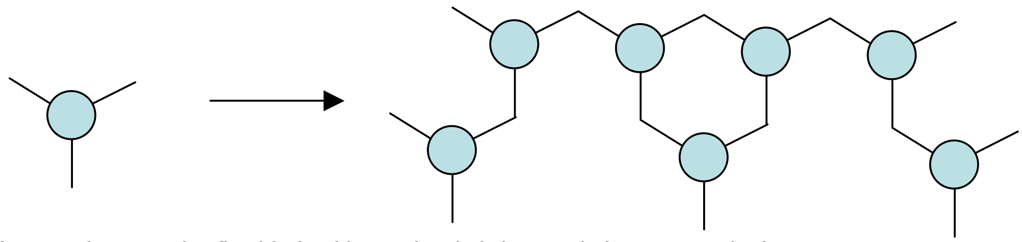
Wood adhesives also exhibit a variety of means of setting, including polymerization and loss of water solvent. Some that polymerize, such as epoxies and phenol-resorcinol-formaldehydes, cure (polymerize) at ambient temperatures, whereas most others require heat or moisture. Moisture-cured adhesives need to have sufficiently wet wood to cure within a reasonable time. In addition to setting by polymerization, most wood adhesives are water borne and set by the wood absorbing water. Those adhesives that set by water removal may cure more slowly when bonding heartwood than sapwood because the former absorbs water more slowly than the latter. Bonding conditions are greatly influenced by type of product being produced. For example, plywood bonding generally requires a different adhesive than does oriented strandboard production because of adhesive application conditions and the amount of compression that the wood experiences for bringing the surfaces together during bonding. For furniture, many phenolics are undesirable because of their dark color and slow setting speed. The traditional poly(vinyl acetate), or white glue, is often being replaced by moisture-cured isocyanates because the latter have better durability. The selection of adhesive depends upon wood substrate, type of joint, bonding conditions, and use conditions for the bonded product.

Bond durability

Wood products are expected to have a long lifespan—over 100 years is fairly typical of wood products, whether used in buildings or furniture. Thus, adhesives need to very "durable." As noted by Kamke (2006), we use the term durability but often do not define service conditions and service life. Service life is generally easier to define—that of inexpensive furniture and cabinets is usually measured in tens of years, while that of structural elements and fine furniture is measured in hundreds of years.

Service conditions involve both applied loads and internal forces as generated by changes in temperature and wood MC. For a beam, bookcase shelf, or vertical member, the load applied to the wood product can generally be calculated and the bonded product tested for its performance. This allows the manufacturer to determine the suitability of the wood product for its load-bearing ability. On the other hand, internal forces generated by the setting process of the adhesive and moisture level changes are harder to quantify, making it more difficult to know the true service life. The shrinking of wood as it dries and the expansion as it picks up moisture are well known, and overall dimensional changes can be measured. For wood to swell and shrink means that much of the water absorption by the cell wall goes into making the cell walls increase in thickness by expanding outward from the lumen during swelling and the opposite during drying. However, even knowing the expansion of the bulk wood, converting the swelling data into actual internal forces on the bondline is difficult. How much of the dimensional change is mitigated by stress relaxation in the wood and how uniform is this stress given that earlywood should have different expansion and contraction values than latewood? How are the forces distributed given the anisotropic nature of wood? How much of the stress is distributed through the adhesive? How much internal force originally exists in the bondline from normal volume shrinkage of the adhesive during the setting process, from loss of the solvent, or the polymerization process? Changes in wood MC while in service are difficult to prevent, but the best practice to maintain the wood during bonding at a moisture content near the average level the product will experience during use.

(A) In-situ polymerized – rigid, multifunctional oligomers that highly crosslink, but can diffuse into cell walls.



(B) Pre-polymerized – flexible backbone that lightly crosslinks or extends during setting.

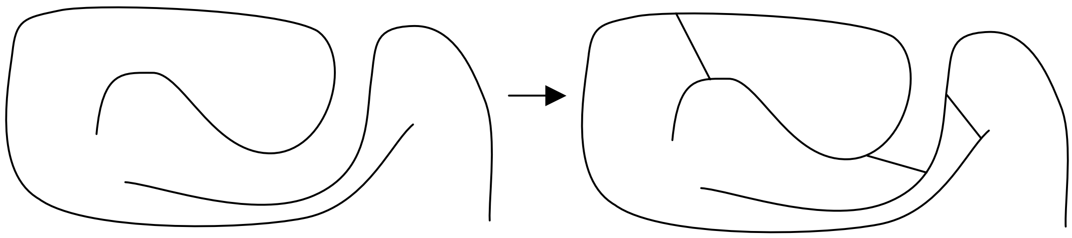


***Fig. 2**. Wood adhesives can generally be classified as either (A) those that are in-situ polymerized where the applied adhesive consists of small molecules that polymerize to form large molecules during the adhesive setting process or (B) those that are pre-polymerized and are often crosslinked during the adhesive setting process.*

Although these questions may not be readily answered, we can better understand the performance of wood bonds if we understand how two general classes of adhesives deal with internal forces (Figure 2). One class, which represents the largest volume of wood adhesives, includes those that are formed from *in situ* polymerization. These adhesives are made of rigid monomers that are often highly crosslinked, yielding an even more rigid cured adhesive, and include the formaldehyde-cured adhesives made from phenol, resorcinol, urea, and melamine, and epoxy adhesives. If the adhesives are rigid, how do they cope with dimensional changes of wood as moisture level changes, especially in exterior exposure? Many of these adhesives are made from chemicals that are known to stabilize the wood by making it more rigid and reducing dimensional change with changing moisture levels. Thus, the difference in dimensional changes between the bulk wood and the bondline is spread through the wood in the interphase region. The other class of wood adhesives is the pre-polymerized adhesives. These generally have a flexible backbone that is often crosslinked during the curing process and include poly(vinyl acetate), polyurethane, emulsion-polymerized isocyanate, and protein. These materials usually have the ability to adjust to dimensional changes of wood by spreading the strain through the adhesive layer. Thus, general comparison of adhesive mechanisms should include evaluation of the polymer morphology, its physical properties, and its mode of interaction with wood.

In the manufacture of panel products, most adhesives are of the *in situ* polymerization class. However, for furniture manufacture, both classes of adhesives are used. The bonds generally are strong enough to give wood failure, although most poly(vinyl acetate) or white glue is not crosslinked, so under high moisture conditions the adhesive softens as the swelling stresses increase, leading to bond failure.

Changes in Wood Supply

In the past 100 years, wood characteristics within species for many American woods have changed as we have moved from cutting old-growth trees to harvesting younger material. Wood quality within a species is governed by factors such as growth rate of the tree (rings per centimeter in a board), overall health and vigor of the tree as affected by silvicultural practices, and final age of the tree when it is cut. Directly related to these are factors such as presence of knots, completion of heartwood formation, proportion of juvenile wood, and greatest width of possible boards cut from the tree. The quality of boards cut from a single tree primarily depends on the proportions of juvenile, reaction, and mature wood. With these changes in tree and board characteristics come changes in the cost of material; as demand exceeds the ready supply of material of a given quality, price increases, often encouraging bold entrepreneurs to explore the use of alternative species.

When attempting to shift to a new species, people are generally seeking an identical wood that is, for the moment, available at a competitive price. Of course, "identical" means different things to different people; some may want the white color of high-quality sugar maple and care little about its strength, whereas others may be seeking only inexpensive, strong secondary wood for hidden parts of a large piece of furniture. Thus, if one's requirements are well defined, a cheap and plentiful alternative may be found by matching the relevant wood properties of the commonly used material with the alternatives available.

This means that a wood user must specify the exact characteristics of interest (e.g., correct scientific name, density, rings per centimeter, number and size of knots), particularly when grading standards are not available or not uniformly enforced (as with many tropical hardwoods). For example, a furniture maker might specify teak, thinking of good, old-growth Burmese *Tectona grandis* (Wood Identification 2007). Their supplier, however, may deliver a load of so-called Brazilian teak (cumaru) *Dipteryx odorata*, which, apart from being nothing like genuine teak in its wood properties, is also an endangered species. Or the supplier may provide Costa Rican teak, which, although it is in fact botanically the correct species (being plantation material), has growth rings of 20 to 26 mm, not at all what the furniture maker had in mind when the order was placed. Thus, to manage the changing wood resource, furniture makers must accurately and precisely specify their needs in contracts with wood suppliers.

Also of critical importance to the issue of wood quality and changing supply is the issue of moisture content in the product. With the relative increase in proportion of juvenile wood in many species, and the ever-decreasing diameter of trees being harvested, control of MC is even more important now than before, because the material itself is more likely to change unevenly with changes in MC. Also, targeting the MC of the wood at the time of processing to the expected MC at the site of end-use will minimize dimensional changes. Likewise, the correct adhesive must be chosen for the specific wood, at the correct MC, and on the correct type of joint for successful application. Targeting all these processes accurately and correctly will take best advantage of the properties of the raw material and will minimize the likelihood of product failure.

Relatively little work has been published on the effects of juvenile wood on mechanical properties of an adhesive–wood bond. The rising supply of wood with higher proportions of juvenile wood and this lack of published literature prompted researchers and the United States Forest Service's Forest Products Laboratory to conduct some basic experiments (Jakes et al. 2007). Using ponderosa pine (*Pinus ponderosa*), compression shear block specimens were constructed using two adherends and a phenol resorcinol formaldehyde resin. Three groups of shear block specimens were tested, one with two mature wood adherends (MM), one with two juvenile wood adherends (JJ), and one with one juvenile and one mature adherend (JM). Interestingly, despite juvenile wood's perceived inferiority to mature wood, the JJ group was significantly stronger than both the MM and JM groups. However, the JM group produced the weakest bonds. The resulting compression shear strengths were 11.3 ± 0.3, 9.1 ± 0.3, and 7.3 ± 0.5 for JJ, MM, and JM groups, respectively, where the uncertainties are 95% confidence intervals on the mean. An analysis of the corresponding load–displacement curves revealed that the JJ group had over twice the work to failure than did the MM and JM groups, and the overall bond stiffness was lower. This work suggests that during the loading of a compression shear block specimen, the more compliant JJ specimen is able to dissipate twice as much energy during loading than is the MM specimen, and the result is an overall stronger bond. However, if mature wood is bonded to juvenile wood, the opposite affect occurs, and a weaker bond results. This work demonstrates that juvenile wood, when present in adhesive–wood bonds, will affect bonding, but not necessarily negatively in all cases.

Conclusions

Understanding both the nature of wood and adhesive interactions with wood is important to making a wood product with an acceptable service life. Wood supplies are changing, with a greater proportion of earlywood or juvenile wood than in the past, which can have a variety of effects on end use. Furthermore, these changes can greatly affect the interaction and performance of adhesives with wood. Two main classes of adhesives, based upon their polymer chemistry and morphology, each respond differently to changes in wood moisture levels. The wood user, especially the furniture maker, needs to be cautious and informed in wood selection or may need to consider altering the design and adhesive used to accommodate these changes.

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Emission of TVOC from materials applied in the furniture manufacture

Przemyslaw Gaca[2]

Abstract: *Volatile organic compounds from particleboards, plywood, finish foil, furniture elements and polyvinyl acetate (PVA) adhesive were measured in the small chamber. Samples emission properties in small chamber was given both qualitative and quantitative characterization with the use of GC/MS system. Volatile organic compounds (VOC) emissions were very high for a furniture element – veneered particleboard and raw pine wood. Relatively high emission was observed also in the case of child bed sample where aromatic derivatives were detected. For plywood, PVA adhesive and finish foil VOC emission was found low and met the IKEA requirements. Significant differences, both qualitative and quantitative, between unfinished and finished samples were observed. There were also conducted TVOC emission tests from different kinds of lacquers put on wood species. High emission levels of TVOC for polyurethane and chemo-hardening lacquer were determined.*

Introduction

The state of the environment is, according to World Health Organization (WHO, 1989), the main reason of health problems of inhabitants, connected with the increase of tumor sicknesses and other civilization diseases. Principally, one of the most important things is indoor air quality (IAQ). It is well known that people spend most of their time inside buildings and apartments. Lower immunity of children and the elder people cause that they are especially exposed to the influence of hazardous compounds present in the air of rooms.

In order to improve indoor air quality, it is generally advised to control sources by reducing pollutant emissions, particularly by the selection of "low emission" building products (Yrieix, 2004 and Zellweger, 1997). Other important thing is creating the labeling schemes which are available in many countries (Wolkoff, 2003).

The furniture - the main indoor equipment of rooms intended for permanent stay of people – should be under constant monitoring, taking into account their manufacture from different materials with the application of synthetic chemical compounds which could cause the emission of volatile organic compounds.

In the Department of Environmental Protection and Wood Conservation of Wood Technology Institute we carry out the research on emission of harmful compounds such as formaldehyde and VOC from materials applied in the furniture manufacture.

The main objective of our studies was the VOC emission research from the chosen materials applied in the furniture manufacture like particleboard, plywood, finish foil, raw pine wood finished furniture elements and polyvinyl acetate glue (PVA). We also examined 4 types of lacquers which were applied into the surface of six wood species and for comparison into the surface of glass plate and fibreboard.

Materials and Methods

In our research we applied following furniture materials: particleboard, plywood, finish foil, finished furniture elements, raw pine wood materials and polyvinyl acetate (PVA) adhesive.

All determinations of the VOC emission from indoor materials and products were conducted by the chamber method (chamber: 0,025m^3 capacity). The chamber was made of glass and was conditioned for proper temperature and relative humidity according to the following conditions:

- air temperature in chamber: 23°C±1°C
- relative humidity of air: 45%±5%
- air exchange in chamber: $1h^{-1}$
- filling up the chamber with the tested material: $1m^2 \cdot m^{-3}$

The chamber was equipped with suitable accessories such as inlet port for airflow and an port for temperature/humidity measurements. The climatic conditions were continuously monitored by the PC software. This chamber was placed in the air-conditioned room.

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The sorbent tubes constructed of glass were used for collection of emissions. The tubes were packed with one layer of Tenax TA (120 mg). Before usage these tubes were conditioned at 260°C for 24 hours and exposed to the flow of helium at 15 ml·min^{-1}.

The sampling flow rate was set at 100 ml·min^{-1}, collecting for analysis 1 liter of air for a sampling period of 10 minutes. After collection of two air samples, chemical compounds adsorbed on traps were thermally desorbed on-line into the chromatographic column.

To characterize and quantify the target compounds the gas chromatograph/mass spectrometer (GC/MS) system was utilized. Target compounds were identified by the retention time, full mass spectra and/or by the presence of some ion fragments. Quantitative evaluation was achieved by comparing the chromatogram peak area of each compound to the corresponding peak area of an internal standard. Analysis was performed on the Thermoquest Finnigan Trace 2000 gas chromatograph coupled with Finnigan Trace mass spectrometer.

Analysis conditions of samples were as follows:

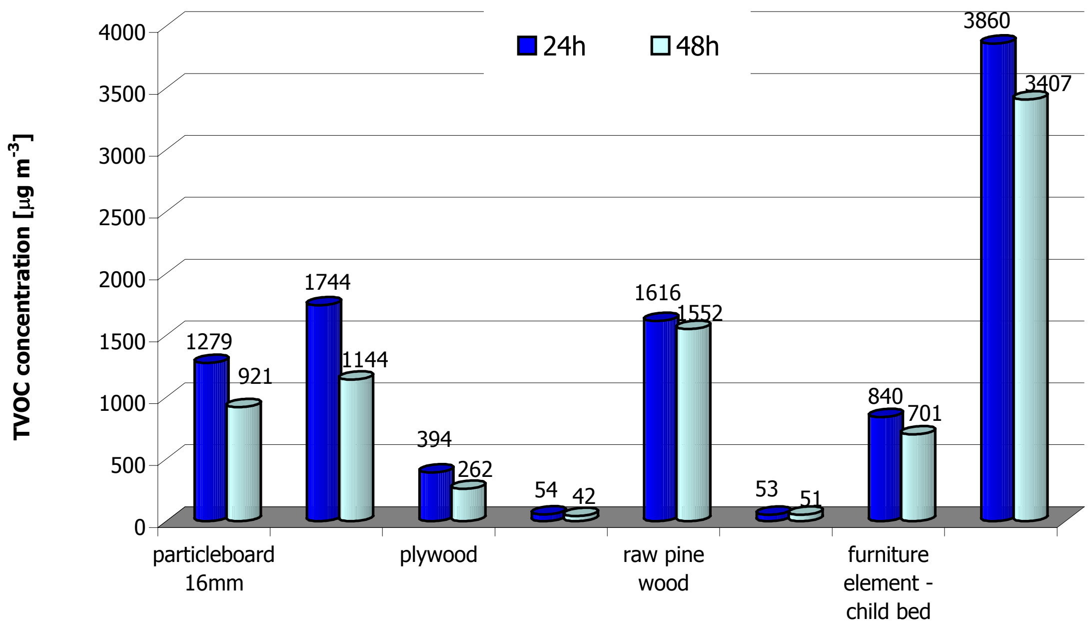
- GC/MS interface temperature - 220°C,
- detector MS - 220°C,
- oven temperature - 35°C (4 min) 5°C/min --> 140°C (0 min) 12°C/min --> 240°C (3 min)
- carrier gas flow - 1ml·min^{-1}
- injector - thermal desorber, 280°C for tube desorption and 280°C for trap desorption

Results

The total concentration of volatile organic compounds (TVOC) emitted from the tested materials is presented in Figure 1 (the test results are expressed by the mean value of two air samples and did not differ by more than 10% from each other).

The highest emission of VOC compounds was observed in the case of the furniture element – three layered particleboard veneered with natural birch veneer - and resulted 3 860 μg·m^{-3} after 24 hours and 3 407 μg·m^{-3} after 48 hours. Also TVOC emission from raw pine wood (1 616 μg·m^{-3} after 24h, 1 552 μg·m^{-3} after 48h) did not fulfill the IKEA requirements and was above 1 200 μg·m^{-3}. For other materials the total concentration of

Fig. 1. *The TVOC emission of selected furniture materials*



the volatile organic compounds after 48 hours was below the IKEA requirements. In the case of finish foil and polyvinyl acetate adhesive there was observed very low emission of volatile organic compounds (relatively: 42 μg·m^{-3} and 51 μg·m^{-3} after 48 hours). Relatively high VOC emission from the child bed sample: 840 μg·m^{-3} after 24 hours and 701 μg·m^{-3} after 48 hours, should be taken into consideration. As it was mentioned above, children have lower immunity, so such a level of hazardous compounds emission could cause different health effects like eye, nose, and throat irritation, headaches, loss of coordination, nausea, damage to liver, kidney, and central nervous system and even cause cancer (Rutkowska I., 1995)

The results of the measurements of the hazardous compounds concentration showed significant qualitative and quantitative differentiation of group VOC compounds emitted from the tested materials.

In Figure 2, the percentage emission distribution of the group compounds emitted after 48 hours from the tested materials is presented.

Aldehydes were the main part in the three samples: particleboards (thickness: 16mm and 25mm) and plywood: 59%, 47% and 30%, relatively. However, in the case of plywood aliphatic compounds were the most important part – 65%. They also played the principal role in the other 2 samples: PVA adhesive (45%) and furniture sample – veneered particleboard (54%).

The large participation of terpenes emitted from raw pine wood sample, above 96%, was not any surprise. It is natural, that these compounds are the main elements of raw pine wood. The child bed sample made of pine wood had also high amount of terpenes (37%). But the most worrying fact is the presence of aromatic compounds. Almost of all compounds emitted from this furniture element have aromatic derivation. It can be explained by the fact of application of varnish, from which these compounds are emitted.

The amounts of alcohols and ketones were relatively low and their participation in the main distribution of group compounds was below 10% for all air samples collected from the chamber filled with the test materials.

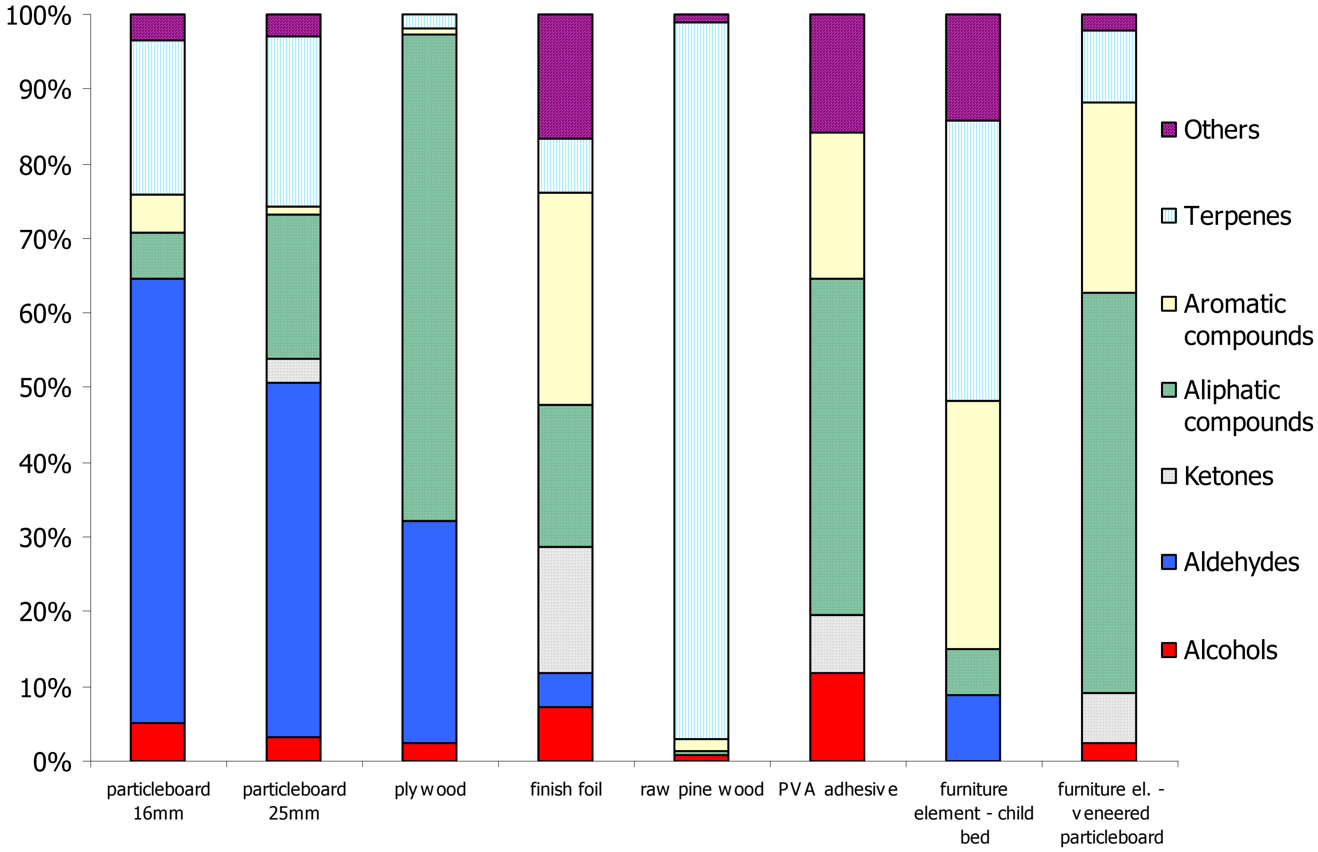


Fig. 2. *Percentage emission distribution of the group compounds (after 48 hours)*

We also conducted the determination of VOC emission from four types of lacquers put on the different wood species and for comparison on the glass plate and fibreboard. The results of tests are presented in the Figure 3.

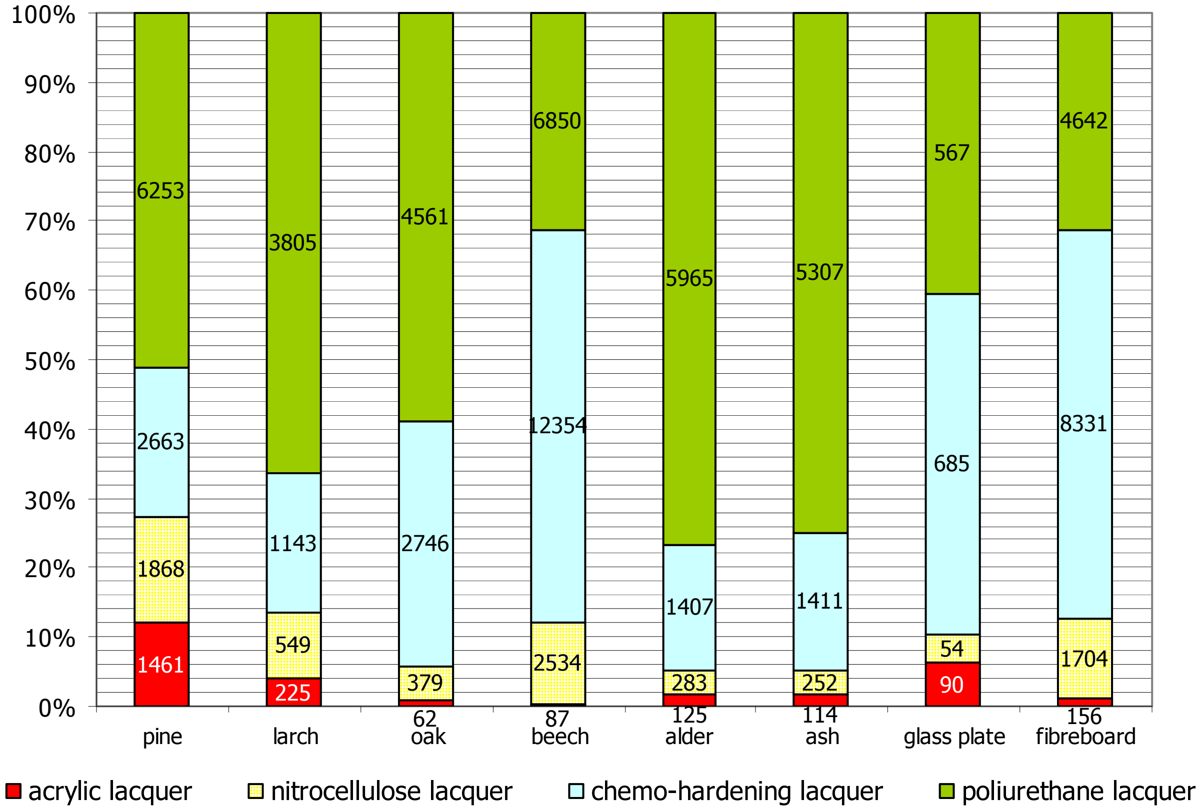


Fig. 3. *Total VOC emission from different kinds of lacquers*

The highest VOC emission was observed from the samples lacquered with polyurethane and chemo-hardening lacquers. The VOC emission from the glass plate lacquered with polyurethane lacquer was only 567 $\mu g{\cdot}m^{-3}$. In the case of wood species and fibreboard there were very high emission levels in the range of 3 805 $\mu g{\cdot}m^{-3}$ (larch wood) and 6 850 $\mu g{\cdot}m^{-3}$ (beech wood). Applying of chemo-hardening lacquer on the beech wood sample gave outstanding high TVOC emission value above 12 000 $\mu g{\cdot}m^{-3}$. There was also high emission from fibreboard (above 8 000 $\mu g{\cdot}m^{-3}$) on which was put this kind of lacquer. Other samples were characterised with much lower TVOC concentrations (from 685 to 2 746 $\mu g{\cdot}m^{-3}$.). The nitrocellulose and acrylic lacquer applied at the surfaces of wood species and glass plate and fibreboard had levels of TVOC mostly not exceeding 600 $\mu g{\cdot}m^{-3}$. The higher emission value for the pine sample is due to the presence of terpenes compounds. It is worth to notice the high concentration of TVOC compounds in the air samples of beech species: 2 534 $\mu g{\cdot}m^{-3}$.

Discussion and Conclusions

The research confirmed the need for qualitative and quantitative characterization of indoor pollutants like volatile organic compounds emitted from furniture and materials applied to their manufacture.

Most of the test samples were not dangerous to the indoor environment in the respect of the amount of emitted volatile organic compounds. The test materials samples emitted VOCs at the level below 1 200 $\mu g/m^3$.

But the thorough qualitative and quantitative research indicates the appearance of some dangerous group compounds which can be present in the air of the indoors. It should be taken into consideration, that unfinished wood boards, like particleboards, are the main source of aldehydes emission. The emission research of the child bed showed relatively high concentration of aromatic compounds which cannot be present in such product. Such compounds could cause different health effects. Nielsen et al. (1994) showed the principles of the evaluation of health effects of VOCs emitted to indoor air.

High emission of terpenes in the case of raw pine wood was confirmed (Jensen, 2001).

It should be noticed that the furniture element – particleboard veneered with natural birch veneer - was characterized by the very high emission of volatile organic compounds. Over 50% of all emitted compounds were aliphatic derivatives. ¼ of all compounds were aromatic derivatives. This large amount of VOCs found suggests that there were applied materials (like adhesives, veneers) characterized by the high content of organic solvents.

Very high emission levels of TVOC characterized the wood samples lacquered with polyurethane and chemo-hardening lacquers with a maximum value of 12 354 $\mu g/m^3$ from beech samples covered with the second varnish. The high differences between glass plate (relatively, 567 and 685 $\mu g \cdot m^{-3}$) and other wood samples and fibreboard (relatively from 1 143 to 12 354 $\mu g \cdot m^{-3}$) may result from the interactions between compounds present in the structure of samples and compounds present in the lacquer. The levels of volatile organic compounds emitted from water-based acrylic lacquer and nitrocellulose lacquer were much lower not exceeding 600 $\mu g/m^3$ in most cases.

In this study there were tested only several materials used to furniture manufacture. Taking into account both qualitative and quantitative differentiation, it is imperative to continue the research of indoor air quality. It is necessary to further develop the knowledge on emission of volatile organic compounds from materials applied in manufacture of furniture. It may contribute to identification and possible extermination of the emission sources and finally to improve indoor air quality and decreasing the risk of disadvantageous indoor environment influence on human health.

Acknowledgement

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Linear and high speed rotational wood welding: Wood furniture and wood structures

Antonio Pizzi[1]

Abstract: *Linear vibrational surface-to-surface wood welding and high speed rotation-induced wood dowels welding, without any adhesive, rapidly yields wood joints of considerable strength. The mechanism of mechanically-induced wood welding is shown to be due to the temperature-induced softening and flowing of some amorphous, cells-interconnecting polymer material in the structure of wood, mainly lignin, but also of hemicelluloses and consequent high densification of the bonded interface. In linear wood welding wood species and grain direction are important parameters. In rotational wood welding wood species, relative diameter differences between the dowel and the receiving hole, dryness of dowel and substrate, and pressing time are parameters yielding significant strength differences, while relative orientation of the fibre grain of the dowel in relation to the fibre grain of the substrate, relative rate of rotation within a limited range and the use of rough or smooth dowels did not have any significant influence. X-ray microdensitometry and scanning electron microscopy are also used in the presentation to show the advantages and limits of wood dowel welding by high speed rotation. The use of dry dowels inserted hot or cold in the substrate after preheating them at high temperature (100°C) yielded consistently better results than that obtained with PVAc gluing. The joint construction also afford a level of water resistence of the doweled joint, although not to the level to produce exterior-grade joints. High-speed dowel rotation welding was used to manufacture several furniture pieces as well as a full-scale suspended floor, hence a building construction structure. This was coupled with obtaining a more light weight floor assembly at equal stiffness by maximizing the rigidity of the suspended floor while minimizing the number of timber planks used to build it, and maintaining its vibration frequency high and its level of vibration low. Deformation under 4 points static load of the floor was carried out to determine displacement under load and the floor vibration behaviour was determined by the use of accelerometers. The properties of the floor and the own frequency measured do satisfy well in excess all the requirements specified by Eurocode 5. The lower is the value of the maximum vibration rate $V_{max,}$ the better is the floor. The floor measured $V_{max,}$ value is low rendering the dowel-welded floor competitive with traditional floors. Both furniture and the structural suspended floor were manufactured by using either fixed-base or hand-held electric drills, rendering the use of the technique particularly easy and inexpensive.*

Introduction

Mechanically-induced friction welding techniques which are widely used in the plastic and automotive industries have recently been applied also to joining wood, without the use of any adhesive. These techniques work by temperature-induced softening up to melting of some wood components and forming at the interface between the two wood surfaces to be joined a composite of entangled wood fibres drowned into a matrix of molten wood intercellular material, such as lignin. Evidence that the temperature induces intercellular material softening and melting is provided by the complete loss of the cellular structure of wood and its high densification at the welded interface.

Linear mechanical friction vibration (Fig. 1) has been used to yield wood joints satisfying the relevant requirements for structural applications by welding at a very rapid rate. Cross-linking chemical reactions have also been shown to occur by CP-MAS ^{13}C NMR. These reactions, however, are relatively minor contributors to the strength of the welded joint during the very short welding period but acquire more importance after the welding period.

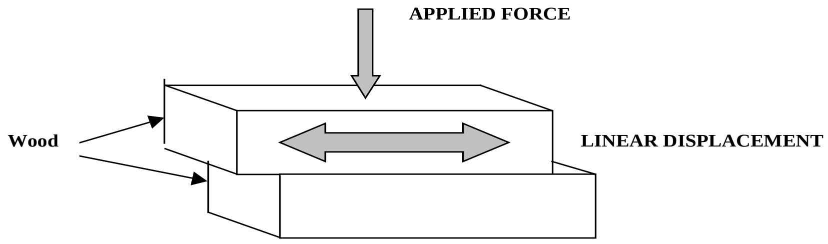

Fig. 1. *Vibrational movements of two solid wood surfaces during linear vibration wood welding.*

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Assembly techniques by dowel insertion, with or without the use of adhesives, are common and particularly useful in joining solid wood in the furniture and wood joining industries. In particular, in the furniture assembly industry the traditional manner to insert dowels in good quality solid timber furniture joints is by percussion, pneumatic or manual means. Adhesives, generally poly(vinyl acetate) (PVAc), are used, but not always. In the most general case, dowels of 10 mm diameter are inserted in a prepared hole of 9.5 to 9.75 mm diameter in the substrate. When poly(vinyl acetate) (PVAc) adhesive is used instead 10 mm diameter dowels are inserted in 10 mm diameter holes.

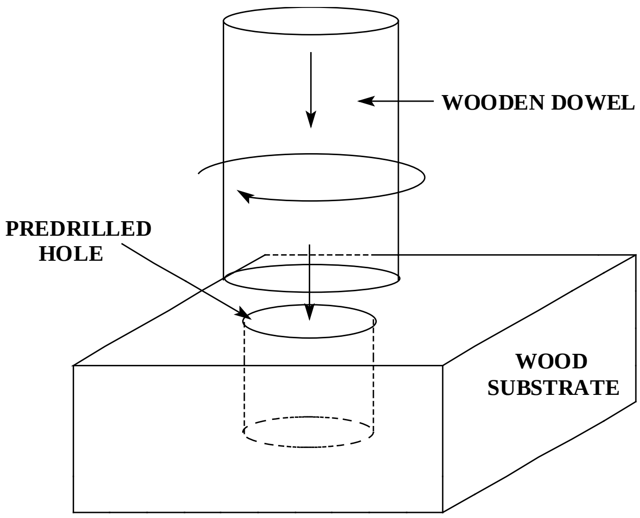


Fig. 2. *Schematic representation of the insertion of a cylindrical dowel in a predrilled hole in a wood substrate.*

Insertion of dowels by high speed rotation, however, cause an increase of temperature, and hence a series of physical and chemical events leading to wood welding comparable to that obtained by vibrational flat surface welding (Fig. 2).

This presentation review what exciting developments has been achieved up to now in this extremely new field of wood binding.

Linear Wood Welding

Linear welding of wood can gives bonding results satisfying the relevant standards, while orbital welding gives much lower results. Some of the parameters that influence welding of metals with the same type of equipment also influence wood welding. Thus, the influence on the final weld of the vibration welding time, the contact holding time after the welding vibration had stopped, the welding pressure exerted on the surfaces, the holding pressure after the welding vibration had stopped, and the amplitude of the shift imparted to one surface relative to the other during vibrational welding are of importance. Welding frequencies of 100 Hz are used. The joint tensile strength depends on vibration amplitude, showing some good bond strength for 3mm vibrational amplitude. The joint tensile strength depends on welding pressure, values of 2 to 2.3 Mpa giving the best results. The joint tensile strength depends on welding time, but less markedly than on welding pressure. In general combinations of 3 seconds welding time and 4-5 seconds holding time give strong joints presenting strength in excess of 10 and sometime of 11 Mpa. The relevant European Norm for these types of joints requires strengths equal to or higher than 10 MPa.

The strong joints obtained are not capable of satisfying specifications for exterior joints as they show relatively poor resistance to water. These joints can then only be considered for interior applications such as for furniture and for interior grade wood joints. Improvements in joint design can however improve their resistance to watrer, but never to exterior grade level. Furthermore the technique at this stage is only usable for solid wood joints and joints between premanufactured panels and not only those presenting the same type of characteristics as solid wood, such as plywood and oriented strand board (OSB), but also MDF and particleboard. The technique has considerable interest for its low cost and in the implementation of totally environment friendly wood joints in joinery and furniture manufacturing.

The mechanism of mechanically-induced wood vibration welding has been shown to be due mostly to the melting and flowing of some amorphous, cells-interconnecting polymer material in the structure of wood, mainly lignin, but also hemicelluloses. This causes partial detachment, the "ungluing" of long wood cells, wood fibres, and the formation of a fibres entanglement network in the matrix of molten cell-interconnecting material which then solidifies (Figs. 3 and 4) . Thus, a wood cells/fibres entanglement network composite having a molten lignin polymer matrix is formed. Figs. 3 and 4 show scanning electron micrographs showing the detail of the type of composite formed in the bondline of a solid wood joint. The interfacial composite formed has lost all the anatomical structure of wood and presents a density more than double than that of wood, this characteristic alone already contributing greatly to the increase in strength of the welded joint (Fig. 5).

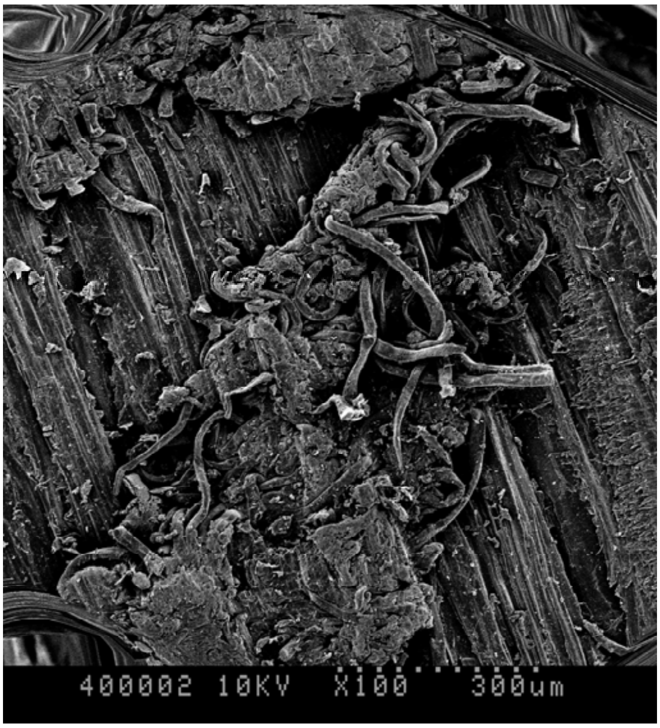


Fig. 3. *Scanning electron microscopy images of lignin fusion band at 100 magnification showing the entangled and detached tracheids, a fused intercellular lignin mass and tracheids and fibers immersed in the fused lignin matrix.*

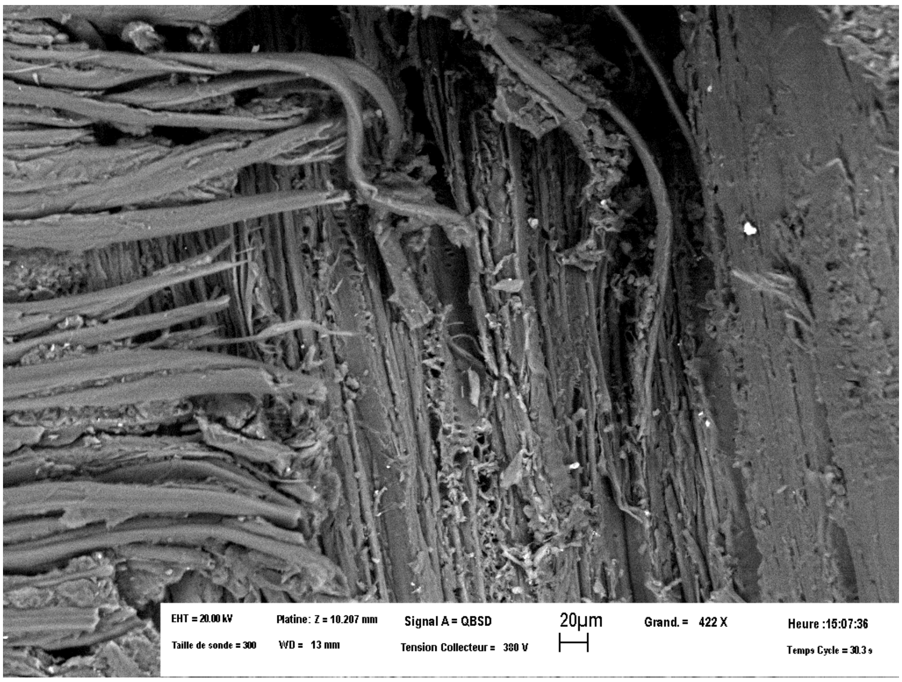


Fig. 4. *SEM micrographs of the surface of a dowel that has been tested: Fibres of the wall of the substrate hole welded to the underlying fibres of the dowel that run 90° cross-grain (422X magnification).*

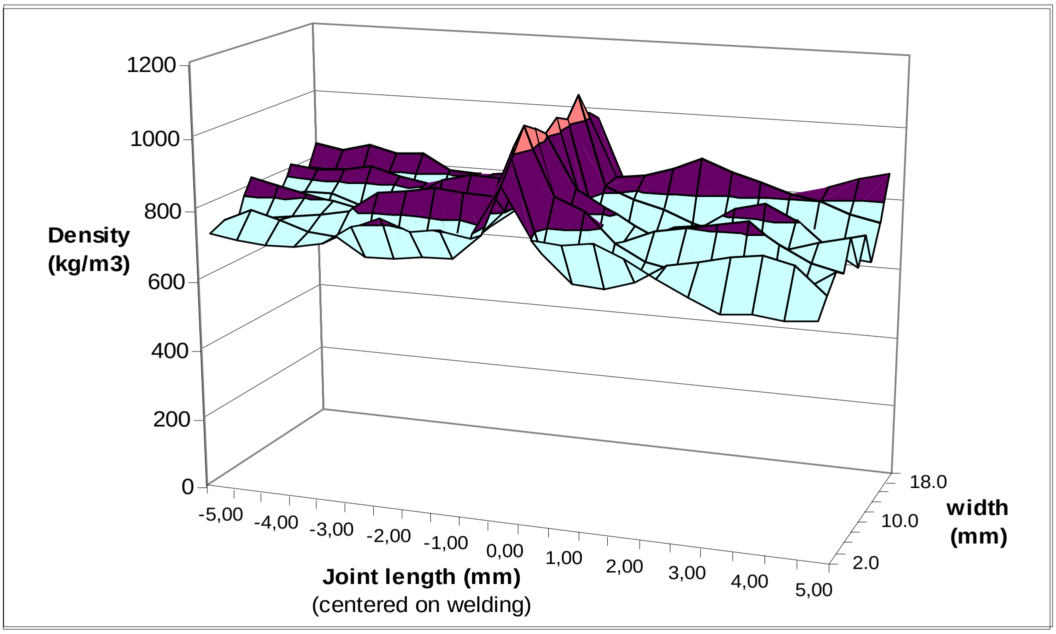


Fig. 5. *X-ray microdensitometry-derived three-dimensional wood density map of a beech wood joint clearly showing high density peaks at the interface where the wood is joined in relation to the lower density of the surrounding wood.*

During the welding period some of the detached wood fibres which are no longer held by the interconnecting material are pushed out of the joint as excess fibres. Cross-linking chemical reactions also have been shown and confirmed to occur (the most likely one of these identified by NMR appears to be a cross-linking reaction of lignin with carbohydrate-derived furfural and furfural self-polymerization). These reactions, however, are relatively minor contributors during the very short welding period. Their contribution increases after welding has finished, explaining why some holding time under pressure, after the end of welding, contributes strongly to obtaining a good bond.

Rotational Dowel Welding

High speed rotation-induced wood dowels welding, without any adhesive, rapidly yield wood joints of considerable strength. The mechanism of mechanically-induced high speed rotation wood welding is due, as already observed in linear vibration welding, to the temperature-induced softening and flowing of some amorphous, cells-interconnecting polymer material in the structure of wood, mainly lignin, but also of hemicelluloses and consequent high densification of the bonded interface. Wood species, relative diameter differences between the dowel and the receiving hole, and pressing time were shown to be parameters yielding significant strength differences; while relative orientation of the fibre grain of the dowel in relation to the fibre grain of the substrate, relative rate of rotation within a limited range and the use of rough or smooth dowels did not have any significant influence. X-ray microdensitometry and scanning electron microscopy were used to determine the limits of wood dowel welding by high speed rotation. The type of parameters that had an influence on strength indicated that the strength values obtained, although often rather high, were often due to welding of only a limited part of the dowels to the substrate. This is due to the forcing of the dowel into a truncated conical shape by the pressure of insertion and the consequent disruption of bonding in some areas. Notwithstanding this effect, the welded contact area is sufficient to yield strength results comparable to, or even slightly higher than those obtained by PVAc adhesive bonding. The use of dry dowels inserted hot or cold in the substrate after preheating them at high temperature (100°C) yielded consistently better results than those obtained with PVAc gluing.

Oven-dry dowels, insertion of hot dowels, cross-cut dowels, substrate holes of step-decreasing diameter as a function of depth, use of ethylene glycol or other compounds able to decrease the glass transition temperature of wood components have all been shown to contribute to improving weld joint strengths in a variety of less drastic conditions than the 10 mm/8 mm dowel/substrate hole diameter difference. The results showed that once the depth of the dowel is much greater than 15 mm, then almost all the conditions used improve the weld strength. This means that the proportion of area welded in relation to the tensile strength of the dowel itself is a determining factor. The greater this area the higher the strength, irrespective of the application conditions used. Thus, over a certain welded area the dowel breaks when tested in tensile, i.e. the joint is stronger than the dowel. Temperatures >180°C are obtained during the quick welding step with the temperature decreasing in less than a minute to 60-70°C (Fig. 6). The same chemical reactions as occurring in vibrational welding have been shown by solid state ^{13}C NMR analysis to also occur in dowel rotation welding. In dowel rotation welding the production of carbohydrate-derived furanic aldehydes is higher (a) from the wood material of the substrate in which the hole is pre-drilled rather than from the material of the wood dowel itself, (b) when the weld joint strength is good, and (c) when the rate of dowel insertion is higher.

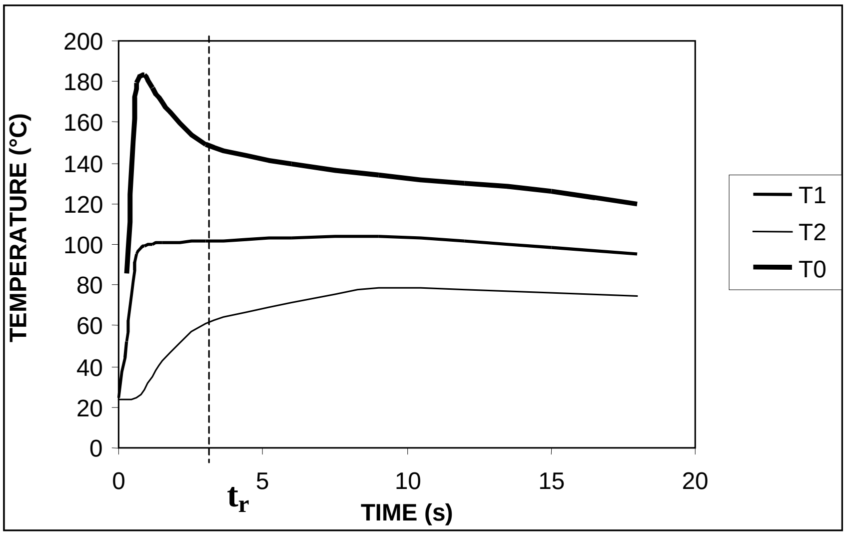


Fig. 6. *Experimental and extrapolated curves of temperature increase as a function of time for the case of 10 mm dowel diameter and 8 mm substrate hole diameter. T_1 experimental curve at 1 mm from substate hole, and T_2 experimental curve at 2 mm from substrate hole. T_0 is the extrapolated curve of the interface temperature. It increases as a function of time compared to the experimental curves for temperatures T_1 and T_2. t_r is the time at which rotation was stopped.*

The interactions between parameters found to be determinant in wood dowels welding by high speed rotation have also been evaluated. Of these the interactions that proved to be the most significant, in descending order, were rotation rate/dowel moisture content, followed by rotation rate/ethylene glycol, and finally at a lower level of significance the interactions rotation rate/dowel temperature, wood grain direction/wood species, and dowel temperature/wood species. Of the single factors, once the most detereminant factor already optimized in previous studies, namely the dowel/hole diameter difference has been fixed, the most significant were: wood grain direction, dowel's moisture content (dryness) and wood species. The optimized process yielded exellent strength results and equations able to predict the strength obtainable with certain variable values have been developed. The torque for the insertion of the dowel in the substrate hole was measured. In no cases the value of the torque needed for insertion was excessive and insertion is therefore easy. Wood joints composed of two pieces of timber held together by a dowel welded to both of them were assembled. The results obtained (Table 1), advantages and points of importance to improve in the technology have already been presented and discussed. X-ray densitometry of the samples prepared showed some interesting features (Fig. 7 a,b,c,d).

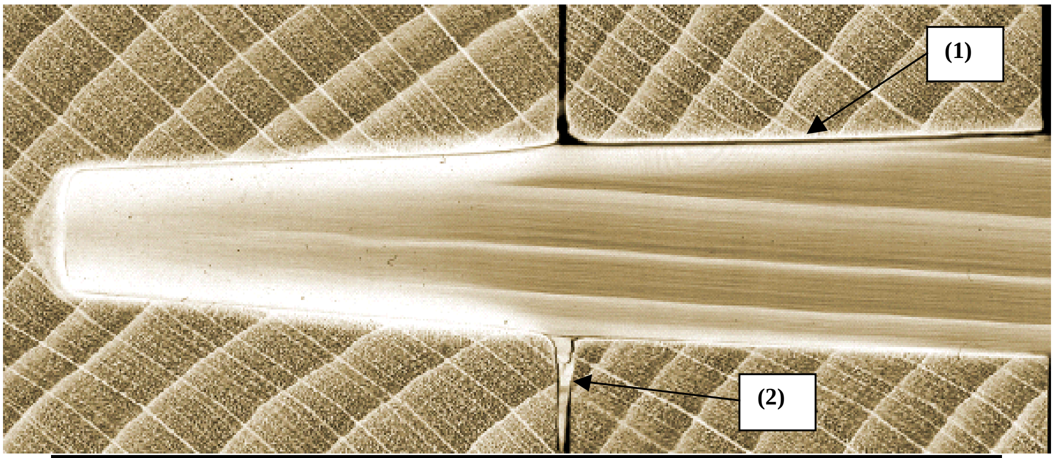
Table 1. *Tensile strength results of two wood blocks joined by a welded dowel.*

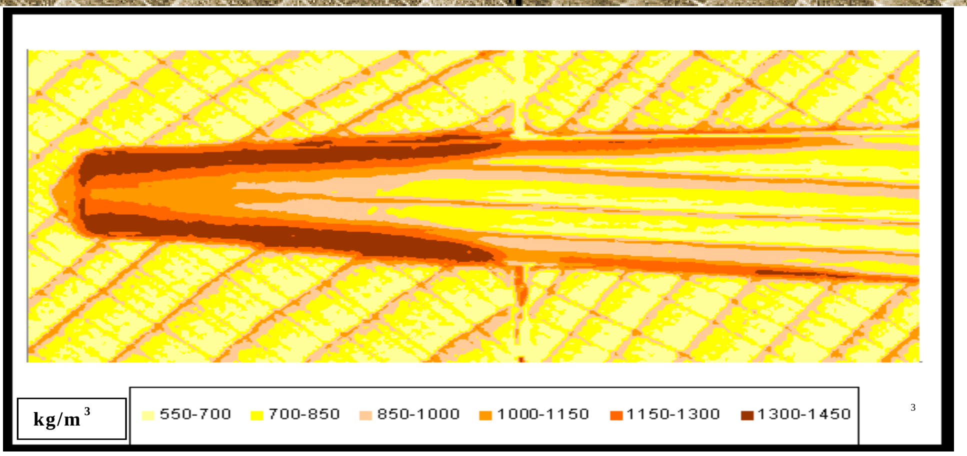
Test Number	Tensile Strength (N)
1	3584
2	3941
3	3669
4	3626
5	3632
6	3611
7	3771
8	4320
9	4231
10	4199
Average *(Standard deviation: 291)*	**3858**

Two-block wood joints were obtained by insertion and welding without adhesives of dowels by high speed rotation. Their strengths were better than that obtained by poly(vinyl acetate) (PVAc) gluing. X-ray-microdensitometry analysis showed that a complete welding of the dowel to the substrate occurred and that a perfectly tight joint was formed. Isolation of the flow material allowed CP-MAS ^{13}C NMR analysis of its composition with possibly low interference from the constituents from the substrate. The flow material appeared to be composed of hemicelluloses, apparently xylans, and lignin. Scanning electron microscopy coupled with the NMR analysis results showed that microfibrils of cellulose, in both amorphous and crystalline states, torn from the wood surface during welding as well as very small proportions possibly of recrystallized xylans and furanic compounds formed by heat transformation of the carbohydrates were present. Most important, the geometry of the dowel joint allowed the joint to maintain up to 88% of its initial tensile strength after 24-hour immersion in cold water and 15% after redrying (Table 2).

Table 2. *Resistance of 10 mm dowels to 24 hours in cold water – dowels inserted for 20 mm only*

	Dry strength (N)	Wet strength (N)
Welded dowels	1979$\pm$103	1746$\pm$153
PVAc-bonded dowels	1844$\pm$177	1286$\pm$224





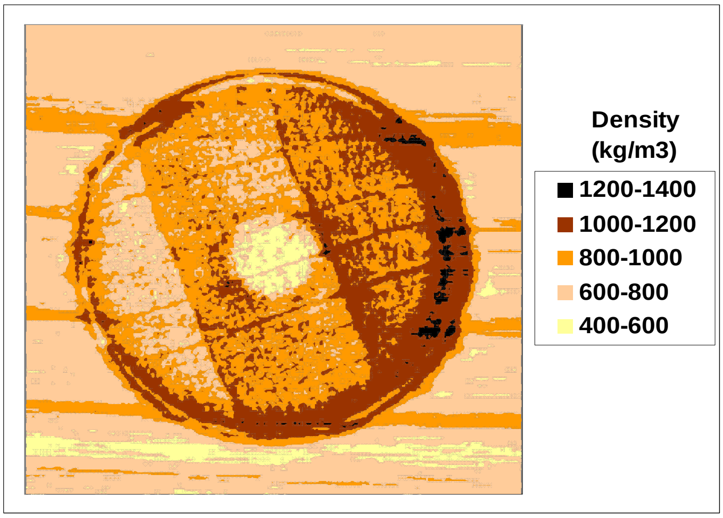


Fig. 7. *(a) X-ray micrograph of a fluted groove beech dowel inserted and welded to two separate blocks of beech wood. The white areas correspond to the areas of greater densification. For the microcrack indicated by arrow (1) welding has not occurred or welding has been broken. Arrow (2) indicates molten intercellular material which has seeped in the sfissure between the two blocks. (b) X-ray microdensitometry map in kg/m^3 of the same rotation welded fluted groove beech dowel inserted in a beech wood substrate (beech substrate tangential section). (c) X-ray micrograph of a fluted groove beech dowel inserted and welded to two separate blocks of beech wood when the two blocks are kept firm together during dowel insertion. Note that interface between the two blocks is tight and inviusible: it be only deducted from the different directions of the growth rings of the two wood pieces. (d) X-ray microdensitometry map in kg/m^3 of dowel section, sliced perpendicularly to the length of the dowel, of a welded fluted groove beech dowel.*

Two-block wood joints were obtained by insertion and welding without adhesives of dowels by high speed rotation. Their strengths were better than that obtained by poly(vinyl acetate) (PVAc) gluing. X-ray-microdensitometry analysis showed that a complete welding of the dowel to the substrate occurred and that a perfectly tight joint was formed. Isolation of the flow material allowed CP-MAS ^{13}C NMR analysis of its composition with possibly low interference from the constituents from the substrate. The flow material appeared to be composed of hemicelluloses, apparently xylans, and lignin. Scanning electron microscopy coupled with the NMR analysis results showed that microfibrils of cellulose, in both amorphous and crystalline states, torn from the wood surface during welding as well as very small proportions possibly of recrystallized xylans and furanic compounds formed by heat transformation of the carbohydrates were present. Most important, the geometry of the dowel joint allowed the joint to maintain up to 88% of its initial tensile strength after 24-hour immersion in cold water and 15% after redrying (Table 2).

Applications already developed

Dowel welding by high speed rotation was used to join two wood blocks. Strong joints were obtained. Dowel angle to the surface of the wood blocks to be joined had a marked influence on the mechanical performance of the joint. When the dowel was inserted at 90° to the substrate, the dowel worked only in shear. When introduced at an angle such as 30° or 45°, the dowel worked in shear plus tension, resulting in better joint strength. The joint almost always failed by dowel fracture. The interfacial welding dowel/substrate was almost always stronger and did not break. Short two-layer beams joined exclusively by a series of welded dowels were prepared, tested in shear according to structural standards, and their performance was compared to that of solid wood and of glulam beams of the same dimensions (Fig. 8). The short two-layer beams prepared for testing met the Eurocode 5 standard requirements when the best dowel insertion angle was used. Thus, while this technology is ideal for the assembly of furniture without any adhesive or nail, by satisfying the Eurocode 5 requirements for structural application, the technology was found to be of interest also in structural and civil engineering applications. Then 2-m-long two-layer wood beams were prepared, with the two layers connected exclusively by a series of welded dowels, and tested in bending (Figs. 9, 10, 11) and compared to double-nailes, with steel nails, beams of the same dimensions (Figs. 9, 10, 11). Their maximum failure strength and stiffness in bending were determined. These beams outperformed both nailed beams and glued-dowel beams (Figs. 12, 13). All the beams had the same length and conformation. The number of nails was double the number of dowels used.

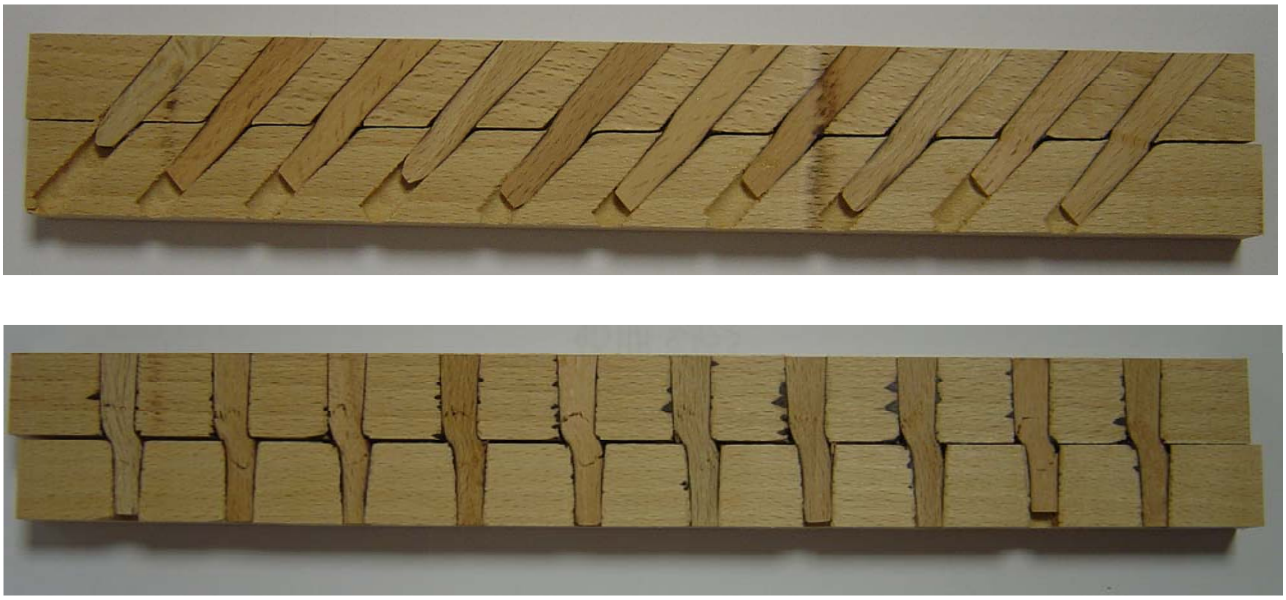


Fig. 8. *Section of 300 mm long 45° and 90° welded-dowel beams after testing according to EN 1380 and EN 383. Note the frequent double cracking of the dowels in the 90° case due to the force exercised being purely in shear. Note in the 45° case: (i) the absence of dowel cracks, (ii) the small dowel deformation due to the shear component of the force exercised on the joint, and the very short length welding breaks at the dowel/substrate interface this causes. (iii) the slight separation of the laminae due to the tension component of the force exercised. Note that in both cases the greatest majority of the welding is still intact. Note that the unfilled lower parts of the holes in the 45° case are not due to any dowel slippage but the joints was conceived for this length of dowel insertion.*

Fig. 9. *Two-meter long double-row nailed and single-row doweled two-layers beams. Photograph of the spruce beams after preparation.*

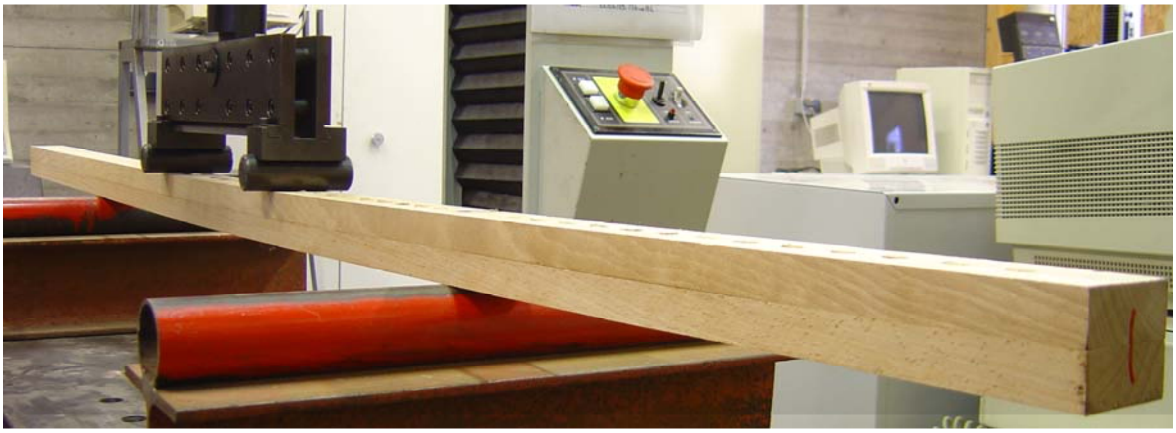


Fig. 10. *Sequence of events during the 4 points bending test of a 2 meters long beech wood beam joined by welded dowels: test start.*

Fig. 11. *Sequence of events during the 4 points bending test of a 2 meters long beech wood beam joined by welded dowels: maximum bending before break.*

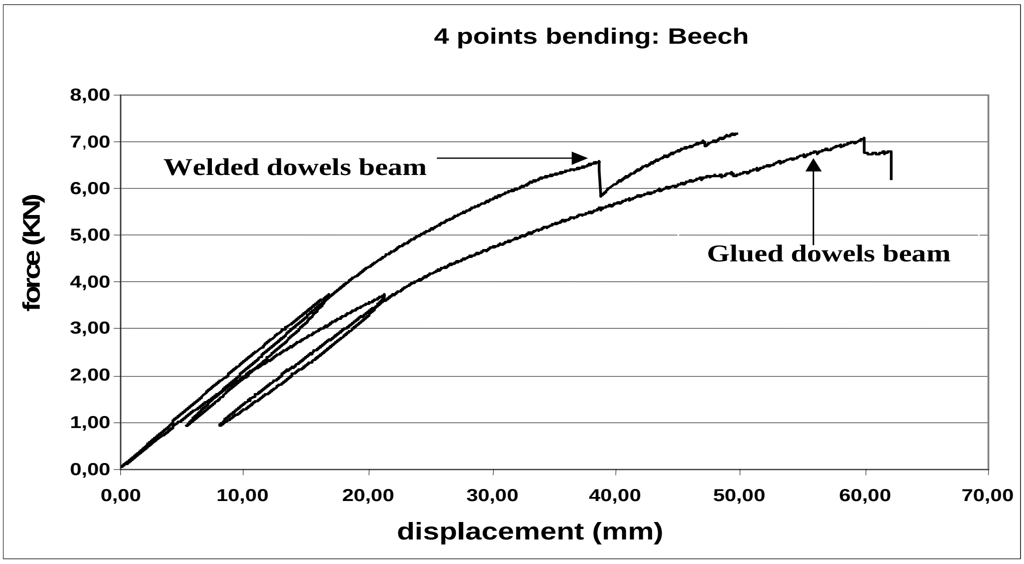


Fig. 12. *Force as a function of displacement during the 4-point bending test of two layers beech 2 meters long beams joined by welded dowels and PVAc-glued-in dowels, with the fluted groove dowels in both cases inserted at 30°. Compare with Fig. 13.*

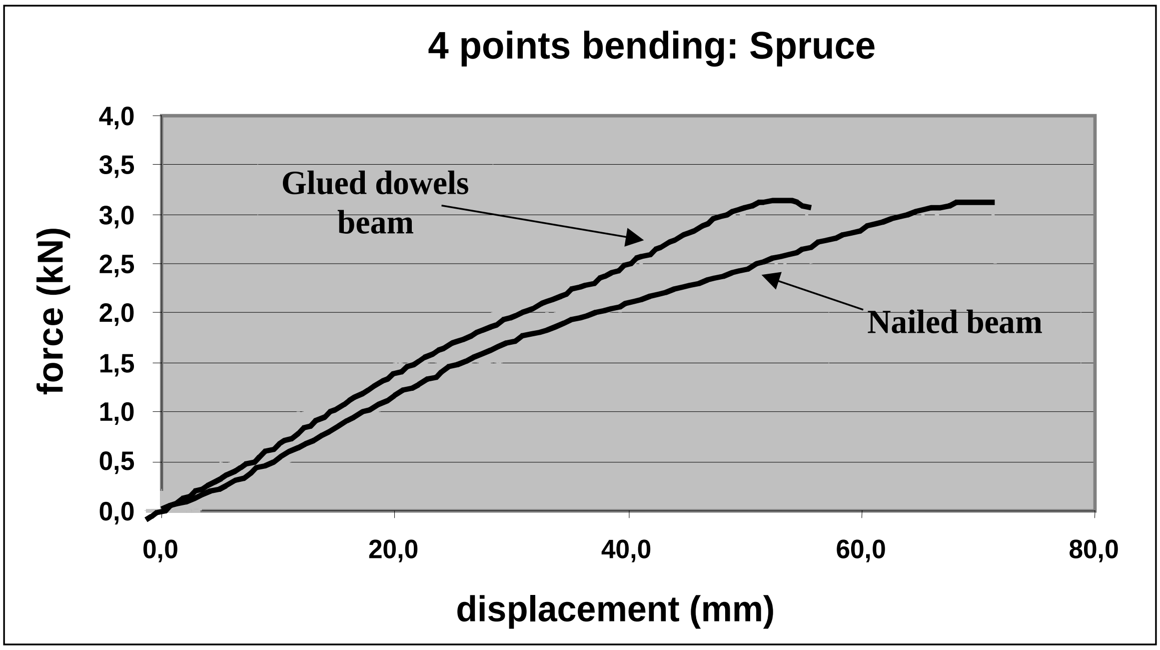


Fig. 13. *Force as a function of displacement during the 4-point bending test of two layers 2 meters long spruce beams joined by nails and by PVAc-glued-in dowels, with both nails and fluted groove dowels inserted at 90°. Compare with Fig. 12.*

High-speed dowel rotation welding was used to manufacture a full-scale 4metres by 4 metres by 22 cm suspended floor, hence an applicable civil engineering structure, to demonstrate that scaling up of such welding technique was feasible (Figs. 14, 15, 16) . This was coupled with obtaining a more light weight floor assembly with equal stiffness by maximizing the rigidity of the suspended floor while minimizing the number of timber planks used to build it, and maintaining its vibration frequency high and its level of vibration low. Several assembly and connection combinations of two and three slats linked through welded wood dowels were tried to determine the mechanical resistance of the cross-over joints that had to be used in the building of the floor. Deformation under 4-point static load of the floor was carried out to determine displacement under load and the floor vibration behaviour was determined by the use of accelerometers. The fundamental first natural frequency measured did satisfy well the requirements specified by Eurocode 5 (Fig. 17).

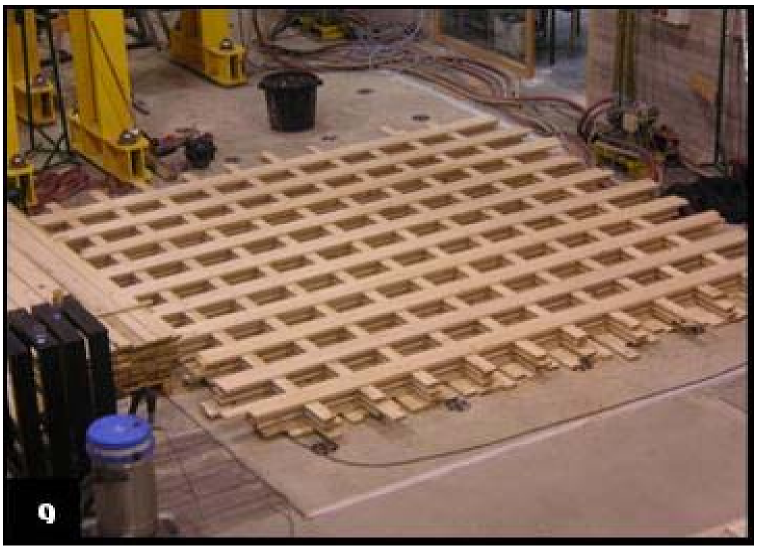


Fig. 14. *Photographs of the suspended floor at the beginning of its construction.*

Fig. 15. *Photographs of the suspended floor on its final placement ready for testing.*

Fig. 16. *Photographs of the suspended floor. Configuration of welded dowels during floor construction.*

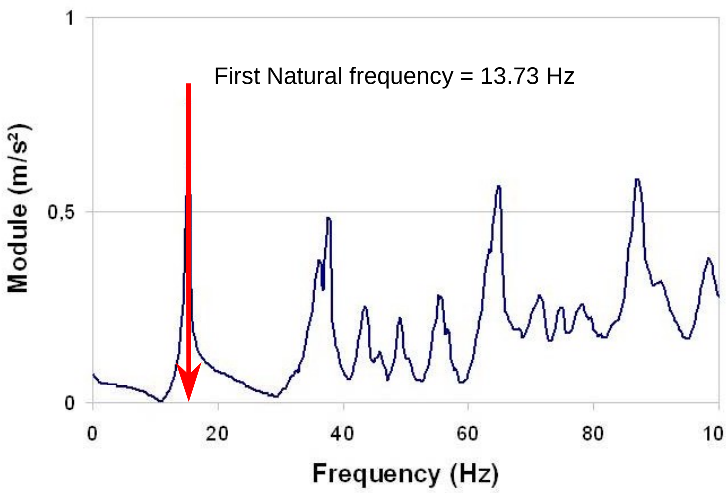
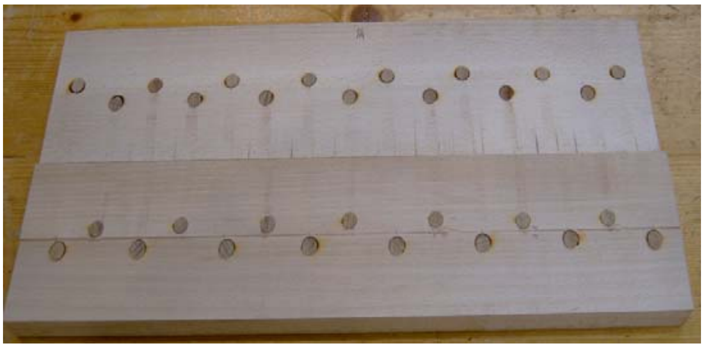


Fig. 17. *Accelerometer Fast Fourier Transform trace showing modulus as a function of frequency and showing the first natural frequency (13.73 Hz) of the suspended floor in the centre of the floor. Eurocode 5 and European Norms require a first natural frequency* $\geq$ *8 Hz.*

Furthermore, panels based on a cross-insertion of welded dowels yielded but joints strength inferior to UF-bonded butt joints but of considerably superior water resistance than the glued joints. Thus after 2 hours in boiling water, both tested wet or tested after redrying they conserved approximately 60% of their original strength indicating that in the case of dowel welding without any adhesive the geometry of the joint allows to produce full exterior grade joints (Fig. 18 and 19. Table 3). These panels were conceived to be used as the bottom panel of wooden coffins.



***Figs. 18** & **19.** Block panels in which the but joint is kept together by cross-welded dowels accross the planck to planck interface. Before the dowels are cut (right) and after the dowels are cut and the piece planed (left).*

***Table 3.** Butt joints joined by crossed dowels, tested in tension*

	Welded (5dowels)	*UF-bonded (without dowels)*
Dry strength (N/mm^2)	0.58$\pm$0.04	1.42+0.16
2 hours in boiling water, tested wet (N/mm^2)	0.34$\pm$0.02	0.0
2 hours in boiling water + redrying (N/mm^2)	0.35$\pm$0.02	0.0

In the case of linear vibration welding, joints obtained in this manner have been used succesfully, and commercialized, for the production of high class, high price, snow boards of which many have been sold already (by BFH, Biel, Switzerland). One of the important applications of both welding systems is for the joining without any adhesives of wood panels. Thus, linear vibration welding has been used succesfully for the edge to edge joining of different wood panels and also for joining a particulate composite panels to solid wood pieces. Equally, rotational dowel welding has been used to join different types of panels in rigs to be tested succesfully for structural application according to european norms and specifications (Fig. 20).

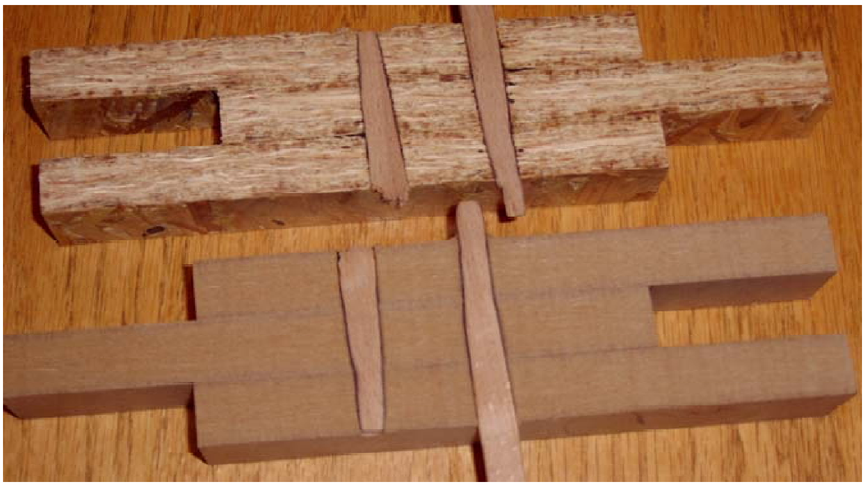


***Fig. 20.** Dowel-welded test pieces of OSB panels and MDF panels tested according to European Norms EN 26891 (ISO 6891) (P21-310) (1991), EN 1380 (P21-375) (1999) and prEN 1995-1-1:2003 Eurocode 5 for structural application.*

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The potential for modified materials in the furniture industry – Potential and availability

Dennis Jones[1]

Abstract: *The use of wood modification techniques has gained acceptance as a means of improving some of the properties of European timber species. Among the main improvements are increased dimensional stability and natural durability. Commercial activities increase on an annual basis, meaning more modified material is becoming available for a range of uses. This paper considers a range of commercial developments and how they may be applied within the furniture industry. Among the methods considered are: acetylation, furfurylation, heat treatment and hot oil treatment.*

Introduction

There has been a considerable drive towards sustainability within the timber industry. However such demands are often met with the need to use a species of lower qualities than, for example, a traditional tropical hardwood species. For several decades, there has been an interest in using wood modification as a means of altering the properties of a 'common' softwood species, so that some or its properties mimic those of a tropical hardwood. Whilst early work in this field was carried out in the US and Japan, more recent developments have taken place within Europe.

Within the UK, several research groups, including the Building Research Establishment (BRE) have been actively promoting wood modification as a means of improving timber properties. BRE have recently undertaken several projects involving the use of wood modification techniques to upgrade UK grown timber, mainly for use within the construction industry. This represents a single market opportunity for modified wood. Among other markets that have been considered have been panel products (as reported by Jones and Enjily within COST E49). This paper considers opportunities within furniture manufacture.

There are a variety of treatments that may be considered and these are summarised below.

Acetylation

This is recognised as the most common form of chemical modification, having been demonstrated on laboratory scale several decades ago. The process involves the reaction of wood with an acetylating agent (such as acetic anhydride or ketene) resulting in the esterification of many of the hydroxyl groups within the wood. These then become more hydrophobic, making the wood more dimensionally stable. Recent developments have seen attempts at commercialisation in two European countries: The Netherlands (Figure 1) and Sweden (Figure 2).

More recently, considerable financial investment has been placed into acetylation within The Netherlands, with the establishment of a new company, TitanWood. Indeed TitanWood have launched an acetylated product called Accoya™, which is currently being marketed within the UK by BSW (a major sawmill within the UK).

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__Fig. 1:__ Acetylation plant in The Netherlands

__Fig. 2:__ Acetylation plant in Sweden

Furfurylation

A process that has emerged on the market place is wood treated with furfuryl alcohol, a compound that may be produced in biomass conversion. This reagent can react with the wood hydroxyl groups as well as undergoing polymerization reactions, resulting in polymeric blocking of the hydroxyl groups. Development of this process has been done in Norway. It has been shown that furfurylation can prevent decay, increase dimensional stability and decrease hygroscopicity among other properties. Figure 3 shows the pilot scale treatment vessel, and full commercialisation has now been established at Porsgrunn in Norway.

***Fig. 3.** Pilot scale furfurylation plant*

A range of commercialised products have been produced from the furfurylisation process, including VisorWood™ (a low level treatment for pine and spruce), and Kebony™ (a higher level treatment with properties and appearance similar to that of tropical hardwood).

Heat treatment

There are a variety of companies currently working on thermal modification of wood. Heat treatments are based on the limited thermal cleavage of active groups within the wood structure, which may be released as volatile agents, or trapped within the cell matrix and repolymerized into more stable forms. Treatments are usually carried out in the absence of oxygen, due to the excessive thermal degradation processes that can occur in the presence of oxygen. Thus, atmospheres of nitrogen, steam or treatment *in vacuo* are used. Since this, in theory, is a simpler process than chemical modification, plant construction costs are generally lower, resulting in a slightly lower priced product. Among recent and current companies working on heat treatments include New Option Wood and Retitech of France, Plato Wood and Lignius of The Netherlands, whilst the largest company is a Finnish consortium, which produces ThermoWood®. There are a range of other companies (large and small scale) operating, such that a CEN technical committee has been set up to help evaluate product performance in an attempt to standardise processes and products.

Hot Oil treatment

Hot oil treatments can be considered as a "marriage" between thermal treatments and impregnation processes. The hot oil can act both as an oligomer capable of polymerising within the wood cell wall (to produce a non-leachable hydrophobic treatment), as well as providing an excellent medium for heat transfer during the thermal process. A treatment that is increasing in popularity is the use of a reactive hot oil. BRE are currently working with SHR, Bangor University and several industrial partners on a Fifth Framework CRAFT project for the development of a commercial process using a modified linseed oil system. Other hot oil treatments have been developed in the past, most notably the Menz Holz system in Germany.

Where modified wood may be marketed

Wood modification has shown itself to be suited to many uses in the wood industry, where an increase in the durability and dimensional stability offer added value to a product. The First European Conference on Wood Modification (ECWM1), held in Gent, Belgium in April 2003 showed that there was considerable interest in wood modification, with several companies being present at a technical forum. This increasing interest was shown at ECWM2, held in Gottingen, Germany in 2005, with further interest already declared for ECWM3, to be held in Cardiff, UK in October 2007.Within these conferences, it has been possible to demonstrate products and processes as well as expand marketing and research links. Many of these links have led to increased levels of commercialization, with further activities expected in the coming years.

The marketing of a new product will ultimately determine its success and profitability. Without the correct interpretation of consumer requirements and desires, even the best products will be doomed to failure. Thus it will be necessary to undertake a careful consideration of what products should be targeted for a given modification process. In terms of furniture manufacture it is necessary to determine which properties will benefit from the use of modified wood. The main properties of furniture are strength, looks and in-service durability, with secondary properties including dimensional stability, water stability and fire resistance. Based on predicted properties following wood modification, wood impregnation / polymerization processes, as well as chemical modification processes may appear to offer best options on first examination. This does not preclude other methods (such as thermal treatments), though their use may be more relevant within flooring sectors. Indeed the use of thermally modified end-grain tiles has already been demonstrated within Wales, with small scale production currently underway.

A survey in Sweden predicted an interest by local companies in modified wood in the following areas (Table 1), along with the anticipated treatment of interest. The last line (referring to furniture manufacture), represents an assessment of current methods available.

Table 1. *Overview of perceived modification techniques in various uses.*

Product range	*Possible treatment process*
Garden Wood	Thermally modified wood
Window Companies	Acetylated wood / Polymer impregnated
Exterior Door Companies	Acetylated MDF
Flooring Companies	Modified wood / MDF / Polymer impregnated
Wet Room & Façade Panels	Acetylated fibres
Building products etc.	Acetylated / Heat modified wood
Automotive / Nautical industry	Furfurylated wood
Architects /Gov. organizations	Acetylated /Heat modified wood
Furniture manufacture	Acetylated / Furfurylated / Polymer impregnated

In terms of furniture manufacture, modified veneers (for enhanced plywood products) and modified fibres (for use in composites) also offer potential, given the use of these materials in many furniture products. MDF manufacture is approximately 10 million m^3 per year annually, so even a small capture of this market would represent a lucrative move for wood modification companies.

Current and anticipated availability of modified wood

Table 2 provides an evaluation of current and imminent availability of modified materials.

Table 2. *Availability of modified wood*

Modification	Likelihood of availability		
	Solid wood	*Veneers*	*Fibres*
Acetylation	Yes Within 1-2 years	Yes < 5 yrs	Yes <10 yrs
Furfurylation	Yes Current	Yes <10 yrs	Possible
DMDHEU	Yes Within 1-2 years	Yes <5yrs	Possible
Heat treatment	Yes Current	Yes <5 yrs	Possible <10 yrs

In addition to the modification methods listed in Table 2, other methods may become more available in the next 5-10 years. All wood modification methods require a 'leap of faith', with considerable investment required. Any development requires a strong market assessment before investment can take place.

Conclusions

Wood modification has moved from an interesting laboratory experiment to full scale commercial production in several cases. Furfurylation and heat treatment are established, with acetylation reaching full commercial production in spring 2007. Other methods may also become available once market entry points have been selected.

The furniture industry can benefit from modified wood, not only through the use of solid wood, but also veneers and fibres. Benefits can include improved stability, increased surface hardness and water repellency.

Acknowledgements
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The use of *Origanum vulgare* ssp. *hirtum*, as a substitute for particleboard production

Ntalos, G.[1]**, Cholmpa, V.**

Abstract: *Origanum vulgare ssp. hirtum, is the 'true Greek' plant of oregano, a plant very high in essential oils. It is found only in Greece, Turkey, and the islands of the Aegean Sea (it is sometimes incorrectly referred to as Origanum heracleoticum) and it is the strongest and most pungent form of the species, and also much more limited geographic spread and rarely available. Known as 'origini' in Greece, it is only summer flowering heads that are dried and used. The flowers are always white. The leaves are fuzzy, oval and somewhat coarse in relation to the other species. With the present work was intended to study the suitability of Greek oregano stalks as a raw material for the particleboard production. A combination of oregano particles and industrial wood particles in different proportions were been used as a raw material for the production of one-layer particleboards. As a binder were been used a commercial urea-formaldehyde (UF) E1 resin. The strength properties was good up to 20 percent of wood replacement but not very good in 50% origano stalks and the hygroscopical properties were lower than EN 312-1 norm requirements for general use. We have also to mention the very good odor from the boards based on the essential oils that emit.*

Introduction

In the last decades, the decline in forest areas is a common problem. This is a result of forest fires, and also of the increased demand for wood and agricultural lands. This situation leads people to consider seriously about deforestation and generally for forest degradation. It is a common conscience to provide the environmental and aesthetic goods of the forests. Very often, many countries try to find a special plan concerning the forest harvesting, which leads to wood storage. All the above in combination with the growing demands for wood products and especially for wood based panels (Table 1) for furniture and construction proposes (Papadopoulos 2006), leads people to find new wood resources as an alternative sources to forest wood.

The prospective of using different lignocellulosic materials, which comes from agricultural residues, as a raw material replacement, of wood for the wood based panel production, is been thinking seriously from several researchers from the past (Baum 1967,Wang and Sun 2002, Ntalos 2002, Papadopoulos et al. 2004) till now (Zheng et al. 2006, 2007, Kalaycioglu and Nemli 2006, Guru et al. 2006). The stalks of Origanum vulgare ssp. hirtum, is been testing as one of those lignocellulosics agricultural residues, which could replace wood as a raw material for the production of particleboards. Origanum vulgare spp hirtum - Greek Oregano, is a subspecies of the widespread wild oregano, and is found only in Greece, Turkey, and the islands of the Aegean Sea (it is sometimes incorrectly referred to as Origanum heracleoticum). Known as 'origini' in Greece, it is only summer flowering heads that are dried and used. The flowers are always white. The leaves are fuzzy, oval and somewhat coarse in relation to the other species.

The flavor is strong, austerely and hotly aromatic, penetrating and slightly bitter. This is the strongest flavored 'oregano'. It is the species used for extraction of essential oils, the dried foliage having around 3% of oils, depending on growing conditions and seedling variability. The concentration of oils is so high that lengthy handling of large amounts of the dried product can cause irritation to sensitive skins. This species of Origanum is a creeping rhizomatous perennial, and needs sandy or well drained soil, is hardy, and will stand a little frost. As long as it gets good snow cover, it has survived as far north as Alberta, Canada. True Greek oregano can easily be grown from seeds sown inside in late winter or spring (depending on climate), or from cuttings in Autumn or Spring. It also easily propogates from rooted portions of the creeping stem. Transplant the seedlings after danger from frost has passed.

This paper intends to test if the stalks of Greek Oregano can be used as a raw material for the production of particleboards. This could give in wood industry an alternative material instead of forest wood, and also an alternative use of this plant. Together with all above mentioned we can also provide boards wit special odors from the essential oil that this stalks have in small concentration. This odor can keep away insect like mouth and the boards made from this material can used for special uses like closets or food packaging.

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Table 1. *Worldwide production of wood products from 1970 to 2005 (1000 m3) (database results FAOSTAT, 2006)*

	1970	1980	1990	2000	2005
Veneers	3,202,900	4,440,004	5,216,404	8,038,474	9,372,420
Plywood	33,413,700	39,432,191	48,156,808	58,168,245	68,902,288
Particleboards	19,141,400	40,508,700	55,418,300	84,997,203	99,667,560
Fiberboards	14,021,865	16,961,401	20,215,627	34,098,598	56,870,971
Sawn Wood	389,189,129	420,867,511	466,457,815	386,014,342	428,459,032

Material and methods

As it is already mentioned the raw material for the present work is the stalks of Greek Oregano plant, which were collected in September 2006, from Central Greece.

Concerning the evaluation of the basic properties and characteristics representative samples were chosen, for the determination of those properties. The extractives, ash content and moisture content were determined in stalk material. Concerning the determination of extractives content solubles in hot water, dichloromethane (Co Ridel-de Haen) and wood free of extractives was have been done according to ASTM standards D1110-84, D1107-84, D1108-84 and D1105-84 (ASTM-D, 1984). Ash content were determined according to ASTM standard D1102-84, respectively (ASTM-D,84).

The moisture measurement has been done according to the European norms, EN 322 (EN,1993) before board production and was in a acceptable level for this purpose (Skarvelis, 2001). A hammermill, with an 8 mm round role screen, has been used for the chipping and the particles were dried in a laboratory drier with hot air, from moisture ranged between 40 and 50%, down to 3% m.c. A variety mixture of origanum particles and wood chips (which have been supplied from a local particleboard plant) has been used as furnishes for one-layer particleboards.

Two experimental particleboards dimension 50X50 cm of 16mm thickness were manufactured for each board type (100% wood chips, 10%origanum particles and 90% wood chips, 20% origanum particles and 80% wood chips ,50% origanum particles and 50% wood chips, 25% origanum leaves and 75% wood chips). The temperature of the press (dynamic) was 180°C and the pressing time was 5 minutes. The target density in all board types was 0.70g/cm^3. A commercial UF-resin of E1 grade was applied to particles by spraying. The UF resin is the most commonly used resin in Greek particleboard industry at the present. The resin (at 74% solids but spayed with 50% solids) was supplied from the same local particleboard industry where the wood particles were supplied. For the particleboards the glue solids were 8% of the wood. It is also worth mentioning that there was no water-repelling agent during the particleboards construction.

Next step was the sample cutting from boards, in order to determine the following properties, always in accordance with the appropriate EN and ASTM standards. The determination of density has been done according the EN323; EN,1993, the static bending (modulus of rupture (MOR), modulus of elasticity (MOE))according to EN310; EN,1993, the internal bond according to EN319; EN,1993, the thickness swelling and water absorption according to EN 317; EN, 1993 and the screw holding strength according ASTM D-1037; ASTM D-, 1996 standard by using screws with d=3.5mm, l-45mm and the hole diameter 2.5mm.

Results and discussion

Table 2 displays the results of the chemical analysis of the oregano stalks. The average of soluble in water is less than in dichlomethane because of the essential oil that the stalks contains.

Table 2. *Extractives of oregano stalks*

Extractives soluble in	*%*
Hot water	7,78
Dichlomethane	10,11

Table 3. *Ash and moisture content of the oregano stalks*

Property	*%*
Ash content	8,08
Moisture content	3,25

In Table 4 we can see the mechanical properties of the boards .

Table 4. *Mechanical properties of UF-bonded single-layer experimental particleboards made from origanum particles.*

Wood : Origanum	*MOR (N/mm^2)*	*MOE (N/mm^2)*	*IB (N/mm^2)*	*Screw Holding = (N)*	*Screw Holding + (N)*
100:0	13,54	2132,65	0,45	1693,84	2619,26
90:10	13,80	2381,67	0,40	1452,76	3508,28
80:20	13,76	2336,43	0,41	1521,05	3209,355
50:50	11,51	2155,61	0,37	1328,22	2662,21
75:25 origanum leaves	6,19	1355,78	0,23	741,39	2176,09
EN 310 & EN 319 P2 General use	11.5	--	0.24	--	--
EN 310 & EN 319P3 Interior Fitment	13.0	--	0.35	--	--
EN 310 ; 317; 319 P4 Load Bearing -Dry	15.0	--	0.35	--	--

Table 5. *Hygroscopic properties of UF-bonded single-layer experimental particleboards made from origanum particles (Standard deviations in parentheses)*

Wood : Origanum	*TS (%) 24h*	*WA(%)24h*
90:10	144,99	125,07
80:20	137,96	127,96
50:50	140,27	126,13
0:100	106,69	134,62
75:25 origanum leaves	144,99	125,07
EN 310 & EN 319 P2 General use	N/A	
EN 310 & EN 319P3 Interior Fitment	N/A	
EN 310 ; 317; 319 P4 Load Bearing -Dry	14	

Conclusions

Partial substitution of wood by origanum stalk particles resulted in the slightly improvement of all board mechanical properties. This my be the result of the longer and thinner appearance of the origanum particles than industrial wood particles. However, the strength properties of boards containing up to 20% origanum particles met the minimum EN requirements for interior boards and up to 50% for general use. It is suggested that alternative resins, such as isocyanates, might give improvements in panel properties (Papadopoulos et al. 2002). On the other hand Hygroscopical properties are detiorated. The usage of water-repelling agent might help in the improvement of these properties.

Of course we are waiting better properties in three layer board where the origanum particles will replace wood only in the core layer.

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"Construction & Architecture"

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Ξύλινες κατοικίες. Κλασική και σύγχρονη τεχνολογία – Ελληνική πραγματικότητα.

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Περίληψη: *Οι προκατασκευές κατοικιών με ξύλινο σκελετό παρουσιάζουν ενδιαφέρον για την Ελλάδα, για τους παρακάτω λόγους:*

- *Το ξύλο ως δομικό υλικό είναι πολύ παρεξηγημένο στην Ελλάδα, όπου το μπετό κυριαρχεί σε επικίνδυνο βαθμό. Η άποψη ότι δεν υπάρχει ξύλο είναι εσφαλμένη, γιατί για ένα σπίτι 100m² απαιτείται μόνο μια ποσότητα 8-10 m³ δομικής πριστής ξυλείας κωνοφόρων για σκελετό, εσωτερική και εξωτερική τοιχοποιία και στέγη, η οποία παράγεται στην Ελλάδα. Απαιτούνται επίσης και διάφορες ξυλοπλάκες οι οποίες όχι μόνο παράγονται στην Ελλάδα αλλά και εξάγονται.*
- *Οι κατασκευές με ξύλινο σκελετό είναι οι πλέον αντισεισμικές και ως εκ τούτου κατάλληλες για περιοχές με έντονη σεισμικότητα όπως η Ελλάδα. Πέραν αυτού το ξύλο είναι ένα υπέροχο υλικό που κατεργάζεται εύκολα, είναι μονωτικό και έχει υψηλή αισθητική αξία.*
- *Η αντοχή της ξύλινης κατοικίας σε φωτιά ενισχύεται με χρήση υλικών επικάλυψης – επένδυσης τοίχων και μόνωσης που αντέχουν στη φωτιά όπως: σοβάς, χτίσιμο με τούβλο – πέτρα, γυψοσανίδες, τσιμεντοσανίδες ή κόντρα πλακέ εμποτισμένο με αντιπυρικές ουσίες.*

Για τους παραπάνω λόγους θα πρέπει να ασκηθεί κατάλληλη πολιτική από μέρους της πολιτείας και να θεσπιστούν προδιαγραφές σχετικά με τις κατασκευές αυτές.

1. Χρησιμοποιούμενα υλικά

Τα προϊόντα ξύλου και άλλα βασικά υλικά, που χρησιμοποιούνται σε προκατασκευές ξύλινων σπιτιών, είναι τα ακόλουθα:

- Πριστή ξυλεία κωνοφόρων (πεύκη, ελάτη, ερυθρελάτη, λάρικα, ψευδοτσούγκα).
- Λεπτή στρογγύλη ξυλεία (στύλοι) πεύκης, ψευδοτσούγκας.
- Διάφορα είδη ξυλοπλακών (μοριοσανίδες, ινοσανίδες, ξυλοπλάκες από προσανατολισμένα ξυλοτεμαχίδια μεγάλων διαστάσεων (OSB), αντικολλητά, ξυλοδοκούς από συγκολλημένες λωρίδες ξυλοφύλλων (PSL), σύνθετη ξυλεία από συγκολλημένα ξυλόφυλλα (LVL), ξυλοπλάκες από συγκολλημένες λωρίδες συμπαγούς ξύλου).
- Γυψοσανίδες, τσιμεντοσανίδες, μονωτικά υλικά, καρφιά, κοχλίες, μεταλλικές πλακέτες συνδέσεων, ασφαλτικά, πισσόχαρτο, μεμβράνες, οργανικές επικαλύψεις τοίχων, πατωμάτων κ.α.

Τα προϊόντα αυτά πρέπει να είναι πιστοποιημένα για δομικές εφαρμογές σύμφωνα με τις ισχύουσες ευρωπαϊκές προδιαγραφές (βλ. Πίν. 1.1). Ειδικότερα για τα προϊόντα ξύλου σε ότι αφορά την ποιότητα, ισχύουν τα ακόλουθα:

<u>Ξυλεία σκελετού – Επενδύσεις τοίχων</u>

Τα κορμίδια ως στοιχεία σκελετού πρέπει να είναι ευθυτενή, χωρίς βασικά σφάλματα στρεψοϊνιας, κωνικομορφίας, έντονης ροζοβρίθειας, έντονων ραγαδώσεων και προσβολών από μύκητες και έντομα. Η διάμετρος κορμιδίων κυμαίνεται από 12 έως 22cm. Αν χρησιμοποιείται πριστή ξυλεία θα πρέπει επίσης να είναι απαλλαγμένη από σφάλματα δομής και προσβολές. Η ξυλεία σκελετού θα πρέπει να είναι ξηραμένη στο 10-12%. Οι επικρατέστερες διατομές για ορθοστάτες είναι στις σύγχρονες κατοικίες: πάχος: 5cm, πλάτος: 10cm για απόσταση κατακόρυφων στοιχείων (ορθοστατών) 50 – 60cm (κέντρο από κέντρο). Για μεγαλύτερες αποστάσεις κατακόρυφων στοιχείων σκελετού, οι διατομές είναι ενισχυμένες και ποικίλουν ανάλογα με το είδος της κατοικίας και τον αριθμό των ορόφων (10x10cm, 10x12cm, 12x12cm, κτλ.)

Για την κατασκευή του σκελετού προτιμάται η ξυλεία πεύκης η οποία διαθέτει μεγαλύτερη ανθεκτικότητα.

Σε ότι αφορά τις *ξυλοπλάκες* και *σανίδες επικάλυψης* των τοίχων και της στέγης, ισχύουν επίσης διεθνώς αυστηρές προδιαγραφές. Έτσι για <u>εξωτερική επικάλυψη</u> επιτρέπεται η χρήση:

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- Σανίδων τύπου ραμποτέ πάχους τουλάχιστον 22mm από πεύκη, ψευδοτσούγκα.
- Αντικολλητά εξωτερικής χρήσεως από ανθεκτικά είδη όπως πεύκη, δρυς, ευκάλυπτος, ανθεκτικά τροπικά, συγκολλημένα με κόλλα εξωτερικής χρήσεως.
- Τσιμεντοσανίδες εξωτερικής χρήσεως (ίνες ξύλου ή ίνες πολυμερών αναμεμιγμένες με τσιμέντο υψηλής πίεσης και πιεσμένες σε συμπαγή πλάκα). Χρησιμοποιούνται επίσης και δομικές πλάκες τύπου 'σάντουϊτς' κατασκευασμένες από πυρήνα διογκωμένης πολυστερίνης με αμφίπλευρη επένδυση τσιμεντοσανίδων τύπου *Ηρακλίτης*. Τα προϊόντα αυτά μπορεί να σοβατιστούν.

Τις τελευταίες δεκαετίες η έλλειψη αντικολλητών οδήγησε στην ανάπτυξη νέων τύπων ξυλοπλακών για κατασκευές, από υπολείμματα κατεργασίας και ξύλο μικρών διαστάσεων. Τέτοια νέα προϊόντα είναι οι ινοσανίδες υψηλής πυκνότητας (High Density Fiberboard – HDF), οι μοριοσανίδες με κατευθυνόμενη διάταξη ξυλοτεμαχιδίων (Oriented Structural Board – OSB) και η πριστή ξυλεία από επικολλητά ξυλόφυλλα (Laminated Veneer Lumber). Τα νέα αυτά προϊόντα που ήλθαν να υποκαταστήσουν την πριστή ξυλεία και το κόντρα πλακέ, αν και χρησιμοποιούνται σε ευρεία κλίμακα στις ΗΠΑ, Καναδά και Αυστραλία, αντιμετωπίζονται από ορισμένους επιστήμονες με κάποια αμφιβολία.

Για <u>εσωτερική επένδυση τοίχων</u> χρησιμοποιείται ξυλεία ραμποτέ, μοριοσανίδες – ινοσανίδες αντικολλητά κατηγορίες Ε1 (χαμηλή περιεκτικότητα σε φορμαλδεϋδη: 6,5-8 mg/100g ξυλοπλάκας), γυψοσανίδες, τσιμεντοσανίδες τύπου δομικών πλακών *«Ηρακλείτης»* (ξυλοτεμαχίδια με τσιμέντο υψηλής αντοχής), που σοβατίζονται και βάφονται.

<u>Πάνελς τοιχοποιίας σε μορφή σάντουϊτς</u>

Με σκοπό τη βιομηχανική παραγωγή ημιέτοιμων ή και έτοιμων στοιχείων τοιχοποιίας και στέγης, έχουν παραχθεί πολλά πάνελς σε μορφή σάντουϊτς διαφόρων στρώσεων για συγκεκριμένες εφαρμογές, όπως:

- Εξωτερικές και εσωτερικές επενδύσεις τοίχων προκατασκευασμένων σπιτιών, εσωτερικά χωρίσματα σπιτιών και άλλων χώρων καταστημάτων, για υποδομή ειδικών πατωμάτων, για πόρτες.
- Ειδικά ηχομονωτικά – θερμομονωτικά – διακοσμητικά πάνελς.

Οι στρώσεις που αποτελούν τα πάνελς αυτά είναι: αντικολλητά – MDF – κοινές μοριοσανίδες – μοριοσανίδες τύπου OSB, διακοσμητικά ξυλόφυλλα, διογκωμένη πολυουρεθάνη, στρώσεις φελού, φύλλα αλουμινίου, στρώσεις πολυμερών. Οι στρώσεις αυτές συγκολλούνται στο επιθυμητό πάχος. Η σύνθεση των στρώσεων εξαρτάται από την τελική χρήση.

Οι πιο συνηθισμένοι τύποι τέτοιων πάνελς με τις αντίστοιχες εφαρμογές, είναι οι ακόλουθοι:

<u>Πάνελ 3 τρώσεων</u>

MDF - OSB - MDF.: χρησιμοποιείται για εσωτερικά χωρίσματα χώρων.

<u>Πάνελ 3 ή 5 στρώσεων</u>

- αντικολλητό 3 ή 5 στρώσεων – διογκωμένη πολυουρεθάνη αντικολλητό 3 ή 5 στρώσεων: χρησιμοποιείται για εξωτερική επένδυση σπιτιών από ξύλινο σκελετό ή για εσωτερικά χωρίσματα.
- αντικολλητό 3 ή 5 στρώσεων – μονωτική ινοπλάκα – αντικολλητό 3 ή 5 στρώσεων: χρησιμοποιείται για εσωτερικά χωρίσματα χώρων.
- OSB – διογκωμένη πολυουρεθάνη – OSB: χρησιμοποιείται για εξωτερική επένδυση σπιτιών με ξύλινο σκελετό και επικάλυψη σκελετού στεγών.

<u>Πάνελ 5 στρώσεων</u>

ξυλόφυλλο – OSB – ξυλόφυλλο – OSB – ξυλόφυλλο: Χρησιμοποιείται για επικάλυψη τοίχων εξωτερικής χρήσεως.

<u>Πάνελ για πόρτες</u>

αντικολλητό 3 ή 5 στρώσεων – λεπτή στρώση αλουμινίου – στρώση πολυουρεθάνης – λεπτή στρώση αλουμινιμίου – αντικολλητό 3 ή 5 στρώσεων.

<u>Πάνελ υψηλής ηχομόνωσης για επικάλυψη τοίχων</u>

αντικολλητό 5 στρώσεων – πολυμερές υψηλής πυκνότητας – αντικολλητό 5 στρώσεων.

<u>Πάνελ για φάτσες κτιρίων και νταμπλάδες πορτών και χωρισμάτων</u>

PVC – εξηλασμένη πολυστυρόλη – PVC.

Για την κατασκευή της στέγης χρησιμοποιείται επίσης ξυλεία κωνοφόρων χωρίς σφάλματα. Για ειδικές κατασκευές στέγης με εμφανή στοιχεία σκελετού στέγης και για μεγάλα ανοίγματα χρησιμοποιείται επίσης η επικολλητή πριστή ξυλεία (σύνθετη ξυλεία). Σύνθετη ξυλεία χρησιμοποιείται επίσης και στο σκελετό και τα μπαλκόνια, λόγω του πλεονεκτήματος της ευθυτένειας και της μεγαλύτερης μηχανικής αντοχής.

Για όλα τα συγκολλημένα προϊόντα ξύλου που χρησιμοποιούνται σε εξωτερικές εφαρμογές είναι επιβεβλημένη η χρήση συγκολλητικής ουσίας εξωτερικής χρήσεως (φαινόλη ή μελαμίνη φορμαλδεϋδη, κόλλες ισοκυανίου).

Σε πολλές χώρες επιτρέπεται και συνιστάται ο εμποτισμός της ξυλείας του σκελετού και της εξωτερικής επικάλυψης υπό πίεση με υδατοδιαλυτά συντηρητικά άλατα χαλκού, χρωμίου, βορίου (τύπου CCB) ή βορικά άλατα.

Τα υλικά που χρησιμοποιούνται στις θέσεις συνδέσεων των ξύλινων στοιχείων του σκελετού (κοχλίες, ξυλόβιδες, μεταλλικές λάμες), καθώς και τα υλικά αγκύρωσης του σκελετού στη βάση θεμελίωσης (κοχλίες, αγκυρόβιδες) θα πρέπει να συνδυάζουν την υψηλή μηχανική αντοχή με την αντοχή σε οξείδωση. Για αυτό το λόγο παράγονται από ειδικά ανθεκτικά κράματα σύμφωνα με ειδικές προδιαγραφές.

Διαβαθμισμένα ποιοτικά και σύμφωνα με προδιαγραφές είναι και τα υλικά μόνωσης, όπου εκτός από τις θερμοηχομονωτικές και αντιπυρικές ιδιότητες, λαμβάνεται σοβαρά υπόψη και ο παράγων υγεία. Επιτρεπόμενα υλικά μόνωσης είναι: ο υαλοβάμβακας και άλλες ίνες ορυκτών (ορυκτοβάμβακας), η διογκωμένη ή εξηλασμένη πολυστερίνη, η πολυουρεθάνη, οι μονωτικές ινοπλάκες ξύλου, το πισσόχαρτο, τα ασφαλτόπανα, οι μεμβράνες πολυμερών PVC – CPE, οι θερμομονωτικές πλάκες φελλού.

Για ηχομόνωση χρησιμοποιούνται, εκτός των αναφερθέντων υλικών:

- Θερμομονωτικές πλάκες φελλού, οι οποίες παρασκευάζονται από διογκωμένα τρίμματα φυσικού φελλού (ο οποίος παράγεται από το ξηρόφλοιο της φελλοδρυός), που μορφοποιούνται σε πλάκες με πίεση και υψηλή θερμοκρασία. Το προϊόν αυτό είναι το μοναδικό φυσικό θερμομονωτικό υλικό τελείως αβλαβές για τον άνθρωπο.
- Ηχομονωτικά σάντουϊτς από ηχοαπορροφητικό υλικό από αφρώδη PU + ηχοανακλαστικό φύλλο + αφρώδη PU για απορρόφηση κραδασμών, συνολικού πάχους 30MM.

Στον Πιν. 1.1 αναφέρονται οι ισχύουσες ευρωπαϊκές προδιαγραφές καθώς και η ημερομηνία που καθιστά υποχρεωτική την εφαρμογή τους, για όλες τις κατηγορίες των προϊόντων ξύλου και προκατασκευασμένων στοιχείων που χρησιμοποιούνται σε ξύλινες δομικές κατασκευές.

Πιν. 1.1. *Ευρωπαϊκές προδιαγραφές προϊόντων ξύλου και προκατασκευασμένων στοιχείων που χρησιμοποιούνται σε ξύλινες δομικές κατασκευές.*

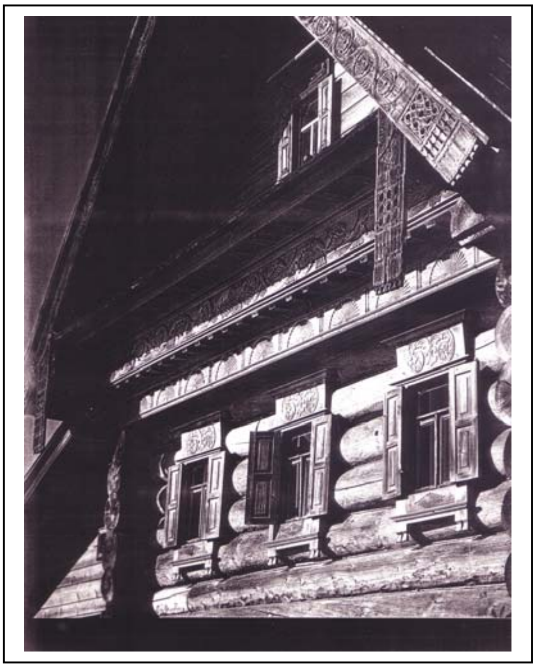
Προϊόν	Προδιαγραφή	Ημερομηνία εφαρμογής
Ξυλοπλάκες	EN 13986	01-04-06
Προκατασκευασμένα δομικά στοιχεία	EN 14250	01-09-06
LVL για κατασκευές	EN 14374	01-09-06
Ξυλεία πατωμάτων	EN 14342	01-03-07
Επικολλητή ξυλεία	EN 14080	01-04-07
Πελεκητή ξυλεία κατασκευών	EN 14081	01-09-07
Ξυλοπλάκες και επενδύσεις από μασίφ ξύλο	EN 14915	Καλοκαίρι 08
Ξυλεία κατασκευών με finger joint	EN 15497	
Εμποτισμένη ξυλεία κατασκευών	EN 15228	
Στύλοι	EN 14229	Άνοιξη 09
Στρογγύλη ξυλεία κατασκευών	EN 14544	Άνοιξη 09
Μεταλλικές συνδέσεις	EN 14545 / EN 14592	Άνοιξη 09
Προκατασκευασμένα στοιχεία	EN 14732	Άνοιξη 09

Διεθνής πρακτική

2.1. Κορμόσπιτα με οριζόντια τοποθέτηση κορμών

Το βασικό κατασκευαστικό υλικό των κορμόσπιτων είναι κατάλληλα κατεργασμένα κορμίδια πεύκης – ψευδοτσούγκας – κυπαρισσιού – καστανιάς, διαμέτρου 12-25cm (Εικ. 2.1). Παλαιότερα χρησιμοποιούντο και μεγάλης διαμέτρου κορμοί ενώ η κατεργασία τους γινόταν με παραδοσιακά εργαλεία (σκεπάρνια, τσεκούρια, αποφλοιωτήρες) (Εικ. 2.2). Τέτοια παραδοσιακά κορμόσπιτα σώζονται για δεκάδες αιώνες αποδεικνύοντας ότι το ξύλο σαν κατασκευαστικό υλικό έχει μεγάλη διάρκεια, αρκεί να υφίσταται στοιχειώδη συντήρηση και φροντίδα.

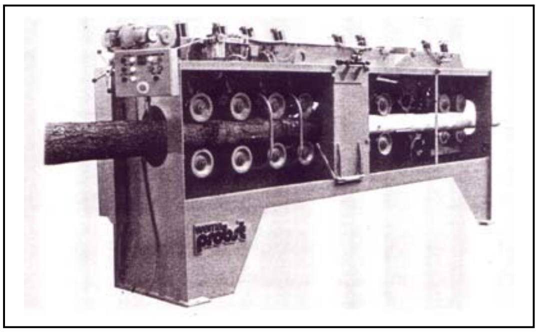
Εικ. 2.1. *Κορμόσπιτο από κατεργασμένα κορμίδια.*

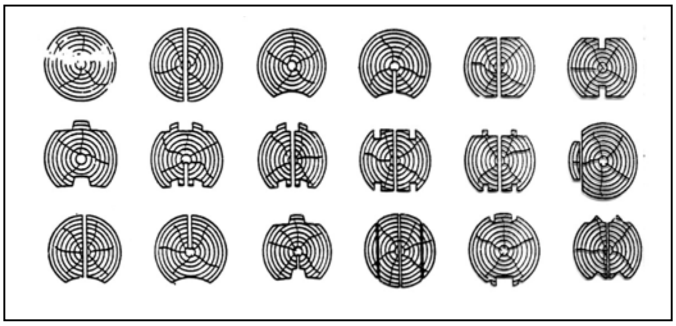


Εικ. 2.2. *Παραδοσιακή κατοικία από κορμούς μεγάλης διαμέτρου.*

Τα τοιχώματα των κορμόσπιτων αποτελούνται από τα κορμίδια, τα οποία μετά την κατεργασία τους τοποθετούνται σε οριζόντια διάταξη το ένα επάνω από το άλλο. Η κατεργασία των κορμών είναι απλή. Οι κορμοί διέρχονται από ειδικά μηχανήματα (Εικ. 2.3), όπου κυλινδρομορφώνονται και ταυτόχρονα δημιουργούνται κατά μήκος προεξοχές και εσοχές (Εικ. 2.4) που αποσκοπούν στην πλήρη εφαρμογή – σύνδεση των κορμών κατά την οριζόντια τοποθέτηση του ενός πάνω στον άλλο.

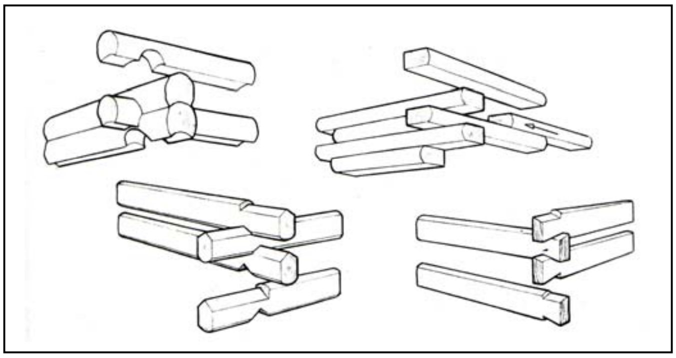
Εικ. 2.3. *Μηχανή μορφοποίησης κορμιδίων.*





Εικ. 2.4. *Διάφορες διατομές κορμιδίων.*

Οι συνδέσεις των κορμών στις γωνίες των τοίχων γίνονται με ανάλογη δημιουργία εσοχών, έτσι ώστε να διασταυρώνονται οι κορμοί και να δένονται μεταξύ τους (Εικ. 2.5)



Εικ. 2.5. *Γωνιακές συνδέσεις κορμιδίων.*

Για το δέσιμο των κορμιδίων σε κάθε τοίχο περιμετρικά και σε αποστάσεις 2-3m, οι κορμοί φέρουν κατακόρυφες οπές μέσα από τις οποίες διέρχονται ανοξείδωτες μεταλλικές ράβδοι οι οποίες στα δύο άκρα τους φέρουν σπείρωμα, όπου βιδώνονται παξιμάδια και σφίγκονται οι κορμοί. Το σφίξιμο των κορμών μπορεί να γίνεται κατά τη διάρκεια του καλοκαιριού, όταν το ξύλο χάνει υγρασία με αποτέλεσμα να ρικνώνεται και οι διαστάσεις του να μικραίνουν. Το αντίθετο γίνεται κατά τη διάρκεια του χειμώνα, όταν τα κορμίδια παίρνουν υγρασία από την ατμόσφαιρα με αποτέλεσμα να διογκώνονται και να αυξάνει η διάμετρός τους. Το εύρος της μεταβολής της σχετικής υγρασίας και της θερμοκρασίας της ατμόσφαιρας εξαρτάται από το τοποκλίμα και καθορίζει την υγρασία ισορροπίας του συγκεκριμένου χώρου, δηλ. την τελική υγρασία που αποκτά το ξύλο αν το αφήσουμε στις συνθήκες αυτές. Το εύρος της υγρασίας ισορροπίας για τα Ελληνικά δεδομένα κυμαίνεται από περιοχή σε περιοχή μεταξύ 8% και 15-16%. Οι μεταβολές αυτές μπορεί να προκαλέσουν μια μέγιστη ρίκνωση ή διόγκωση στη διάμετρο του κάθε κορμού μέχρι και 2mm (θεωρητικά η συνολική μεταβολή του ύψους ενός τοίχου 3m που αποτελείται από 20 κορμούς, μπορεί να φθάσει τα 40mm). Αυτό στην πράξη περιορίζεται σημαντικά με τη συντήρηση των κορμών με ελαιώδη συντηρητικά όπως το λινέλαιο, τα συντηρητικά βερνίκια και άλλα υδροαπωθητικά επιχρίσματα, τα οποία τοποθετούνται τόσο στις εγκάρσιες τομές των κορμών όσο και στην περίμετρό τους.

Τα εσωτερικά χωρίσματα γίνονται είτε από κορμίδια μικρότερης διαμέτρου ή από ξύλινο ελαφρύ σκελετό και επένδυση στις δύο όψεις. Ο φέρον σκελετός της στέγης είναι επίσης κορμοί η ζευκτά βαρέως τύπου. Στα πολύ παλαιά σπίτια η επικάλυψη της στέγης γινόταν με χώμα. Η τεχνική είναι εντυπωσιακή. Επάνω από το σανίδωμα της στέγης απλώνεται φλοιός σημύδας, ο οποίος είναι αδιαπέραστος από το νερό και πάνω από το φλοιό τοποθετείται το χώμα. Η συγκράτηση του χώματος στα άκρα της στέγης γινόταν με πέτρες ή κορμίδια. Η ξυλεία συντηρείτο με πίσσα ή κεδρέλαιο. Οι Νορβηγοί στη χειμερινή Ολυμπιάδα του 1993 κατασκεύασαν ολόκληρα χωριά από τέτοια σπίτια, όπου έμεναν οι αθλητές, προφανώς για να δώσουν ένα παγκόσμιο μήνυμα προστασίας του περιβάλλοντος μέσα από τη χρήση ανανεώσιμων πρώτων υλών.

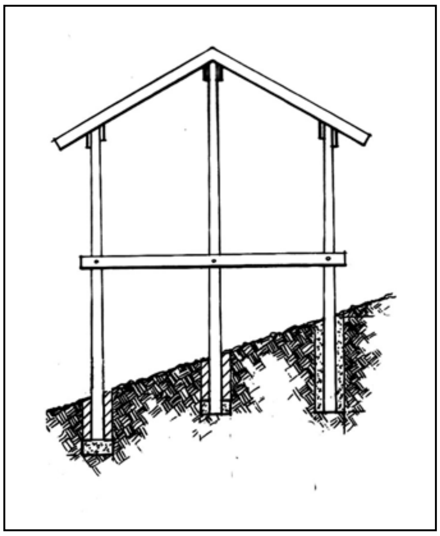
Σε μια άλλη παραλλαγή της τεχνικής αυτής για την επικάλυψη της στέγης χρησιμοποιούντο πριστά, που διέτρεχαν όλη τη στέγη, είτε κατά τη φορά της κλίσης, είτε κάθετα προς αυτή. Σαν υλικό επικάλυψης χρησιμοποιούντο επίσης μικρά τεμάχια ξύλου πεύκης κέδρου η δρυός σε σχήμα σφήνας. Σε μια μικρή παραλλαγή του τύπου των κορμόσπιτων, τα κορμίδια μορφοποιούνται και στις τέσσερις πλευρές τους, επάνω - κάτω με εσοχή - προεξοχή και δεξιά - αριστερά με παρύφωση για δημιουργία επίπεδων πλευρών (Εικ. 2.6).

Εικ. 2.6. *Κορμόσπιτο από ορθογωνισμένα κορμίδια.*

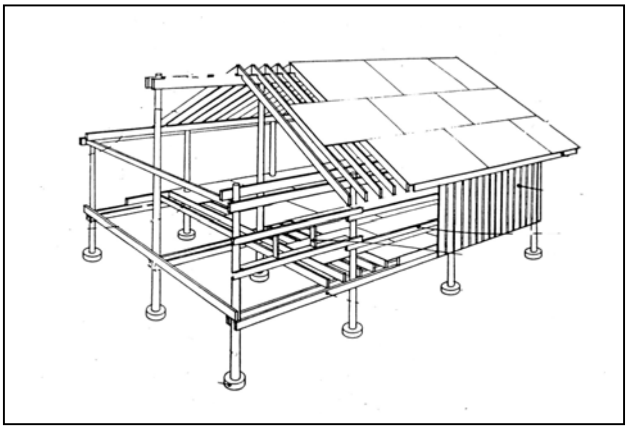
Στη θέση των κορμιδίων μπορεί να χρησιμοποιηθούν και πριστά διαστάσεων πάχους 10-15cm και ύψους 15-22cm με την ανάλογη μορφοποίηση. Ο τύπος αυτός παρουσιάζει από κατασκευαστική άποψη ιδιαίτερο ενδιαφέρον για τα ελληνικά δεδομένα γιατί υπάρχει διαθέσιμη πριστή ξυλεία ελάτης πεύκης, η οποία θα μπορούσε να αξιοποιηθεί με καλύτερο τρόπο για το σκοπό αυτό.

2.2 Κατοικίες με ξύλινους κατακόρυφους στύλους ως φέροντα στοιχεία

Ο σκελετός των σπιτιών αυτών αποτελείται από κατάλληλα εμποτισμένους υπό πίεση στύλους πεύκης, κυπαρισσιού, ψευσοτσούγκας, οι οποίοι θεμελιώνονται κατακόρυφα με ειδικό τρόπο μέσα στο έδαφος (Σχ. 2.1 και 2.2). Η διάμετρος των στύλων εξαρτάται από τον αριθμό των ορόφων του σπιτιού. Για σπίτια ενός ορόφου χρησιμοποιούνται στύλοι με διάμετρο κορυφής 11-13,5cm και για διώροφα με διάμετρο κορυφής 20-21cm, ενώ οι αποστάσεις των στύλων μεταξύ τους σε τυπικές κατασκευές μπορεί να είναι 1,8-3,0m. Σύμφωνα με υπολογισμούς ειδικών μηχανικών, το συνολικό βάρος του οικήματος μπορεί να είναι 3-6 τόνοι ανά στύλο. Οι στύλοι θα πρέπει να είναι ευθείς χωρίς σφάλματα στρεψοϊνίας και κωνικομορφίας, κυκλικής διατομής και υγιείς. Οι στύλοι εμποτίζονται υπό πίεση με υδατοδιαλυτά άλατα βορίου και συγκράτηση 12 κιλών ξηρού άλατος ανά m^3 ξυλείας.

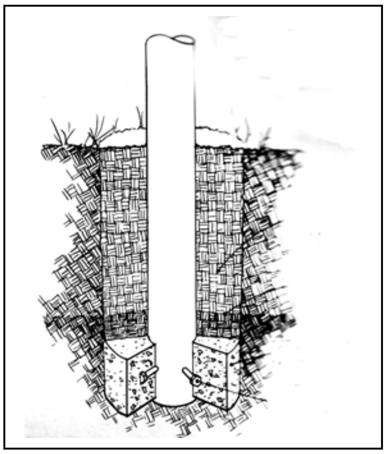


Σχ. 2.1. *Τρόπος θεμελίωσης κατοικίας με ξύλινους κατακόρυφους στύλους ως φέροντα στοιχεία.*



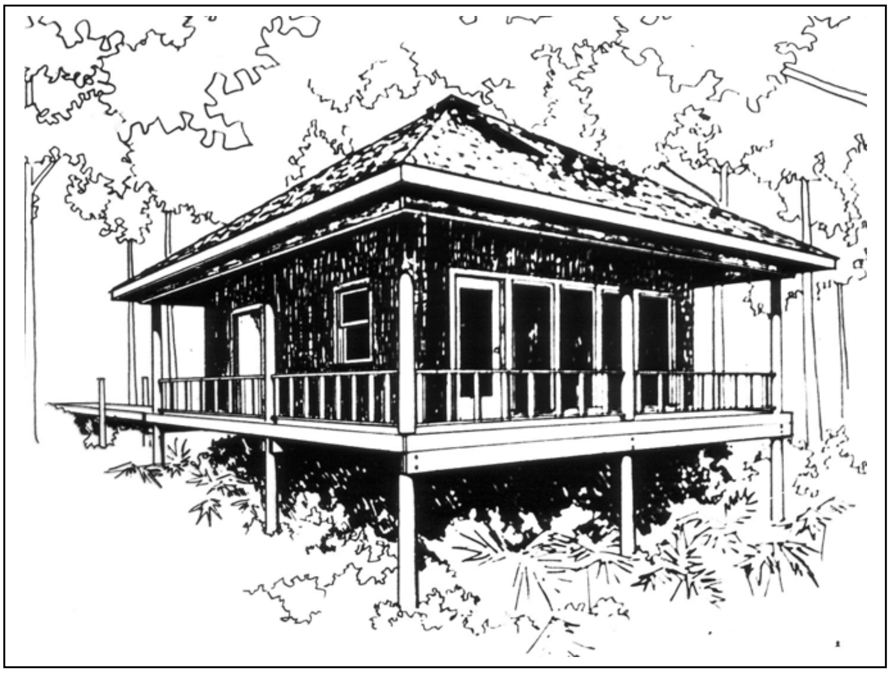
Σχ. 2.2. *Τρόπος κατασκευής κατοικίας με ξύλινους κατακόρυφους στύλους ως φέροντα στοιχεία.*

Η θεμελίωση των στύλων (Σχ. 2.3) είναι πολύ σοβαρή εργασία και γίνεται με τήρηση ορισμένων κανόνων. Το βάθος θεμελίωσης εξαρτάται κυρίως από την κλίση και την ποιότητα του εδάφους, το ύψος των στύλων από την επιφάνεια του εδάφους μέχρι το πάτωμα του ισογείου, τις αποστάσεις των στύλων μεταξύ των, τις διαμέτρους των στύλων και την έκταση της επιφάνειας που καλύπτει η θεμελίωση. Το βάθος αυτό κυμαίνεται από 1μ. μέχρι 2.40μ. η δε διάμετρος των οπών είναι περίπου 40 εκ. Κατά τη θεμελίωση τοποθετείται σκυρόδεμα σε δακτύλιο 30-40 εκ. ύψος γύρο από το στύλο και διάμετρο 40 εκ. στον πυθμένα του λάκκου και πάντα σε βάθος μεγαλύτερο του ορίου παγετού του εδάφους. Ο υπόλοιπος χώρος του λάκκου θεμελίωσης καλύπτεται με άμμο ή χαλίκι και συμπιέζεται.

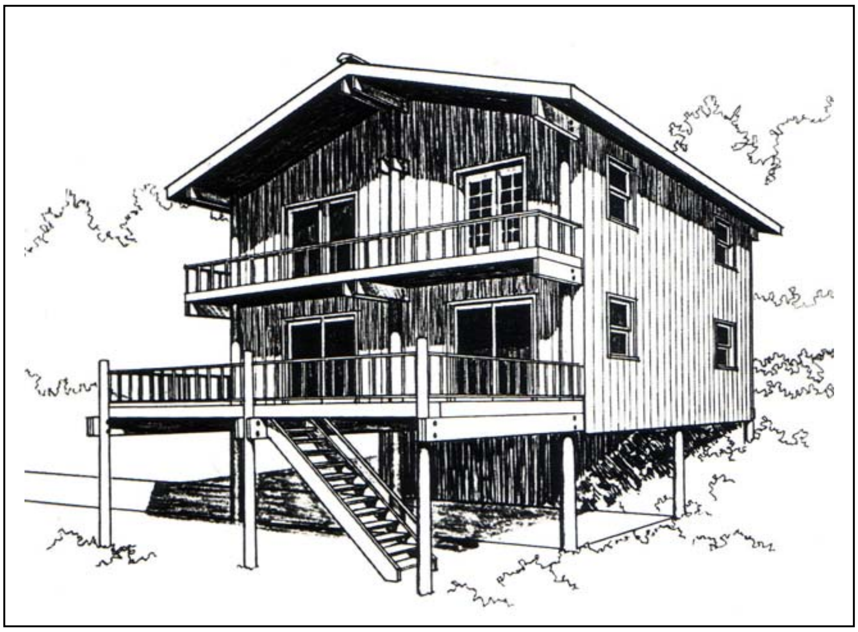


Σχ. 2.3. *Τρόπος θεμελίωσης στύλου στο έδαφος.*

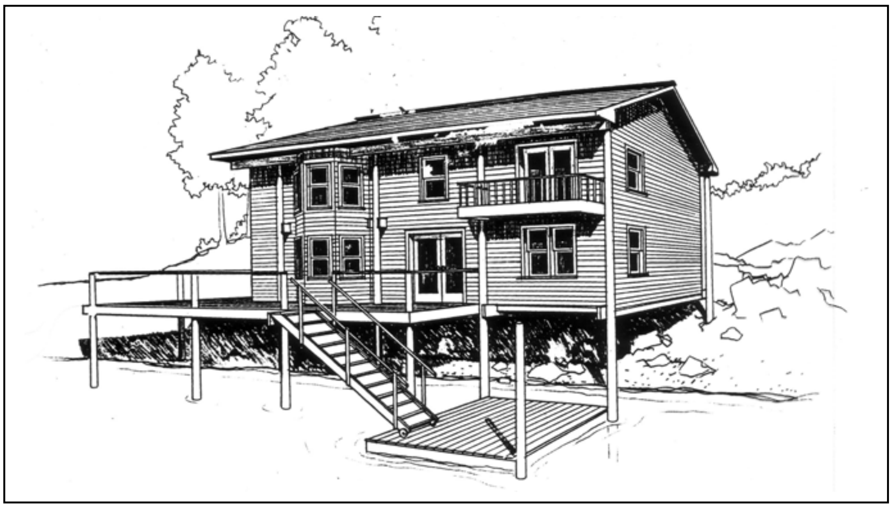
Μετά τη θεμελίωση των στύλων γίνεται η τοποθέτηση των δοκών του πατώματος, των τοιχωμάτων και της σκεπής. Η στερέωση των δοκών επάνω στους στύλους γίνεται με διαμπερείς - ανοξείδωτες ξυλόβιδες και δημιουργία κατάλληλων υποδοχών με απλά εργαλεία. Η ολοκλήρωση των εργασιών στους τοίχους, στη σκεπή, και στα πατώματα γίνεται με τις μεθόδους που περιγράφονται στην επόμενη παράγραφο. Στα Σχ. 2.4, 2.5 και 2.6, παρουσιάζονται 3 αντιπροσωπευτικοί τύποι σπιτιών αυτού του είδους.



Σχ. 2.4. *Κατοικία με ξύλινους κατακόρυφους στύλους ως φέροντα στοιχεία σε δάσος.*



Σχ. 2.5. *Κατοικία με ξύλινους κατακόρυφους στύλους ως φέροντα στοιχεία σε πλαγιά.*

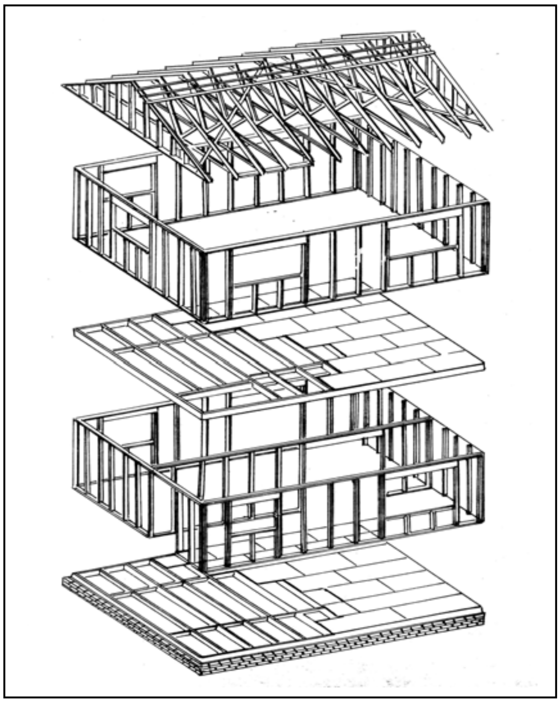


Σχ. 2.6. *Κατοικία με ξύλινους κατακόρυφους στύλους ως φέροντα στοιχεία σε λίμνη.*

Ο τύπος αυτός σπιτιού παρουσιάζει σοβαρά πλεονεκτήματα, όπως: χαμηλό κόστος και μεγάλη αντοχή σε περιπτώσεις σεισμών, πλημμυρών, τυφώνων, διότι οι ξύλινες κολόνες υποστηρίζουν και δένουν την όλη κατασκευή, απορροφώντας και μεταφέροντας τις τάσεις στο έδαφος. Προσφέρει επίσης μεγάλες δυνατότητες αρχιτεκτονικού σχεδιασμού και επιτρέπει την κατασκευή σπιτιού σε πλαγιές, σε λόφους με πετρώδη ή δύσκολα εδάφη, σε παραλίες θαλασσών ή λιμνών, χωρίς να θιχθεί εντελώς η διαμόρφωση του εδάφους. Τα σπίτια αυτά είναι κατάλληλα για παραθεριστική κατοικία σε απομακρυσμένες περιοχές, για εγκαταστάσεις συνεργείων, αποθηκών, γκαράζ κ.α.

2.3 Κατοικίες από ελαφρύ ξύλινο σκελετό

Πρόκειται για την κλασσική τεχνική κατασκευής ξύλινου σκελετού σπιτιού, η οποία έχει επικρατήσει και χρησιμοποιείται τόσο για τη μέθοδο προκατασκευής στο εργοστάσιο, όσο και για τη μέθοδο της επιτόπου κατασκευής σπιτιού (Σχ. 2.7). Ο ξύλινος σκελετός των εξωτερικών τοίχων αποτελείται από κατακόρυφους ορθοστάτες με διατομή περίπου 50 x 100mm, οι οποίοι τοποθετούνται σε διαστήματα ανά 50 έως 60cm (κέντρο από κέντρο). Οι ορθοστάτες αποτελούν τα κατακόρυφα στοιχεία των πλαισίων του σκελετού και στερεώνονται στα άκρα τους με οριζόντια στοιχεία ιδίας διατομής, ενώ δένονται οριζόντια με τραβέρσες ιδίας διατομής ανά 60cm. Η κάθε πλευρά των εξωτερικών και των εσωτερικών τοίχων μπορεί να αποτελείται από δύο, τρία ή και περισσότερα πλαίσια σκελετού, ανάλογα με το μήκος της κάθε πλευράς.



Σχ. 2.7. *Στοιχεία κατοικίας από ελαφρύ ξύλινο σκελετό.*

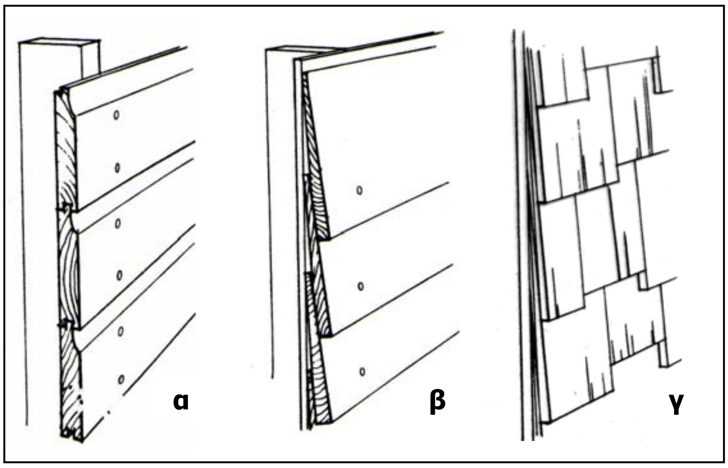
Τα πλαίσια του σκελετού στερεώνονται στο κάτω μέρος τους κατά τη διάρκεια του στησίματος του σκελετού, επάνω σε ξύλινους στρωτήρες διατομής 7x15cm, οι οποίοι πακτώνονται στη βάση από μπετό περιμετρικά με ειδικά μεταλλικά, ανοξείδωτα βύσματα, υψηλής αντοχής. Εναλλακτικά αντί βάσης μπετού ως θεμελίωση μπορούν να χρησιμοποιηθούν εμποτισμένοι στύλοι πεύκης οι οποίοι πακτώνονται περιμετρικά όπως αναλύεται παρακάτω (Εικ. 2.7).

Εικ. 2.7. *Στερέωση πλαισίων σκελετού σε ξύλινους στρωτήρες.*

Οι στρωτήρες αυτοί, συνήθως είναι εμποτισμένοι με υδατοδιαλυτά άλατα εμποτισμού. Οι στρωτήρες θα πρέπει να τοποθετούνται με προσοχή περιμετρικά στη βάση από μπετό, κατά τρόπο ώστε μετά την τοποθέτηση της εξωτερικής επένδυσης του σκελετού, οι εξωτερικού τοίχοι να προεξέχουν κατά 2 εκ. της βάσης μπετού, ώστε η βροχή που χτυπάει πλάγια τους εξωτερικούς τοίχους να μη προσεγγίζει τον ξύλινο σκελετό.

Τα πλαίσια κάθε πλευράς στερεώνονται στην πάνω πλευρά τους κατά το στήσιμο με ξύλινους δοκούς διατομής 7x10cm, ώστε να εξασφαλίζεται αντοχή και περιμετρικό δέσιμο όλου του σκελετού. Η στερέωση γίνεται με ανοξείδωτα καρφιά ή ξυλόβιδες. Τα στοιχεία κάθε πλαισίου στερεώνονται μεταξύ τους πλευρικά με ειδικές μεταλλικές πλακέτες συνδέσεως που φέρουν στην επιφάνειά τους καρφιά. Η τοποθέτηση των πλακετών συνδέσεως γίνεται πλευρικά με ειδικές καρφωτικές μηχανές.

Στην εξωτερική πλευρά των εξωτερικών τοίχων καρφώνονται ξυλοπλάκες εξωτερικής χρήσεως ή σανίδια πεύκης τύπου ραμποτέ (Σχ. 2.8 και Εικ. 2.8).



Σχ. 2.8. *Επικαλύψεις εξωτερικών τοίχων με:*
α: σανίδες τύπου ραμποτέ, β και γ: σανίδες επικάλυψης.

Εικ. 2.8. *Επικάλυψη εξωτερικού τοίχου με σανίδες επικάλυψης.*

Στην εσωτερική πλευρά τοποθετείται φράγμα υδρατμών (PVC) και μετά γυψοσανίδα πάχους 12mm ή τσιμεντοσανίδα τύπου ηρακλείτη, η οποία σοβατίζεται, ή διάφοροι τύποι ξυλοπλακών, ή σανίδια τύπου ραμποτέ. Στο πάχος του σκελετού τοποθετείται συνήθως μόνωση από υαλοβάμβακα, πετροβάμβακα ή πολυουρεθάνη και διαπερνώνται ειδικοί σωλήνες για παροχή ρεύματος, νερού, τηλεφώνου κτλ.
Για λόγους ευκολίας στην προκατασκευή και το στήσιμο του σπιτιού η όλη κατασκευή χωρίζεται σε τμήματα υπό μορφή πλαισίων (βλ. Εικ. 2.7). Τα πλαίσια προκατασκευάζονται στο εργοστάσιο σε διάφορα μεγέθη και είτε είναι ολοκληρωμένα (σκελετός, μόνωση, εξωτερική – εσωτερική επένδυση), είτε αποτελούνται μόνο από τον σκελετό και οι υπόλοιπες εργασίες ολοκληρώνονται επί τόπου (Εικ. 2.9).

Εικ. 2.9. *Προκατασκευή πλαισίων τοιχωμάτων.*

Η κατασκευή των εσωτερικών τοίχων είναι ανάλογη με την κατασκευή των εξωτερικών. Η χρήση της γυψοσανίδας σαν υλικό επένδυσης παρέχει και αντιπυρική προστασία.

Στις θέσεις των παραθύρων και θυρών προβλέπονται αντίστοιχα ανοίγματα κατά το σχεδιασμό και την κατασκευή των πλαισίων. Οι πόρτες και τα παράθυρα μπορεί επίσης να ενσωματωθούν στα πλαίσια ή να τοποθετηθούν επί τόπου. Μεταξύ ισογείου και πρώτου ορόφου μεσολαβεί ο σκελετός του πατώματος του πρώτου ορόφου, ο οποίος έχει τη μορφή πλατφόρμας (βλ. Σχ. 2.7) και πάνω σ αυτόν στερεώνεται ο σκελετός των τοίχων του πρώτου ορόφου. Η απλή περιγραφή όπως έγινε παραπάνω, αντιπροσωπεύει την τυπική παραδοσιακή κατασκευή του σκελετού σπιτιών που κυριάρχησε στη Βρετανία την περίοδο 1960-1970. Ο τύπος αυτός με μικρές παραλλαγές έχει υιοθετηθεί και χρησιμοποιείται σε μεγάλη κλίμακα στις ΗΠΑ και τον Καναδά. Τα σπίτια του τύπου αυτού χτίζονται εξωτερικά με τούβλα και για το λόγο αυτό δεν διαφέρουν από τα συνήθη για τα δικά μας δεδομένα σπίτια. Η τεχνική αυτή προσφέρει γενικά ενισχυμένη θερμοηχομόνωση και πυρασφάλεια, ενώ η κατασκευή είναι γρηγορότερη και σε ανταγωνιστικότερη τιμή για τα Βρετανικά δεδομένα.

Θεμελίωση

Η θεμελίωση σπιτιών με ξύλινο σκελετό γίνεται με διάφορους τρόπους όπως:
- Από οπλισμένο σκυρόδεμα (πλάκα μπετού) πάνω στην οποία στερεώνεται κατάλληλα ο σκελετός του σπιτιού. Μπορεί ακόμη να έχει τη μορφή τοιχίου που διατρέχει περιφερειακά την κατοικία και πάνω στο οποίο στηρίζονται οι εξωτερικοί τοίχοι του σπιτιού. Στην τελευταία περίπτωση οι εσωτερικοί τοίχοι και οι

κολόνες του πατώματος στηρίζονται σε εσωτερικές κολόνες από μπετό ή εμποτισμένο ξύλο που πακτώνεται κατάλληλα μέσα στο έδαφος.

- Από εμποτισμένους ξύλινους στύλους οι οποίοι πακτώνονται μέσα στο έδαφος σε ορισμένα διαστήματα (Εικ. 2.10). Η τελευταία μέθοδος ενδείκνυται για κατασκευές σε πολύ υγρές περιοχές ή απομακρυσμένες, όπου είναι δύσκολη η μεταφορά μηχανημάτων και υλικών, κυρίως για παραθεριστική κατοικία ή οικήματα συνεργείων, εργοταξίων κλπ.

Εικ. 2.10. *Θεμελίωση σπιτιού με πακτωμένους ξύλινους στύλους.*

Κατασκευή στέγης

Η στέγη είναι δυνατό να αποτελείται από προκατασκευασμένα ζευκτά, τα οποία τοποθετούνται και στερεώνονται στο σκελετό των τοιχωμάτων, σε απόσταση 80-100cm μεταξύ τους (κέντρο από κέντρο) (Εικ. 2.11 και 2.12). Ο σκελετός της στέγης μπορεί να κατασκευάζεται και επί τόπου σε πιο απλή μορφή, κυρίως σε δίρρικτη ή τετράρριχτη στέγη.

Εικ. 2.11. *Τοποθέτηση προκατα-σκευασμένων ζευκτών στέγης.*

Εικ. 2.12. *Κατασκευή στέγης από προκατασκευασμένα ζευκτά.*

Εικ. 2.13. *Τοποθέτηση σανιδώματος στη στέγη.*

Μετά την τοποθέτηση του σκελετού της στέγης ακολουθεί το κάρφωμα του σανιδώματος της στέγης με σανίδια (Εικ. 2.13) ή αντικολλητά εξωτερικής χρήσεως ή πλάκες μοριοσανίδας τύπου OSB. Πάνω από το σανίδωμα τοποθετείται ειδική μεμβράνη ή πισσόχαρτο στεγανοποίησης, τα καδρόνια στέγης και η επικάλυψη της στέγης (κεραμίδια, λαμαρίνα ή άλλα υλικά) (Εικ. 2.14 και 2.15).

Για καλύτερη θερμομόνωση πάνω από το πισσόχαρτο ή την ειδική μεμβράνη καρφώνονται δύο σειρές καρδονιών στέγης διατομής 5x7cm με τη διάσταση των 7cm κατακόρυφα. Η πρώτη σειρά καδρονιών καρφώνεται σε απόσταση 50cm μεταξύ τους και παράλληλα προς την κλίση της στέγης. Στα κενά των καδρονιών αυτών τοποθετείται μόνωση από εξηλασμένη πολυστυρόλη, πετροβάμβακα ή άλλο υλικό. Η δεύτερη σειρά καδρονιών διατομής 5x7cm καρφώνεται κάθετα προς την πρώτη σειρά με την διάσταση των 7cm κατακόρυφα και σε αποστάσεις μεταξύ τους 30cm κέντρο από κέντρο, εάν ως επικάλυψη πρόκειται να στερεωθούν κεραμίδια (Εικ. 2.14 και 2.15).

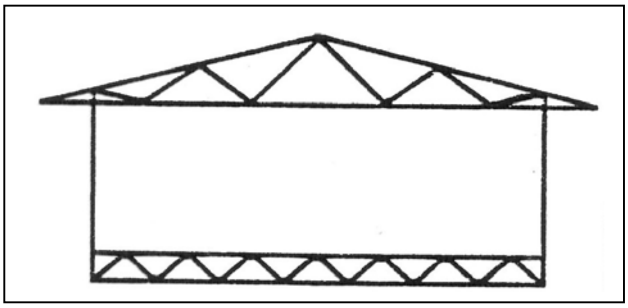
Εικ. 2.14. *Τοποθέτηση πισσόχαρτου στεγανοποίησης και καδρονικού για στερέωση κεραμιδιών.*

Εικ. 2.15. *Τοποθέτηση κεραμιδιών στη στέγη.*

Εάν η επικάλυψη είναι αυλακωτή λαμαρίνα ή σάντουϊτς από δύο λαμαρίνες και μόνωση στη μεσαία στρώση, τα καδρόνια στερεώνονται ανά 50cm. Στη χώρα μας συνήθως αποφεύγεται το σανίδωμα και το καδρονικό της στέγης καρφώνεται κατευθείαν επάνω στα ζευκτά. Η τεχνική όμως αυτή περιορίζει τη μόνωση.

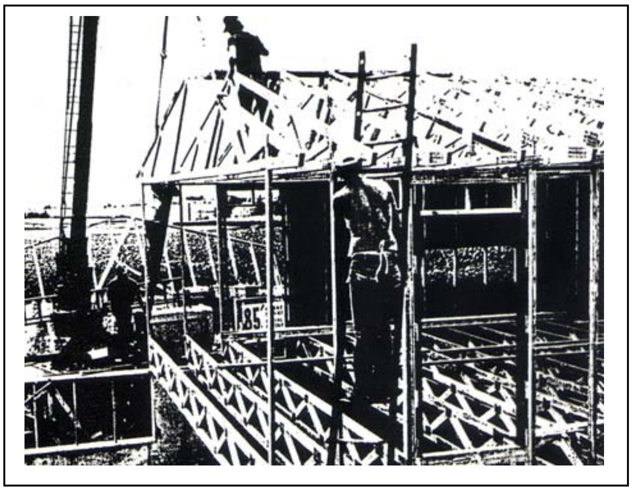
2.4. Κατοικίες με σκελετό τύπου Truss Framed System (TFS)

Πρόκειται για νέο σύστημα ελαφρού ξύλινου σκελετού από ολόσωμα πλαίσια, το καθένα από τα οποία περιλαμβάνει μια δικτυωτή δοκό σκελετού του πατώματος, δύο κοινούς ορθοστάτες εξωτερικών τοίχων και ένα αιωρούμενο δικτυωτό στέγης (Σχ. 2.9). Όλη η εγκάρσια τομή του φέροντος σκελετού του πατώματος, των εξωτερικών τοίχων και της στέγης, είναι δεμένη σε μια κατασκευαστική μονάδα.



Σχ. 2.9. *Τύπος ολόσωμου πλαισίου.*

Ο τύπος αυτός της προκατασκευής δημιουργήθηκε από το Εργαστήριο Δασικών Προϊόντων της Δασικής Υπηρεσίας των ΗΠΑ με σκοπό την πληρέστερη αξιοποίηση των δασικών προϊόντων. Παράλληλα στόχευε στη βελτίωση των υπαρχόντων τύπων ξύλινων σκελετών και την αντιμετώπιση προβλημάτων ανεπαρκών συνδέσεων μεταξύ των στοιχείων στέγης – τοιχωμάτων και των στοιχείων τοιχωμάτων – βάσης, που παρουσιάζονταν μετά από θύελλες, σεισμούς, τυφώνες, στους σκελετούς αυτούς.



Εικ. 2.17. *Κατασκευή κατοικίας με ολόσωμα πλαίσια.*

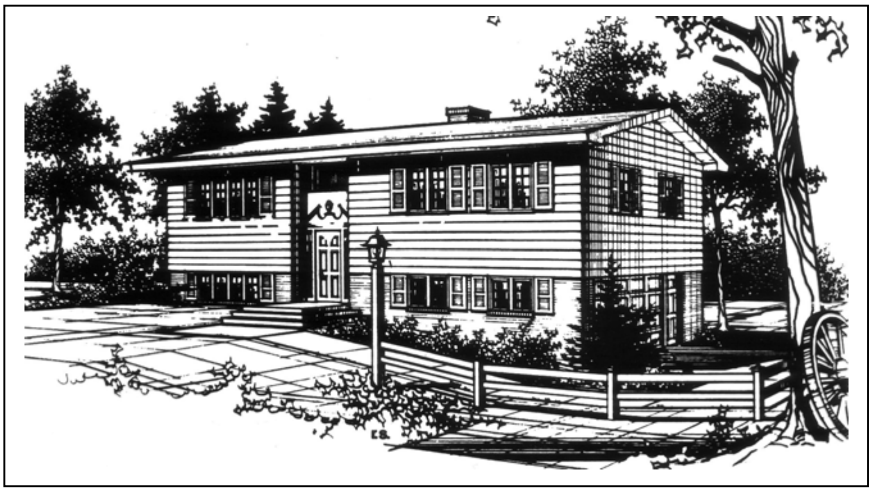
Με το σύστημα T.S.F. εξασφαλίζεται κατασκευαστική συνέχεια από τα θεμέλια μέχρι την οροφή και αντιμετωπίζονται κοινά προβλήματα συνδέσεων (Εικ. 2.17).

Κάθε στοιχείο του σκελετού ανθίσταται σε επίδραση δυνατού αέρα, φορτίων χιονιού και σε άλλες φορτίσεις που ασκούνται σε οποιοδήποτε σημείο της κατασκευής. Με το σύστημα αυτό σύμφωνα με την άποψη των μηχανικών ξύλου που το σχεδίασαν, επιτυγχάνεται χαμηλό κόστος κατασκευής, άνετος σχεδιασμός, γρήγορη προκατασκευή, γρήγορη ανέγερση της κατοικίας και υψηλή ασφάλεια.

Για τις περισσότερες εφαρμογές, η κατασκευή των πλαισίων γίνεται αποκλειστικά από καδρόνια πάχους 5cm και πλάτους 10cm, ενώ δεν είναι απαραίτητοι οι δοκοί και οι κολόνες υποστήριξης στο ισόγειο. Άλλα πλεονεκτήματα του συστήματος είναι:

- Μειωμένη ποσότητα ξυλείας, μέχρι και 30%, σε σύγκριση με κοινές προκατασκευές.
- Μειωμένη εργασία κατασκευής αγωγών για θέρμανση και κλιματισμό, γιατί μπορεί να χρησιμοποιηθεί το κενό των 52cm του σκελετού πατώματος.
- Επειδή δεν απαιτούνται ενδιάμεσα υποστηρίγματα μεταξύ των εξωτερικών τοίχων του σπιτιού (άνοιγμα ζευκτών), αποφεύγονται κολόνες, δοκοί και φέροντες εσωτερικοί τοίχοι διαίρεσης.

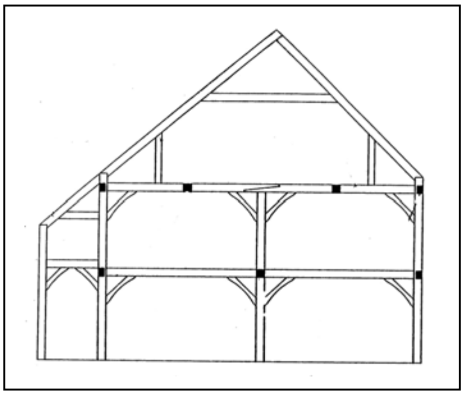
Ο τύπος T.F.S. ωστόσο υιοθετήθηκε μόνο για μικρό αριθμό κατασκευών μέχρι σήμερα. Στο Σχ. 2.10 παρατηρούμε μια ολοκληρωμένη κατοικία.



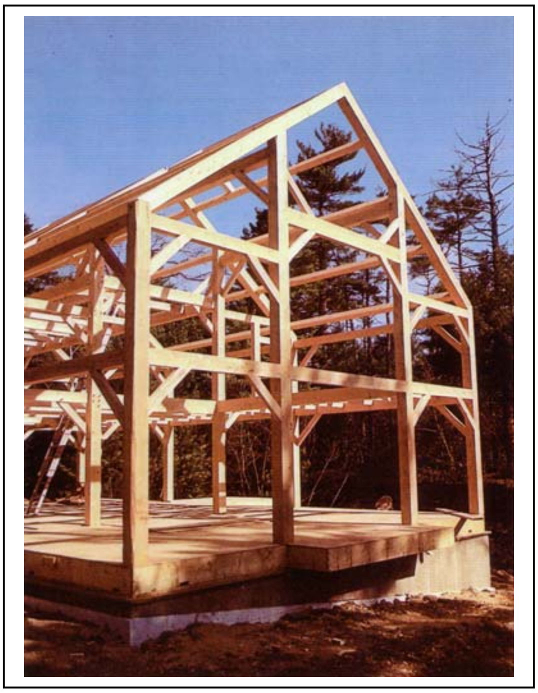
Σχ. 2.10. *Κατοικία με σκελετό τύπου Truss Framed System (TFS)*

2.5. Διώροφες κατοικίες από ενισχυμένο ξύλινο σκελετό και ενισχυμένα πάνελς τύπου σάντουϊτς

Στον τύπο αυτό προκατασκευάζεται και στήνεται ενισχυμένος ξύλινος σκελετός τοίχων και στέγης (Σχ. 2.11 και Εικ. 2.18). Στην εξωτερική πλευρά του σκελετού και στα πατώματα στερεώνονται ενισχυμένα προκατασκευασμένα πάνελς. Σε κάθε ενισχυμένο πλαίσιο σκελετού ενσωματώνονται στοιχεία σκελετού τοίχων ισογείου και ορόφου (κολόνα), οριζόντια δοκάρια τοίχων και πατωμάτων, καθώς και δοκοί στέγης (Εικ. 2.19). Η τεχνική αυτή εξασφαλίζει υψηλή μηχανική αντοχή και διευκολύνει στο στήσιμο των ολόσωμων πλαισίων με γερανό. Τα ολόσωμα πλαίσια συνδέονται μεταξύ τους με οριζόντιες δοκούς. Τα ενισχυμένα πάνελς τύπου σάντουϊτς αποτελούνται από δύο επιφάνειες ξυλοπλακών (αντικολλητά, O.S.B.), μεταξύ των οποίων συγκολλάται στρώση μόνωσης από διογκωμένη πολυστυρόλη ή πολυουρεθάνη. Η μία από τις ξυλοπλάκες είναι εξωτερικής χρήσεως και η άλλη εσωτερικής. Τα πάνελς αυτά, γνωστά ως stress skin panels, έχουν υψηλή μηχανική αντοχή και συμπεριφέρονται κατά τη φόρτιση ως δοκός τύπου διπλού ταφ. (Ι). Εξασφαλίζουν υψηλή μόνωση, χαμηλό κόστος παραγωγής και μεγάλη ταχύτητα τοποθέτησης. Δοκιμάσθηκαν για πρώτη φορά στις ΗΠΑ το 1937. Στην αγορά εμφανίσθηκαν το 1959 και η νέα τεχνολογία άρχισε να γίνεται αποδεκτή το 1980. Σήμερα αντιπροσωπεύουν λιγότερο από 1% των νέων κατοικιών στις ΗΠΑ.



Σχ. 2.11. *Ενισχυμένος ξύλινος σκελετός.*



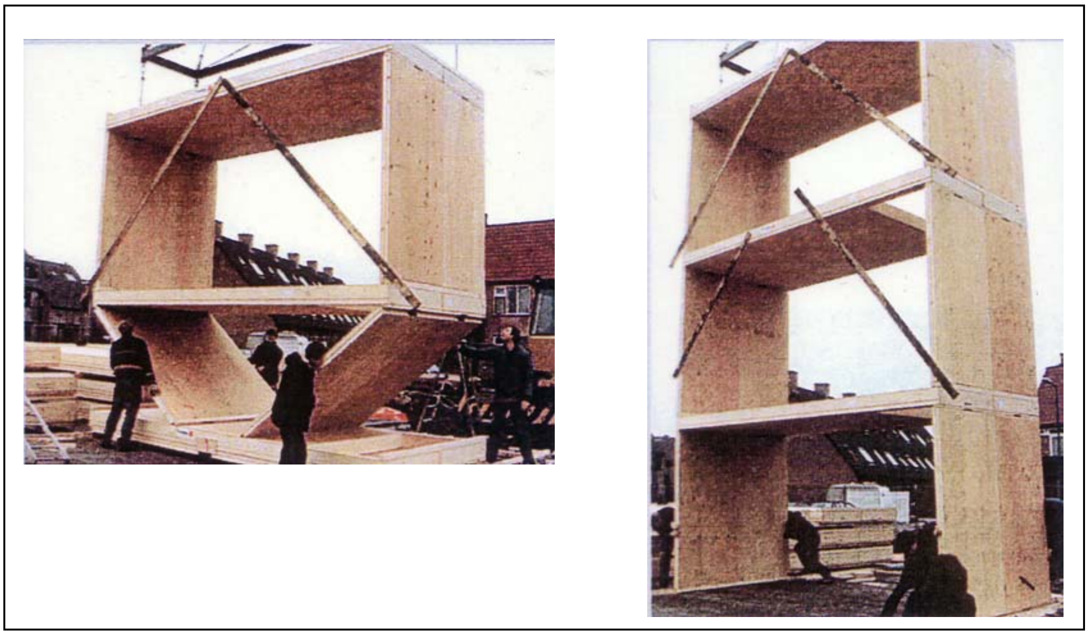
Εικ. 2.18. *Κατασκευή κατοικίας με ενισχυμένο ξύλινο σκελετό.*

Εικ. 2.19. *Εσωτερικό κατοικίας από ενισχυμένο ξύλινο σκελετό.*

2.6 Προκατασκευασμένα κτίρια με πτυσσόμενα (αναδιπλούμενα) πλαίσια (τεχνολογία FOLDEX)

Με την τεχνική FOLDEX προκατασκευάζονται κτίρια μέχρι 2 ορόφους και δώμα από προκατασκευασμένα αναδιπλούμενα πλαίσια (Εικ. 2.20 και 2.21). Διακρίνονται πλαίσια οροφής, τοίχων και πατωμάτων. Το κάθε πλαίσιο τύπου σάντουϊτς αποτελείται από 3 στρώσεις, όπως τα πλαίσια που περιγράφηκαν παραπάνω, με τη διαφορά ότι στη μεσαία στρώση εμπεριέχεται και ο ξύλινος σκελετός. Το γεγονός αυτό προσδίδει στα πλαίσια πολύ μεγάλη μηχανική αντοχή, που επιτρέπει τη χρήση τους ως στοιχεία στέγης, τοίχων και πατωμάτων. Η εξωτερική στρώση των πλαισίων τοίχων και στέγης είναι τσιμεντοσανίδα. Τα πλαίσια προκατασκευάζονται σε ειδικά εργοστάσια με θερμή πολυώροφη πρέσα υψισύχνων, γεγονός που εξασφαλίζει υψηλή παραγωγή και χαμηλό κόστος. Η τεχνολογία FOLDEX ενδείκνυται για συγκροτήματα κατοικιών, γραφείων, Σχολείων, καταστημάτων και εξασφαλίζει μαζική παραγωγή, χαμηλό κόστος, πολύ γρήγορη εγκατάσταση με γερανό (5 ώρες για κατοικία 120m^2) και μεγάλη οικονομία υλικών.

Εικ. 2.20. *Μεταφορά αναδιπλούμενων πλαισίων τύπου FOLDEX.*



Εικ. 2.21. *Εγκατάσταση κτιρίου με αναδιπλούμενα πλαίσια.*

2.7. Πολυόροφα κτίρια με ξύλινο σκελετό

Τα πολυόροφα κτίρια με ξύλινο σκελετό αποτελούν κοινή πρακτική στη Β. Αμερική. Στην Κ. Ευρώπη τα κτίρια αυτά βρίσκονται σε αρχικό στάδιο εφαρμογής. Τα τελευταία 10 χρόνια στη Γερμανία, Αυστρία, Ελβετία, Γαλλία κατασκευάσθηκαν περίπου 500 πολυώροφα ξύλινα κτίρια με 6.000 διαμερίσματα. Παρατηρείται μια τάση αύξησης της ζήτησης, η οποία οφείλεται στην ανερχόμενη ζήτηση προϊόντων από ανανεώσιμες πηγές ενέργειας.. Στη Β. Ευρώπη η κατασκευή τέτοιων κτιρίων είναι πιο διαδεδομένη.

Ο σκελετός σε Ευρώπη και Αμερική κατασκευάζεται με την κλασική μέθοδο πλατφόρμας (5x10cm). Τα πλαίσια σκελετού τοίχων σε κάθε όροφο χωριστά επενδύονται στις δύο όψεις με πάνελς, τα οποία είτε προκατασκευάζονται (Ευρώπη), ή κατασκευάζονται επί τόπου (Αμερική). Τα πατώματα είναι τύπου πλατφόρμας, και οι κολόνες έχουν ύψος μόνο ενός ορόφου. Μια άλλη μέθοδος προβλέπει κατασκευή των πλαισίων από ξύλο μασίφ. Μια τρίτη μέθοδος προβλέπει κολόνες που διαπερνούν πολλούς ορόφους, επάνω στις οποίες στηρίζονται τα οριζόντια δοκάρια του σκελετού.

Το 2000 κατασκευάσθηκε με συνεργασία πολλών μηχανικών ξύλου από διάφορες χώρες της Ευρώπης εξαόροφο ξύλινο κτίριο 2.000m^2, για να μελετηθούν οι μέθοδοι τεχνικής στην προκατασκευή και το στήσιμο, η χρήση νέων προϊόντων, η βελτίωση της πυρανθεκτικότητας και της θερμοηχομόνωσης, η μείωση του κόστους και ο εναρμονισμός των μεθόδων παραγωγής και των προτύπων σε διάφορες χώρες της Ευρώπης (Εικ. 2.22).

__Εικ.__ 2.22. Εξαώροφο κτίριο με ξύλινο σκελετό.

3. Συνήθη σφάλματα σε δομικές κατασκευές ξύλου στην Ελλάδα

Στη χώρα μας είναι γεγονός ότι κυριαρχεί το μπετό σε κατασκευές με τις γνωστές δυσάρεστες συνέπειες. Η έλλειψη ειδικών μηχανικών, εργολάβων και τεχνητών ξύλινων δομικών κατασκευών έχει ως αποτέλεσμα την εσφαλμένη εφαρμογή της απαιτούμενης τεχνολογίας. Τα συνήθη σφάλματα που παρουσιάζονται στη χώρα μας είναι τα ακόλουθα:

- Μπερδεύουμε τα διάφορα είδη δομικής ξυλείας κυρίως τη λευκή ξυλεία κωνοφόρων (ελάτη, ερυθρελάτη) με την ξυλεία πεύκης (σουηδική ξυλεία). Κατά κανόνα η ξυλεία πεύκης έχει μεγαλύτερη μηχανική αντοχή και ανθεκτικότητα και χρησιμοποιείται σε εξωτερικές και εσωτερικές κατασκευές, ενώ η λευκή ξυλεία σε εσωτερικές.
- Χρησιμοποιούμε υγρό ξύλο με τις γνωστές συνέπειες. Η υγρασία του ξύλου για εξωτερικές χρήσεις πρέπει να είναι 12-15% και για εσωτερικές 8-10%.
- Χρησιμοποιούμε ξύλινα στοιχεία στέγης με μικρές διατομές.
- Ενίοτε γίνεται λανθασμένη επιλογή εμποτισμένου ξύλου σε εσωτερικές κατασκευές. Η χρήση CCA απαγορεύτηκε, ενώ ο εμποτισμός με βορικά άλατα επιτρέπεται σε δομικές κατασκευές όπου ο άνθρωπος δεν έρχεται σε επαφή με το εμποτισμένο ξύλο.

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Experimental studies on structural timber glass adhesive bonding

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Abstract: *The utilization of timber in building structures has been increasing in the last years, partly due to technological developments in this field. New tectonic concepts such as prefabricated and industrialised systems arose, though very different from the ones presented by traditional solutions. Hence, the combination of timber with other constructive materials is emerging. Steel, composites materials or glass can be combined with timber, in order to obtain resistant products, highly typified, standardised and with low behavioural variation.The structural utilization of timber glass composite solutions is a daring construtive system, which although still in a very early stage, already presents an important potential of applicability in architecture. In order to fully benefit from composite timber glass cross sections, an adequate bonding between these two elements is essential. With the purpose of achieving the ideal balance between strength and flexibility, an extensive set of experimental tests is being carried out at the University of Minho.This presentation focus on the analysis of results regarding shear stress tests with timber glass bondings, using adhesive as structural bonding system. In these tests, various adhesives were applied, including different trades and adhesive types such as silicone, methacrylate, polyurethane, acrylics and superflex polymers.*

Introduction

The technical evolution has been responsible for a significant rise in the structural utilization of timber in construction. Prefabricated industrialised systems currently present an area of strong architectural and constructive development. The uniformity, traditionally inherent in this concept, is nowadays superimposed by the feasability of unitary series. As a complement, the increasing resort to composite solutions diversifies the range of functional, expressive and structural solutions of products, leading to results that any material alone, with its limitations, could not achieve. Therefore, any composite solution will aim at enhancing the intrinsic increased value of its components and the simultaneous minimization of the disadvantages that each material, separately, presents. This is the central idea behind composite systems and also the starting point for the development of the present research.

Structural timber-glass composite solutions present all conditions for, in a near future, assume great architectural significance. First of all, it will allow benefiting from natural lighting in a way not much explored so far, with consequent advantages at other levels. On the other hand, the transparency of glass, associated with its structural employment, could achieve the most transcendent features of this material, magic and illusion. Also at a structural level, glass compression capacity and timber tension resistance must be enhanced, the same way that accumutaded and tensile stresses must be avoided on glass surfaces. Simultaneously, the specificities of natural behaviour of timber have to be assumed, understood and contextualised.

However, no matter what the object of the strutural composite system is – beams, columns, plates or slabs –, another aspect must be taken into consideration: the bonding system. While assuming a role which is as important as, or even more important than the two original elements, the bonding system will be the main responsible for the unity of the set and, at the same time, preserve the diversity of components. As far as the present research is concerned, the structural adhesive bonding was chosen as the bonding system since it presents a superior guarantee of effectiveness regarding the above mentioned intention. Uniform distribution of forces, reduction of fragilization of materials by avoiding drilling, averting of high peak stresses and aim at the ductility in the unity of the set were the criteria for this decision.

It is also of great importance that the adhesive brings together strength and flexibility. That is the path to its structural employment, necessarily subject to transmission of heavy loading. The adhesive must also allow bending, expansion and shrinkage of timber, according to loading and humidity variations. Given the basic difference of characteristics between glass (brittle) and timber (ductile), it is believed that this could be the best way of enhancing the performance of the different composing elements in a unitary set.

Nowadays, there are still few examples of buildings constructed, in which structural load transfering by means of adhesives is applied. The research now presented fits in a wider project, whose main objectives are the feasability and optimization of architectural potentialities of timber glass constructive solutions.

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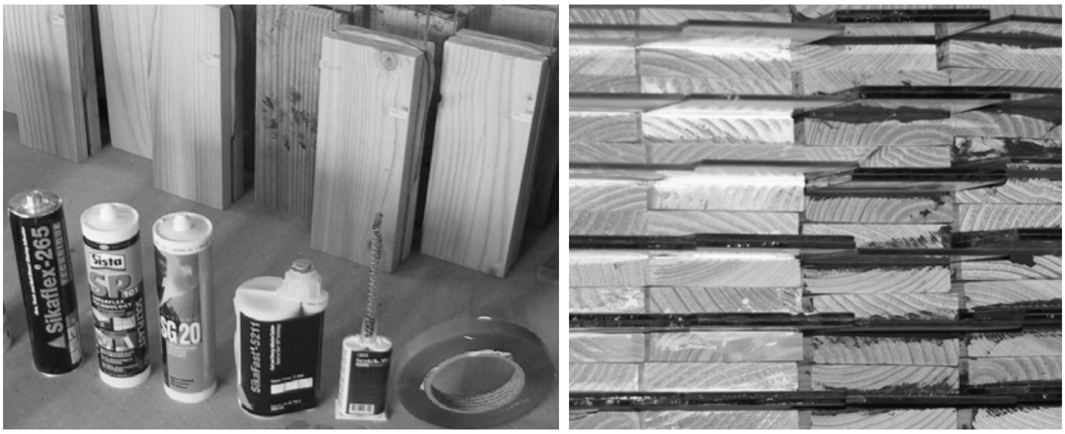
Materials and Methods

Given the unpredictability resulting from the many variables related to timber glass structural adhesive bonding system, it would be impossible to firmly move towards a technical and pratical solution without producing a wide set of experimental tests, which could reveal most of the possibilities.

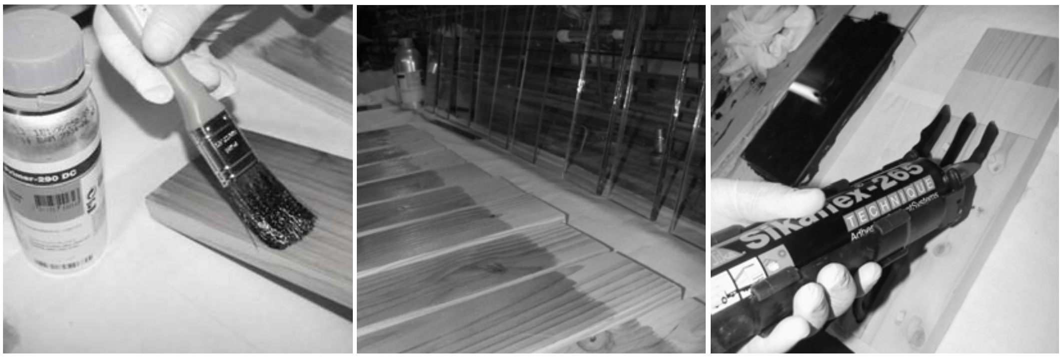
Hence, shear stress tests were carried out, according to the implementation of some important variables. Based on such tests, the results of a set of six different products are hereby presented.



Figs. 1 *and **2**: Adhesives and specimens used in the set of experimental tests*

All six products, shown on figure 1, represent different types of adhesives: polyurethane, superflex polymers, silicone, methacrylate and acrylic – in two-component format and bi-adhesive tape.

These adhesives, suggested for this purpose by the manufacturing companies themselves, within their range of products, gave rise to 54 specimens – figure 2 –, tested according to variables presented on Table 1: primers utilization and glass type. It is also important to state that, when composing and preparing the specimens – sequence of figures 3, 4 and 5 –, and apart from the use of primers, glass plates were duly degreased with dimethyl ketone and dried. Timber elements were cleaned of dust with compressed air.

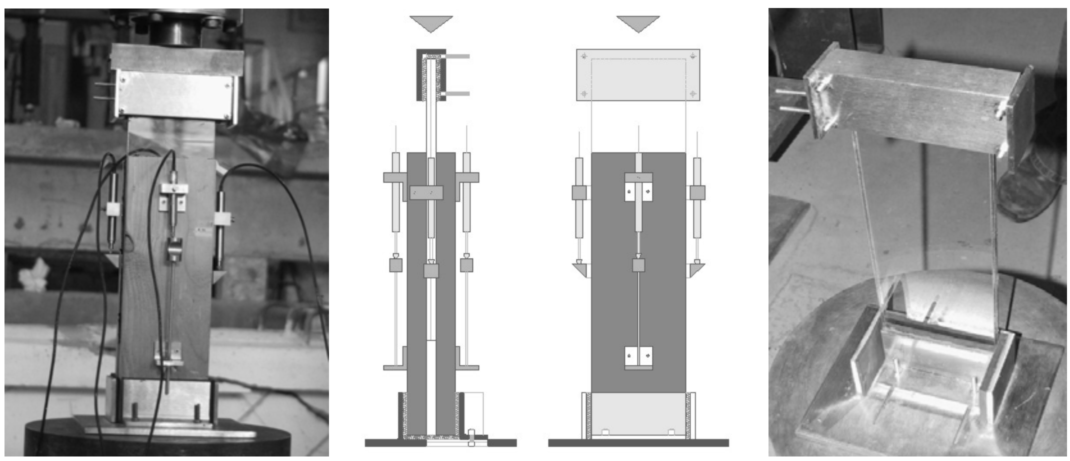


Figs. 3, 4 *and **5**: Preparation of specimens – treatments and adhesive application*

***Table 1**: Products and variables applied to the set of experimental tests*

Adhesive Type	*Product*	*Cp^S*	*Primers*	*Glass Type*	*Specimen series*
Polyurethane	Sikaflex®265	1C	-	Laminated	A04; A05; A06
			Sika®Activator Sika®Primer290DC	Tempered	A07; A08; A09
				Laminated	A10; A11; A12
Silicone	Sikasil®SG-20	1C	-	Laminated	B04; B05; B06
			Sika®Primer290DC	Tempered	B07; B08; B09
				Laminated	B10; B11; B12
Superflex Polymers	Sista Solyplast SP101	1C	-	Laminated	C04; C05; C06
			Generic Primer	Tempered	C07; C08; C09
				Laminated	C10; C11; C12
Methacrylate	Sikafast®5211	2C	-	Laminated	D04; D05; D06
			Sika®ADPrep	Tempered	D07; D08; D09
				Laminated	D10; D11; D12
Acrylic	3M™ Scotch-Weld™ DP-810	2C	-	Laminated	E04; E05; E06
			3M Glass Silane Generic Primer	Tempered	E07; E08; E09
				Laminated	E10; E11; E12
Acrylic Tape	3M™ VHB™ 4910F	-	-	Laminated	F04; F05; F06
			3M Glass Silane Generic Primer	Tempered	F07; F08; F09
				Laminated	F10; F11; F12

According to the scheme of tests used, as presented in figures 6 and 7, specimens were submitted to shear loading, in series of three, at the speed of 15 microns/s. This leads to the collection of data regarding strength resistance; timber glass relative displacement allowed by the adhesive and deformation of timber following the longitudinal orientation of the grain, resulting from loading transmission through the adhesive. The loading is applied by means of metalic grips, adjustable to the dimensional variation of specimens and internally covered with neoprene, which prevents glass from cracking. Each specimen, made up of a glass plate fixed between two timber boards, presents a total contact surface of 40 000 mm^2 (200mm X 100mm X 2). The area of this surface, though not limiting of the results obtained, proved exaggerated as far as adhesives of greater resistance and stiffness are concerned. The end result was glass failure, very common among adhesives of great resistance which, many times, proved stronger than the materials themselves.



***Figs. 6, 7** and **8**: Test scheme; glass behavioural characterisation test*

The types of glass used in tests – 5.5.1 laminated glass and 5 mm tempered glass – were also tested, as shown in figure 8, so as to check their level of resistance and behavioural variability. In the end, high uniformity was revealed. The timber employed was *Pseudotsuga Menziessi*, or Coast Douglas Fir, a type of softwood, properly dried, sawn and polished.

Results

The set of tests was prepared in order that the data obtained could directly be compared. Taking that into account, figure 9 presents a comparison between several load/relative displacement curves, representative of different adhesive performances. The balance between load-bearing capacity and flexibility of each case is, thus, highlighted.

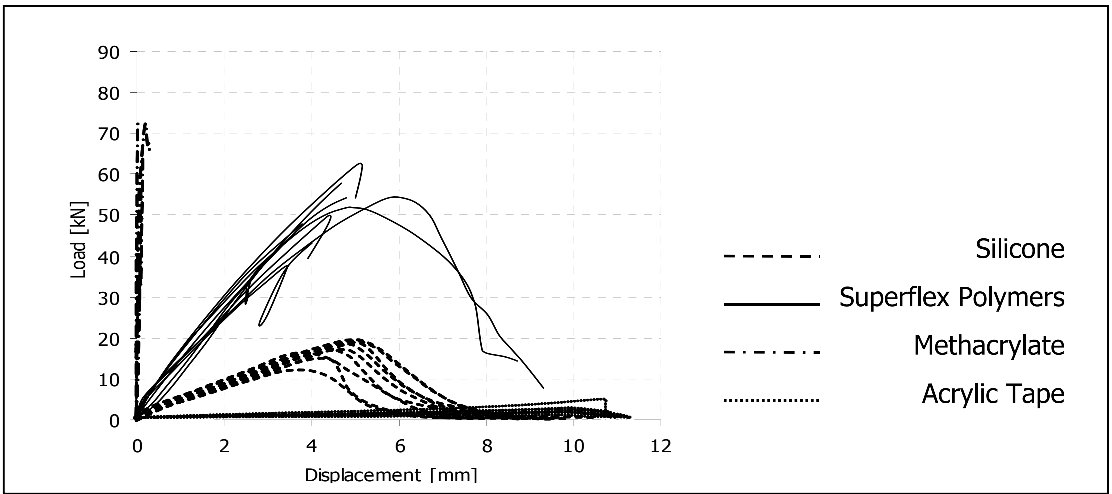


Fig. 9: *Load/relative displacement graph comparing adhesives B, C, D and F*

Table 2: *Summary of results*

Product	*Specimen series*	*Max. Load kN (average)*	*Displacement mm (average)*	*Failure mode*[1]
Sikaflex®265	A04; A05; A06	68,37	5,11	G
	A07; A08; A09	39,45	5,80	G
	A10; A11; A12	15,35	4,26	C; Ag
Sikasil®SG-20	B04; B05; B06	15,86	3,90	Aw; Ag; C
	B07; B08; B09	19,60	4,84	Aw; Ag; C
	B10; B11; B12	20,05	4,91	Aw; Ag; C
Sista Solyplast SP101	C04; C05; C06	60,15	4,96	G
	C07; C08; C09	45,41	3,78	G
	C10; C11; C12	54,62	5,20	Aw; C; G
Sikafast®5211	D04; D05; D06	71,51	0,08	G
	D07; D08; D09	49,76	0,09	G
	D10; D11; D12	62,31	0,15	G
3M™Scotch-Weld™DP-810	E04; E05; E06	87,94	0,05	Ag; C; G
	E07; E08; E09	57,12	0,01	G
	E10; E11; E12	73,44	0,05	Aw; C; G
3M™ VHB™ 4910F	F04; F05; F06	1,95	9,60	Aw; C
	F07; F08; F09	1,30	10,06	Aw; C
	F10; F11; F12	3,28	10,37	Aw; C

[1] *Failure mode*: Aw–Wood Adhesion; Ag–Glass Adhesion; C-Cohesion; G-Glass failure

Table 2 conveys a summary of some of the most important results obtained, that is, the maximum loading average for each series of specimens, as well as the registered timber glass relative displacement average at the maximum loading referred points.

Strength and relative timber glass displacement

Figure 9 illustrate the curves of all tests carried out with silicone, superflex polymers, methacrylate and acrylic tape. These tests unveiled the existence of three different groups: adhesives highly resistant and insufficiently flexible in this context – methacrylate (and also two-component acrylic adhesive); highly flexible adhesives, yet insufficiently resistant to the loading they could be subject to in real situations – silicone and acrylic tape –

and, finally, adhesives that balance both key factors in this research: strength and flexibility – this being the case of superflex polymers. One can clearly observe this situation in figure 10.

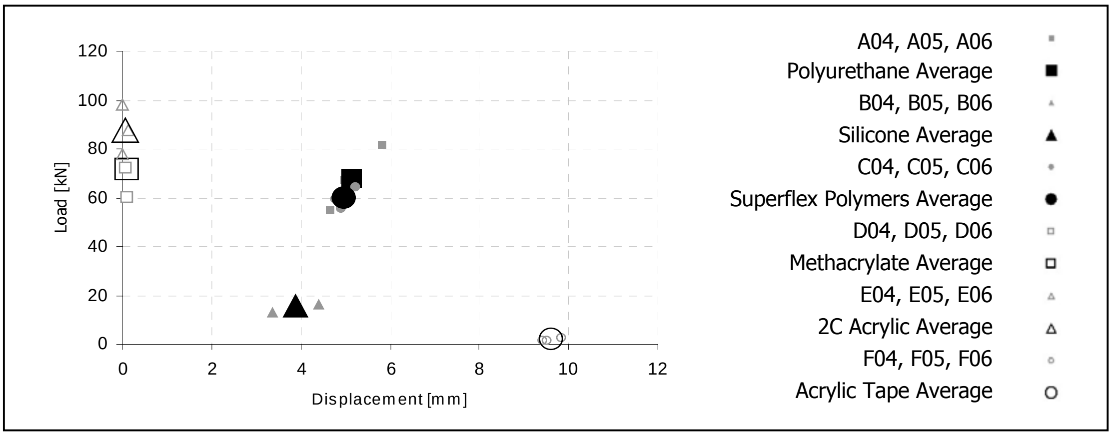


***Fig. 10**: Maximum loading resistance and respecting displacement – all adhesives*

Figures 9 and 10 also demonstrate that one could search for a suitable solution in the latter of the referred groups, the superflex polymer group, to which polyurethane also belongs. Figure 11 compares these two adhesives, and introduces another important aspect – behavioural variability. It is possible to observe that superflex polymer, contrary to polyurethane, presents in all circumstances uniformity convergent with safety criteria, essential in this kind of structure. However, it is pertinent to refer that variation in polyurethane fundamentally results from already mentioned variables applied to the test. This also highlights the influence of such variables, as will be observed further on.

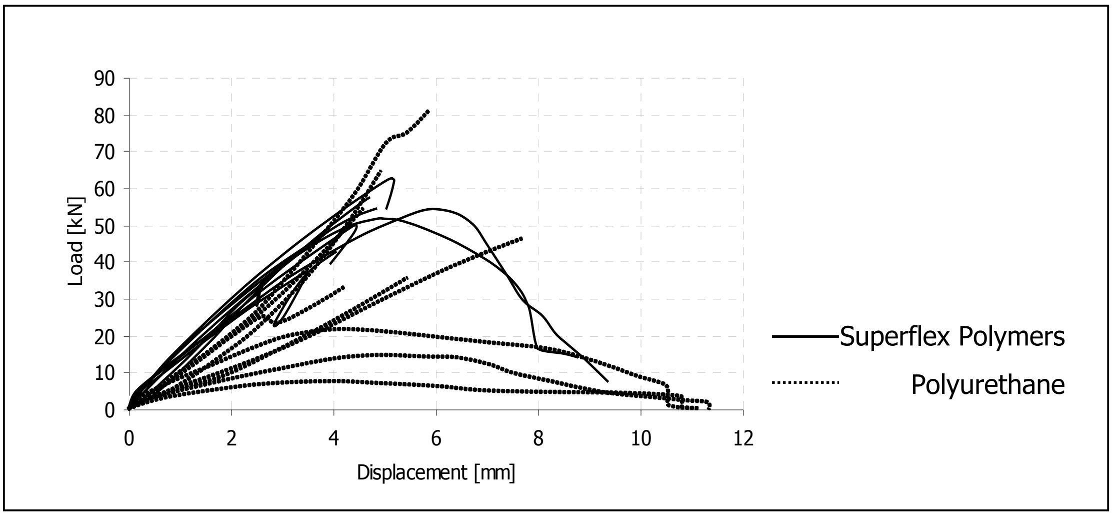


***Fig. 11**: Load/relative displacement graph comparing adhesives A and C*

Nevertheless, one must underline that flexibility of adhesives is relative, subject to arising diverse interpretations if observed in a wide range or according to its contextualization in their specific group. Taking, as an example, the results obtained for methacrylate, which in figure 9 – and comparing with other curves in the same graph –, seems to superimpose on the Y-axis, it is possible to conclude that, in the tested context, this adhesive is extremely rigid and resistant, as demonstrated by the repeated glass failure. Apart from that, and within its specific group, this adhesive can be considered relatively flexible when compared to others. As shown in figure 12, timber glass relative displacement regarding two-component acrylic adhesive is merely of centesimal fractions of mm. However, methacrylate registered twice the relative displacement when compared to the latter. In any case, this relative flexibility seems insufficient to being considered applicable in this context, due to the inherent characteristics of the materials and the intended scope of application.

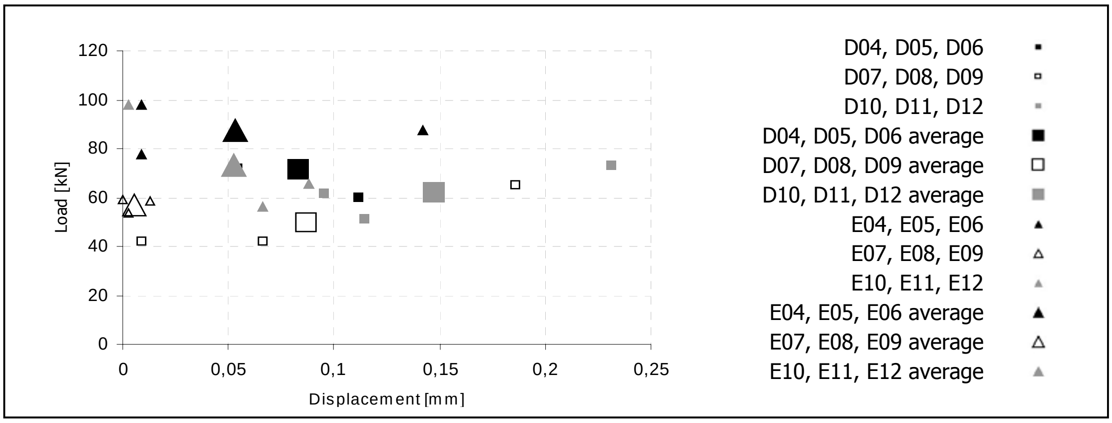
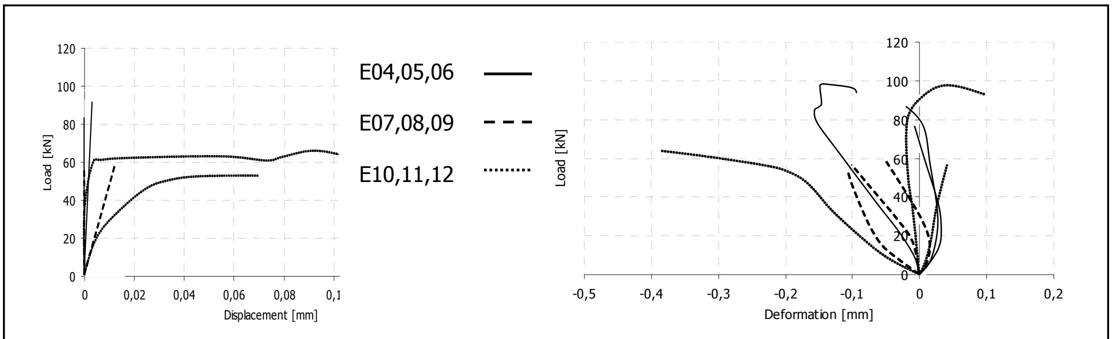


***Fig. 12**: Maximum loading resistance and respecting displacement – methacrylate (D) and 2C acrylic (E) series*

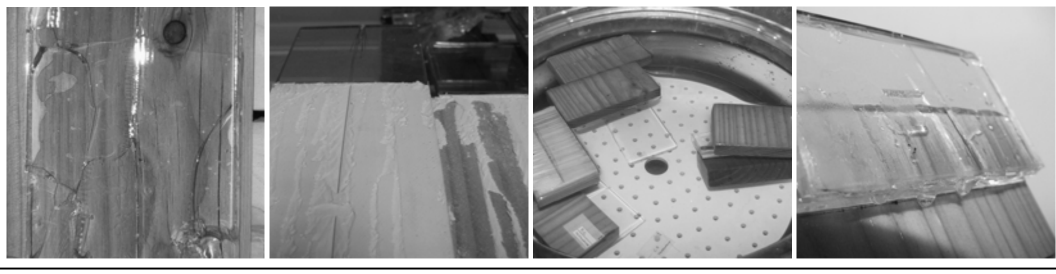
In fact, glass has low tension resistance, whereas timber is ductil, hygroscopic, of variable dimension and influenced by atmospheric agents to which it is exposed. Therefore, if the adhesive is not able to minimally absorb the various resulting stresses, it will not efficiently play the role of interface, as it will not prevent from a direct confrontation of different behaviours of materials.

Strength and timber deformation – consequences

As a logical conclusion, the greater the loading endured by the adhesive, the greater the strength to which both timber and glass will be submited. Conversely, a comparison between the graphs below – figures 13 and 14 –, regarding two-component acrylic adhesive, shows that longitudinal deformation of timber is higher than timber glass displacement. As a consequence of the anisotropic character of timber, this longitudinal shrinkage has repercussions in its tangential expansion. This is precisely the occurrence that must be minimised, as it is responsible for the application of tension stress on the glass surface in contact with timber. Figures 15 and 16 illustrate this failure mode. Even without applying external loading, the same occurrence can take place during normal specimen saturation in water, as shown in figures 17 and 18.



***Figs. 13** and **14**: Load/relative displacement and load/deformation (of timber) graphs regarding acrylic adhesive – comparison of X-axis data*



***Figs. 15, 16, 17** and **18**: Failure modes, due to loading or water saturation*

A highly resistant adhesive – tolerating stressing up to 15/20 MPa – even if considered exceedingly flexible within its group can, under certain circumstances, easily fail due to the particular characteristics of the materials under study. Limiting the tangential dimension of timber in contact with glass would surely represent a valid solution for this situation.

Since behavioural uniformity of the used types of glass was initially verified, it is important to note that the occurrence described in the previous paragraph originates discrepancies within data presented on table 2 concerning the specimens which collapsed through glass. This leads to the conclusion that the type of adhesive directly influences the failure of glass itself.

Failure modes and primers utilisation

In the tests carried out, different failure modes were observed, as characterized in table 2, according to the type of adhesive and variables implemented. In general, it is possible to perceive that, in adhesives of higher resistance, glass always ended up collapsing. However, except for methacrylate, situations occur in which, besides glass, collapse takes place simultaneously with adhesion break – either with glass or timber – and/or cohesion break of the ahdesive itself.

According to figures 19 to 22, failure mode patterns can be observed in two of the most resistant adhesives – superflex polymers and two-component acrylic. In the first case, only noticed in series C10, 11 and 12, which shows that the best results regarding this adhesive are obtained without any primer use, collapse involved glass break, glass adhesion break and cohesion break. Hence, it was possible to observe a sliding pattern in the cohesion break of this adhesive – figure 20. Similarly, two-component acrylic adhesive also stood heavier loading without surface treatment. However, this adhesive presents three significant differences: adhesive break occurred in series E10, 11 12, but also in E04, 05, 06, regarless of the fact that in the latter it took place under strength rates superior than those in series E10, 11, 12 and also series C04, 05, 06. Series E10, 11, 12, in which primers were applied, collapsed through timber adhesion – figure 21 – whereas series E04, 05, 06, where primers were not used, collapsed through glass adhesion – figure 22. Ultimately, the post-collapse surface of this adhesive did not indicate any sliding. Instead, it was possible to observe an apparent vitrification.

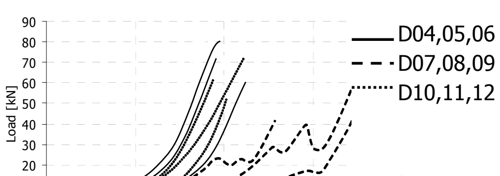
***Figs. 19, 20, 21** & **22**: Failure modes – superflex polymers and 2C acrylic*

Based on these examples, one may then conclude that surface treatment has a decisive influence in the adhesive bonding failure. Still, depending on the adhesive observed, that influence may be positive or negative, thus improving or worsening its performance. The use of primers is not always advisable. For each situation there is an ideal solution which, should it imply total absence of primer treatment, can surely ba an important advantage – of time and cost – in the employment of adhesive.

Glass type influence

Tempered glass, though more resistant to superficial stresses than laminated glass, presents two considerable and decisive disadvantages in relation to the latter: brittle properties and an irregular surface, as shown in figure 24. These characteristics prevent tempered glass from being considered an adequate solution for the intended situation. The results obtained demonstrate that, from certain loads onwards, there are oscillations in the loading/displacement curves of the loading unit – figure 23 –, which do not occur with laminated glass. Moreover, the superior resistance it holds – despite the differences in thickness used in both types of glass – becomes irrelevant when the occurrence illustrated in figures 15 to 18 takes place. Due to the characteristics of tempered glass, the occurrence mentioned results in its immediate collapse, even before the applied loading can affect the adhesive bonding. As shown in table 2, except for polyurethane, all series corresponding to tempered glass with primer treatment – 07, 08, 09 – always resisted less than series with laminated glass – 10, 11, 12.

In the case of bi-adhesive acrylic tape, the difference is of less than half the effectiveness, due to the incapaciy for compensating superficial imprecisions, as observed in figure 25. This irregularity in the glass surface tends to affect, more than any other, fluid adhesives, as one may apprehend from the difference obtained in the case of two-component acrylic adhesive, the greatest registered (57,12 kN to 73,44 kN), as shown in table 2.



Figs. 23, 24 *and* ***25****: Load/Loading unit displacement graph – methacrylate; tempered glass; bonding irregularities – bi-adhesive acrylic tape with tempered glass*

Both analysed variables – primers utilization and glass type – directly influence the performance of adhesives and lead to conclusions regarding the behavioural uniformity each presents, according to the mentioned variables – figure 26. Behavioural uniformity is an important safety factor, which must be guaranteed.

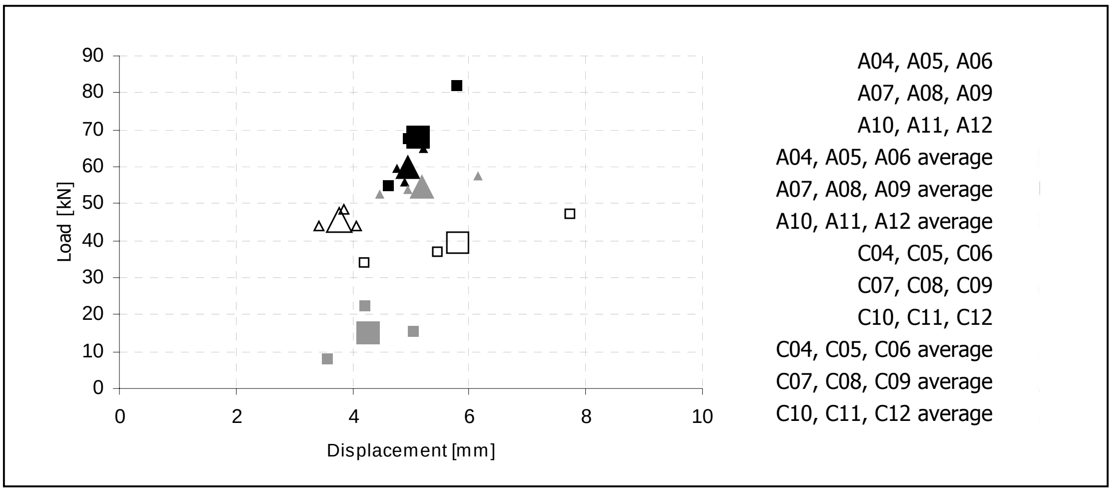


Fig. 26*: Maximum loading resistance and respecting displacement – polyurethane (A) and superflex polymers (C) series*

Safety precautions

Safety is a crucial and indispensable aspect. It implies deformability or ductility criteria, instead of brittle characteristics, unable to absorb tensions, usually much resistant but easy to suddenly and unexpectedly collapse. The behaviour of the structural element in periods ranging from the first cracking to the maximum loading resistance, and from this to collapse, is of essential importance in this context, where the possible maximum margin of time should be kept and any fragile tendency avoided. This applies to adhesives and the materials themselves. In this case, the elementary choice of laminated glass - usually less resistant than tempered glass - results from safety concerns.

Conclusions

The bonding solution pointed out in the present research can be a practical system regarding the structural use of several timber glass composite elements. Depending on the geometry of the composite cross section, the specific mechanical characteristics of its components and the loading involved, it may be necessary to apply a more rigid or ductile adhesive.

This paper summarises the results of 54 laboratorial tests on shear stress, involving different types of adhesives. The results obtained, concerning strength and flexibility, demonstrate a wide range of mechanical behaviours - from extremely rigid to significantly ductile – and support the feasibility of this solution to the applications envisaged.

This solution, however, must undergo other tests in order to be accepted as a structural constructive solution: temperature and relative humidity variation, UV radiation, ageing, aesthetics and applicability are considering aspects. The work hereby presented is a stage in a long path towards the technical and scientific validation that is intended to be achieved. Its main purpose is the practical, safe and generalised implementation of an innovative, daring and promissing constructive system.

Acknowledgements
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Evaluation of the integrity of glued laminated timber structures in service

Cruz, H.[4], **Gaspar, F. M.**[5], **Rodrigues, J.**[6]

Abstract: *This paper stresses the importance of monitoring and following up glued laminated timber structures. It reports the experience of the authors concerning the structure of Atlantic Pavilion (Lisbon), describing the principal aspects that are taken into account. It also refers some non-destructive methods that may be used for the assessment of glued laminated timber structures in service. Finally, it refers an ongoing PhD research program, which is expected to provide some useful information on this subject and presents some early results on chemical analysis with this purpose.*

1. Introduction

Monitoring and follow up of glued laminated timber structures in service is essential to control their durability and to prevent premature degradation and failure. This is particularly needed for structures subjected to weathering, but also required for indoor elements. Nevertheless, a huge amount of glued laminated timber structures are not regularly assessed. In many cases, the building owners are not aware of the need for this, namely when there is little experience on building with timber and particularly with glued laminated timber. They do not realise how simple periodical visual inspection may help detecting possible problems at early stages of damage process, enabling corrective action to take place in time.

Despite the increasing interest on this subject from the scientific community, due to some recent disasters reported in the literature, we still face the lack of practical and reliable methods to evaluate the actual strength of bond lines on service. This is of special concern because bond lines ageing caused by weathering or by indoor moisture cycles may not be detected with simple visual inspection. To provide this evaluation it is necessary to understand the bond line degradation mechanisms, why it occurs, how to detect it and how to quantify the associated loss of strength. These are not yet totally identified and need to be studied.

2. Ageing and degradation of glued laminated timber structures

Durability of glued laminated timber structures implies sufficient durability of timber and glued joints. Deterioration of the wood itself may be due to biological attack (insects and fungy), water induced stresses associated with drying cycles, and UV radiation (only surface degradation, in this case). Deterioration of glued joints may result from excessive loading imposed to the structural member, from internal stresses due to shrinking and swelling of the wood (associated with water uptake or drying), from environmental temperature or from chemical reaction of the cured adhesive with water and/or other products. It can cause strength loss and delamination (Figure 1).

A wood joint properly bonded with a phenolic adhesive is said to be stronger and more durable than wood (Gillespie, 1980). However, during ageing, wood and adhesive will not maintain their original properties and some degree of degradation will occur. When joints separate an appreciable amount at the surface, special worries about the strength of the structure can rise and the quality of glue bond in the other parts of the glued laminated timber is also questioned.

Delamination is the complete separation of the glue lines and it is the most visible symptom of problems. Curved glued laminated timber structures are one example where delamination due to external loads can occur. Bending moments in curved members cause radial stresses perpendicular to the grain (Ehlbeck and Kürth, 1995). One example of this is the case study reported by Giustina (1985) where, due to deficient detailing, serious delamination occurred on the structure (Figure 1a).

Dimensional changes of wood due to weathering can cause cross-sectional tension stress levels perpendicular to the grain up to 0.3 MPa which can be in the range of about two thirds of the characteristic strength value

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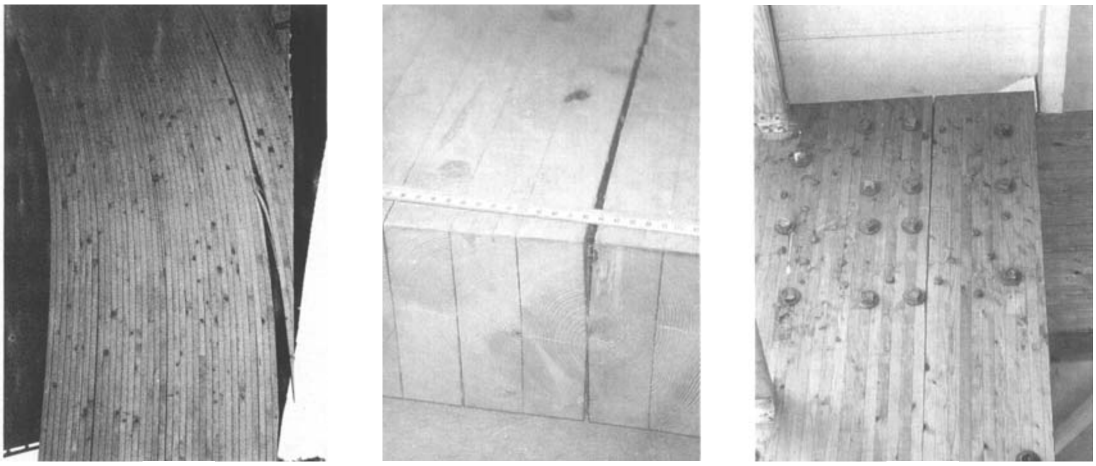
(Aicher and Dill-Langer, 1997). Maximal stresses are obtained when a long dry period is followed by a long humid period (Ranta-Maunus, 1998).

According to Gramatikov (1996) the occurrence of tensile stresses perpendicular to the grain, due to shrinkage of wood and/or external loads, caused a large number of reported collapses of timber structures. Many cases of severe glue line delamination in structures have also been reported (Biger, 1995; McIntosh, 1996; Gramatikov, 1996; Zakić, 1998).



a) b) c)

Fig. 1. *a) Delamination in a three-pin portal frame with curved haunches (Giustina, 1985); b) Crack due defective gluing (Biger, 1995); c) Cracked frame corner (Biger, 1995).*

Simply changing the growth-ring orientation in adjoining laminate can alter the possibility of fracture in the vicinity of the joint caused by a change in moisture content of the laminated member (Laufenberg, 1982; Nestic and Milner, 1991). Additionally, the closer the pith is to a bond line, the greater the stresses in the bond line induced by a moisture content change in the wood (Nestic and Milner, 1991; Ranta-Maunus, 1998).

Water, temperature and UV radiation may induce degradation of adhesives. However, for wood adhesives, the most important are changes in temperature and moisture (Frihart, 2005; Gillespie, 1980). Wood joints are especially sensitive to moisture effects as a result of the permeability of wood, which allows access of moisture to both the interior of the wood member and the adhesive layer.

Once moisture is present, it can attack the bond by (Davis, 2003):

- Altering the properties of the adhesive in a reversible manner, e.g., plasticisation;
- Swelling the adhesive and introducing concomitant stresses;
- Disrupting secondary bonds across the adherend/adhesive interface;
- Altering the properties of the adhesive in an irreversible manner, e.g., hydrolysis, cracking, or crazing.

Plasticisation and swelling are both reversible processes. Water depresses the glass transition temperature of adhesives and lowers modulus and strength (Comyn, 1983). On the other hand, Irle and Bolton (1988) showed that the superior durability of wood-based panels bonded with an alkaline PF adhesive compared to panels bonded with a UF adhesive was due to the ability of the phenolic adhesive to absorb and to be plasticized by water. In the plasticized state, the phenolic adhesive is able to reduce stress concentrations that otherwise fracture the wood or the adhesive in urea-bonded panels (River, 2003).

Urea-formaldehyde adhesives are known to be sensitive to high moisture levels. These depolymerise under humid environments, as shown by increased release of formaldehyde (Dunky, 2003), especially if they are exposed to humid and acid media where the consecutive processes of post-cure and hydrolytic degradation take place (Dutkiewicz, 1983). On the other hand, wood adhesives, like phenol-formaldehyde and resorcinol-formaldehyde, do not change drastically in their adhesion to wood at higher moisture levels.

3. Monitoring of structures – the example of the Atlantic Pavilion

Glued laminated timber structures are relatively new in Portugal. It can be said that they were first introduced in this country with the Atlantic Pavilion built for the World Exhibition EXPO'98. The timber structure is composed of 17 truss arches and glulam members are made of spruce bonded with type I adhesive and received a surface preservative treatment suitable for hazard class 2.

This was the first large structure (150m x 120m in plan, 47m high above the arena) designed according to Eurocode 5 (Figure 2). Because of that, LNEC was involved in the specification, fabrication of glued laminated timber members and construction of the timber structure. After 2001 LNEC has also set a follow up routine, which involves:

- Monitoring of temperature and relative humidity of inside environment;
- Monitoring of moisture content of timber;
- Measurement of vertical and horizontal displacements;
- Periodical inspections of the structure.

Fig. 2. *Atlantic Pavilion built for the World Exhibition EXPO'98*

Measurement of relative humidity (RH) and temperature (T) is done because RH and T influence moisture content of timber and also have implications to the risk of fastener corrosion. Besides, preservative treatment was specified as a function of the expected humidity conditions. The adhesive was specified as a function of T and HR. Continuous measurement of the air temperature and relative humidity is made with equipment installed at various points of the structure, covering different highs, orientation and solar incidence (Figure 3. a).

Measurement of moisture content of timber is also made in several locations of the structure and several depths from the timber surface (Figures 3.a and 3.f). In this structure, moisture meter needles were inserted and left in the timber. Periodic measurements are made by attaching a wood moisture meter to each pair of needles. Moisture content is of utmost importance since it determines timber strength and durability. It also determines dimensional variations of timber members and can help to explain the global movement of the structure in the long run.

Vertical and horizontal displacements give important information, as increasing movements may indicate an instable situation and the imminence of collapse. In the case of the Atlantic Pavilion, measurements are made with the help of topographic equipment (Figure 3.e), twice a year (summer and winter).

Visual inspections are however the most useful tool one can use to follow a timber structure and can't be replaced by any other method (Figure 3.b). They allow early detection of water infiltration, biological attack and other problems, enabling prompt intervention to take place. In the case of Atlantic Pavilion a general summer inspection is made every year to assess fissures, delamination (Figure 3.d), fasteners looseness (Figure 3.c), joints opening and other possible problems. Resident responsible persons also carry out frequent inspections along the rainy season in order to detect and promptly solve possible water intake situations.

So far, this structure does not present significant problems. In one of the arches one glue line opening was detected and subsequently consolidated by bonding on site with epoxy adhesives. This is believed, however, to be a fabrication defect associated with glue starvation and not an evolving situation.

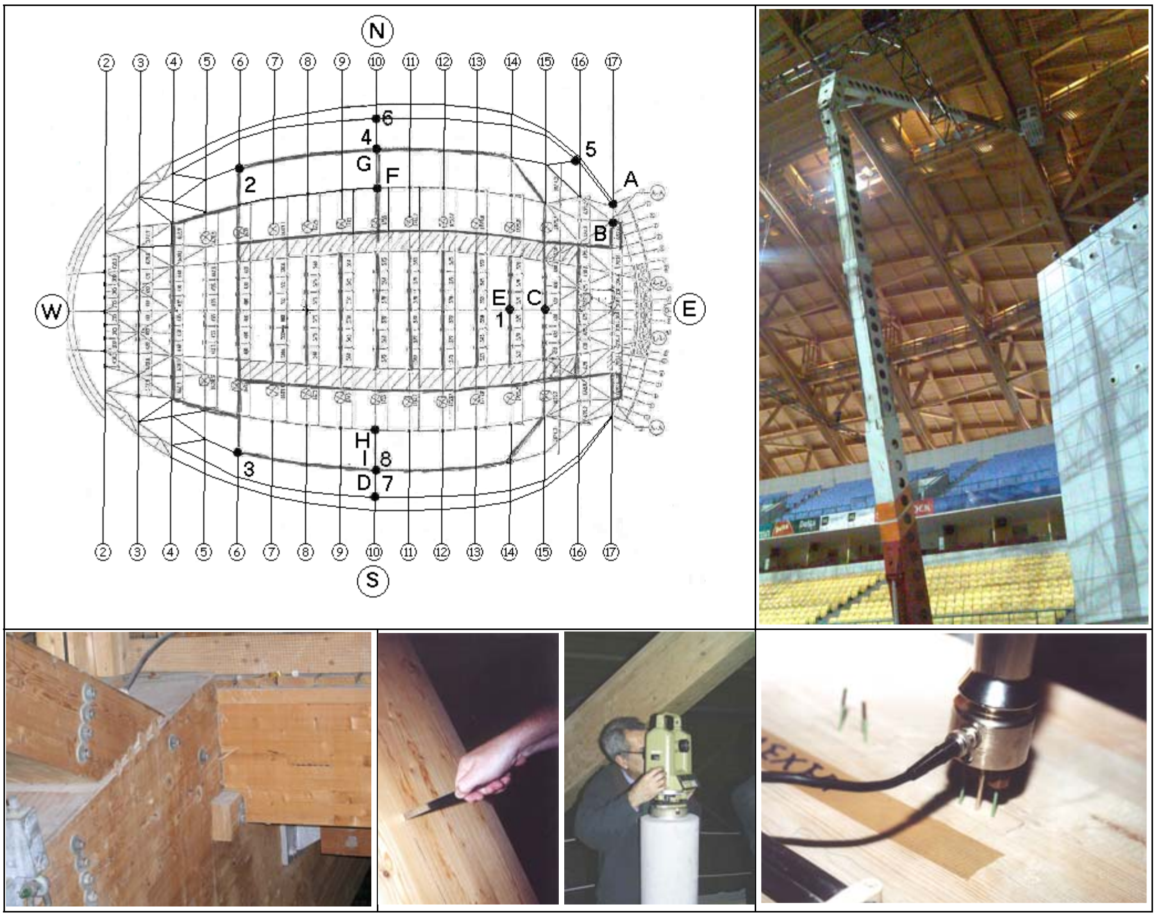


Fig. 3. *a) Plant indicating points of measurement of ambient conditions (1 to 8) and moisture content (A to I); b) Visual inspection of arches; c) joints inspection; d) delamination measurement; e) measurement of vertical and horizontal displacements; f) moisture content measurement*

4. Non-destructive and semi-destructive evaluation methods

Non-destructive techniques are generally based on correlations between non-destructive measurements and strength. However, for strength prediction in the case of timber structures, the major drawback of the non-destructive techniques is the relatively poor correlation between the measured non-destructive quantity and material strength (Kasal and Anthony, 2004). Visual inspection and ultrasonic techniques are examples of non-destructive evaluation methods.

Complementing non-destructive techniques with semi-destructive methods can enhance the reliability of material strength and stiffness evaluation. In semi-destructive techniques, a small specimen is removed form a member and tested destructively. The size of the specimen relative to the size of the member must be sufficiently small that reduction in cross-section is negligible. The so-called core-drilling technique is one possible semi-destructive test.

4.1. Ultrasonic methods

Ultrasonic methods have been applied to non-destructive evaluation of glued laminated timber.

Both acoustic emission (AE) and acousto-ultrasonic (AU) techniques have been applied for testing defective or weak glued lines. The ultrasound transmission technique was, for instance, used by Dill-Langer and Aicher (2005) to show the feasibility of detection glue-line defects. Acousto-ultrasonic technique differs from conventional ultrasonic techniques since more subtle flaws, such as poor quality adhesive bonding, can be detected (Beall, 2002). However, additional basic research is needed to clarify the effects of wood characteristics on attenuation of ultrasonic waves and on AU/AE parameters (Kawamoto and Williams, 2002).

The acousto-ultrasonic technique was also used to assess the performance of finger joints. Correlations can be established between the acousto-ultrasonic parameters and the tensile strength or the reduced finger-joint strength over the time (Bucur, 2006).

4.2. Core-drilling technique

Standard EN 392 (1995) specifies two specimen types for evaluation of glue lines shear strength: the block-shear test specimen and the cylindrical test specimen obtained by drilling along the glue line. Cylindrical drilled specimen is then notched and test results are comparable to the ones obtained with the block-shear test.

Core-drilling has been used to establish physical properties of wood and other materials for some time and cylindrical specimens of various types have been reported (Selbo, 1962; Outinen and Koponen, 2001; Lear, 2005). Selbo (1962) used a cylindrical specimen of 25 mm in diameter and Outinen and Koponen (2001) made some experiments with a so-called drilled shear specimen of 32 mm in diameter. The results demonstrated that this method is sufficiently promising to test glued laminated timber members in service.

Some requirements have to be fulfilled to the success of this technique. Owing to the fibrous character of the wood material, a good core cutter is needed to not leave marks or torn grain on the specimen, to allow extraction of high quality cores, needed for subsequent mechanical testing. The shearing tool must have a locking device with exactly the same diameter of the core and it will be desirable that shear toll allows aligning the position of the glue line with the shear plane. When testing, additional care should be taken to certify that shearing force is parallel with the grain direction of the adjoining laminations and the moisture content of specimens shall be appropriate to assure the comparability of values obtained for the same diameter of the shearing tool.

5. Chemical analysis methods – trial evaluation by NIR

The potential of NIR analysis to detect chemical anomalies or modification of glue lines due to ageing was investigated in the scope of strength evaluation of structures in service. Should this give relevant information, a reasonable amount of small samples of the glue lines could be collected from structures causing minor disturbance.

5.1. On-going research – General frame

A PhD research program is undergoing at LNEC aimed to develop methods for evaluating the performance of glued laminated timber in service. It focus on glued laminated beams made of Spruce (*Picea abies*) and type I adhesive, as the kind of glulam mostly used in Portugal, and glulam made of deep impregnated preservative treated maritime pine (*Pinus pinaster*, Ait.) and type I adhesive, as a suitable material to apply in hazard classes 3 and 4 defined in EN 335-2.

Prior to bonding, maritime pine was deep treated with Tanalith E 3492 (one possible alternative to CCA products) to the target retentions of 7.1 kg/m^3 and 16.4 kg/m^3, respectively appropriate to hazard classes 3 (weather exposure without ground contact) and 4 (ground contact), followed by kiln drying.

Glued laminated beams of untreated (Z), lightly treated (L) and highly treated (H) maritime pine were manufactured at LNEC in laboratory environment. PRF adhesive by Dynea ASA was used. Four curing temperatures were used (20ºC, 30ºC, 40ºC and 45ºC). After 24 hours, clamping was released and beams returned to standard conditions (20$\pm$2ºC; 65$\pm$5%).

The influence of preservative treatment and cure temperature on the performance of freshly glued laminated beams was assessed by shear strength and delamination tests. Shear tests were also carried out on the material that had been subjected to delamination test cycles.

A number of destructive, semi-destructive and non-destructive tests will be carried out after different levels of natural and artificial ageing of these beams to monitor possible modification of shear strength of the glue lines.

The possibility of using chemical analysis techniques to assess the state of glue lines in service is also investigated as these only require a small amount of material to be sampled from the structures. Their use was believed to provide also useful information to explain different behaviour of glue lines on different timber species or different treatment levels.

5.2. Near Infrared Spectroscopy (NIR) analysis and results

The glue lines analysed covered all the combinations of treatment level (untreated maritime pine (Z), pine with the lower level of preservative treatment (L), pine with the higher level of preservative treatment (H)) and cure temperature (20ºC, 30ºC, 45ºC). NIR analysis was done on specimens that suffered delamination test and were subsequently shear tested to failure.

NIR spectra of the glued joints in the shear plan were collected in the wavenumber range from 12,000 to 5,100 cm^{-1} with a Bruker Vector 22 N/I spectrometer in diffuse reflectance mode using a fibre optic sampling probe. Each spectrum was obtained with 100 scans at a spectral resolution of 8 cm^{-1}.

The NIR results (Figure 4) show clear chemical differences in the cured adhesive with different levels of treatment and also with different cure temperatures.

These differences may be correlated to delamination test results, which show a clear increase of delamination with increasing preservative retention and with decreasing cure temperature.

These preliminary results, although limited, seem to indicate that NIR analysis may be a promising tool to assess the quality of glue lines in service. Further tests are under way to fully understand the potential of this method.

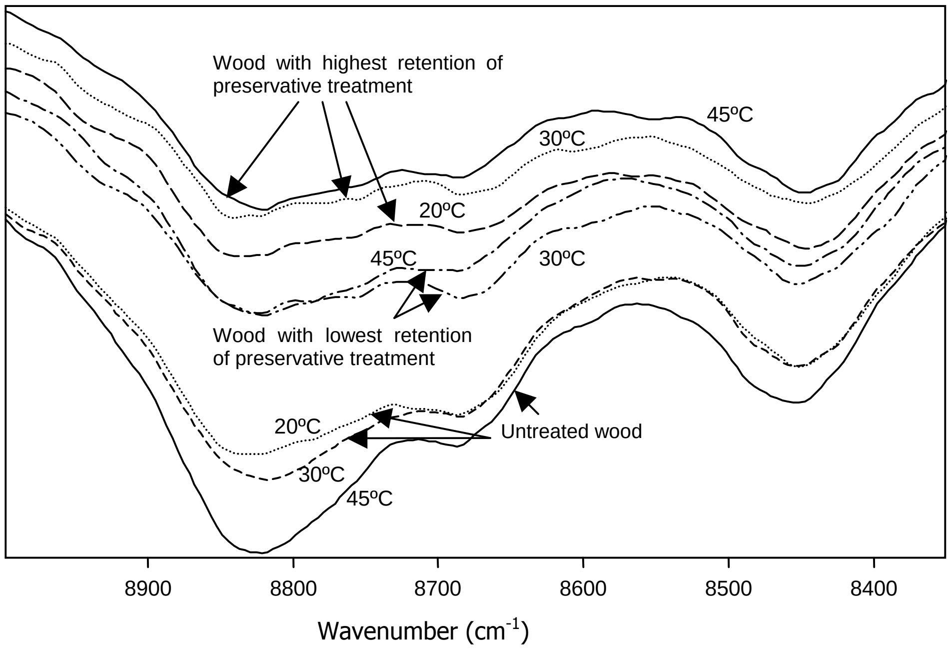


Fig. 4. *Spectral analysis of glue lines by NIR after ageing*

6. Conclusions

Monitoring and follow up of glued laminated timber structures in service is essential to control their durability and to prevent premature degradation and failure. This is particularly needed for structures subjected to weathering, but also required for indoor elements. Nevertheless, a huge amount of glued laminated timber structures are not regularly assessed. In many cases, the building owners are not aware of the need for this, namely when there is little experience on building with timber and particularly with glued laminated timber.

Durability of glued laminated timber structures implies sufficient durability of timber and glued joints. Access of water to the glue lines has been reported to be the main cause of degradation of the glued joints, as it is also the major cause of degradation of wood.

Periodical visual inspection of structures is essential as it may help detecting possible problems at early stages of damage process, enabling corrective action to take place in time. Monitoring and measurement of displacements, environmental conditions and moisture content of timber may also give relevant information in order to identify rising stability problems or hazard situations.

Despite the valuable information obtained with visual inspection methods and monitoring, there is a need for complementary practical and reliable non-destructive and semi-destructive techniques to evaluate the actual strength of bond lines on service. This is of special concern because bond lines ageing caused by weathering or by indoor moisture cycles may not be detected with simple visual inspection only, especially in early stages of deterioration.

Acousto-ultrasonic method shows to be promising to evaluate glue lines and it has already proven its efficacy for the evaluation of finger joint strength. Its use for the assessment of structures in service may however present practical difficulties. Although being a semi-destructive approach, the core-drilling technique can also be used to evaluate glue lines strength with a major level of certainty.

To provide new methods it is necessary to understand the bond line formation and degradation mechanisms and to know the associated loss of strength. These are not yet totally identified and need to be studied.

Early results of Near Infrared Spectroscopy analysis of cured adhesives, carried out by the authors, suggest that NIR analysis may be a promising tool to detect adhesive problems and modifications on the glue lines due to ageing. Further tests are under way to fully understand the potential of this method.

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Detailing of timber structures in seismic areas

Ceccotti, A.[7], **Touliatos, P.**[8]

Abstract: *Structural detailing is a very important issue for earthquake resistant buildings. This is particularly true in the case of timber structures where the conception of the structural behavior as whole and of single joints plays a fundamental role. In this lecture particular attention is therefore made to the structural form and to ductility and dissipation properties of joints that can be reached only if details are properly conceived. The intention is not to present an exhaustive list of possible cases but to give the key for a better understanding of real needs. Examples are taken also from ancient constructions in order to try to educate modern designers to learn from the past experience about earthquake - safe constructions – when calculation codes where not yet available.*

Introduction

In the design process of constructions in seismic zones great attention is always paid to the calculation of load – bearing elements: Codes dedicate a large space to it. Nevertheless designers must be aware that the diligent application of the calculus rules will not be enough for the success of the construction. The reality is that constructional arrangements and details are also important. In other words calculus per se is not sufficient without good detailing. Eventually the opposite may be true: i.e. for constructions of small dimensions and regularly arranged in plane and in height, with some minimum dimensioning of element sections, few details about the connections had been in great sufficient – the experience showed – to resist earthquakes, and calcula had never been performed at all… Therefore in many seismic codes around the world a list of minimum dimensions and requirements with structural examples are given for small structures which do not need to be calculated.

In the present version of the European Seismic Design Code (EC 8) a similar possibility has been not considered because of the diversity of construction techniques through different Countries of the European Union. Anyway in EC 8 in addition to calculus criteria a lot of detailing rules are given in order to:

- to assure the compliance with some very important hypothesis at the base of calculation methods and to give at the same time advices against the most dangerous mistakes.
- to assure the attainment of the foreseen ductility level and, consequently, of the relevant "behavior factor" used for the evaluation of the inertia forces. For example in the part 1.3, ch. 5 – "Timber Structures" – of Eurocode 8 some detailing rules is given in order to assure a good ductility behavior of mechanical joints.

Actually, in order to not confine the progress of the building activity with wood, EC 8 is a performance-based Code and in principle any joint may be acceptable if it fulfills some ductility performance test requirements (see Lecture C 17), but in most current cases few detailing rules are sufficient to avoid tests.

Nevertheless the designer must also be aware that also if the part of the Code dedicated to detailing would be full of structural detailing examples he could not be completely satisfied by the a critic application of these rules. The most important thing is to understand the real meaning of such detailing rules and behave consequently. And let us say that, hopefully, a Code could not ever contain and solve respectively all kinds of details and problems. Consequently in this lecture the leading idea is to focus the main important points to be controlled for the best performance of a generic timber structure in a seismic zone and leave to the designer the joy to solve his particular case.

Structural continuity

Basically, the earthquake action may be considered like a horizontal action that, differently from the vertical actions, involves the entire structure and not only the small part of it just underneath the load (Figure 1). Obviously one can say the same thing for wind, but if we exclude hurricanes, that are fortunately no present in Europe, the action of the earthquake, according to the modern point of view of the Eurocodes, can be more important in respect to the wind especially for heavy structures (Ceccotti & Larsen, 1989).

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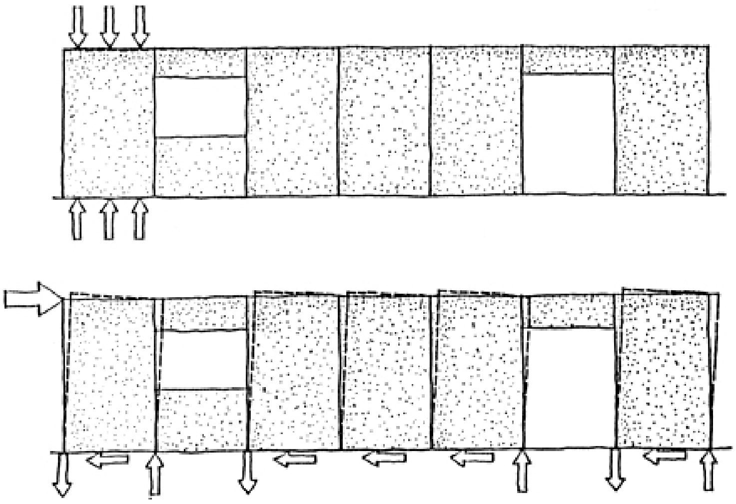

Fig. 1: *Different structural involvement by vertical and horizontal loading*

This means that particularly important is the continuity of the liaison between different members at all positions and the bilateralism of these connections. All the components of the shear walls and diaphragm systems (see Lecture E 10) must be adequately fastened together so that the structure acts as an effective unit.

In figure 2, for example, the main points where such features are mostly necessary are focused, and possible solution for realize continuity is presented. At the floor level a presence of a continuous girder all around the floor should be considered in order to collect the tension forces that will arise when the floor is loaded laterally and it is considered rigid in plane acting as a diaphragm (Figure 2b) and the necessary continuity at the corners will be assured through the diaphragm paneling putting edge nails at closer spacing (Figure 2a).

Also in the height the load bearing vertical elements should be continuously connected in order to guarantee the transmission of the vertical efforts (Figure 2c).

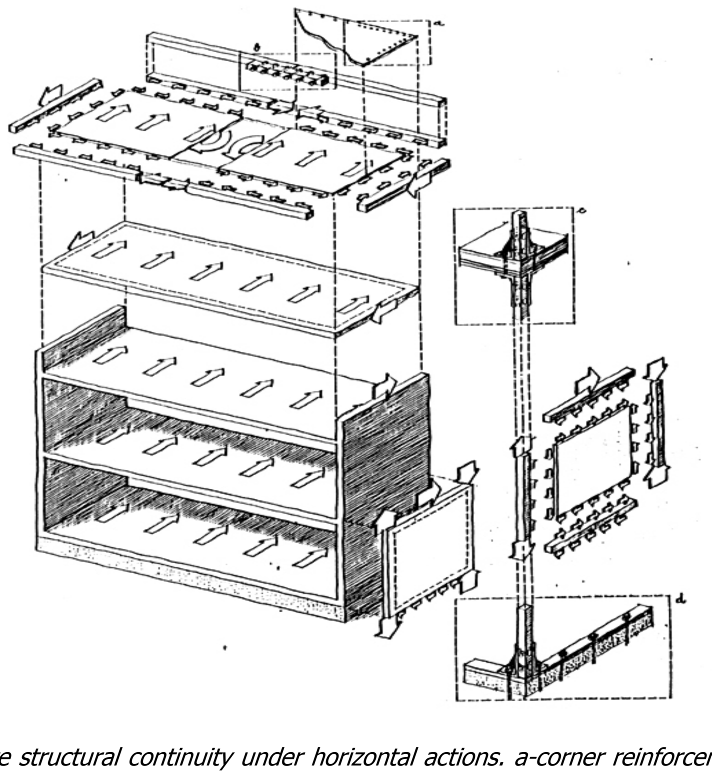

Fig. 2: *Details assure structural continuity under horizontal actions. a-corner reinforcement; b-tension girder continuity; c-continuity of tension studs; d-prevention of uplifting from foundation and sliding out of foundation.*

In some cases at floor level the liaison between two corresponding – upper and down – shear walls is merely obtained by panel sheathing nailing through the header beam of the floor, that I not suitable when tension efforts are important as in the case of an earthquake.

Particularly attention should be paid to connections between the timber structure and foundations both in order to prevent uplift and sliding (Figure 2d). Openings weaken both diaphragms and shear walls, therefore opening must be reinforced around them in order to try to maintain as much as possible the same in plain rigidity. In figure 3 an example about a real application is given.

Great attention shall be paid to the tension perpendicular to grain. For that reason EC 8 prescribes, with reference to Figure 4d, that be has to be more than 2/3h, where h is the depth of the member, so that splitting due to tension perpendicular to the grain is less possible; and when using a strap this should surround the timber piece (Figure 4e).

Connections shall be obviously able to work in both directions because the action of the earthquake is b-directional: for that reason simply contact joints without any possibility to react against opposite actions and prone to take off support are not suitable (In figure 5 some possible provisions are given).

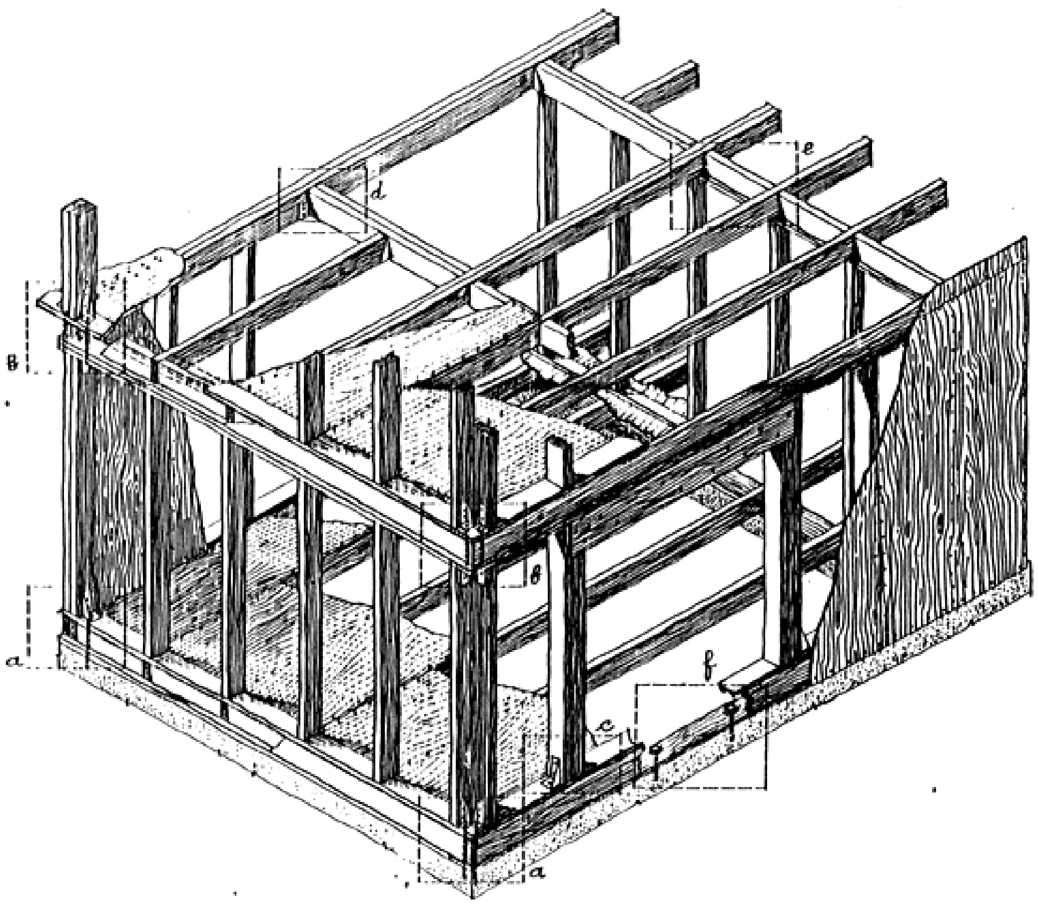

Fig. 3: *Detailing examples for a timber framed house, a-prevention of uplifting from foundations; b-continuity of tension members; c-stiffening of openings in shear walls by framing with additional studs, lintels and corner hangers; d-stiffening of openings in diaphragms by framing with doubling of trimmer and header joists; e-stiffening of diaphragm floors (blocking); f-prevention of sliding out of foundations.*

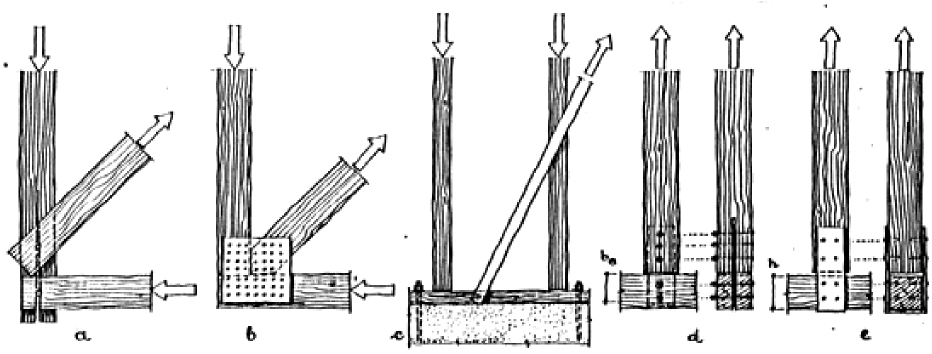

Fig. 4: *Dealing about tension perpendicular to grain. a,c: poor; b,d and e: good.*

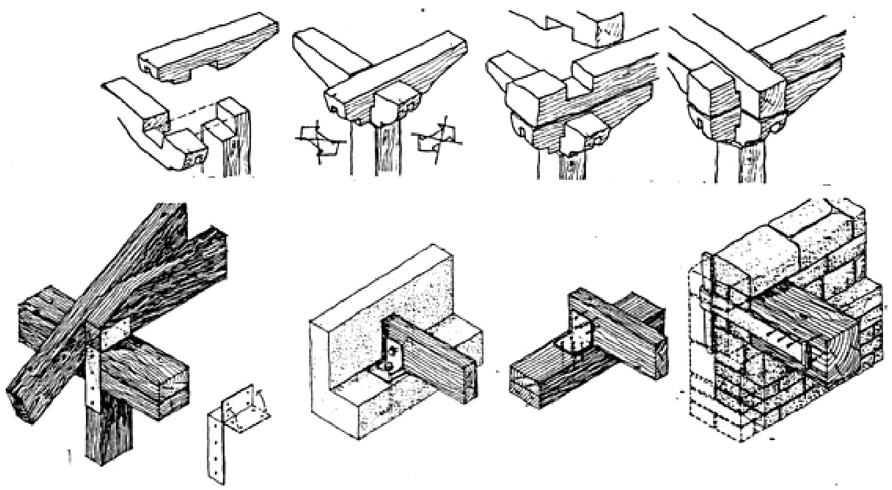

Fig. 5: *Possible provisions against the loosening of support in old and modern constructions.*

Foundations shall be tied each other as much is possible in order to minimize the effects of differential ground movements. In particular foundations of houses should be interconnected so much to act as a whole especially if the nature of the ground is soft in order to realize a rigid foundation functioning as a "raft" when soil is moving.

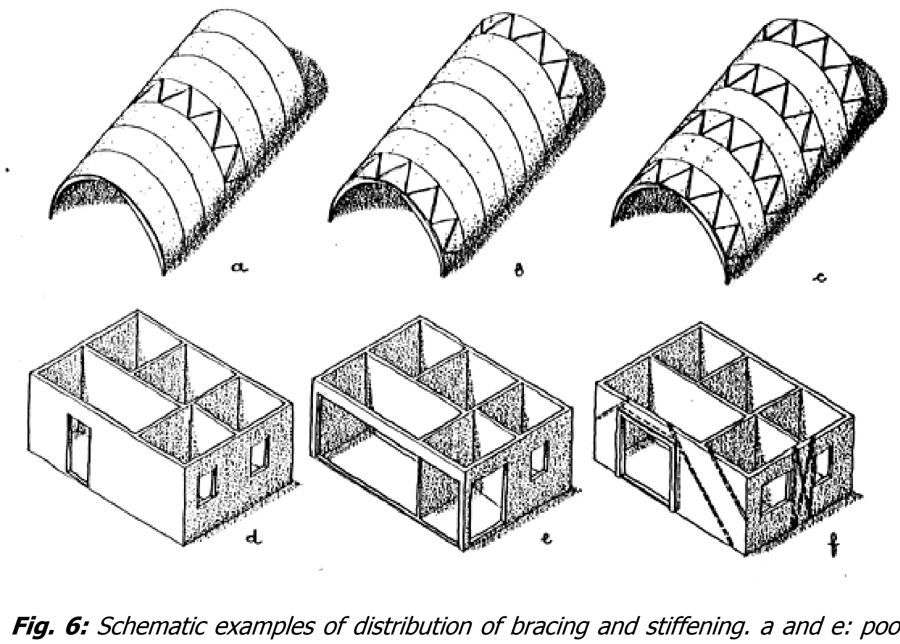

Fig. 6: *Schematic examples of distribution of bracing and stiffening. a and e: poor; b and f: fair; c and d: best.*

Building regularity

Regularity in plan and height is very important in order to assure a good behavior under earthquakes. The reason is that torsion effects induced by irregularity are not easily dominated by the calculus, especially the static one. On the other hand, unfortunately, the tentative to control torsion effect by sophisticated dynamic calculation is often a no more than an academic exercise: it should be much better that one would try to realize the more possible regular building, also without real axes of symmetry but at least with seismic resistant parts regularly distributed and better, also homogenously distributed (Figure 6). In that way torsion effect are quite limited, strength properties are more uniformly distributed and calculus results more reliable. In presence of large unbalanced openings in order to reduce the tendency of the building to twist under lateral forces, the best solution is to try to approximate the rigidity afforded by the shear wall at the opposite and by means of additional bracing, or by increasing panel thickness, with edge nailing at closer spacing, or by affixing panels to both sides of the framing around the openings. Internal partitions have usually a positive effect as they contribute to increase the dissipation of energy by hysteresis and friction.

Structural compatibility

The problem arise in the case of connection between parts with different rigidities, i.e. for the liaison between the load bearing timber structure and a chimney of an external wall – often only decorative – of masonry (or even glass).

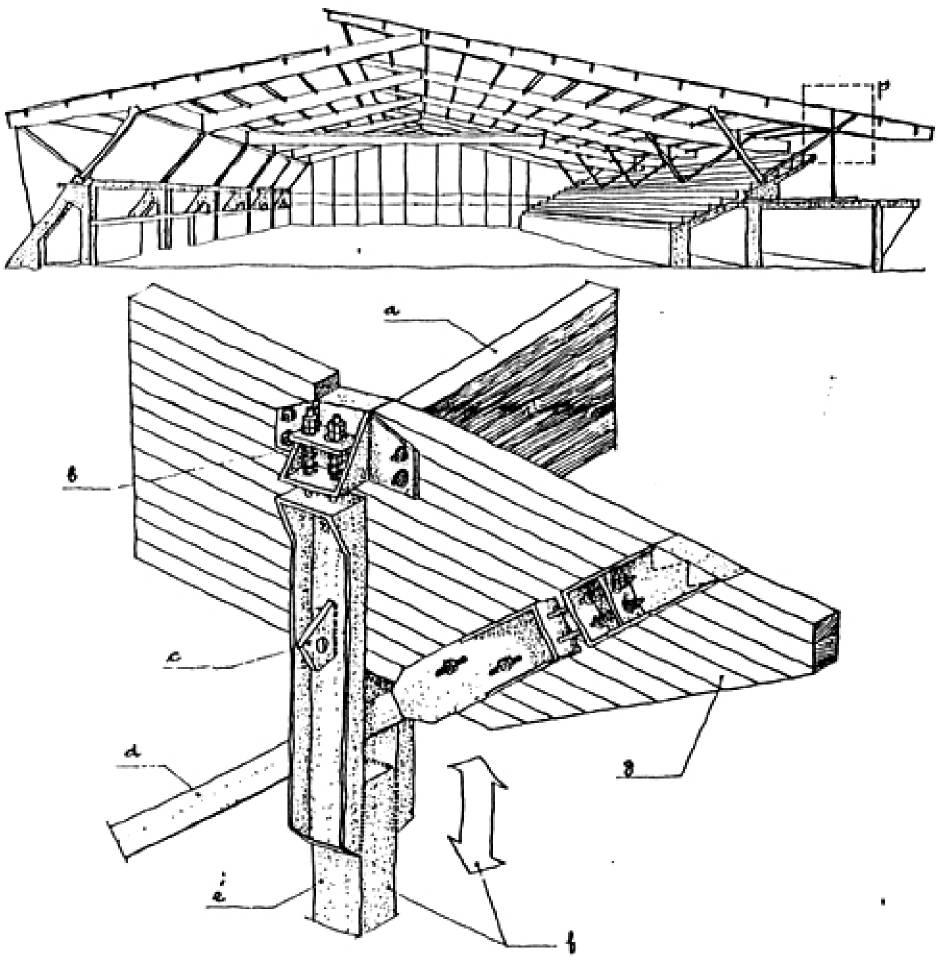

Fig. 7: *Timber framed sport - hall with special joint allowing independent movements between structural and not structural parts. In the enlarged detail p are visible: a - frame stiffening beam; b – heavy duty springs; c – bracing system joint; d – main frame steel rod with end spring; e – building façade steel column; f – independent movements between main frame and façade frame; g – main frame glulam beam.*

Designer has two possibilities: to realize a masonry structure rigid, self bearing and independent from the flexible timber structure or to connect the masonry part to the timber part so strictly that the two structures will act together. In the examples of figure 7 the glass façade wall is independent from the main structure. In most cases the connections between the external masonry walls and the inside timber structure are not properly conceived so that they just increase the horizontal action on the masonry wall because they add to the inertia forces acting on the wall per se, the pushing – action due to the larger movement of the timber structure.

If different material parts are connected, two simplified alternative design approaches are possible according to two different limit situations. On one hand one can consider that the masonry weight should be carried by the timber structure (i.e. light masonry with mass but no rigidity, that means masonry is considered fissured); on the other hand one must consider that masonry, more rigid than timber, will collect the totality of larger actions (i.e. heavy masonry). In other words the timber structure will still carry the vertical loads but will learn on the masonry structure regarding to the horizontal forces (Figure 8).

In case of figure 9, when because of an earthquake the masonry would collapse, dissipating a lot of energy, the timber structure has still the possibility to stand up after the shock and the masonry will be easily repaired.

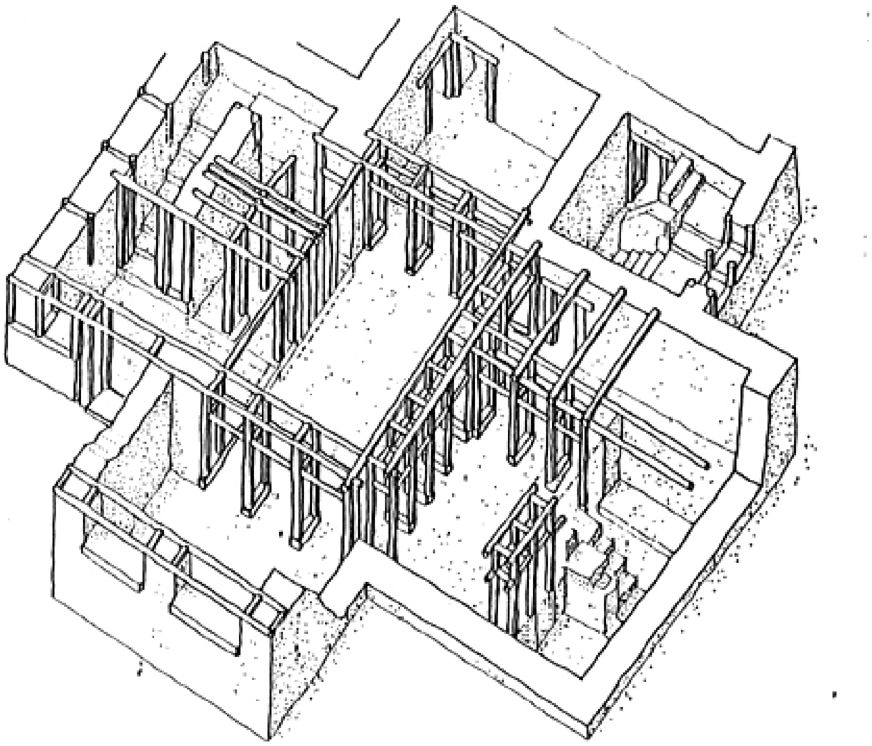

Fig. 8: *Example of mixed stone – timber building (Greek islands, 1500 b.C.). Timber framing contribute to support vertical loads but only heavy masonry resist lateral loads. Note that floors because of their workmanship cannot be considered rigid in plane.*

Ductility and dissipation of energy

In old buildings (i.e. timber framed buildings with brick infill, like in figure 9), dissipation of energy was obtained by friction between timber and masonry, and by hysteresis due to compression perpendicular to grain.

In modern structures, in order to reduce inertia forces, unless one decide to try to increase the natural period of vibration like in figure 7d, the most usual way is to dissipate energy by hysteresis in mechanical joints (see Lecture C17).

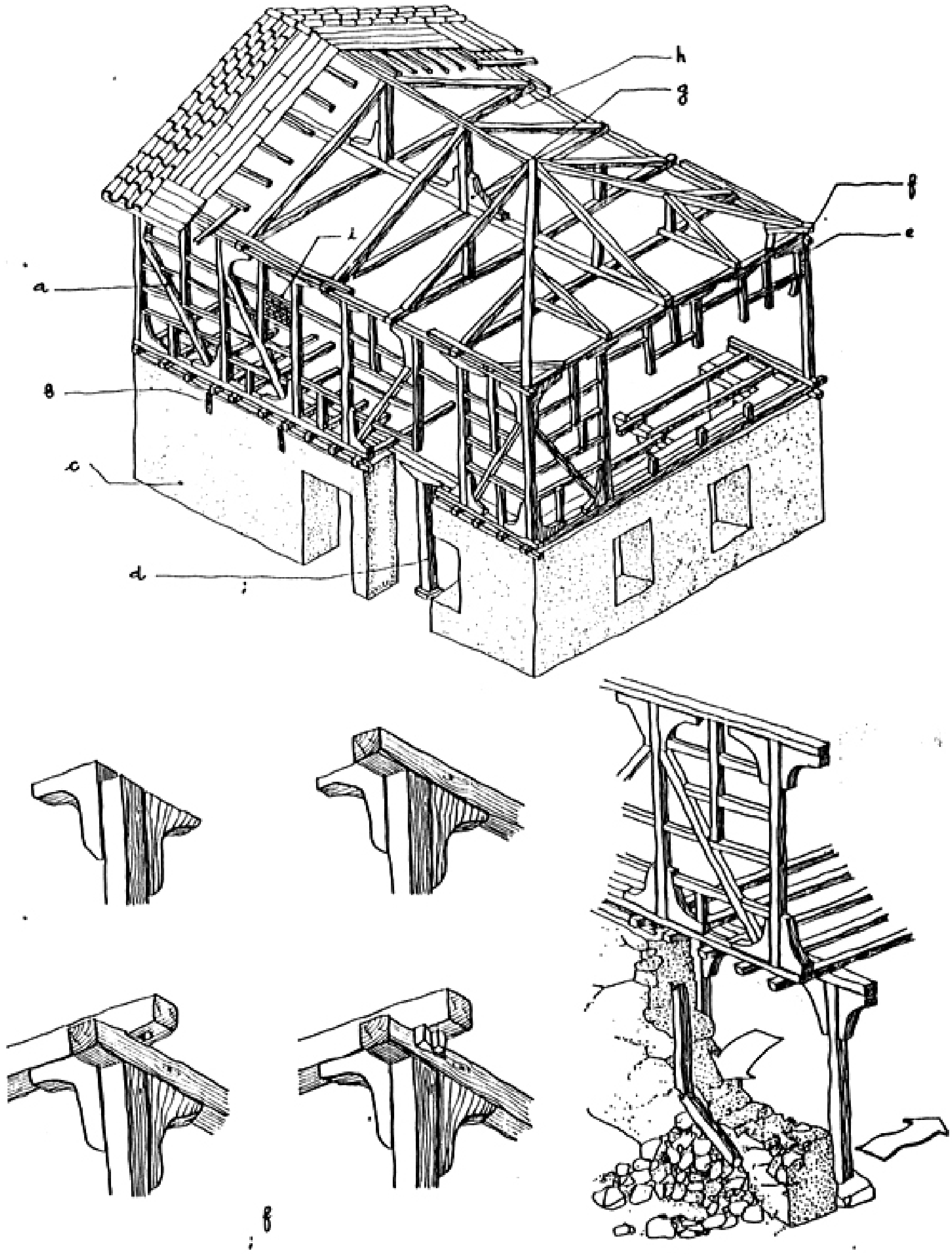

Fig. 9: *Example of mixed masonry – timber structure building in Greek islands (1800 a.C.). a – diagonal stiffening timber rod acting perpendicular to grain at the corner; b – anchoring detail of the wooden frame to the masonry wall; c – masonry wall bearing the wooden frame of the upper floors; d – secondary load bearing system of wooden columns, just behind the main load bearing system of masonry walls, able to support provisionally the entire structure after a severe earthquake (see the below detail, right hand side); e – wooden curved one piece component stiffening the timber wall frame; f – special joint easy to replace when damaged after an earthquake (see enlarged detail, below on the left hand side); g – wooden curved one piece component stiffening the roof; h – tie beam to avoid thrust on walls; I – brick infill. Also in this case floors cannot be considered rigid in plain due to their workmanship.*

EC 8 prescribes that when designing with reduced inertia forces (q>1) ductility and energy dissipation properties shall be demonstrated by tests. Anyway for some particular cases tests may be avoided if some details are followed. These details are mainly based on the past experience.

Shear panel systems had given excellent ductile behavior, much better than any diagonal bracing system. For that reason in the connection of sheathing to the timber framing, in EC 8 it is stated that for proper ductility it is sufficient that the sheathing material is wood based and the thickness t1 of the sheathing material is at least 4d, where d, nail diameter, does not exceed 3.1 mm. This is valid provided that the wood – based materials meet one of the following conditions: particleboard – panels with a density of at least 650 kg/m^{3,} plywood – sheathing at least 9mm thick, or particleboard – and fireboard – sheathing at least at 13mm thick. Smooth nails and staples are still suitable in diaphragms when connecting sheathing materials to the timber frame. A t2 nail length of 4 – 6 times the sheathing thickness is appropriate. However in the general case smooth nails are not recommended without additional provisions against withdrawal.

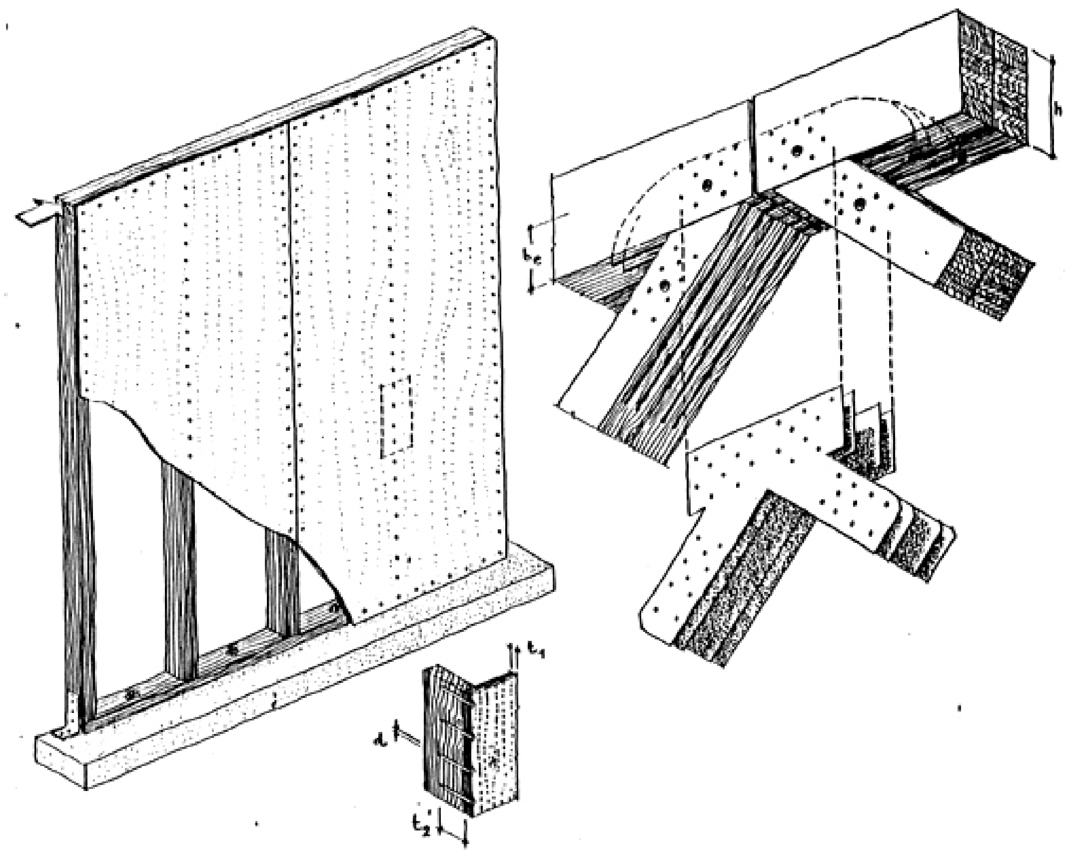

Fig. 10: *References for basic detailing requirements in EC 8.*

Besides EC 8 considers that doweled and nailed timber - to – timber and steel – to timber joints, when minimum thickness of the connected members is at least 8d and the dowel diameter does not exceed 12 mm, are sufficiently ductile. The reason why this sentence applies is that for the best performance under cyclic load a mode III failure of the joints is desirable (that means thick timbers and slender dowels). Now, if one looks at the diagrams in lecture C3 (see figures 11 and 12) it is easy to recognize that the fields corresponding to the desirable behavior are characterized by values of t1/√(My/fhd) and t2/√(My/fhd) rations bigger than 3.5.

Therefore with reference to the usual values of timber embedding strength and fasteners steel yielding strength it is possible to state that the minimum thickness of the connected timbers – 8d- is widely on the safe side, in ductility terms speaking.

Anyway if one wants to propose different fasteners arrangements or different materials, he is allowed, provided he will be able to demonstrate by tests that the EC 8 performance requirements under cyclic loading of the joint are fulfilled (see Lecture C17). Obviously before to propose new arrangements it is important to have clear in mind that the basic idea underneath is to try to obtain mode failure number III in order to couple the dissipation due to the embedding of the timber with the dissipation due to the plasticization of the fasteners steel.

Note: it is evident that some detailing rules given by EC 8 are thought for achieving the necessary ductility level. But what about the case that the designer chooses the possibility to design his structure without making reference to ductile and dissipative behavior, i.,e. q=1? In principle he is not obliged, for example, to use slender dowels, but anyway the use of slender dowels will for sure give to his structure a reserve of ductility always very welcome, without any extra cost. So the suggestion of the Authors is to follow as much is possible the detailing for ductile and dissipative behavior also for structures calculated as non dissipative.

Discussion and Conclusions

Structural detailing is very important in timber structures in seismic areas to guarantee the flow of the efforts through the entire resistant structure from foundations and ground to roofing and vice versa (structural form and continuity); and to ensure a sufficient resistance reserve (ductility and dissipation of energy). Codes and experience can give guidelines for a proper detailing but designer has the challenge to find for each case the most appropriate solution.

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OSB cement bonded structural panels

Papadopoulos, A., Ntalos, G.[9]

Abstract: *The objective of this research work was to evaluate the mechanical and physical properties of oriented strand board (OSB) using cement as binder. It was found that an increase of cement-wood ratio resulted to an increase in all, but MOR values. The MOE, IBS and TS values obtained with 2.0 wood-cement ratio conformed to the more stringent requirements of EN 300 for OSB/4. Further increase of cement-wood ratio to 3.0, only marginally improved these three board properties. Finally, it was observed that a lower cement-wood ratio required in order to manufacture acceptable OSB than particleboards and this maybe due to the strands geometry.*

Introduction

Cement-bonded wood composite panels are not a novel concept, having been on the market for over a century. In the past, these panels have consisted of excelsior and magnesite and have been used primarily as low-density insulating materials. By the early 1960's, a high-density cement-bonded structural flakeboard was developed leading to expanded applications (Deppe, 1974). Today, wood-cement panels have found acceptance in a number of countries as a result of certain desirable characteristics. The development and use of wood-cement panels attest to their attraction as building materials. In addition to their resistance to fire, these materials have a special attraction for use in warm, humid climates where decay and termites are a major concern (Jorge et al. 2004). The cement binder provides a durable surface as well as one that can be easily embossed and colored for an alternate, low maintenance finished product. The raw materials used are compatible with a range of processing methods to provide a variety of products that are easily machined with conventional wood-working tools. Although heavier than resin-bonded panels, they are lighter than concrete and, therefore, wood-cement panels can replace it in construction, namely prefabricated construction, in elements that are not subjected to loads, like walls. These attributes appeal to engineers, architects and contractors for use in public and multifamily residential buildings.

The majority of research in this field has been carried out on particleboards and flakeboards. The focused topics include the problem of the compatibility between cement and wood and ways of overcoming the problem, methods of manufacture and the properties exhibited by common wood composites, special techniques to accelerate the curing of cement and to improve the properties and finally manufacture of nonwood raw materials – cement composites. An excellent review can be found elsewhere (Jorge et al. 2004).

The objective of this work was to look at ways of manufacturing oriented strand boards (OSB) using cement as binder. Oriented strand board is a structural reconstituted panel that consists of wood strands glued with an exterior-type, waterproof resin. In the last decade, OSB has gained significant growth in the structural wood based panel market. This is the first study, as far as the authors are aware, where cement is applied to manufacture OSB.

Materials and Methods

Aspen ring-cut strands (*Populus alba*) were used in this study, with average strand size of 75 mm x 20 mm x 0.75 mm (length x width x thickness). The strands were air-dried to approximately 10% moisture content (MC). The bonding agent employed was commercial grade Portland cement, type I. Ammonium chloride (2% based on weight of cement) was introduced into cement slurry to accelerate cement set during hydration. A predetermined amount of air-dried strands and a NH_4Cl_2 (anhydrous) distilled water solution were thoroughly blended. Cement was subsequently added and the constituents were mixed until the cement paste completely hydrated. The quantity of distilled water added, was calculated using a relationship developed by Simatumpang (1979) and applied by other researchers as well (Jones et al. 1985; Moslemi and Pfister, 1987; Fuwape, 1995; Sudin et al. 1995). In his formulation, the water requirement was determined as follows:

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$$water\ (litres) = 0.35\ C + (0.30 - MC)\ W$$

where:

C = cement weight (kg)

MC = wood strands moisture content (oven-dry basis)

W = oven-dry wood strand weight (kg)

After 15 minutes of manual mixing, the cement-wood water mixture was screened onto to a caul. The mat was evenly distributed to provide as uniform a density as possible and pre-pressed to a thickness of approximately 50 mm. Cold pressing took place under an initial pressure of 2 – 5 MPa, depending on the cement –wood ratio to a 16mm thickness, after which the board was retained in compression for 24 hours. Target board density was 1000 Kg/m^3. Two replication of each board were made at cement-wood ratios of 3.0, 2.0, and 1.5 (by weight), giving a total of 6 boards. To minimize cement capillary desiccation and enhance hydrartion, boards were misted with distilled water, then wrapped in cellophane before storing for curing at 20^{0}C and 65% relative humidity for a month. Boards were tested for internal bond strength (IBS) (EN 319), modulus of rupture (MOR), modulus of elasticity (MOE) (EN 310) and thickness swelling (TS) (EN 317).

Results and Discussion

The data from boards made from various cement-wood ratios are summarised in Table 1. Table 1, reveals the inability of 1.0 wood-cement ratio to give boards with acceptable properties. It is interesting to notice the very low value of IBS, which denotes the complete failure of the boards. At this wood-cement ratio, IBS values of similar magnitude (0.27 N/mm^2) have also been reported by Fuwape (1989). The lower value obtained in this study may be consequence of the manual mixing of strands. Preliminary results showed, that mechanical mixing resulted to the breakdown of wood strands and therefore to the alteration of their final dimensions. Table 1, also shows that an increase of cement-wood ratio resulted to an increase in all, but MOR values. This is in line with the observations made by Moslemi and Pfister (1987). They found that the modulus of elasticity, internal bond strength and thickness swell increased linearly with greater cement-wood ratio (R^2 values of 0.89, 0.91 and 0.93 were observed from the data shown in Table 1, for MOE, IBS and TS respectively). The relationship between cement-wood ratio and MOR values is considerably different that of MOE or IBS or TS. The MOR values are decreased with an increase of cement-wood ratio, if we exclude the failure in boards made from 1.0 cement-wood ratio. The higher proportion of wood in the board may enhance the flexural property of the board. When wood occupies more volume in the board, the areas of stress concentration around the component particles are more diffused, resulting in increased to applied stresses (Moslemi and Pfister, 1987). Our results are in conformity with those made by other workers (Fuwape, 1985; Sudin et al. 1995) regarding the use of cement in particleboard and flakeboard manufacture.

Table 1. *Mechanical and physical properties of cement-bonded OSB. (Standard deviations in parentheses).*

Cement : Wood	**Density (g/cm^3)**	**MOR** (N/mm^2)	**MOE** (N/mm^2)	**IBS** (N/mm^2)	**TS** (%)
1.0	0.972 (0.08)	3.10 (0.6)	466.7 (31.1)	0.13 (0.009)	34.14 (4.8)
2.0	1.111 (0.09)	12.25 (0.9)	4949.1 (176.3)	0.87 (0.030)	9.42 (1.1)
3.0	1. 087 (0.11)	8.27 (0.9)	5212. 5 (155)	0.94 (0.05)	4.28 (0.33)

From the data presented in Table 1, it can be assumed that an optimum cement-wood ratio in order to manufacture OSB with acceptable bending properties (MOR in particular), is lower than 2.0; probably lower than 1.5. This proposed cement-wood ratio for OSB manufacture, appears to be lower than the corresponding ratio for particleboard and flakeboard manufacture. Commercial cement-bonded particleboards, made with cement-wood ratio of 2.75 to 3.0 are reported to attain acceptable properties (Bahre and Greten, 1977). Research with southern pine showed that cement-wood ratio higher than 2.0 has enabled the manufacture of cement excelsior boards with acceptable bending strength (Lee, 1985), whereas research with oil palm chips reported an optimum cement-wood ratio of 2.5 (Sudin et al. 1995). A ratio of 2.0 has been reported by Moslemi and Pfister (1987) and Fuwape (1995), both for particleboard manufacture. The lower cement-wood ratio required to manufacture acceptable OSB than particleboards and flakeboards maybe due to the strands geometry. A study carried out by Badego (1988) has shown that flake geometry (length and thickness) is highly correlated with board key properties, like MOR, MOE, IBS and TS. He found that the longer and thinner the raw material is, the stronger, stiffer and more dimensionally stable the boards are.

Conclusions

The objective of this work was to look at ways of manufacturing oriented strand boards (OSB) using commercial cement as binder. It was found that an increase of cement-wood ratio resulted to an increase in all, but MOR values. The MOE, IBS and TS values obtained with 2.0 wood-cement ratio conformed to the more stringent requirements of EN 300 for OSB/4. Further increase of cement-wood ratio to 3.0, only marginally improved these three board properties. Finally, it was observed that a lower cement-wood ratio required in order to manufacture acceptable OSB than particleboards and flakeboards and this maybe due to the strands geometry.

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Pull-out strength of glued-in rods

Widmann, R., Steiger, R. [10]

Abstract: *Bonded-in steel rods are very efficient in withstanding high forces applied to timber members. Investigations of bonded-in rods started in the late eighties of the last century and several design models were published since. An extensive literature review showed a certain degree of discrepancy and partly even contradiction on the influence of single parameters on the pull-out strength. Therefore a series of tests on glued-in steel rods with metric threads M12, M16 and M20 were executed. The rods were bonded in glulam made of Norway spruce lamellas perpendicular and parallel to the grain by means of an epoxy-type adhesive using the GSA®-system. The slenderness ratios of the rods calculated from the anchoring lengths with respect to the diameter of the drill hole varied between 7.5 and 12.5. Registered failure loads were compared with design values derived from different existing approaches. The pull-out strength was found to be almost directly proportional to the surface area of the bond line. Dependence between pull-out strength and anchoring length, slenderness ratio and diameter of the rod were examined. The influence of the wood density on the pull-out strength was verified. Compared to rods bonded in parallel to the grain, pull-out strength of rods with same diameter and anchoring length set perpendicular to the grain are 20 to 50% higher.*

Introduction

Glued-in rods are used in practice since the 1980's for strengthening glulam beams for transferring loads into timber members. A large number of investigations were executed on the structural performance of such connections in the past. However, literature reviews in Steiger et al. (2007) and Widmann et al. showed that certain discrepancies in particular on the influence of single parameters on the pull-out strength of rods existed. Differences in the assessment of wood density, rod to grain orientation, glued length and rod diameter in conjunction with the pull-out strength could be determined. One consequence of this fact was the late introduction of the structural behaviour of glued-in rods into construction standards. While recent standards like Eurocode 5 (2005) and the Swiss standard SIA 265 (2003) don't provide information about designing respective joints, only DIN 1052 (2004) covers glued-in rods and offers a design model.

A modification of the system of glued-in rods by the Swiss company n'H (Neue Holzbau AG, Switzerland) was introduced by Strahm (2000). This system, called GSA®, is characterized by rods with a reduced diameter on a certain length. The reduced diameter can cause - depending on its design and dimension - that the failure of the joint occurs in the steel and not in the wood and/or adhesive. This might be desired in practice, in particular to guarantee a good load-bearing behaviour of joints which rely on more than one glued-in rod. Fabris (2001) showed that removing the rod's thread at the upper section leads also to a shift of the anchorage zone to the interior of the specimens. Stress concentrations are reduced and splitting due to shear forces and stresses perpendicular to the grain are less likely to happen. With this a higher load bearing capacity of the joint can be achieved and/or a design with reduced edge distances can be realized.

In order to investigate the performance of the modified glued-in rod system with paying special attention to the assessment of the influence of the above mentioned parameters a respective test program was executed. Steiger et al. (2007) for rods glued-in parallel to grain and Widmann et al. (2007) for rods glued in perpendicular to grain reported the results in detail. In the following a compilation of the most important findings is presented.

Materials

Timber

The specimens were cut from glued-laminated timber made of Norway spruce lamellas of 40 mm thickness. The lamellas were free from any finger-joints or significant anatomical defects such as big knots and deviations of grain angle, in order to avoid negative influence on the results by these parameters. The glulam members were assembled using a melamine urea formaldehyde (MUF) adhesive. Two series of glulam beams were produced from lamellas with clearly distinct distributions of density as shown in table 1, in order to quantify the influence of the timber density on the pull-out strength of the rods. All specimens were cut from beams with a desired high or low density respectively.

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Table 1. *Densities of glulam lamella samples (Steiger et al., 2007)*

Parameter	*Oven-dry density ρ_0*			
	Low 1	*High 1*	*Low 2*	*High 2*
Sample size n	12	12	12	12
Mean value	371	493	378	498
Maximum	388	508	390	515
Minimum	353	481	365	492
Coeff. of variation	3.6%	2.2%	2.2%	1.5%
ρ_k [1)]	349	475	364	485
[1)] 5th percentile assuming normal distribution and $n = \infty$				

Adhesive

For the GSA®-system a special epoxy-type adhesive, free from any solvent and curing at ambient temperature has been developed by ASTORit AG, Switzerland. This adhesive performs well, as company internal tests showed. Shear strengths up to 35 N/mm^2 were reached between two threaded steel surfaces bonded together. Tests on threaded steel rods bonded in ash established shear failure in timber at nominal shear strength levels of 16 to 18 N/mm^2.

Steel rods

The GSA®-system is characterized by a reduced cross-section of the steel rods within a certain length λ_v. Removing the rod's thread within the length λv leads to a shift of the anchorage zone to the interior of the specimens. The drill-hole was filled with glue on its whole length $\lambda + \lambda_v$. However, it was assumed that the zone along the sinking length λ_v does not contribute to the pull-out resistance due to the lack of mechanical indentation of rod and adhesive. The sinking length λ_v was taken to be $5 \cdot d$.

The zinc coated steel rods with metric threads M12, M16 and M20 corresponded to strength grades 8.8 and 10.9 with a characteristic tensile strength f_{ub} of 800 and 1000 N/mm2 and a characteristic yield limit f_{yb} of 640 and 900 N/mm2 respectively. The rods were set in holes with diameters d_h that exceeded the rods diameter by 2 mm.

Specimens, equipment and procedure

The series of parallel to grain and perpendicular to grain had identical geometries regarding the joints (see Table 2). This should enable a direct comparison of the results.

Specimens parallel to grain

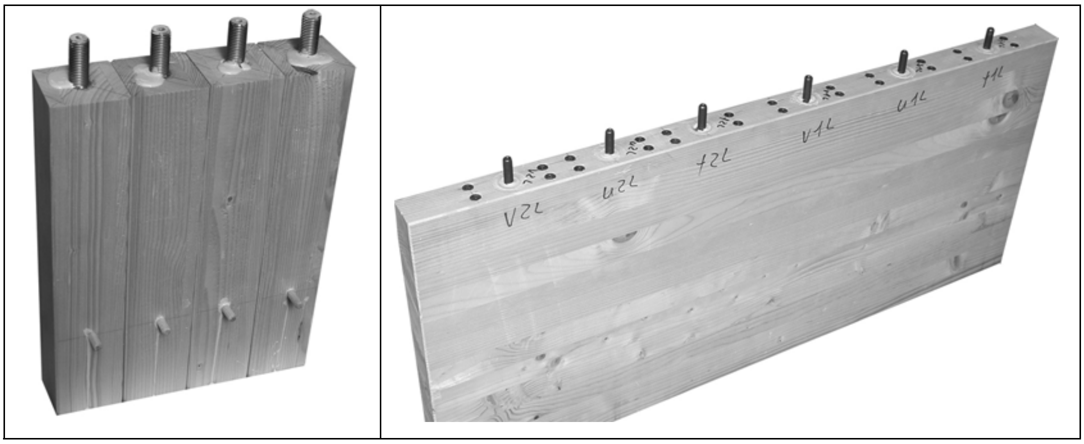
The rods set parallel to the grain were tested in a double-ended (pull-pull) configuration. This leads on one side to an efficient test set-up and on the other side increases the base for a statistical data analyse (Steiger et al., 2005) even if the failure occurs only on one end of the specimen, which is the normal case. In order to give the joints an optimal performance both the timber and the steel elements should have a similar stiffness ($E \cdot A$). However, to prevent early splitting of timber due to stress concentrations, the cross-sections of the specimens had to be designed in such way that bigger timber cross sections resulted. Geometry and dimensions of the specimens are shown in figure 1 and in table 2. In general there were four specimens per combination of rod diameter, timber density and anchorage length resulting in a total number of 72 specimens with 144 glued-in rods. In Steiger et al (2007) also the tests of other configurations are reported with a total of 192 glued-in rods.

Specimens perpendicular to grain

The perpendicular to grain specimens consisted of glulam beams with several rods bonded-in as shown in figure 2. Because for perpendicular to grain tests a pull-pull configuration is not possible a pull-pile foundation configuration was used. As piles a set of four screws for the specimens with rods of the smallest diameter (Figure 2) and a set of four glued-in rods for the specimens with 16mm and 20mm rods were used. The transfer of the support reaction via the piles into the wood excluded high compression stresses perpendicular to the grain in the loading zone which could have influenced the pull-out strength of the rods. Additionally it was assumed, that with this configuration a more or less uniformed stress distribution between the glued-in rods and the piles could develop. A total of 86 rods were tested perpendicular to grain.

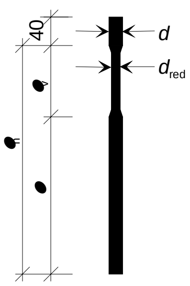
Equipment and procedure

All of the tests were carried out in a universal tension testing machine. The rate of loading was taken in accordance to EN 26891.



Figs. 1 & 2. *Specimens with rods glued-in parallel and perpendicular to grain. Only one end of the 4 shown parallel specimens is visible. At the top ends of these specimens leaked-out glue is present and in the lower part the closed filling holes for the glue can be seen. The sets of screws next to each rod in the specimens parallel to grain served as pile foundations for the support reaction.*

Table 2. *Specimen and rod dimensions for parallel and perpendicular to grain specimens*

Symbol	*Description*	*Geometry*	*Rod*
d	nominal rod diameters	12mm, 16mm, 20mm	
d_h	drill-hole diameter	d + 2mm	
λ	target slenderness ratios d_h/λ for each d	7.5 – 10 – 12.5	
λ_v	length of reduced diameter	$5 \cdot d$	
λ	glued length	$\lambda\ d_h\cdot$	
λ_h	length of drill-hole	$\lambda + \lambda_v$	
a	width of timber specimen	55mm, 75mm, 95mm	
e	distance to pile foundations (perpendicular specimens)	50mm, 80mm, 80mm	

Results and discussion

Detailed results and an extended discussion are published in Steiger et al. (2007) for tests parallel to grain and Widmann et al. (2007) for tests perpendicular to grain.

All specimens parallel to grain failed due to pull-out of the rods. Previous tests perpendicular to grain, with rod diameter 12mm and the longest glued-in length showed steel failure and the tests were repeated with use of a higher steel quality.

While about 65% of the specimens parallel to grain showed splitting of the wood cross-section (figure 3), no external signs for wood failure could be found for the specimens perpendicular to grain. As no significant difference existed between the failure loads of parallel specimens with and without splitting it is assumed that splitting was not the main failure mode but a consequence of the pull-out of the rods.

Some specimens were opened after failure to analyse the failure modes. The parallel to grain specimens showed shear failures parallel to grain in the wood close to the adhesive layer, shear failures in the wood-adhesive interface and also shear failures in the adhesive layer along the threaded part of the rod. The prevailing failure mode was found to be shear failure within the wood. For specimens that showed more than one failure mode it was not possible to determine primary and secondary failure mode(s).

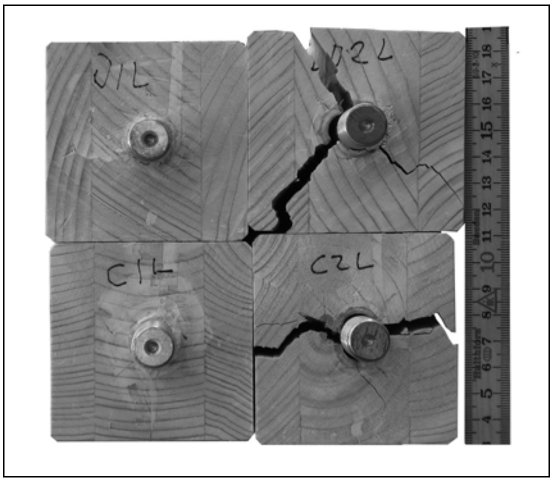


Fig. 3. *Specimens parallel to grain with and without end splits after testing*

Cutting single specimens out of the perpendicular to grain beams revealed cracks perpendicular to grain in the wood that were running parallel to the rods axis (figure 4). Big deformations of the wood in the direction of the pull-out together with respective wood compression and tension failures perpendicular to grain appeared close to the rods. Remaining wood fibres on the surface of the adhesive indicated a shear failure of the wood along the wood-adhesive interface.

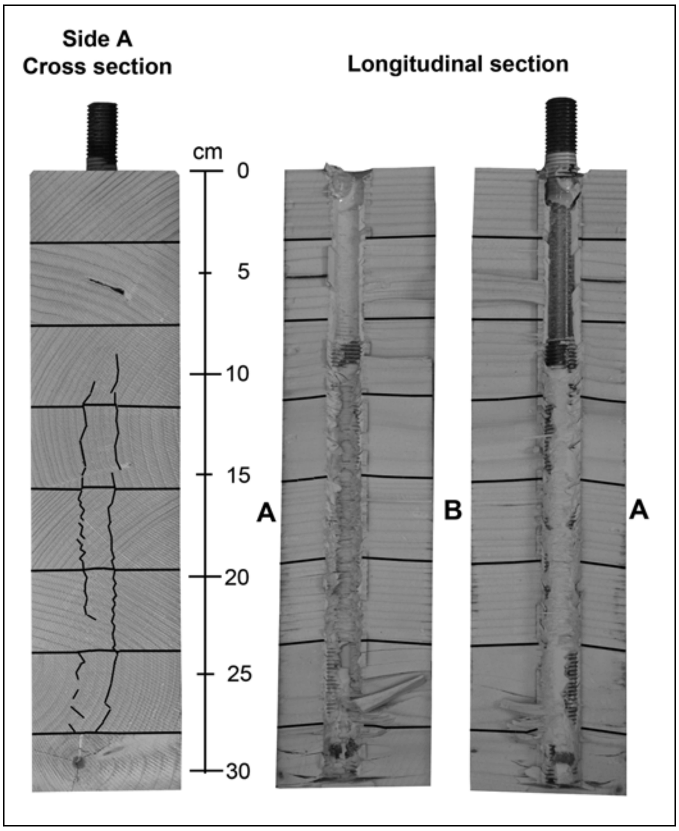


Fig. 4. *Specimen perpendicular to grain cutted out of the beam and opened by sawing and splitting. Cracks as well as adhesive layers of the glulam are highlighted. In the cross section cracks perpendicular to grain can be observed. In the longitudinal section the inclination of the adhesive layers indicate the remaining deformation of the wood after failure. The two upper layers don't show an inclination which confirms that in the zone of the sinking length no significant loads were transferred to the timber (Widmann et al., 2007)*

However, adhesive failure in the interface to the wood in some sections (no wood fibre on adhesive) as well as shear failure in the adhesive layer along the thread of the rod in other sections could be verified. Like for the specimens parallel to grain it was also not possible to keep primary and secondary mode(s) distinct for specimens perpendicular to grain that showed a combination of failure modes.

For analysing, it was assumed, that shear failure was the predominant mode and the respective ultimate shear stresses along the wood-adhesive interface were evenly distributed over the glued area which is defined by the glued length λ and the drill-hole diameter d_h.

An overview of the results is given in Figure 5. It can be seen, that the pull-out strength perpendicular to grain was higher than parallel to grain. The differences between the strengths of single series perpendicular/parallel with identical geometries were found to vary between 20% and 50%.

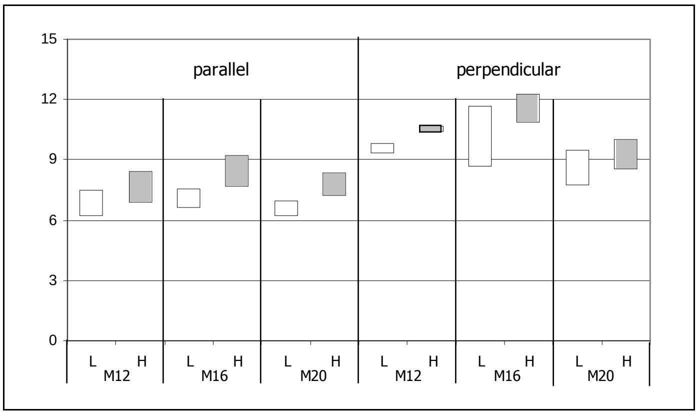


Fig. 5. *Range of mean pull-out strengths for each rod diameter, wood density (L = low, H = high) and orientation in reference to the grain on base of Steiger et al. (2007) and Widmann et al. (2007)*

The influence of single parameters on the nominal shear strength was investigated for specimens parallel (Steiger et al., 2007) and perpendicular (Widmann et al., 2007) to grain. The following table 3 contains an overview.

Table 3. *Dependency of nominal pull-out strength on single parameters.*

Parameter	*parallel*	*perpendicular*
Wood density	yes, $\rho^{0.6}$	yes, but big scatter, $\rho^{0.25}$
glued length	yes, $\lambda^{-1/3}$	yes, $\lambda^{-1/3}$
rod/hole diameter	no	no
slenderness ratio	yes, $\lambda^{-1/3}$	yes, $\lambda^{-1/3}$

In Widmann et al. (2007) the results obtained perpendicular to grain were compared to several existing design models. These models are based on different single or multiple parameters, like glued length (Riberholt (1988) and DIN 1052 (2004)), rod diameter (Bernasconi 2001) and several geometrical, stiffness and fracture-mechanical parameters which are combined in a ω value (Gustafsson et al. 2001). It turned out that the progression of the obtained strengths followed most of the models tolerably but always on a clearly higher level. The strengths exceeded the different models between 20% and 100%. This proved that the shift of the anchorage zone away from the surface of the wood specimen, as used used in GSA®, provides a high performance system for the application of loads into timber via steel anchors.

Based on the results of the tests suggestions for describing the load bearing capacity of the investigated GSA® system were published:

For rods set parallel to grain a shear strength formulation is given:

$$f_{v,0,mean} = 7.8N/mm^2 \cdot \left(\frac{\lambda}{10}\right)^{-1/3} \cdot \left(\frac{\rho}{480}\right)^{0.6} \qquad \text{Steiger et al. (2007)}$$

The pull-out strength of the rod $F_{ax,0,mean}$ can be calculated by multiplying the obtained nominal shear strength with the surface of the glued zone A_g which is defined by the glued length l and the hole diameter d_h.

For rods perpendicular to grain the pull-out strength of the rod can be determined directly on base of the surface of the glued zone A_g (mm^2) by:

$$F_{ax,90,mean} = 0.045 \cdot A_g^{\ 0.8} \quad \text{(kN)} \qquad \text{Widmann et al. (2007)}$$

These design-proposals are valid within the boundaries of the tested system parameters, in particular rod diameters, slenderness ratios of the holes and wood densities as defined above.

Conclusions

The investigated GSA® system shows in comparison to existing design models for glued-in rods pull-out strengths on a higher level. This confirms the positive effect of shifting the anchorage zone of glued-in rods from the surface to the interior of the timber on the performance of the joint. The pull-out strength depends on wood density for both, rods glued-in parallel and perpendicular to grain with rods bonded in high density wood having a higher pull-out resistance than those bonded in low density wood. On the assumption, that shear failure is the primary failure mode and that the shear stresses are evenly distributed over the entire glued zone the ultimate shear stresses decrease with an increase of the glued length and the slenderness of the glued zone. The diameter of the rod/hole as a single parameter does not influence the ultimate shear strength of the system.

The pull-out strength of rods set perpendicular to grain is about 20% to 50% higher than of those glued in parallel to grain.

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"Structural Bonding"

Chaired by Dr. **Klaus Richter**

Structural rehabilitation of wooden ceilings using a high-tech composite system

Schober, K.U., Rautenstrauch, K.[11]

Abstract: *The structural rehabilitation of wooden ceilings is widely-used with steel and timber members in residential buildings. Apart from these traditional methods, recent investigation indicates that carbon composites can be effectively used to improve strength and stiffness of timber beams in the tension zone. The immediate potential seems, however, to be for FRP reinforcement by retrofit. Fiber reinforced polymers have been evaluated for engineered timber products, but only a few investigations have been carried out on their use for strengthening old timber beams. The limits are distinguishable in timber crushing under bending loads and the quality of the bond line itself. A high strength material can be the optimal add-on to restore the load-bearing capacity in the compression zone. This paper presents a fundamental approach of a new on-site application technique using high-tech composite materials for restoration and structural upgrading of a historical wooden ceiling in Mansfeld Castle / Germany. The idea of this easy-to-use application was focused on combining the favorable characteristics of an epoxy resin-bound polymer concrete layer, the existing timber beams and FRP reinforcement in a composite structure. Hereby, the tension-stressed timber and FRP and compression-stressed PC layer are offering a good load-carrying behavior. The material properties of the composite beam and the application are described.*

Introduction

Timber structures in residential houses from the past centuries are designed for lower live loads as specified in current design standards. Most of them have a couple of damages insect and fungal attack and need specific interventions to restore these damages for renovation. For renovation they need new technologies to increase the load-carrying capacity of the members for state-of-the-art housing conditions, including reinforcement or repair. Such techniques are very promising as they minimize disturbance of the building and to its occupants during the intervention.

A study of reinforcement techniques for restoration and strengthening of existing timber floors under bending loads has been carried out at the Bauhaus-University of Weimar. The experimental and numerical study consists of two parts: The use of epoxy resin-based polymer concrete as strengthening material, whereby the removal of the suspended ceiling on timber floors is not necessary and the use of structural adhesives on the building site, whereby the removal of the overhanging part of the structure as well as the inserted ceiling is not necessary. The main task of the test series performed was gathering and generating qualitative and quantitative knowledge on high-tech composite materials suitable for on site repair and upgrading of timber structures. This will enable the effective and safe application of reinforcement techniques especially in high demanding situations where the present lack of knowledge and reliability of these products restrain their application.

Reinforcement techniques for structural timber elements, based on the use of structural adhesives on site, have been applied for some decades as an extension of procedures that became very common for the repair or the upgrading of other structures. Reinforcement techniques using adhesives are very promising as they minimize disturbance of the building and to its occupants during the intervention. However, some problems have prevented the wider use of structural adhesives, particularly in historical timber structures, where sufficient reliability cannot yet be guaranteed. One reason is that the service life has not yet been fully proven for synthetic adhesives, since the oldest bonded joints are around sixty years. Greater ages cannot be simulated by existing accelerated ageing tests. Besides, although the failure of a structural bonded joint may be responsible for the collapse of the whole structure. Short and long-term performance of glued bonds highly depends on the bonding process, which is especially difficult to control on the building site. Since properties of reinforced elements very much depend on the carefully work on site, such difficulties have to overcome and first procedures for applying and controlling were established (CEN TC 193/SC1/WG11 2003).

Apart from this, each piece of wood differs in the amount of stiffness-reducing defects such as knots, splits, and checks and therefore, it is hard to say at what stress level the reinforced beams would have failed if they had not been reinforced. Such is the nature of wood. But the wooden beams reinforced with CFRP reveal more ductile behavior with respect to un-reinforced beams. The presence of CFRP reinforcement arrest crack opening, confines local rupture and bridges local defects in the timber. Therefore, the specimen can support higher loads before failure. Moreover, as can be observed from the experimental study, there is an increase of

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the load-carrying capacity because of the quoted crack opening arrest but the limits are distinguishable in timber crushing under bending loads and the quality of the bond line itself.

A high strength material can be the optimal add-on to restore the load-bearing capacity in the compression zone. This add-on was found in an epoxy resin-bounded polymer concrete. The idea of this application was focused on combining the favorable characteristics of the polymer concrete layer, the existing timber beam and FRP reinforcement in a composite structure. There, the disadvantages of the single components are compensated and the tension-stressed timber and FRP and the compression-stressed PC layer offering a much better load-bearing behavior. Polymer Concrete is a composite material formed by combining mineral aggregates such as sand or gravel with a monomer. Rapid-setting organic polymers are used in PC as binders. Studies on epoxy polymers have shown that curing method, temperature and strain rate influences the strength and stress-strain relationships. PC is increasingly being used as an alternative to cement concrete in many applications. Today, polymer concrete is used for finishing work in cast-in-place applications, precast products, highway pavements, bridge decks and waste water pipes. PC exhibits a brittle failure and therefore improving its post-peak stress-strain behavior is important. Hence developing better PC systems and characterizing the compressive strength in terms of constituents are essential for the efficient utilization of PC. However, the data on epoxy PC are rather limited, and there is an increasing interest in the deformation characteristics under working conditions in combination with other materials such as wood for composite structures.

Materials

Adhesive and CFRP reinforcement

Two basic approaches are discussed here:

- The use of reinforcement materials embedded internal in the wood specimen.
- The use of external reinforcement resulting in a system of composite type.

The old solid wood beams were reinforced with a continuous carbon fiber lamella S&P 150/2000 with intermediate modulus fibers from S&P Reinforcement Ltd. and 3.15 m length within the clear span for external reinforcement, otherwise over the full length of 3.50 m embedded in the wooden beams. The CFRP layer with a cross-section of 1.4 x 50 mm was glued / embedded by means of the commercially available epoxy resin StoPox SK 41 from StoCretec Ltd, which has a technical approval for gluing together with the carbon fiber lamellas. The mean mechanical properties are shown in table 1.

Table 1: *Mean material properties of used epoxy resin and CFRP*

	Unit	*Epoxy Matrix*	*CFRP*
Tensile Strength	MPa	75	2 200
Tensile Modulus of Elasticity	MPa	2 800	164 000
Ultimate elongation	%	3.5	1.4
Density	g/cm³	--	--

Polymer concrete

The polymer concrete Compono® 100S consist of the epoxy resin Compono® 100H, the accelerator Compono® 100 and special gravel with a grading of 0-4 mm. The components were mixed in a ratio of 100:19:595 by volume (14:2.67:83.33 by weight). Comparing the used PC with high-strength concrete C100/115 on a curing temperature of 20°C after seven days, the mean compressive strength is about the same, whereas epoxy PC has a triple value of the bending strength of C100/115. A comparison with the usually on building site used concrete C25/30 shows table 2.

Table 2: *Mean material properties of used epoxy resin-bounded PC*

Property	*Unit*	*Epoxy PC Compono® 100S*	*Concrete C25/30*	*Comparison*
Density	g/cm³	~ 2.0	2.0 … 2.6	1 … 0.77
Tensile MOE	MPa	19.600	30,000	4.00
Bending strength	MPa	30	5.5	5.45
Compressive strength	MPa	110	30	3.37

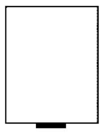
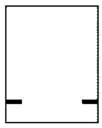
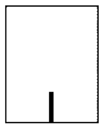
The mechanical properties of the used epoxy PC were investigated depending on the ambient air temperature when cast on site and the curing time. Because of the laboratory tests, only a small increase of the compressive strength with cumulative curing time and storage temperature could be observed. Low temperatures result in an interruption of the chemical reaction in cement bounded concrete. In epoxy resin bounded PC the chemical reaction continues when the ambient air temperature is rising. Higher temperatures cause in higher early compressive strength under standard climate conditions. The post curing under standard climate conditions result in strength increase up to 10% compared with the values after one day (Schober and Rautenstrauch 2006).

Reinforcement systems for wooden ceilings

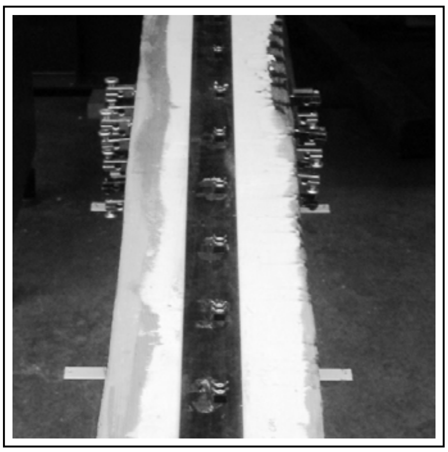
Structural upgrading with FRP reinforcement on bottom

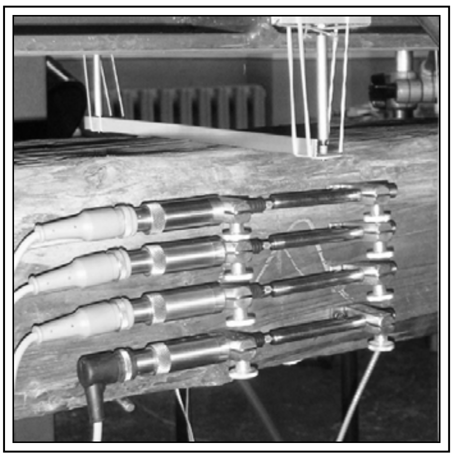
Three different reinforcement schemes were evaluated in the testing program (table 3). The gluing of the lamellas was done under practice-related conditions. The timber surface was primed using StoJet HIS, a two-part epoxy to saturate the wood prior gluing to avoid desiccation of the resin and to ensure a full compound between resin and wood surface.

***Table 3:** Dimensions of used timber beams and reinforcement schemes*

Series	Height [cm]	Width [cm]	Type	Description
Vh	15.42…19.33	15.14…20.30		1 x 1.4 x 50 mm bonded centrally to the tension zone, horizontal on bottom
Vs	14.64…21.28	15.30…18.28		2 x 1.4 x 25 mm bonded laterally to tension zone 3 cm from bottom in slot
Vv	15.74…21.09	15.00…19.33		1 x 1.4 x 50 mm bonded centrally to the tension zone, vertical on bottom

The main task of the test series performed was gathering and generating qualitative and quantitative knowledge on CFRP suitable for on site repair of timber structures. We have studied three practical solutions of bonding on the building site in order to investigate the different performance of the composite structure. For investigating the strengthening effect of the carbon fiber reinforcement in different, practice-related positions, the flexural behavior of all specimen were tested first within the elastic range with and without reinforcement (Schober and Rautenstrauch 2005a, 2005b). The tests were executed as a four-point bending test according to EN 408. The vertical and horizontal displacements of the beams were measured using inductive transducers in the span and on the support brackets (figure 1), as well as 2D close range photogrammetry in the midspan of the specimen (Rautenstrauch et al. 2004). Strain gauges were placed on the external bonded CFRP lamella to obtain the elongation of the reinforcement itself (figure 2).





***Figs. 1-2:** Deflection measurement on face and bottom of the specimen*

Structural upgrading with polymer concrete facing on top

The solid wood beams were reinforced with a commercially available epoxy PC Compono® facing on top in a thickness of 2.5 and 3.5 cm over the full beam length of 2.50 m (Figure 3, 4). The casting of the epoxy PC was done under practice-related conditions. The timber surface was primed using the commercial primer StoJet IHL, a two-part epoxy to saturate the wood prior casting to avoid desiccation of the PC and to ensure a full compound between PC and wood surface.

The main advantages are:

- The section design can de done easily by a timber formwork on the level of the necessary construction height.
- All work can be done from top side; the suspended ceiling will remain unaffected.
- The floor below the reconstruction work can be used without any restrictions.
- The full load-carrying capacity is achieved after one day.

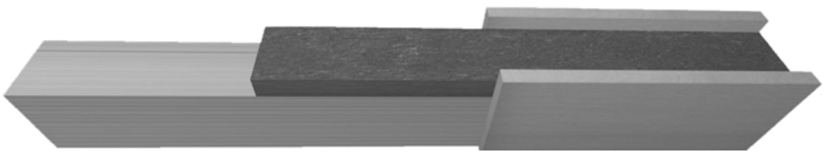


Fig. 3: *Timber beam with formwork and PC facing*

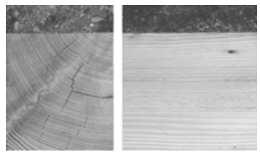
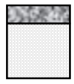
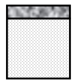
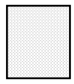
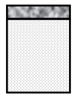
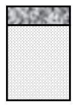


Fig. 4: *Side views*

For characterization of the load-carrying behavior of the hybrid composite structure the mechanical properties of the bonding joint between polymer concrete and timber are important. Therefore, different tests for investigating the structural performance in tension and shear were done. Since these types of building systems have been realized as timber-concrete composite beams mainly taken for bridges or revitalization of timber floors, dowel type connectors were usually studied and taken for transfer of shear forces. These steel connectors realize the transmission of shear stress in the contact area of timber and surface layer in timber-concrete composite structures. In the building system described here, only the natural adhesive bond between timber and epoxy PC should be sufficient to transfer the shear- and tension forces in the compound joint. To obtain practice related conditions of the wood surface, the surface roughness of the specimen was modified as planed, sawn, milled and without any modification.

Under tension forces, the fracture of the specimen appeared in most of the tests as cohesion fracture in timber. In some cases failure occurred as adhesion fracture in the primed bounding zone. The bonding behavior of the test specimen can be assumed as a rigid joint where the tensile strength perpendicular to the grain was measured. Due to damages from former insect attack, the axial tensile strength perpendicular to the grain for the matured spruce has around 30% lower values than new timber. Under shear forces, all test specimens failed also by timber fracture with sudden bursts. Measurable deflections in the composite joint were not recorded. Therefore, a rigid bonding behavior between timber and epoxy resin bounded polymer concrete can be assumed. Under bending loads, we have studied five practical solutions of timber-PC configuration in order to investigate the different performance of the composite structure and to compare the strengthening effect by shortening the beam height and substitute with PC as well as by adding additional stiffness in form of epoxy PC on top to attain the highest possible strengthening effect with regard to of construction time and costs (Table 4).

Table 4: *Mean dimensions and upgrading schemes*

Series	*Length*	*Width*	*Height*	*Section*
B/10/3	2.50 m	12 cm	10.5 cm timber + 3.5 cm PC	
B/11/2	2.50 m	12 cm	11.5 cm timber + 2.5 cm PC	
B/14/0	2.50 m	12 cm	14.0 cm timber	
B/14/2	2.50 m	12 cm	14.0 cm timber + 2.5 cm PC	
B/14/3	2.50 m	12 cm	14.0 cm timber + 3.5 cm PC	

Results

FRP reinforcement

The tests have shown that the most frequent failure mechanism is the one in which traction failure and shear failure occurs with or without partial plasticization of the compressed zone. The different failure modes for different strengthening methods are described in Schober and Rautenstrauch 2005a. The reinforcing scheme increased the load-bearing capacity by mean 6% in comparison to the values measured for the un-reinforced wooden beams. The strength increase was defined as the bending stress of the reinforced specimen at the deflection in linear range before failure divided by the bending stress of the un-reinforced specimen at the same deflection value and shown for each series in figure 5. Predicting the stiffness and strength increase that occurs when a timber beam is reinforced with carbon fibers is a complex problem and difficult to establish because of the natural defects that occur in wood and drastically reduce the stiffness. However, design of wood structures has been accomplished for decades by applying stress modification factors to the allowable design stress values according to national codes for a given size and grade of timber. Accordingly, allowable stress modification factors have been conceptually developed based on the experimental results from this study.

For investigation of the different reinforcement schemes and failure modes a non-linear finite element analysis was obtained using a delamination analysis and interface damage law described mathematically by Barenblatt (1962) and Needleman (1987). Delamination of composite laminates, such as CFRP, have the peculiarity that the non-linear constitutive behavior can be limited to a small region around the lamella where the crack propagates, while the remaining part of the structure can be assumed as linear elastic. The timber beam was modeled using symmetry conditions and the anisotropic properties obtained from the beam specimen, for the glue line cohesive interface elements were used. The numerical model, geometry and load conditions are described in Schober et al. 2006. The comparison of the obtained flexural behavior and the numerical simulation are shown in figure 6. Pre-existing cracks and fissures in the structure result in an overestimation of the bending stiffness in the delamination model.

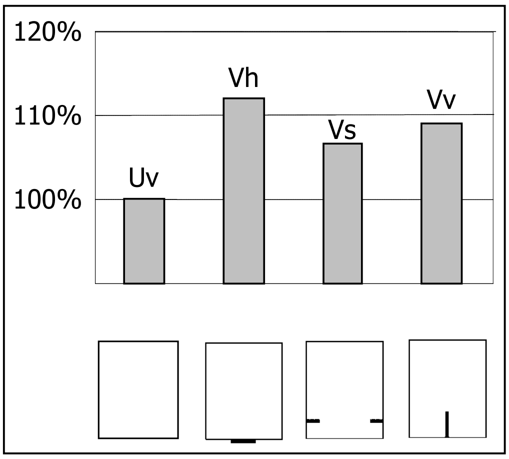


__Fig. 5:__ Increasing of load-carrying capacity with CFRP, comparison with unreinforced series

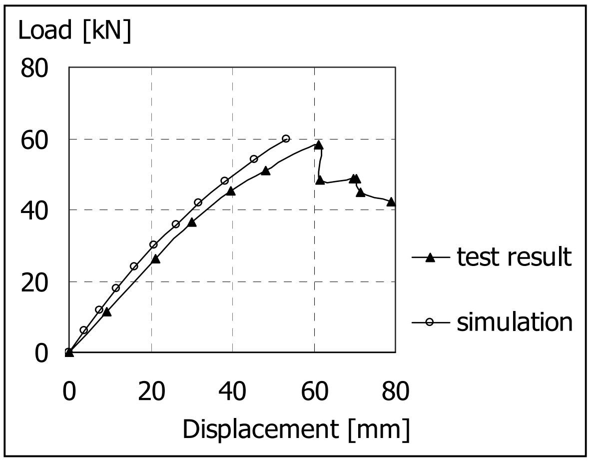


__Fig. 6:__ Load-deflection behavior of specimen S02/2 Vs with lateral reinforcement

PC reinforcement

As result of the axial bonding and shear tests, the structural behavior of the compound between timber and epoxy resin-bounded polymer concrete can be assumed as rigid. This assumption could be confirmed by the bending tests where failure occurred in the timber traction zone without complete plasticization of the compression region in all specimen and test series. The structural performance is shown to be linear elastic up to local failures induced by the presence of defects e.g. knots and cracks, followed by wood fracture in the tension zone. The tests have shown that the most frequent failure mechanism is the one in which traction failure and shear failure occurs. Therefore, additional FRP reinforcement of the traction zone will increase the load-bearing capacity and confines local rupture.

The results of the bending test show an increase of the load carrying capacity by 2.5- to 3-times the value without PC facing, where the difference of the bearable load between the series with 2.5 cm and 3.5 cm epoxy PC is not so high (figure 7). Therefore, the structural system with 2.5 cm polymer concrete is the more economical one and preferable.

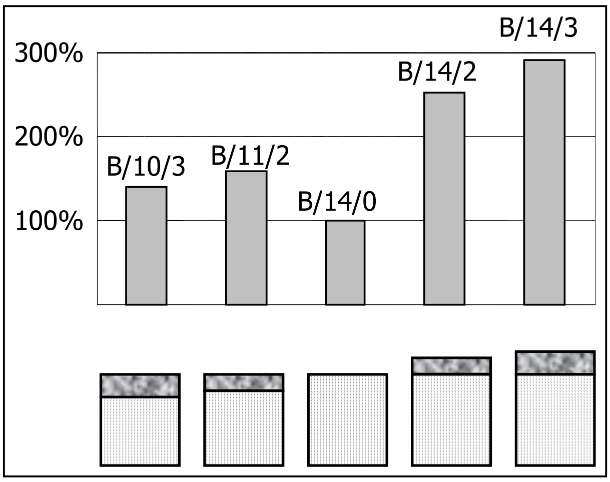


__Fig. 7:__ Increasing of load-carrying capacity with epoxy PC, comparison with unreinforced series (12 x 14 cm cross-section)

Combined CFRP and PC reinforcement

On the one hand, the worldwide development of the timber-concrete composite structures has shown that hybrid composite constructions are a very efficient solution to increase the load-carrying capacity of timber structures. On the other hand, the advantages are limited by the wide scattered mechanical properties of the timber. The answer seems to be the combination of the two described strengthening methods into a composite system consisting of an epoxy resin-bound polymer concrete layer on top, the timber beam and FRP reinforcement on bottom. Hereby, the close mechanical interaction of the tension-stressed timber and FRP and compression-stressed PC layer are offering a good load-carrying behavior. For comparison, the different levels of the ultimate bending moment carried by the structure are shown in Figure 8.

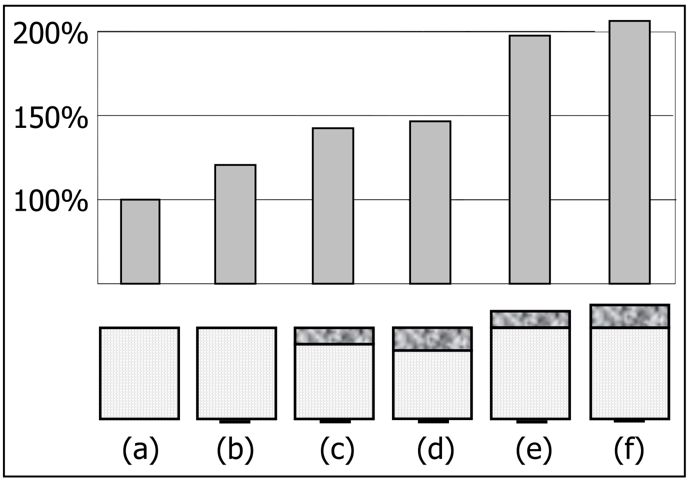


Fig. 8: *Ultimate bending moment compared to the unreinforced beam*

The most economical combination of both systems seems to be configuration (e) with only a few centimeters of PC on top (here 25 mm) and a light reinforcement of the tension zone (here 50x1.4 mm CFRP with intermediate modulus) to confine the scattering of the mechanical properties and local defects. The increase of m_u is propagated up to 200%. For any restrictions, e. g. of the construction height the configuration (c) offers a good value with 1.5 time's load-bearing capacity.

Example: In-situ reinforcement of Mansfeld Castle

Accompanying to the theoretical investigations, both systems were tested for renovation and upgrading of existing structures independently and together under practice-related conditions on site. First practical insights have shown primarily doubts regarding cleanliness and feasibility could not confirmed and the work progress was done accurate and efficient after short briefing of the construction worker. The new occupancy and floor design of the Mansfeld Castle required an increase of the existing dead loads and the life loads for the structural design of the waffle slab above the „Blauer Saal (blue hall, figure 9, 12)". Due to high loading from the secondary girders and the specific construction of the waffle slab and main girder the combination of both reinforcement systems was chosen – upgrading of the tension zone with carbon fiber lamellas S&P 200/2000 (figure 10) and an additional PC layer topping with Compono® (figure 11)". The structural design was done using a finite-element model and a beam model independently.

***Fig. 9:** Waffle slab, bottom view*

***Fig. 10:** Sticking of the FRP lamellas*

***Fig. 11:** PC reinforcement from top floor*

***Fig. 12:** Finished reconstruction work*

Conclusions

Epoxy resin-based polymer concrete can be combined with timber floors for upgrading without necessitating the removal of the suspended ceiling. This technique is very promising in many cases of reinforcement of timber floors and historical structural wood parts. The properties of the glue line can be described as a rigid continuous joint. This includes the good adhesive penetration into the wood surface and the high cohesive strength of the glue line in terms of further design and calculations. The use of CFRP as a strengthening technique can be applied without necessitating the removal of the overhanging part of the structure. For practice related investigations effective cross-sectional data including existing cracks, knots and damages were used with a reduction of the initial bending stiffness and the moment of inertia respectively of about 17 % compared to the cross-sections without defects. The presence of CFRP reinforcement arrests crack opening, confines local rupture and bridges local defects in the timber especially for reinforcement types other than on the bottom face. The combination of both systems seems to be an optimal configuration with a load-carrying capacity of up to 200% compared to the unreinforced beam.

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Use of reinforced epoxy plates for the rehabilitation of timber beams

António C. R. Duarte[12], **João H. O. Negrão**[13], **Helena M. P. Cruz**[14]

Abstract: *This paper discusses a rehabilitation system for timber beams, consisting of the removal of the damaged parts and its replacement by elements of similar sound wood. The connection between new and old timber is achieved by mean of a reinforced epoxy plate, obtained through the insertion of reinforcement into a groove and pouring of a low viscosity adhesive in it. Mild steel rough bars, stainless steel threaded bars and pultruded glass-fiber bars were tested as reinforcement alternatives. Though similar systems are frequently used in the replacement of beam ends, in which shear is the dominating stress, this study emphasizes the bending behaviour. Therefore, its results and conclusions are particularly relevant for elements damaged by that type of stress, as is the case with midspan bending collapse of overloaded beams. Although it was not possible to restore the original strength and stiffness of the solid beam, the system revealed to be suitable for actual works, because the estimated design value of the rehabilitated beam is larger than that provided by Eurocode 5 for the solid timber beam. Besides, the results showed that the solution may still be improved by increasing the amount or grade of the reinforcement.*

Introduction

Timber beams in ancient structures often show damage due to biological, chemical or mechanical causes. While in some cases such damage is likely to affect the whole element volume, as is the case with beetle infestation, in others it may occur in more limited zones, as the decay caused by high moisture content in the ends of beams supported by masonry walls, or the midspan fracture caused by bending collapse of overloaded beams.

The rehabilitation works should thus be adapted to the specific pathology. The indiscriminate removal of the whole structural element may be an inappropriate technique, if only a limited area is at risk. Besides, the wood species/grade or the element size may not be available at suppliers, leading to the need of using an alternative material, such as glulam, which may not be acceptable from the point of view of authenticity.

The obvious alternative is that of repairing the existing element by removing the damaged parts and replacing them with sound wood, preferably of the same grade or species. The relevant issue here is how to connect the old to the new timber. Either steel plates with fasteners or resin-based splices are the most usual solutions.

This paper discusses a rehabilitation technique of the latter type. The connection of the wooden parts is achieved through a reinforced epoxy mortar plate inserted in a slot. Different reinforcement materials are tested, namely mild steel A400 NR (S275 to EN10025) ribbed rebars, stainless steel threaded rods and pultruded glass fibre rods (GFRP).

The proposed technique is moderately intrusive and is especially suited for the rehabilitation of timber structures locally affected by decay or subterranean termites. This problem is frequent in ancient buildings and occurs mainly in the beam ends, the middle part of the element remaining in good condition.

The practical application of this rehabilitation technique can be done in a number of ways, all sharing some common basic procedures (Freas, 1982).

Though the practical use of these techniques started in the 80's, there is not yet a complete and generally accepted knowledge about them, from both the theoretical and experimental points of view. The availability of calculation methods is still limited and in some repair situations the design isn't even done or consists of using some rule of thumb (Gemert, 1987).

The research work described here was thus focused on issues concerning the experimental behaviour and the analytical procedure for the design of bending splices made up with reinforced epoxy plates. A procedure for the reinforcement design is proposed and the predicted ultimate load is compared to that determined by tests.

Some other aspects are also relevant for the safety of the connection, such as the adhesion strength between the timber and the epoxy mortar, required for the definition of the interface area, or the mortar-reinforcement adhesion, essential for setting the bonding length of the reinforcement bars. The limited human and material resources available, however, prevented those topics to be addressed within the scope of the described work.

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Bonding Properties

As previously referred, the bonding between the rehabilitation system elements is achieved by adhesion between timber and the epoxy mortar poured into a slot drilled in the timber, after having placed the reinforcement bars.

Figure 1 shows the geometry of the splice, for the case using steel rebars. A 10mm wide gap between old and new timber elements was considered. The cross-section is bxh=75x125 mm^2. The groove width was set to 21 mm, in order to allow for minimum coverage of the 10mm diameter rebars, which were placed in three layers (V1 to V5 beams). In order to improve the adhesion, rods were first sand-blasted and then abraded with a steel wire brush, resulting in a rough and rust-free surface (Duarte, 2004).

Although the basic concept remains, the scheme of Figure 1 has gone through slight adaptations in the cases of reinforced beams with stainless steel (V6 and V7) and with fibre glass composite rods - GFRP (V8 and V9 beams). In these cases, timber beams with cross-section bxh=75x110 mm^2 were used. The reason for this was simply the economic need of using the timber sizes available for the research. The stainless steel reinforcement consisted in two threaded M14 rods displayed in two layers, while the GFRP reinforcement was made up with six 5mm diameter rods placed in three layers. In both cases, the rods were carefully cleaned before placement and, in the GFRP case, the surfaces were roughened with sandpaper. The spacings between rods and between bottom face and the first rod were set equal to the rod diameter, for both reinforcement types.

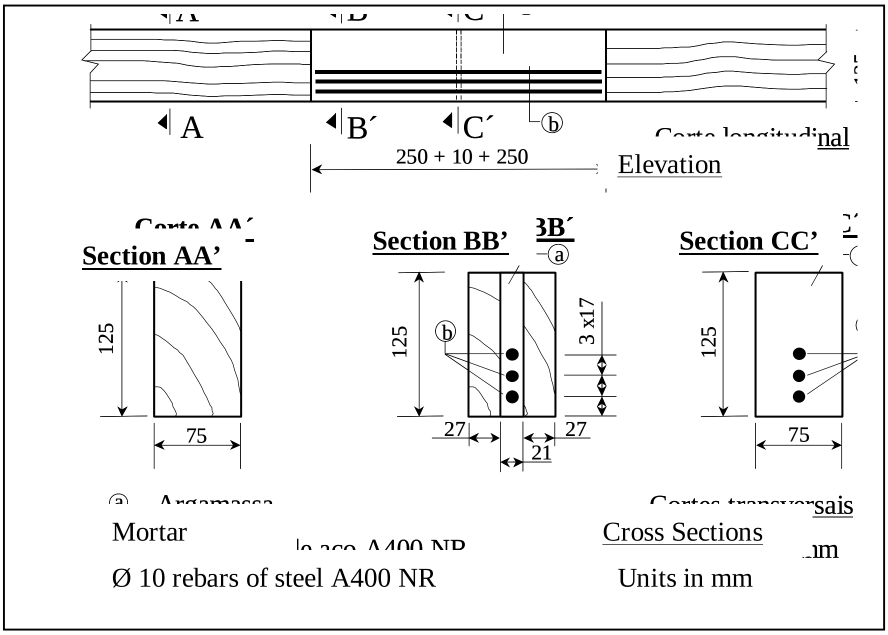


Fig. 1: *Rehabilitation system with A400 NR rebars*

Test Setup

In order to ensure the correct execution of the designed rehabilitation system, the following provisions and procedures were adopted (Duarte, 2004):

- The parts to be bonded were put in place with the guidance of 25x25x1,5 aluminium corner studs, regularly screwed along the length to be spliced, as shown in Figure 2 . The surface of the studs was previously covered with plastic film, in order to prevent the bonding of the corners with the mortar.
- To prevent the mortar to drop away, 2mm-thick wood veneer stripes covered the joint sides and the bottom of the slot. These containing elements were cut with the appropriate dimensions, covered with PVC film and fixed to wood with adhesive tape.

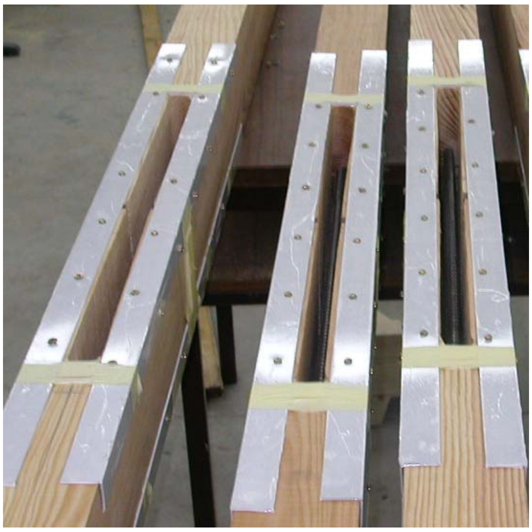


Fig. 2: *Assembly of the timber parts* ***Fig. 3:*** *Reinforcement positioning*

- The steel rods were kept in position with steel wire, inserted into transversally drilled holes. The wire was bent at an angle in order to produce a slope for keeping the rods centered through the beam width (Figure 3). This allowed the control of the mortar cover under the bottom rod and of the spacing between rods.
- The epoxy mortar is made of three components, respectively resin, hardener and inert material. In the filling operation, the mortar top level was left 3mm up above the beam surface, to compensate the lowering mainly due to the mortar absorption by timber.
- After the mortar hardening, the aluminium corner studs and the sealing side and bottom elements were removed, and the splice surfaces were smoothed (Figure 4). Should the rehabilitation system be hidden, the mortar surfaces might be covered with wood sheeting.

Fig. 4*: Rehabilitated beams*

Bending Strength of the Rehabilitation System

Bending tests

First, solid wood (*Pinus pinaster*, Ait) beams V1 to V5, 75x125mm^2 cross section, were tested to flexural failure. These beams were then rehabilitated, according to the system illustrated in Figure 1, and tested again.

Another four beams, with cross-section bxh=75x110 mm^2, were also tested incorporating alternative reinforcement materials. Two of them were reinforced with threaded stainless steel rods (V6 and V7 beams), while the other two were reinforced with fibre glass pultruded rods (V8 and V9 beams).

The tests followed the EN 408 provisions. Figure 5 shows the relevant dimensions for the loading scheme. Besides the ultimate moment, both the global and local values of MOE were measured.

The repair operation required the removal of the damaged parts. As the original beams were longer than what was strictly required to fulfil the static scheme of Figure 5, it was possible to comply with it even for the shorter rehabilitated beams.

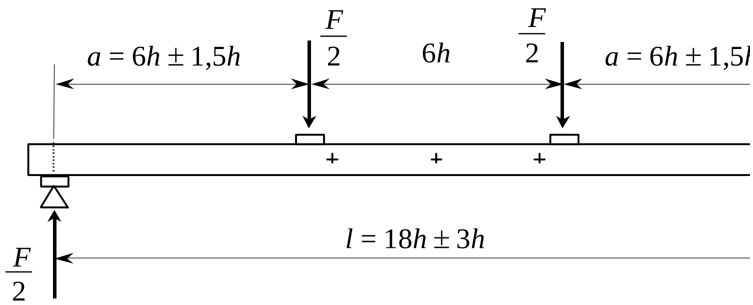


Fig. 5: *Loading scheme according to EN 408*

Test results

Figure 6 illustrates the load-midspan deflection curve for beam V1. In order to ease the visual comparison of the behaviour before and after the rehabilitation work, the curves for both the original and rehabilitated beam were superimposed.

The original solid wood beams response followed a nearly linear stress-strain behaviour up to failure, as expected.

After the rehabilitation with the A400 NR rebars, the beams exhibited elastic-plastic behaviour, with a ductile collapse caused by the yielding of the rebars in the bonding joint. This caused a plastic hinge to forme at the joint section, with significant relative rotation and the opening of a wide gap in the bottom face of it, as shown in Figure 7.

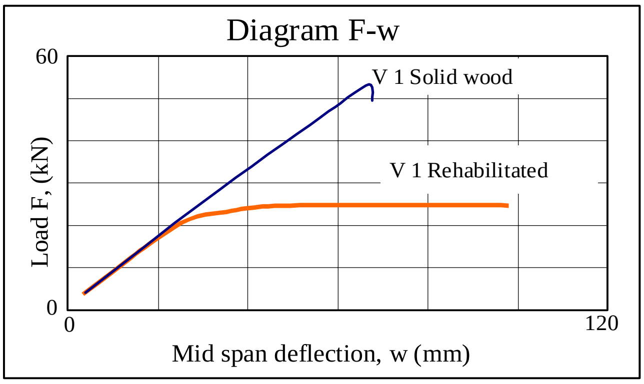


Fig. 6: *Load-deflection curves of beam V1, before and after rehabilitation.*

Fig. 7: *Failure mode with A400 NR rebars.*

The plastic failure of the reinforcement rods is typical of under-reinforced beams. It is thus possible to increase the effectiveness of the rehabilitation system through the increase of the number or size of reinforcement bars. However, excessive reinforcement strength increase may lead to the modification of the failure mode. Other possible causes of structural failure are the compression crush of the mortar or timber, the timber failure in tension in sections beyond the splice region and the bond failure between mortar and reinforcement or between mortar and timber.

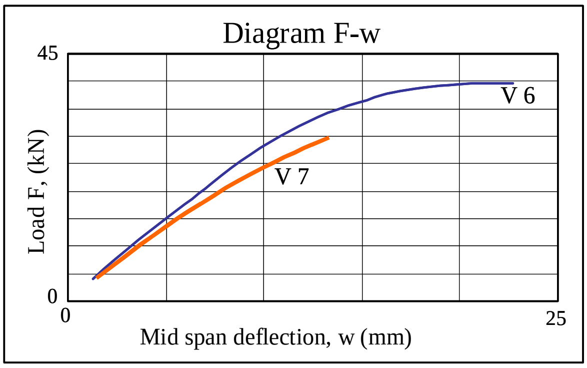


Fig. 8: *Load-deflection curves of rehabilitated beams V6 and V7.*

Fig. 9: *Failure mode for beams with stainless steel threaded rods.*

Figure 8 shows the load-deflection curves for the beams rehabilitated with stainless steel rods. This was the case with the highest reinforcement mechanical percentage, mostly due to the superior tensile strength of this material. Although the stainless steel rods show a ductile behaviour, the plasticization at the joint section, near failure, is incipient in beam V6 and not at all distinguishable in beam V7. The high reinforcement percentage increased the plasticization onset, allowing other failure modes to prevail.

In both beams, failure occurred in the tensile side, outside the splice length, and was caused by a sloping crack which propagated towards the upper face, as shown in Figure 9. The main cause of failure was thus the low timber tensile strength perpendicular to the grain.

One may see the fracture of the epoxy grout plate and the inner interface, with local bonding failure between the grout and timber. The bonding between the reinforcement and the mortar behaved satisfactorily, with the bars remaining perfectly involved by the mortar when the collapse took place.

Regarding the beams reinforced with GFRP bars, the load-deflection curve initially follows a linear path, but a significant stiffness decrease may be noticed as the ultimate load approaches (Figure 10). The GFRP bars behave elastically and linearly up to its failure, which does not favour the plasticization of the cross section of the joint. The two beams of this type behaved very similarly, as can be seen in Figure 10, where the load-deflection curves for both tests are displayed.

In both cases, failure was caused by bonding failure between the GFRP rods and the involving grout, which can be clearly seen in Figure 11.

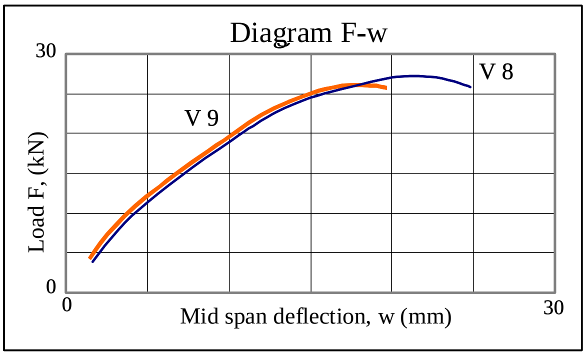
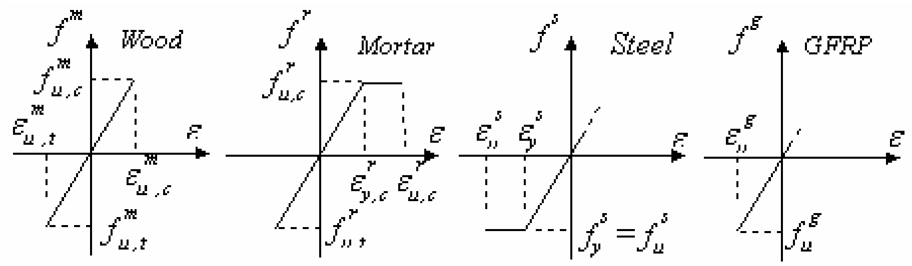


Fig. 10: *Load-deformation curves of rehabilitated V8 and V9 beams.*

Fig. 11: *Beams failure mode with GFRP reinforcement.*

Proposed Analytical Model for the Bending Behaviour

The proposed analytical model for the bending behaviour of the repair system provides an estimate of the ultimate bending moment, which can be compared to those obtained experimentally. The model is based on the static equilibrium of stresses within the cross-section and the assumption of plane sections after deformation. The constitutive relations adopted for the materials are as shown in Figure 12 and Table 1.



The mechanical properties used in the design were set through tests or, whenever available and reliable, in technical documentation of the materials. EN 408 was used as guidance for bending tests, as previously mentioned. The experimental program also included compressive tests of epoxy mortar specimens and tensile tests of GFRP and steel bars. The latter tests were made according to EN 10002.1 (Duarte, 2004).

Table 1: *Mechanical properties of the materials (mean values).*

	Mortar	*A400NR*	*St. steel*	*GFRP*	*Timber*
Elastic Modulus (MPa)	19000	213000	174000	47000	14804
Ultimate tensile strength (MPa)	10	586	962	870	90
Ultimate compres. strength (MPa)	66	-	-	-	90
Elastic limit strain ($^0/_{00}$)	3,47	2,75	5,53	18,51	6,10
Ultimate strain ($^0/_{00}$)	6,10	10,00	10,00	18,51	6,10

Results and Discussion

Table 2 lists the experimental and expected (analytical) ultimate bending moments.

Comparing the resisting bending moments before and after rehabilitation, one may see that the average value of the ultimate moment has decreased some 46% in beams rehabilitated with A400NR rebars, 36% for the stainless steels case and close to 52% in those reinforced with GFRP bars. These relations essentially depend on the reinforcement ammount, more than on the type of rods. Especially in the A400NR case, the epoxy plate is clearly under-reinforced and it is possible to increase the ultimate moment by increasing the reinforcement.

Table 2 : *Failure moments.*

Description	*Beam*	*Failure moments (kN.m)*			
		Rehabilitated beams		*Solid beams*	
		Test	*Calculus*	*Test*	*EC5* (2)
Timber beams with A400 NR steel reinforcement. Section: 75x125	V1	9,3	10,5	19,9	5,3
	V2	9,2		13,0	
	V3	9,4		22,2	
	V4	8,9		13,6	
	V5	8,2		15,1	
	Average	9,0		16,8	
Timber beams with stainless steel reinforcement. Section: 75x110	V6	9,9	10,6	13,7 (1)	4,1
	V7	7,5			
	Average	8,7			
Timber beams with GFRP reinforcement. Section: 75x110	V8	6,6	7,5	13,7 (1)	4,1
	V9	6,5			
	Average	6,6			

(1) Value extrapolated for the results of the original 75x125 beams, assuming the same maximum stress and elastic behaviour.
(2) Design value for Grade EE (NP 4305) of Maritime pine ($f_{m,k}$=35N/mm^2), according to the EC5 provisions.

Another reason for the relatively low efficiency of the repair is the very good quality of the specific solid timber beams used. This issue brings no significant benefit to the strength of the critical cross-section of the rehabilitation system, but dramatically enhances the behaviour of the original solid timber beams.

Despite the significant reduction in the resisting moment, the rehabilitation system is still feasible, because the experimental ultimate moments of the rehabilitated beams are higher than the design values of the solid timber resisting moment, evaluated with the EC5 rules. Although the number of tests was not large enough to allow for a reliable statistical analysis, it was decided to estimate the 5% value of the resisting bending moment, for the case of beams rehabilitated with A400NR rebars. The value of 8.2kNm was found and, considering a material safety factor γ_M =1,1, because the dominating failure mode is associated with reinforcement yielding, one may set the experimental value for the design moment in about 7.4kNm.

This value is directly comparable with the design moment for the solid timber section, obtained after the EC5 rules, exceeding it in about 41%. This means again that, according to the current procedures for timber structures design, the rehabilitation system is acceptable.

The failure moment of the rehabilitated beams is similar to that obtained from the proposed analytical model. For the A400NR case, the average actual value of the ultimate moment was about 85% of the value provided by the model. This difference may partially be due to the insufficient knowledge on the material's behaviour.

In the stainless steel reinforced beams that percentage was 82% and in the GFRP reinforced beams it was 88%. Some divergence is probably due to the fact that flexural failure has not been the cause of failure, which was anticipated by the combination of other causes, not considered in the model, such as the mortar fracture, the loss of adhesion between the involved materials or the tensile strength perpendicular to the grain. In beam V6, in which flexural collapse prevailed, the ultimate moment was only 7% lower than the calculated moment, showing that the proposed analysis model is in good agreement with reality, for that specific failure mode.

Conclusions

The experimental results show the effectiveness of the rehabilitation system and encourage the development of new investigations. Based in the insight from the present work, some suggestions for the improvement of the system and its strength may be drawn. One such proposal is the use of reinforcement perpendicularly to the grain, in order to delay the brittle tensile failure mode observed in Figure 9. This can be done, for instance, by using stirrups glued in transverse holes drilled in the element. Another possibility is the use of GFRP fabrics, when such solution is aesthetically acceptable.

The use of stainless steel threaded rods is a good choice comparatively to the use of A400NR rebars. The thread greatly improves the adhesion of the epoxy mortar and only a simple surface cleaning is required for the preparation, because there is no rust development. Besides, the tensile strength is much higher than that of the A400NR rebars.

Despite their high strength, the GFRP rods have a low elastic modulus and exhibit a large ultimate strain. As a consequence, considerable mortar cracking arises in the reinforcement surrounding area, due to its incapability to cope with those deformations. This fact and the low roughness of the GFRP rods surface decrease the adhesion between the two materials.

The proposed design model is a helpful mean for the conception and design of rehabilitation systems similar to the one discussed in here. The model can still be improved by a more comprehensive investigation on the materials behaviour.

The solution may also benefit from the possibility of developing a simple and cheap execution methodology, using pre-fabricated reinforcement and easy fixing elements.

Acknowledgements

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Selection of adhesives and quality control of structural repairs by bonding on site

Rodrigues, D., Custódio, J., Cruz, H.[15]

Abstract: *Bonding on site has been used successfully for many years in the repair and strengthening of timber structures. However, there is still a lack of knowledge about its service durability, especially under high service temperature conditions. Not only the performance and durability of epoxy adhesives depend on their thermal stability but also on the preparation and application procedures followed on site, as confirmed by tests conducted by the authors to evaluate the effects of mixture ratios and curing conditions. This paper discusses a number of test methods that can be used for quality control of structural repair jobs involving bonding on site. Two studies conducted at LNEC where these methods were applied are presented: one concerning a structural intervention on a decorative suspended ceiling, the other concerning the selection of adhesives suitable for high service temperature conditions as expected in some roofs. DMA and FTIR analysis results are presented and their high potential for quality control during and after adhesive application is discussed.*

1. Introduction

Epoxy adhesives have been used successfully for many years for the repair and strengthening of structures on site. These adhesives have good adhesion to a large variety of materials (wood, concrete, steel, FRP, GRP, etc.), cure at ambient temperature, don't require pressure during the cure process and exhibit low sensitivity to thickness variations of the glue line. However, because epoxy adhesives exhibit excellent initial joint strength when tested in standard climate conditions, there hasn't been a major concern about its service durability.

The lack of knowledge about the way in which fire and high service temperature affects the performance of epoxy adhesives lead recently to the development of research studies to address those concerns. Experimental and modelling work developed by the authors in previous studies clearly showed that the service temperature to which the timber structure is exposed dictates the temperature reached by the glue lines placed inside the bonded elements, despite the insulation provided by the timber cover (Cruz 2004 & 2005, Custódio 2006).

Although the heating effects on the long-term durability of glued joints require further investigation, the ("hot" tensile) tests performed so far with a range of commercial epoxy adhesives clearly show that its immediate effects may be critical for structural safety.

Selection of the adhesives for bonding on site should bear in mind the requirements of the job (viscosity, open time and close time, sensitivity to incorrect mixing or other possible mistakes) and the required performance (adhesion to the intended adherends, strength, stiffness, ductility, thermal stability).

Not only the performance and durability of adhesives depend on their intrinsic properties but also on the preparation and application procedures followed on site. On-site quality control of the intervention and regular monitoring of the joint should be made to ensure good service durability.

2. Test methods for adhesives selection and quality control

2.1 Selection of adhesives

Most commercial adhesives were developed for applications other than bonding timber. Generally they present too high stiffness and modest ductile behaviour. Moreover, information of their adhesion to timber products is not always available.

Adhesion tests are therefore required covering the specific adherends to be bonded and surface preparation methods considered. Pull-off tests are relatively easy to carry out and may help selecting the suitable adhesive and method of application.

Even when presenting working properties and strength and stiffness properties of the adhesive, rarely Product Data Sheets include quantitative information on its sensitivity to cure temperature and to service temperature – which may be highly relevant for certain applications and for certain regions.

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Important information can be gathered through direct measurement of mechanical properties at specific temperatures, the so-called “hot” tensile tests. This can be complemented with the measurement of mechanical properties at standard room temperature after cycles of relative humidity and temperature, “cold” tensile tests after artificial ageing.

Another possible approach consists in the assessment of the Glass Transition temperature of the adhesive, Tg. This property is believed to give a good indication regarding the sensitivity of the adhesive to high environment temperature during service life. Nevertheless, it is also important to know the strength variation of the adhesive throughout the expected service conditions. Thus, the correlation between Tg and mechanical properties reduction with increasing service temperature requires further analysis.

2.2. Quality control methods of on-site bonding

A satisfactory on-site bonding structural intervention requires: a well established intervention action plan; proper qualification of operatives; suitable quality control of the supply of all materials, tools and equipment considered necessary; and suitable quality control of the execution as well as of the final work.

One likely source of defects is related to the incorrect mixing and application of adhesives. Despite a number of procedures that should be implemented on site to prevent these mistakes, it was recognised useful to check this in time for necessary corrective actions to take place.

A 3-part standard on "Adhesives for on-site assembling or restoration of timber structures. On-site acceptance testing.” has been developed within the European Committee for Standardization CEN TC193/SC1/WG11. This standard covers: (Part 1, document N20) Adhesive cure characterization tests; (Part 2, doc. N21) Shear strength check of adhesive assembled parts; (Part 3, doc. N22) Pull-out resistance of glued rods.

Exothermic cure monitoring (specified in part 1 of the above standard) is probably the most interesting test for on-site quality control since it gives on-time information about the adhesive that has been applied on site. Should any problem be detected during cure, prompt corrective action can be adopted (removal, further strengthening, etc.).

The principle followed by the standard relies on the measurement of the adhesive’s cure schedule, by the exothermic rise in temperature and peak temperature reached by a specified adhesive volume kept under insulated conditions. This method is used to evaluate adhesive mixtures and the results are closely related to the local ambient conditions, especially the temperature. The adhesive manufacturer should therefore provide reference exothermic curves for a range of ambient temperatures.

A possible alternative could be again the Dynamic Mechanical Analysis (DMA) technique, which is also believed to be a helpful tool to detect mixture or cure temperature problems, since mix ratio and cure temperature influences the adhesive glass transition temperature and its storage modulus.

2.3. Quality control of bonded structures in use

When an adhesive fails to meet the requirements or when its performance, in the short or the long run, is questioned, it is possible to determine whether or not the chemical composition of the cured adhesive corresponds to the designed one and then investigate the source of any detected alteration.

Chemical analysis of small samples of the adhesive can be a less disturbing alternative to collecting larger samples from the bonded joints for shear testing, or to proof-loading the bonded structural member.

The molecular structure of polymeric materials can change over time, as they are mixed, cured or exposed to heat, moisture or radiation. Fourier Transform Infrared Spectroscopy (FTIR) is thought to easily differentiate the composition of the epoxy adhesives, analyse the hardening process after mixing the two components, identify and measure the nature of the changes in the functional groups.

3. Case studies

3.1. Choosing the right adhesive for a ceiling repair

A decorative/acoustic suspended ceiling made of plywood panels nailed to a solid timber structure was found to require extensive repair due to insect attack of many solid timber pieces. This was the ceiling of a small concert hall, situated high above the amphitheatre.

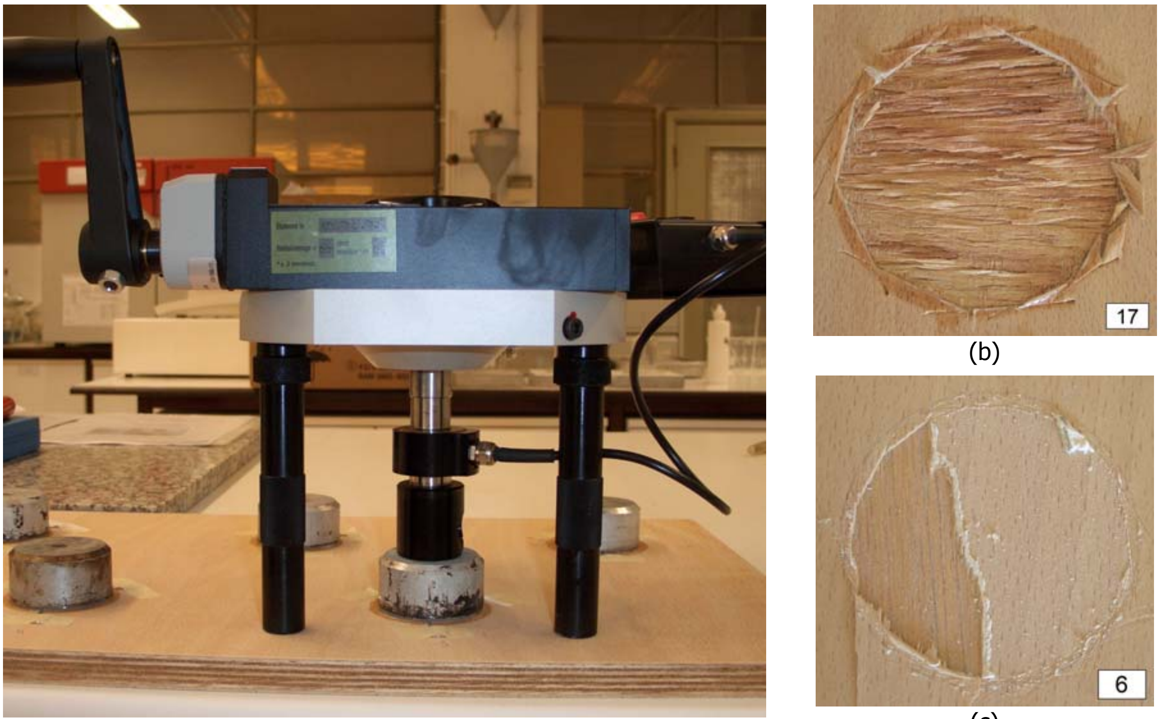
The first considered repair plans involved propping of the whole structure and replacement of every damaged solid timber piece with a new treated one; this approach was considered too time consuming and expensive. An alternative approach was therefore considered, and this consisted in leaving the whole ceiling in place by creating a new redundant suspension system consisting of metal plates bonded to the upper face of the plywood panels and suspended by steel cables from the concrete ceiling above.

Identifying an adhesive suitable for the job and an efficient but simple cleaning method for the dusty plywood, as well as obtaining information on the bond strength was therefore necessary.

– pull-off tests

Pull-of tests (EN ISO 4624:2003) were carried out in laboratory environment (Figure 1a), involving a number of adhesives (one epoxy, two polyurethane) and cleaning methods (dust removal + washing with a mild detergent solution, with and sand paper abrasion in between). New plywood similar to the one applied in the ceiling was used for these tests. Finally, a sample of plywood collected from the ceiling was also used for confirmation of pull-off test results.

Tests on the new plywood indicated that the sand paper operation did not increase adhesion, and could be dropped in this case. Furthermore, the epoxy adhesive gave the strongest bond and the highest wood failure percentage inside the plywood (Figure 1b), whereas with the PU adhesives most failures were either cohesive-type in the adhesive or adhesive-type at the interface between the decorative wood veneer and the core plywood (Figure 1c). The epoxy adhesive was therefore adopted based on pull-off test results since it conducted to a safer type of failure.



(a) (b) (c)

Fig. 1: *(a) Portable pull-off test equipment; (b) Plywood after pull-off test with epoxy adhesive; (c) Plywood after pull-off test with polyurethane adhesive.*

3.2. Evaluating thermal stability of adhesives for roof repairs

Many commercial epoxy adhesives show significant strength and stiffness decrease when exposed to relatively low temperature. The reason for this behaviour is related to the fact that adhesives solidify without crystallizing, presenting in normal conditions a disordered solid structure (glass state). The increase of the temperature results in a transition from a non-crystalline solid phase to a liquid phase (glass transition temperature). This phenomenon occurs also in the case of epoxy polymers at temperatures in the range of service temperatures expected in summer). The transition reflects macroscopically through changes in the adhesive mechanical, dielectric and viscoelastic properties (Yamaki, 2001).

– hot tensile tests

A comprehensive research program was set up at LNEC intended to study a number of commercial adhesives and bonding products having in consideration the required properties for structural repair of floor and roof structures. The adhesives must cure at room temperature without pressure, have low sensitivity to the adhesive layer thickness, high mechanical resistance, ductility, durability and good adherence to various materials (wood, steel, FRPs and concrete). Besides this, these adhesives should have little sensitivity to high service temperature, since temperatures in excess of 40ºC are anticipated in roof structures. The selection was made thought literature review and direct enquiries to national and international structural adhesives manufacturers, focusing mainly in epoxy and polyurethane polymers base products (Table 1).

Table 1. *Selected adhesives.*

Manufacturer	***Adhesive***	***Polymer***	***No. Components***	***Mix***
3M	5200	Polyurethane	1	-
Degussa	Mbrace HT65	Epoxy	2	Hand
Mapei	Mapewood Gel 120	Epoxy	2	
	Mapewood Gel 140	Epoxy	2	
Matesica	Soldepox	Epoxy	2	
Rotafix	CB10T	Epoxy	2	
	Timberset	Epoxy	2	
	TG6	Epoxy	2	
Tecnocrete	Stapox AS	Epoxy	2	
SIKA	Sikadur 31	Epoxy	2	
	Sikadur 32N	Epoxy	2	
	Sikadur 42	Epoxy	2	
SP Systems	SP 106	Epoxy	2	

The use of the adhesives was done after thoroughly mixing the resin/hardener together. The ratio amount of resin/hardener indicated on the Product data sheet was meticulously respected. The thermal stability of the adhesives included in table 1 was evaluated through "hot" tensile tests following European standard EN ISO 527-2: 1996. Cured test specimens (dumb-bell-shaped type 1B) were subjected to tensile tests carried out in a range of temperatures (23, 30, 40, 50 and 60 ºC). The specimen is exposed to the test temperature for 10 minutes prior to being tested in tension inside the temperature chamber. Six tests specimens (replicates) are considered for each adhesive and test temperature. The specimens were prepared at 20±2ºC, 65±5% RH and cured at 23±2ºC, 50±5% RH.

Figure 2 and table 2 present some test results. The commercial identification of each adhesive has been omitted.

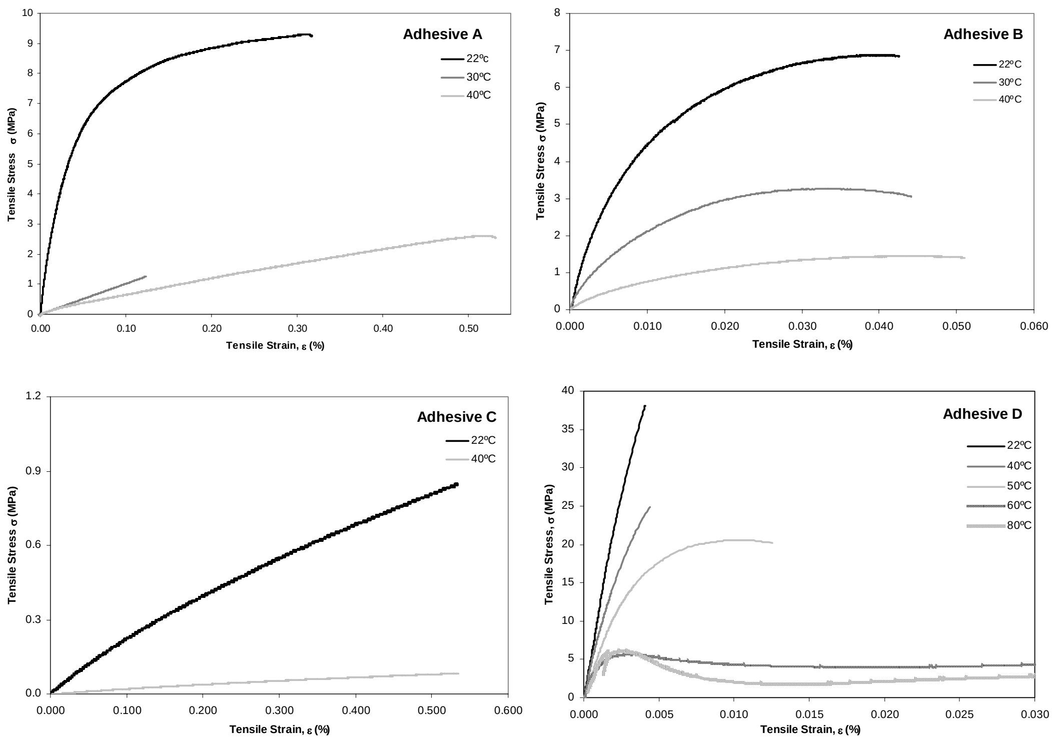


Fig. 2: *Results of the "hot" tensile test obtained for 3 epoxy adhesives (adhesives A, B and D) and for 1 polyurethane adhesive (adhesive C).*

Table 2. *Young modulus and ultimate tensile strength (UTS) of various adhesives at different temperatures.*

Adhesive	***Test Temperature (ºC)***	***Young Modulus (MPa)***	***UTS (MPa)***
A	22	295.5	9.3
	30	10.1	-
	40	26.6	2.6
B	22	753.3	6.9
	30	305.6	3.3
	40	194.2	1.4
C	22	3.3	-
	40	0.7	-
D	22	10442.1	38.1
	40	6460.0	24.9
	50	6311.4 / 4688.4	26.2 / 20.6
	60	1295.6	5.7
	80	1318.1	6.1

Figure 2 and Table 2 show that some adhesives that are commonly used for structural repair show a perceptible decrease of mechanical properties at relatively low temperatures (between 20ºC and 40ºC). This behavior may be related to the adhesive Tg being close to the test temperatures.

3.3. Checking adhesive sensitivity to mixing mistakes and cure conditions

– Adhesive's cure schedule monitoring

The exotherm depends on the environment temperature and the mix ratio. Two adhesives were prepared with different component mix ratios at various cure conditions (Table 3) and the corresponding cure schedule was accessed according to TC193/SC1/WG11 - Part 1.

Table 3. *Adhesive mix ratios and cure conditions.*

Mixture Ratio	*Cure Conditions*		
	17ºC – 70% RH	*23ºC – 50%RH*	*31ºC – 32%RH*
-20% hardener		●	
-10% hardener		**x** ●	**x**
Prescribed	●	**x** ●	**x** ●
+10% hardener		**x** ●	**x**
+20% hardener		●	

x – Adhesive D ● – Adhesive E

Immediately after mixture, three 20 cm^3 plastic capsules located in an adequate thermal isolating container were filled with adhesive. The cure was monitored through a portable K-thermocouple-type measuring device using an adjustable data logger and evaluated on site with reference to the Product Data Sheet. The room temperature was also recorded and the cure exothermic curves obtained are presented in Figure 3.

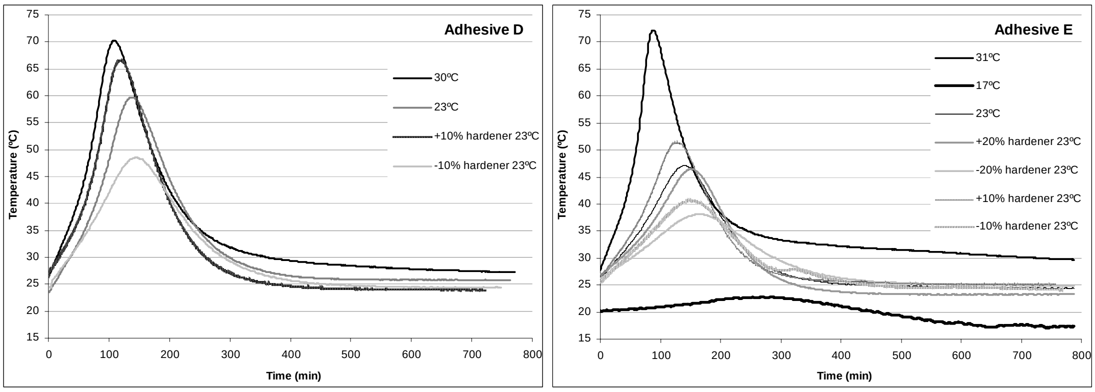


Fig. 3: *Exothermic profiles during cure process for Adhesive D and Adhesive E.*

The adhesives with higher cure temperatures and/or upper hardener quantity not only present a faster cure process but also a higher exothermic maximum temperature. From the two, adhesive D presents a larger variation of the exothermic maximum temperature due to mix ratio variation. On the other hand, the adhesive E presents higher variation of exothermic curves due to the cure temperature.

From the results presented the exothermic curves appear to be a good indicator of the adhesive mix ratio and the effectiveness of the mix process. However, it is also important, to assess the consequences of mix ratio problems and cure temperature on the adhesive mechanical performance.

– DMA analysis

The properties of adhesives are believed to depend on the hardener to resin ratio. Variation in this ratio introduces off-stoichiometric mixtures and this may lead to variation on its mechanical behaviour. In order to investigate this effect, adhesive D was prepared with various mix ratios (-10%, +10%, -30% and +30% of hardener) at two cure temperatures (20ºC and 30ºC).

The Tg for the various adhesives was obtained by DMA analysis, using a TA Instruments Dynamic Mechanical Analyser DMA Q800 V7.0 Build 113. The storage modulus, G□, and tan□ against the scanning temperature were obtained at a heating rate of 2ºC/min from ambient temperature to 120ºC, at a frequency of 1 Hz and amplitude of 15□m. Three samples (replicates) of cured adhesives 6x10x4mm^3 were used for each mix ratio and cure conditions. The test was performed in the 3-point bending mode (3PB), with a preload force of 0,2N and a force track of 150%. DMA results are presented in table 4.

Table 4. *Glass transition temperature for adhesive D, with different mix ratios and cure temperatures.*

Cure Temperature (ºC)	***Mixture Ratio***	***Glass Transition Temperature (ºC)***	
		Average	***Standard Deviation***
20	-10% hardener	50.1	0.12
	Prescribed	49.4	0.21
	+10% hardener	50.8	0.43
30	-10% hardener	50.7	0.39
	Prescribed	52.7	0.13
	+10% hardener	53.0	0.27

From table 4, as expected, it is observed that the Tg tends to increase with the hardener quantity and cure temperatures.

The adhesive storage modulus for different mix ratios and cure conditions were also obtained at 20ºC, 30ºC and 40ºC (Table 5). These are closely related to the MoE in bending.

Table 5. *Storage modulus for adhesive D at various temperatures, with different mix ratios and cure temperatures.*

Cure Temperature (ºC)	***Mixture Ratio***	***Storage Modulus (MPa)***		
		20ºC	***30ºC***	***40ºC***
20	-10% hardener	7546.83	6437.52	3369.47
	Prescribed	7380.01	6085.46	2721.41
	+10% hardener	7546.32	6618.69	4112.22
30	-10% hardener	8230.61	6875.58	3494.07
	Prescribed	7498.75	6514.43	4002.60
	+10% hardener	8182.53	7599.30	5623.05

Again, as expected, the adhesive storage modulus decreases with increasing temperature. In addition, the adhesives cured at 30ºC present higher storage modulus than the ones cured at 20ºC. The ones that present a smaller and higher decrease of the storage modulus are, respectively, the adhesives with +10% hardener and with -10% hardener. Being these last results in line with the behaviour observed in table 4.

Therefore, it is possible to use this method to control an incorrect mix ratio, to determine improper mixing, and to assure batch-to-batch uniformity.

Moreover, there is a good correlation between faster cure and higher exothermic maximum temperature (both assessed by exothermic monitoring) and higher Tg and storage modulus assessed by DMA analysis.

– FTIR analysis

To access the usefulness of FTIR as a quality control technique, adhesive D was prepared with various mix ratios at two cure temperatures (Table 5) and then analysed by FTIR.

The spectras were acquired with a Nicolet model Magna-IR 550 Series II Fourier-transform IR spectrophotometer equipped with IR source, KBr beamsplitter, and deuterated triglycine sulfate (DTGS) detector. Each spectra was obtained in the transmission mode and represents 32 co-added scans at a spectral resolution of 4 cm^{-1}, with Happ-Genzel apodization and Mertz phase correction. The samples were prepared in KBr pellets with a mix ratio of approximately 1:100 (w/w), of cured adhesive and potassium bromide. The mixture (100mg) was pressed to 10 tons into a pellet with a thickness of about 1mm. The obtained spectras were normalized with the infrared absorption band at 1084 cm^{-1} (Figure 4).

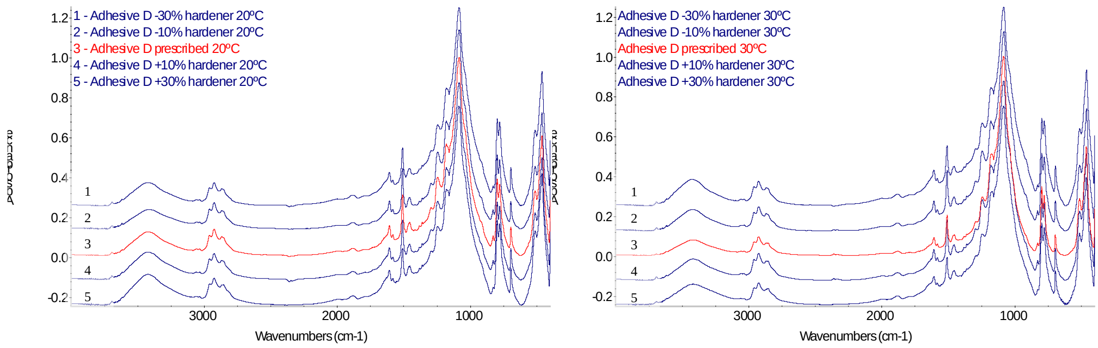


Fig. 4: *FTIR Spectras for cured adhesive at 20ºC and 30ºC with different component mix ratios.*

Typical characteristic infrared bands of epoxy polymers can easily be identified. The variation of the bands intensity for the several adhesive mix ratios indicates differences in the components concentrations, which allows using this method for differentiating different adhesive mix ratios.

4. Conclusions

The performance and durability of epoxy adhesives depend on their intrinsic thermal stability (formulation) but also on the preparation and application procedures followed on site, as confirmed by tests conducted by the authors to evaluate the effects of mixture ratios and cure temperature.

Quality control is essential in all phases of the work and people involved should be aware of the test methods available and their possibilities.

Chemical analysis like DMA and FTIR show a high potential for selecting adhesives on the base of their higher or lower Tg, for checking if proper mixture and cure took place and later for assessing possible ageing effects on the bonded joint.

When considering joint strengthening with structural adhesives, careful selection of the adhesive, adequate design considerations, on-site quality control of the intervention, and regular monitoring of the joint should be made to ensure good service durability. These procedures will increase the confidence in on-site bonding techniques for the repair and reinforcement of timber structures.

Acknowledgements

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European Standard CEN TC193/SC1/WG11 "Adhesives for on-site assembling or restoration of timber structures - Part 3: Verification of the adhesive bond strength using tensile proof-loading." (document N22).

Timber-concrete-composite with adhesive interface

Brunner, M.[1], **Schnüriger, M.**[2]

Abstract: *An adhesive connection is proposed for timber-concrete-composite structures. The first project phase was concerned with the optimization of manufacturing parameters in order to ensure a good and stable bond. Many different timber-concrete-composite elements of large size were investigated. The bending tests fully met the expectations. Adhesive creep was not a problem.*

Introduction

Timber-concrete-composite structures are traditionally connected mechanically with screws, bolts or concrete indentations into the wood. An adhesive connection is also possible: it would be slip-free with an even distribution of the shear forces over the interface and no force concentrations.

Between 1998 and 2000, a feasibility study was made at Berner Fachhochschule. In 2004, a follow-up COST C12-project was initiated with the cooperation of industrial partners who supplied the test specimens and helped to ensure that practical parameters were studied. The project comprised 3 phases:

1. Optimization of the manufacturing process to reduce dangers of a displacement of the wet adhesive during the pouring of the concrete.
2. The optimised manufacturing process from the 1st project phase was used to prepare large-scale timber-concrete-composite slabs of different dimensions. The reliability of the adhesive bond in cases of reduced contact area was also tested.
3. Study of the long-term behaviour of timber-concrete-composite slabs: possible influence of the adhesive on the deflections due to creep or shrinkage.

Process optimization for a reliable adhesive bond

Parameters und Materials

The possible displacement of the wet adhesive during the pouring of the concrete was investigated with many parameters. The materials used to manufacture the test specimens were:

- Timber: 3-ply wooden slab, quality C24
- Concrete types:
    - Normal concrete (grade C25/30)
    - SCC (self compacting concrete, C30/37)
- Adhesive: 2-component, epoxy-based, SIKA AG

The test specimens were produced with the following constant parameters:

- dropping height of the wet concrete (50 cm)
- amount of adhesive: 925 g/m^2
- 3-ply timber slab, 30 mm

The following parameters were variables:

- Concrete quality ("normal" and SCC with no vibrators)
- Time interval between mixing adhesive and pouring the concrete: 15 and 90 minutes
- Concrete thickness: 8 cm, 16 cm or 24 cm

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[2] Creabeton Matériaux, Müntschemier, Switzerland.

Timber-concrete-composite slabs 30 cm wide and 90 cm long were manufactured. Figs. 1 and 2 show the manufacturing process. The timber was fixed in the steel formwork. The adhesive - coloured red for a better visualization - was mixed and then applied on the surface of the timber. After a suitable time interval the fresh concrete was poured onto the adhesive. Normal concrete was compacted with a vibrator which was carefully prevented from touching the adhesive. Fig. 3 shows the cutting of the hardened timber-concrete-composite slabs into the different test specimens shown in Fig. 4.

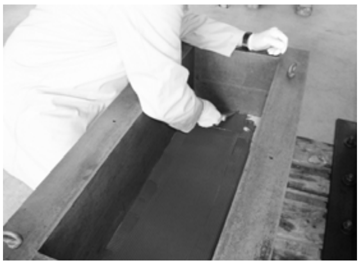


Fig. 1: *Application of adhesive on timber at the base of the steel formwork*

Fig. 2: *Pouring of concrete (SCC in this case) on the wet adhesive.*

Fig. 3: *Cutting specimens: inspection, tests*

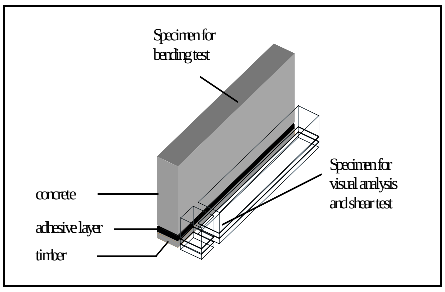


Fig. 4*: Scheme for cutting the specimens.*

Visual analysis of prism sections

Sections from different longitudinal segments of the specimen were inspected for signs of some disturbance of the adhesive. The results of the visual analysis are:

- The adhesive was markedly displaced only at the points where the concrete was poured in.
- The fine mixing of the adhesive with the concrete was more pronounced when SCC was used. In the case of conventional concrete, apart from some local protruding into the concrete, the adhesive stayed in place (Fig. 5).
- The time interval between the mixing of the adhesive and the pouring of the concrete seemed to be a key factor: the mixing of the adhesive with the concrete was markedly reduced when the time interval was increased from 15 to 90 minutes.

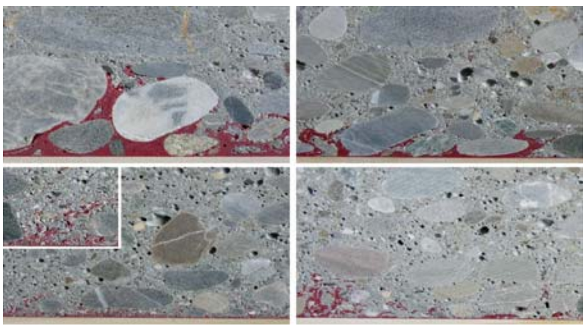


Fig. 5: *Cross section of timber-concrete-composite slabs. Top: conventional concrete; bottom: SCC. Time interval between mixing of adhesive and pouring of concrete: 15 minutes (left) and 90 minutes (right).*

Loading tests with small specimens

Four-point-bending tests were carried out in accordance with EN 12390-5. Four specimens each with concrete thickness of 8 cm and 16 cm were tested. The load and the corresponding deflection at midspan were measured. The tests results varied greatly: most of the specimens failed in shear. Some local displacement of the adhesive at the point where the fresh concrete had been poured in was observed in some of the SCC test specimens.

Shear tests were performed in accordance with the European standard EN 392 [11]. 5 specimens each with conventional concrete and SCC were cut and prepared. The contact area between concrete and timber was 100mm wide and 40mm high. The following parameters were fixed: 16cm concrete layer, 90 minutes time interval between mixing the adhesives and pouring the concrete. A comparison of the test results shows that the shear resistance was higher and the variation was also much less when conventional concrete as opposed to SCC was used (Table 1). The inspection also showed that SCC specimens suffered more from adhesion failure: when conventional concrete was used, most of the shearing lines ran through the concrete.

Table 1 *- Results of the shear tests*

Concrete type	*Shear stress [N/mm^2]*		*Timber fails [%]*	*Adhesive fails [%]*	*Concrete fails [%]*
C25/30	Values (min. –max.)	2,67 – 3,42	5	45	50
	Average	2,97			
	Standard Deviation	0,30			
SCC	Values (min. – max.)	1,30 – 3,42	0	80	20
	Average	2,10			
	Standard Deviation	0,88			

Bending tests with large-scale specimens

Materials used

Based on the parameter study of the manufacturing process, the following materials and parameters were chosen to produce large-scale specimens for bending tests:

- Concrete: C25/30 according to SIA 262 [9]
- 90 minutes interval between mixing of adhesive and pouring of concrete
- Careful pouring of concrete, gentle use of vibrators
- Timber: C24 rectangular wood, glued side by side to form a slab, according to SIA 265 [8]
- Adhesive: 2-component epoxy adhesive produced by SIKA AG

Bending tests on slabs with full bonding area

Four-point-bending tests were carried out in accordance with the standard EN 408 [12]. Three specimens were tested for each of the four series with the different parameters with regard to the thickness of the timber and the concrete (Fig. 6).

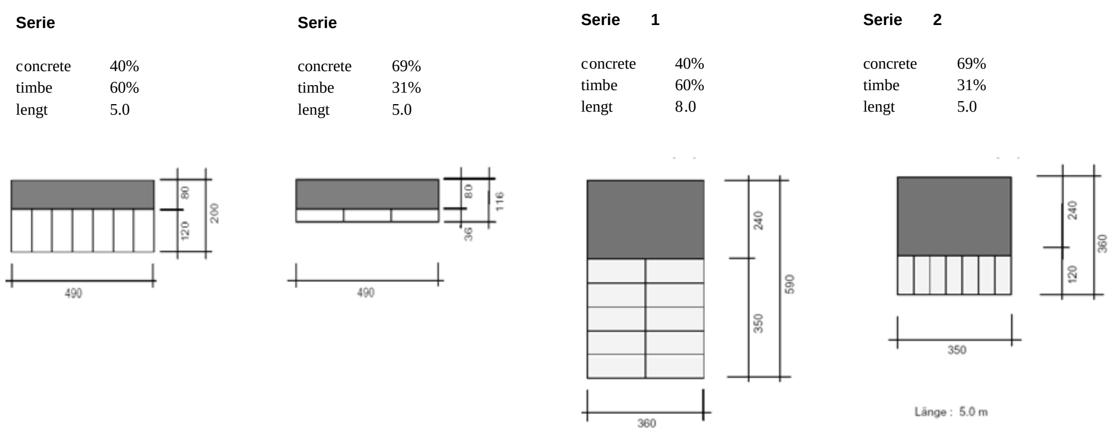


Fig. 6 *– Dimensions of large scale timber-concrete-composite slabs for the bending tests.*

For the calculation, the 5% fractile values listed in the Swiss standards SIA 262 (concrete) and 265 (timber) were multiplied by 1,33 to obtain the average failure values expected in the tests. The test results met all expectations and are summarized in Table 2:

Table 2 *- Results of the bending tests: specimen with complete bonding area*

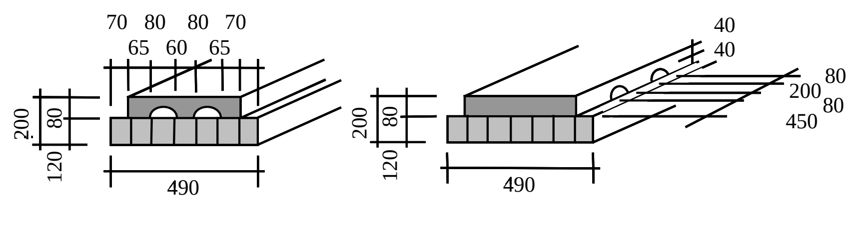
Series	*Failure loads - maximum moment*				*failure modes*
		calculated [kNm]	*measured [kNm]*	*mes./calc [%]*	
111	Values		99,3-102,0		Concrete in compression
	Average	108,0	101,03	- 6,4	
	Standard dev./COV [%]		1,50 [1,5%]		
112	Values		30,2-33,6		Timber in tension
	Average	30,0	31,90	+ 6,3	
	Standard dev./COV [%]		1,70 [5,3%]		
121	Values		378,1-497,9		Shear
	Average	525,5	444,84	- 15,3	
	Standard dev./COV [%]		61,07 [13,7%]		
122	Values		191,1-213,2		Shear
	Average	183,0	201,07	+ 9,8	
	Standard dev./COV [%]		11,21 [5,6%]		

- No horizontal displacement of the concrete relative to the timber could be measured, confirming the assumption that the adhesive connection was rigid.
- An evaluation of the load-deflection lines confirmed that the measured stiffness EI of the slab corresponded quite well to the calculated value assuming a rigid connection.
- In all test series the specimens failed quite as predicted by the calculation models, whether it was a case of shear failure, failure of timber in tension or concrete in compression.
- In test series 111, 112 and 122 the measured failure loads of the different specimens exhibited little variation and corresponded to the calculated values.
- In the 3rd test series 121, no specimens attained the predicted failure load. An inspection indicated that the local displacement of the adhesive was rather extensive in places.

Shrinkage of concrete was clearly recognizable in the test series 121 und 122, where the concrete layer was 240 mm thick. The corresponding specimens exhibited cracks at the beam ends ranging approximately 100 mm into the timber-concrete interface. The maximum crack opening was 2,5 mm. At the sides of the beams the cracks were visible for approximately 50 mm towards the midsection of the beam. Curiously, the cracks did not seem to adversely affect the load-bearing capacity: in the test series 122 in particular, the measured loading capacity corresponded quite well to the calculated values. The lower values for the test series 121 may have been due more to the greater adhesive displacement mentioned above.

Bending tests on slabs with reduced bonding area

Two series of bending tests were carried out on 2 slabs each with a reduced bonding area. The full contact area between the concrete and the timber was avoided with the help of PVC half-pipes running parallel and perpendicular to the beam length (Fig. 7). The voids thus obtained could be used to place pipes and conduits in the floors of practical buildings. The bending tests (Fig. 8 and 9) were carried out in accordance with the European standard EN 408 (four-point-loading) [12].



Series PL: Pipe Longitudinal *Series PT: Pipe Transverse*

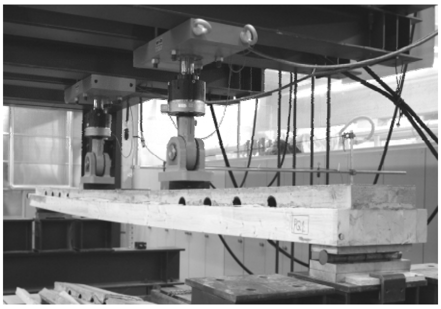


Fig. 8: *Bending test set-up with a timber-concrete-composite slab.*

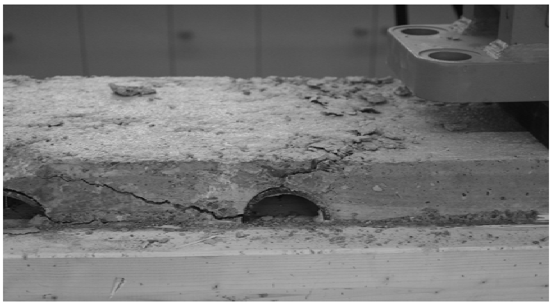


Fig. 9: *The test specimens of the test series PT failed on combined shear and compression in the concrete induced by the load transmission over the voids.*

The test results met the expectations and are summarized in Table 3:

- In all test series, the failure mode of the test specimens corresponded to the predictions of the calculation models.
- Test series PL: The test specimens all failed in compression of the concrete layer. The calculations suggested that a further reduction of the glued area could have been carried out before the shear stresses in the reduced contact area would have led to a premature failure of the concrete in shear.
- Test series PT: Failure occurred in the concrete, partly due to shear, and partly due to compressive stresses caused by the load transmission to bypass the voids in the concrete. The position and the size of the voids have an influence on the load-bearing capacity, since they influence the load-transmission possibilities.
- For comparison purposes, a theoretical cross section with the same dimensions and materials but without voids was calculated as a fictitious test series 111_{red}. The implication is clear that the longitudinal pipes do not result in a high strength reduction as compared to the solid concrete slab. However, the load transmission problems over transversely placed pipes may lead to a greater reduction in the load-bearing capacity.

Table 3 *- Results of the bending tests: specimen with reduced bonding area*

Series	*Failure loads - maximum moment*				*failure modes*
		calculated [N/mm^2]	*measured [N/mm^2]*	*calc./mes. [%]*	
111_{red}	calculated values from 111 and PT	83,0	-	-	concrete in compression
PL	Values		79,4-81,5		concrete in compression
	Average	81,0	80,36	- 0,7	
	Standard dev./COV [%]		1,06 [1,3%]		
	compared to 111_{red}		96,8%		
PT	Values		72,6-76,1		concrete in compression and shear
	Average	64,0	74,82	+ 16,9	
	Standard dev./COV [%]		70,4 [2,6%]		
	compared to 111_{red}		90,1%		

Long-term tests

Two elements with longitudinal and transverse pipes in the concrete and of the same dimensions as described in fig. 7 were selected for the long-term tests. They were loaded with 30kN - which corresponds to 30% of the failure load - applied at two points at a third of the span.

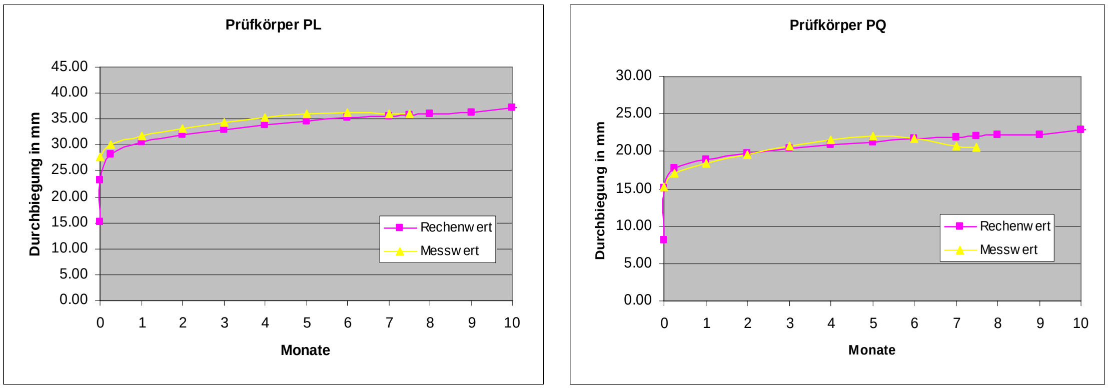


***Fig. 10**: Good agreement between the measured and calculated deflections for the specimen PL (fig. left) and PT (fig. right) during a period of 7 months.*

The test specimens have been under observation since 2000. The measured deflections during the first few months have been compared to the calculated values in fig. 10 above: for both elements there is quite good agreement. Thus the adhesive bond remains quite stable over time and does not exhibit significant creep.

Conclusions

In the 1st project phase, the manufacturing process could be optimised. The risk of displacements of the adhesive could be minimized when the following parameters were used:

- Adhesive: 2-component epoxy-based adhesive (SIKA Product, 925 g/m2), carefully applied over the horizontal surface of the timber slab.
- Time interval between mixing adhesive and pouring concrete: 90 minutes (± 20 minutes). This recommendation is valid for "normal" conditions (20°C, 50-60% relative air humidity) because the optimization tests took place under these conditions. There are as yet no test results for other temperature and humidity conditions.
- In order to avoid the risk of adhesive displacement, normal concrete should be carefully poured at different places and gently vibrated.

In the 2nd project phase, large specimens were tested in bending to check the usefulness of the manufacturing parameters chosen in the 1st phase. The results for composite slabs with a complete and a reduced bonding area agreed very well with the values predicted with calculation models. They can be summarised as follows:

- Signs of local displacement of the adhesive always occurred at the points where the concrete was poured in, but this seemed to have no significant influence on the load-bearing capacity.
- Shrinkage of the concrete led to horizontal cracks at the beam ends only in the specimens with a very thick concrete layer (240mm). However, these cracks did not seem to have a negative influence on the load-bearing behaviour. It is recommended that the concrete layer should be limited in thickness to 200mm.
- The test results have confirmed the theoretical assumption of a rigid connection between the concrete and timber. The calculation and design of the elements can be done with the usual calculation models for a composite section with a rigid connection.
- Our investigations have revealed that even under controlled, almost laboratory-like conditions, the production of wet-in-wet timber-concrete-composite elements is still quite delicate because of the danger of adhesive displacement during the pouring of the concrete. It is therefore too early to permit the use on site, where quality control may be more difficult.

The 3rd project phase was concerned with the long-term behaviour of timber-concrete-composites with the adhesive connection. Of particular interest was the possible influence which the rheological properties of the adhesive might have on the long-term deflections. The measured deflections agreed well with theoretical values based on known calculation models such as [13-16]: the adhesive does not suffer significant creep.

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European timber trade analysis: An economical overview and regional market potential

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Abstract: *The analysis of internal and external European timber trade is the purpose of this survey, in order to provide a precious tool in the decision-making process for the European enterprises of the timber sector; particularly those that import raw material of sawnwood (furniture sector, building activities etc), but also those that process round wood and export their products or think about exports. The analysis can also contribute to the determination of suitable policies of the member - countries of EU for the above - mentioned sector. The analysis of a 39 year time series (1964-2002), the creation of indicators, and the determination of forecasting models for the imports, exports, production, apparent consumption and sawnwood self-sufficiency indicators for the EU25, result that the above variables show augmentative tendencies in volume, a reduction of product price at 35.6%, for the last decade and a self-sufficiency indicator of 93%, while countries such as Finland, Sweden, Latvia, Lithuania, Estonia, Czech Republic, Slovakia and Denmark have an overabundance of sawnwood and can develop their comparative advantage for the conquest of new global markets.*

Introduction

The study and the analysis of the existing situation of timber product trade in the European Union constitutes an important decision-making tool for the timber sector enterprises. The shrinkage of the timber sector in all Europe, as well as Greece and the intensely competitive market, at a global level, impose the knowledge and spread of research results about the perspectives (opportunities, dangers, threats) that curve the future of enterprises and the possibility of their adaptation to the exterior environment (Bolkesjo and Buongiorno 2006, Shogren 2007). Indicatively, we report the example of China's entry in WTO with an increase of 25.6% in timber products imports, at a value of US$ 446.2 million, after 2003 (Gan 2004). China is considered to be a target – market for countries with timber overabundance, such as the Scandinavian countries, Estonia, Latvia, Czech Republic etc. The information about the demand and offer of various products of timber, as well as the forecast for their future are often capable, alone, to limit the risk of enterprises to some important degree.

Furthermore, the knowledge and the possession of criteria for the choice of various timber products and new markets in various geographic exports regions, the international experience, the market orientation and the planning of particular marketing strategies in industrial markets curve new directions for wood enterprises.

Within the E.U. policy framework, both the trade and the environment constitute the central subject of multifaceted commercial negotiations, after the completion of Uruguay Round/ and the organisation of World Organism of Trade (Blake et all, 1999).

In general, if we analyze the existing situation of timber production in the developed countries worldwide, we will realise that their economic derotation that is forecasted for next years, will cause the increase of their needs in timber and its products. Besides, as long as the entry of Canada and the countries of former USSR is shifted to the most inaccessible regions of their forests, the cost of coniferous exported timber production will arise and consequently its prices. The already intense demand for tropical timber imports, from the tropical forests of underdeveloped countries, will intensify still more (Stamou 2005).

The tendencies and the prospects of European timber for the 21st century are described in a particularly important and recent research (UNECE and FAO, 2004), in which the mainer ascertainments are focused in the following: a) The demand for the forest products in Europe will be continuously, regularly increasing for the next 15 years at least, with a given rate of GNP increase, roughly at 1-2% annually, b) European production of forestal products will increase up to 2020, in constant real prices and expenses, by a 25-35% for the sawnwood. c) The degree of self-sufficiency in products of timber in Europe will approach 100, provided that: 1st) the timber from the European forests is competitive, regarding price and quality, compared to the main competitors in the world markets and 2nd) the industries that will process this raw material must also be competitive at a world scale, that is to say, big units, with sufficient invested capital and strict production cost controls; and d) markets of sawnwood production and sales are very likely to be rather different in 2020 from the today ones, mainly because of the application of technological innovations, which will aim at the reduction of production cost, as well as the improvement of products' quality and the development of new uses and markets. There will appear new production processes, new composite materials, new glues and surface finishings, etc.

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The removal of restrictions for the release of trade in the European Union has contributed considerably to the economic growth of certain countries, among which is also Greece, and this action strengthened their position in the global free economy.

The EU produces the 25% of world production of relative products. Nevertheless, for a consumption of 300 million of m³ of round wood as raw material, the EU imports 20 to 25 million of m³, mainly from Russia (coniferous and non coniferous), the CEECs (coniferous and non coniferous), Northern America (non coniferous) and the southern hemisphere (coniferous and non coniferous) (FBI 2006).

The value of imports and exports of sawnwood at a world level for 2004 amounts to 30 billions $ roughly for a quantity of 90 millions m³ (UNECE and FAO 2006). Generally, the EU has a negative commercial balance for timber products and imports sawnwood of non coniferous from Northern America and panel wood – based types from South-eastern Asia. Recent increases inside the EU have begun to decrease these flows, e.g. the important export of plywood from the EU to the USA.

The production and the trade of European forestal products accelerated in 2000 as a result of the powerful requirement within the regions of EU / EFTA, that increases the consumption in the CEECs and the increasing demand from exterior Europe. Nevertheless, the forecasts for 2001 were for a deceleration in a lot of markets, which were followed by the general improvement in 2002 ((UNECE and FAO, 2004).

The EU is the biggest trader and the second biggest consumer of forest products in the world, with a positive trade balance overall. However, within this context, the EU is a net importer of raw materials, mainly round wood which comes mostly from the CEECs and CIS.

The effects of globalisation on trade in the EU F-BI can be seen in the increased worldwide procurement of raw materials, such as wood and pulp, as well as in growing worldwide trade in forest-based products and technology products needed by the forest industry.

New low-cost competitors from Asia, Latin America and the CEECs constantly challenge the industry in the EU. Asia showed the highest growth in new investment outside the EU during the 1990's, which has resulted in a loss of market share of the EU F-BI in certain sectors, such as in the wood-based panels industry (F-BI 2007).

Regarding Greece, the general course of Greek exports of timber sector was not particularly good the last decade, as the export extroversion indicator of the Greek economy (the value of Greek exports as percentage of nominal GNP), follows a declining course from 1995 and afterwards (from 10.5% to 7.6%) (SEVE et all 2004).

The mainer countries –recipients of Greek exports in timber industry, in value, are 12. The 5 most important ones at a declining order are: Albania, Cyprus, Holland, Lebanon and Italy. It is particularly important to notice the percentage increase of the sector's export activity in 2002 to countries such as Holland (10fold increase), Italy (255.1%) and Germany (200.4%). The mean increase of exports of Greek timber industry amounted in 2002 to 26.7% (IERS and SEVE 2006).

The analysis of the representative indicators of export extraversion of Greece indicates a progressive, light reduction of exports, concerning the total crude value of timber industry production (4.4% in 2002 and 6.8% in 1995). The very same tendency is also observed by the analysis of the total value changes of sector exports for the 1995-2002 period, as well as by the percentage (%) of exports change on the total of Greek exports, which started at a 0.5% in 1995 and have reached a 0.3% in 2002. Furthermore, the share of imports on the total of Greek imports is maintained constant at 1.3% (1995 -2002) with certain small losses during the specific time interval.

The aim of this study is the presentation and supply of condensed information to timber enterprises not only about the existing situation, but also about the prospects of the internal and exterior trade of sawnwood in the EU25 generally, and more specifically, the analysis of data for each country in the Union.

Methodology

For the achievement of the aims of research, we used the important UNECE and FAO data base (2004), which constitutes a time series of 39 years, for the 1964-2002 period. This data base was retabulated and processed and new data were derived regarding the production, imports, exports and the apparent consumption in quantity, as well as in value, of sawnwood of the 25 countries in the European Union for the above mentioned period. We applied the time series analysis (Koutsogianni 1977, Ostrom 1990, Anderson et all 1996, Mutanen 2006). Then we compared and analysed the changes of all parameters for the last decade (1992-2002), and we also created a system of indicators, such as: self-sufficiency indicator, the percentage of net exports on the production, the domestic consumed production and the imports –exports ratio (Stamou 2005) for every country in 2002. The time series analysis of the self-sufficiency indicator for each country constitutes a basic criterion for evaluating the strengths and weaknesses and determining the growth opportunities of international trade - at least between the EU countries.

Finally, the suitable regression models were determined. They reflect the relations between the dependent variables: imports, exports, production and apparent consumption and the independent variable of time. The analysis was done using the statistic SPSS ver11 (Dennis and Duncan 2003).

Results

The diachronic development of the quantity (in volumes) of production, imports, exports and apparent consumption of sawnwood (coniferous and non coniferous) in the countries of EU is presented in Figure 1. Initially, we can notice a progressive light increase of all the above parameters, at least up to 1992, while afterwards the increase becomes bigger. Particularly after 2000, the apparent consumption exceeds the dam of the 100 million m^3. This fact reveals the increased consumption of the particular product from the EU consumers, and furthermore, the great tendency of the EU enterprises to increase the exports, naturally in profit - making prices.

Fig. 1. *Production, imports, exports and apparent consumption of sawnwood in 1000 m^3 in Europe at the period 1964-2002.*

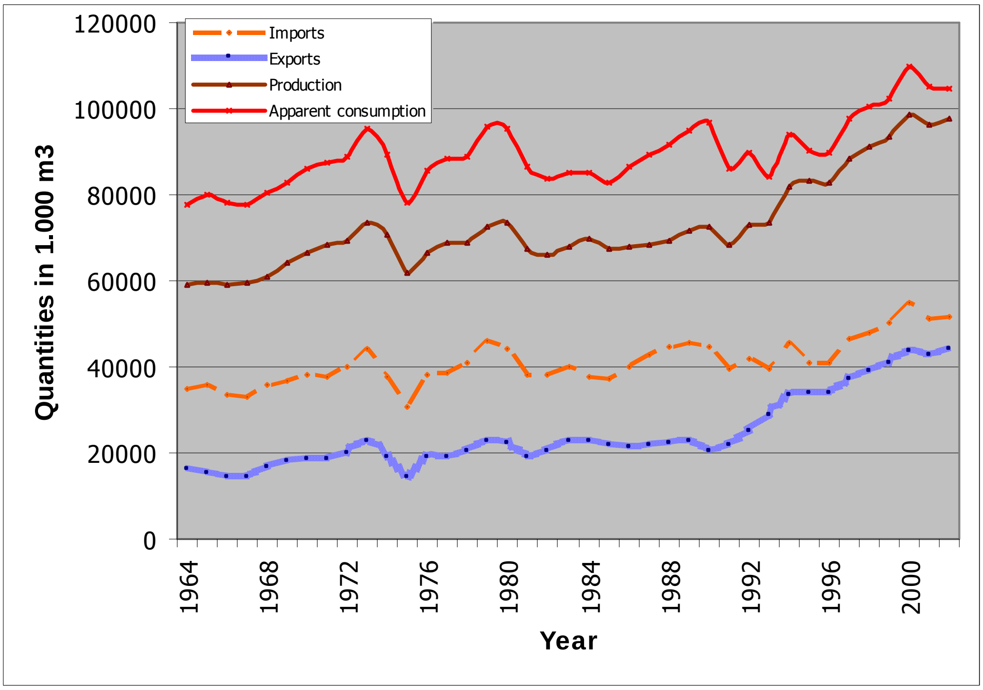
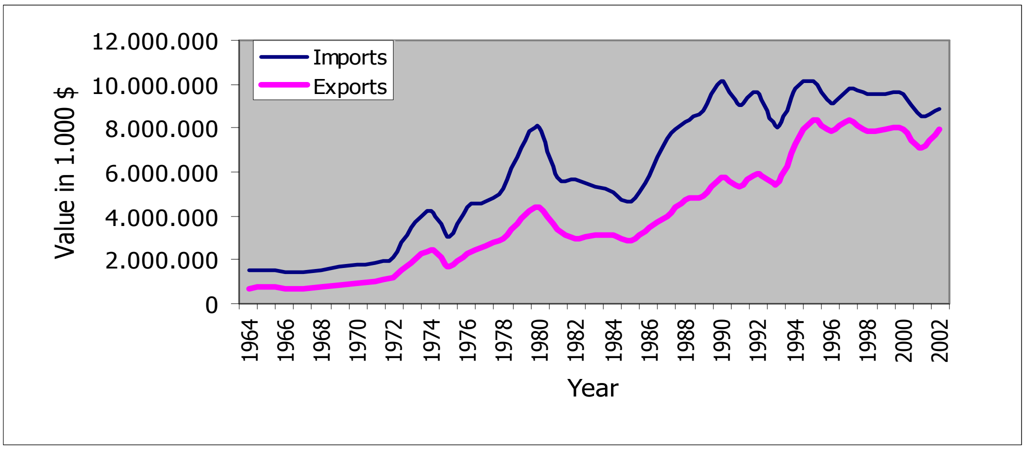


Figure 2 denotes the bigger value of imports related to the exports one, which confirms the above observation on the increase of consumer faith and preference of European customers to European products. It also underlines the leading role of EU25 in the export activity of sawnwood in the world trade, so that the achievement of economies of scale is feasible. Besides, the EU policy is alone a strong point for the timber sector, both because of the enlargement, and also because of the increased amounts, supplied for innovation and particularly for R&D.

The value of sawnwood imports (coniferous) is bigger – almost double - than that of non coniferous. The difference between imports and exports of coniferous and non coniferous is very big, since, the last years, the exports of coniferous appear to exceed the imports, while the exports of non coniferous from EU countries appear to be minimal, in value and quantities.

Fig. 2. *Value of imports and exports of sawnwood in 1000$ in Europe at the period 1964-2002.*



In Table 1 we note that the mainer countries regarding coniferous sawnwood imports in the EU are UK (Tomson 2004), Italy, Germany, Denmark and France, while Greece possesses the 11nd place, among the 25 countries of EU. The mean price of coniferous sawnwood import has decreased by 35.6% and more specifically from 240.5 to 155.0 $/m3, during the period 1992-2002. All the above countries (except Germany) have increased their quantities of imports of coniferous sawnwood during that period.

Table 2 presents the mainer countries of coniferous sawnwood exports in EU, which are Sweden, Finland and Austria, with a significant increase in change of quantities and value during the period 1992 - 2002. The mean price of coniferous sawnwood export has decreased by 25.9% and more specifically from 231.6 to 171.6 $/m3, for the above period.

Table 1. *Change of quantities (in 1000 m^3) and value (in 1000 $) of imports of coniferous sawnwood; period 1992-2002 in the 5 first countries of Europe and Greece*

i/n	Countries	**Quantities (in 1000 m3)**			**Value (in 1.000 $)**		
		1992	**2002**	**Change %**	**1992**	**2002**	**Change %**
1	UK	6899.3	7584.9	9.9%	1340000	1353220	1.0%
2	Italy	4402.0	6092.0	38.4%	1170000	905859	-22.6%
3	Germany	4458.3	4173.0	-6.4%	1340000	703264	-47.5%
4	Denmark	1660.0	4005.0	141.3%	420000	383166	-8.8%
5	France	1404.0	2749.2	96.2%	386092	483338	25.2%
11	Greece	473.0	915.0	93.4%	122000	159224	30.5%
Total EU25		**26464.5**	**37453.6**	**41.5%**	**6365005**	**5805251**	**-8.8%**

Source: Elaboration of UNECE TIMBER database 2003

Table 2*. Change of quantities (in 1000 m^3) and value (in 1000 $) of exports of coniferous sawnwood; period 1992-2002 in the 5 first countries of Europe and Greece*

i/n	Countries	**Quantities (in 1000 m3)**			**Value (in 1.000 $)**		
		1992	**2002**	**Change %**	**1992**	**2002**	**Change %**
1	Sweden	8240.0	11454.0	39.0%	1925113	2160975	12.3%
2	Finland	4268.0	8167.2	91.4%	1027496	1355148	31.9%
3	Austria	3871.0	6462.5	66.9%	866673	1011316	16.7%
4	Germany	945.1	3850.0	307.4%	288673	652207	125.8%
5	Latvia	42.3	2289.6	5312.8%	288860	310281	6247.8%
23	Greece	3.1	4.0	29.0%	557	634	13.7%
Total EU25		**22447.3**	**40474.5**	**80.3%**	**5199629**	**6944515**	**33.6%**

Source: Elaboration of UNECE TIMBER database 2003

Four indicators are reported in Table 3 for each country separately, in 2002: self-sufficiency, the percentage of net exports on the production, the domestic consumed production and the imports – exports ratio. Thus, the indicator of self-sufficiency shows that the EU25 is almost self-sufficient in sawnwood (93%). On the contrary, Hungary, Italy, Greece, Cyprus, Holland, Denmark and Malta are found at a level of under 20%, which proves their weakness and the dependence - to a large extent- on importing sawnwood from the rest EU countries, as well as from other countries of the world market. This fact can be attributed to the following reasons: (a) these countries do not have the suitable natural resources (forests) for the production of timber, (b) the forests' management was not the proper one during the last 100-200 years, (c) the forests carry out only protective and recreation aims and are managed in a relative way, (d) there have not been made any appropriate investments on the creation of wood processing units, and (e) there exists a delay in productivity. On the other hand, countries like the Scandinavian ones (Finland, Sweden), the former USSR (Latvia, Lithuania, Estonia), as well as Czech Republic, Slovakia and Denmark enjoy an overabundance of sawnwood, mainly because of the rich natural vegetation, but also because of the proper and sustainable management of their forests. This fact strengthens these countries' position in the international timber market (e.g. China, India etc.).

Table 3. *Relations of domestic production, international trade, consumption and relative indicators of sawnwood (in 1000 m^3) in 2002 for the 25 EU countries*

i/n	COUNTRIES	IMPORTS (I)	EXPORTS (E)	Production (P)	Ca	Sg%	pE%	pEI%	pP%
1	Austria	1674.1	6645.6	10415.0	5443.5	191%	48%	25%	36%
2	Belgium&Lux	2057.3	1016.7	1308.4	2349.0	56%	-80%	202%	22%
3	France	3287.4	1406.4	10540.0	12421.0	85%	-18%	234%	87%
4	Germany	4862.0	4439.0	16879.2	17302.2	98%	-3%	110%	74%
5	Denmark	10206.7	105.0	281.0	10382.7	3%	-3595%	9721%	63%
6	Greece	1117.0	14.0	122.6	1225.6	10%	-900%	7979%	89%
7	Estonia	236.0	1248.0	1900.0	888.0	214%	53%	19%	34%
8	Un. Kingdom	8263.0	294.0	2539.0	10508.0	24%	-314%	2811%	88%
9	Ireland	842.4	315.9	969.0	1495.5	65%	-54%	267%	67%
10	Spain	2916.2	256.8	3524.0	6183.4	57%	-75%	1136%	93%
11	Italy	7857.0	187.0	1605.0	9275.0	17%	-478%	4202%	88%
12	Cyprus	77.3	0.2	7.5	84.6	9%			97%
13	Latvia	157.9	2857.2	3947.2	1247.9	316%	68%	6%	28%
14	Lithuania	306.6	918.4	1250.0	638.2	196%	49%	33%	27%
15	Malta	18.5	0.0	0.0	18.5	0%			
16	Norway	941.0	625.7	2225.0	2540.3	88%	-14%	150%	72%
17	Holland	3294.0	304.0	253.0	3243.0	8%		1084%	-20%
18	Hungary	1227.0	286.0	221.0	1162.0	19%	-426%	429%	-29%
19	Poland	495.8	788.6	2910.0	2617.2	111%	10%	63%	73%
20	Portugal	274.0	250.0	1298.0	1322.0	98%	-2%	110%	81%
21	Slovakia	50.0	864.0	1265.0	451.0	280%	64%	6%	32%
22	Slovenia	186.7	368.5	446.0	264.2	169%		51%	
23	Sweden	439.0	11475.6	16560.0	5523.4	300%	67%	4%	31%
24	Czech Rep.	381.0	1448.0	3800.0	2733.0	139%	28%	26%	62%
25	Finland	257.4	8187.0	13390.0	5460.4	245%	59%	3%	39%
TOTAL EU		***51425.3***	***44301.6***	***97655.9***	**104779.6**	**93%**	**-7%**	**116%**	**55%**

Ca=Apparent Consumption = Production (P) + Imports (I) - Exports (E)
Sg% = Indicator of self-sufficiency = P * 100 /(P-E+I)
pE% = Percentage of net exports on the production = (E-I)*100/P
pP% = Domestic consumed production = (P-E)*100/P
pEI% = Imports - exports ratio (percentage) = (I-E) * 100

The indicator pE% stands for the percentage of net exports (exports –imports) on the production. Denmark presents a very high percentage (3595%), while the indicator is negative for Greece, Italy, Hungary, Ireland and Belgium.

The indicator of net consumed production appears to be particularly low in Latvia, Slovakia, Estonia, Sweden and Finland, and shows the overproduction of sawnwood in relation to the quantity, consumed in this countries.

Finally, the indicator of imports - exports ratio presents a mean percentage of 55%, in the EU25. A percentage bigger than 80% appears for Cyprus (97%), Spain (93%), Greece (89%), Un. Kingdom (88%), Italy (88%), France (87%), Portugal (81%) and reveals their particular dependence on sawnwood exports.

Figures 3,4 and 5 present the diachronic development of self-sufficiency indicator among the 25 EU countries, sorted in 3 groups (the first group with an indicator > 100%, the second one presents an indicator between 50% and 100% and the third one with an indicator smaller than 50%).

Figure 3 shows that all countries follow a relatively constant course, with the exception of Latvia and Slovakia, which present an intense fluctuation of self-sufficiency indicator. Upward trends figure for Sweden and a light bending for Austria.

Figure 4 records an intense fluctuation of Portugal's self-sufficiency indicator, which has notably decreased the last decade. We record a continuous rising course for Ireland and Germany and naturally for the mean of all EU25 countries.

Figure 5 presents Hungary and Cyprus with an intense reduction of self-sufficiency indicator of sawnwood, particularly during the last decade. The same reduction is noted for Greece and Italy, but to a smaller degree. United Kingdom is the only one to present a slight upward course.

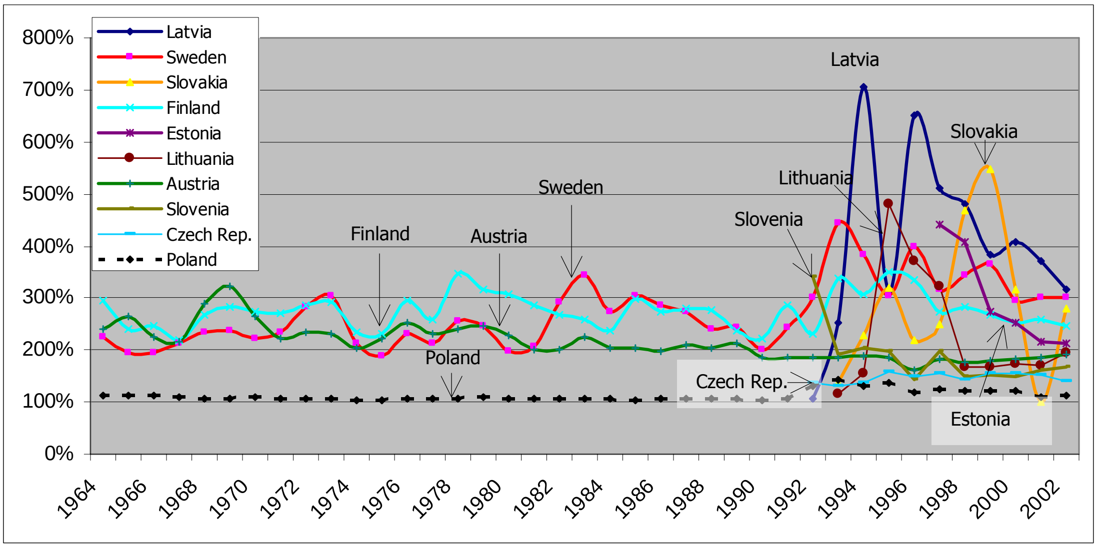


Fig. 3. *Diachronic development of sawnwood self-sufficiency indicator (SFI); First group EU25 countries with SFI> 100, at the period 1964-2002.*

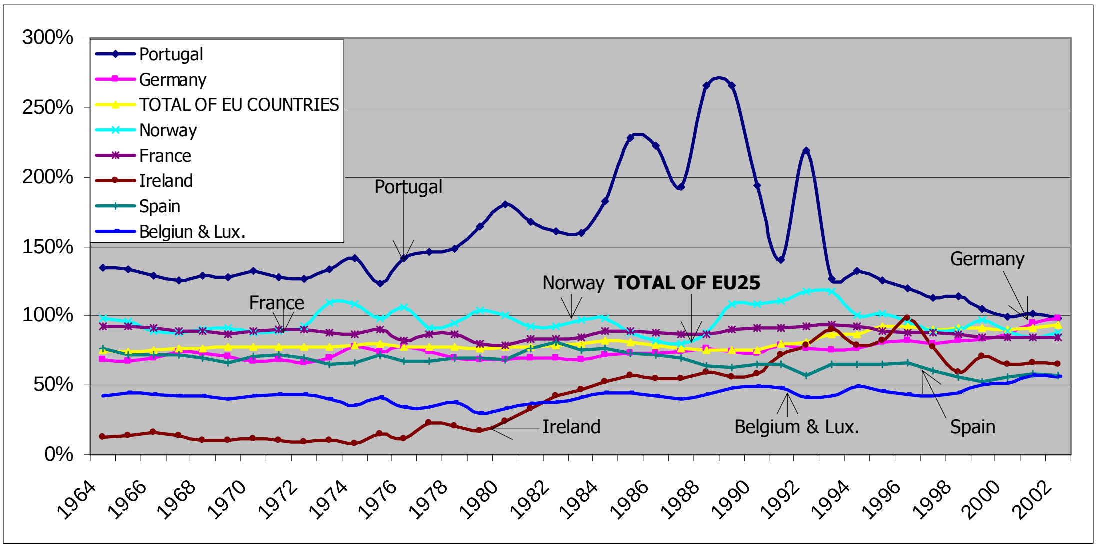


Fig. 4. *Diachronic development of sawnwood self-sufficiency indicator(SFI); Second group EU25 countries with 50<SFI< 100, at the period 1964-2002.*

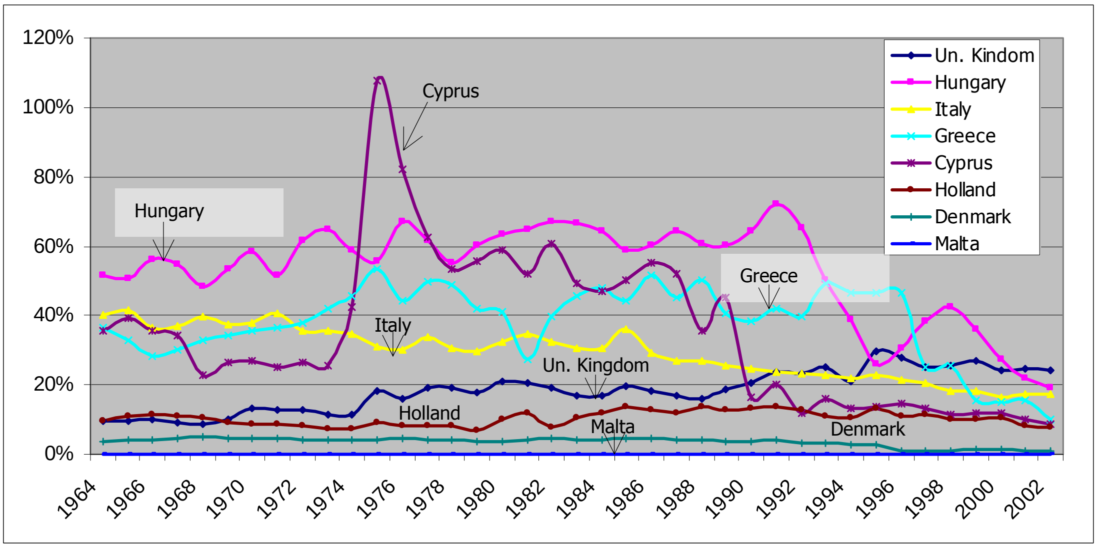


Fig. 5. *Diachronic development of sawnwood self-sufficiency indicator (SFI); Third group EU25 countries with SFI< 50, at the period 1964-2002.*

The following forecast models for the European sawnwood resulted from the regression analysis:

(1) For imports: $Y(I) = 32110.853 + 1171.886t - 62.404t^2 + 1.184t^3$ (R^2 = 0.698, F=26,9, Sig = 0.000 and S.E. = 3086.49).
(2) For exports : $Y(E) = 13464.727 + 1053.03t - 65.402t^2 + 1.561t^3$ (R^2 = 0.943, F=193.9, Sig = 0.000 and S.E. = 2158.11).
(3) For the production: $Y(P) = 54669.883 + 2495.417t - 150.115t^2 + 3.029t^3$ (R^2 = 0.930 και F=154.6, Sig = 0.000 and S.E. = 3021.32)
(4) For the apparent consumption: $Y(Ca) = 73316.008 + 2614.272t - 147.118t^2 + 2.652t^3$ (R^2 = 0.723, F=30.5, Sig = 0.000 and S.E. = 4349.04)
(5) For EU25 self - efficiency: $Y(Sg) = 0.752 + 0.004t + 8.15E\text{-}0.006t^3$ (R^2 = 0.813, F=50,8, Sig = 0.000 and S.E. = 0.027)
$t_{1964} = 1$, for all models.

The above relations can forecast the development of all parameters at a very satisfactory degree, with a significant reliability and precision. The forecast is more precise for the parameters that refer to exports, production and sawnwood self-sufficiency of EU25.

Indicatively, the analysis of model (5) results to the conclusion that the 100% sufficiency of EU25 in sawnwood will be achieved in year 2030, provided that all parameters that so far influence it, develop at the same rate.

Conclusions & Proposals

The tendencies in volume quantity of production, imports, exports and apparent consumption of sawnwood (coniferous and non coniferous) in the 25 countries of EU present to be regularly augmentative, with the apparent consumption to have already exceeded the amount of the 100 million m^3, since 2000.

The EU25 has a leading role in the export activity of sawnwood in the world trade and thus the achievement of economies of scale is feasible. During the period 1992-2002, the mean price of roundwood import has decreased by 35.6%, and more specifically from 240,5 to 155.0 $/$m^3$. The mainer countries of coniferous sawnwood imports in the EU are UK, Italy, Germany, Denmark and France, while for exports are Sweden, Finland and Austria.

The self-sufficiency indicator in sawnwood of EU25 amounts to 93% and is expected to reach the 100% percentage, up to 2030. Countries with the lower indicator of self-sufficiency (< 20%) today, are: Hungary, Italy, Greece, Cyprus, Holland, Denmark and Malta. On the other hand, Finland, Sweden, Latvia, Lithuania, Estonia, Czech Republic, Slovakia and Denmark have an overabundance in sawnwood and can exploit this comparative advantage to conquest new global markets.

The forecast models of production, imports, exports, apparent consumption and sufficiency indicator of sawnwood of EU25, which resulted from the present research, can be applied with a significant reliability and precision.

In the today's international competitive environment, the identification and translation of the external factors (e.g. the competitive position, the market uncertainty etc), as well as the internal ones (e.g. the product development process, the technological competitiveness, the administrative capabilities, the internal information in market knowledge acquisition, the communication and the marketing) constitute a critical point for the successful prospects of a product (Brent et all, 2000).

Strategic plans are imposed to be drawn up, for the timber sector enterprises to penetrate in new markets at international level. The plans should include: a) the evaluation of enterprise's strengths and weaknesses, b) the mission statement and the definition of the comparative advantage, c) the analysis of the enterprise environment, including the changes that take place in it, d) the definition of long-range aims of the enterprise, e) the recognition and the choice of specific markets / products that offer the more attractive opportunities for the enterprise, its possibilities given, f) the determination of specific and measurable objectives that are required for the achievement of the longterm aims of the enterprise and e) the growth of strategic programs.

The promotion of exports of certain countries should constitute a permanent goal and state policy, through the mobilisation of their diplomatic services and the imitation of best practices of countries with exemplary extraversion, successful internationalisation and export performance (Finland, Ireland, N. Zealand and Chile), the close collaboration of the public and private sector, based on objectives and strategy and the linkage between technological research and exports. At the same time, the European timber enterprises should invest in entrepreneurship abroad, branding, research and technology, the creation of distribution networks abroad and in the quality, certification and labelling of both enterprises and their products.

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