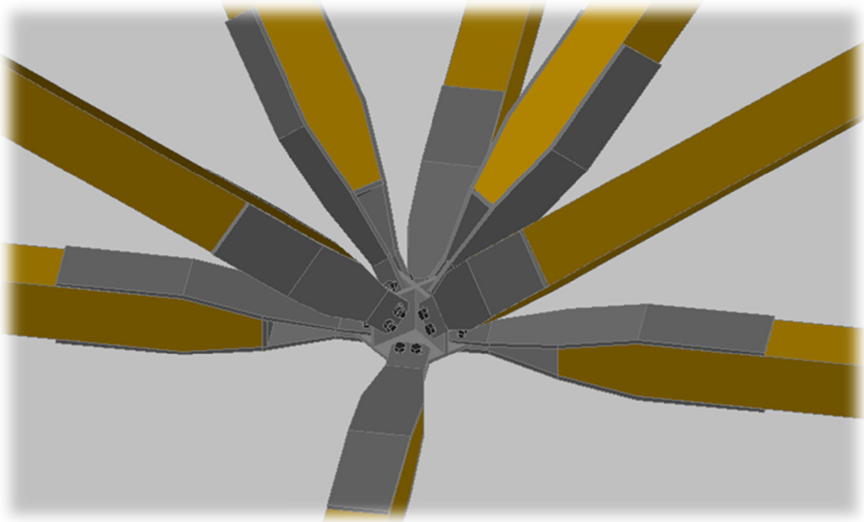
# Glued laminated timber space truss systems



**Nicola Moggio**

Avdelningen för Konstruktionsteknik
Lunds Tekniska Högskola
Lunds Universitet, 2012

Rapport TVBK - 5218

Università di Trento

Lunds Tekniska Högskola

MASTER THESIS

# *Glued laminated timber space truss systems*

Nicola Moggio

July 26$^{th}$, 2012

Supervisors:

prof. Roberto Crocetti
prof. Maurizio Piazza

With the special collaboration of Arne Emilsson of *Limträteknik AB*.

Rapport TVBK-5218
ISSN 0349-4969
ISRN: LUTVDG/TVBK-12/5218(138)

Examensarbete
Handledare: Roberto Crocetti
July 2012

*To my friends and my family*

*«Prima ancora che l'uomo preistorico si rizzasse in piedi, stava a contatto con il legno. Boschi, acque, montagne e praterie furono gli elementi dove si mosse e che, assieme a sole e luna, lo accompagnavano nella lenta evoluzione. Ad un certo punto decise di rialzarsi e camminare su due gambe anziché quattro. E in quel momento iniziò a fare uso del legno, partendo dalla clava. Poi scoprì il fuoco.*

*Presumibilmente lo accese un fulmine, sta di fatto che il nostro antenato iniziò a bruciare legna. Per scaldarsi e più tardi per arrostire selvaggina o danzarci attorno. Sortì dalla caverna e, sempre col legno, costruì una capanna per ripararsi. Non da ultimo, scoprì che i tronchi galleggiavano . Allora ne scavò uno col fuoco e costruì la canoa. In seguito edificò palafitte, scale, spostò pesi tramite rulli di tronchi, scoprendo in quel modo la ruota. Tirò avanti di di tale passo fino a costruire navi e quant'altro.*

*Questo preambolo per dire che, fin dai primordi, l'uomo ha guardato in faccia il legno. E' stato così a contatto, e ne ha avuto talmente bisogno da farselo entrare nel DNA.»*

Mauro Corona

# Acknowledgments

It is rather difficult to list all the people I want to express my sincere gratitude in the most spontaneous and complete way. The goal that I'm about to reach at the end of my long course of study makes me think back to all the time spent in the recent years, the commitment, the hard work, the disappointments and the satisfactions obtained along the way. Everything would never have happened in the absence of all the people who have been close to me and the innumerable possibilities that I had the lucky to have.

The opportunity to work in Sweden from January to July 2012, enabled me to expand my horizons both in terms of technical and personal growth. I would like to warmly thank all the people who have contributed to the development and achievement of this great experience starting with my two supervisors **prof. Maurizio Piazza** *(University of Trento)* and **prof. Roberto Crocetti** *(Lunds Tekniska Högskola)* who have always been available from the beginning welcoming immediately my ambition and suggesting the thesis-work about which I immediately showed great interest. Their support has been essential until the end, I refer to the great technical competence and the constructive attitude with which they have followed me during the research.

A warm thanks is to **Arne Emilsson** of the engineering office *Limträteknik AB* who hosted me in his office for three months in which I could completely get a feel for working in Sweden. During those months I had the fortune of collaborateing with one of the greatest Swedish engineers within the timber structures, he proved to be not only a patient teacher but also a dear friend. His competence and his infinite patience lay the basis of my work which was originally proposed by him, providing the material and the necessary support for the development of the research.

Finally, a deutiful thanks is for the **Provincia Autonoma di Trento** for the granting of the scholarship available to encourage research activities in the international arena. The opportunity has been for me a fundamental economic aid

Lund (Sweden), *July 2012*

# Contents

# Preface

Space structures are a leaf taken from Nature. Natural forms possess exceptional rigidity and use minimum materials to maximum structural advantage. To quote Makowski[1], "*Natural forms act in the direction of minimum effor*t". Man has not been slow in copying these examples drawn from Nature [1].

The main structural aim which drives the use of this construction system is to use the least material possible to achieve a lightweight structure and at the same time have good characteristics of rigidity and restricted deformation. The proposed construction technology can provide ample opportunity for structural engineering; it lends itself to the creation both of bridges of considerable importance and of three-dimensional structures for covering large areas without the use of bracing elements.

Nowadays, the development and diffusion of timber construction systems demonstrate their validity and competitiveness especially in the construction of roofs and bridges, even with long spans. This has given rise to the idea to use timber even for the space frame technology. The research will be directed to explore the possible applications of the system, initially scheduled for tubular steel structures, in the field of wooden construction.

Always used as a structural material in the construction of frames, floors, roofs and bridges, timber material has found high regard in modern civil constructions. In the building industry we often talk about sustainability, energy conservation and the need to relate the constructions with the hosting environment, from this perspective timber has been greatly revalued rediscovering the qualities well-known in traditional building regarding the excellent mechanical characteristics, the simplicity and the versatility for esthetic and modern architectural use.

The idea of applying this technology to laminated wood structures was proposed by ing. Arne Emilsson of the engineering company LIMTRÄTEKNIK AB which boasts a long career in the design of timber structures in Sweden and always looks for new applications of the material in modern buildings. The proposed research project is inserted harmonically in a larger context that sees the Università degli studi di Trento and the Lunds Tekniska Högskola busy with their own research institutes and associated companies in the development, use and promotion of wood as a building material.

The research is directed to explore the possible applications of the space frames

[1] Professor Z.S.Makowski is the author of Analysis, Design and Construction of Braced Domes (1984), he is acknowledged as the foremost promoter of the modern steel space frames.

system in the field of wooden construction, the proposed construction technology can provide ample opportunity even for timber engineering. The aim of the thesis is to prove the possible application of the timber material considering that the design of wood structures is very complex and requires detailed theoretical knowledge accompanied by the intuition and the ability which comes from an understanding of the critical points of the structures. The work is organized into several parts that try to consider all the thematic relating to the design of the specific construction technology and the material particularities.

*Part I* describes in general the timber space frames system; it elencates the description, the development, the components and the advantages of the technology. It also introduces and correlates the use of timber material in the space frames system listing any possible issue related.

*Part II* is about the development of the connection system for transferring the axial stresses from the member to the node. The connections must be able to resist cyclic stresses, exhibiting a good ductile behavior, allowing, among other things, to increase the general robustness of the construction. Different technologies like screws inclined 45° to the grain and pre-stressed bolts working in shear are mentioned for an advantageous application in the truss system.

*Part III* treats a specific case study of a space frame court mentioned in the book *«Analysis, design and construction of Steel space frames»* (G. S. Ramaswamy et al. 2002) [1] . The design of the same construction in glued laminated timber instead of high resistance steel, using the same geometry and loads demostrates the complete applicability of the topics treated in the previous parts. The design is performed according to the new regulations checking the resistance of any element at the ultimate limit state. The deformation of the connection with preloades bolts and the deflection of the court are checked to be within the limits recommended.

# Part I

# Timber space frame systems

# Chapter 1

# Introduction to Space Frames

## 1.1 Space frames

A space frame is a structural system, assembled of linear elements so arranged that the loads transfer in a three-dimensional manner. In some cases, the constituent elements may be two-dimensional. According to the structural analysis approach, a *space frame* is analyzed by assuming rigid joints that cause internal torsions and moments in the members, whereas a *space truss* is assumed as hinged joints and therefore has no internal member moments. The choice between space frame and space truss action is mainly determined by the joint-connection detailing and the member geometry is no different for both. However, in engineering practice, there are no absolutely rigid or hinged joints. For example, a double-layer flat surface space frame is usually analyzed as hinged connections. In this thesis we will consider just «*space trusses*» but in literature the term «*space frame*» is generally used to refer both space frames and «*space trusses*».

Macroscopically, a space frame often takes the form of a flat or curved surface.

The space frame is one of the most important structural systems very often adopted in modern buildings of large dimensions. Among other structural systems the space frame has a few essential advantages. One of those advantages is its lightness. The lightness of the structure is due mainly to the fact that material is distributed spatially in such a way that the load transfer mechanism is primarily axial, either tension or compression, so that in any given element all material is fully utilized. In large-span roofs, where the self weight of the structure constitutes an important part of the total load, the lightness of the constituent elements largely contributes to the rationality and economy of the whole building. Another advantage of the space frame is that the production and construction techniques are industrialized to an extent much greater than is the ease in conventional structural systems. The linear elements and the joints are both prefabricated, so that the jointing work at the site is relatively simple.

Thus a space frame can sometimes be erected by semi-skilled workers. The light weight of the individual elements also makes the handling and assembly work easier.

A space frame is sufficiently stiff in spite of its lightness. This is due to its three-dimensional character and to the full participation of the constituent elements, which adapt themselves equally well to almost all types of loading.

Figure 1.1.1: Flat space frame structures in India.

Figure 1.1.2: The roof structure of the Palafolls Sport Hall, near Barcelona, Spain demonstrates graphically that space grids do not have to be constructed as a flat plane nor to a rectangular plan [2].

## 1.2 Brief history and development of system

Alexander Graham Bell may be considered the first to construct modular space grids.

It is not well known that Alexander Graham Bell (1847 -1922) experimented with space truss structures made of octahedral and tetrahedral units early in this century. He appreciated the dual properties of high strength and lightweight exhibited by these rigid structural forms and incorporated them into several of his projects.

The MERO system was the first commercial space grid system.

Despite Bell's construction of modular space trusses early this century, the MERO system was the first space grid system widely available commercially. This was developed in Germany by Dr. Ing. Max Mengeringhausen (1903 - 1988) but was not introduced until the early 1940s.

The MERO system is probably the most elegant concept for the construction of space grid structures, the connection of circular tube members to 'ball' joints at the nodes by a single concealed bolt. It developed from Mengeringhausen's study of natural structures such as wheat stalks and bamboo canes. The name MERO derives from an abbreviation of the original name Mengeringhausen Rohrbauweise.

Usually the members are circular hollow steel or aluminum tubes that have tapered cone sections welded to each end (complete with connection bolt and sleeve) and the nodes are hot forged solid steel or drop forged aluminum spheres with drilled and tapped holes and profiled to receive the tube ends. It is also possible to use laminated timber members that have short tubular steel inserts at the ends for connection to the nodes.

Originally the MERO system had only one type of standard joint, a sphere with 18 threaded holes and machined bearing surfaces at angles of 45, 60 and 90° to each other. A series of standard bar lengths were also produced such that the node to node lengths were increased by a factor of 2 with each increment in the series. However, with modern numerically controlled precision drilling techniques, holes may now be at almost any required angle and members are manufactured to the appropriate length thus allowing great flexibility in geometry.

There are, of course, many examples of MERO space frames worldwide as one would expect for a system nearly 60 years old and there are many alternative node joints and member cross sections. However, one of the most dramatic uses of the MERO system was for the grandstand roofs at the stadium in Split, Yugoslavia. These roofs are segments of a 452 m diameter cylinder inclined at 11.2° and the free edge spans 215 m with an arc length of 220 m. An impressive use of space grids and glazing can be seen in the atrium wall and roof at the Presidential Circle Office Building in Hollywood, Florida.

The MERO system uses the most common method of space truss construction consisting of individual bars connected at 'ball' shaped node joints. The popularity of this system has endured to the present day and there are now many alternatives based on the MERO ball joint concept. Because of its elegance, the MERO system it is not only used for building structures but also for shop displays and exhibition stands using light weight materials.

Both systems were based on prefabricated steel pyramidal modules (1.22 m x 1.22 m in plan and 1.05 m or 0.61 m deep respectively). The Nenk system was used for roof and floor construction in barrack blocks where it could span 12.2 m with normal floor loads and 26.8 m with normal roof load. Space Deck has been widely used for roof and floor structures with only slight modification to module dimensions and materials ever since.

There are many examples of Space Deck structures as it has been available for almost 50 years in essentially the same form with only minor changes in materials and metrication of module dimensions. Some examples of Space Deck projects: a pitch-roofed atrium at the Hyatt Hotel, Birmingham; a barrel vault at the Bentall Center, Kingston-upon-Thames and the Pearl Hotel, Libya.

Elsewhere in the world, during the 1950s and 60s, space frame systems were proliferating as architects explored the new aesthetic and engineers experimented with alternative jointing systems, materials and configurations.

For instance, in the USA, Richard Buckminster Fuller (1895 - 1981) developed the Octetruss system using tubular steel members and a bolted node joint and Konrad Wachsmann (1901 - 1980) developed a space grid system for large span aircraft hangars for the U.S. Air Force.

Wachsmann's system incorporated a relatively complicated universal connector made from a combination of four standard die forged elements which allowed up to 20 tubular members to be connected at each joint. Stephane du Chateau, in France, developed Tridirectionelle S.D.C., Tridimatic and Pyramitec forerunner of the Unibat system. In Canada, the Triodetic system, predominantly using aluminum as the material for the bars and joints, was introduced by Fentiman Bros. of Ottawa.

Notable examples of long span space grids constructed in 1970 and 1973 were the British Airways maintenance hangars (formerly owned by BOAC) at Heathrow Airport, designed by Z.S. Makowski and Associates.

The hangar roofs are diagonal, double-layer grids 3.66 m deep and provide a column-free covered area 67 x 138 m in plan. In this case, the space grids were not constructed from a proprietary system but were manufactured from tubular steel prefabricated elements joined on site with bolted grid connectors.

Advances in analytical capabilities led to further developments of new systems and the use of space grids for longer spans.

Around the same time, the introduction of electronic computers and the development of programs to enable space grid structures to be analyzed more accurately increased confidence in their use for longer span structures. In the late 1960s and early 1970s many of the pioneering space grid systems were superseded by second-generation systems such as Nodus, Orona, TM Truss etc.

The Nodus system was developed during the late 1960s by the Tubes Division of the British Steel Corporation (now Corus) and was introduced in the early 1970s. After 1985 the system was owned by Space Decks Ltd. of Chard but is no longer available.

The Nodus system was developed with a sophisticated node joint in a range of different sizes and load capacities appropriate for their tubular section products. All of the joints were tested to failure in a special rig, at their Research Center in Corby, to prove their effectiveness and a full size 30.5 x 30.5 x 1.52 m deep space grid was also tested. This structure was dismantled after testing and re-erected as the Space Structures Research Laboratory at the University of Surrey.

The Nodus joint uses cast steel connectors which are butt welded to the chord or bracing members in jigs to ensure dimensional accuracy. Chord members are clamped together between the halves of the node castings using a high strength friction grip bolt and the bracing members are joined to lugs on the node plate by steel pins through the forked end connectors (see exploded view of the joint).

The joint configuration with pinned connection of the bracing members permits variation of the space grid depth, limited only by the requirements of structural efficiency or interference between members at the joint. However, the standard joints are only produced in two forms with lugs for the connection of bracing members either in line with the chord members or alternatively at 45° to the chords when viewed in plan. This limits the possible grid configurations to variations of the square on square or square on diagonal layouts.

Some examples of the use of Nodus are for a canopy at the Hyatt Hotel, Birmingham, the refurbishment of Cwmbran Town Center and in the construction of Terminal 2 at Manchester Airport.

The CUBIC space frame system is a ’true’ space frame that was developed during the late 1970s by Leszek Kubik and his son Leslie and is now marketed by ASW-CUBIC Structures a subsidiary of ASW Construction Systems. It is formed from rigid-jointed modules with no diagonal web members.

As this is a modular system containing no triangulation the chords and vertical members are subjected to bending and shear, in addition to axial forces, as they resist the applied loads by frame action. If an orthogonal grid is formed from Vierendeel girders in both directions and joints are introduced in the top and bottom chords midway between each chord intersection, the grid can be constructed of modules ’X’, ’T’ or ’L’ shaped in plan. These are the basic modules of the CUBIC Space Frame system. No special components are required in fabricating the modules as they are made from standard hot-rolled steel sections and plates welded together in a jig using standard fabrication techniques. Final assembly of the space frame is by site bolting of the lap joints in the chords.

The CUBIC Space Frame system was first used for re-roofing a rehearsal theater 12 m x 20 m at Trent Polytechnic (now Nottingham Trent University) in 1979. It has since been used successfully to roof many building types including factory units and supermarkets where the absence of bracing members has allowed installation of plant, services and even offices within the depth of the space frame.

Recently, Hall 3 at the International Convention Center in Birmingham, approximately octagonal in plan and around 55 m in span, was covered by a CUBIC space frame capable of carrying a point load of 30 tonnes at any of the nodes.

A modification of the system has been developed for composite floor construction in office buildings to exploit the facility for service provision within the structural depth.

The 1980s saw the development of lightweight space truss systems using cold-rolled sections such as the Conder Harley System 80 and Space Deck "Multiframe".

The Harley system was introduced into Europe by Conder Group plc in 1989. It was manufactured under license from the patent holders in Australia where it has been available since 1980. This system, which is no longer available in the UK, was used for re-roofing 8500 $m^2$ at the Eagle Center Market in Derby in 1991. Space Decks Ltd. have also developed a lightweight space truss system called Multiframe which has only been used for a limited number of small projects despite its competitive price.

The chords of the space trusses are continuous over several bays (up to about 12 m long) and do not intersect in the same plane. This makes construction easier but means that bending moments are introduced into the chords due to the slight eccentricity of forces at the nodes caused by the channels in the two orthogonal directions being connected back-to back and not in the same plane (see photograph of Harley system and diagram of Multiframe node).

## 1.3 Components and structure of the space frames

### 1.3.1 Members

A space frame consists of axial members and connectors, which join the members together. The majority of space frame systems for building structures are manufactured from steel or aluminium although timber, concrete and reinforced plastics are also used.

The more common steel frames are preferably tubes, also known as circular hollow sections or rectangular hollow sections, other structural sections such as I- and H-sections are also occasionally used, especially if loads that are transferred to the members between nodes cause bending in addition to axial forces. Where loads are applied only at nodes, circular and rectangular hollow sections have an edge over other section types because they are more efficient in compression, offering a higher radius of gyration for the same area. Circular hollow sections have the further advantage that their moments of inertia are the same in all directions. Square sections have good behavior as well especially for timber member, allowing to get good performances and cheap production too.

Occasionally, the term space truss appears in the technical literature. According to the structural analysis approach, a space frame is analyzed by assuming rigid joints that cause internal torsions and moments in the members, whereas a space truss is assumed as hinged joints and therefore has no internal member moments. The choice between space frame and space truss action is mainly determined by the joint-connection detailing and the member geometry is no different for both. However, in engineering practice, there are no absolutely rigid or hinged joints. For example, a double-layer flat surface space frame is usually analyzed as hinged connections. The term «space frame» will be used to refer both space frames and «space trusses».

The main structural considerations in the design of space truss elements are the buckling of compression chords and web bracing members and the design of joints to effectively and efficiently transmit tension forces between the bars and nodes whilst minimizing secondary bending effects.

Although the weight of a typical aluminium alloy is only one third that of steel for an equivalent volume, it also has a lower modulus of elasticity. Thus, for an aluminium alloy of equivalent strength to steel the resulting aluminium alloy structure may be lighter unless deflections are critical. In this case, additional material may be required to keep deflections within acceptable limits. As the material cost for aluminium alloy is greater than that of steel the choice of material will depend very much on individual circumstances. Greater care is required to weld aluminium than steel and as many space grid systems involve at least some welding in their manufacture steel, in various strength grades, is the most commonly used material for the members. Many systems use cast steel for end connectors and node joints.

### 1.3.2 Node connectors

The nodes may be of many forms depending on the system used (e.g. 'ball' joints, hollow spheres, profiled plates etc.) and this tremendous variety of jointing systems demonstrates the difficulty of achieving a simple, aesthetically pleasing tension joint.

**MERO SYSTEM:** it consists of a threaded spherical ball of hot forged steel with as many as 18 tapped holes, at different angles, distributed evenly over its surface, to receive the members at different angles. The sphere has flat surfaces around the threaded holes to improve the seating of the spartner sleeve. The holes are precisely drilled so that the center lines of the tubes at at node meet at the center of the sphere (figure 1.3.1).

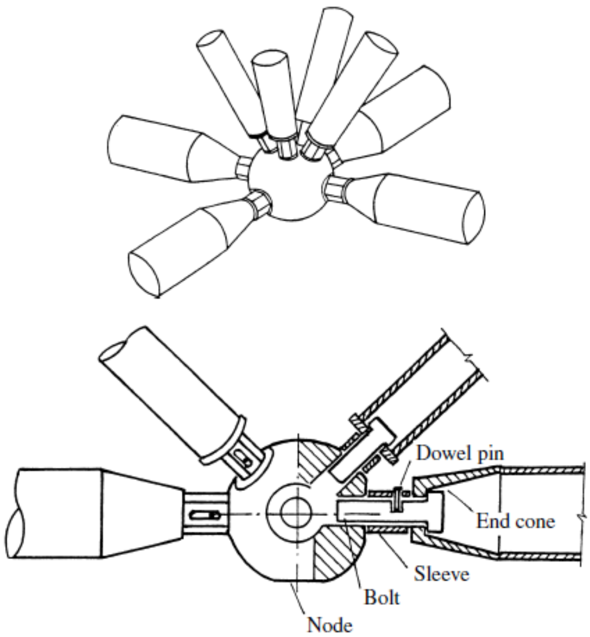



Figure 1.3.1: The MERO system (reproduced from the book «Handbook of structural engineering»[2])

**OCTATUBE SYSTEM:** the system was developed by Prof. Dr Ir. Mick Eekhout of The Netherlands consists of an octagonal base plate to which are welded two semi-octagonal plates placed at right angles to each other. The Octatube node connector is a plate connector, the tubes meeting at a node are flattened and connected by means of high-strength bolts. Developed in 1973, this node connector can be manufactured in any workshop. The connector is designed for space frames meant to roof workshops, warehouses and other structures where cost rather than aesthetics is the governing consideration. It is possible to use sections other than tubes to effect the connection if a plate is welded to the end of the member (figure 1.3.2).

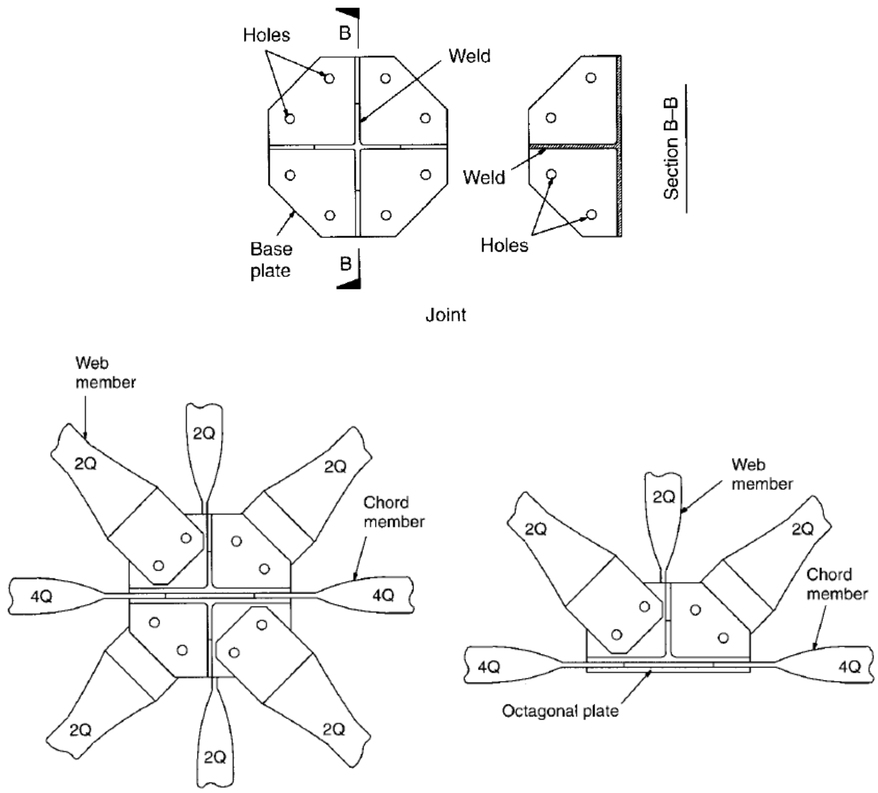



Figure 1.3.2: The OCTATUBE system (reproduced from the book «Analysis, design and construction of Steel space frames»[1])

**TUBALL CONNECTOR:** developed by Eekhout in 1984 , is a hollow sphere made of spheroidal graphite. One-fourth of the sphere comprises a cap and the rest is a cup .The end of THC circular or rectangular hollow section member to be connected is fitted at its ends with threaded solid props by welding. Working from inside the cup, high-strength bolts, normally of 8.8 or 10.9 grade are driven into the threaded prop by means of a torque wrench. Coning of the ends of the tube is resorted to if tubes of large diameter are to be accommodated without congestion over the surface of the cup. If the tension to be transmitted to a node exceeds the permissible tensile strength of the node. the member is carried through the node by using a threaded rod to connect the ends of the tubes. Being hollow, the Tuball node tends to be lighter than a solid forged node. It is also less expensive, because spheroidal graphite costs less than forged steel. The Tubal node has been used successfully for building numerous space frames in India. the United Arab Emirates and the Far East using relatively unskilled labours.

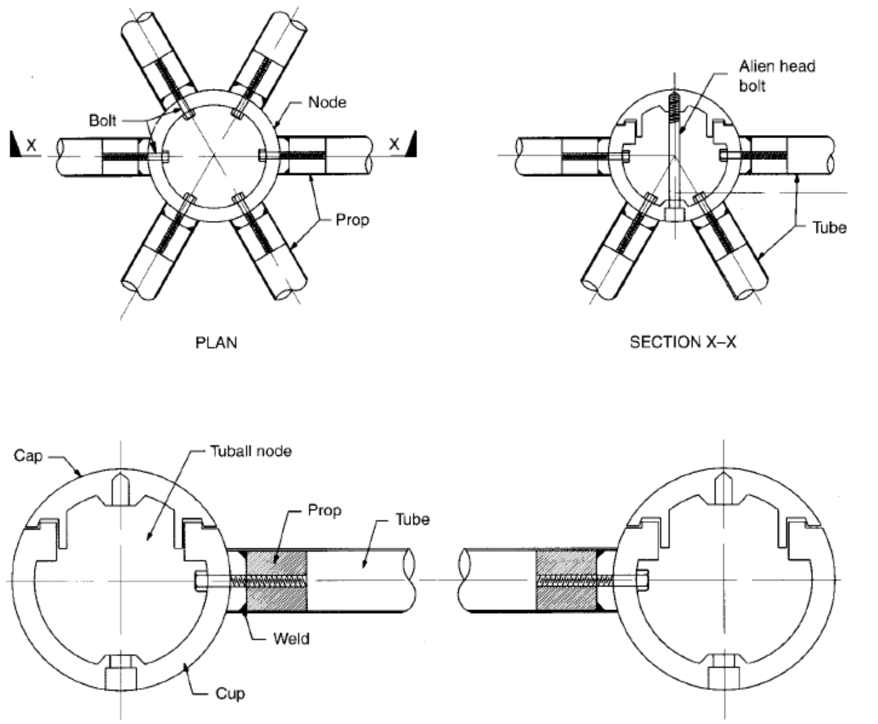



Figure 1.3.3: The TUBALL connection (reproduced from the book «Analysis, design and construction of Steel space frames»[1])

### 1.3.3 Span and design parameters for space grid structures

It is difficult to generalize about span/depth ratios for space grid structures as they depend on the method of support, type of loading and to a large extent on the system being considered. Before any work can proceed on the analysis of a double-layer grid, it is necessary to determine the depth and the module size. The depth is the distance between the top and bottom-layers and the module is the distance between two joints in the layer of the grid. Although these two parameters seem simple enough to determine, yet they will play an important role in the economy of the roof design. There are many factors influencing these parameters, such as the type of double-layer grid, the span between the supports, the roof cladding, and the proprietary system used. In fact, the depth and module size are mutually dependent, which is related by the permissible angle between the center line of web members and the plane of the top and bottom chord members.

The depth and module size of double-layer grids are usually determined by practical experience. In some of the papers and handbooks, figures on these parameters are recommended, and one may find that the difference is quite large. For example, the span–depth ratio varies from 12.5 to 25, or even more. Z.S. Makowski suggests that span/depth ratios may vary from 20 to 40 depending on the rigidity of the system used. High span/depth ratios are appropriate if there are full edge supports but they should be reduced to about 15 to 20 when the supports are only at or near the corners of the grid. Span tables produced by Space Deck indicate that for typical loadings roof span/depth ratios of about 30 are possible using their standard modules. It is usually considered that the depth of space frame can be relatively small when compared with more conventional structures. This is generally true because double-layer grids produce smaller deflections under load. However, depths that are small in relation to span will tend to use smaller modules, and hence a heavier structure will result. In the design, almost unlimited possibilities exist in practice for the choice of geometry. It is best to determine these parameters through structural optimization.

Through regression analysis of the calculated values by optimization method where the costs are within the 3% optimum, the following empirical formulas for optimum span–depth ratio are obtained. It was found that the optimum depths are distributed in a belt and all the span–depth ratios within this range will have optimum effect in construction.

– For roofing system composed of reinforced concrete slabs:

$L/d = 12 \pm 2$

– for roofing system composed of steel purlins and metal decks:

$L/d = (510 - L)/34 \pm 2$

where $L$ is the short span and $d$ is the depth of the double-layer grids.

Few data could be obtained from the past works.

### 1.3.4 Grid configurations

There are many possible ways of dividing up a flat plane by a grid of lines connecting points in a regular or irregular pattern but this may produce considerable variation in the length of lines and the angles between them. However, in modular structural systems such as single or double-layer grids it is advantageous if, in any structure, the number of different member lengths can be limited and connection angles at the joints standardized. Therefore, regular shaped grids are usually adopted for both the top and bottom layers of space grids but there are only three regular polygons, the equilateral triangle, square and hexagon, that can be used exclusively to completely fill a plane.

In square grids the grid lines can be parallel to the edges of the grid or set on the diagonal, usually at 45° to the edges. Both of these are two-way grids having members orientated in two directions, however, plane grids of triangles and hexagons produce three-way grids with members orientated in three directions.

More complex grid geometries may be produced by combining the regular polygons or by using them in combination with other polygonal shapes (e.g. triangles and squares, triangles and hexagons, squares and octagons).

In space grid structures, where two plane grids are separated by web members to form a double-layer grid, the top and bottom grids do not necessarily have to have the same pattern or orientation but in practice, for reasons of cost and the required connection of web members, the number of common configurations is limited.

The common forms of double-layer grids are:

- *square-on-square offset* (type 1) where the bottom grid is offset by half a grid square relative to the upper grid with web members connecting the intersection points on the top and bottom grids;

- *square on square offset*, set diagonally (type 2), is more rigid than the square on square offset;

- *square on larger square* (type 3);

- *square on large square set diagonally* (type 4);

- *square on square diagonal* (type 5);

- *diagonal on square* (type 6).

More open grid geometries are often possible in the lower layer of a double-layer grid because the members are generally in tension, i.e. not subject to buckling, and may, therefore, be longer than the upper compression members even though the forces in the lower chords may be greater. In modular systems it is often possible to omit complete modules in a regular pattern to produce a more open geometry and reduce the self-weight of the structure.

Choice of grid configuration and depth will affect the economy of the space grid as the node joints are usually the most expensive components, therefore, the more there are in a given plan area the higher the cost is likely to be.

Increasing the grid module size reduces the number of joints for a given plan area but there may be adverse consequences. The depth between the two grid layers may have to be increased to accommodate the web members at an appropriate angle and individual members will inevitably be longer. When the longer members are subject to compressive forces they will almost certainly be larger in cross section to avoid buckling and, consequently, heavier and more costly. Where very high rigidity is called for because of large spans, heavy loading or infrequent supports, three-way grids such as the triangle on triangle offset type may be the logical choice.

For multi-layer, three-dimensional grids the use of quasi-crystal geometry has recently been explored.

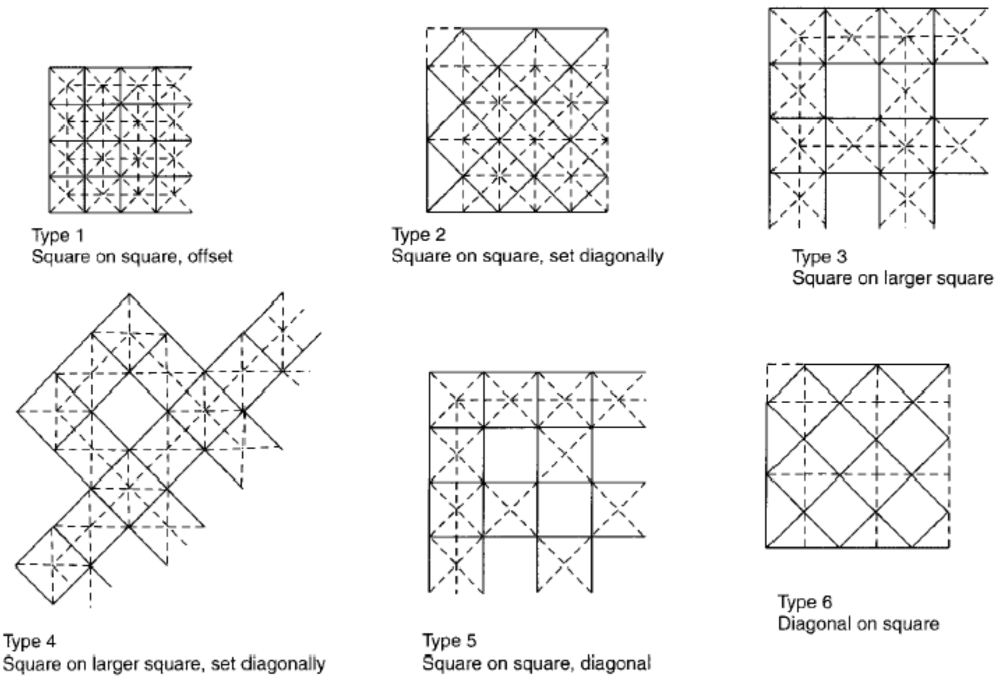



Figure 1.3.4: The six grid arrangements normally used for space frames (reproduced from the book «Analysis, design and construction of Steel space frames»[1])

Quasi-crystals are formed from six rhombic faces of equal side that can be assembled to make both a "fat" and a "thin" version. These can then be combined to make "non-repeating", three-dimensional grids that have a constant member length and dodecahedral nodes that are all oriented in space in the same way.

## 1.4 General construction details

A variety of issues need to be considered in detailing space grid structures.

The main considerations for the detailed design of bars, members, modules, nodes and joints are material properties, element structural behavior and dimensional accuracy. Other issues which must be considered include support details, cladding systems, site construction.

In three-dimensional structures, and long-span structures in particular, dimensional accuracy is of paramount importance as small variations in element dimensions may accumulate to produce gross errors in the dimensions of the final structure. This property is exploited to produce pre-camber of space grids by controlled variation of element dimensions. Components are cut to length with tolerances of less than 1mm and the members of the Nodus and Mero systems, for instance, have their cast end connectors welded to the tubes in accurately dimensioned jigs. The modules for the Space Deck, CUBIC Space Frame and other modular systems are also welded up from accurately cut components in precisely dimensioned jigs. This ensures overall dimensional accuracy for the modules, in this case, in three dimensions.

### 1.4.1 Bearing Joints

Space grids form quite rigid plates so it is very important that any potential movements are accommodated in the support details. A major source of movement in metal structures is change of ambient temperature, especially when long spans are involved. It is necessary to install the space grid on a combination of fixed and sliding bearings to permit free thermal movement whilst restraining the lateral movement caused by wind forces and transmitting these forces to the supports. To hold the structure against lateral loads in any direction a minimum of three lateral restraints are required. The position of the lateral restraints will depend on the distribution and rigidity of the supporting structure. A bearing allowing movement in one direction whilst restraining movement in the orthogonal direction is shown in figure 1.4.1.

Figure 1.4.1: Lateral restraints which permits lateral deformations [3].

Alternatively, the space grid may be rigidly fixed to some or all of its supports; in this case, both the space grid and the sub-structure must be designed to cater for the forces generated by temperature change. Although the structural advantages obtained imposing horizontal rigid restraints, this configuration is not recommended, that is due to the fact that is hard to get a support behavior stiff enough to take care of the huge horizontal force transmitted from the structures.

Typical details for bearing joints are shown in figure 1.4.2. The simplest form of bearing is to establish the joint on a flat plate and anchored by bolts as shown in figure 1.4.2 a and b. This joint seems to be fixed at the support, but in structural analysis it has to be incorporated with the supporting structure, such as columns or walls that have a lateral flexibility. figure 1.4.2 c shows the joint resting on a curved bearing block, which allows rotation along the curved surface. This type of construction is considered as a hinged joint. If a laminated elastomeric pad is used under the joint as shown in figure 1.4.2 d, a new type of bearing joint is formed. Due to the shear deformation of the elastomeric pad, the joint can produce both rotation and horizontal movements. It is very effective to accommodate the horizontal deformation caused by temperature variation or earthquake action.

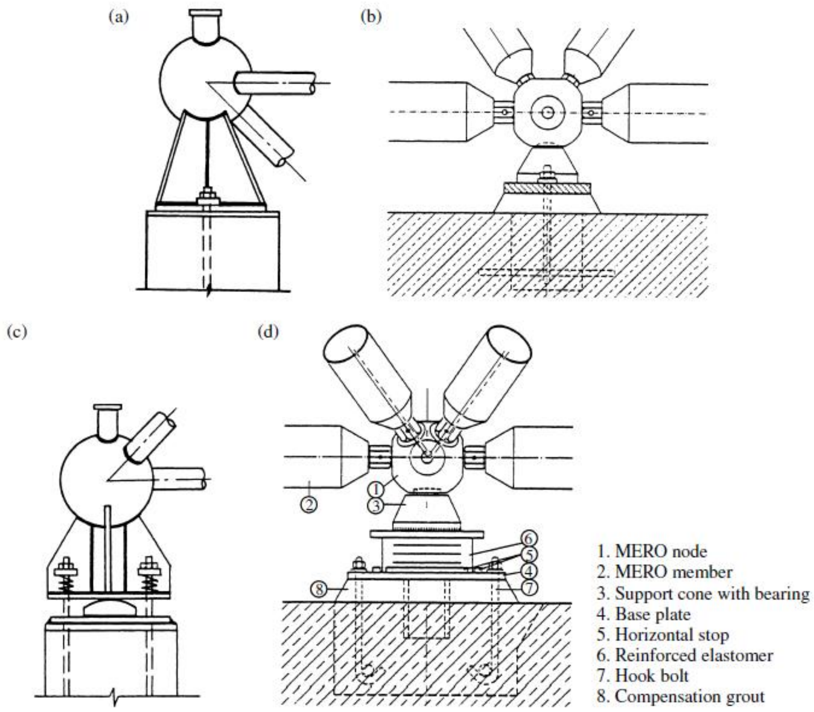



Figure 1.4.2: Bearing joints [2].

### 1.4.2 Method of Support

Ideal double-layer grids would be square, circular, or other polygonal shapes with overhanging and continuous supports along perimeters. This approaches more of a plate type of design, which minimizes the maximum bending moment. However, the configuration of building has a great number of varieties, and the support of the double-layer grids can take the following locations:

1. *Support along perimeters*. This is the most commonly used support location. The supports of double-layer grids may directly rest on the columns or on ring beams connecting the columns or exterior walls. Care should be taken that the module size of grids should match the column spacing.

2. *Multicolumn supports*. For single-span buildings, like sports hall, double-layer grids can be supported on four intermediate columns as shown in figure 1.4.3 a. For buildings like workshops, usually multispan columns in the form of grids as shown in figure 1.4.3 b are used. Sometimes the column grids are used in combination with supports along perimeters as shown in figure 1.4.3 c. Overhangs should be employed where possible in order to provide some amount of stress reversal to reduce the interior chord forces and deflections. For those double-layer grids supported on intermediate columns, it is best to design with overhangs, which are taken as quarter to one third of the midspan. Corner supports should be avoided if possible, since this cause large forces in the edge chords. If only four supports are to be provided, then it is more desirable to locate them in the middle of the sides rather at the corners of the building.

3. *Support along perimeters on three sides and free on the other side*. For buildings of rectangular shape, it is necessary to have one side open, such as in the case of airplane hangar or for future extension. Instead of establishing the supporting girder or truss on the free side, triple-layer grids can be formed by simply adding another layer of several module widths. For shorter spans, this can also be solved by increasing the depth of the double-layer grids. The sectional area of the members along the free side will increase accordingly.

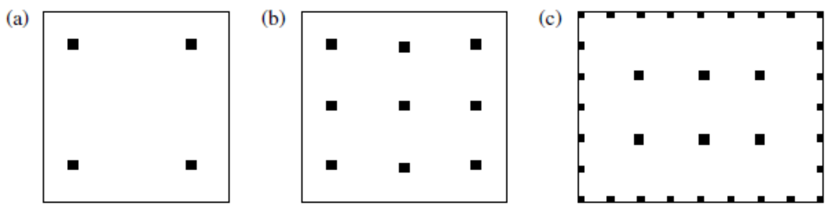



Figure 1.4.3: Multicolumn supports [2].

The columns for double-layer grids must support gravity loads and possible lateral forces. Typical types of support on multicolumns are shown in figure 1.4.4. Usually, the member forces around the support will be excessively large, and some means of transferring the loads to column is necessary. It may carry the space grids down to the column top by an inverted pyramid as shown in figure 1.4.4 a or by triple-layer grids as shown in figure 1.4.4 b, which can be employed to carry skylights. If necessary, the inverted pyramids may be extended down to the ground level as shown in figure 1.4.4 c. The spreading out of the concentrated column reaction on the space grids reduces the maximum chord and web member forces adjacent to the column supports and reduces the effective spans. The use of a vertical strut on column tops as shown in figure

1.4.4 d enables the space grids to be supported on top chords, but the vertical strut and the connecting joint have to be very strong. The use of crosshead beams on column tops as shown in figure 1.4.4 e produces the same effect as the inverted pyramid but usually costs more in material and special fabrication.

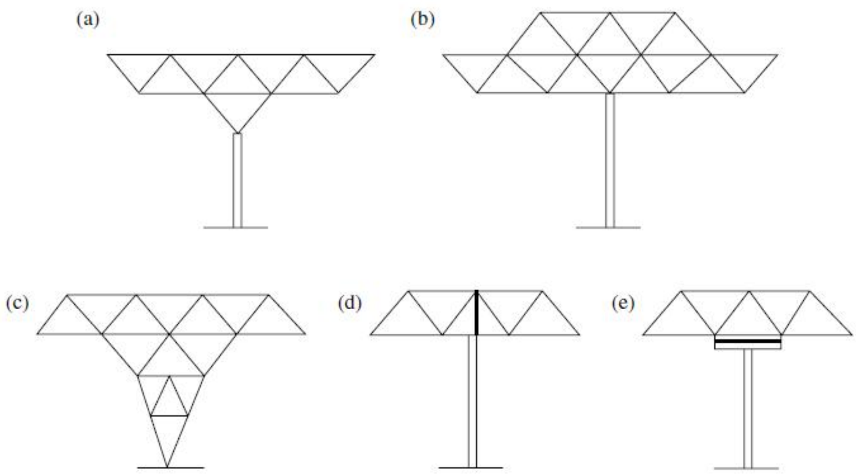



Figure 1.4.4: Supporting columns [2].

## 1.5 Assembly and hoisting

Normally, space frames are assembled in modules at ground level and hoisted up almost vertically by mobile cranes to be placed on columns and walls. The size of the modules depends on the load capacity of the crane and the encumbrance.

There are several alternative methods of erection for space grids and more than one may be used in the construction of a single grid. To some extent the method chosen will depend on the system being used but overall grid size, site access, and component size will also be determining factors.

The most commonly used methods are:

– Assembly of the individual space grid elements or modules on a temporary scaffold support - this is normally only used when no other method is possible as the scaffolding is expensive; however, it may be required in large grids to establish a structurally stable area for later connection of pre-assembled sections.

– Connection in the air, in which space grid elements or modules are lifted individually by crane for connection to areas of the grid that have already been installed - this is more appropriate for heavy modular systems when the site cannot be obstructed by assembly of the grid at ground level and was primarily used for the erection of the CUBIC Space Frame hangar roof at Stansted Airport.

- Assembly of grid elements or modules into panels before lifting by crane and connection in the air - this is a good compromise where it would be difficult to lift the whole space grid as one piece or where it is not possible to assemble the whole grid on the ground, due to lack of space.

- Assembly of the whole grid on the ground before lifting onto the permanent supports by crane in one lift.

- Assembly of a part of the whole grid on the ground before by jacking or winching into position over temporary or permanent supports.

Of these methods the last two are preferred, especially if the space grid components can be manhandled but crane use is expensive and should be avoided where possible. Sometimes this procedure is not feasible and an alternative scheme has to be worked out. Special precautions usually have to be taken to reinforce the diagonal lower chord member squares at their open ends in order to prevent them from deforming into rhombuses. So the squares are provided with temporary tensile elements inserted in the direction in which the squares are predicted to elongate. Because of the open end of the space frame, which does not have a closed geometry, the upward deflection during hoisting has to be considered.

An important advantage may be gained from assembling the grid at or slightly above ground level prior to lifting it to its final position. The installation of services and/or cladding is facilitated as this can be carried out from the ground, eliminating the need for temporary access scaffolding.

Figure 1.5.1: Erection process of the modules [3]

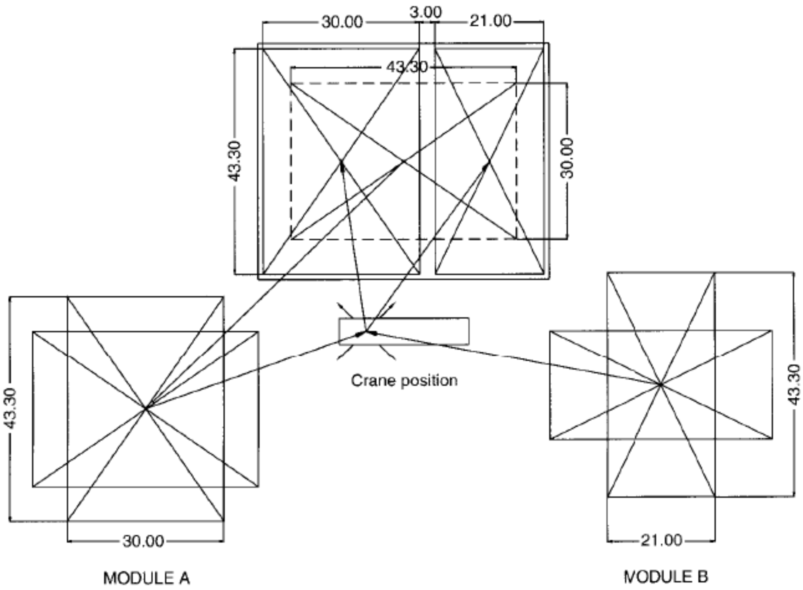



Figure 1.5.2: The erection sequence of a space frame [1]

# Chapter 2

# Advantages of space frames

There are many benefits to be gained from the use of space grid structures, including structural efficiency, reduced deflections, integration of services, resilience, ease of construction, and regularity.

The principal advantages of space grids are as follows:

- All elements of the space grid contribute to the load carrying capacity. This is particularly useful when point loads are applied to the structure. Whereas when planar beams or trusses are employed they must be, individually, capable of carrying any possible point load.

- Loads are distributed more evenly to the supports. This can reduce the cost of the supporting structures especially when heavy moving loads may be applied to the space grid (e.g. overhead cranes). Being light structures, dead loads are very much less and there are consequent savings in columns and substructures.

- Deflections are reduced compared to plane structures of equivalent span, depth and applied loading, assuming that the structural elements are of similar size. Alternatively, a lighter three-dimensional structure may be used resulting in deflections no greater than those of the planar structure.

- The open nature of the structure between the two plane grids allows easy installation of mechanical and electrical services and air-handling ducts within the structural depth. Their fixing is simplified as there is a regular grid of supports available thus reducing or eliminating the need for secondary steelwork.

- The statical redundancy of space grids means that, in general, failure of one or a limited number of elements, for instance, the buckling of a compression member, does not lead to overall collapse of the structure. There have been exceptions to this, notably, the collapse of the Hartford Civic Center, Coliseum, space truss roof in January 1978. It depends which members fail and whether an adequate alternative load path exists.

- Space grid structures are resistant to damage caused by fire, explosion or seismic activity. Unless critical elements (e.g. those adjacent to individual column supports) are removed or weakened by explosion or fire collapse it is unlikely for the same reason as above.

- Modular space grids are usually factory fabricated (thus producing accurate components) easily transportable and simple to assemble on site, since they are put together by using precise, factory-made components, unskilled labour is adequate for their assembly and erection. Because of their modular nature they may be extended without difficulty and even taken down and re-erected elsewhere. Space frames save construction time, because they use factory-produced components that can be manufactured by fast production techniques, transported to the site and easily erected.

- Within reason supports can be located almost anywhere. This give the architect considerable freedom in space planning although approximately square structural bays are preferable. Support location is discussed in a separate section.

- Easiness of construction. Most space grids have a regular grid pattern which may be exploited architecturally to great effect, they offer the architect unrestricted freedom in locating supports and planning the subdivision of the covered space. If the colour chosen for the structure contrasts with the colour of the cladding, or against the sky in fully glazed applications, particularly striking effects can be achieved. In fact the colour chosen for the grid as well as the grid pattern can influence the perceived weight of the structure even more than the actual member sizes.

- No need to extra bracing systems as for the case of planar structures (see section §2.1).

The disadvantages to using space grids are associated with cost, complexity, problems of fire protection, and standardized layout.

There are few disadvantages to using space grids but some are described below:

- The main criticism of space grids is their cost, which can be high when compared with alternative structural systems. This is particularly true when space grids are used for short spans. The definition of a short span is very dependent on the system under consideration but less than 20 to 30m can probably be considered short. For instance, in a portal framed structure additional purlins will be required to support roof decking and secondary steelwork may be necessary to carry services and equipment, neither of which may be needed if a space grid were used instead.

- Visually, space grid structures are very 'busy'. They are rarely seen in plan or in true elevation and at some viewing angles the lightweight structure can appear to be very dense. Grid size and depth as well as the grid configuration can have considerable influence on the perceived density of the structure.
- The number and complexity of joints can lead to longer erection times on site. This is obviously very dependent on the system being used and the grid module chosen.
- When space grids are used to support floors some form of fire protection may be required. This is difficult to achieve economically due to the high number and relatively large surface area of the space grid elements.
- The standardized modular nature of most space grids can impose a geometric discipline of their own. This sometimes makes difficult the use of irregular plan shapes and imposes control on the location of supports.

## 2.1 Space frames or planar structures?

The major characteristic of grid construction is the omnidirectional spread of the load as opposed to the linear transfer of the load in an ordinary framing system. This is the main structural difference between the two systems.

The concept of space frame can be best explained by the example reported in the Handbook of structural engineering (Tien T. Lan 2005) [2]: taking a roof structure for a square building. Fig. 2.1.1 *a* and *b* show two different ways of roof framing. The roof system shown in fig. 2.1.1 *a* is a complex roof composed of planar latticed trusses. Each truss resists the load acting on it independently and transfers the load to the columns on each end. To ensure the integrity of the roof system, usually purlins and bracings are used between trusses. In fig. 2.1.1 *b* latticed trusses are laid orthogonally to form a system of space latticed grids that will resist the roof load through its integrated action as a whole and transfer the loads to the columns along the perimeters. Since the loads can be taken by the members in three dimensions, the corresponding forces in space latticed grids are usually less than that in planar trusses and hence the depth can be decreased in a space frame.

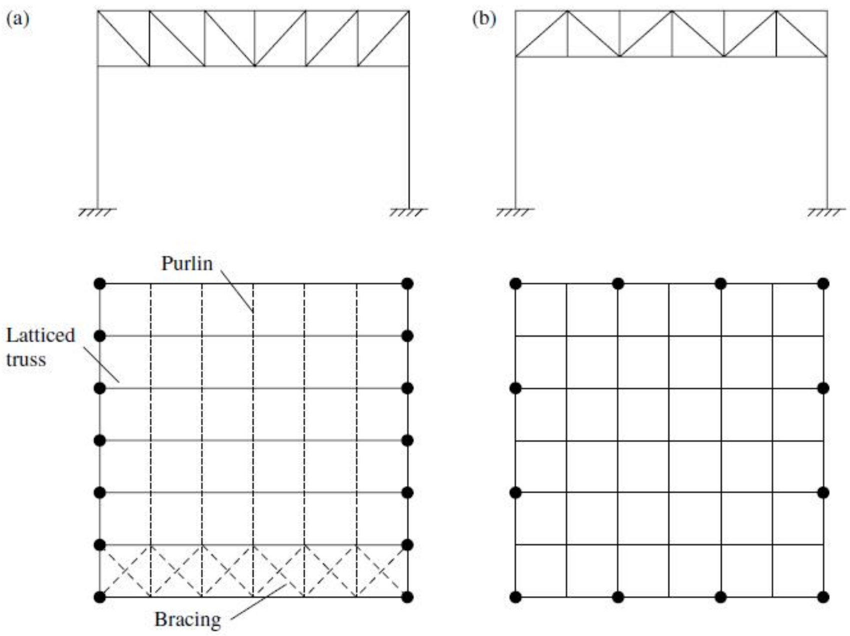



Figure 2.1.1: Example of a roof framing for a square plan [2].

The difference between planar structures and space frames can be understood also by examining the sequence of flow of forces. In a planar system, the force due to the roof load is transferred successively through the secondary elements, the primary elements, and then finally to the foundation. In each case, loads are transferred from the elements of lighter class to the elements of heavier class. As the sequence proceeds, the magnitude of the load to be transferred increases, so does the span of the element. Thus, elements in a planar structure are characterized by their distinctive ranks, not only by the size of their cross-sections but also by the importance of the task assigned to them. In contrast, in a space system there is no sequence of load transfer, and all elements contribute to the task of resisting the roof load in accordance with the three-dimensional geometry of the structure. For this reason, the ranking of the constituent elements similar to planar structures is not observed in a space frame.

# Chapter 3

# Use of timber in space frames

In many ways, timber is very similar to steel as a structural material. Both materials are available in prefabricated shapes and even jointing of timber or steel members respectively, is often comparable. On the other hand, there are marked difference between both materials leading to different design problems. Table 3.1 shows an overview of similarities and differences regarding steel and timber.

| STEEL | TIMBER |
|---|---|
| Similarities | |
| *hollow sections* | *poles* |
| *bars, angles* | *sawn timber* |
| *I-beams* | *rectangular cross sections* |
| *sheets* | *panels* |
| *welding* | *gluing* |
| *bolting* | *bolting* |
| Differences | |
| *isotropic* | *anisotropic* |
| *manufactured* | *grown, graded* |
| *uniform* | *variable* |
| *affected by temperature* | *affected by moisture* |

Table 3.1: Similarities and differences between steel and timber as structural materials [7].

Timber members are particularly capable of acting as tension, compression and bending members. If tension perpendicular to the grain occurs, however, timber is prone to cleavage along the grain. Because of its high to weight ratio it is widely used as a structural material for roof and pedestrian or bicycle bridges.

Compared to steel or concrete, the modules of elasticity is low. This is often counteracted by choosing a stiff structural form, for example I-beams instead of rectangular cross sections for bending members. Another example would be the use of folded plate and shell structures as roofs.

During evolution trees have specialized in resisting their natural environment. In this respect it is a high quality fiber composite, optimally designed to resist loads acting on the tree but also to provide transport of water and nutritional agents. The stem and branches of the tree are designed to resist gravity loads and wind loads. The wood structure is adapted to create maximum strength in stressed directions, whereas in other directions the strength is quite low. Timber has no or very little ductility in the tensile area, while in compression linear elastic–plastic behavior can be assumed. As there is obvious plasticity in compression, some argue that there should also be ductility supposed in bending. A numerous number of researchers proposed to use a plastic rather than elastic behavior for the ultimate bending behavior of timber but also an interaction between the bending and the axial force can make the problem more complex.

Historically space frames have been made using steel members, because of the good structural efficiency that permit to get light structure using thin sections. In several respects, timber as a structural material is similar to steel, this is one of the reasons why in this study it was decided to study this system using structural timber elements instead of steel.

For stress parallel to grain, wood has excellent structural efficiency, when compared to other building materials. A possible criterion for defining such efficiency, in terms of resistance, is the ratio of a yield parameter $f$ of the material (for example the compressive strength) and its density $\rho$ : the value of this ratio is similar to the steel one and is 5 times the concrete one (Table 3.2). These values show that with the same strength it is possible, using wooden elements, greatly alleviate the structure getting many advantages e.g. in seismic field. Equally significant as a structural parameter is the ratio of the Young's modulus $E$ and the yield parameter $f$, which assumes, for timber, values of about one third of those of concrete and comparable with those of steel. This fact is particularly significant for the designers who have to take on the importance of verification of deformation and buckling control, in perfect analogy to what happens in metal structures. Furthermore, the ratio $E^{0,5}/\varrho$, which for timber is about five times the ones of steel and reinforced concrete, shows how it is possible have trees so high with no problems of buckling for self weight.

Due to the ease of workability, timber members can be produced in many sizes and shapes. However, designing timber structures often requires more effort than designing comparable steel or concrete structures. This is caused by the orthotropic properties of timber and by the requirements of mechanical fasteners used to connect timber members. In the fabrication of trussed rafters using punched metal plate fasteners, the design process is automated using Computer Aided Design thus substantially reducing design costs and resulting

| MATERIAL | $f/\rho$ | $E/f$ | $E^{0,5}/\varrho$ |
|---|---|---|---|
| Glue-lam ($GL24$) | $\backsim 63.000$ | $\backsim 480$ | $\backsim 28$ |
| Glue-lam ($L40h$) | $\backsim 80.000$ | $\backsim 400$ | $\backsim 27$ |
| sawn timber (spruce) | $\backsim 183.000$ | $\backsim 147$ | $\backsim 26$ |
| Concrete ($C25/30$) | $\backsim 10.400$ | $\backsim 1.200$ | $\backsim 12$ |
| Steel Fe430 ($f_t = 430MPa$) | $\backsim 55.000$ | $\backsim 480$ | $\backsim 26$ |
| Aluminium ($f_t = 355MPa$) | $\backsim 130.000$ | $\backsim 200$ | $\backsim 26$ |

Table 3.2: Static efficiency of different building materials[4]

in very competitive structures.

## 3.1 Performances of timber structures

Despite of the merits of the material, the design of wood structures is very complex and requires detailed theoretical knowledge accompanied by the intuition and the ability to understand the critical points of the structure. The research on the wood material offers many incentives and the possibility of arriving at some interesting findings. The laminated wood products and wood-based materials on the mechanical characteristics are microscopic, macroscopic and project not yet defined with sufficient accuracy, both for the relative lack of published studies, when compared to those related to other materials, both for the intrinsic complexity of wood material.

### 3.1.1 Wood Strength Definitions

STRENGTH may be defined as the ability to resist applied stress: the greater the resistance, the stronger the material. Resistance may be measured in several ways. One is the maximum stress that the material can endure before "failure" occurs. Another approach is to measure the deformation or strain that results from a given level of stress before the point of total failure. Strength of wood is often thought of in terms of bending strength. This is certainly a useful yardstick of strength but is by no means the only one. A number of other strength criteria are described below.

STRAIN is defined as unit deformation or movement per unit of original length. It is typically expressed in $mm^2$.

ELASTICITY is a property of wood in which strains or deformations are recoverable after an applied stress is removed, up to a certain level of stress known as the proportional limit. Below this point, each increment of stress will produce a proportional increment of strain (the stress/strain ratio is constant) and the wood will return to its original position once the stress is removed. Beyond the proportional limit, each increment of stress will cause increasingly larger increments of strain (as failure is approached) and removal of the stress will only result in a partial recovery of the strain.

MODULUS OF ELASTICITY or Young's modulus is the ratio of stress to strain. Within the elastic range below the proportional limit, this ratio is a constant for a given piece of wood, making it useful in static bending tests for determining the relative stiffness of a board. The modulus of elasticity is normally measured in $N/mm^2$ $[MPa]$ and is abbreviated as $MOE$ or $E$.

MODULUS OF RUPTURE is the maximum load carrying capacity of a member. It is generally used in tests of bending strength to quantify the stress required to cause failure. It is reported in units of $MPa$.

FIBER STRESS AT PROPORTIONAL LIMIT represents the maximum stress a board can be subjected to without exceeding the elastic range of the wood. Permanent set will result if an applied stress exceeds the proportional limit. This property is typically reported in units of $MPa$.

MAXIMUM CRUSHING STRENGTH is the maximum stress sustained by a board when pressure is applied parallel to the grain.

STIFFNESS may be quantified using the modulus of elasticity, E. The higher the E value, the stiffer the wood and the lower the deformation under a given load. A board rated at $2.0 \cdot E$ is twice as stiff as one rated at $1.0 \cdot E$.

COMPRESSION STRESS shortens or compresses the material. For the woodworker, the primary types of compression to consider are parallel to the grain and perpendicular to the grain. Compression parallel to the grain shortens the fibers in the wood lengthwise. An example would be chair or table legs which are primarily subjected to downward, rather than lateral pressure. Wood is very strong in compression parallel to the grain and this is seldom a limiting factor in furniture design. It is considerably weaker in compression perpendicular to the grain. An example of this type of compression would be the pressure that chair legs exert on a wooden floor. If the applied pressure (weight) exceeds the fiber stress at proportional limit for the wood, permanent indentations will result in the floor. Compression stress is measured in $MPa$.

TENSILE STRESS elongates or expands an object. Measurements of tensile stress perpendicular to the grain are useful for quantifying resistance to splitting. Examples of such stress include splitting firewood, driving nails, and forcing cupped boards to be flat. Wood is relatively weak in tension perpendicular to the grain but it is very strong in tension parallel to the grain (visualize a board being pulled from both ends). Due to difficulties in testing and the limited use for such data, tension parallel to the grain has not been extensively measured and/or reported to date. Tensile stress is measured in $MPa$.

SHEAR STRESS involves the application of stress from two opposite directions causing portions of an object to move in parallel but opposite directions. Wood is very resistant to shearing perpendicular to the grain and this property is not measured via a standard test. Wood shears much easier in a direction parallel to the grain - consider a screw running perpendicular to the grain: it will shear out to the nearest end-grain if a sufficiently large force is applied to the board parallel to the grain. Shear stress is measured in $MPa$.

DENSITY is weight per unit volume. For wood, density is expressed as kilograms per cubic meter - at a specified moisture content. Density is the single most important indicator of strength in wood: a wood that is heavier (i.e., more wood substance per unit volume) will generally tend to be stronger than a lighter one.

### 3.1.2 Fire resistance

Although timber is classified as combustible material, a properly designed timber structure has been recognized as performing very well in fire. Timber construction is normally protected from fire by fire resistant cladding materials, while heavy timber construction has good inherent fire resistance because a char layer is formed that retards the heat penetration.

When timber members are exposed to fire, the temperature of the fire exposed surface of the members is close to fire temperature. When the outer layer of wood reaches its burning point (about 300°C), the wood ignites and burns rapidly. The burned wood becomes a layer of char which loses all strength but retains a role as an insulating layer preventing excessive temperature rise in the core.

The low conductivity of char will cause a steep thermal gradient across the char layer. Underneath the char layer, there is a layer of heated wood with a temperature of above 200°C, which is known as the pyrolysis zone. This part of wood is undergoing irreversible chemical decomposition caused solely by a rise in temperature, accompanied by loss of weight and discoloration.

The inner core wood is slightly temperature affected with some loss of strength and stiffness properties, mainly due to the moisture evaporation in the wood. The charring rate is more or less constant and depends on the density and moisture content of the wood and heat exposure.

The fire performance of timber is dependent on the charring rate and the loss in strength and modulus of elasticity. Strength and stiffness properties depend on temperature and moisture content.

The types of timber described include softwoods, hardwoods and glued laminated timbers (glulam), in the forms of solid timber, plywood and wood-based panels. Wood-based panels include wood fiberboard, wood particle-board, medium density fiberboard, oriented strand boards and cement bonded particleboard.

Due to large variation in the type and quality of timber, a system of strength classes has been established to group grades and species with similar strength properties. It gives characteristic strength and stiffness properties and density values for each class (see Swedish classification for glued laminated timber on page 131).

### 3.1.3 Durability

Durability of timber of timber elements is defined as its capacity to perform its designated function for a pre-determined time period. Potential Hazard might be photochemical, chemical, mechanical or thermal degradation , as well as decay and mould fungi, insects, marine borers and bacteria.

The common major risk factors associated with timber are moisture, insects, fungi and ultra violet light. From these, a range of durability issues arise including deformation of members due to moisture movement, premature breakdown of surfaces (including weathering and attack by chemicals), fungal and insect decay, structural performance phenomena such as excessive creep and levels of resistance to wear and tear (including abrasion).

The following are design principles which can be employed to combat durability hazards:

- protective detailing, adequate ventilation, protection from direct and indirect wetting;
- separation from wet materials;
- avoidance of interstitial condensation;
- avoidance of water traps;
- details to accommodate timber movement;
- choice of naturally durable species;

– appropriate preservative treatment and finishes;

– moisture control at installation and maintenance.

## 3.2 Material properties

The strength, stiffness and density of wood vary to a great extent. To be able to use this material in load-bearing structures it is necessary to have a better knowledge of the properties. The wood material is produced by nature and it is therefore not possible to control the variation in properties by changing the manufacturing process. For wood it is instead necessary to get an estimate of the properties and grade the material into different strength classes.

The characteristic strength value for all materials is normally defined as the 5% fractile in the distribution of strength. By grading the material into different strength classes it is possible to:

– improve the control of timber characteristic such as strength and stiffness;

– have a common classification within a market;

– optimize the yield from the raw material;

– optimize the use (good enough quality).

According to most standards the material is graded based on its bending strength. The mean modulus of elasticity in bending and density also have to be checked so that it is within the limits of the grade. All other parameters are estimated based on these values. The relationship between the characteristic bending strength $f_{m,k}$ and other strength and stiffness properties used in the European standards for softwoods are according to *EN338*.

The characteristic values of a strength properties are given from the producer of the glue laminated timber which provides to grade the material with two kind of approach:

– visual strength grading;

– machine strength grading.

Inappendix B the values of the *L40* class (which will be used for the design in this thesis) will be shown.

### 3.2.1 Influence of the properties

The design value of a strength property shall be calculated taking account of:

– the load-duration,

- the moisture influences on strength,
- the material quality,
- the model uncertainties and
- the dimensional variations.

### 3.2.2 Load-duration and moisture influences on strength

The modification factors for the influence of load-duration and moisture content on strength is $k_{mod}$, the values of this factors are given in 3.1.3 of Eurocode 3.

For glued laminated timber, the values of $k_{mod}$ are shown in the following table:

| MATERIAL | SERVICE CLASS | LOAD DURATION CLASS | | | | |
|---|---|---|---|---|---|---|
| | | Permanent | Long term | Medium term | Short term | Instantaneous |
| GLUE LAMINATED TIMBER | 1 | 0,60 | 0,70 | 0,80 | 0,90 | 1,10 |
| | 2 | 0,60 | 0,70 | 0,80 | 0,90 | 1,10 |
| | 3 | 0,50 | 0,55 | 0,65 | 0,70 | 0,90 |

Table 3.3: Values of $k_{mod}$ for glue laminated timber.

### 3.2.3 Partial factor

$\gamma_M$ is the partial factor for a material property. It also takes account for model uncertainties and dimensional variations.

According to the *Eurocode 3*, $\gamma_M = 1,25$ for glue laminated timber.

### 3.2.4 Design value of material property

The design value $X_d$ of a strength property shall be calculated as:

$$X_d = k_{mod} \cdot \frac{X_k}{\gamma_M} \tag{3.2.1}$$

where:

$X_k$ is the characteristic value of a strength property;

$\gamma_M$ is the partial factor for a material property (see 3.2.3);

$k_{mod}$ is a modification factor taking into account the effect of the duration of load and moisture content (see 3.2.2).

## 3.3 Imposition of external constraints

For steel space frames, thermal problems are usually very important to consider because the internal stresses generated could influence a lot the tensional state of the whole structure when long spans are involved. That is the main reason why steel space systems requires carefulness especially for the imposition of the external constraints which have to permit free thermal movement. The restraint of the bearing joint has a distinct influence on the joint displacement and member forces. The construction detail of bearing support should conform to the restraint assumed in the design as near as possible. If this requirement is not satisfied, the magnitude or even the sign of the member forces may be changed. The axes of all connecting members and the reaction should be intersected at one point at the support where a hinged joint is used. This will allow free rotation of the joint. From an engineering standpoint, the space frame may be fixed in the vertical direction, while in the horizontal direction, it may be fixed either tangential or normal to the boundary or both. The way the space frame is fixed often depends on the temperature effect. If the bearing support can allow a horizontal motion normal to the boundary, then the member forces due to the temperature variation can be neglected. In such a case, the bearing should be constructed so that it can slide horizontally. For those space frames with large-span or complicated configuration, especially curved surface structures supported on a sloped base, care should be exercised to ensure a reliable bearing support.

Timber constructions in general do not have the same thermal problems as the steel ones, the thermal deformation of the material is not so important. This property could give some advantages as long as the assumptions are considered appropriately. In fact it is possible to reduce the stresses in the structure by exploiting the horizontal load-bearing capacity of the external constraints.

However it is very hard to get a rigid external support that can carry the horizontal stresses as a perfect fixed hinge, the tension transmitted from the structure is in general very huge and always bring displacement not acceptable considering that condition.

Generally the best way is to model the behavior of the complete structure considering the rigidity of the support system; the advantages however are usually not really different from the theoretical conception of a horizontal roller. Thus, in case of uncertainty of the border conditions and in favor of security, it is recommended to consider the theoretical horizontal free displacement.

## 3.4 Verifications of the members

### 3.4.1 Member in tension

#### 3.4.1.1 Tension in the timber net area

Although the tensile strength $f_{t,0,k}$, of clear wood samples is greater than the compression strength, $f_{c,0,k}$, because tension failure occurs in a brittle rather than in a ductile mode and also because of its sensitivity to the effects of grain slope, knots and other defects, the tensile strength of structural timber is generally less than the compression strength. This is particularly the case for the lower strength classes. Design with respect to tension is normally very simple and the main consideration is the strength value of the wooden member. In some cases it can be relevant to make considerations of the volume subjected to the tensile stresses since the strength is size dependent. In fact, according to the Euro-code UNI 1995-1-1, for rectangular glued laminated timber, the reference depth in bending or width in tension is 600 mm. For depths in bending or widths in tension of glued laminated timber less than 600 mm the characteristic values for $f_{m,k}$ and $f_{t,k}$ may be increased by the factor $k_h$ given by:

$$k_h = min \begin{cases} \left(\frac{600}{h}\right)^{0,2} \\ 1,1 \end{cases}$$

The following expression shall be satisfied:

$$\sigma_{t,0,d} \leq f_{t,0,d} \tag{3.4.1}$$

where:

$\sigma_{t,0,d}$ is the design tensile stress along the grain $[MPa]$;

$f_{t,0,d}$ is the design tensile strength along the grain $[MPa]$.

### 3.4.2 Member in compression

Space frames are often designed fairly slender. Therefore, it is crucial to pay due attention during the design process, in order to avoid problems related to stability. When subjected to an axial load, as the slenderness of the member increases there is a tendency for it to displace laterally and to eventually fail by buckling. Then the main focus of members subjected to compression parallel to grain is on buckling stability.

#### 3.4.2.1 Buckling in the timber frame

As said previously there is a tendency for axial loaded members subjected to compression to displace laterally which eventually can lead to failure by buckling as shown in figure 3.4.1. The more slender the member is the larger the risk for buckling. The slenderness ratio $\lambda$ is defined as the effective buckling length of the member, $L_e$, divided by the radius of gyration, $i$, of the cross section.

$$\lambda = \frac{L_e}{i} \qquad [-]$$

where:

| | |
|---|---|
| $i = \sqrt{I/A}$ | is the radius of gyration about an axis $[mm]$; |
| $I$ | is the second moment of area $[mm^4]$ |
| $A$ | is the cross-sectional area of the member $[mm^2]$; |
| $L_e$ | is the effective buckling length of the member calculated as a function of the real length $L$ and the edge constraints as shown in figure 3.4.1. It is the distance along its length between adjacent points of contra-flexure, where the bending moment in the member is zero. $[mm]$; |

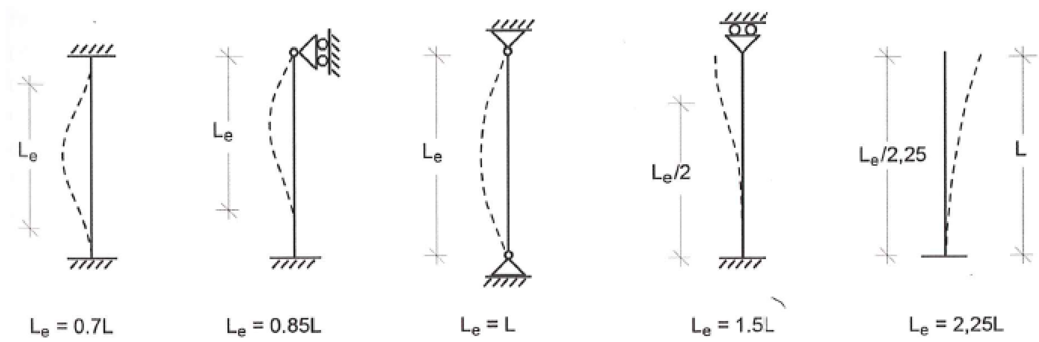



Figure 3.4.1: Effective buckling length $L_e$for different end conditions. $L$ is actual column length.

The Euler buckling load is a theoretical value of the load capacity since no real column is perfectly straight:

$$N_{Cr} = \frac{\pi^2 \cdot E \cdot I}{L_e^2} \qquad [kN]$$

However, other factors will influence the behavior of a column subjected to axial compression such as:

- strength/stiffness — compressive strength and modulus of elasticity of the material;
- geometry of the member — cross-sectional sizes and length;
- support conditions, which are accounted for by effective buckling length;
- geometric imperfections — deviations from nominal sizes, initial curvature and inclination;
- material variations and imperfections — density, effect of knots, effect of compression, wood and moisture content.

This is usually described by introducing a reduction factor factor $\chi$ that depends on the slenderness ratio $\lambda$ and on the above mentioned factors so that the compression strength is given by:

$$N_{c,Rd} = \chi \cdot f_{c,d} \cdot A \qquad [kN] \tag{3.4.2}$$

where:

$f_{c,d}$ is the design value of the compression strength $[MPa]$;

$A$ is the cross section area $[mm^2]$;

$\chi$ is the reduction factor factor $[-]$.

For wooden members the influence of the above mentioned factors have to be considered in the calculations. The reduction factor $\chi$or $k_c$ has been evaluated from simulations for a large number of columns. The columns were modeled by assigning them material properties and geometric imperfections based on observations of real columns. Correlation between different properties was taken into account.

The calculation of the ultimate load for the simulated columns was based on second order plastic analysis, using the plastic deformation potential of timber subjected to compression. Based on the simulations, buckling curves for different circumstances were developed. In order to do so a measure called the relative slenderness ratio $\lambda_{rel}$, was defined

$$\lambda_{rel} = \frac{\lambda}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \qquad [-]$$

where:

$f_{c,0,k}$ is the characteristic compressive strength of the timber parallel to the grain $[MPa]$;

$E_{0,05}$ is the fifth percentile of the elastic modulus $[MPa]$.

The coefficient $\lambda_{rel}$ takes off every dependance from material properties and geometric dimensions in order to get univocally the $\chi$ factor as shown in figure 3.4.2.

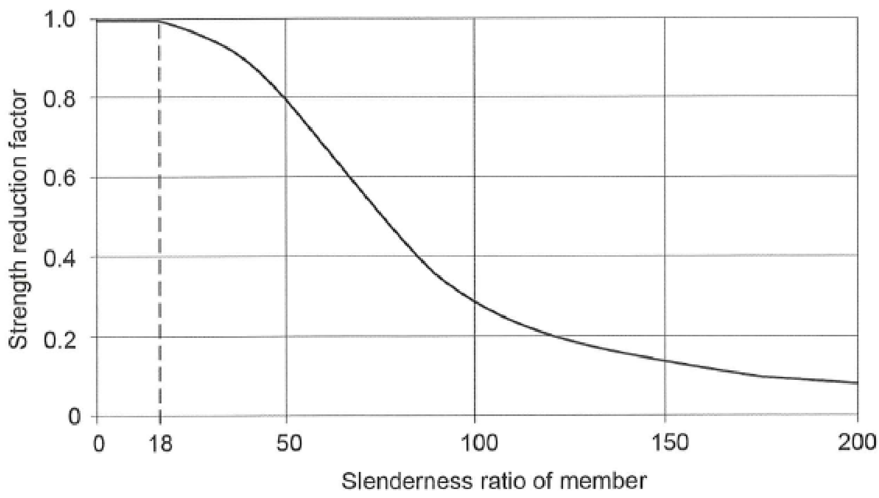



Figure 3.4.2: Typical graph showing the strength reduction $\chi$ or $k_{c,y}$ in a solid timber compression member as function of the slenderness ratio $\lambda_{rel}$.

*Eurocode 5* considers this check in a tensional approach so the expression given for axially loaded members is:

$$\frac{\sigma_{c,0,d}}{k_c \cdot f_{c,0,d}} \leq 1 \tag{3.4.3}$$

where:

$\sigma_{c,0,d}$ is the stress acting on the section of the frame;

$$k_c = \frac{1}{k_y \cdot \sqrt{k_y^2 - \lambda_{rel}^2}};$$

$$k_y = 0,5 \cdot (1 + \beta_c \cdot (\lambda_{rel} - 0,3) + \lambda_{rel}^2);$$

$\beta = 0,1$ per glue laminated timber.

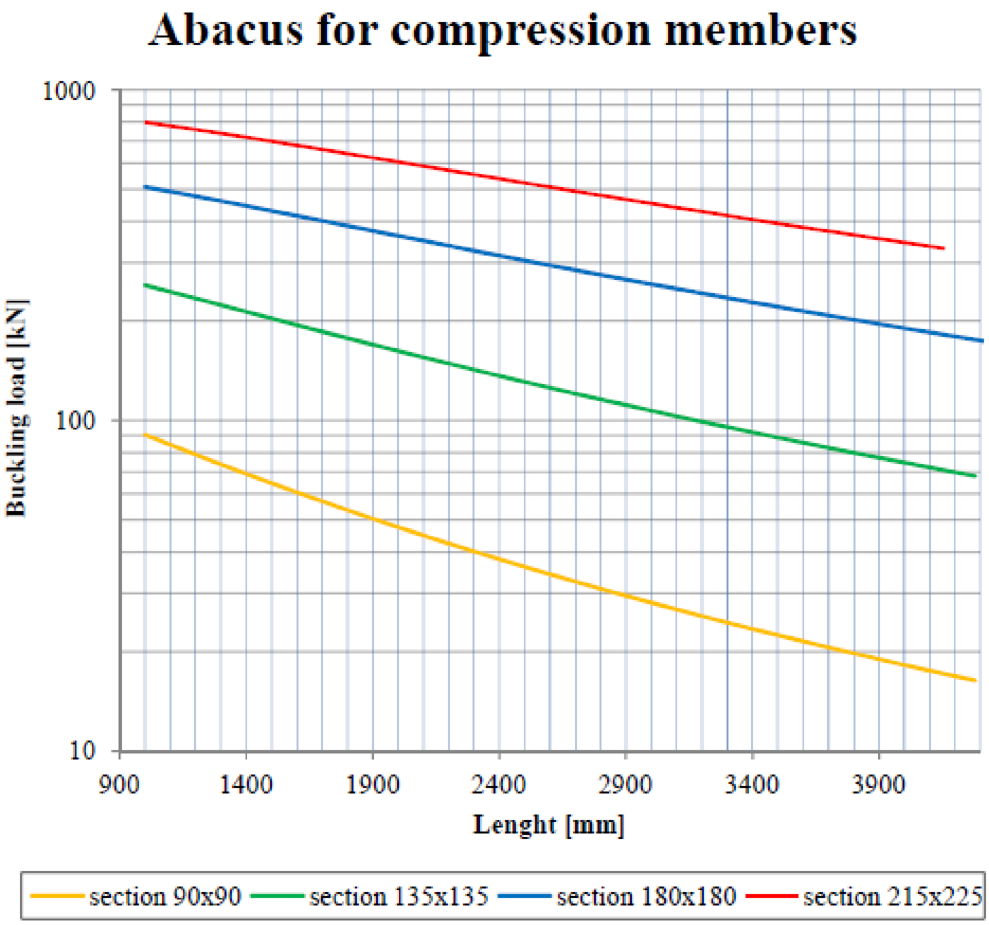



Figure 3.4.3: Abacus for the design buckling resistance $N_d$ for some common section suitable for space trusses.

### 3.4.2.2 Compression in the timber net area

The following expression must always be satisfied especially for the local checks:

$$\sigma_{C,0,d} \leq f_{c,0,d} \tag{3.4.4}$$

where:

$\sigma_{C,0,d}$ is the design compressive stress along the grain;

$f_{c,0,d}$ is the design compressive strength along the grain.

## 3.5 Check of the deflection

The space frame system permits to get a stiff behavior concerning the deformations even with timber members, that is due to the fact that the members work just with normal forces which have the stiffest performances.

The influence of moisture content and load duration is important when designing a timber structure. Engineered wood products, produced of wood and some kind of adhesive, may have different creep properties than sawn timber. In these types of material the creep behavior may be governed by the properties of the adhesives. the creep is larger for some engineered wood products than sawn timber.

Creep is often taken into account in design, the increase in deformation due to creep after an infinite time is definite as creep factor, denoted as $k_{def}$ in *Eurocode 5*.

The final deformation may be calculated taking into account of the creep effect as:

$$u_{fin} = u_{inst} + u_{creep} = u_{inst} \cdot (1 + k_{def})$$

With the help of the creep coefficient it is possible to estimate how large the deformation structure will be after a long time with a constant load. The creep factor $k_{def}$ for glue laminated timber is 0,6 in service class 1.

According to the *Eurocode 5,* the instantaneous deformation, $u_{inst}$ should be calculated for the characteristic combination of actions (see equation (6.2.3)), using mean values of the appropriate moduli of elasticity $E_{mean}$. For structures consisting of members, components and connections with the same creep behavior and under the assumption of a linear relationship between the actions and the corresponding deformations, the final deformation, $u_{fin}$, may be taken as:

$$u_{fin} = u_{fin,G} + u_{fin,Q_1} + u_{fin,Q_i} \tag{3.5.1}$$

where:

$u_{fin,G} = u_{inst,G1}(1 + k_{def})$ for a permanent action;

$u_{fin,Q,1} = u_{inst,Q,1}(1 + \psi_{2,1}k_{def})$ for the leading variable action $Q_1$;

$u_{fin,Q,1} = u_{inst,Q,1}(\psi_{0,i} + \psi_{2,i}k_{def})$ for accompanying variable actions $Q_i$ $(i > 1)$;

$u_{inst,G}$, $u_{inst,Q,1}$, $u_{inst,Q,i}$ are the instantaneous deformations for action $G$, $Q_1$, $Q_i$ respectively;

$\psi_{2,1}$, $\psi_{2,i}$ are the factors for the quasi-permanent value of variable actions;

$\psi_{0,i}$ are the factors for the combination value of variable actions;

$k_{def}$ is the creep coefficient, $0,6$ for glue laminated timber in service class 1.

Different regulations generally propose the following limits of vertical deflection:

$$u_{ist,Q} \leq l/300$$

$$u_{fin,Q} \leq l/300$$

$$u_{fin} \leq l/250$$

where $l$ is the span of the structure.

# Part II

# Connections for timber space frames

# Chapter 4

# Ductility and robustness in timber structures

Timber structures are mostly designed as a set of individual statically determinate elements connected to each other. However, most structures end up being highly statically indeterminate due to the (semi) rigidity of most connections modeled as perfectly pinned in the design phase. Consequently, it can be expected that failures in systems initiate and develop in ways that are inconsistent with the design practice or intent. This can be more problematic than, for example, in steel or reinforced concrete systems because mechanical responses of components in timber structures may be completely different from the expectations.

The large differences in component response do not affect the system response at serviceability level. However, the system response at ultimate limit state can be completely different than intended during the design causing unexpected failures at unexpected load levels.

Furthermore, the connections should be regarded as multiple fastener connections in strength, stiffness and ductility analysis and not as a set of single fastener connections. Depending on the connection geometry, the failure mode may change from ductile to brittle. Nowadays the design theories are based on that the timber and the dowels should behave as an ideal rigid-plastic material. This is, however, not totally reliable due to irregularities in the wood and uneven load distribution between the dowels. The joints instead often show brittle failures, which is a serious problem for dowel-type joints owing to that the collapses of the structure may occur suddenly and dangerously.

This study will focus on special "hinged nodes" that can carry the the loads with the typical behavior of the ductile connections, this behavior is required in order to protect the structure from the intrinsic fragility of the wood material.

It is also of importance to keep a stiff response of the joint during the serviceability limit state. If the joint shows deformations for rather small load the whole structural system will be affected and deformations in the joint contributes to deflections of the whole timber structure.

## 4.1 General notes on ductility

At ultimate limit state, timber structural elements generally fail in a brittle manner. Such behavior is due to the elastic-brittle stress-strain relationship of the material in both tension and shear. Non-linear load-deformation relationships characterized by some plasticization can only be achieved in timber engineering by elements loaded in compression when buckling is prevented, both parallel and perpendicular to the grain, and through connections. In bending, the load-deformation relationship is generally close to linear due to the elastic-brittle stress-strain relationship in tension, which hinders extensive plasticization in compression.

From probabilistic analysis it was demonstrated that ductile structural behavior positively influences structural robustness. It is therefore of importance to investigate possible ductile behavior of structural components for implementation into the probabilistic structural robustness analysis.

Ductility is an important requirement in structural design. Traditional literature references quote three main reasons for achieving ductile behavior:

1. to ensure the failure will occur with large deformations, so as to warn the occupants in the case of an unexpected load (e.g. exceptional snow load, etc.);

2. to allow stress redistribution within a cross-section and force redistribution among different cross-sections in statically indeterminate structures (plastic analysis), so as to increase the load-bearing capacity of the structure with respect to the value calculated in elastic analysis. Plastic analysis can only be carried out for structures which exhibit a minimum amount of ductility;

3. to allow energy dissipation under dynamic loading. Energy dissipation reduces the effect of the earthquake on a structure, leading to an overall better behavior. Roughly speaking, the larger the ductility, the lower the seismic action that has to be considered in design. The seismic actions considered in design are therefore related to the ductility of the structure.

4. to ensure the fulfillment of structural robustness, where the members must be able to accommodate large displacement and rotation demand caused by sudden failure of a single member within the whole structural system.

Under static load conditions, design for ductile failure in the connections is desirable because this allows redistribution of forces between components and sub-assemblies. This is one of the means of avoiding progressive and disproportionate collapse in structural systems. In seismic design, ductile failure in the connections is desirable because mechanical connections are the only significant sources of ductility and energy absorption during cyclic loading.

## 4.2 General notes on robustness

For reduction of the risk of collapse in the event of loss of structural element(s), a structural engineer may take necessary steps to design a collapse-resistant structure that is insensitive to accidental circumstances, e.g. by incorporating characteristics like redundancy, ties, ductility in the structural design. However the designer can also use key elements, alternate load path(s) etc. These issues are strongly related to the robustness of structural systems, which has obtained a renewed interest due to a much more frequent use of advanced types of structures with limited redundancy and serious consequences in case of failure. The increased focus on robustness is also due to recently severe structural failures such as the incident at Ronan Point in 1968 and the World Trade Center towers in 2001. In order to minimize the likelihood of such disproportionate structural failures, many modern building codes consider the need for robustness in structures and provide strategies and methods to obtain robustness. One of the main issues related to robustness of structures is the definition of robustness. The most general definitions are very similar to each other, particularly those taken from codes despite the use of different terms (robustness, structural integrity, but also progressive collapse prevention). These definitions are focused on the prevention of an escalation of damage within the structure, given a certain initial (localized) failure/damage.

Due to many potential means by which a local collapse in a given structure can propagate from its initial extent to its final state, there is no universal approach for evaluating the potential for disproportionate collapse, or for robustness. E.g. it has been proposed to incorporate physical characteristics like redundancy, ties, ductility in the structural design, or by choosing a structural system with inherent key elements, alternate load path(s) etc. In general these characteristics can have a positive influence on the robustness of a structure, however in *Euro-codes* ductility is only awarded for concrete and steel structures and not for timber structures. Structural codes for timber do not award that ductility will result in a semi-rigid behavior plus a higher level of safety due to a lower probability that premature brittle failures occur [9].

# 4.3 Fasteners

## 4.3.1 Ductility of fasteners

Regarding the aspect of timber joints everyone agrees that the way to achieve high ductility is to take advantage of the plasticity of mechanical connectors (nails, dowels, bolts, etc.) and prevent failure in the timber material.

There is a wide variety of fasteners and types of joint details that can be used in timber connections. Two main ways to attach two structural members are either by dowel-type fasteners or by adhesives. Nails, screws, bolts and dowels belong to the first group, all with a common load transfer. This transfer involves both the bending behavior of the fastener and the bearing and shear stresses along the shank. The second type of fastener is when the load transmission mainly is achieved by a large bearing area at the surface of the members. The members in this part are glue and different types of punched metal plates such as shear-plates and split-rings. The stiffness of the joint is depending of what type of fasteners that is used, different fasteners give different load-slip curves, see figure 4.3.1.

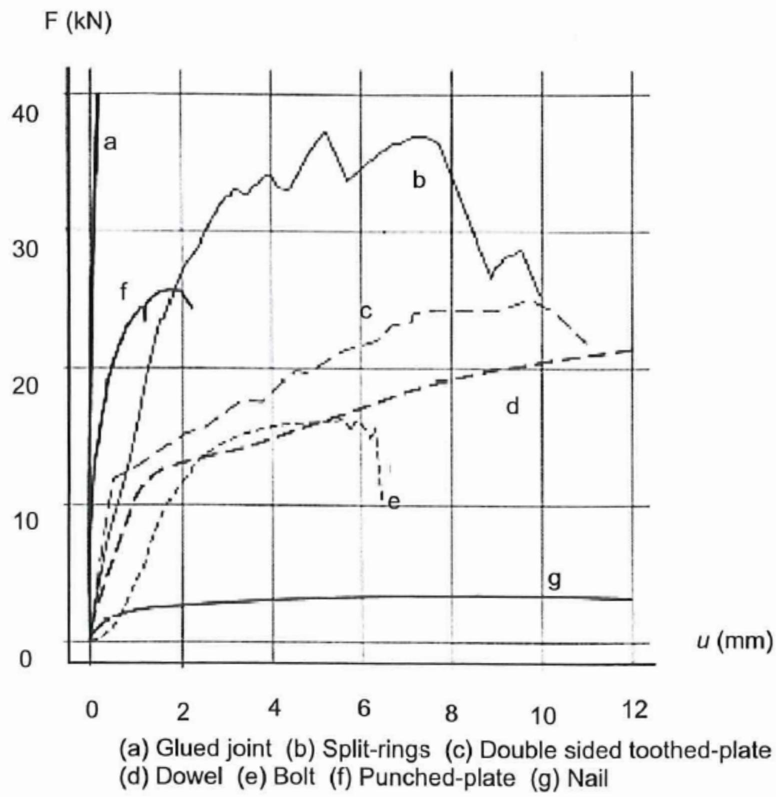



Figure 4.3.1: Typical load-slip curve [13].

### 4.3.2 Dowel-type fasteners

**Nails:** Nails are the most common fasteners used in engineered structures. They are in many cases the most effective fasteners due to the possibility to be driven into place directly without pre-drilling. Other favorable distinguishing features of nails are; they can be placed close together compared to other connectors and they are a relatively cheap fastener. Cold drawing of steel wire gives the nail its characteristic circular cross-section. The withdrawal strength is low compared to other fasteners, but to improve the pullout resistance the nail surface can be corrugated. However, in the other direction, the lateral load—bearing capacity (shear capacity) of the nails is very high. The preferences for these types of connections are based on the expected ductile behavior of nailed joints.

**Bolts:** In more heavy timber structures bolts are to prefer. In this case pre-drilled holes are always needed with an oversize of 1-2 mm compared to the bolt. These connections have a two-side connection or more which means a connection with more than two shear planes to analyst. This will be done by taking the design value of the weakest single shear plane multiplied with the numbers of shear planes. This gives the total connection capacity of the bolted connection.

**Screws:** Other types of dowel-type fasteners are screws. They are often used in single-sided connections when two members connect each other and they are primarily used where the demanded withdrawal strength is high. A limitation to their use is a consequence from the need of pre-drilling, when installing screws with dimension larger than 8 mm, to prevent wood splitting. This is also necessary to avoid torsion failure of the screw itself, caused by the moment when driving the screw.

The use of screws is often important for the great resistance guaranteed from the treated part which gives a lot of possibilities of use. An advantageous way to use the screws will be described in this thesis on sec. 5.3.

### 4.3.3 Adhesives

A glued joint, which is correctly made on a timber surface parallel to the grain, has the same properties as the timber. This means that the joint between two different materials has the same capacity as the weakest part of the materials.

Timber members joined with adhesives are normally manufactured in a controlled environment under the control of a formal quality assurance program. Therefore site-made glued joints are usually excluded from engineered structures because of concerns about quality control on the job site. The quality control is of large importance since a failed glued connection is known as a brittle failure and to avoid that, adhesives in the field are usually used in combination with nails. In large structures such as prefabricated roof trusses, punched metal plates are commonly used. The plates are made of a thin steel plate with a thickness of 1-2 mm with punched-out nails perpendicular to the plane. They can be used in roof structures with spans up to 30-40 meters to join the members together. The plates are embedded into the timber with a hydraulic pressing tool at the factory, and work well as truss connectors. The ability to transfer member forces with small connection areas makes the plates cost efficient regarding savings in materials. The main force is transferred from one member to the plate and from the plate into the next member, therefore the strength of the joint will be decided either by the anchorage in one of the joined members or in the sectional steel area between the members.

## 4.4 Material Parameters

Doweled joints consist in three different members: the steel dowel, the plate and the timber beam. The different joint members are all of great importance to carry the load and have to be evaluated both one by one and as an assembled joint.

### 4.4.1 Timber

Timber is an organic material that has a wide spectrum with different strength classes depending on the timber quality. Due to that the material is anisotropic and inhomogeneous with different strength in different directions, the material parameters will be rather complicated. The highest tensile capacity in timber is tension parallel to the grain and the weakest is tension perpendicular to the grain. This is due to the macroscopic structure of a timber piece, where the wood fibers are all oriented in the longitudinal directions along the tree trunk. These fibers are surrounded by a middle lamella, which glues the fibers together with adhesive lignin. However, the modulus of elasticity is obtained as a kind of average value, obtained from the strength grading process.

Timber also has advantages as a construction material, it has low weight in relation to its strength and it is a material that is not hard to form and shape a member.

The main properties of the material are shown in sec. 3.2 on page 45.

### 4.4.2 Steel Material

Steel is an isotropic material with approximately the same strength in all directions in opposite to timber. The behavior is distinguished by a very stiff response and a high elasticity-modulus in the beginning of the loading sequence. This behavior will have an abrupt end, when the material yields and the strain will increase without any large variation of stress. After a certain strain the microscopic orientation of the steel atoms will find a new stable structure and the material will introduce its strain-hardening interval. Here the material is weaker compared to in the elastic interval and large deformations will appear due to small increases of the loading. The whole typical stress-strain curve for steel is shown in figure 4.4.1 and gives a clear understanding of the behavior of the material in the steel plate and the dowels.

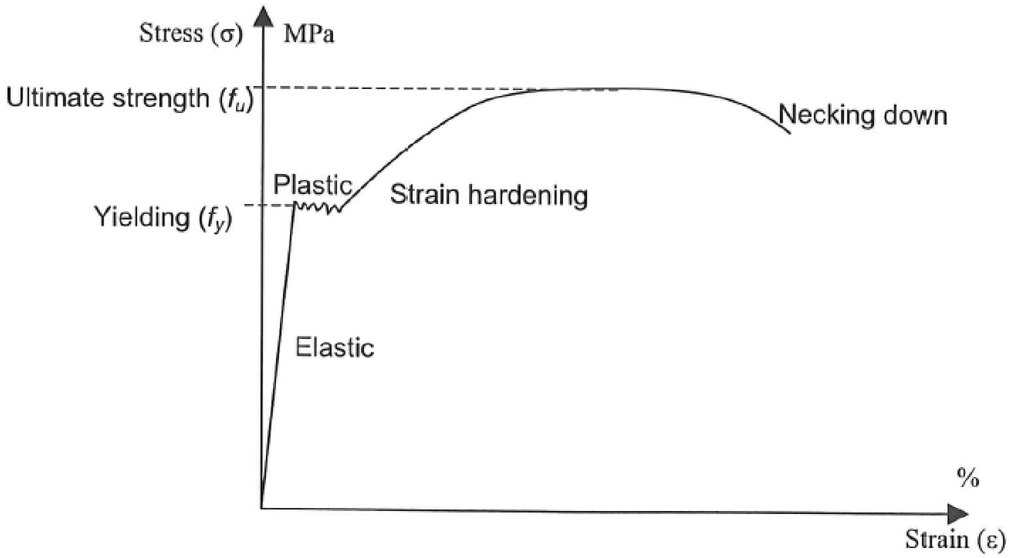



Figure 4.4.1: Typical stress-strain curve for steel [13].

## 4.5 Connections with Preloaded Bolts in the connection

When a connection is subjected to a load reversal or to dynamic loading, a shear connection which acts by shear stress in the bolt and bearing stress in the plates, normally is not suitable. By pretensioning of the bolts, however, a clamping pressure occurs between the connected parts which enables load to be transferred by frictional resistance.

Of course the frictional resistance of a bolt usually is lower than its shear resistance (see part 5.5.1.3 on page 81) but this property can be exploited for the capacity design required in this kind of structure. These bolts behave very stiffly at service loads. In fact they behave almost in the same manner as welded joints. However, when the load applied exceeds the friction generated by the pre-stressing ,a slip occurs and the bolt starts behaving as a "common bolt".

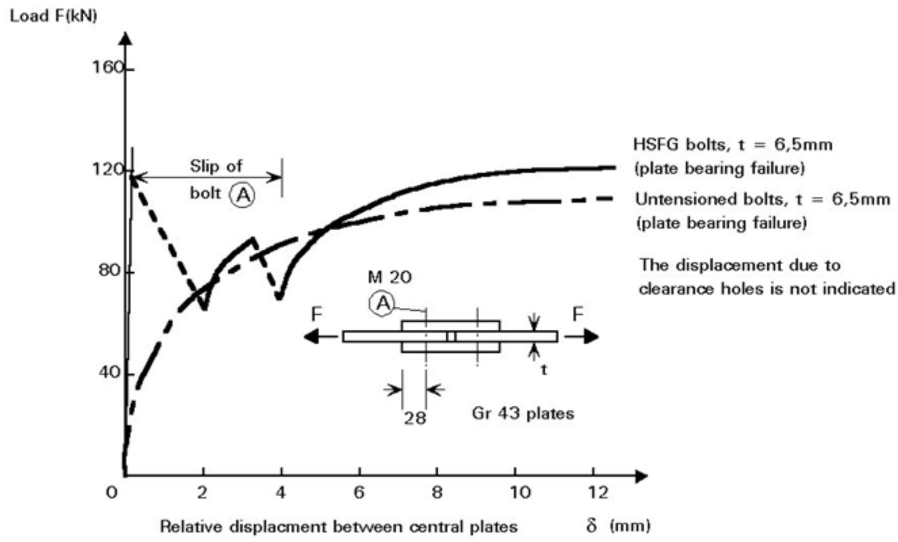



Figure 4.5.1: Comparison of a load-deformation response of a lap joint.

The aim is to apply a pre-load on the bolts enough to carry the forces applied until the end of the serviceability limit state in such a way, slip at bolt will occur before failure, thus allowing for a possible *load redistribution* among other struts converging to the node.

### 4.5.1 Load transmission

Preloaded bolts exert a compressive stress on the connected plates working in shear. The compression gives rise to high frictional resistance, which enables load to be transferred between the connected parts. When the applied load exceeds the frictional resistance, which is developed between the plates, the plates will slip relative to each other allowing the bolt to act in bearing.

Bolts which transfer load by friction are known as High Strength Friction Grip (HSFG) bolts. Controlled tightening of the bolts allows the frictional action to be quantified for design. The main advantages of HSFG bolted connections are their greater stiffness and their ability to withstand alternating forces. Their behavior under fatigue loading is also better than that of bearing bolted connections.

Against these advantages are the costs of HSFG bolted connections. The preparation of the friction grip surfaces and the controlled tightening require additional care (training of people). The costs are greater than for bearing connections. As a result, HSFG bolted connections are usually used only where the stiffness of the connection is important, where alternating loading would cause alternating slip, or where fatigue loading is present.

In this case HSFG bolts will be used in shear connections transmitting the

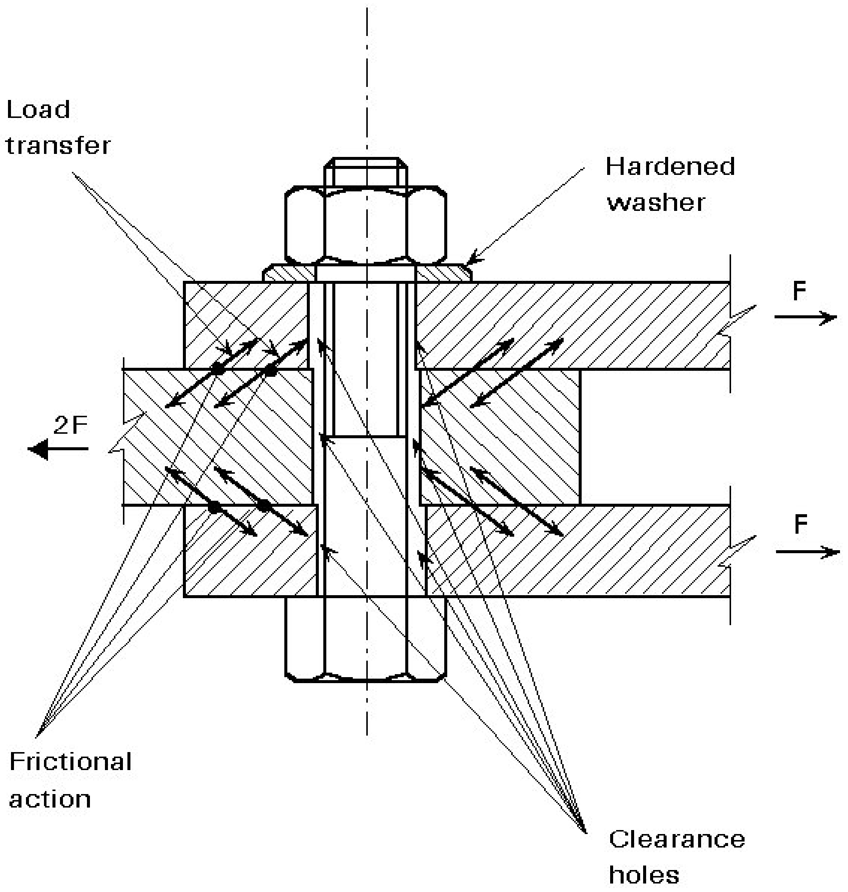



Figure 4.5.2: Load transmission in a shear connection through friction.

force by friction between the contact faces. This requirement is taken at least until the serviceability limit state, when the structure has to maintain its elastic behavior; after that there could be some slip between the bolt and the hole which should be oversized to permit the tolerance during the assembly phase.

## 4.5.2 Resistance of preloaded bolts for the serviceability limit state

The pre-stress to be applied to HSFG bolts is:

$$F_{P,Cd} = 0,7 \cdot f_{u,b} \cdot A_s \qquad [kN] \tag{4.5.1}$$

where:

$A_s$ is the tensile stress area of the bolt $[mm^2]$;

$f_{u,b}$ is the ultimate tensile strength of the bolt $[MPa]$;

The resistance of these connections depends on the preload $F_{P,Cd}$, the slip factor $\mu$ and the number of friction faces $n$. The design slip resistance of a preloaded high strength bolt has a value:

$$F_{s,Rd} = n \cdot \mu \cdot \frac{F_{P,Cd}}{\gamma_{Ms}} \qquad [kN] \tag{4.5.2}$$

where:

$n$ is the number of friction faces $[-]$;

$\mu$ is the the slip factor depending on the preparation of the surfaces. For surfaces not treated $\mu = 0,3[-]$;

$\gamma_{Ms}$ is the the partial safety factor which for the serviceability limit state is $1,1$ and for the ultimate limit state is $1,25[-]$.

# Chapter 5

# Connection system

Since in these kind of structure consists of axial members and connectors, the joints do not have to transfer any outer moment or shear force. One aim of this thesis is to find the solution which summarizes the best performances in terms of stiffness, strength and ductility.

The use of timber as a material for space frame structures has already been tried recently in few cases, both for roofing and bridges of modest importance. In all cases the connections was concepted with large metal dowels or glued joints. The issues related to those types of connection are the excessive rigidity which interferes with the ductility of the entire structure, it causes a limited use of this type of construction and the impossibility of use in structures which require high static performances.

Figure 5.0.1: Glued-joint timber space structure, `www.cenci.com`


Figure 5.0.2: Dowel type joint in a space frame bridge crossing river Isar, Munich Thalrichen *1991*.

## 5.1 Properties required from the connection

- Ductility
- Tolerance
- Resistance for the ultimate state
- Behavior during the service state
- Cheapness of the system

## 5.2 Limits of the connections and hazards to avoid

Wood has drastically different strength properties across the grain than it has parallel to the grain. Across the grain wood is weak, both with respect to a homogenous tensile state of stress and with respect to crack propagation. The tensile strength is only a few percentage of that parallel to grain, and there is a difference in resistance to crack propagation of a similar magnitude.

In structural design efforts are therefore made to avoid tension perpendicular to grain and shear. It is, however, not possible to completely avoid such stresses and several cases of timber structural damage have occurred due to fracture perpendicular to grain. It is probable that more cases of structural damage have occurred due to tension perpendicular to grain than due to tension parallel to grain, although the prime modes of structural loading are bending, tension and compression along the structural member.

A reason for this can be lack of knowledge. Comparatively little material testing and research has been carried out and available methods for engineering strength analysis are comparatively crude. A complication is that several non-trivial factors are significantly affecting both the perpendicular to grain strength and the perpendicular to grain stress, making load-carrying predictions difficult and uncertain. In addition, in practical design it is not possible to distinguish the fiber orientation from the orientation of the structural member. Commonly, it is tacitly assumed that these orientations coincide, which in some cases is far from true and adds to uncertainties and scatter. In codes of timber design, consideration to risk of perpendicular to grain tensile fracture is commonly made by empirically found figures for the load-bearing capacity of the structural element or joint, or by empirical rules such as rules for minimum edge distance in joints.

### 5.2.1 Examples of fracture

As said before, most of the cases of fracture is due to the stress transmitted perpendicular to the grain. Some case of them are illustrated in figure 5.2.1

- Mechanical and adhesive joints commonly give rise to tension across grain in the local vicinity of the joint. One example (5.2.1 d) is the risk of cleavage of the wood in the vicinity of a steel rod glued in parallel to grain. In other examples (5.2.1 e, f and 5.2.1 g), there is direct loading across grain from the connector to the wood. Also, loading along grain (5.2.1 h) is known to produce tensile stress and risk for tensile fracture across the grain. In nail joints (5.2.1 i), fracture may develop even at zero external load due to the wedge action of the nails if the edge distance or the distance between the nails is too small. In adhesive joints (5.2.1 j), tension perpendicular to grain may develop as a result of the geometry of the joint.

- The non-homogenous character of wood and defects such as knots and abnormal anatomy may give tension perpendicular to grain, stress concentrations and strength reduction. As an example, (figure 5.2.1 o) shows how the growth ring induced variation of the orthotropic material orientation may result in tensile fracture when a piece of glue-lam is loaded in compression perpendicular to grain. Figure 5.2.1p illustrates how grain deviation around a knot in the lover edge of a beam may yield perpendicular to grain tensile fracture, and as a result, reduction of the bending strength of the beam.

- As a spherical and troublesome example of perpendicular to grain fracture, (figure 5.2.1 q) shows how a cleavage crack develops during cutting of the stem into pieces when the tree is being harvested by use of a forest machine.

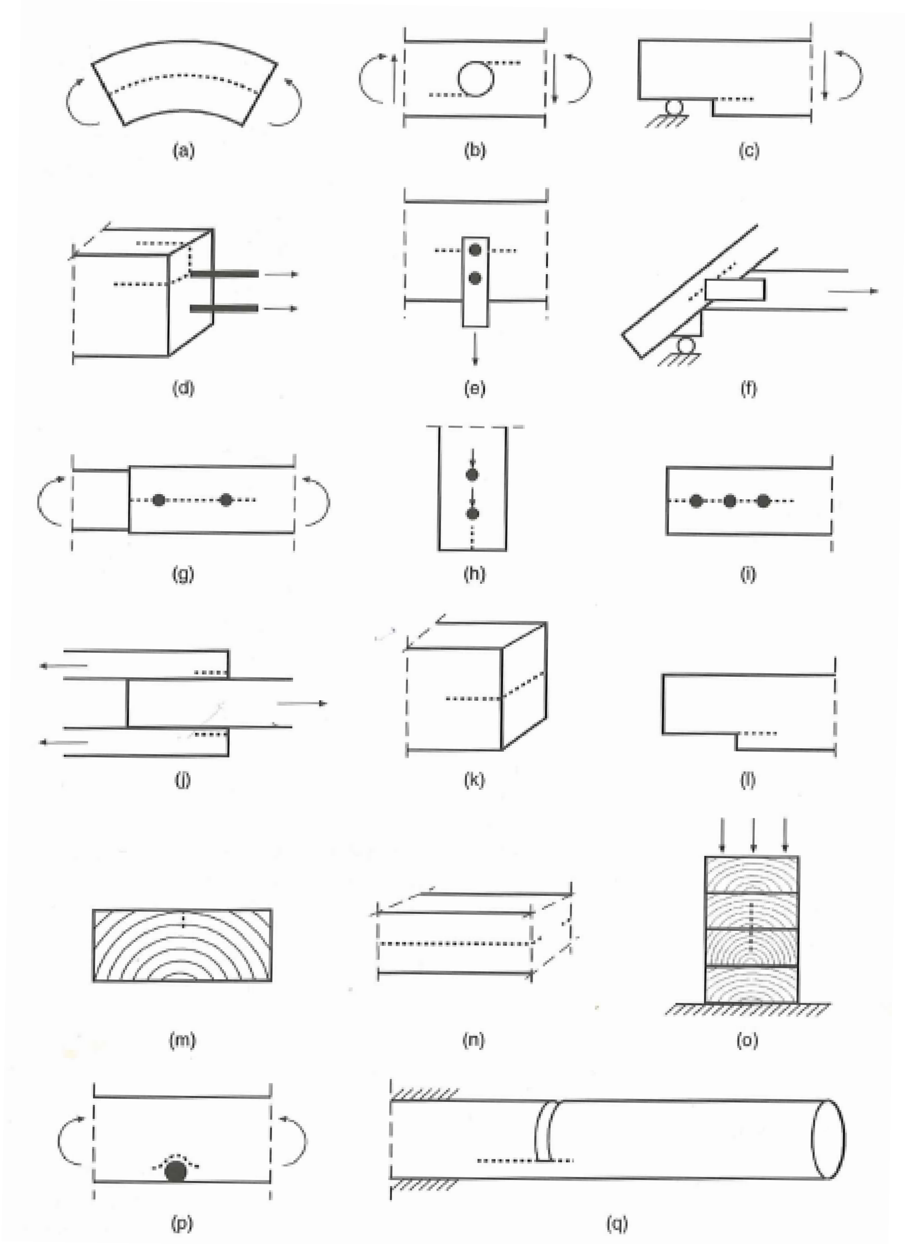



Figure 5.2.1: Various types and causes of fracture perpendicular to grain [7].

- In traditional carpentry joints, not illustrated in figure 5.2.1, there are commonly high local shear stresses that may yield perpendicular to grain fracture and govern the strength of the joint.

## 5.3 Conception of the steel plate system with screws in tension

In order to get a connection with good performances, avoiding the risks elencated in the part 5.2.1 on page 69, it is necessary to find a solution which satisfies both a good behavior in tension and compression but even the properties previously shown.

The system that will be studied consists on a connection with an external steel plate which covers externally the edges of the space frames, fixed with screws inclined 45° to the grain. The solution will be adapted for being used both with bolts working in tension (spherical node MERO) and with the bolt working in shear (plate node Octatube).

**SCREWS:** using the screws inclined 45° to the grain is one advantageous way for tension members.

In this system the tension forces, from the steel plate, are transmitted to the screws through the pre-drilled an shaped holes which distribute the component on every screw head. The Johansen theory can be applied not only to timber-to-timber connections but also to timber-to-steel. The load-bearing behavior is composed of two effects. The first one is the "dowel effect" which depends on the screw resistance to bending and the resistance of the wood to crushing. The second one, the "tensional effect" depends on the screw resistance to tension and on the presence of friction between abutting surfaces.

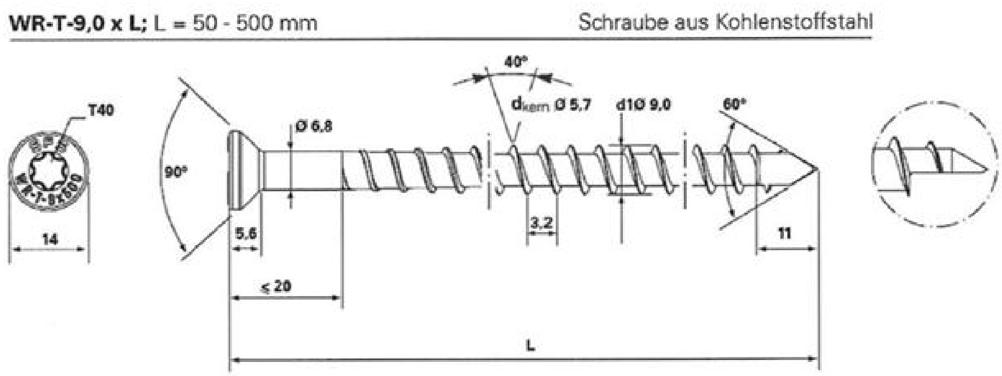



Figure 5.3.1: Screw WR-T-90xL [13].

**STEEL PLATES:** used to connect the frame from the timber element to the node, the dimensions and the thickness are chosen in order to respect the plugging of the screws and the resistance from the stress that might cause both crisis for resistance and buckling. The system must be as simple as possible so the idea is to prepare pre-bended plates, shaped to be immediately applied to the timber element with the screws.

Since the screws, which are the main fasteners of the connection, are insert inclined with an angle of 45° through the plate, it is necessary to prepare appropriate holes that permit the correct propagation of the efforts from the screw-hed to the plate. This requirement is binding on the choice of the thickness to be taken to the plate, in fact, as design choice, it is necessary not to make work with shear the screws but just in tension. To avoid this, it is necessary to prepare pre-drilled holes in the steel clamp with the right inclination and appropriate seats to hold the screw head in the suitable way as figure 5.3.2 shows.

Figure 5.3.2: Example to inclined pre-drilled holes in the steel plate to host the head of the screws

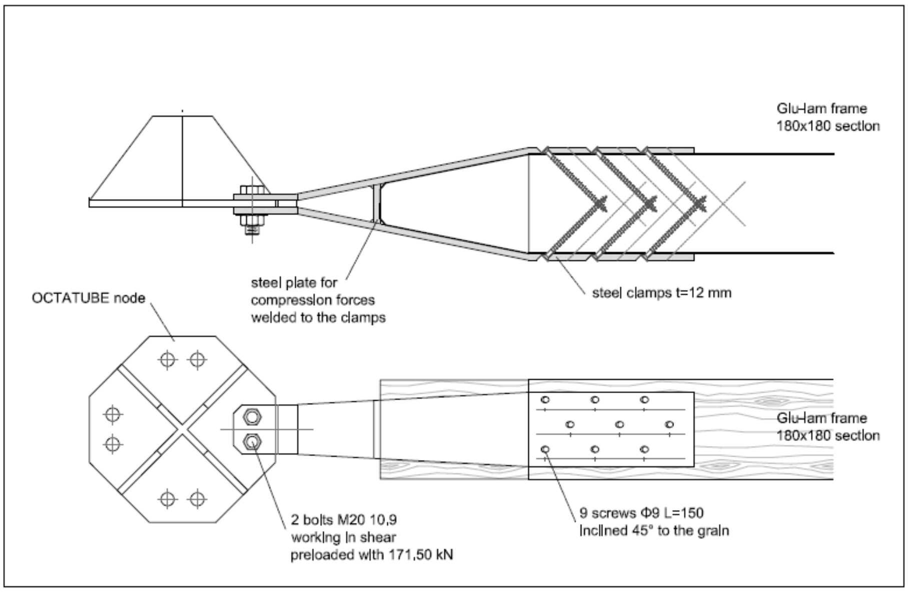
The transmission of the compression forces from the connection to the section of the truss is completely assigned to the plate welded between the clamps constituent the connection, the tapered shape of the frame, is a design choice to permit that the compression stress is more homogeneously distributed to the whole section of the frame and even to permit a gentle curvature of the plates; furthermore it allows to have more space in the area of the node (where usually eight trusses converge).

**CONNECTIONS WITH BOLTS:** There are two possibilities to be able to insert the connections at the end of the plates:

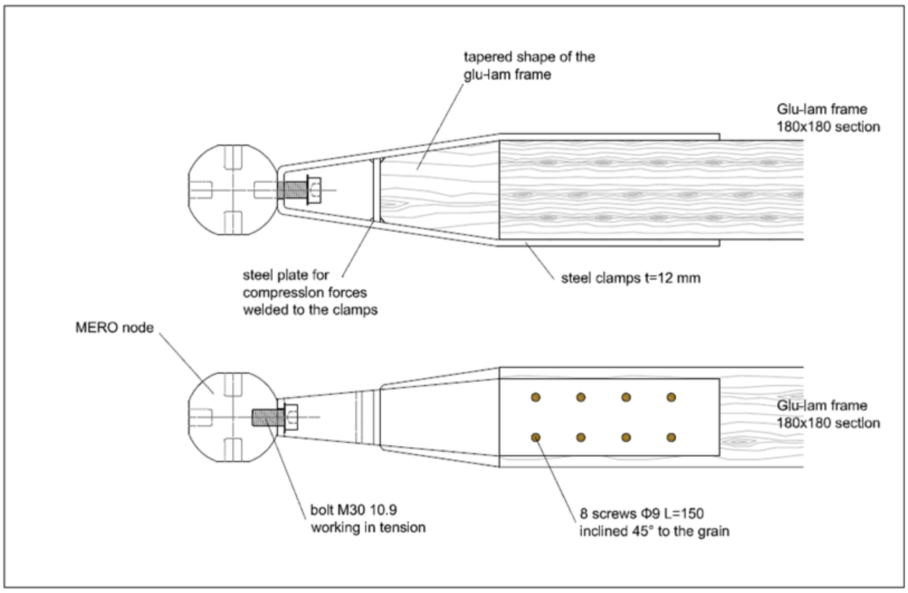
*a)* *Bolt in tension,* connected to a threated spherical node;

*b)* *Bolts working in shear*, connected to a plate node (see Chap. 6 on page 89) .

Where slip is not acceptable in a joint subject to reversal of shear load preloaded bolts in a slip-resistant connection, fitted bolts, injection bolts or other bolts with the same effect should be used.




a) Technology with bolts working in shear




a) Technology with bolts working in tension

Figure 5.3.3: Two possible technologies with the steel plates working with the inclined screws

## 5.4 Forces in the screws under tensile loads

The connection is divided in two steel clamps so the tension force $T_{Ed}$ will be divided by two and distributed in every screw.

The ultimate lateral load of timber-steel joints using inclined screw connectors can be defined using a theory of "yielding" (Johansen yielding theory) which assumes plasticity in both the wood and the fastener. K. W. Johansen first applied the theory of plasticity to dowel-type connectors in wood. Those design criteria for the single connector now form the basis for the design of nailed timber-to-timber joints given in the *Eurocode*. In order to obtain the load bearing capacity of timber-to-steel connection with inclined screws, which are principally loaded in tension, the Johansen theory is extended, taking into the account the withdrawal capacity of fastener and friction between contact interfaces of connected members. In[12], basic equations for the determination of ultimate load bearing capacity of timber-concrete and timber-steel joints with inclined screws, and the comparison between theoretical and experimental results can be found.

For the characteristic withdrawal strength of the screws, the approach of the DIBT (Deutsches Institut für Bautechnik [**?**]) is adopted. The choice is due to the fact that the formulas are derived from experimental results that are considered more reliable than other normative sources.

### 5.4.1 Principal equations and failure modes

**Johansen yielding theory**

The three cinematically possible failure modes and the internal forces and stresses as well as the occurring plastic hinges in the screw for timber-to-concrete joints with inclined screws are schematized in figure 5.4.1.

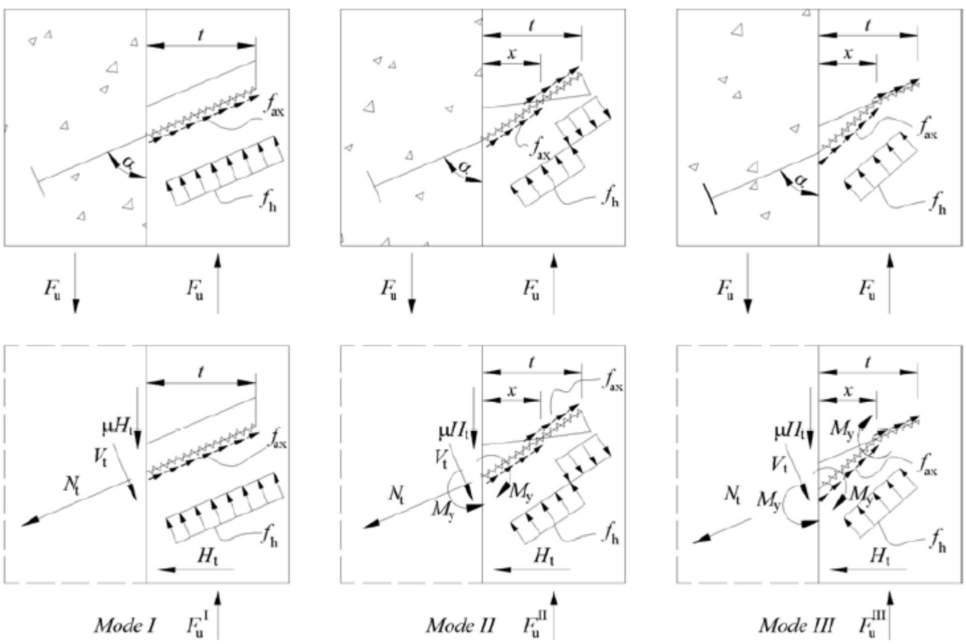



Figure 5.4.1: Stresses in a timber-to-concrete or steel-to-concrete connection with an inclined screw for three Johansen's failure modes [12].

– In MODE I the ultimate load-bearing capacity is reached when the wood yields plastically along the screw.

– The MODE II is realized when the embedment stresses are distributed over the length of the screw so that a plastic hinge at the interface between timber and concrete is formed and the screw rotates as a stiff member in the wood. Such failure mode is possible if the embedded length $t$ of the fastener in wood is enough to enable the plastic hinge formation in the screw.

– The MODE III failure occurs when the embedment stresses are distributed over the length $(t - x)$ of the screw forming an additional plastic hinge.

When the inclination angle between the screw axis and the timber plane is $\alpha = 45°$ and the friction between the interface of the members may be neglected ($\mu = 0$) so the formulation for the three analyzed failure modes may be written as:

$$F_u^I = f_{ak} \cdot d \cdot t + f_h \cdot d \cdot t \qquad Mode\ I \tag{5.4.1}$$

$$F_u^{II} = f_{ak} \cdot d \cdot t + f_h \cdot d \cdot t \cdot \left[ \sqrt{2} \cdot \sqrt{\frac{M_y}{f_h \cdot d \cdot t^2} + 1} - 1 \right] \qquad Mode\ II \tag{5.4.2}$$

$$F_u^{III} = f_{ak} \cdot d \cdot t + \sqrt{2 \cdot M_y \cdot f_h \cdot d} \qquad Mode\ III \tag{5.4.3}$$

where:

$f_{ak}$ is the characteristic withdrawal strength at an angle $\alpha$ to the grain according to the DIBT [**?**]

$f_{ak} = k_{ax} \cdot f_{1,k}$

$f_{1,k}$ is the characteristic withdrawal strength depending on the density of the wood $[MPa]$

$$f_{1,k} = \begin{cases} 90 \cdot 10^{-6} \cdot \rho_k^2 & 6,5 \leq d \leq 9\,mm \\ 80 \cdot 10^{-6} \cdot \rho_k^2 & d = 13\,mm \end{cases};$$

$k_{ax}$ taken into consideration for the coefficient of the angle between the screw axis and wood fiber direction $[-]$

$$k_{ax} = \begin{cases} 0,3 + \dfrac{0,7 \cdot \alpha}{45°} & for\ 0° \leq \alpha < 45° \\ 1 & for\ \alpha \geq 45° \end{cases};$$

$f_h$ is the characteristic embedment strengths in timber according to *Eurocode5* $[MPa]$

$$f_h = \begin{cases} 0,082 \cdot \rho_k \cdot d^{-3} & without\ predrilled\ holes \\ 0,082 \cdot (1 - 0,01d) \cdot \rho_k & with\ predrilled\ holes \end{cases};$$

$M_y$ is the characteristic value for the yield moment according to according to the DIBT **[?]** $[N \cdot mm]$;

$d$ is the outer diameter of thread $[mm]$;

$t$ is the embedded length of the fastener in wood $[mm]$;

$\rho_k$ the characteristic density, $[kg/m^3]$.

Of course the minimum value of the three modes will determinate the load capacity of the fastener:

$$F_{ax,\alpha,k} = min\left\{F_u^I;\ F_u^{II};\ F_u^{III}\right\} \tag{5.4.4}$$

#### THE TENSION STRENGTH OF THE SCREW

The withdrawal strength can not exceed the tension strength of the section of the screw.

$$R_{ax,k} < R_{ax,u,k} \tag{5.4.5}$$

$R_{ax,k} = f_{ak} \cdot d \cdot t$ is the withdrawal strength of the screw $[kN]$;

$R_{ax,u,k}$ is the tensile strength of the screw provided by by the producer, in this thesis will be used the informations given by DIBT Z-9.1-472 **[?]**.

It considers also the pull-through strength of the screw head. For screws used in combination with steel plates, the tear-off capacity of the screw head should be greater than the tensile strength of the screw

### 5.4.2 Design requirement for the screw system

The characteristic value of the single screw, got to the calculation, must be reduced because the influence of many factors in the following way:

$$F_{ax,\alpha,d} = k_{mod} \cdot \frac{F_{u,k}}{\gamma_M} \qquad [kN] \tag{5.4.6}$$

where:

$\gamma_M = 1,3$ is the partial factor for fastener properties $[-]$;

$k_{mod}$ is a modification factor taking into account the effect of the duration of load and moisture content for glue laminated timber in the 2°class of service see Table 3.3 of UNI 1995-1-1 $[-]$.

**NUMBER OF CONNECTORS**

The characteristic withdrawal capacity of connections with axially loaded screws should be taken as:

$$F_{ax,\alpha,t} = n_{ef} \cdot R_{ax,d} \qquad [kN] \tag{5.4.7}$$

For a connection with a group of screws loaded by a force component parallel to the shank, the effective number of screws is given by:

$$n_{ef} = n^{0.9}$$

where:

$n_{ef}$ is the effective number of screws;

$n$ is the number of screws acting together in a connection.

This reduction is due to the influence of the tension perpendicular to the grain that develops in correspondence of the holes. That requirement may be omitted for screws since the holes are very little and, if the distance from the holes in the direction of the fibers is respected, there are not great problems.

**PRE-DRILLED HOLES REQUIREMENTS**

For screws in softwoods with a smooth shank diameter d $\leq$ 6 mm, pre-drilling is not required. For all screws in hardwoods and for screws in softwoods with a diameter d $>$ 6 mm, pre-drilling is required, with the following requirements:

– The lead hole for the shank should have the same diameter as the shank and the same depth as the length of the shank

– The lead hole for the threaded portion should have a diameter of approximately 70 % of the shank diameter.

**DISTANCE OF THE HOLES**

Minimum spacings and end and edge distances for axially loaded screws, see figure 5.4.2, should be taken from Table 5.1.

| Minimum screw spacing in a plane parallel to the grain $a_1$ | Minimum screw spacing perpendicular to a plane parallel to the grain $a_2$ | Minimum end distance of the center of gravity of the threaded part of the screw in the member $a_{1,CG}$ | Minimum edge distance of the center of gravity of the threaded part of the screw in the member $a_{2,CG}$ |
|---|---|---|---|
| $7d$ | $5d$ | $10d$ | $4d$ |

Table 5.1: Minimum spacings and end and edge distances for axially loaded screws according to *Eurocode5*

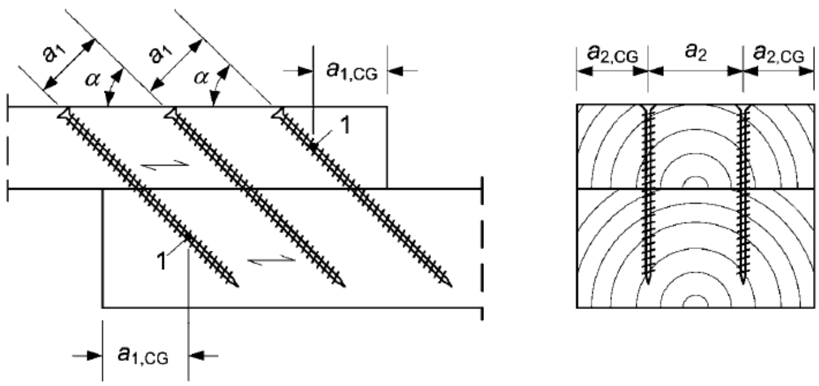



Figure 5.4.2: Spacings and end and edge distances.

# 5.5 Verifications of the steel connection

## 5.5.1 Tension

### 5.5.1.1 Tension in the steel plate

Both in the union with the bolt in tension and the union with bolts working in shear, the connection is divided in two ways so the force will be divided. For this reason the design value of the axial force $N_{Ed}$ acting to the member will be divided by two.

According to the Euro-code UNI 1993-1-1 the design value of the tension force $N_{Ed}$ at each cross section should satisfy:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1$$

For sections with holes the design tension resistance $N_{t,Rd}$ should be taken as the smaller of:

a) the design plastic resistance of the gross cross-section

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \qquad [kN] \tag{5.5.1}$$

b) the design ultimate resistance of the net cross-section at holes for fasteners

$$N_{u,Rd} = \frac{A_{net} \cdot f_u}{\gamma_{M2}} \qquad [kN] \tag{5.5.2}$$

where:

$A$ is the area of the cross section $[mm^2]$;

$A_{net}$ is the net area of the cross section, it should be taken as its gross area less appropriate deductions for all holes and other openings $[mm^2]$;

$f_y$ is the specified minimum yield strength of the steel $[MPa]$;

$f_u$ is the ultimate tensile strength of the steel $[MPa]$;

$\gamma_{M0}$ is the partial factor for resistance of cross-sections $[-]$;

$\gamma_{M2}$ is the partial factor for resistance of cross-sections in tension to fracture $[-]$.

When capacity design is requested, that means getting a ductile behavior, the design plastic resistance $N_{pl,Rd}$ should be less than the design ultimate resistance of the net section at fasteners holes $N_{u,Rd}$.

#### 5.5.1.2 Tension resistance in bolts

Bolts in tension can be used to connect the frame to a treated spherical node (e.g. MERO), in this case a single bolt is usually used that has to carry the whole tension force.

Axial tension resistance of a bolt is based on the stress area $A_s$ and is given by:

$$F_{t,Rd} = 0,9 \cdot \frac{f_{u,b} \cdot A_s}{\gamma_{Mb}} \qquad [kN] \tag{5.5.3}$$

where:

$0,9$ is the result of a statistical evaluation based on a very large number of test results $[-]$;

$A_s$ is the area of the threaded part - stress area[1] $[mm^2]$;

$f_{u,b}$ is the ultimate tensile strength of the bolt $[MPa]$;

#### 5.5.1.3 Shear resistance in bolts

In structural connections, bolts are used to transfer loads from one plate to another. The shear load is transmitted into and out of the bolts by bearing on the connected plates. The principal action on a bolt in a splice joint of the type shown in figure 5.5.1 is shearing on its cross-sectional plane caused by bearing between opposing plates in the joint. The elastic distribution of these bearing stresses and the stresses produced in the bolt are complex. However, for fully developed plastic conditions, the distribution of shear stress is effectively uniform so that the shear strength is the product of the cross-section area of the bolt in the shear plane and the shear strength of the material. If threads are excluded from the shear plane, the shank area may be used. Otherwise the stress area of the threaded portion should be used. In modern detailing practice it is common to use the smaller area and not to contrive to exclude the threads from the shear plane.

The design shear resistance of a bolt ($F_{v,Rd}$) in normal conditions, per shear plane, is:

---

[1] $As = \pi \cdot d_s^2/4$ where $d_s$ is the mean value between the core diameter ($d_c$) and the flank diameter ($d_f$) of the thread.

$$F_{v,Rd} = 0,6 \cdot \frac{f_{u,b} \cdot A}{\gamma_{Mb}} \qquad [kN] \tag{5.5.4}$$

where:

$0,6$ is the result of a statistical evaluation based on a very large number of test results, it is valid only for the shear plane passing through the unthreaded portion of the bolt $[-]$;

$f_{u,b}$ is the ultimate tensile strength of the bolt $[MPa]$;

$A$ is the area of the shank - nominal area $[mm^2]$;

$\gamma_{Mb}$ is the partial safety factor for the bolt $[-]$.

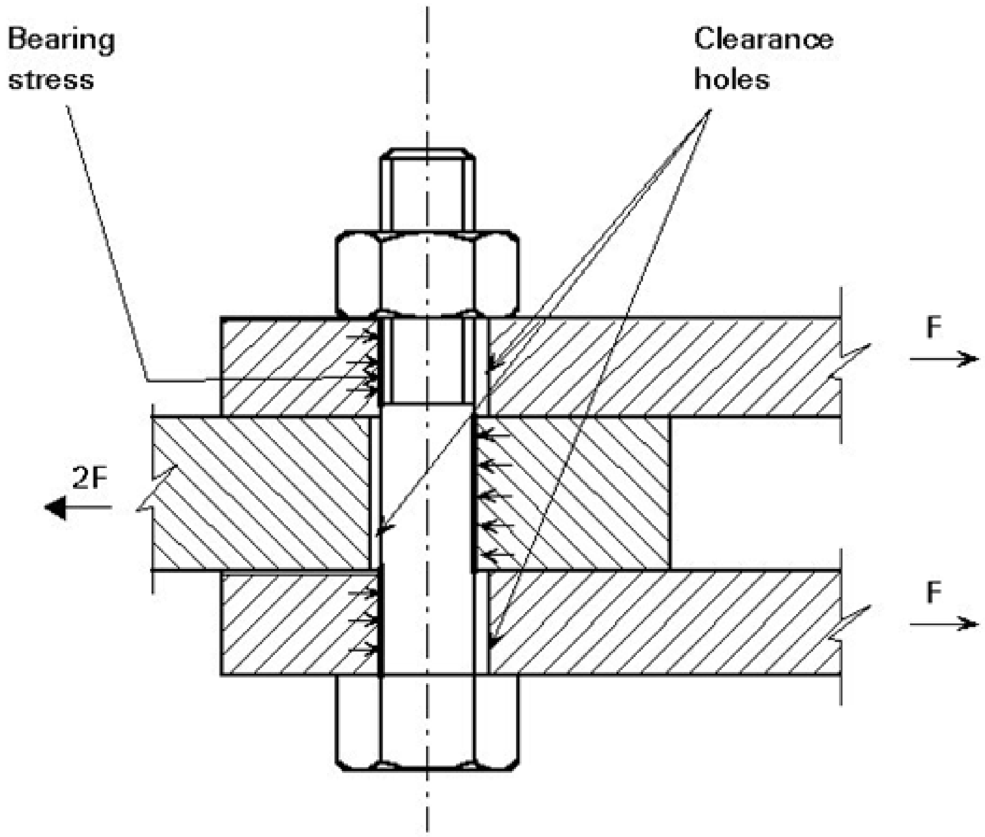



Figure 5.5.1: Load transmission in a splice joint.

#### 5.5.1.4 Bearing resistance of the steel plate

Yielding due to pressure between the bolt shank and plate material may result in excessive deformation of the plate around the bolt hole and possibly some distortion of the bolt. The area resisting the bearing pressure is assumed to be the product of the plate thickness and the nominal bolt diameter.

The distance ($e1$) of the bolt from the end of the plate must be sufficient to provide adequate resistance to the shearing-out mode of failure, which is governed by the area of the shear path.

The design bearing resistance of a bolt is given by:

$$F_{b,Rd} = 2,5 \cdot \alpha \cdot d \cdot t \cdot \frac{f_u}{\gamma_{Mb}} \qquad [kN] \tag{5.5.5}$$

where:

$\alpha$ is the smallest between $\left\{ \frac{e_1}{3 \cdot d_0}; \frac{p_1}{3 \cdot d} - \frac{1}{4}; \frac{f_{u,b}}{f_u}; 1 \right\}$[2] $[-]$;

$d$ is the outer diameter of the bolt $[mm]$;

$d_0$ is the diameter of the hole $[mm]$;

$t$ is the thickness of he plate $[mm]$;

$f_{u,b}$ is the ultimate tensile strength of the bolt $[MPa]$;

$f_u$ is the ultimate tensile strength of the steel element $[MPa]$;

$\gamma_{Mb}$ is the partial safety factor for the bolt $[-]$.

If the net section of the member is small, net section rupture may govern the failure load of the connection. If the shear resistance is greater than the bearing resistance of the plates, *shearing-out* or *tension* failures will occour (see part 5.5.1.1 on page 80). In this case, the deformation capacity of the connection is very large and the joint has a "ductile" behavior. In the other case, when the failure is due to the shearing of the bolts, the deformation capacity of the connection is very small and the joint has a "brittle" behavior.

#### 5.5.1.5 Sliding of the bent plate

Sliding of the bent plate is an important problem to avoid. Because of the folded form, there could be a straightening that, despite of the ductile behavior, reduces the bearing capacity at the serviceability limit state.

[2] This reduction coefficient is necessary because when the end distance is short, the capacity of deformation is small.

A simple and economic system to avoid this problem, could be to insert a welded piece of steel between the steel clamps with the function to keep within the bended edges during tension.

This solution is simply applicable for the first technology (plates working in shear) interposing a welded piece of the same thickness of the node.

For the second technology it will be enough to do the same with a welded plate joining the bent corners, in this case, there will be even advantages for the transmission of the compression from the connector to the member section (it will be seen in the part 5.5.2.1).

In both the cases, the tension carried from the reinforcement interposed is about 15÷20 % of the whole tension of the single plate[3].

## 5.5.2 Compression

### 5.5.2.1 Transmission of the forces from the connector to the member section

The compression is transfered from the connection to the member directly to the section by way of an horizontal plate welded between the two plates.

The tapered shape has been developed to converge the steel clamps to the node and to limitate the encumbrance around the joint space; that permits the meeting of more frames at the same node and facilitates the assembly work.

### 5.5.2.2 Compression to the steel plate

**Resistance** According with the *Eurocode* UNI 1993-1-1, the design value of the compression force $N_{Ed}$ at each cross-section should satisfy:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1$$

The design resistance of the cross-section for uniform compression $N_{t,Rd}$ should be determined as follows:

$$N_{t,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \qquad [kN] \tag{5.5.6}$$

where:

$A$ is the area of the cross section for class 1, 2 or 3 [4] $[mm^2]$;

[3] With an inclination of the taper of about 10°

[4] In case of class 4, see 6.2.4.6.11 UNI 1993-1-1

$f_y$ is is the specified minimum yield strength of the steel $[MPa]$;

$\gamma_{M0}$ is the partial factor for resistance of cross-sections $[-]$.

Fastener holes except for oversize and slotted holes as defined in EN 1090 need not be allowed for in compression members, provided that they are filled by fasteners.

**Buckling** As mentioned before[5], the buckling behavior of steel is very similar to that of wood (see part 3.4.2.1 on page 49).

Both for the plates working in shear and the plates shaped to work with the bolt in tension ( see figures5.3.3 .*a* and .*b* on page 74), it is necessary to check the buckling under compression.

According with the *Eurocode* UNI 1993-1-1, a compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1,0$$

where:

$N_{Ed}$ is the design value of the compression force $[kN]$;

$N_{b,Rd}$ is is the design buckling resistance of the compression member $[kN]$.

The design buckling resistance of a compression member should be taken as:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \qquad [kN] \tag{5.5.7}$$

where:

$A$ is the area of the cross section for class 1, 2 or 3 [6] $[mm^2]$;

$f_y$ is is the specified minimum yield strength of the steel $[MPa]$;

$\gamma_{M1}$ is the partial factor for resistance of members to instability assessed by member checks $[-]$;

$\chi$ is is the reduction factor for the relevant buckling mode $[-]$.

For axial compression in members the value of $\chi$ for the appropriate non-dimensional slenderness $\bar{\lambda}$ should be determined from the relevant buckling curve according the *Eurocode* UNI 1993-1-1 with a similar approach as seen in the part 3.4.2.1 on page 49 for timber.

[5]See on page 41

[6]In case of class 4, see 6.2.4.6.48 UNI 1993-1-1

#### 5.5.2.3 Shear resistance in the bolts

See part 5.5.1.3 on page 81.

#### 5.5.2.4 Bearing resistance of the steel plate

The approach is the same of the part 5.5.1.4 «Bearing resistance of the steel plate» on page 83 but the compression effect is reduced using $\alpha = 3$ in the (5.5.5).

# Part III

# Case study

# Chapter 6

# Space frame with plate connectors (OCTATUBE)

## 6.1 Redesign of an existing steel structure

The case study chosen for the analysis is a space frame with plate connectors for supporting a tennis court at Deira City center, United Emirates [1].

The project included the design and construction of a flat double-layer space frame to support a concrete tennis court. The idea to play tennis on the top of the space frame, faced the engineers with the formidable challenge posed by the abnormally heavy loading and the stringent restriction on deflections under live load. It was quite clear from the very beginning that, to win the contract against stiff global competition, the key to meeting the two conflicting objectives of minimizing weight while ensuring adequate stiffness to keep deflections under control lay in the judicious selection of the appropriate topology.

The concrete, which was poured into steel deck sheets, rests on a tubular space frame, of rectangular plan, measuring 58.8 m x 50.4 m. The structure is supported on perimetral columns, spaced 8.40 m apart. and four interior latticed columns by means of inverted pyramidal 'tree' supports, which are a part of the space frame.

### 6.1.1 Selection of typology

The six topologies normally used are shown in figure 1.3.4 on page 27. The topology of square on square set diagonally (type 2) was opted for the very stiff and structurally efficiency .

The choice of a grid spacing of 2.97 m in the diagonal direction permitted purlins to be spaced 2.1 m, a structural depth of 2.1 m was chosen because it takes the length of the diagonal bracing members equal to the diagonally laid out top and bottom chords. The layout of the top and bottom chords and the member numbering schemes adopted are shown in figure 6.2.5, 6.2.6 and

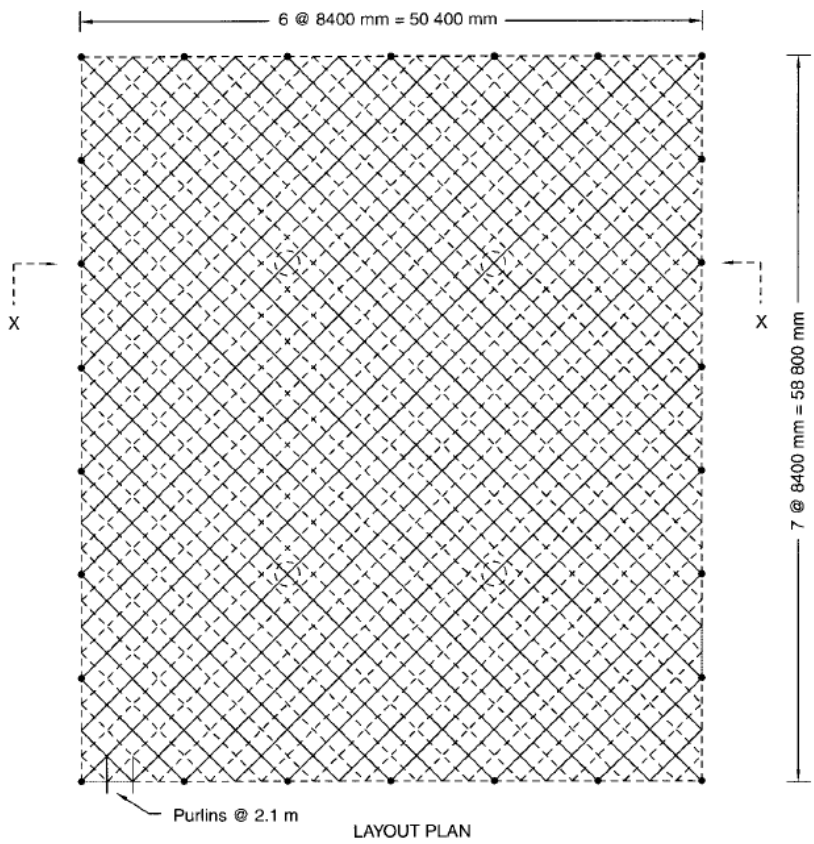



Figure 6.1.1: The space frame plan of Deira City centre [1].

6.2.7.

### 6.1.2 Design choices/strategies

A multi-pronged strategy was worked out to win the contract in the face of stiff international competition. The factors that eventually tilted the award of the contract in their favor were:

- The design using Flocoat tubes of 450 MPa yield strength had an edge over the European designs using tubes of 240 MPa yield strength. This factor helped in pruning the weight of the space frame.

- A further reduction in weight was possible by adopting a square over diagonal topology, which resulted in a drastic reduction in the number of diagonal bracing members and a consequent reduction in weight.

- Flocoat tubes have a superior corrosion resistance due to the rust·resistant treatment that they undergo on-line during manufacture.

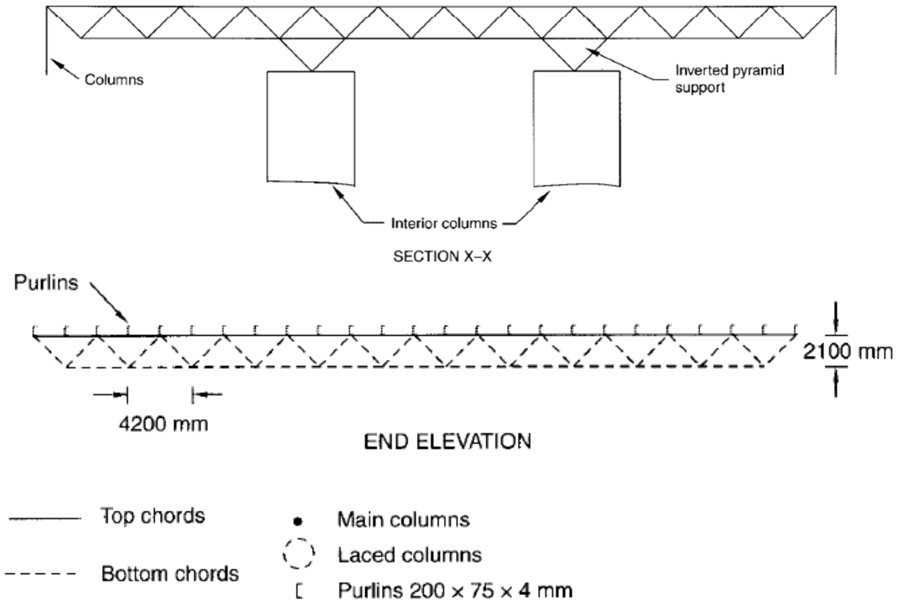



Figure 6.1.2: Support system and section of the space frame of Deira City centre [1].

– There was a substantial reduction in the number of node connectors required. clue to the adoption of a square over diagonal topology that required bottom nodes only in alternate bays.

### 6.1.2.1 Specifications of members

***Top-chords*** tubes of $127.0mm \times 4.50mm$[1].

***Bottom-Chords*** tubes of $127.0mm \times 4.50mm$.

***Diagonals*** it was adopted a distinction between the zones of heavy shear (near the central supports) and the others. For the heavily stressed a tube of $139.70mm \times 5.40mm$, meanwhile for the other a tube of $76.10mmx3.25mm$ were adopted.

### 6.1.2.2 Node connectors

Octatube node connectors ( see on page 22 ). The yield stress of the node plate of grade 40 was 235 $MPa$. Bolts of 20 $mm$ diameter and grade 8.8 were used in 21 $mm$ holes.

[1]Except the member meeting above the central supports that needed a section of 139.70mm×5.40mm

#### 6.1.2.3 Loads

The space frame was designed to carry the following characteristic loads[2]:

- dead loads from concrete and steel deck 150 $kg/m^2$
- estimated weight of installations 50 $kg/m^2$
- weight of space frame, purlins, nodes, stubs and stools 50 $kg/m^2$
- live load 200 $kg/m^2$

#### 6.1.2.4 Code of practice

Design was based on the method of limit states. Design is generally to confirm to *BS 5950: Part 1:1985.* The members was designed in accordance with *Eurocode 3/BS 5950: Part 1:1985* code provisions.

## 6.2 Conception of the same structure in glue-lam

The challenge of this part is to demonstrate how, the technologies studied in the previous chapters, can be used advantageously applied even to this particular case.

It will be utilized the same geometry and grid configuration of the original structure. Only the material used for the trusses will be changed, trying to utilize glue-lam frames with available sections[3].

Even the connection system utilized will be the same (Octatube node connectors), it will be adapted to the shape necessary to the glue-lam technology and dimensioned according to the forces derived from the new analysis. However in this case pre-loaded bolts will be used in order to get a stiffer behavior during the exercise condition (as seen on part 5.3 on page 73).

For the load analysis the same loads will be used provided for the original structure but applied with the new Swedish rules both at the ultimate and serviceability limit states[4].

---

[2]As specified in part 2.14.2.1 of the book Analysis, design and construction of steel space frame [1] it will not be added other wind and snow loads in order to compare the results as faithfully as possible.

[3]The lamella's thickness of Swedish glue-lam is 45 mm

[4]There will be inserted some hypothesis about the service class and load-duration class of the structure, necessary for timber structures according with Eurocode5.

### 6.2.1 Analysis and computer output

#### 6.2.1.1 Load analysis

The same loads will be used provided for the design of the original structure in steel as shown on the preceding page combined according to the current standards given by the EN 1990 *Eurocode 0*.

According to the normative, a distinction shall be made between ultimate limit states and serviceability limit states.

**ULTIMATE LIMIT STATES** The limit states that concern the safety of people and/or the safety of the structure shall be classified as ultimate limit states.

**SERVICEABILITY LIMIT STATES** The limit states that concern the functioning of the structure or structural members under normal use, the comfort of people and the appearance of the construction works shall be classified as serviceability limit states. The verification of serviceability limit states should be based on criteria concerning the aspects of deformations (the appearance, the comfort of users or the functioning of the structure), vibrations (that cause discomfort to people) and damage (that is likely to adversely affect the appearance, the durability or the functioning of the structure).

Design for limit states shall be based on the use of structural and load models for relevant limit states. It shall be verified that no limit state is exceeded when relevant design values for

- actions,
- material properties,
- product properties,
- geometrical data.

The verifications shall be carried out for all relevant design situations and load cases.

Actions shall be classified by their variation in time as follows :

- *permanent actions* ($G$), e.g. self-weight of structures, fixed equipment and road surfacing, and indirect actions caused by shrinkage and uneven settlements ;
- *variable actions* ($Q$), e.g. imposed loads on building floors, beams and roofs, wind actions or snow loads ;
- *accidental actions* ($A$), e.g. explosions, or impact from vehicles.

The characteristic value $F_k$ of an action is its main representative value. For every action, the characteristic value shall correspond to either :

- an upper value with an intended probability of not being exceeded or a lower value with an intended probability of being achieved, during some specific reference period;
- a nominal value, which may be specified in cases where a statistical distribution is not known.

Other representative values of a variable action shall be as the combination value, represented as a product $\Psi_0 \cdot Q_k$, used for the verification of ultimate limit states and irreversible serviceability limit states[5].

The structure will be verified by the *Partial factor method*, in which will be inserted the partial factors ($\gamma_{i,inf}$ and $\gamma_{i,sup}$) for the actions which takes account of the possibility of unfavorable deviations of the action values from the representative values.

The general format of effects of actions should be:

### Ultimate limit states

$$E_{d,sup} = \sum_1^N \{\gamma_{i,sup} \cdot G_{k,i} + \gamma_{i,sup} \cdot \Psi_{0,1} \cdot Q_{k,i}\} \tag{6.2.1}$$

$$E_{d,inf} = \sum_1^N \{\gamma_{i,inf} \cdot G_{k,i} + \gamma_{i,inf} \cdot \Psi_{0,1} \cdot Q_{k,i}\} \tag{6.2.2}$$

### Serviceability limit states

$$E_{d,k} = \sum_1^N \{G_{k,1} + \Psi_{0,1} \cdot Q_{k,i}\}^{\ 6} \tag{6.2.3}$$

The values of the data used for the design of the treated structure are shown in the Tab. 6.1[7].

[5]For buildings, for example, the frequent value is chosen so that the time it is exceeded is 0,01 of the reference period.

[6]In the combinations for serviceability limit state, it is meant that the loads are omitted $Q_{k,1}$ that give a contribution favorable for the purpose of checks.

[7]The values of the and factors for actions should be obtained from EN 1991.

| LOAD TYPE | SYMBOL | CHARACTERISTIC VALUE | $\gamma_{i,inf}$ | $\gamma_{i,sup}$ | $\Psi_{0,i}$ |
|---|---|---|---|---|---|
| Self weight of glue-lam space frames | $G_{k,frames}$ | *450* $kg/m^3$ | 1 | 1.2 | 1 |
| Weight of the node at every joint | $G_{k,nodes}$ | 15 $kg$ | 1 | 1.2 | 1 |
| Dead load from concrete and steel deck | $G_{k,floor}$ | 1,50 $kN/m^2$ | 1 | 1.2 | 1 |
| Estimated load of installation | $Q_{k,inst.}$ | 0,50 $kN/m^2$ | 0 | 1.5 | 0.7 |
| Live load (*Quasi static load*) | $Q_{k,live}$ | 2,00 $kN/m^2$ | 0 | 1.5 | 1 |

Table 6.1: Load characteristic values and coefficients according to *Eurocode 0*.

#### 6.2.1.2 Modeling of the structure

Taking advantage of the double symmetry, it is necessary to analyze just a quarter of the structure using SAP*2000* software. The input data necessary are:

- geometry of the frame system: imported in 3*D* from a .dwg file;
- material properties: class *L40h* (Swedish classification, see appendix B);
- frame section: $180 \times 180\ mm^2$;
- internal constraints: release of the moment capacity of the frames (necessary for space trusses),
- boundary conditions: assignment of the hinges in correspondence of the columns and modeling along the symmetry border.

See appendix A for the detailed description of the finite element analysis with SAP*2000*.

#### 6.2.1.3 Load distribution and combination

The distribution of the loads on the structure is divided on each node of the superior layer (top chords) of the space frame system except the self weight of the frame and the weight of every node (node and connection included). In the analysis will be given particular attention in modeling the assignment of loads more coherent as possible.

The distributed loads and the installation loads shown in the Tab. 6.1 will be concentrated on each node of the top chords simulating the force transmitted from the corresponding area of influence meanwhile the weight of every node will be applied on itself. The self weight of the frames is imposed to the software in order to consider it automatically computed in the analysis.

The areas of influence for every node can be considered the same for each top-node and in this case it is $A_{inf} = 2,97m \times 2,97m = 8,82m^2$ where $2,97m$ is the length of the frames. The forces for the ultimate limit state, are calculated using the 6.2.1 and 6.2.2 in order to get the *superior* ($F_{sup}$) and *inferior* ($F_{inf}$) values necessary for the application of the *Partial factor method*. The estimated forces are shown in the Tab. 6.2.

| LOAD TYPE | POSITION | $A_{inf}$ | $F_{inf}\,[kN]$ | $F_{sup}\,[kN]$ |
|---|---|---|---|---|
| Weight of the node at every joint | *Each node* | - | 0,15 | 0,18 |
| Dead load from concrete and steel deck | *Top* | 8,82 $m^2$ | 13,23 | 15,90 |
| Estimated load of installation | *Top* | 8,82 $m^2$ | 0 | 6,61 |
| Live load (*Quasi static load*) | *Top* | 8,82 $m^2$ | 0 | 26,46 |

Table 6.2: Loads at the Ultimate limit State

In order to provide every condition of stress in each frame it is necessary to choose carefully the distribution of the forces on the area of the court; in fact there could be different cases of distribution of the loads on every partition of the area which may determinate different states of solicitation in the frames (see figure 6.2.1 on the next page). Hence the actions of the frames will be determined from the envelope of every significant case of distribution.

The next paragraph will demonstrate that many elements may be both in tension and compression depending on the case.

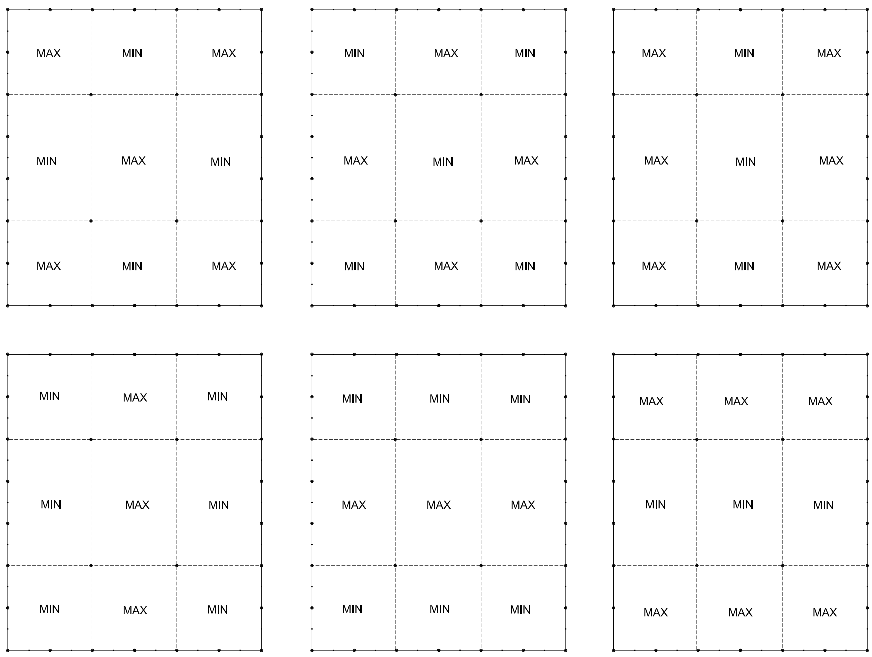


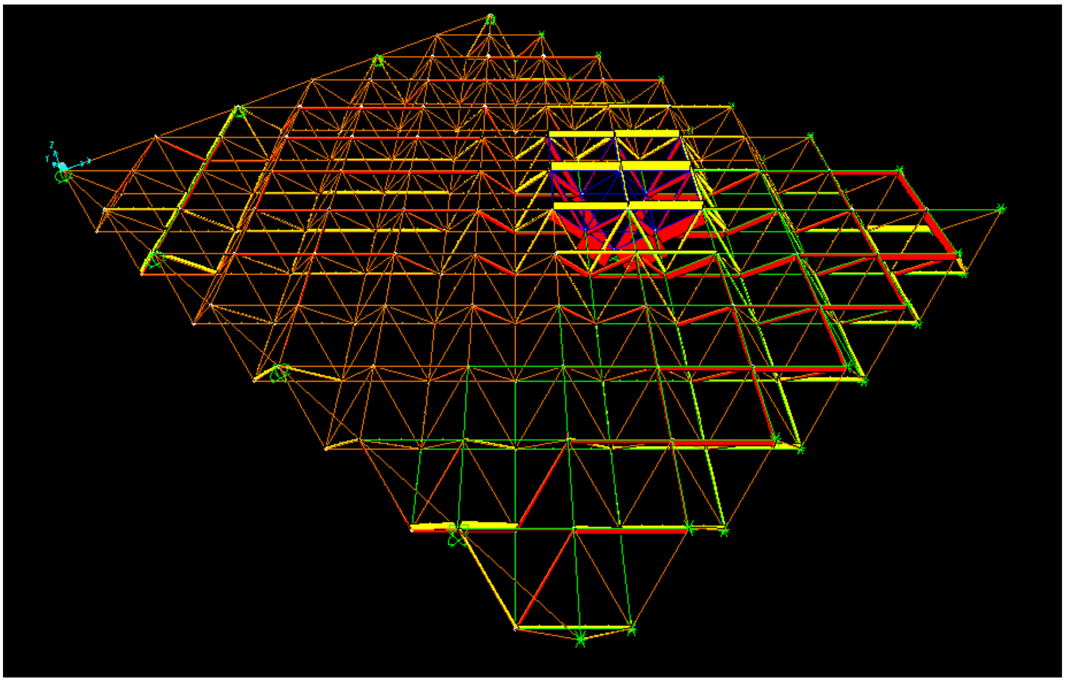

Figure 6.2.1: Main cases of load distribution in the different partitions of the court considered for the envelope.

#### 6.2.1.4 Computer output

After the modeling of the structure and the assignment of the data shown on page 95, it is necessary to apply the Load distribution and combination (6.2.1.3) to the model in order to get the results from the envelope of every load case considered.

As expected from the envelope analysis, which provides the *maximum* and the *minimum* forces from the various combinations, many elements result both in tension and compression; this is due to the fact that different cases of distribution of the loads on every partition of the area determinate different states of solicitation in the frames. In fact, on a number of 714 elements of the model, 434 are in *tension* and 479 are in *compression*.

In the tables 6.3 and 6.4 the main values for every significant frame are shown, both for *tension* and *compression*.



: TENSION ELEMENTS

: COMPRESSION ELEMENTS

Figure 6.2.2: Member forces diagram for frames.

The maximum tension of $318,96\ kN$ was found for the member n° 2769.
The maximum compression of $-334,57\ kN$ was found for the member n° 659.

( See the numbering from page 103 to page 105).

| Frame | Lenght | OutputCase | StepType | Section | P |
|---|---|---|---|---|---|
| Text | [mm] | Text | Text | Text | [kN] |
| 814 | 2970 | ULS envelope | Max | 180x180 | 238,5 |
| 1855 | 2970 | ULS envelope | Max | 180x180 | 252,2 |
| 2753 | 2970 | ULS envelope | Max | 180x180 | 289,9 |
| 2755 | 2970 | ULS envelope | Max | 180x180 | 277,4 |
| 2757 | 2970 | ULS envelope | Max | 180x180 | 260,3 |
| 2759 | 2970 | ULS envelope | Max | 180x180 | 296,1 |
| 2760 | 2970 | ULS envelope | Max | 180x180 | 231,4 |
| 2765 | 2970 | ULS envelope | Max | 180x180 | 245,1 |
| 2769 | 2970 | ULS envelope | Max | 180x180 | 319,0 |
| 2770 | 2970 | ULS envelope | Max | 180x180 | 265,7 |
| 2795 | 2970 | ULS envelope | Max | 180x180 | 304,1 |
| 2797 | 2970 | ULS envelope | Max | 180x180 | 260,5 |
| 2799 | 2970 | ULS envelope | Max | 180x180 | 246,8 |
| 2801 | 2970 | ULS envelope | Max | 180x180 | 255,0 |
| 3380 | 2970 | ULS envelope | Max | 180x180 | 234,5 |
| 3556 | 2970 | ULS envelope | Max | 180x180 | 257,9 |

Table 6.3: Tension values of the main elements

| Frame | Lenght | OutputCase | StepType | Section | P |
|---|---|---|---|---|---|
| Text | [mm] | Text | Text | Text | [kN] |
| 617 | 2970 | ULS envelope | Min | 180x180 | -278,1 |
| 625 | 2970 | ULS envelope | Min | 180x180 | -298,9 |
| 627 | 2970 | ULS envelope | Min | 180x180 | -316,1 |
| 635 | 2970 | ULS envelope | Min | 180x180 | -295,4 |
| 649 | 2970 | ULS envelope | Min | 180x180 | -334,6 |
| 652 | 2970 | ULS envelope | Min | 180x180 | -266,3 |
| 659 | 2970 | ULS envelope | Min | 180x180 | -334,6 |
| 705 | 2970 | ULS envelope | Min | 180x180 | -261,5 |
| 707 | 2970 | ULS envelope | Min | 180x180 | -263,7 |
| 713 | 2970 | ULS envelope | Min | 180x180 | -319,2 |
| 715 | 2970 | ULS envelope | Min | 180x180 | -280,1 |
| 1059 | 2970 | ULS envelope | Min | 180x180 | -251,9 |
| 1965 | 2970 | ULS envelope | Min | 180x180 | -252,7 |
| 2845 | 2970 | ULS envelope | Min | 180x180 | -283,8 |
| 2850 | 2970 | ULS envelope | Min | 180x180 | -327,3 |
| 2852 | 2970 | ULS envelope | Min | 180x180 | -333,1 |

Table 6.4: Compression values of the main elements

### 6.2.2 Conception and design of the members

The members are designed in *L40h* class glue-lam according to the Swedish classification[8]; this high-performance class is available especially for small section, composed just to few laminations. In fact the 180×180 section is composed of four *45mm* laminations. This resistance class permits to get better performance both for tension and compression than other lower classes.

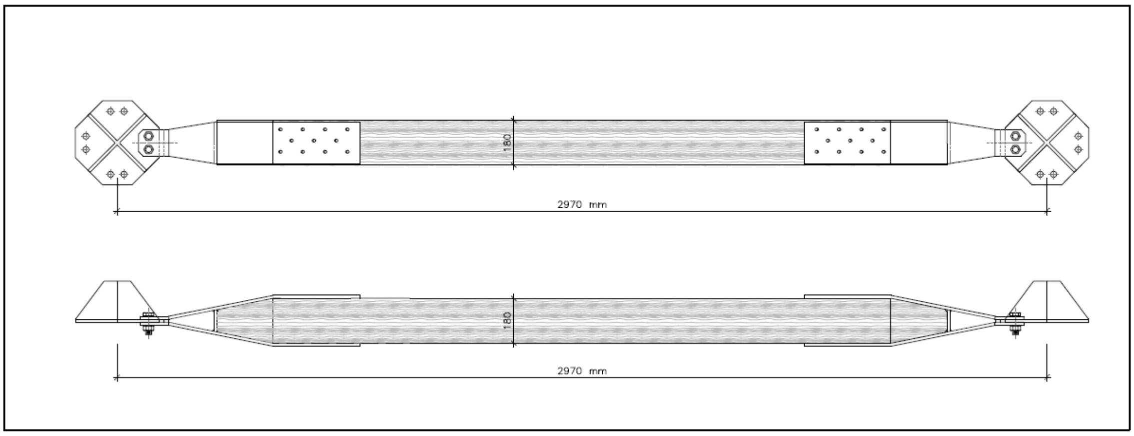



Figure 6.2.3: Glued laminated timber truss with OCTATUBE connectors.

Since the live loads (tennis court) is quasi-static , we consider to have a short term duration load which provides a $k_{mod} = 0,9$ (see tab. 3.3 on page 46). The partial factor for material property $\gamma_M$ for glue laminated timber is $1,25$.

In order to standardize as much as possible the assembly of the frame elements, the connection system is chosen to be the same of every cross section. The plate connector material is *S355J0* Steel ( $f_{y,k} = 355MPa$ and $f_{u,k} = 490MPa$ ) with a thickness of 12 mm.

Since the forces in the members are very different, at least two different kinds of connections will be used, changing the number of the screws required and trying to find out a logical arrangement which facilitates the assembly work.

**A TYPE**

This system will be used for members subjected both to tension and to compression with forces below 180 kN (see figure 6.2.4 on page 102):

- The plate connector is in *S355J0* Steel ( $f_{y,k} = 355MPa$ and $f_{u,k} = 490MPa$ ) with a thickness of 12 mm.
- The connection between the plate connection and the node is achieved by two M20 bolts 10.9 ( $f_{u,k} = 1000MPa$ ).

[8]See par. «Swedish classification for glued laminated timber» on page 131.

- The connection between the plate and the frame is provided with $9 \times 150$ screws, 9 per side, inserted with an angle of 45° to the grain.

**B TYPE**

This system will be used for members solicited both for tension and compression with forces exceeding 180 kN (see figure 6.2.4 on the following page):

- The plate connector is in *S355J0* Steel ( $f_{y,k} = 355MPa$ and $f_{u,k} = 490MPa$ ) with a thickness of 12 mm.
- The connection between the connection and the node will be composed by two M24 bolts 10.9 ( $f_{u,k} = 1000MPa$ ).
- The connection between the plate and the frame is provided with $9 \times 200$ screws, 11 per side, inserted with an angle of 45° to the grain.

The lay-outs proposed for the two kinds of connections are shown in figure 6.2.5, figure 6.2.6 and figure 6.2.7. The checks of the trusses, regarding the member part and the connection part will be carried out according to the rules shown in the sections 3.4 and 5.5 of this thesis.

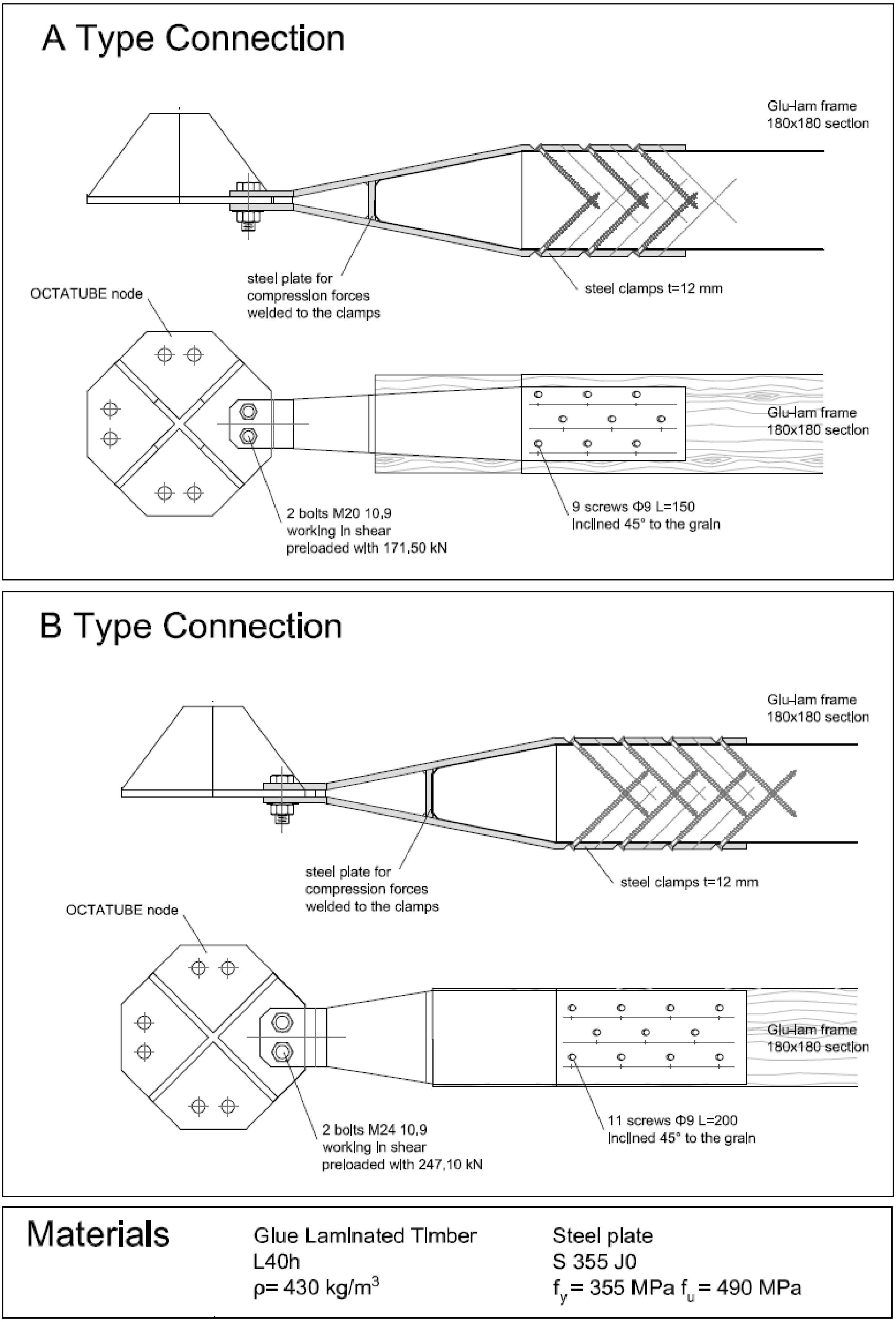



Figure 6.2.4: OCTATUBE connection system.

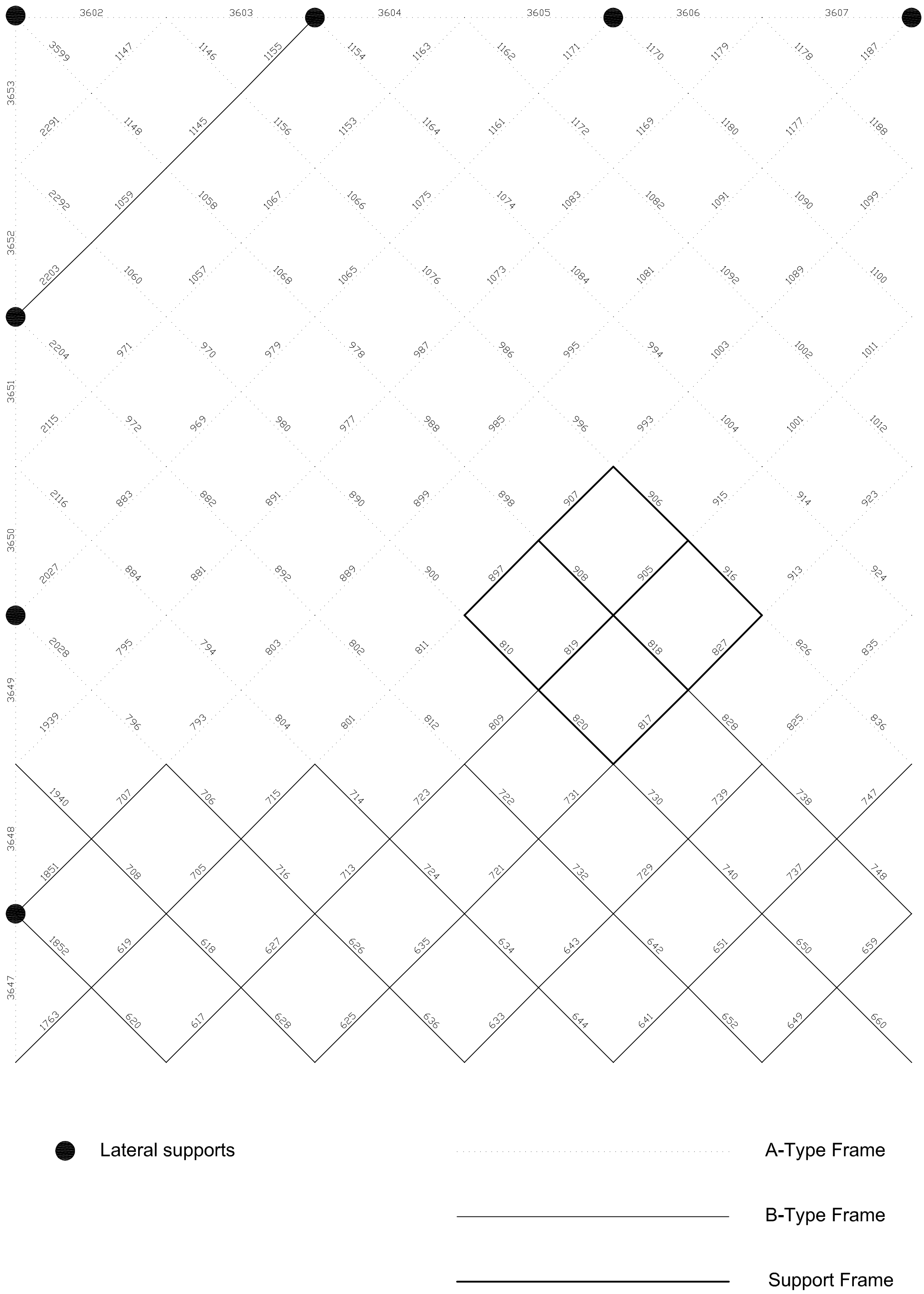



Figure 6.2.5: Quarter of plan showing members numbering: **Top Chords.**

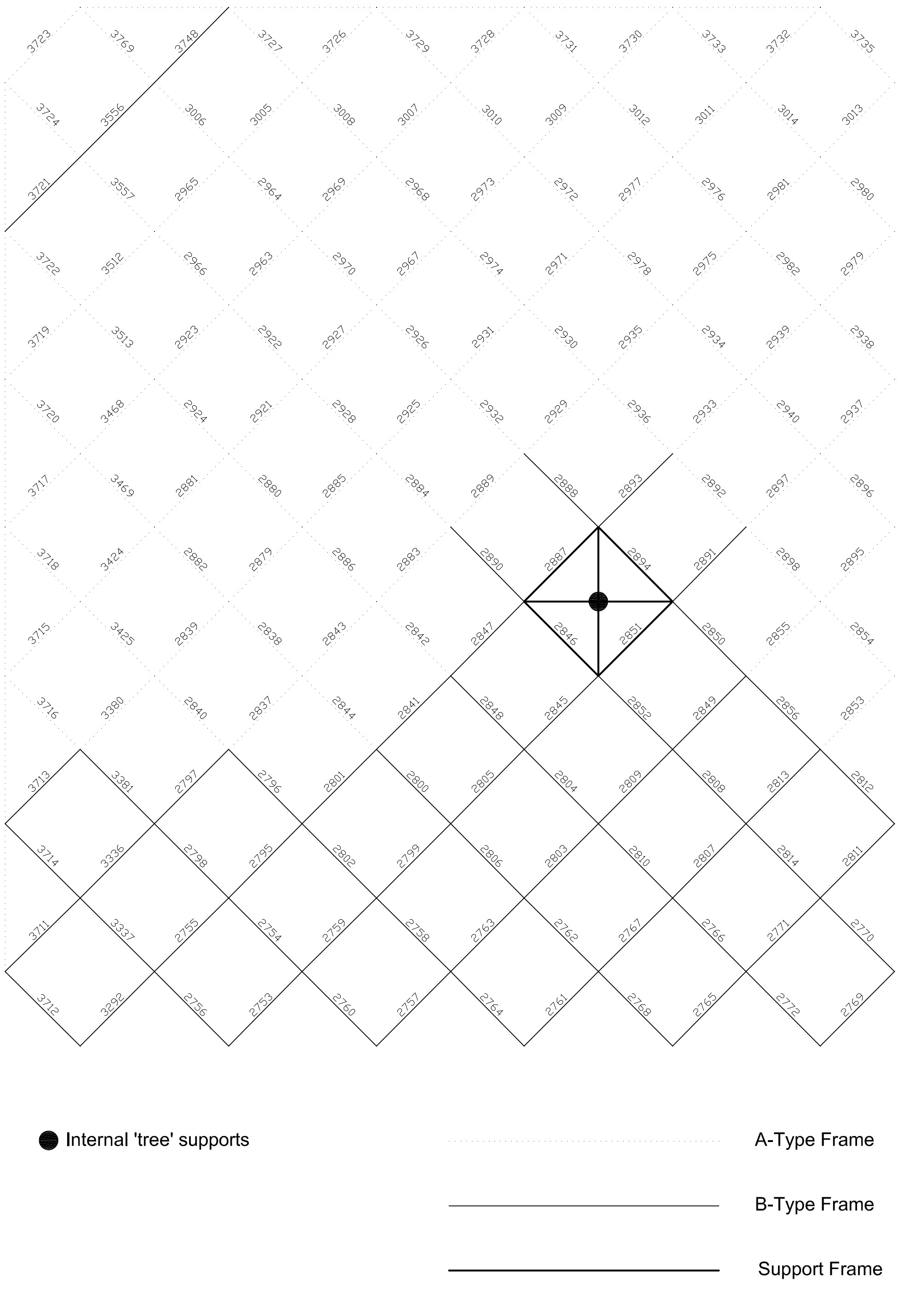



Figure 6.2.6: Quarter of plan showing members numbering: **Bottom Chords.**

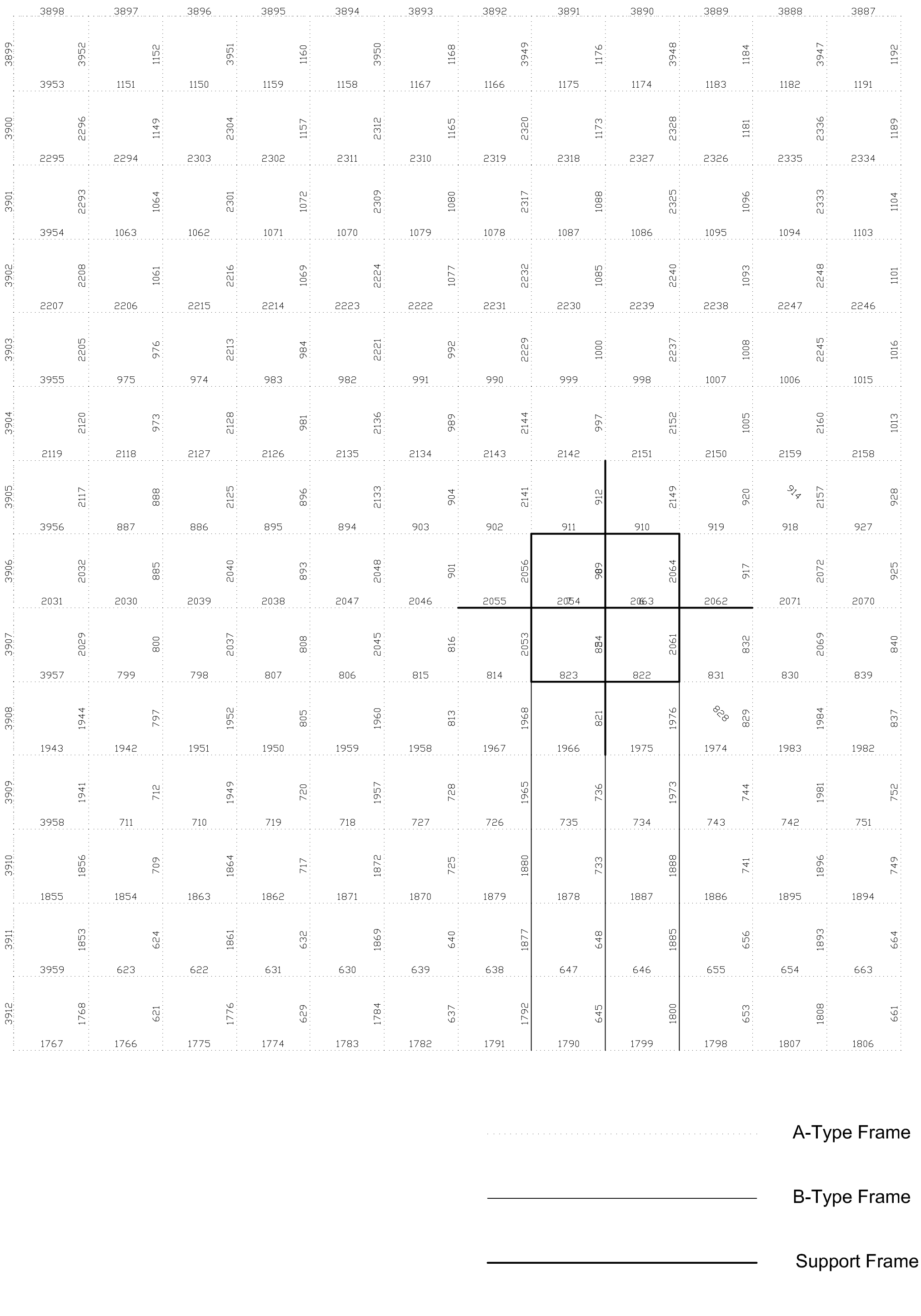



Figure 6.2.7: Quarter of plan showing members numbering: **Diagonals.**

### 6.2.3 Checks of the members at the ultimate state

The most highly stressed elements will be designed for the two typologies of member both in tension and compression.

#### 6.2.3.1 Tension

The maximum tension of $318,96\ kN$ was found for the member n° 2769 .

**Section resistance** It must be verified the eq.(3.4.1) about the tension in the net timber area.

*Tension resistance:*

$$f_{t,0,d} = f_{t,0,k} \cdot k_{mod}/\gamma_M = 22,5 \cdot 0,9/1,25 = 16,20\,MPa$$

180 section resistance*:*

$$F_{sec,180} = f_{t,0,d} \cdot A_{net} = 16,20\,MPa \cdot 31636,59\,mm^2 = 512,51\,kN$$

The net timber area $A_{net}$is the area of the timber removed from the screw holes. In this case it shall be considered a standard of 12 holes with a diameter of 9mm.

**Screws resistance** As said in the section 5.4 on page 74, the ultimate lateral load of timber-steel joints using inclined screw connectors can be defined using the Johansen yielding theory that for this kind of system provides three kinematically possible failure modes but according to the eq.5.4.4 on page 77, just the lower value has to be considered.

The characteristic resistance of the connection between the two steel plates and the glue-lam body of the frame is calculated according to the section 5.4.

A TYPE SCREW-SYSTEM RESISTANCE*:*

$Mode\ I$ $46,51\,kN$ *equation* (5.4.1)

$Mode\ II$ $28,77\,kN$ *equation* (5.4.2)

$Mode\ III$ $20,05\,kN$ *equation* (5.4.3)

$F_{ax,\alpha,k} = min\left\{F_u^I;\ F_u^{II};\ F_u^{III}\right\} = 20,05\,kN$ (*Mode III*)

$F_{ax,\alpha,d} = 0,9 \cdot 20,05/1,3 = 13,88\,kN$ equation (5.4.6)

$F_{ax,\alpha,tot} = 2 \cdot 9 \cdot 13,88 = 249,83\,kN$ equation (5.4.7)

B TYPE SCREW-SYSTEM RESISTANCE*:*

$Mode\ I$ $\quad 62,02\,kN$ *equation* (5.4.1)

$Mode\ II$ $\quad 38,25\,kN$*equation* (5.4.2)

$Mode\ III$ $\quad 25,34\,kN$ *equation* (5.4.3)

$F_{ax,\alpha,k} = min\left\{F_u^I;\ F_u^{II};\ F_u^{III}\right\} = 25,34\,kN$ (*Mode III*)

$F_{ax,\alpha,d} = 0,9 \cdot 20,05/1,3 = 17,54\,kN$ equation (5.4.6)

$F_{ax,\alpha,tot} = 2 \cdot 11 \cdot 17,54 = 386,00\,kN$ equation (5.4.7)

As expected, the use of a limited number of long screws causes the development of failure *Mode III* which allows a ductile behavior of the connection under breaking.

**Connection resistance** The following checks concern the steel connection capacity. The calculations are carried out considering the resistance of the two clamps of steel forming the connection.

A TYPE CONNECTION RESISTANCE*:*

*Plastic resistance (5.5.1):*

$$N_{pl,Rd} = n_{plates} \cdot \frac{A \cdot f_y}{\gamma_{M0}}$$

$$N_{pl,Rd} = 2 \cdot (1080 \cdot 355/1) \cdot 10^{-3} = 766,80\,kN$$

*Ultimate resistance (5.5.2):*

$$N_{u,Rd} = n_{plates} \cdot \frac{A_{net} \cdot f_u}{\gamma_{M2}}$$

$$N_{u,Rd} = 2 \cdot (0,9 \cdot 552 \cdot 490/1,25) \cdot 10^{-3} = 389,49\,kN$$

*Shear resistance in bolts (5.5.4):*

$$F_{v,Rd} = n_{bolts} \cdot n_{surfaces} 0,6 \cdot \frac{f_{u,b} \cdot A}{\gamma_{Mb}}$$

$$F_{v,Rd} = 2 \cdot 2 \cdot (0,6 \cdot 1000 \cdot 245/1,3) \cdot 10^{-3} = 588,00\,kN$$

*Bearing resistance*[9] *(5.5.5):*

$\alpha = 0,3636$

$f_u = 490\ MPa$

[9] *Two shear planes*

$F_{b,Rd} = 2 \cdot 2 \cdot (2,5 \cdot \alpha \cdot d \cdot t \cdot f_u/\gamma_M) \cdot 10^{-3} = 342,10\,kN$

B TYPE CONNECTION RESISTANCE:

*Plastic resistance (5.5.1):*
$N_{pl,Rd} = n_{plates} \cdot \dfrac{A \cdot f_y}{\gamma_{M0}}$
$N_{pl,Rd} = 2 \cdot (1080 \cdot 355/1) \cdot 10^{-3} = 766,80\,kN$

*Ultimate resistance (5.5.2):*
$N_{u,Rd} = n_{plates} \cdot \dfrac{A_{net} \cdot f_u}{\gamma_{M2}}$
$N_{u,Rd} = 2 \cdot (0,9 \cdot 696 \cdot 490/1,25) \cdot 10^{-3} = 491,09\,kN$

*Shear resistance in bolts (5.5.4):*
$F_{v,Rd} = n_{bolts} \cdot n_{surfaces} 0,6 \cdot \dfrac{f_{u,b} \cdot A}{\gamma_{Mb}}$
$F_{v,Rd} = 2 \cdot 3 \cdot (0,6 \cdot 1000 \cdot 353/1,3) \cdot 10^{-3} = 882,00\,kN$

*Bearing resistance(5.5.5):*
$\alpha = 0,3846$
$f_u = 490\,MPa$
$F_{b,Rd} = 2 \cdot 2 \cdot (2,5 \cdot \alpha \cdot d \cdot t \cdot f_u/\gamma_M) \cdot 10^{-3} = 434,21\,kN$

The following table shows the summary of the tension checks for the two type of systems:

| CONNECTION | N°SCREWS | $\mathbf{T}_{max}$ | $\mathbf{T}_{R,d\,SECTION}$ | | $\mathbf{T}_{R,d\,SCREWS}$ | | $\mathbf{T}_{R,d\,CONN.}$ | |
|---|---|---|---|---|---|---|---|---|
| | per side | $[KN]$ | $[KN]$ | | $[KN]$ | | $[KN]$ | |
| A TYPE | 9 | **248,84** | 512,51 | **OK** | 249,83 | **OK** | 342,10 | **OK** |
| B TYPE | 11 | **318,96** | 512,51 | **OK** | 434,21 | **OK** | 321,75 | **OK** |

Table 6.5: Summary of the checks for the maximum tension elements

#### 6.2.3.2 Compression

The maximum compression of $-334,57\,kN$ was found in the member 659.

**Buckling resistance** This check is carried on following the indications seen in part 3.4.2.1 on page 49 applying the eq.3.4.3.

*material parameters:*

$f_{c,0,k} = 29,00\,MPa$
$E_{0,05} = 11100\,MPa$
$f_{c,0,d} = f_{c,0,k} \cdot k_{mod}/\gamma_M = 29 \cdot 0,9/1,25 = 20,88\,MPa$

*frame properties :*

$A = 180 \times 180 = 32400\,mm^2$
$L_o = 2970 - 2 \times 200 = 2570\,mm$[10]

*buckling resistance :*

$i = 180/\sqrt{12} = 51,96\,mm$

$\lambda = L_0/i = 49,46\,mm$

$\lambda_{rel} = 49,49/\pi \cdot \sqrt{29/11100} = 0,8047$

$k_y = 0,5 \cdot (1 + 0,1 \cdot (\lambda_{rel} - 0,3) + \lambda_{rel}^2) = 0,8490$

$$k_{c,y} = \frac{1}{k_y \cdot \sqrt{k_y^2 - \lambda_{rel}^2}} = 0,4953$$

$\sigma_{MAX} = k_{c,y} \cdot f_{c,0,d} = 10,34\,MPa$

$N_{b,0,d} = A \cdot \sigma_{MAX} = 335,10\,kN$

| **Length** | **Section** | **i** | $\lambda$ | $\lambda rel$ | **k**$y$ | **k**$c,y$ | $\sigma_{MAX}$ | **A** | **F**$_{MAX}$ |
|---|---|---|---|---|---|---|---|---|---|
| [mm] | | [mm] | | | | | | [mm$^2$] | [kN] |
| 2970 | *180x180* | 51,96 | 49,46 | 0,80 | 0,85 | 0,50 | 10,34 | 32400 | **335,10** |

Table 6.6: The allowable buckling resistance of the glue-lam section

**Connection resistance** The following checks are about the steel connection capacity. The calculations are carried out considering the resistance of the two clamps of steel forming the connection.

A TYPE CONNECTION RESISTANCE*:*

*Shear resistance in bolts (5.5.4):*

$$F_{v,Rd} = n_{bolts} \cdot n_{surfaces} 0,6 \cdot \frac{f_{u,b} \cdot A}{\gamma_{Mb}}$$

[10]The buckling length taken for the calculation is reduced because of the rigidity in correspondence of the steel node.

$F_{v,Rd} = 2 \cdot 2 \cdot (0,6 \cdot 1000 \cdot 245/1,3) \cdot 10^{-3} = 588,00\,kN$

*Bearing resistance (5.5.5):*
$\alpha = 3$
$f_u = 490\,MPa$
$F_{b,Rd} = 2 \cdot 2 \cdot (2,5 \cdot \alpha \cdot d \cdot t \cdot f_u/\gamma_M) \cdot 10^{-3} = 1128,96\,kN$

*Buckling in the plate (5.5.7):*
$A = 12 \cdot 90 = 1080\,mm^2$
$L_0 = 140\,mm$
$\chi = 0,8266$
$N_{b,Rd} = 2 \cdot (\frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}) \cdot 10^{-3} = 633,90\,kN$

B TYPE CONNECTION RESISTANCE:

*Shear resistance in bolts (5.5.4):*
$F_{v,Rd} = n_{bolts} \cdot n_{surfaces} 0,6 \cdot \dfrac{f_{u,b} \cdot A}{\gamma_{Mb}}$
$F_{v,Rd} = 2 \cdot 3 \cdot (0,6 \cdot 1000 \cdot 353/1,3) \cdot 10^{-3} = 882,00\,kN$

*Bearing resistance (5.5.5):*
$\alpha = 3$
$f_u = 490\,MPa$
$F_{b,Rd} = 2 \cdot 2 \cdot (2,5 \cdot \alpha \cdot d \cdot t \cdot f_u/\gamma_M) \cdot 10^{-3} = 1354,75\,kN$

*Buckling in the plate (5.5.7):*
$A = 140 \cdot 90 = 12600\,mm^2$
$L_0 = 140\,mm$
$\chi = 0,7253$
$N_{b,Rd} = 2 \cdot (\frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}) \cdot 10^{-3} = 865,18\,kN$

**Local compression** The compression force is transmitted directly to the section of the frame through the traversal steel plate so the force can be considered completely imposed to the timber section without stressing the screws against the inclined direction.

Because of the tapered shape of the frames, the area solicited from the steel plate is lower than the frame section so it must be locally verified for pure compression in order not to get local compression failure in timber.

A TYPE CONNECTION RESISTANCE:

$f_{c,0,d} = f_{c,0,k} \cdot k_{mod}/\gamma_M = 29 \cdot 0,9/1,25 = 20,88\,MPa$

$A_c = 180 \times 90 = 16\,200\,mm^2$

$F_{R,d,COMPRESSION} = 20,88 \times 16\,200 \times 10^{-3} = 338,26\,kN$

A TYPE CONNECTION RESISTANCE:

$f_{c,0,d} = f_{c,0,k} \cdot k_{mod}/\gamma_M = 29 \cdot 0,9/1,25 = 20,88\,MPa$

$A_c = 118 \times 76 = 8\,974\,mm^2$

$F_{R,d,COMPRESSION} = 20,88 \times 8\,974 \times 10^{-3} = 187,37\,kN$

The following table shows the summary of the compression checks for the two type of systems:

| **CONNECTION** | **BOLTS** | **GRADE** | $\mathbf{N}_{maxULS}$ | $\mathbf{F}_{R,d\,BUCK.}$ | | $\mathbf{F}_{R,d\,CONN.}$ | | $\mathbf{F}_{R,d\,COMP.}$ | |
|---|---|---|---|---|---|---|---|---|---|
| | | | $[kN]$ | $[kN]$ | | $[kN]$ | | $[kN]$ | |
| A TYPE | 2 *M20* | 10.9 | **180,00** | 335,11 | **OK** | 588,00 | **OK** | 187,37 | **OK** |
| B TYPE | 2 *M24* | 10.9 | **334,57** | 335,11 | **OK** | 865,18 | **OK** | 338,26 | **OK** |

Table 6.7: Summary of the checks for the maximum compressed elements

### 6.2.4 Performance of the preloaded bolts

This check is the more onerous regarding the serviceability limit state and the ductility of the whole structure. The choice of the bolts in fact has been made depending on the the pre-load required to guarantee the frictional resistance of the connection at the characteristic load combination. The conception of the Octatube system permits to take advantage of the friction offered from the surfaces of the steel plates within the load transmitted from the pre-load of the high resistance bolts.

The system is developed to resist the loads under the characteristic combination in order to keep the stiff behavior under exercise conditions. At the ultimate limit state this factor has not been considered, in fact the slip at the bolts is provided before failure.

The choice and the dimensioning of the bolts size are chosen exclusively to guarantee the necessary strength to ensure the necessary friction resistance to satisfy at every node, the force provided from the exercise-load analysis obtained from the equation (6.2.3) (see in tab.6.8).

| LOAD TYPE | POSITION | $A_{inf}$ | $F_{SLS}\ [kN]$ |
|---|---|---|---|
| Weight of the node at every joint | *Each node* | - | 0,15 |
| Dead load from concrete and steel deck | *Top* | 8,82 $m^2$ | 13,23 |
| Estimated load of installation | *Top* | 8,82 $m^2$ | 4,41 |
| Live load (*Quasi static load*) | *Top* | 8,82 $m^2$ | 17,64 |

Table 6.8: Loads at the Serviceability limit State

In the table 6.9 on the facing page the maximum forces obtained from the analysis are shown, both for the tension and the compression elements.

| **Frame** | **OutputCase** | **State** | **StepType** | **P** |
|---|---|---|---|---|
| Text | Text | Text | Text | [kN] |
| 1855 | SLS combo | Tension | Max | 182,8 |
| 2753 | SLS combo | Tension | Max | 216,5 |
| 2755 | SLS combo | Tension | Max | 207,0 |
| 2757 | SLS combo | Tension | Max | 194,5 |
| 2759 | SLS combo | Tension | Max | 220,8 |
| 2769 | SLS combo | Tension | Max | **233,9** |
| 2770 | SLS combo | Tension | Max | 199,4 |
| 2795 | SLS combo | Tension | Max | 226,7 |
| 2797 | SLS combo | Tension | Max | 194,1 |
| 2799 | SLS combo | Tension | Max | 184,0 |
| 2801 | SLS combo | Tension | Max | 189,9 |
| 3292 | SLS combo | Tension | Max | 182,6 |
| 3556 | SLS combo | Tension | Max | 191,3 |
| | | | | |
| 617 | SLS combo | Compression | Min | -207,7 |
| 625 | SLS combo | Compression | Min | -223,3 |
| 627 | SLS combo | Compression | Min | -235,9 |
| 635 | SLS combo | Compression | Min | -220,6 |
| 659 | SLS combo | Compression | Min | **-267,6** |
| 659 | SLS combo | Compression | Min | -230,4 |
| 705 | SLS combo | Compression | Min | -195,1 |
| 713 | SLS combo | Compression | Min | -238,0 |
| 715 | SLS combo | Compression | Min | -208,7 |
| 2845 | SLS combo | Compression | Min | -211,7 |
| 2850 | SLS combo | Compression | Min | -244,7 |
| 2852 | SLS combo | Compression | Min | -268,5 |

Table 6.9: Tension and compression of the main elements at the serviceability limit state

The maximum tension of $233,90\ kN$was found for the member n° 2769 .

The maximum compression of$-267,61\ kN$ was found for the member n° 659.

Of course the forces in the whole structure are not homogeneous so the frame system is designed using at least two typologies of connection, working with different size of bolts. A-TYPE is used for the frame solicited with forces lower than 180 kN and B-TYPE is used for the others. This device allows a better optimization of the material[11]. The logical dispositions proposed for the two kinds of connections (that also fulfill the tests at the ultimate limit state)

[11] Of course it could be possible to use an higher number of system, in the limit not to over complicate the organization and the assembly work.

are shown on page 103.

The two connections are developed in the following way:

**A TYPE** n°2 bolts *M20* grade 10.9 ($f_{u,k} = 1000MPa$ ), preloaded with the maximum force of 171,50 kN[12].

**B TYPE** n°2 bolts *M24* grade 10.9 ($f_{u,k} = 1000MPa$ ), preloaded with the maximum force of 247,10 kN .

#### 6.2.4.1 Checks of the preloaded bolts

**A TYPE**

| | | |
|---|---|---|
| Bolt: | *M20* | |
| Grade: | 10.9 | ($f_{u,k} = 1000MPa$ ) |
| $A_s$: | $245\,mm^2$ | |
| n$_b$: | 2 | (number of bolts) |
| n$_f$: | 2 | (number of friction surfaces) |
| $\gamma_{Ms}$ : | 1,1 | (serviceability limit state) |
| $\mu$ : | 0,3 | (not painted steel to steel) |

Preloading force required (4.5.1):

$$F_{P,Cd} = 0,7 \cdot f_{u,b} \cdot A_s$$
$$F_{P,Cd} = 0,7 \cdot f_{u,b} \cdot A_s = 0,7 \cdot 1000 \cdot 245 \cdot 10^{-3} = 171,50\,kN$$

Friction resistance of the connector (4.5.2):

$$F_{s,Rd} = n \cdot \mu \cdot \frac{F_{P,Cd}}{\gamma_{Ms}}$$
$$F_{s,Rd} = n_b \cdot n \cdot \mu \cdot \frac{F_{P,Cd}}{\gamma_{Ms}} = 2 \cdot 2 \cdot 0,3 \cdot \frac{46,67}{1,1} = 187,09\,kN$$

**B TYPE**

| | | |
|---|---|---|
| Bolt: | *M24* | |
| Grade: | 10.9 | ($f_{u,k} = 1000MPa$ ) |
| $A_s$: | $353\,mm^2$ | |
| n$_b$: | 2 | (number of bolts) |
| n$_f$: | 2 | (number of friction surfaces) |
| $\gamma_{Ms}$ : | 1,1 | (serviceability limit state) |
| $\mu$ : | 0,3 | (not painted steel to steel) |

[12] See par. «Resistance of preloaded bolts for the serviceability limit state» on page 65

Preloading force required (4.5.1):

$$F_{P,Cd} = 0,7 \cdot f_{u,b} \cdot A_s$$
$$F_{P,Cd} = 0,7 \cdot f_{u,b} \cdot A_s = 0,7 \cdot 1000 \cdot 245 \cdot 10^{-3} = 247,10\,kN$$

Friction resistance of the connector (4.5.2):

$$F_{s,Rd} = n \cdot \mu \cdot \frac{F_{P,Cd}}{\gamma_{Ms}}$$
$$F_{s,Rd} = n_b \cdot n \cdot \mu \cdot \frac{F_{P,Cd}}{\gamma_{Ms}} = 2 \cdot 2 \cdot 0,3 \cdot \frac{46,67}{1,1} = 269,56\,kN$$

The following table shows the summary of the compression checks for the two types of systems:

| CONNECTION | MEMBER | BOLTS | GRADE | $\mathbf{N}_{maxSLS}$ $[KN]$ | $\mathbf{F}_{s,Rd,FRICTION}$ $[KN]$ | |
|---|---|---|---|---|---|---|
| A TYPE | 1855 | 2 *M20* | 10.9 | **182,78** | 187,09 | **OK** |
| B TYPE | 659 | 2 *M24* | 10.9 | **267,61** | 269,56 | **OK** |

Table 6.10: Summary of the checks for the preloaded bolt resistance

### 6.2.5 Check of the deflection

The deflection for the structure considered is calculated with SAP*2000* to get the instantaneous deflection for every load case. The vertical deformation is calculated for every significant point of the structure that is in the middle of every span see figure 6.2.8 on the next page.

following the eq. 3.5.1 on page 53 the deformation for the main points of the structure are:

**Point A**

| LOAD TYPE | $u_{ist}[mm]$ | $u_{fin}[mm]$ |
|---|---|---|
| Self weight of glue-lam space frames | 1,25 | 2,00 |
| Weight of the node at every joint | 4,90 | 7,83 |
| Dead load from concrete and steel deck | 0,11 | 0,18 |
| Estimated load of installation | 1,63 | 2,62 |
| Live load (*Quasi static load*) | 6,53 | 7,70 |

$u_{inst,Q} = 8,16\,mm < (l/300)$

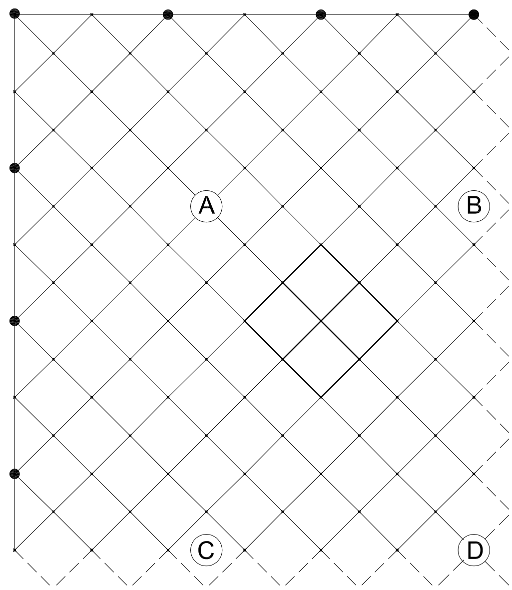



Figure 6.2.8: Quarter of plan showing the points checked for deflection.

$u_{fin,Q} = 10,32\, mm < (l/300)$
$u_{fin,tot} = 20,33\, mm < (l/250)$

**Point B**

| LOAD TYPE | $u_{ist}[mm]$ | $u_{fin}[mm]$ |
|---|---|---|
| Self weight of glue-lam space frames | 1,20 | 1,93 |
| Weight of the node at every joint | 4,60 | 7,37 |
| Dead load from concrete and steel deck | 0,10 | 0,16 |
| Estimated load of installation | 1,54 | 2,46 |
| Live load (*Quasi static load*) | 6,14 | 7,24 |

$u_{inst,Q} = 7,68\, mm < (l/300)$
$u_{fin,Q} = 9,70\, mm < (l/300)$
$u_{fin,tot} = 19,17\, mm < (l/250)$

**Point C**

| LOAD TYPE | $u_{ist}[mm]$ | $u_{fin}[mm]$ |
|---|---|---|
| Self weight of glue-lam space frames | 2,59 | 4,19 |
| Weight of the node at every joint | 9,87 | 15,80 |
| Dead load from concrete and steel deck | 0,22 | 0,35 |
| Estimated load of installation | 3,29 | 5,27 |
| Live load (*Quasi static load*) | 13,17 | 15,53 |

$u_{inst,Q} = 16,46\,mm < (l/300)$
$u_{fin,Q} = 20,81\,mm < (l/300)$
$u_{fin,tot} = 41,12\,mm < (l/250)$

**Point D**

| LOAD TYPE | $u_{ist}[mm]$ | $u_{fin}[mm]$ |
|---|---|---|
| Self weight of glue-lam space frames | 3,39 | 5,43 |
| Weight of the node at every joint | 12,96 | 20,73 |
| Dead load from concrete and steel deck | 0,29 | 0,47 |
| Estimated load of installation | 4,33 | 6,92 |
| Live load (*Quasi static load*) | 17,28 | 20,38 |

$u_{inst,Q} = 21,61\,mm < (l/300)$
$u_{fin,Q} = 27,31\,mm < (l/300)$
$u_{fin,tot} = 53,94\,mm < (l/250)$

where:

$l = 16,80\,m$ is the minimum span of the floor between the support;

$l/300 = 56,00\,mm$;

$l/250 = 67,20\,mm$.

As provided the deflection is largely verified using this system.

# Chapter 7

# Conclusions

The proposed construction technology is able to provide ample opportunity for structural timber engineering. It lends the realization both of bridges of considerable importance and of three-dimensional structures for covering large areas without the use of bracing elements.

The main topics that this research project aims to have developed are the following:

- Structural safety requires adeguate strength and ductility before failure;
- easiness of imanifacturing and erection, which is critical need to meet the competitiveness and the current market needs;
- design of a simple and economical joint solution;
- durability of structures with particular emphasis on details;
- dynamic loads resistance (wind, earthquake and sport activities);
- sustainability in the use of wood material.

The application of glued laminated timber and the proposed connection technology in the space frame system allow to understand the benefits elencated in the first chapters of this thesis. The use of timber material is a valid alternative to steel because of the similar properties in the elastic range and, if provided with an appropriate connection technology, a good ductile behavior can be achieved as well.

**Elastic range behavior** At the serviceability limit state, under characteristic loads, the structure maintains a stiff behavior in term of deflection (see part. 6.2.5) and stresses transmitted to the connections (see part. 6.2.4).

With a good design and an appropriate connection choice, the elastic range in the timber elements is mantained until the ultimate limit state before buckling or brittle failure happen.

**Ductile behavior** The choice of the connection technology is supposed to be taken as the more ductile as possible in order to avoid the failure hazards presented in section §5.2 which determinate brittle behaviors.

The use of the long screws working inclined to the grain allows the development of a ductile mechanism for tension connections which does not favorite dangerous crack propagations in the members after the ultimate limit state. Furthermore, for a good statical efficiency, the conception of the steel plate must favor a great deformation before cracking.

**Robustness** The statical redundancy of space grids means that, in general, failure of one or a limited number of elements, for instance, the buckling of a compression member, does not lead to overall collapse of the structure. These issues are strongly related to the robustness of structural systems.

In order to minimize the likelihood of disproportionated structural failures, many modern building codes consider the need for robustness in structures and provide strategies and methods to obtain robustness. Traditional types of structures with limited redundancy may have serious consequences in case of failure since they can not withstand failures of the main members.

Space grid structures are resistant to damage caused by fire, explosion or seismic activity unless critical elements (e.g. those adjacent to individual column supports) are removed or weakened by explosion or fire collapse.

**Lightness and fire performances** The weight of the glued laminated structure does not exceed the 10% of the original steel structure, built with Flocoat tubes of 450 MPa yield strength. The massive shape of the square section, compared to the $127.0mm \times 4.50mm$ tube, allow to get a better fire resistance[1].

**Versatility** Because of the large number of nodes generally present in this space structures, the simplicity and the cheapness of the node system represent crucial points to consider concerning the cost of the whole construction.

The choice of the system with the bolts working in tension or in shear, shows many architectural opportunities combined with the different performances required.

*Octatube node* presents good performances and good tolerance in phase of assembly; it requires a special accurancy because of the preloading of the bolts but it permits a good and stiff behavior especially for the dynamic loads. The production of the huge number of nodes required, can be cheaply produced with *«cast steel»* system.

[1] Steel components shall be painted with a special fire-protective paint.

*Mero node* presents more freedom of shape. The inclination between the trusses can be changed depending on the pre-drilled holes prepared in the spherical connection by high precision machines.

**Cheapness and standard production** The use of standard members allows the use of industrial production with numerical control machines. Both the production of the wooden trusses and the steel connection can easily factory made increasing the cheapness of the production.

The accuracy required from the assembly of the elements is very important to ensure both the tolerance under the erection phases and the good adherence of the members to the steel connections which allows an optimal transmission of the forces. The assembly of the connections to the wooden body shall be carried under controlled industrial environment before the erection in order to obtain the suitable precision.

# Appendix A

# Modeling and analysis of the structure with SAP*2000*

Taking advantage of the double symmetry, it is necessary to analyze just a quarter of the structure using the software SAP*2000*.

## A.1 Geometry of the frame system

The *3D* draw was prepared in AutoCAD and imported in SAP*2000* from a .dwg file (see fig. A.1.1).

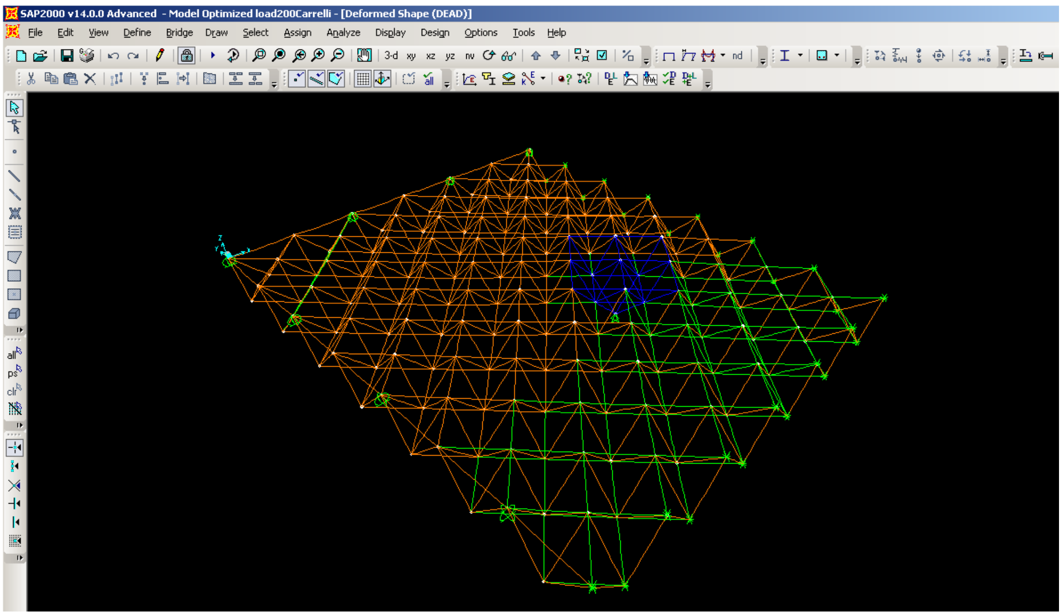



Figure A.1.1: Model of a quarter of the structure in SAP*2000* interface.

## A.2 Material properties and frame section

Since just normal forces are involved in the structure, the only material properties required to a truss-structure are just the Modul of Elasticity and Weight per unit Volume (see figure A.2.1).

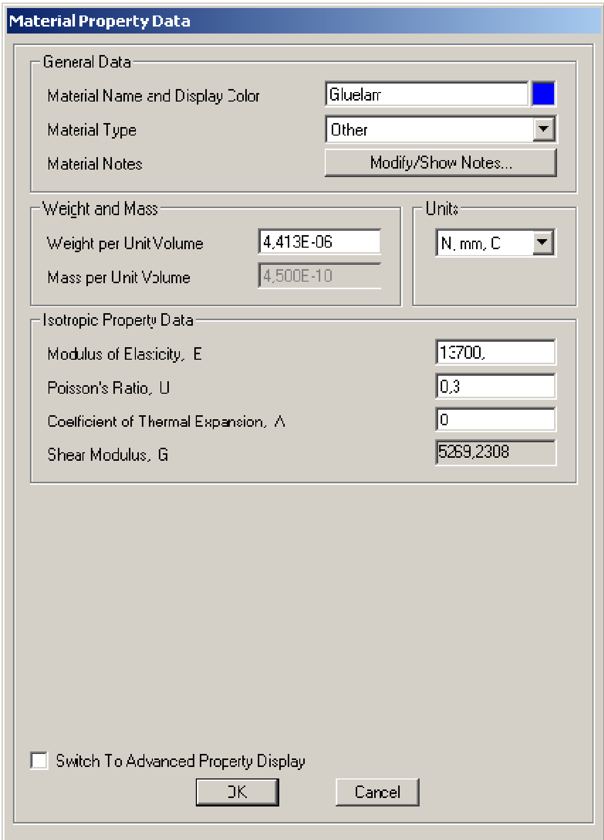
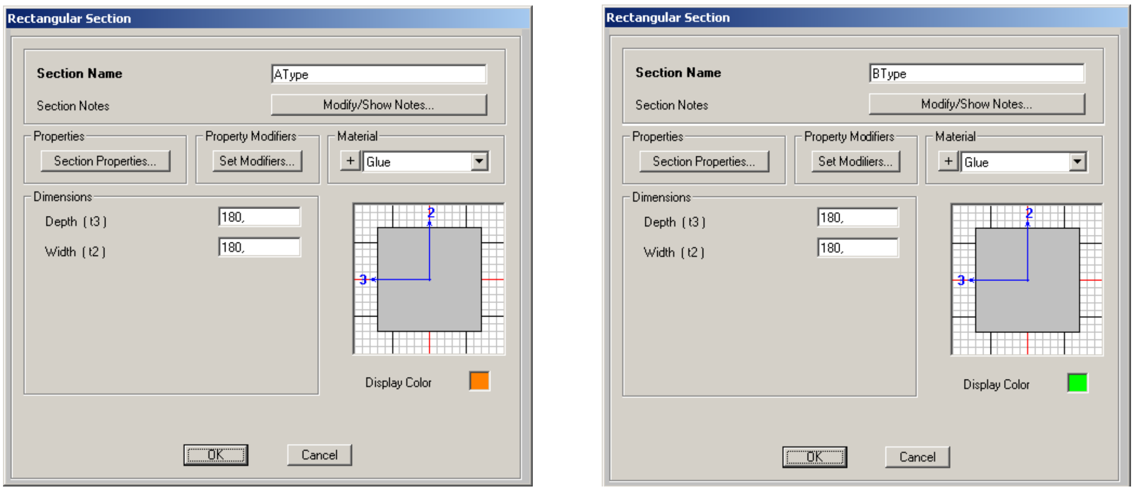



Figure A.2.1: Material properties imposed to the software.

**Modeling of the frames** Based on the previous experience of the existing space frame in steel, a section of $180 \times 180\ mm^2$ was assigned to every chord. The force found from the first cycle, corresponded to the tension and compression testing. The results was scanned and the section assignments of the frames had not been changed because of the hyper-static configuration of the frame work system that in this case requires an homogeneous assignment of the section.

In correspondence of the central support however, that section was not enough so in that zone, on which rests the constrain reaction of the central pillars, the elements must be dimensioned with steel chords.

A section of $180 \times 180\ mm^2$ was adopted for every truss. A distinction was made to separate the results of the two different connection typologies (see figures A.2.2 *a* and *b*).




a) Section of the members with the A-Type connection system (orange).

b) Section of the members with the B-Type connection system (green).

Figure A.2.2: Definition of the truss rectangular section in Sap2000 interface.

## A.3 Internal constraints and boundary conditions

**Modeling of the internal joint-constraints** Since we are treating a reticular space truss roofing system, and the nodes will be designed to focus the components to the same point, the frames can transmit only normal forces. Therefore it is necessary to model the joint between every element releasing the bending capacity of all the nodes. Frames that are designed and modeled to transmit only axial force are called «*trusses*».

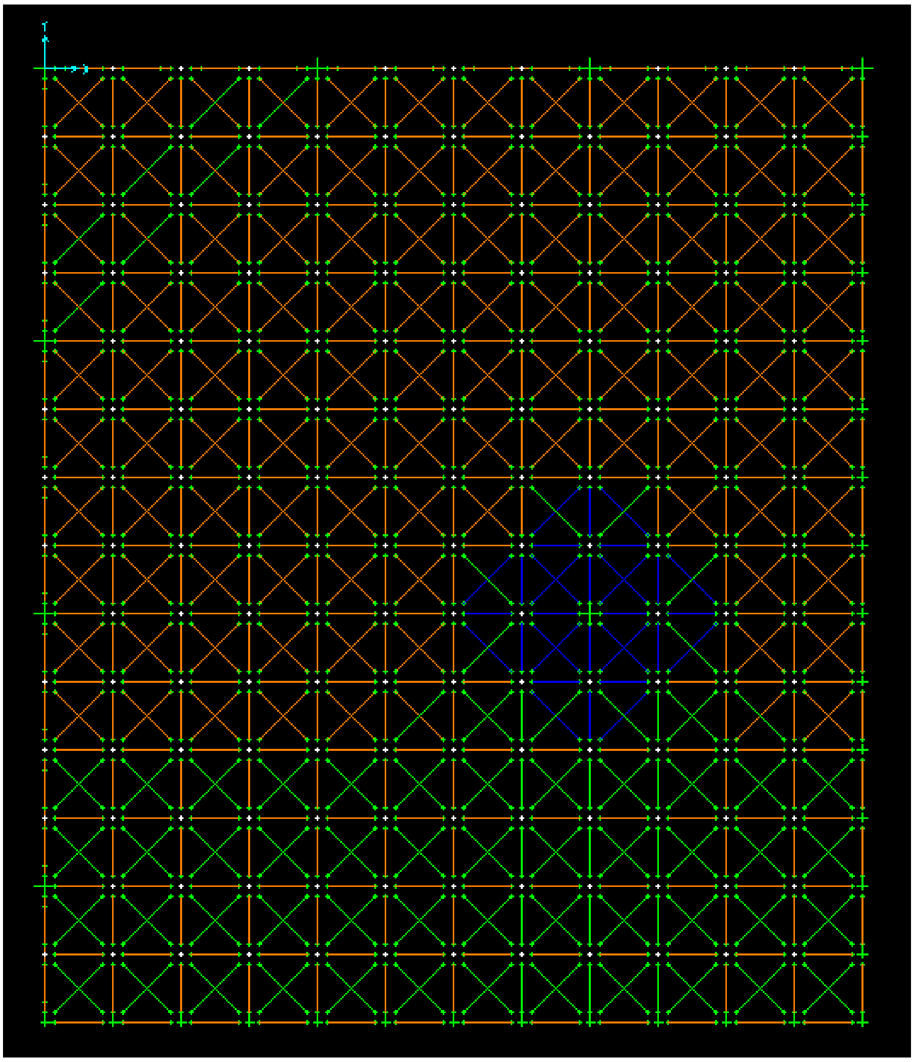
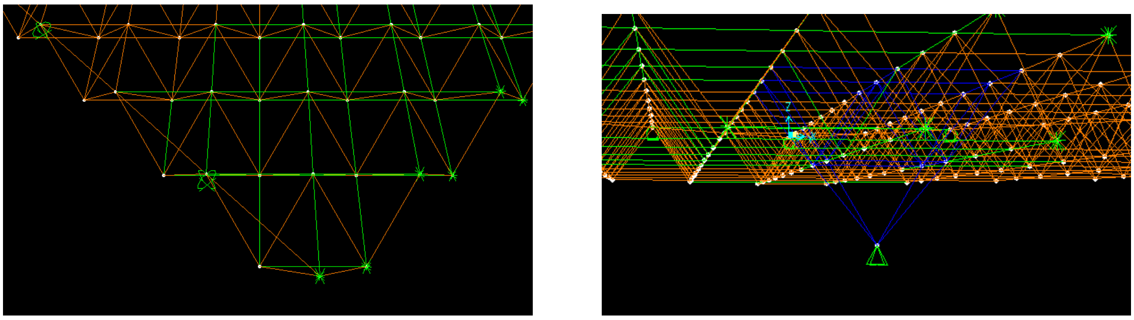


Figure A.3.1: Assign of the frames releases in SAP*2000* interface.

**Boundary conditions** The structure is supported by peripheral columns, spaced 8.40 m apart, and four interior latticed columns by means of inverted pyramidal 'tree' supports, which are part of the space frame. Since the internal supports also guarantee the horizontal restraints it might model the support as hinge (translation binding in $x$, $y$ and $z$ directions), the peripheral columns will be modeled like a connecting rod which can only carry vertical load ($z$ directions).

Concerning the assignments of the constraints along the symmetry border, it will be necessary to put a link that does not permit the translation in the direction perpendicular to the symmetry lines.



a) Perimetral supports and constraints along the symmetry border

b) Pyramidal 'tree' central supports

Figure A.3.2: Boundary condition interface in SAP*2000*.

## A.4 Load assignment

The distribution of the loads on the structure is divided on each node of the superior layer (top chords) of the space frame system except the self weight of the frame and the weight of every node (node and connection included).

The distributed loads and the installation loads shown in the table 6.1 on page 95 will be concentrated on each node of the top chords simulating the force transmitted from the corresponding area of influence meanwhile the weight of every node will be applied on itself.

**Load patterns** A load pattern is a specified spatial distribution of forces, displacements, temperatures, and other effects that act upon the structure. The definition of the load patterns for the structure is concerning the table 6.1 on page 95. The self weight of the timber structure, assigned in the table A.2.1 on page 124, is considered automatically by imposing the Self Weight Multiplier.

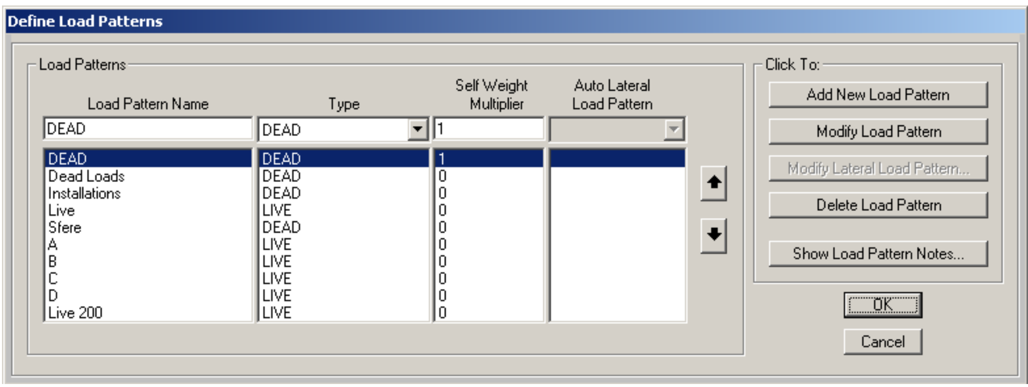
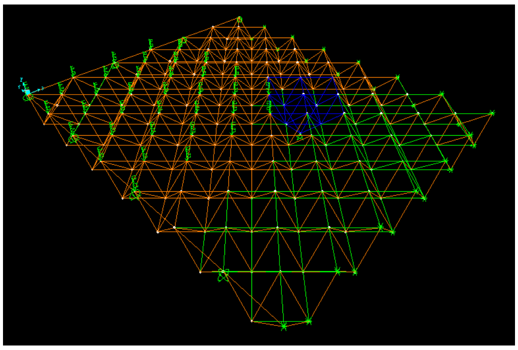


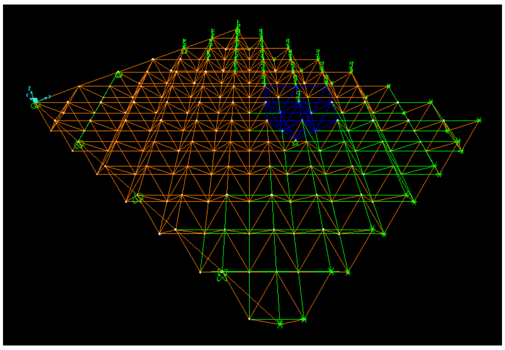

Figure A.4.1: Load patterns definition in SAP*2000* interface.

**Load cases** Load patterns must be applied in load cases in order to produce results. A load case defines how loads are to be applied to the structure, and how the structural response is to be calculated.

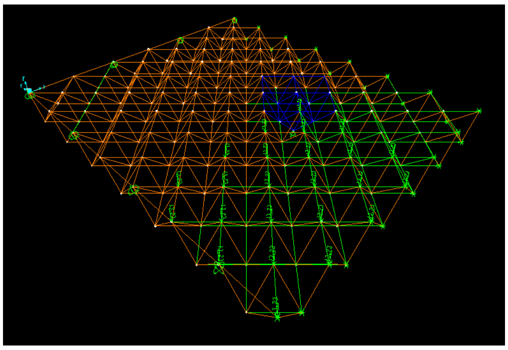
The forces are put separately on the four partitions of the court (see figure A.4.2) with the minimum value in order to define the necessary cases needed for the envelope (see figure A.4.3).



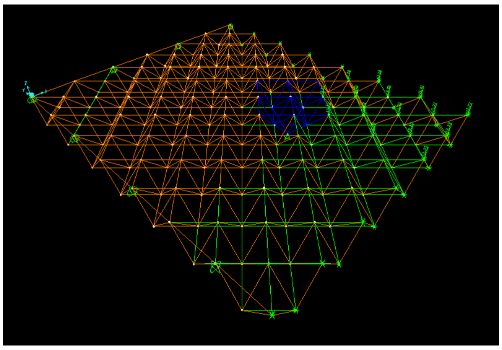
A case



B case



C case



D case

Figure A.4.2: Forces distribution in the four partitions of the quarter of the court.

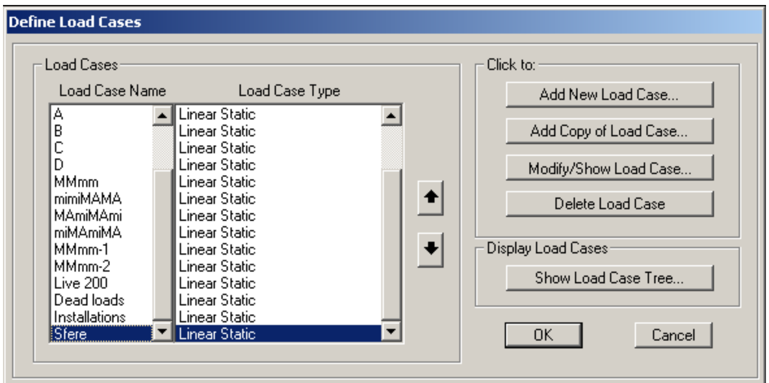
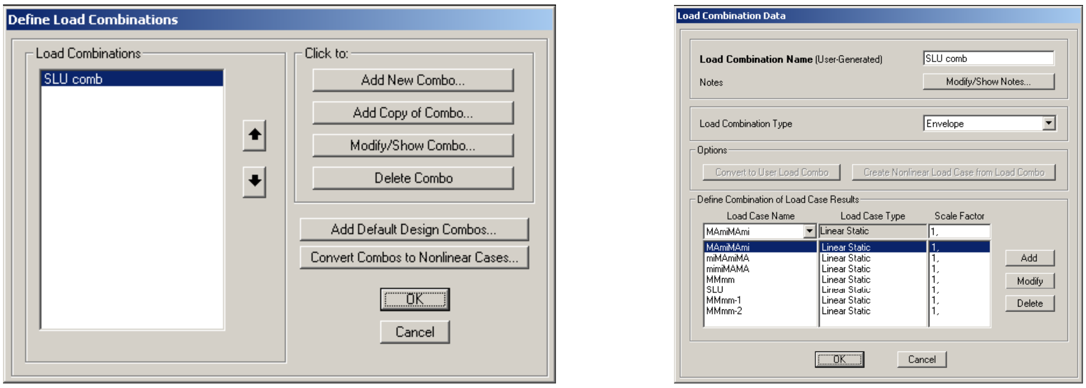



Figure A.4.3: Load cases definition in SAP*2000* interface.

### Load combination

The actions of the frames are determined from the envelope of every significant case of distribution. The load cases that give the maximum and minimum components are used for this combo.




a) Load cases list a) Envelope combination

Figure A.4.4: Load combinations definition in SAP*2000* interface.

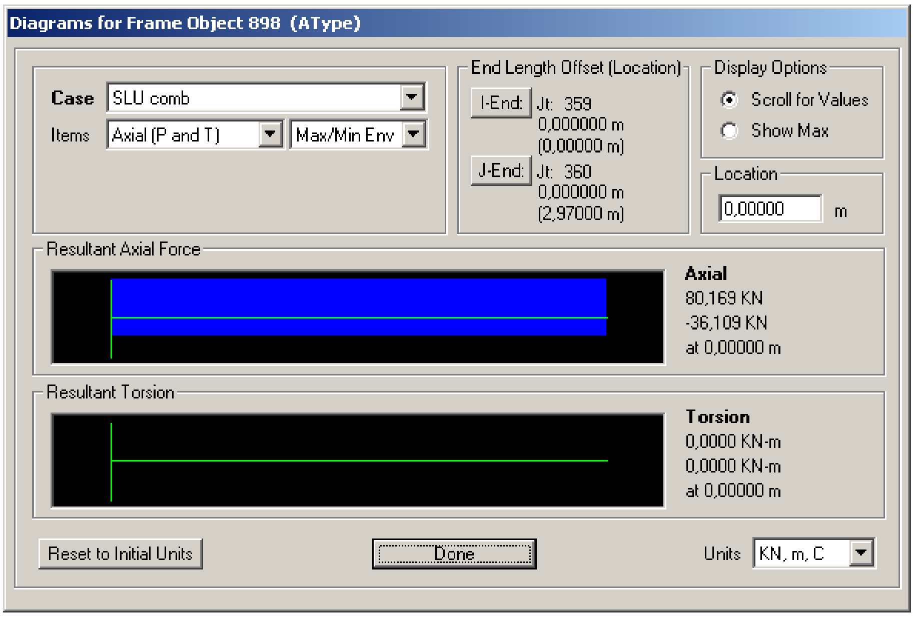



Figure A.4.5: Diagram of the envelope of a typical member which can be both in compression and in tension depending on the case.

# Appendix B

# Swedish classification for glued laminated timber

This class applies glue-lam cleaved vertically into two parts with width/height ratio$\leq$1/8. Values have been calculated according to *EN14080* (version January 2011).

**Results**
With the above conditions the different classes can be given by the fallowing declared values.

| QUANTITY | | L40h | L40c | L40s |
|---|---|---|---|---|
| $f_{m,k}$ | $[MPa]$ | 32,0 | 30,8 | 30,0 |
| $f_{t,0,k}$ | $[MPa]$ | 22,5 | 17,6 | 22,5 |
| $f_{t,90,k}$ | $[MPa]$ | 0,5 | 0,4 | 0,4 |
| $f_{c,0,k}$ | $[MPa]$ | 29,0 | 25,4 | 29,0 |
| $f_{c,90,k}$ | $[MPa]$ | 3,3 | 2,7 | 2,7 |
| $f_{v,k}$ | $[MPa]$ | 3,5 | 3,5 | 3,5 |
| $E_{0,mean}$ | $[MPa]$ | 13 700 | 13 000 | 13 200 |
| $E_{0,05}$ | $[MPa]$ | 11 100 | 10 500 | 11 100 |
| $E_{90,mean}$ | $[MPa]$ | 460 | 410 | 410 |
| $G_{mean}$ | $[MPa]$ | 850 | 760 | 760 |
| $\gamma_k$ | $[kg/m^3]$ | 430 | 400 | 430 |

Table B.1: Characteristic values of the properties.

These values are the maximum that a manufacturer may choose to declare. Lower values can be declared in order to minimize technical and/or financial risks.

# List of Tables

# List of Figures

# Bibliography


[1] G. S. Ramaswamy et al. (2002), «*Analysis, design and construction of Steel space frames*», Thomas Telford, UNITED KINGDOM.

[2] Tien T. Lan (2005) «Handbook of structural engineering»,Chinese Academy of Building Research, Beijing, CHINA

[3] Tata Steel Europe Limited (2012), `http://www.tatasteelconstruction.com`, UNITED KINGDOM.

[4] M.Piazza et al. (2009), «*Strutture in legno*» Biblioteca tecnica HOEPLI, ITALIA.

[5] R.Crocetti et al. (2011), «*Design of timber structures*» Swedish wood, SVERIGE.

[6] Holger Gross (2001), «*Limträ Handbok*» Svenskt Limträ AB, SVERIGE.

[7] H.J. Blaas et al. (1995), «*Timber Engineering*» STEP, THE NETHERLANDS .

[8] A. Jorissen, M. Fragiacomo (2010), «*Ductility in timber structures*» Meeting forty three Nelson, NEW ZELAND.

[9] P.H. Kirkegaard et al. (2011) «*System reliability of timber structures with ductile behavior*» Department of Civil Engineering, Aalborg University, Aalborg, DENMARK

[10] A. Emilsson et al. (2011), «*Assesment of failures and malfunctions*» COST, SVERIGE.

[11] S. Thelandersson & H. J. Larsen (2003), «*Timber Engineering*» WILEY, SVERIGE & DENMARK.

[12] S. Kavaliauskas, A. Kazimieras Kvedaras & B. Valiūnas (2007), «*Mechanical behavior of timber-to-concrete connections with inclined screws*», Dept of Steel and Timber Structures, Vilnius Technical University, LIETUVA

[13] DIBT Z-9.1-472, Allgemeine bauaufsichtliche Zulassung: *SFS Befesiger W-S-6,5; WT-T-8,2; WR-T-13 als Holzverbindungsmittel*.

## Normative sources


- EN 1990 *Eurocode 0: Basis of structural Design*
- EN 1993-1-1, *Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings.*
- EN 1995-1-1, *Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings.*