THE PERFORMANCE OF MEDIUM AND LONG SPAN TIMBER ROOF STRUCTURES: A COMPARATIVE STUDY BETWEEN STRUCTURAL TIMBER AND STEEL

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ABSTRACT

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This thesis analyzes the performance of structural timber and steel in medium and long span roof structures. A technical background about roof structures including structural elements and roof structure types, span definitions, and classification of roof structures are discussed. Roof structures are detailed with traditional and the contemporary forms. The thesis comprises the comparison between structural timber and steel by using structural, constructional and material properties. Structural forms and the performance of timber and steel are discussed. The research also includes the roof structures built with structural timber in Turkey, application, marketing and examples in Turkey are indicated. In the conclusion part the performance criteria of timber and steel are summarized, the researcher has prepared a table to compare the performance of timber and steel.

Keywords: Timber, Steel, Roof, Structure, Span

ÖZ

ORTA VE GENİŞ AÇIKLIKLI AHŞAP ÇATILARIN PERFORMANSI:
AHŞAP VE ÇELİĞİN KARŞILAŞTIRMALI ÇALIŞMASI

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Bu çalışma strüktürel ahşap ve çeliğin orta ve geniş açıklıklı çatılarda performanslarının karşılaştırılmasını kapsamaktadır. Çatı sistemleri hakkında, strüktürel elemanları, çatı çeşitlerini, açıklık tanımını ve çatı sınıflandırmasını kapsayan teknik bir özet tartışılmıştır. Çati sistemleri geleneksel ve güncel sınıflandırılmalarla detaylı anlatılmıştır. Tez strüktürel ahşap ve çeliği strüktürel, yapım ve malzeme performanslarına göre karşılaştırmaktadır. Araştırma Türkiye'de strüktürel ahşap kullanılarak tasarlanmış çatıları üretim, uygulama ve pazarlama kriterlerine göre de kapsamaktadır. Sonuş bölümünde ahşap ve çeliğin performans kriterleri özetlenmiş, sonuç tablosunda performans kriterleri ahşap ve çelik için karşılaştırılmıştır.

Anahtar Kelimeler: Ahşap, Çelik, Çatı, Strüktür, Açıklık

To my father who follows me from heaven,
my mother and
dear Uğur

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ABBREVIATIONS

DL: Dead Load

E: Young’s modulus of elasticity;

EMC: Equilibrium Moisture Content

F: Force

Glulam: Glued Laminated Timber

L: Load

LL: Live Load

LVL: Laminated Veneer Lumber

MC: Moisture Content

SCL: Structural Composite Lumber

STC: Sound Transmission Class

CHAPTER I

INTRODUCTION

As the necessity for a clear and wide span in architecture, various forms, span systems and materials have been designed and developed by architects and engineers. Due to technological development, cost rates and consideration of aesthetic appeal, structural timber is widely used for medium and long span roof structure purposes in Europe and U.S., on the contrary, among many structural materials, steel is probably the most widely used material in Turkey for similar needs. This research comprises structural timber and steel, their advantages and disadvantages are compared due to performance criteria for medium and long span roof structures.

Deciding the accurate structural system for a construction, depends not only on the span or the ceiling height but also to the architectural form and image of the building. Theoretically, many supporting systems can be applied to any length of the span. On the contrary, when economy is taken into consideration; if the span increases, the structural system has to bear more loads. Erection time, transportation, material strength, workmanship, avaibility of the material may effect on the decision of the structural system.

Although there is a timber construction heritage in Turkey’s architecture history, it is not widely used in spanning of a roof structure. The possible reasons are considered as the lack of technology in timber construction, high cost rates, lack of high quality raw material and workmanship. Due to preceeding reasons, steel is the first choice in construction of large span roof structures. Without considering if it is the appropriate solution for the space or not, it is used at sports halls, swimming pools, market places, super and gross markets, factories, ateliers, storages wherever a medium or long span structure is needed. However structural timber and steel have different properties when compared to each other, specifying the material and the construction method should be done by considering these properties.

Understanding the timber roof structures and its performances, may lead an increase in the application of timber roof spanning in Turkey. The research is not prepared to defence timber against steel, but a comparison may atract the attention of architecture students and young architetcs, so that timber roof structures for medium and long span roof structures may be introduced and developed in Turkey's architecture medium.

I.1 Definition of the Problem

Timber has a potential for designing a medium and long span roof structure. Although timber is a stuctural material, its structural capacity is not well known and steel is prefererred when a medium or long span roof structure is needed. Different forms and systems have been developed in the western countries, however in Turkey its potential is not well known and not applicated due to various reasons such as cost, lack of knowledge, lack of demand and production. This is a problem for architects and architecture students, because whenever a medium and long span roof structure is needed, steel has the priority in the first choice. This creates serious problems for buildings, because the originality and appropriateness of the structural systems and materials do not satisfy the designers. An architecture composed of same repetitions of standard steel roof forms is not satisfactory for professionals.

There are a few firms that are specialized in structural timber application and design, especially for glued laminated timber, moreover the applications are mainly for hotels at touristic resorts for swimming pools. The use of timber has many advantages, of this its warm appearence. Especially roof structures made out of glulam elements is an attraction point for people. Besides its aesthetic appeal, timber has a high performance for medium and long span due to its structural, constructional and material properties.

The research is needed because at the design stage of a project, architects should be very knowledgeable while deciding the structural system. Generally the firms which are the vendor of a foreign glulam firm, are knowledgeable about the subject; at least an ordinary architect or engineer should be able design the preliminary projects and conceptual drawings. Many of the architects do not have the necassary knowledge on the span and the choice of the structure. Due to lack of information about the subject, generally steel is the first choice for a roof span. On the other hand, this may cause some problems about the general view of the cities.

Some time later, the man made environment may become very dull and unsatisfactory, due to the repetitions of the same building techniques and materials without any or less quality. Additionally this negatively effects the culture of architecture due to the same roof structures. Originality and locality is an important factor for the culture and civilization. When structural timber is studied, architects may find out that they have a different solution for spanning of a space which will lead alternative projects. For some kinds of roofs such as swimming pools and factories, structural timber gives very satisfactory results, because it is resistant to many of the chemicals and it is not affected easly like steel.

When erection time, fire resistance, dead weight, durability, transportation, heat resistance and expansion, and cost are compared between timber and steel, different results can be acquired. Considering these results, the choice for the span system and the material should be made. At different cases and different projects both steel and timber may be the only best solution for the project. But in order to make the best choice, potentials and performances of the two materials should be well known.

I.2 Objective of the Research

Timber and steel are the two closest structural material alternatives for roof spanning of a space. Both, the structures designed with two materials are erected on site or can be prefabricated. The objective of this research is to compare the performances of timber and steel thus, an architect, could be able to visualize the potential of timber. The list of performances are selected regarding the structural, constructional and material characteristics of these two elements.

Timber design considerations effect not only the structure; also the image of the building. In timber designs, every detail effect the joints and this results in the change of the appearance. In timber design, the appropriate structural system should be carefully selected and the details should be accurately decided. Timber, as a structural material allows us to design various roof forms when used on the large spaces.

Thus, the main objective of the research is to create a basis and konowledge on the characteristics and the potential of timber. The systematic of the research is kept general so that it may be extended in the future for other types of structural materials for the construction of medium and long spans. By comparing the performances of structural timber and steel, evaluation of the two materials will be possible. At the end of the research different results for both materials is acquiered.

I.3 Scope of the Research

The scope of the research for the examples and documents is limited with the European countries, the U.S.A., Canada and Australia, where structural timber is widely used for roof structures. In the technical background chapter, photographs and drawings are used to visualize original projects from these countries. The scope is limited with the previously listed countries due to the technology and the knowledge developed in the structural timber subject. The applicated projects from Turkey are also given at the end of the research to have a comparison with foreign countries. The projects are selected from the last 15 years due to the development in solid timber and engineered wood products such as glued laminated timber.

The research is limited with the performance criteria namely: structural, constructional and material characteristics of timber and steel. The research is enriched with formulas, engineering data and tables. Data on material characteristics of timber and steel is especially introduced in order to have a comprasion in a scientific scope. Various searchers have prepared pages of tables about the strength properties of structural timber and steel, these kind of technical information is especially not included, in order to limit the framework of the research. The scope also includes the contemporary situation for structural timber and steel in Turkey, the problems in production, erection, transportation, cost and maintanence are also introduced.

I.4 Materials and the Method

The method is a comparative study for the roof structure performance of timber and steel. Literature survey is done to examine and construct a basis for the roof structures, timber and steel. The method is a comparetive study, so that the properties of timber would be understood better. A direct comparison is limited in the research, because in a test specimen timber or steel would have different results; on the other hand a structure is not only evaluated by unit strength properties. When a structure is assessed in terms of strength efficiency, total load of the structure, diagonal members, connections, sections depths, volume all effect the strength properties.

The materials used in the research are composed of documents for visualization, tables for technical data and figures for detailing, introduced to construct a basis for the subject. In order to provide reference, Code of Practice, Britisih Standards, National Design Specification ‘NDS’, Uniform Building Code ‘UBC’, Timber Construction Manual ‘TMC’ is used. Internet sources are accessed to collect current information. Contemporary buildings are selected for the examples, so that the technology and the development in western countries are researched. Surveys are done to follow contemporary events and facts in Turkey and the foreign countries. Structural timber and steel producers, factories and the internet media is surveyed.

I.5 Terms and Definitions

Terms and definitions depend on the country and geographical condition for structural timber and steel. The terms ‘log’, ‘timber’, ‘wood’, ‘lumber’, ‘board’ may change by the cultures. Generally in the American sources the term lumber is used, but the British refer it is as rough timber. The term timber is used for solid timber pieces which have dimensions more than 10 cm in the American sources. Some researchers defined timber as structural lumber. When construction is suggested timber, additionally when production or the raw material is claimed, wood is used. Engineered products are defined as wood products. The researcher defined and used ‘wood’ as the main substance of the tree, ‘timber’ as the general term of the wood products, ‘log’ as the trunk of the tree cleaned from branches, ‘lumber’ as the rectangular cut pieces of the logs.

In the further chapters the terms about wood’s structure and strength are used. Further information about the wood’s structure and strength is given in the Appendix part. While using the terms the researcher was sensitive on not to use detailed definitions and information. The general content is on the performance of timber and steel on roof structure, in order to be close to the subject, unnecessary terms and definitions about timber and steel are not indicated. Terms related with performance criteria such as moisture content, strength, stress, erection, acoustics, chemical properties are indicated in the related chapters and the glossary.

CHAPTER II

TECHNICAL BACKGROUND

In order to compare the performance of timber and steel, a technical background is necessary for roof structures and structural elements. Basicly roof structures is summarized in terms of structural elements, spans, classification of roof structures and structural systems for roofs.

II.1 Structural Elements for Timber and Steel

Structural timber is produced from lumber, which varies by the given species, grade, cross-sectional size, and length; additionally it differs in one country from another. Different searchers defined and classified lumber, and sizes in various kinds. The designer should contact a local lumber supplier, to determine what kind of lumber is commonly stocked or produced (Fig.II.1). In order to use lumber commercially, moisture content shall be reduced by kiln drying or air drying. According to Olin, (1990:201.18) drying the lumber increases strength, reduces shrinkage and fungus attack, moreover improves the capacity to receive pressure preservative treatment. Kiln drying is a more effective method to reduce the moisture content and mostly used for structural products.

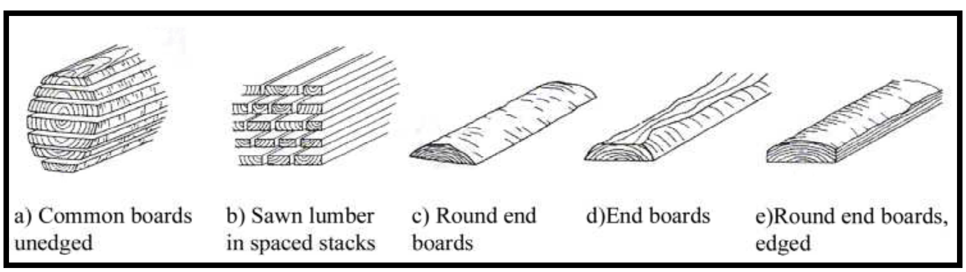


Figure II. 1 Five cutting ways of sawn lumber (Götz *et al.*, 1989:21).

Theoretically, lumber is cut from a log in two distinct ways: a) tangentially to the annual growth ring, called plain sawed in hardwoods, flat grained in softwoods;

b) radially to the rings, called quarter sawed in hardwoods, edge grained in softwoods (Olin, 1990:201.22). Plain sawed lumber is less expensive, requires less labor and waste; shrinking and swelling is less in thickness; generally has a less weaking effect (Fig. II.2). Quarter sawed lumber is more costly to produce, shrinking and swelling is less in width; it tends to wear more evenly because the radial surface is more uniform.

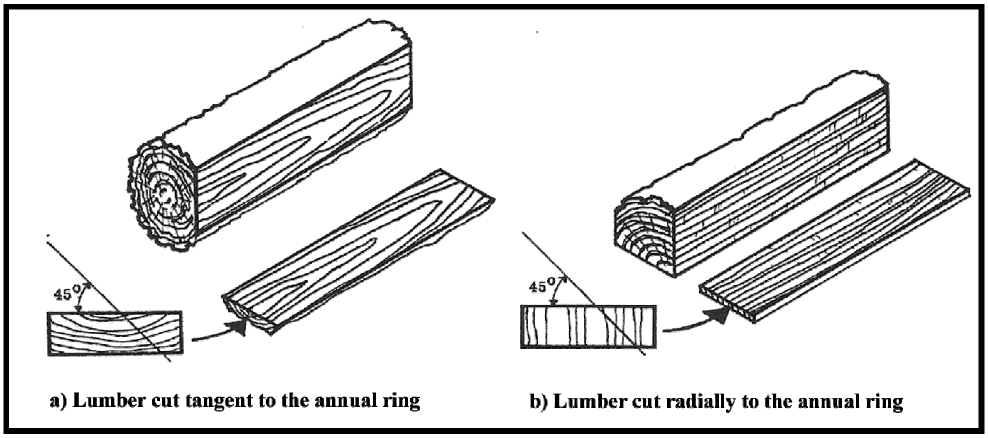


Figure II. 2 Lumber is cut tangent or radially to the annual ring (Olin, 1990:201.22).

Sizes and types of lumber differ in various parts of the world. Faherty, stated the standards and types in America; Götz, Hoor, Möhler and Natterer considered the standards and applications in Germany. Terms ‘dimension’, ‘timber’ and ‘boards’ are used to define a specific type of lumber. According to Faherty (1989:21-25) structural elements for structural timber are in three basic types and this classification is mainly used in North America:

1. Sawn Lumber: a) Boards: Less than 5 cm in nominal[1] thickness; b) Dimension: From 5 cm to 10 cm in nominal thickness; c) ‘Timber’: Nominal thickness of 10 cm or more. 2. Structural Composite Lumber (SCL): Structural Composite Lumber (SCL) core is manufactured with a network of lumber strands laminated together with a waterproof adhesive to form a single, solid, stable core reducing the risk of warp. Industry standards for SCL are still under development. SCL is avaible as in both

[1] The term nominal is derived from the dimension resulting from the rough sawing of unseasoned (green) lumber. Planing or seasoning results in a product of smaller cross section, but the nominal dimension is applied. A nominal 5x10 cm stud is 3.8x8.9 cm in the fully dried and planed state (Faherty, 1989:22).

'boards' and 'dimensions', and produced in dimensions essentially compatible with sawn lumber sizes. It can be obtained in widths of up to 60 cm net on order. 3. Glued Laminated Timber: Structural timber, components of large sizes and complex shapes can be fabricated from smaller sawn sections (laminates) by the process of glued lamination, known as 'glulam' (Fig. II.3). Glued laminated timber is mainly used for flexural members, columns, arches, truss members and decking. Because a degree of homogenization is achieved, glulam structural members and products have higher allowable stresses and more reliable stiffness than sawn timber. Glulam is produced of kiln-dried lumber and do not have the problems of shrinkage and secondary stress generation.

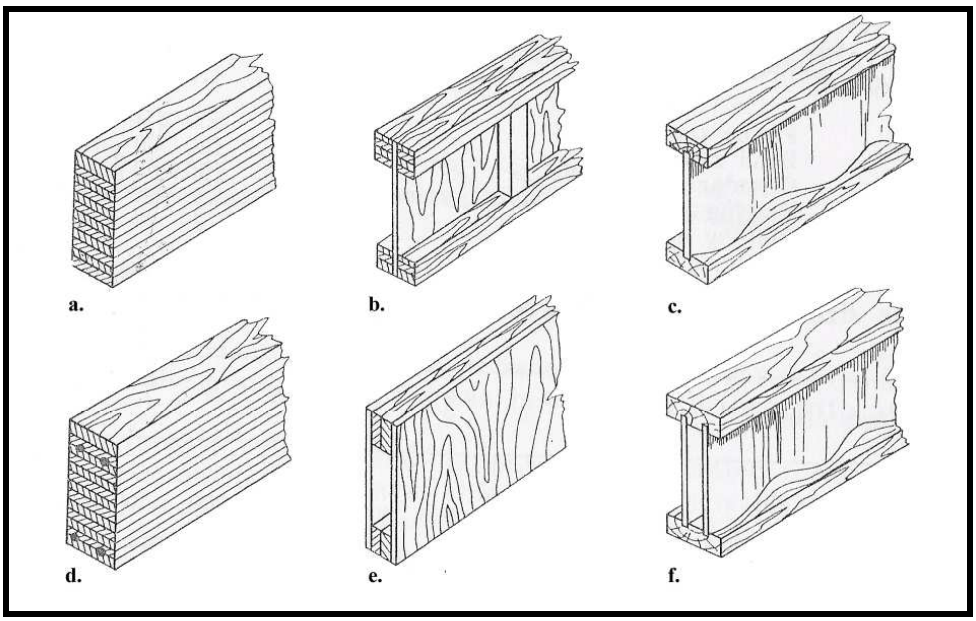


Figure II. 3 Typical sections of glued laminated structures: a) solid rectangular section, b) 'I' section with a plywood web, c) 'I' section with a corrugated web, d) reinforced section, e) box shaped section, f) box shaped corrugated section (Karlsen, 1989:377).

Halperin (1994:2) stated the sizes of structural timber according to thickness, width and grading (Table II.1). Götz *et al.* (1989:19), classified lumber into five types and defined the sizes and standards referring to German application: a) Strips: Timber having a cross section of up to 32 cm² and a width of less than 8 cm. b) Boards: Lumber having a thickness of 0.8 cm minimum and 4 cm maximum, and a minimum width of 8 cm. c) Planks: Lumber having a minimum thickness of 4 cm. The larger side of the cross section is at least twice as large as the smaller side. d) Dimension lumber: Lumber with a square or rectangular cross section having a relation of side widths of up to 1:3. Minimum side width is 6 cm. e) Timbers: Lumber where the width of its larger side is greater than or equal to 20 cm.

Table II. 1 Size classification and grades of structural timber (Halperin, 1994:2).

Category	Thickness (cm)	Width (cm)	Grades
Dimensional lumber	5-10	Any width	
Structural light framing	5-10	5-10	Select structural Nos, 1,2,3
Light framing	5-10	5-10	Construction utility
Stud	5-10	5-15	Select structural Nos, 1,2,3
Joists and planks	5-10	10 and wider	Select structural Nos, 1,2,3
Beams and stringers	10 and thicker	Widths more than 5 greater than thickness	
Posts and timbers	10 and thicker	10 and wider	Select structural Nos, 1,2,3

In the U.S.A. practice, softwood is sawed into three broad classes of lumber: a) Yard; b) Factory and shop; c) Structural lumber. Yard lumber is used for light construction and finish work. Factory and shop lumber is used for door, sash, cabinet and other millwork elements. It may contain defects such as knots and defects which may be disregarded. Structural lumber is intended for heavy construction and it is 5 cm thick or greater (Karlsen, 1989:34).

Structural timber is classified by a grading system[2] according to the type and size of the defects; moreover gradings are given due to the strength properties of the selected specie (Fig. II.4). This permits dealers to provide materials which are uniformly suitable for their intended uses, it is an advantage for both the dealer and the user (Oberg, 1963:84). DIN 4074 establishes three grading systems:

a) Grade 1: Lumber with especially high structural strength;

b) Grade 2: Lumber with ordinary strength;

c) Grade 3: Lumber with lesser structural strength (Götz *et al.*, 1989:16).

[2] For the grading system, the growth rings should not be more than 5 mm wide and should contain at least 20% latewood and no pith areas are allowed in grade 1 and 2 to be used in the outer tension zone (Karlsen, 1989:34).

Wayne, (1991:78) stated that glulam is not only rated for stress but also for appearence: industrial, architectural and premium. The three differ in the visible defects such as knots, knot holes, knot size, voids, areas of insufficient surfacing, color, grain laminations and eased corners.

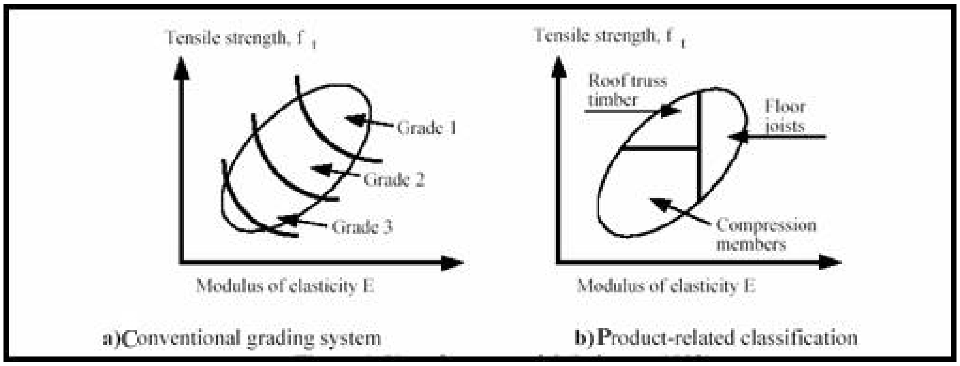


Figure II. 4 Mechanical properties of timber products required by end users (Kliger, 2000, document on-line).

According to The Engineered Wood Association, APA, engineered wood products are classified as a) Glulam, b) Structural composite lumber: laminated veneer lumber, parallel strand lumber, oriented strand lumber.

a) Glulam: Glulam is an engineered stress-rated product created by bonding together individual pieces of lumber having a thickness of 5 cm or less. Glulam is manufactured from laminates[3] of sawn timber, planed to a smooth surface, before being glued together with the grain in the laminates running essentially parallel. Güller, (2001:151) claimed that for curved members laminates with 2.54 cm (1 inch) width are used, for linear or slightly curved members laminates with 5.08 cm width are used. Basicly, glulam is not stronger than lumber since gluing has no effect on strength, nonetheless by wisely selected laminations the bending strength exceeds that of lumber. Virtually any size of cross section and length of component can be

3 Individual laminates can be end-jointed by the process of finger jointing to produce long lengths. Beams wider than the normal commercially available laminates can be manufactured by edge-gluing after finger jointing. The lay-up of these wide laminates is arranged so that both edge and finger joints are staggered.

produced from glulam (Fig.II.5). The constraints are those imposed by transportation and individual manufacturer's facilities. Spans of 40 m and depths of 2 m have been produced, but spans up to about 30 m are more usual. In the USA, highly competitive domed structures spanning 150 m have been built using glulam ribs.

b) Structural composite lumber: It is the combination of wood veneers, veneer strands are bonded together with adhesives. It is classified as:

1) Laminated veneer lumber (LVL): It is produced by bonding thin wood veneers together in a large billet. The grain of all veneers is parallel to the long direction. The LVL billet is then sawn to desired dimensions depending on the construction application. Güller, (2001:151) suggested that veneers which are bounded parallel or vertical to each other, have 3.2 cm width, greater dimensions are used in glulam manufacturing.

2) Parallel strand lumber (PSL): PSL consists of long veneer strands laid in parallel formation and bonded together with an adhesive to form the finished structural section. Like LVL and glulams, this product is used for beam and header applications where high bending strength is needed. Veneers are sawn from lumber, pieces with 18 cm width and 20.32 cm length are produced. In the further step these pieces are pressed with adhesives and necassary dimensions for the market are produced (Güller:2001:151).

3) Oriented strand lumber (OSL): Similar to PSL, oriented strand lumber is made from flaked wood strands that have a high length to thickness ratio. Combined with an adhesive, the strands are oriented and formed into a large mat or billet and pressed (The Engineered Wood Association, document on-line).

Hornbostel and Hornung, suggested that for spans up to 7.31 m solid timbers are generally cheaper than laminated sections except where extremely heavy loads are implied. Beyond this limit glulam beams or other structural systems should be used (1982:180). Erdoğan, (2002:114) claimed that glulam is more expensive than solid wood due to the need of more materials, high fabricating costs and the difficulties in transporting large members. However glulam gives a freedom in design, ease of installation, durability and efficient utilization. According to Desch, (1996:200) the most favourable property of glulam is, its stength to weigth ratio and high flexural rigidity compared with other materials. It gives a great degree of freedom in design when compared to solid timber.

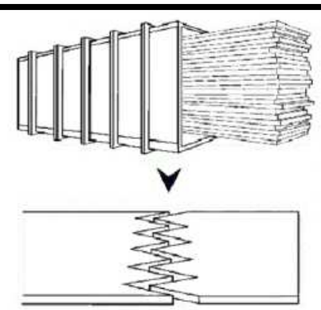


Figure II. 5 Production of glulam is composed of drying and jointing periods of laminates (Glued Laminated Timber Association, Glulam Specifiers Guide, document on-line).

In steel construction, structural steels are identified by the ASTM designation number that specifies the steel (Table II.2). The most commonly used steel is graded as A 36 steel. There are several steels that have more structural abilities and that are known as high strength steel. ASTM 588 is intended primarily for use in welded bridges and buildings where decreased weight of steel, improved durability and weather resistance is required (Ellison, 1987:198). Steel is also produced in different forms and techniques. Parker, (1994:263) summarized the production techniques of steel as:

a) Rolled shapes; b) Wire; c) Extrusion; d) Casting; e) Forging.

a) Rolled shapes: The process of rolling is used to produce a variety of linear elements including rods, bars and plates. b) Wire: This is formed by pulling the steel through a small opening. c) Extrusion: This is similar to drawing, although the sections produced are other than simple round shapes. d) Casting: This is done by pouring the molten steel in to a form. e) Forging: This consists of pounding the softened steel into a mold until it takes the shape of the mold.

Table II. 2 Types of structural steel is graded with a designation (Callender, 1982:2.77).

ASTM Designation	**Primary Use in Construction**
	Carbon Steel
A36	All purpose carbon grade steel used for construction of buildings and bridges.
	High Strength Low Alloy Steel
A242	For exceptionally high corrosion resistance, more expensive, suitable for use in uncoated condition.
A440	Riveted or bolted structures where weight saving is important , corrosion resistance twice that of carbon steel, not recommended for welding.
A441	Welded structures where weigth saving is important, corrosion resistance twice that of carbon steel.
A572	Excellent formability and weldability, economical where strength and light weight are vital design objectives.
A588	The atmospheric corrosion resistance of this steel is 4-6 times that of carbon steel, if unpainted, for uses where weight reduction, weldability and maintanence costs are considerations.

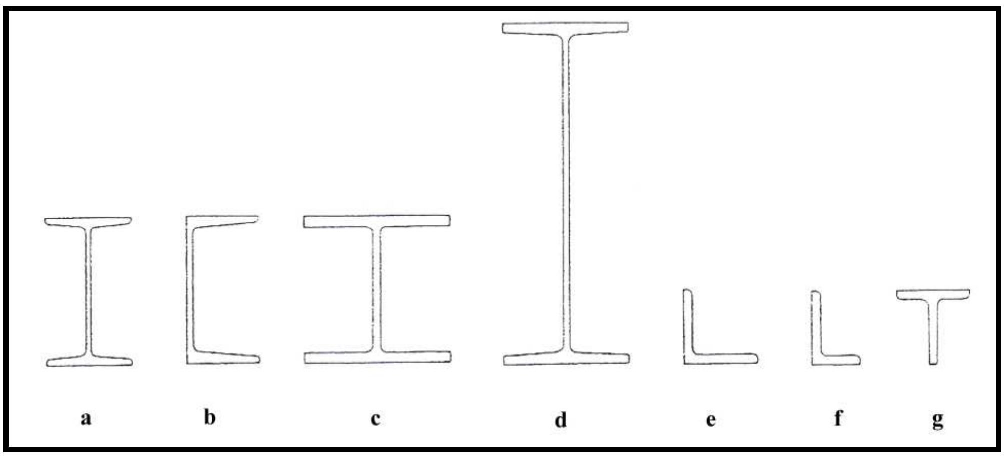


Figure II. 6 Typical hot rolled steel sections (Foster, 1983:143).

Continuous casting, which is used to transform molten steel into finished forms is, the most widely used method of steel production. Rolled products can be divided into sections and plates. Sections can be hot rolled or cold rolled, that hot rolling must be done in red hot state, whereas cold rolling is done at room temperature (Fig. II.6,

Fig. II.7). Steel trusses are commonly produced from welded hot rolled shapes, using 'T' sections as top and bottom chords and angles as diagonal members or entirely composed of welded tubular sections (Eggen *et al.,* 1995:37-38).

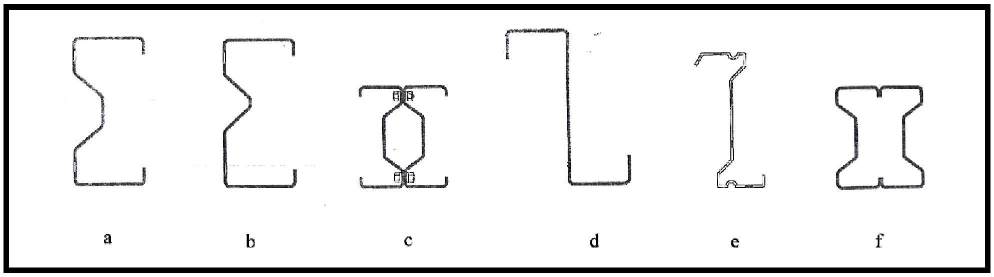


Figure II. 7 Typical cold rolled steel sections (Owens, 1992:429).

According to IIIingworth, (1993:278-279) the availability of steel sections in variety of dimensional changes provides designer with a wide variety of options. The addition of hollow sections widens more the scope of steel, where the hollow section is strong but light means of construction.

Table II. 3 Identification of structural steel shapes (Merrit, 1982:8.2).

Section	**Symbol**
Wide flange shapes	W
Standard I shapes	S
Bearing-pile shapes	HP
Similar shapes that cannot be grouped in W, S or HP	M
Structural tees cut from W,S or M shapes	WT, ST, MT
American standard channels	C
All other channel shapes	MC
Angles	L

Hot rolled shapes include bars, plates and angles, primarily to withstand tension forces. Steel shapes like 'W', 'S' and 'M' are designed for compression and bending, such as in columns and beams. Merrit, (1982:8.1) claimed that shapes are identified by their cross-sectional characteristics: angles, channels, beams, columns, tees, pipe, tubing and piles. Structural shapes are simply identified by letter symbols (Table II.3). The shapes are delivered in standard lengths but can also be produced in custom-cut dimensions. Plate products are classified as: a) Thick, 5 mm or more; b) Medium, 3-5 mm; c) Thin, under 3 mm. Built up steel beams and products are designed for different and specific structural purposes. They can be special-ordered and delivered in a short time. Welded plate girders are generally used in bridge construction.

Castellated beams are made by cutting a ‘W’ beam or ‘S’ beam longitudinally along the web in a straight or curved zigzag shape (Fig. II.8). The two halves of the beam seperated, displaced and later welded together; so that bearing capacity of the beam increases by 100% without any increase in weight. Tubular steel is produced in circular or rectangular sections, withstand torsion forces. Corrugated steel sheets are produced from cold milled sheets and generally coated with zinc, aluminium, or both. Corrugated steel is frequently used as siding or roofing of light prefabricated buildings, industrial sheds and the like (Eggen *et al.,* 1995:40).

Carbon greatly effects the physical properties of steel. The amount of carbon may vary between 0 to 1.5%. Increasing the carbon content, increases the strength, hardness and brittleness of steel but decreases its ductility. Ellison, (1987:196) classified the steel according to carbon content as: a) Low-carbon steel with a carbon content between 0.06% and 0.30%, b) Medium carbon steel with a carbon content between 0.30% and 0.50%, c) High carbon steel with a carbon content between 0.50% and 0.80%. The steel production starts with combining together three principle raw materials: iron ore, coal and limestone. Various alloying elements are introduced into steel to change the properties in order to increase strength and resistance to rust. These elements are silicon, manganese, copper, nickel, chromium, tungsten, molybdenum and vanadium. Such steels are called alloy steels.

Owens stated that steel is graded in the Eurocode and BS 4360 specification for the weldable structural steels, notch ductility requirements are specified by grades A to F. According to these requirements steel should show a minimum of 27 J energy absorption corresponding to the letter grade. Grade A no requirement, grade B +20ºC, grade C at 0ºC, grade D at -20ºC, grade E at -50ºC and grade F at -60ºC (1992:215). The European based standard, BS EN 10 025 have the two steel strength grades 43 and 50 having the yield strengths 275 N/mm² and 355 N/mm² (Owens,1992:224).

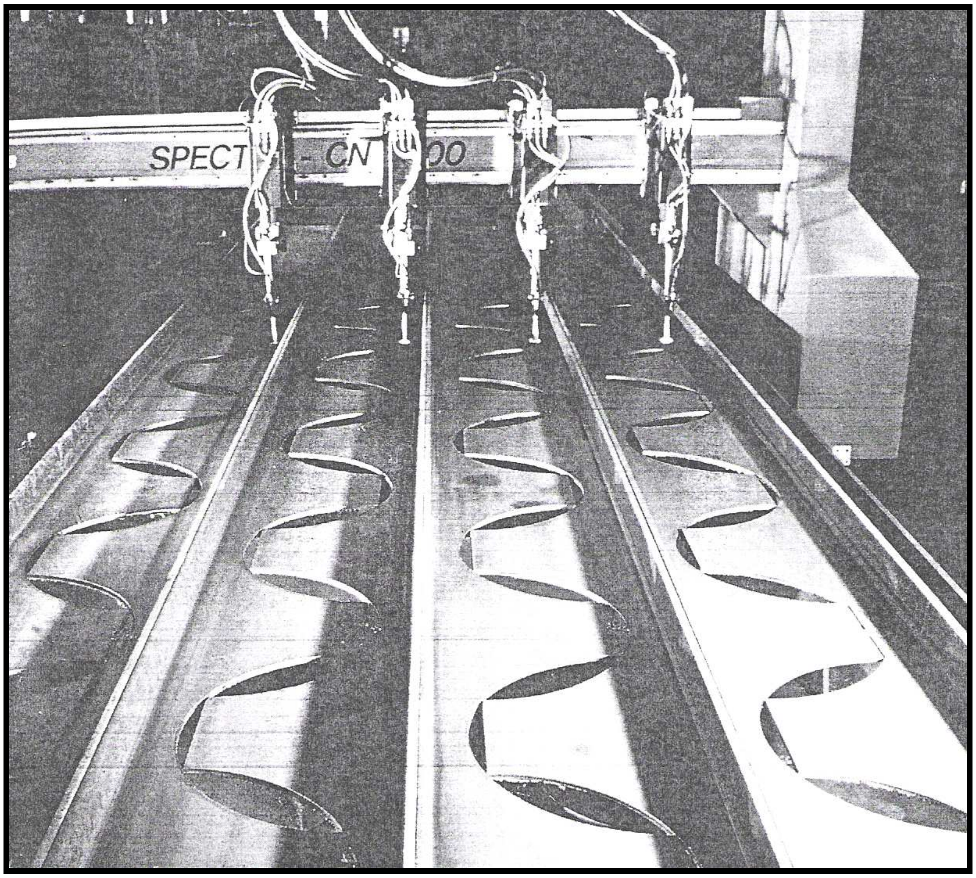


Figure II. 8 Production of castellated beams with the help of digital blow torches at Westok Structural Services Ltd., Wakfield, England (Eggen *et al.*, 1995:39).

The researcher claimed that both structural timber and steel are graded with special designations depending on the region of the production. This grading system is necassary for a uniform construction, to simplify calculations and site applicaitons. Manufacturers produce the raw material according to these specifications, so that faults are decreased during application period. Generally accepted gradings are inserted in the Chapter II.1. Unfortunately in Turkey there is not a specific grading system for structural timber. Raw lumber is graded as grade 1, 2 and 3; but for structural purposes there is not an established system to simplify stress calculations.

II.2 Roof Structures

The basic function of a roof is to enclose a space and protect it from outer conditions. A roof structure should satisfy the requirements such as strength and stability, weather resistance, thermal insulation, fire resistance and sound insulation (Foster, 1983:162). According to Ambrose, (1990:66) designing a roof structure requires to deal with two major functional problems: the generation of the general roof construction, form, structure, and the development of the means for making the roof watertight. Vandenberg, (1974:191) denoted that a roof structure must keep out water, has to give protection against sun, resist the destructive effects of radiation and alternate exposure to heat and cold, moderate heat losses to acceptable limits.

Every building contains one or more structural elements and the overall structural system is composed of the combination of these elements which generally consists of the main framework, secondary members roofing, stiffeners, and foundations. Once the layout, the interior space, the span of a building is determined, the structural design starts with the computation of the roof, determining to insulate the roof or not, the kind of the material of roofing can effect the length and appearance.

These complex and many sided considerations make it difficult to choose the appropriate structural system and material. Such considerations need a three dimensional visualisation of the flow of forces. But structural optimisation is not the only criteria in the choice of the system and in the comparison of the alternatives. The arrangement depend also on the other functional needs such as heating, ventilating, sprinklers due to trussed members, lightning governing the structural plan, or loud speakers moreover lighting can provide additional load. These considerations and the architectural form influence the choice of the structural system. According to Toydemir, roof structures design parameters depend on the roof structures' bearing system, insulation for heat, light, sound and water; design and architectural concept (2004:18). The most appropriate form of the roof structure is decided due to the type of the building, foundation conditions, spans to be covered, nature and magnitude of the loads, lighting requirements, accomodation for services, the possibility of the future alteration, speed of erection, and aesthetic considerations (Foster, 1983:339).

Ambrose, (1990:67) suggested that the selection of the basic roof structure and the general roof construction must respond to various issues including: a) The materials and basic construction of the rest of the building; b) The dimension of required horizontal clear span; c) Special needs for a particular geometric form; d) The form and the layout of the supports; e) Penetrations of the roof for vents, elevators, skylights, and so on; f) Acceptable range for deformations; g) General architectural impact of the roof in terms of its form, visibility of the top surface, and the possible exposure of the underside structure.

The researcher claimed that a roof structure is composed of series of elements:
a) Structural members like beams, 'I' joists, purlins, struts, rafters; b) Lighting fixtures, c) Ventilating elements like air ducts; d) Covering materials for insulation, acoustics and decking purposes. If the structure is exposed, these elements are designed harmonious with the form; on the contrary if the structural elements are hidden, secondary members may pass over the suspended ceiling level. A roof composed of structural members, these members are: beam, truss, frame 'I' joist, shaped lineer elements to construct a dome or shell and shaped ribs to form a plate or frame. Further information about main structural elements are indicated in Chapter II.5.

Roof structures is summarized in terms of classification and span. Firstly, classification of roof structures will be done; secondly, span definition, the lengths to discuss medium and long span will be defined and then length of simple, medium and long span will be discussed. Type of the roof structure and span length is related to each other; one type of roof structure may be appropriate in medium span but inefficient in long span.

II.2.1 Structural Members in a Roof Construction:

Structural members that span the distance between main elements, hold insulation, cladding, lighting and ventilation, moreover they constitude three dimensional stiffness. Structural elements (Fig. II.9) that compose a timber roof structure are:
a) Rafter (*mertek*); b) Purlin (*aşık*); c) Strut (d*ikme*); d) Diagonal (*göğüsleme*);
e) Collar (*kuşaklama*); f) Siding (*yanlama*); h) Tension beam (*gergi kirişi*); i) Cladding board (*kaplama tahtası*).

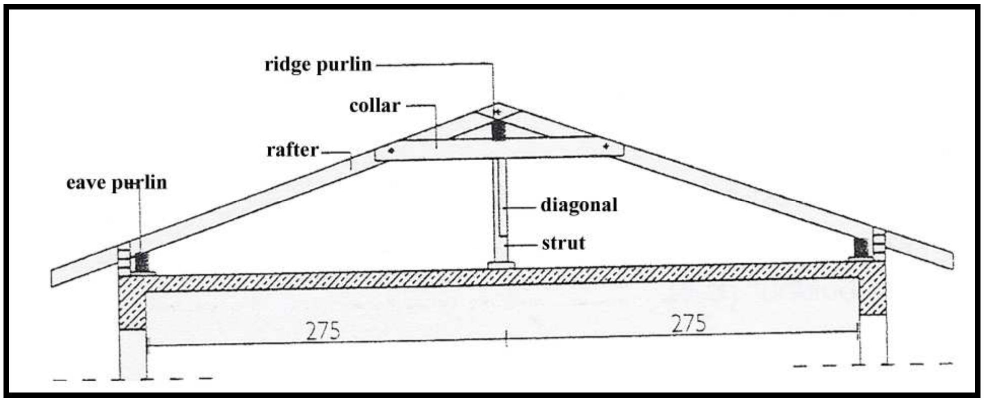


Figure II. 9 Structural members in a hip roof construction (Toydemir, 2004:40).

Rafter, is the construction element that transfers the loads from under cover plank to purlins. Rafters are placed for 2.75-3.00 m span with 45-55 cm axial dimensions, and have section dimensions 5x10 cm or 6x12 cm.

Purlin, in a roof is the element that transmits the roof loads from rafters to struts. Purlins are perpendicular to the slope of the roof. Purlins are generally have the dimensions of 10x14 cm, 10x16 cm or 12x16 cm. Purlins are placed in a 3.75-4.00 m span. The span length may be increased by diagonals. If the purlin is placed over the rafter it is named as rafter purlin, when it is placed at the edge of the rafter it is called edge purlin.

Struts, are the vertical elements that transmit the loads from rafters and purlins to a floor or buildings load bearing system. In the hip roofs, struts transmit the loads to floor or walls. In the hung roofs the name of this vertical member is king. Struts generally have the dimensions about 10x10 cm or 12 x12 cm. Struts are bounded to floor, reinforcement or structural system by cushions, that have the dimensions about 4x12 cm or 5x12 cm.

Diagonal, is the member that is placed between strut and purlin with 45º and acquire the the stability of strut and purlin. Diagonals are placed to double sides of the struts, and should be connected by bolts. Diagonals generally have the dimensions of 8x8 cm or 10x10 cm.

Collar, is the element in the roof that connects the struts horizontally and vertical to the purlins. Collars are generally horizontal in case purlins are not at the same level, collar angle may change. Collars are generally applicated double and continuous. The connection between collars and struts must be with bolts, but nails to connection the collars with rafters is enough.

Siding, is an element of hung roof. The vertical load come from purlins to struts are transmitted through siding slope. Sidings are in rectangular section and have the dimensions 10x10 cm, 12x12 cm or 14x14 cm. If double sidings are placed, section dimensions are more than the half of the necassary dimension.

Cladding board, is below the roof tile or metal cover in order to have a flat surface. Under cladding board are generally in the dimensions of 2.5 by 20-25 cm sections for roof tile cladding; however if metal cover is used, under cladding boards have the same dimensions but a tongue and groove joint should be added. Under cladding plank transfers the roof loads like roof tile or snow to rafters. Over the under cladding tile, a membrane must be set in order to prevent water penetration.

If it is assumed that roofing material and the flashing prevent water penetration from exterior, and then any potential problem is likely to be caused by the penetration of water vapour from inside of the building. The accumulation of water vapour in roof spaces must be prevented by the provision of adequate ventilation. In BS 5250, it is paraphrased as a) A 10 mm gap continuously along each side of a roof pitch over 15º, b) A 25 mm gap continuously along each side of a roof with a pitch under 15º.

According to Burchell and Sunter, (1987:27) ventilation should be done by ridge vents, gable end vents or individual eaves vents. In cold flat roof construction a vapour barrier must be installed between the insulation and ceiling lining. It must be specially cared that installation of insulation does not block the passage of ventilation.

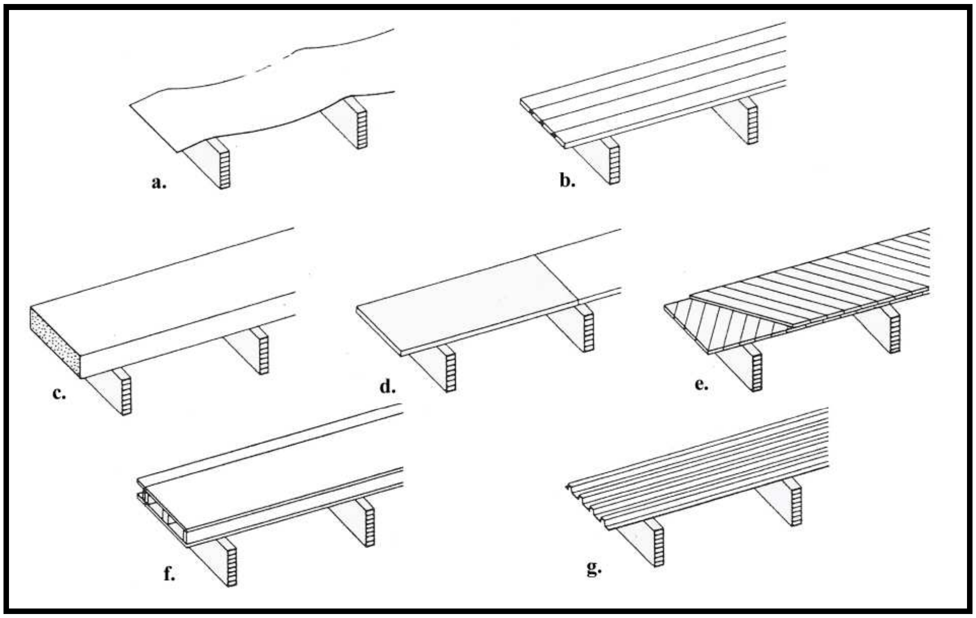


Figure II. 10 Roof decking arrangements are: a) Fabric or plastic roof on braced beams, b) Tongue or groove planking or splined boards, c) Composite materials, d) Wood chipboards, plywood or furniture panels, e) Diagonal planking, f) Panel elements of timber slats or sheating, g) Trapezoidal sheet metal (Götz *et al.,* 1989:127).

Decking is the material to enclose the space between the load bearing ribs (Fig. II.10). Decking can be decided according to the unit weight of the material, architectural image if exposed, span length between the main ribs, amount of secondary members. Some decking systems can support itself without placing secondary supports, of course structural form should be appropriate for such system.

II.2.2 Classification of Roof Structures

During the research, it is observed that different searchers classified roofs due to their point of interest. Roof structures are classified according to the structural system, span and form. According to Binan, classification of roof structures are: a) Hip (*beşik çatı*); b) Ridge (*kırma*); c) Mansard; d) One way slope; e) Cross form; f) Tent and flat (1990:3-7). When the structural system is considered, Binan classified the timber roofs as hip (*oturtma*), hung (*asma*) and mix systems (1990:57). Özdemir (2003:89), classified roof structures as: a) Lean to (*sundurma*); b) Hip; c) Ridge; d) Mansard; e) Tower; f) Monitor (*fenerli çatı*); g) Shed; h) Combined (Fig. II.11).

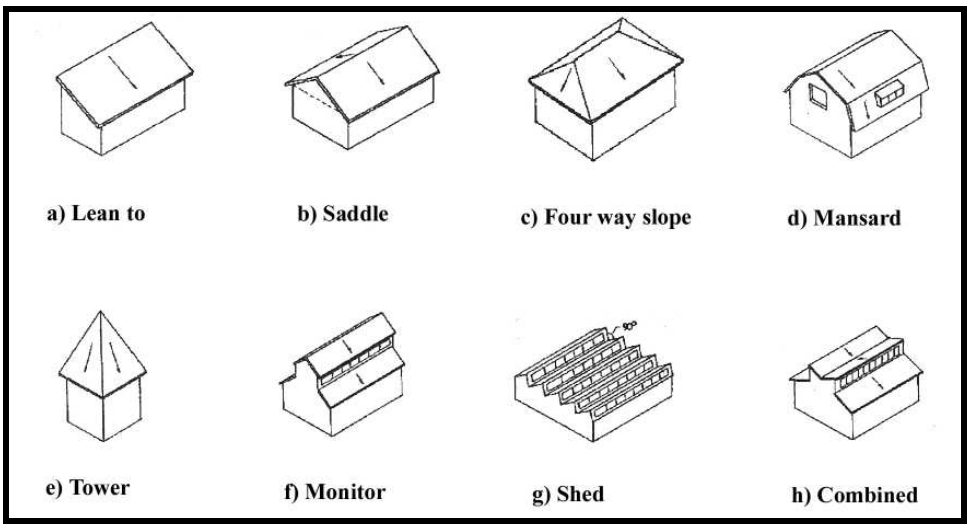


Figure II. 11 Classification of roof forms (Özdemir, 2003:90-91).

Ambrose (1990:66), sorted out the roof structures according to their forms: a) Flat with overhang; b) Flat with parapet; c) Shed; d) Hip; e) Gambrel; f) Mansard; g) Simple gable; h) Compound gable; i) Gable roof dormer; j) Skylight; k) Monitor; l) Sawtooth. If the angle or the slope of the roof surface is considered Ambrose (1990:72), categorized the roof structures as: a) Fast draining, from 3:12 slope and over; b) Intermediate, from 1:12 slope to 3:12 slope; c) Slow draining, from 0 to 1:12 slope. Foster (1983:164), stated that roof structures should be classified under 3 basic types: 1) The plane of the outer surface: a) Horizontal; b) Sloping. 2) Structural principles on which their design based, that is the manner in which the forces set by external loads are resolved within the structure of the roof. 3) Span.

Türkçü (2000:217-220), categorized the roof structures according to: 1) Geometry: a) One way slope, b) Saddle, c) Folded, d) Lean to, e) Shed, f) Mace, g) Mansard, h) Butterfly, i) Cross, j) Apex; 2) Amount of slope: a) Flat, b) Sloped; 3) Structural material: a) Timber, b) Reinforced concrete, c) Metal, d) Stone; 4) Insulation: a) Flat, b) Converted.

Toydemir (2004:24-27), claimed the classification of the roof structures should be done according to: 1) Usage of the roof: a) Used or able to be used roof structures: for terraced, garden or parking purposes. b) Unused or unable to be used roof structures: sloped roof structures over 33% slope. This slope is appropriate only for maintanence purposes. 2) Rain water: a) Sloped to the outwards; b) Sloped to the inwards. 3) Slope: a) Little sloped roofs: roofs which have a slope between 0%-25%. b) High sloped roofs: roofs which have a slope over 25%. c) Changing sloped roofs: roofs which have an arched, dome, shell, pyramidal shape. In these roofs the slope differs between 0º-90 º. 4) Covering material: a) Roof tile; b) Asbestos; c) Metal; d) Bitumin; e) Polimer; f) Glass; g) Natural stone; h) Plantation; i) Soil. 5) Form: a) One faced; b) Craddle; c) Ridge; d) Mansard; e) Dome; f) Tower; g) Shed; h) Flat or less slope. 6) Under roof ventilation: a) Cold roof-air under the roof; b) Hot roof-air without. 7) Structural system: a) Embeded; b) Hung; c) Mixed roof.

The researcher classified the roof structures due to: span; usage of the roof; form; structural material. 1) Span: a) Simple; b) Medium; c) Long. Span is related with the type of the structural material, the system and the design. Further information about span is indicated in the next chapter. 2) Usage of the roof: a) Used; b) Unused. A roof structure may be either used or unused. Generally flat roofs have a tendency to be used, sloped roofs are not used. 3) Form: a) Sloped; b) Flat; c) Dome; d) Arch; e) Shell; f) Pyramid; g) Vault; h) Folded. Form of the structure is related with the general design concept, architectural preference, structural material and interior volume. 4) Structural material: a) Timber, b) Steel, c) Reinforced concrete,

d) Aluminium, e) Plastics. Structural material critically effects the general design concept, span, form and construction technique. According to slope a classification is not indicated in the classification; a vault or dome structure also has a slope changing by the point. Covering material and insulation is not included, because both covering material, insulation and decking does not effect the selection of structural system, design, span or form. The same covering or insulation material may be applicated to a timber or steel structure.

II.3 Span

Span, is the extent or measure of space between two points or extremities, as of a bridge or roof. Span size of a roof structure is so much related with the potential of the structural material and the design. The classification of the span and the appropriate system is related with the efficiency of the structure. Increasing the span length, means the increase of span depth. For example, a truss system may span a medium length, however to increase the length, the design of the roof structure may be changed into a space frame. Roof structures may be classified in three basic span types: a) Simple; b) Medium; c) Long. These span lengths may depend on the structural material, for example for a concrete beam simple span may be considered between 3 m and 6 m, on the other hand with steel 'I' beams it may be considered between 3 m and 12 m.

Karlsen, stated that simple span is between 3 m and 4 m, medium span is up to 36 m, and large span is over 36 m (1989:265). However Foster, claimed that simple span is up to 7.6 m, medium span between 7.6 m and 24.40 m, long span over 24.40 m (1983:165). In all structural systems minumum span is limited due to system's lower feasible span. To decide the span length, system's efficiency in terms of amount of used material, time of erection, cost of the structure, structure's depth, dead weight is very important. When the feasible limits are extended, the structure becomes to be overdesign, and another type of structure related with the limit of the span should be chosen. Span sizes have been increased by years, in the further chapters various searchers have defined span sizes for different roof structures, and it has been observed that from 70s to 90s amount of spans related with structures has been increased. The increase is related with the technological developments in joint details, quality of wood products and steel.

The researcher classified the span as follows: a) Simple span, b) Medium span, c) Long span. Simple span is considered between 3 m and 8 m for structural timber. For a simple span laminated beams, box beams, trusses may be used and would be efficient. Over 8 m straight beams, sawn lumber beams or simple trusses would not be efficient, that a different structural system should be chosen. Medium span is considered between 8 m and 24 m and trusses, arches, space frames, pyramid shaped structures, open web joists, plywood folded plates would be efficient. In these limits structure is efficient with the selected form. Long span is considered over 24 m. Trusses, laminated arches, space frames, curved ribs, shells would be the appropriate solution for long span timber roof structures. Efficient span limits and related forms are summarized at the end of the conclusion (Table IV.4).

II.4 Traditional Roof Structures

Roof structure types have been enhanced due to technological developments and change of needs. In order to overwiev the contemporary structure systems, firstly the background of the topic is introduced; so that the roots of the structural development can be understood. Roof structure types are summarized in two basic topics: traditional and contemporary systems. Traditional systems comprise Anatolian and western traditions.

Traditional roof systems differ in the Anatolian and western culture. Todays contemporary timber structures are mainly based from western rafter roof tradition. Western conutries have developed their timber roof tradition and by including new materials they have enriched the contemporary structural systems.

II.4.1 Anatolian Roof Tradition

In Anatolian tradition, according to Binan (1990:58) elements of the roofs are: roof cover, roof clad, rafter, purlin, strut, tie beam, collar and diagonal. a) Roof cover (*çatı* *örtüsü)*: The upper level cover that prevents the rain and snow to penetrate the roof. b) Roof clad (*çatı kaplaması*): Depending on the covering material timber cladding on the roof. c) Rafter (*mertek*): The secondary elements that the timber cladding is nailed on. d) Purlin (*aşık*): The beams that support the rafter. Purlins are named according to their place in the roof like eave purlin (*damlalık aşığı*), middle purlin (*ara aşık*). e) Strut (*dikme*): The supports that the purlins are placed on and they are erected over a column or wall. f) Tie beam (*bırakma kirişi*): The struts are placed on tie beam is there is no a wall. g) Collar (*kuşak*): Collars connect the strut, purlin and rafter, moreover they join the structural system horizontally. h) Diagonal (*göğüsleme*): In order to decrease the wide supports and to prevent the wind effects diagonals are used. Diagonals are connected from struts to purlins with 45º.

According to Binan (1990:57) traditional structural timber roofs are classified as hip, hung and mix systems. a) Hip: Hip roof do not have long span, loads are transferred to the load bearing elements like beams and columns. b) Hung: Hung systems are used for medium spans and enclosed by beams and trusses. Trusses are shaped due to the slope of the roof. c) Mix: Used where the span changes, various trusses are used together.

Hip roofs are constructed as one purlin; two purlin and three purlin hip. a) One purlin hip roof: In the one purlin hip roof the length of the rafters or the span between the purlins horizontally are not more than 2.5-3 m. Purlin lengths are not more than 3.5-4 m. The structural system is composed of trusses placed one by one, purlins placed along the building and rafters rest on the purlins. Rafters deform under equally scattered loads. Vertical pressure of the rafters are transferred from eave purlin to outer walls, from ridge purlin to interior walls and foundation. Purlins deform by the vertical loads generated by the rafters. b) Two purlin hip roof: In the traditional roofs the span between the purlins determine the rafter spans. These spans are between 2.5-3 m, and the purlin spans are between 2.5-3 m. Two purlin roofs are appropriate for 8 m roof spans (Fig. II.12). When the span between the purlins are more than 3 m, two purlin system is used. In the two purlin roof, rafter ends support as a cantilever moreover the length of the cantilever is not more than 1.5 m. c) Three purlin hip roof: This type is used for the roof where the span length is long and the distance of the ridge and eave is between 5 m and 7 m (Fig.II.13). Construction technique is the same as two purlin hip roof. Load bearing walls should be placed. Purlins are connected by struts and collars.

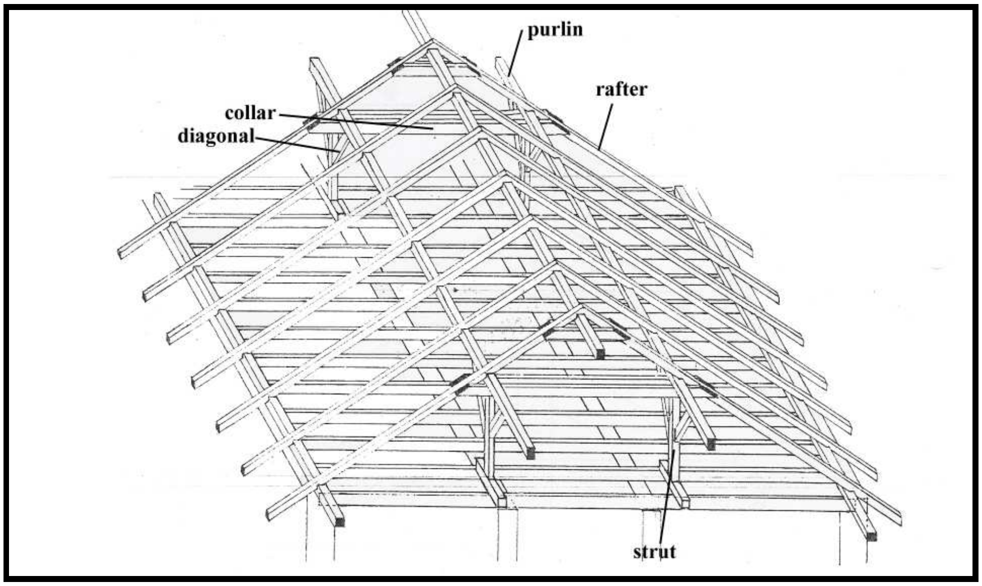


Figure II. 12 Two purlin hip roof is composed of rafters, purlins, diagonals, collars and struts. The ridge purlin is placed for easy installation (Binan, 1990:78).

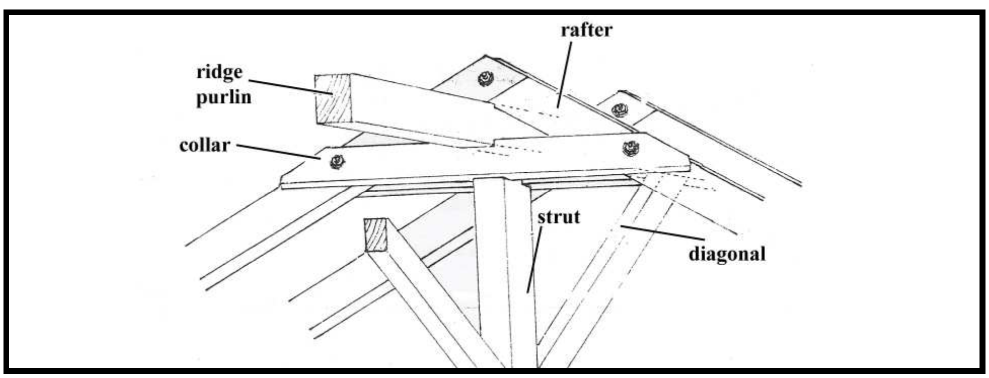


Figure II. 13 Ridge connection in a three purlin hip roof is composed of rafters, purlins, collars, struts and diagonals (Binan, 1990:82).

Hung roofs are used where the span length is more than 6 m. The joist on the ceiling is hung to the roof structure. Depending on the span and building length two or more trusses are placed at 3.5-5 m. Hung roofs are constructed as one purlin, two purlin, three purlin. a) One purlin hung roof: It is used by one purlin at 5-6 m span. If the under roof is flat ceiling, beam is hung to the king and ceiling beams are placed on the hung beam. b) Two purlin hung roof: It is used at 6-10 m span (Fig. II.14). It is composed of tie beam (*gergi kirişi*), chord (*yanlama*), header (*başlık*) and two kings. The distance between the trusses is between 3-4 m. c) Three purlin hung roof: It is used at 10-12 m span. In the long spans double kings are used and the trepozoidal system is connected with a header beam. Siding and header beam are connected between double kings.

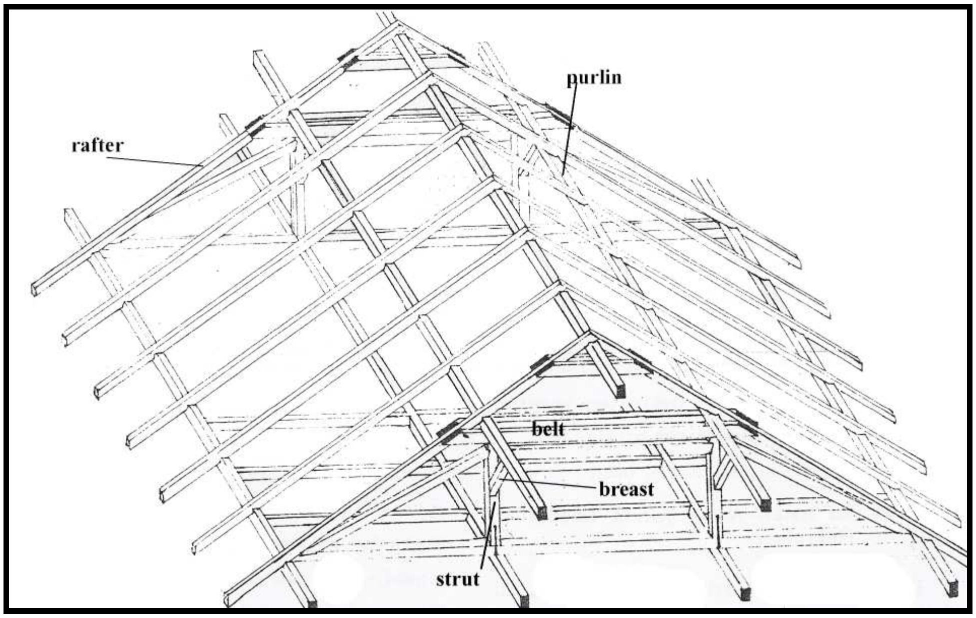


Figure II. 14 Two purlin hung roof is composed of rafters, purlins, diagonals, collars and struts (Binan, 1990:94).

II.4.2 Western Roof Tradition

In western tradition timber pitched roofs are a) single pitched; b) double or purlin, c) triple or trussed. Single pitched roofs are monopitched and ridge. In the monopitch roof, the inclined rafters meet the wall plates at an angle and their load tends to make them slide off the plate. To decrease this tendency to slide and to provide a horizontal surface, the rafters in all pitched roofs are notched over the plates. To avoid weaking, the depth of the notch should not exceed one third of the rafter. Monopitch roofs may be constructed with the rafters laid at right angles to the pitch, that causes a shorter span and permits smaller sections to be used. A ridge roof consists of pairs of rafters pitched against each other at their heads with their feet bearing on opposite walls (Fig. II.15). The depth of the rafters are decreased when compared to a monopitch roof, of the same overall span. In the collar roof tie members are used but at a higher level than the feet of the rafters and they are called collars. It may be used for short spans not exceeding 4.9 m.

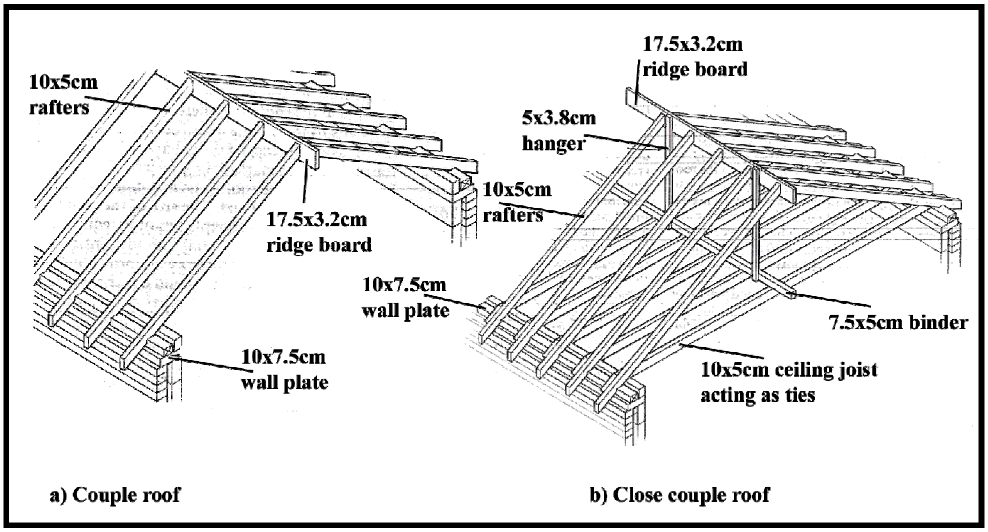


Figure II. 15 Ridge roofs are constructed in couple and close couple form (Foster, 1983:177).

Double or purlin roofs are used when the span of a roof is more than 6.1 m and needs in a couple type roof rafters much more than 10 cm in depth, it is more economical to introduce some support to the rafters along their length (Fig. II.16). This support may be in the form of a strut to the centre of every rafter placed, but it is more economical to introduce purlin, a longitudinal beam on which the rafters bear. By the introduction of the purlins, the total cube of timber in the roof rises with increase in span than if the rafters were increased in size.Depending on the weight and rafter length a 22.5 by 7.5 cm purlin will span from 2.5 m to 3.7 m. Purlins may be placed vertically or normal to the rafters. The former is preferable that the purlin supports on vertical walls or struts; the latter is sometimes appropriate when inclined struts are used.

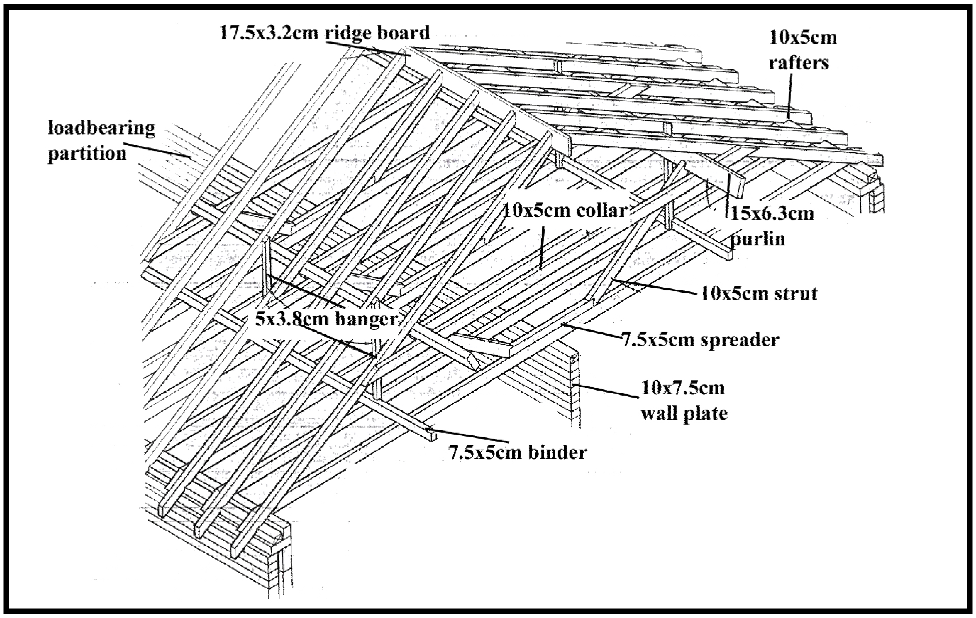


Figure II. 16 Purlin roof construction (Foster, 1983:180).

Triple or trussed roofs consists mainly of a pair of rafters triangulated to provide support for the purlins, preferably at the node points. For short spans two rafters lying in the same plane may be triangulated to carry purlins which are fixed under them, so that purlins are in the same relative position to the other rafters which they in fact support. For wider spans resulting in large loads on the truss members, the size of a normal rafter is usually too small to be used in the truss and seperate rafters are triangulated and carry the purlins on their backs. These rafters are placed below the level of normal rafters and do not directly support the roof covering.

Trussed roofs may be divided under topics such as: a) Nailed trusses, b) Bolted and connectored trusses, c) Glued trusses and d) Trussed rafters (Fig. II.17). Nailed trusses are the least efficient method, but it is a simple method. They are used where the lightweight roof coverings are used and the spans are short. Nailed trusses may be used up to 6.1 m. Bolted and connectored trusses are used to increase the efficiency of the bolted joints. They are embedded half in each adjacent members and transmit the loads from one member to another. Jointing by connectors permits to use thicker timbers. For greater loads split ring connectors are used, but these need accurately cut grooves. Bolted and connectored trusses are efficient up to 7.6 m. Glued trusses construct the most efficient type of joint. The members may be glued to each other by using lapped joints, in some cases lapped members may not provide sufficient gluing area, that double lapped joints or gussests may be used. These trusses may be used in the range of 4.5 m to 9 m.

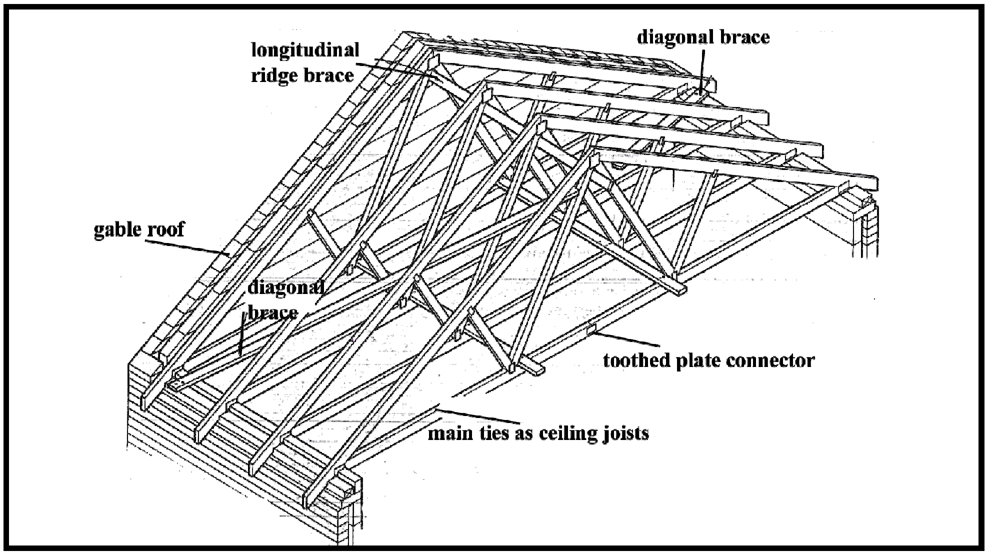


Figure II. 17 Trussed rafter roof construction (Foster, 1983:191).

Trussing every pair of rafters normally require purlins, thus dispensing with purlins. Trussing every pair of rafters permit low pitches, short bracing members, and placement of rafters at 60 cm centers rather than 40 cm centers. Elimination of purlins and ridge board, decreases timber content of whole roof structure and lower dead weight is acquired. The main difference between trussed rafter and a roof truss is that the former carries its own weight on itself, however a truss carries the the loads from a number of adjacent rafters via the purlins.

II.5 Contemporary Roof Structures

In order to understand contemporary roof structure systems basic structures are reviewed in this section. Roof structures may be considered in two dimensional or three dimensional forms. A two dimensional structure has a length and depth, moreover the loads are carried in two dimensions, in a vertical plane. On the other hand, a three dimensional structure has a length, depth and span, additionally forces are resolved within the structure. Two dimensional structures include beams, trusses, rigid frames. Beam is the simplest form of enclosing a span designed with linear members. A frame bears like a 3 dimensional structure, it is totally designed with the column reactions, however a truss can be designed seperately from the column, a timber truss can be designed over a reinforced concrete or steel column. Frame is a more rigid roof structure than truss. In two dimensional structures secondary members are introduced to bear the roof loads.

Three dimensional structures include space frames, folded plates, shells and domes. Space frame bears is three directions, differently from truss and frame; the 3 dimensional stability increases the span lengths for space frame. Folded plates are structures composed of repeating plates. In the terminology there is a dilemma for shell, that some searchers use shell is as the outer layer of the structure, like the skin of the egg, without structural ribs and mainly designed with plastics or reinforced concrete; on the other hand some searchers define shell as the organic form of the structure that can be composed of ribs, or trusses. In the research shell is used as the organic form of the strutcure, combined of ribs, trusses, the 3 dimensional form. Span lengths for the types both in timber and steel, simply summarizing drawings for the forms and photographs are used to discuss the subject.

II.5.1 Beams

Beam, is a squared off log or a large, oblong piece of timber, metal, or stone used especially as a horizontal support in construction. Beams are classified as straight beams, simply supported beams, shaped beams and cantilever beams. A timber beam may be produced from solid timber, glued web plates and laminated wood beams. Beams may be classified under four main topics: straight, simply supported, shaped and cantilever. a) Straight beams: Simply supported straight beams are widely used for purlins, lintels, flat roof joists and similar applications. Relatively light weight, combined with ease of fixing, make the use of timber popular with contractors and visually attractive. b) Simply supported beams: Simply supported beams are usually deflection governed in design. It follows that beams which are continuous over multiple supports are more efficient with consequent cost savings. Care will, however, be needed to allow for the changed distribution of loading on the support structure. c) Shaped beams: For roofs that are nominally flat a generous fall is strongly recommend. Timber beams can be tapered from one end or both ways from centre. This can be done with or without a camber which can considerably enhance the appearance of beams which might otherwise look quite deep mid span. d) Cantilever beams: It is easy to taper Glulam & LVL. Balconies, canopies and larger roofs will look better trimmed to a structurally efficient profile.

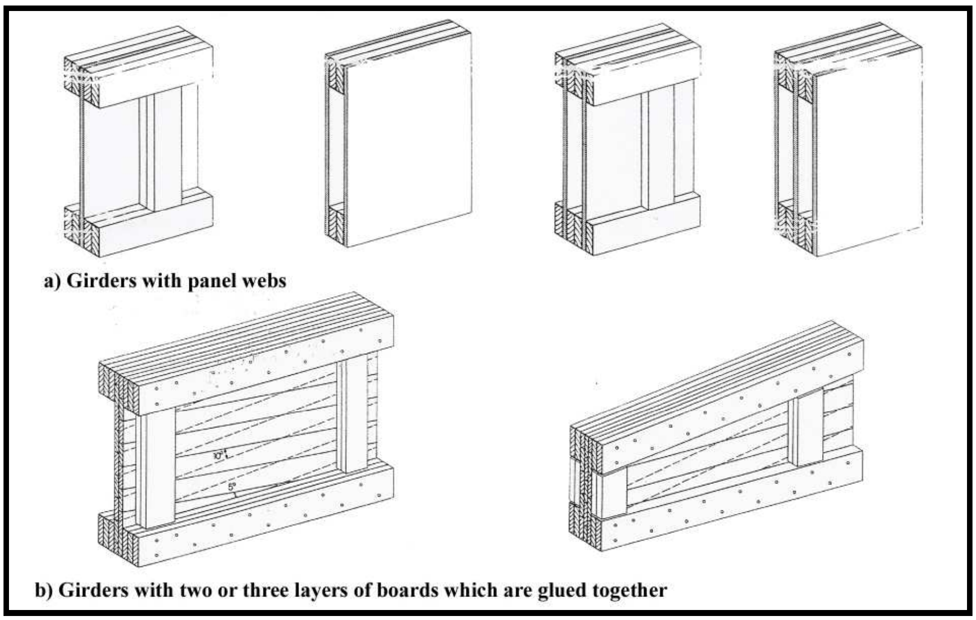


Figure II. 18 Girders are composed of 'I' or box sections with panels or with glued boards (Götz *et al*., 1989:57).

Laminated beams consist of two or more boards glued together along their wider sides. Boards should not exceed a thickness of 3 cm, may be increased to 4 cm if the wood is dried and carefully selected. Glulam sections can be produced in rectangular, 'I' beam and box sections (Fig. II.18). Laminated beams can be formed to any depth or length. The length is limited due to length of the shop, length of the gluing bed and transportation. The depth is limited only by the avaible width of of the assembly machines ranging between 2 m to 2.3 m. Lengths from 30 m to 35 m and depths of up to 2.20 m are commonly avaible. Laminated wood members are generally produced with rectangular cross sections. For bending the ratio of height to width of a section varies mostly between 3 and 8, but should not exceed 10 (Götz *et al.,*1989:21-22).

The design of a beam should be decided by bending, shear stresses, deflections; depending on the cross section of the beam, length of span and type of load. Length may be between 1 m and 7 m straight solid girders. For glued web plates length is between 7 m to 30 m, laminated wood beams length is between 7 m to 40 m (Götz *et al.,*1989:84). According to Schodek (1980:513) laminated beams may have a span range between 3 m and 24 m and have a depth between L/18-L/20, however Götz *et al.* (1989:21), stated that laminated beams span up to 30 m and 35 m. Schodek claimed that box beams may have a span between 5 m and 28 m, and a depth in between L/18-L/20.

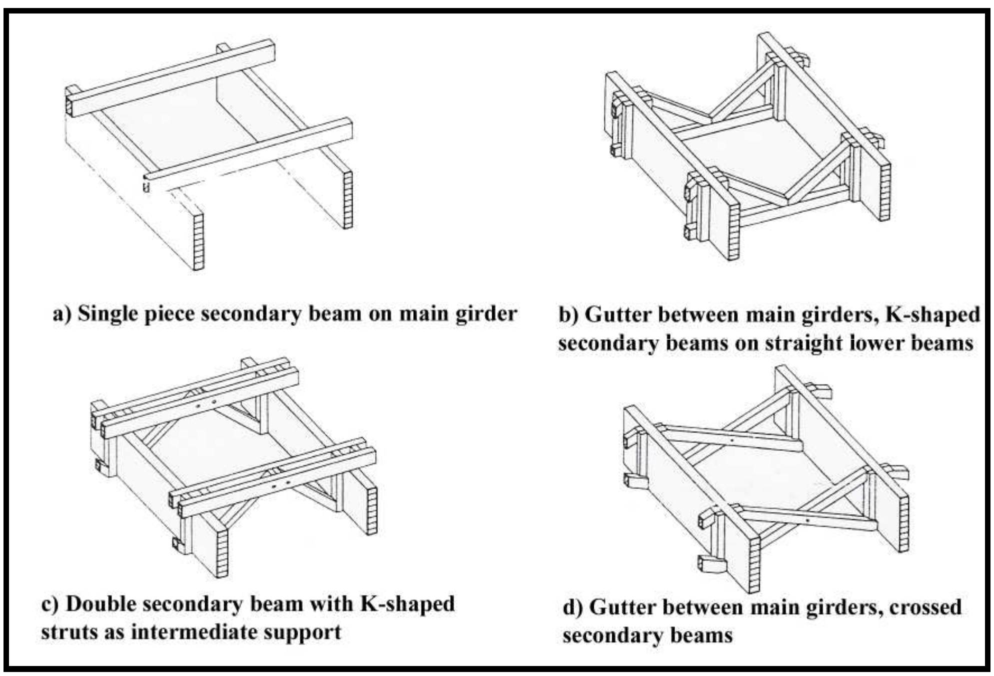


Figure II. 19 Secondary girders are placed on main girders in various alternatives with 90º (Götz *et al.,* 1989:109).

In a beam construction secondary members are placed related with the form of the structure that they can be designed with 90º or with various angles such as 60º or 45º

(Fig. II.19, Fig. II.20). The simplest design is to install secondary girders over main beams, mainly used for shorter spans. When the spans increase, diagonal bracing is necassary related with the beam depth, distance between main beams, weight of the decking material or insulation. In the angular arrangements except for 90º, secondary members can be placed over main girders or they can pass through the main beams.

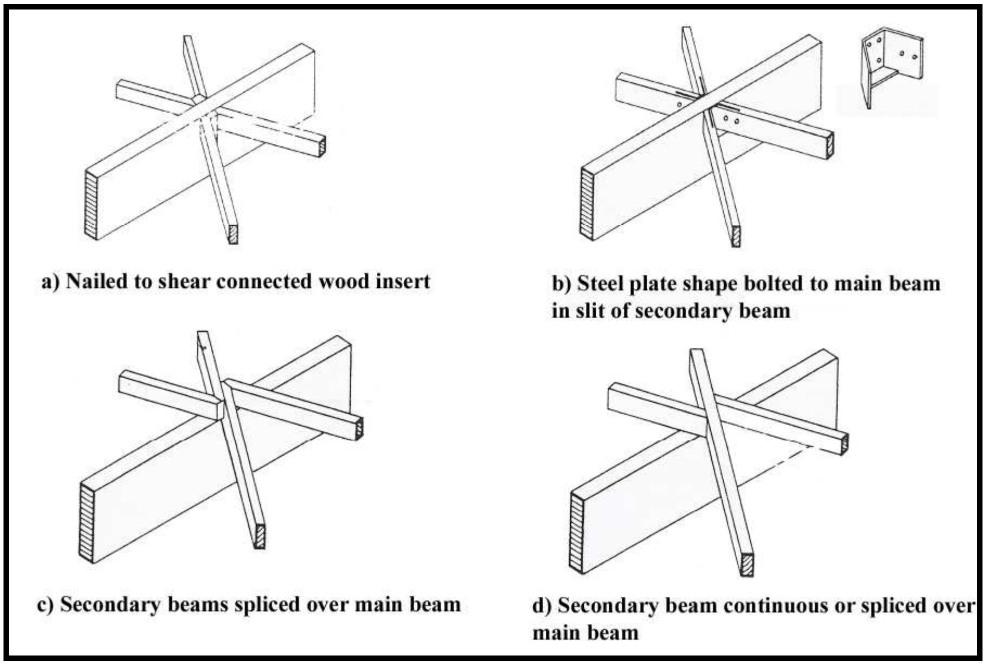


Figure II. 20 Secondary girders are placed on main girders in various alternatives with 60º (Götz *et al.,* 1989:107).

Glued laminated beams are commonly called as glulam, and may be constructed both horizontally or vertically. Horizontal lamination causes a more economical use of timber, particularly for larger section beams, depth of the beam may be formed of multiple laminates and is not limited by the width of any given plank. Each laminate has a thickness about 5 cm for straight members, decreased to 1.6 cm according to the radius of curve for shaped members. Beams much over 18 m length are not usually produced in glulam construction (Foster, 1983:343). Götz *et al.,* suggested that rectangular laminated wood cross sections can be produced for widths of up to 50 cm, depths of 3 m, and lengths over 30 m. Cross sections may have a width of 12 cm to 18 cm, and a depth of 2 m. Laminated wood is especially appropriate for arches spanning up to 100 m (1989:54). Callender (1982:2.61) claimed that sawn beams can be used between 9.15 m and 12.2 m, glued laminated beam can have a span between 18.3 m and 30.5 m. For spans up to 12 m or 15 m, a more economical distribution of material is obtained by the combined use of solid timber flanges and webs of plywood. For spans over 18 m and up to 30 m, laminated glued and nailed beams of 'I' section will produce a more economical structure than solid rectangular laminated beams. The webs and flanges are formed of boards approximately 25 mm thick fixed together with glue and carefully calculated nailing (Foster, 1983:343-344).

Figure II. 21 Roof structure of Highland Park Health and Fitness Center, Buffalo Grove. (Sentinel Structures, Inc., document on-line).

At the Fighland Park Health and Fitness Center the roof structure is composed of glued laminated straight beam (Fig. II.21). For the Viikki Teaching and Research Farm three new buildings are commissioned for the Helsinki University Teaching and Research Farm: two storage halls and a grain drying plant (Fig. II.22, Fig. II.23). The roof structure of the storage buildings consisted of plywood web beams combined with small scale glulam beams which was achieved without the use of tension bars.

Figure II. 22 Roof structure of Viikki Teaching and Research Farm[4]; Viikki, Finland (Timber Research and Development Association, document on-line).

Figure II. 23 Roof structure of Viikki Teaching and Research Farm; Viikki, Finland (Timber Research and Development Association, document on-line).

The rectangular shelter with the dimensions 13.5 m x 19.3 m was designed and installed by Western Wood Structures. Galvanized tube steel columns support the peaked and cambered roof beams (Fig. II.24).

4 Architects: Arkkitehtuuritoimisto Mauri Mäki-Marttunen; Structural Engineers: VM-Suunnittelu Oy; Completed: 1998, Area: 1.080m².

Figure II. 24 Roof structure of Emmaus Church; Cornelius, Oregon (Western Wood Structures, document on-line).

In steel construction steel beams are produced from hot rolled or cold rolled sections. When depths exceed the limit of rolled beams, generally spans more than 20.40 m the girder must be built up from plates and shapes.Welded girders are prefered to the riveted ones. Welded girders are generally composed of three plates that offering the opportunity for simple fabrication, efficient use of material and least weight. The availibility of high strength weldable steels caused development of hybrid girders. For example a high strength A572 steel may be used in a girder for the most highly stressed flanges, lower priced A36 steel can be used for lightly stressed flanges, web plate and detail material. Hybrid girders are efficient and economical for long spans and heavy loading (Merritt, 1982:8.9).

According to Eggen *et al.*, to increase the span while keeping the dimension of the beam at minimum, beam can be supported at one or several places along the beam, forming a trussed beam. The beam should withstand compression loads as well as bending. Eggen *et al.*, stated that simple steel beam has a span length between 3 m to 50 m (1995:102-104). Schodek claimed that wide flange shapes are generally used for steel beams for horizontal spanning elements. According to Schodek, steel beams have a span range between 5 m and 21 m (1980:519). Owens, defined the span lengths as between 4 m and 8 m by cold formed sections, up to 30 m span by rolled sections, between 6 m and 60 m span by castellated beams and between 5 m and 15 m span by compund sections (1992:403).

Merritt (1982:8.98) pointed out that in steel beam construction, generally one support is temporary bolted during erection, later it is fixed when the appropriate position is acquired. Beam is fixed to the columns by cranes, holding the beams from the ground should be prior to erection planned. General technique is to start erection from the beams in the middle and go further to the beams out in the plan view. During erection the number of bolts are kept minimum, just enough to keep the joints up tight and take care of the stresses caused by dead weight, wind and erection forces.

The researcher suggested that, for solid timber is used max. 7 m span is appropriate, because longer timbers can not be produced and the deflection effects the one piece solid beam. For glued web plates a span length between 7 m and 35 m is efficient, smaller spans can be enclosed by solid beams, longer spans can be passed by laminated beams. For laminated beams 6 m to 45 m is the proper span length. If the span length incresases, the system should be changed into a truss or an enhanced system. Spanning longer lengths with beam system is not efficient, due to the increase in the depth, weigth and deflection. For steel beams cold formed sections can be used in 4 m to 8 m, because cold formed sections are appropriate in the simple span. Rolled sections can be used at max. 35 m, longer spans can not be passed by rolled sections due to deflection, weigth and efficiency of the beam. Castellated beams can be used between 6 m to 60 m. Beam system is the most simple form of spanning a space. For longer spans enhanced systems should be used, secondary members must be introduced to have a rigidity on the structure. To span a long space curved or three dimensional forms should be used, depth should be incresead, that a flat or shaped beam may become very simple for long span. In a structure when the complexity of the form and amount of secondary members increase, longer spans can be enclosed.

II.5.2 Trusses

Trusses are girders which have been decreased to an assembly of simple struts connected by pin connections (Fig. II.25). Struts are stressed either in compression or in tension. Size of the compression members are dictated by buckling, while the size of tension members is controlled by tensile stresses at the weakest points which are generally at the connections.

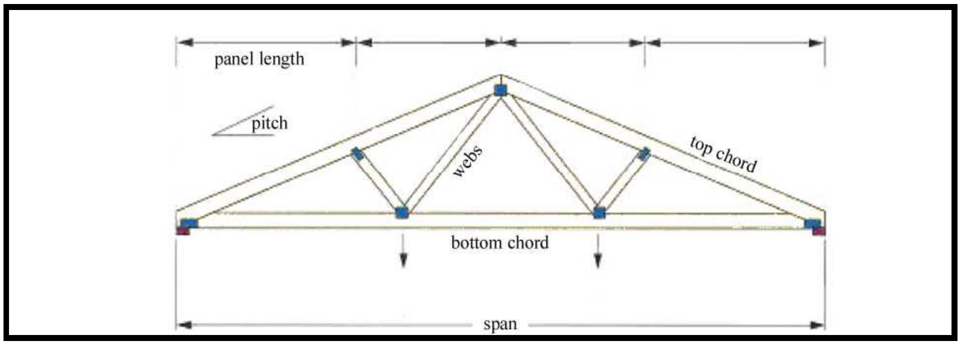


Figure II. 25 Typical truss members and definitions in truss design (MiTek Solutions, document on-line).

Roof trusses may be constructed in steel, timber, aluminium alloy, in spans up to 60 m, even more when required. In the case of very large spans, the pitch is designed in low height; to prevent the excessive internal volume, to reduce the area of roof to be covered and the weight of the structure (Foster, 1983:311).

Various searchers defined different truss types (Fig.II.26). A truss is compund of a bottom chord, top chord and webs. Names of the trusses may change in different cultures and geographical conditions. In a truss form, if any part of the roof is lighted, the parts that are glazed are more vertical. The classification of trusses are done according to the construction technique, joints and the form of the truss.

Götz *et al.*, (1989:55-58), classified the trusses according to their construction technique and connections as: triangular strut; trigonit-glued; nail plate greim and menig.

a) Triangular strut truss: It consists of a truss glued together (Fig. II.27). Upper and lower chords may be parallel or convergent moreover they may be fabricated in any length of finger joint splices. The maximum span for such systems is 10 m for parallel or trapezoidal trusses, and 30 m for triangular type trusses. Trusses shorter than 15 m may be produced in two sections, but those over 15 m must be made in two sections and have a nonglued field splice.

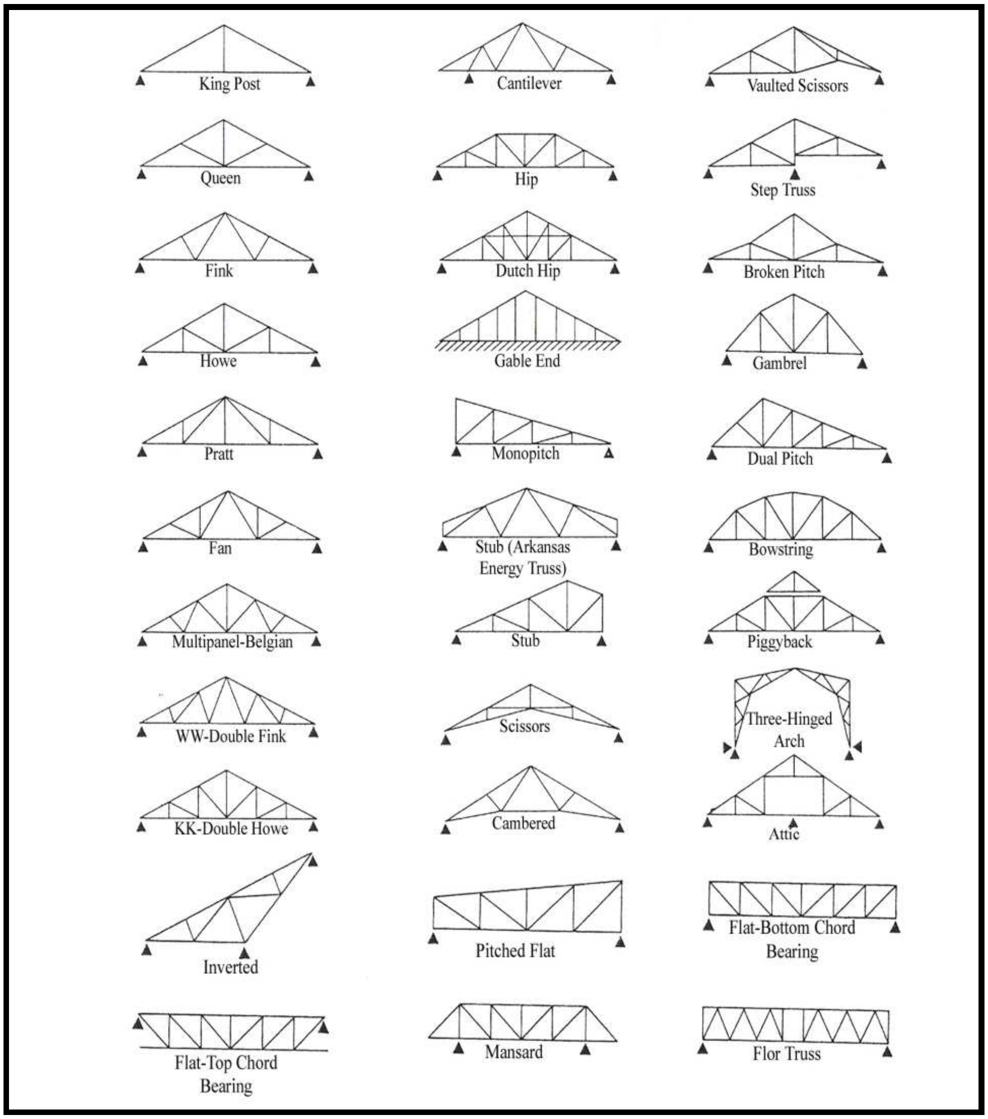


Figure II. 26 Typical truss types have various names in different cultures (Faherty, 1989: 6.7-6.8).

b) Trigonit-glued truss: Trigonit type truss is a glued truss in which the diagonals are jointed together into a web by means of finger joints and then nailed to the flanges (Fig. II.27). The trusses glued in one machine should not be over a length of 15 m.

c) Nail plate truss: Generally the distance between the trusses is about 1.25 m, depending on the load it may vary from 0.625 m to 2.5 m (Fig. II.28). The span of nail plate trusses should not exceed 20 m.

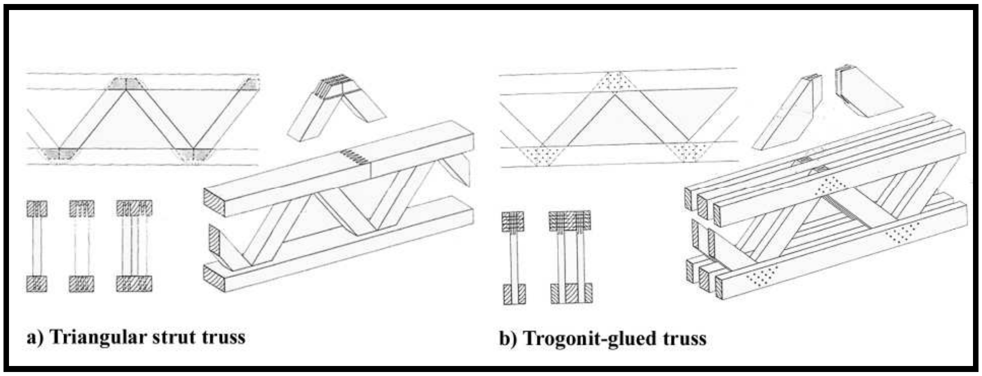


Figure II. 27 Triangular struts are glued by finger joints, in trigonit girder truss diagonals are joined by finger joints, double or triple flanges are nailed together with each nail loaded in double shear (Götz *et al.,* 1989:55-56).

d) Greim truss: It is designed in two kinds: saddle and flat (Fig. II.28). d.1) Saddle roof truss spanning 10-20 m and having a ridge height of 1/10 of the span. d.2) Flat roof truss spanning between 7 m and 35 m with a depth of about 0.08 of the span.
e) Menig truss: These type of trusses are appropriate for heavier loads, optimum spans for this type of truss is between 16 m and 25 m (Fig. II.28). On the other hand, spans up to 45 m have been constructued with menig nail plates, with the depth of the truss at the centerline equal to 1/10 of the span.

Vandenberg (1974:206) claimed that trussed rafters have a span up to 10 m, fink truss up to 10 m, double fink truss between 5 m and 14 m. Schodek, defined the flat truss spans between 12 m and 33 m and have a depth of between L/10 and L/15, moreover shaped trusses may have a span of between 18 m and 46 m and a depth of between L/7 and L/10 (1980:513). Girders up to 45 m and trusses to about 60 m or 75 m may be constructed in timber, the upper chords of girders may be curved, parallel or inclined, and are usually laminated in the larger spans (Foster, 1983:347). The size of compression members is decided by buckling, however the size of tension members is controlled by tensile stresses at the weakest points, which are generally at the connections. Due to this reason, to increase the cross sectional area of a member in order to get a convenient connecting surface or cross section at connections. The top chords of a truss are designed against buckling by lateral bracing (Götz *et al.*, 1989:90).

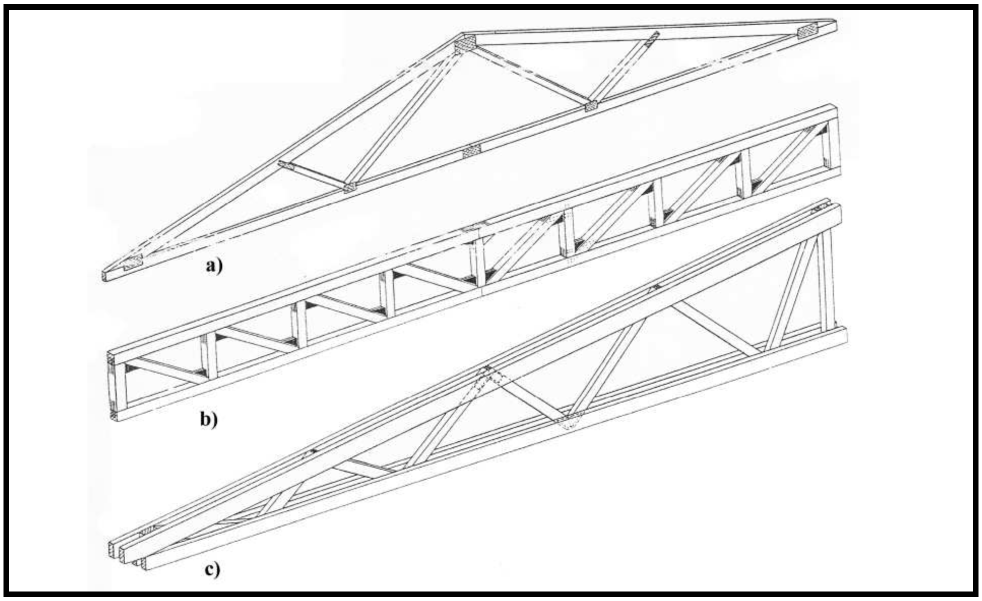


Figure II. 28 a) Nail plate truss: Node plates consist of galvanized steel plates, they are pressed from both sides into wood. b) Greim truss: Stell plates are placed in sawn mortises and nailed from the outside without predrelling. c) Menig truss: Node plates are placed between struts and are not visible in the finished truss (Götz *et al.*, 1989:58).

Hornbostel and Hornung (1982:183-184) stated that laminated wood trusses generally span in a range between 12.2 m and 45.7 m, additionally some manufacturers are equipped for spans larger than these spans. Solid wood trusses are used for spans between 6.1 m and 10.7 m. Shaeffer, considered that timber trusses have a span range between 7 m and 30 m; steel trusses have a span length between 20 m and 60 m (1980:8). Desch (1996:195) denoted that trusses may span up to 30 m. Trussed rafters are designed to carry simply the direct load imposed upon them. According to Mindham, trussed rafters use approxiamately 30% less timber than other truss forms, moreover factory production keeps the labour cost of trussed rafter very low when compared to that necessary bolt and connected jointed truss (1994:12).

Figure II. 29 St Mathews Lutheran Church; Stony Alberta (The Engineered Wood Association, document on-line).

Merritt (1982:7.58) suggested that bowstring truss is the far most popular. Spans of 30.4 m to 60.8 m can be spanned with bowstring truss with glued laminated timber at top and bottom chords and solid sawn timber at webs. Top chord, bottom chord and the heel connections are the major stress carrying components. Since the top chord is nearly the shape of an ideal arch, stresses in chords are almost uniform, throughout the bowstring truss, web stresses are low under uniformly distributed loads. In the flat trusses chord stresses are not uniformly distributed along their length, web stresses are high. Triangular trusses like scissors are used for short spans, they usually have solid sawn members for both chords and webs. Callender (1982:2.61) claimed that bowstring trusses can be used between 12.2 m to 45.7 m; fink, belgian and pratt trusses have a span range between 12.2 m to 27.45 m. In spatial trusses designing structures with large spans using common profiles with relatively short lengths, pozitively effect the erection time, production and transportation. They also cause the combination of members from solid wood, glued laminated timber or steel. These systems are suitable for designing complex geometrical shapes (Fig. II.29).

In steel construction due to the greater depth of trusses, they provide a high stiffness against deflection when compared corresponding rolled beam or plate girder than otherwise would be required (Merritt, 1982:8.9) Owens, classified steel roof trusses as pratt, howe, fink, mansard, pratt, warren, and saw-tooth (1992:529). With steel design for pratt, howe and fink trusses estimated span range is between 6 m and 12 m; with mansard truss range is between 15 m and 30 m; with pratt and warren truss range is between 6 m and 50 m (1992:532). Steel roof trusses may be produced from tubes, angles or rolled sections between a span of 10 m and 100 m. Open web joists may be produced from angles, tubes as chords and rolled sections (1992:403). According to Vandenberg (1974:206-207) double fink truss have a span range between 8 m and 15 m, howe truss up to 10 m, double howe between 8 m and 15 m, fan truss between 8 m and 15 m, bowstring between 20 m and 40 m.

Steel girders may be bolted or welded complete in the fabricating shop, or the ones which are too large for transport are made in two halves with the necassary connecting plates. Steel light lattice beams formed from angles and flats are used for lightly loaded roof structures up to spans of 12 m. Such beams frequently incorporate high tensile steel bottom chords (Foster, 1983:348). Foster, stated that universal steel beams are suitable for spans up to 10.5 m, although this can be extended by the use of castellated beams. The economic depth of the trussed girders is from 1/6 to 1/10 of the span (1983:311). Schodeck, claimed that steel howe truss have a span length between 10 m and 32 m, bowstring truss between 17 m and 37 m and special trusses between 23 m and over 55 m (1980:523). According to Eggen *et al*. (1995:120), steel consumption in truss construction with spans of 50 m or high is enormous, the consumption of steel is so relatively high that it is reasonable to study other structural forms than trusses.

Taymaz (1988:283), suggested that in steel trusses due to movement related with heat, one of the supports should be done hinge support. Steel trusses are placed between 2 m and 6 m and trusses are named as full body truss (*dolu gövdeli makas*) and frame roof (*kafes çatı*). Full body trusses can be formed either as a single extruded (*haddeden çekilmiş*) unit, or as combined beams (*birleşik kiriş*). The roof spaces designed with full body trusses can be used. Full body roof trusses are connected by rivetting or welding. If the roof truss has a full body, height of the truss decreases through the supports. Frame roof trusses are formed of steel bars placed in triangular shapes. In the frame roof trusses the elements that face to compression are shorter and the elements that face to tension are longer. Roof trusses are generally designed symetrically to the middle axis.Purlins are placed over the top header connection points. Frame roof trusses are designed according to the type of the building, truss length and the covering material.

In steel roof trusses purlins are generally made of steel, and rarely made of timber or prefabricated prestressed concrete (Fig. II.30). Steel purlins are in ‘U’, ‘I’, ‘L’ or ‘Z’ sections. The span between the purlins are 1.5 m and 4 m. Steel purlins are connected by bolts, rivetting or welding (Fig. II.31). In steel roof trusses, rafters are placed perpendicular to purlins and between 40 cm and 90 cm span. Timber rafters are placed over steel purlins and connected with bolts or strip. Lath is the element that is connected vertical to the rafters. Timber laths are in 5 cm by 5 cm sections, and placed over timber rafters. Steel laths are in ‘I’, ‘L’ or ‘T’ sections and placed over steel rafters moreover steel laths are connected by bolts or welding.

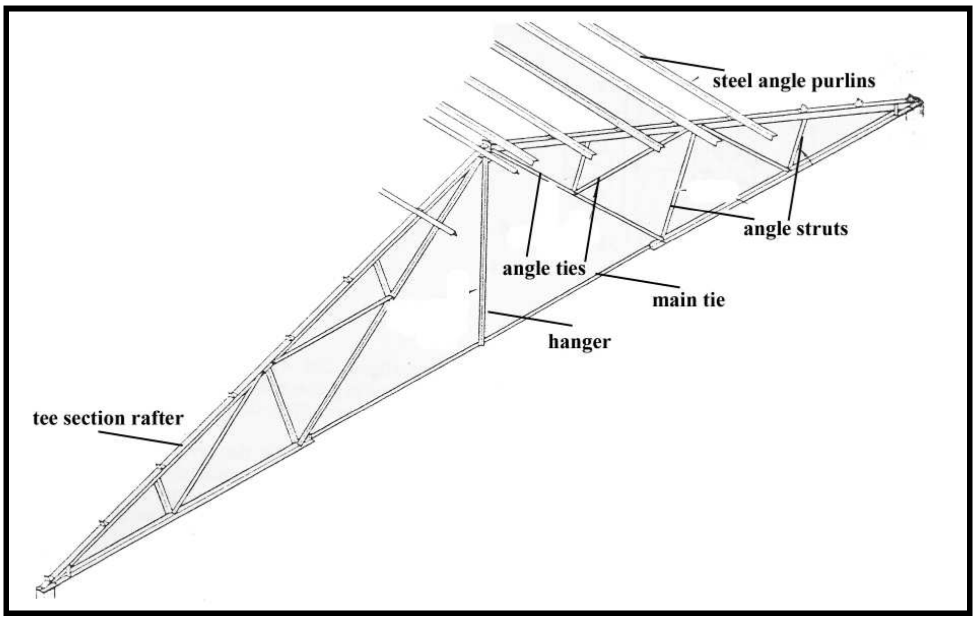


Figure II. 30 Steel trusses are composed of rafters, purlins, hangers, struts and ties (Foster, 1983:346).

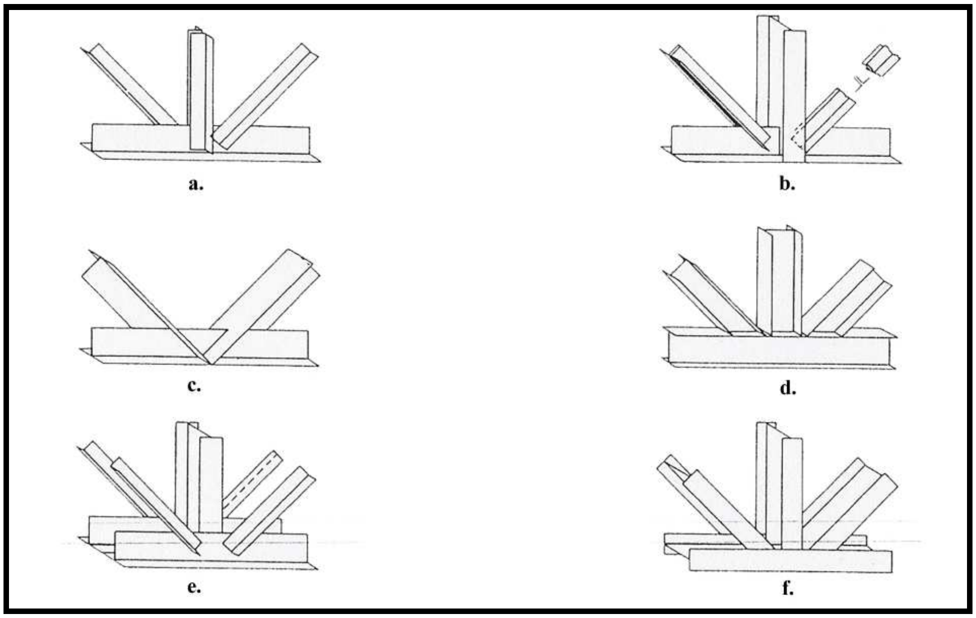


Figure II. 31 Connections for simple welded trusses (Ellison, 1987:211).

According to the researcher trusses made of solid timber may be designed up to 11 m . For solid timber span length is limited both in beam and truss design, because solid pieces deflect, deformed due to torque, moreover strength and size of solid member is limited. Trusses composed of girders or laminated sections have a span up to 45 m. With laminated sections truss span increase, deeper sections and longer members can be achieved. When steel is used for trusses maximum span is possible up to 55 m, larger spans are not feasible with steel. In truss design crucial point is that, when the span increases, the amount of secondary members rise the dead weight of the total truss which ends up in higher depths. Higher depth and more dead weigth is not feasible for a structure, so a different system should be designed for longer spans.

II.5.3 Arches

One of the advantages of glued laminated wood is that the laminations can be bent to various curvatures, one common use of curved glulam members is in the fabrication of arches, which may have several configurations. According to Schodek, laminated arches have a span between 12 m and 43 m and have a depth of L/4 and L/6 (1980:

513). For spans up to 24 m and with the depth to length ratio 1/8 to 1/6, two hinged arch configurations are more feasible, because it is more economical of materials wherever the curved arch members can be shipped from the factory to the site.

Figure II. 32 Arch structure of Cantina Fossoli; Colognola, Italy; Architect: Antonio Masconi (Holzbau Spa, document on-line).

As a rule, arches have a constant radius of curvature, because it is easier to bend the component boards in a circular fashion. To decrease bending, the thickness of the laminations should not be over 1/300 of the curvature radius or 3.3 cm (Karlsen, 1989:201). Götz *et al.* (1989:54), claimed that laminated arches have been executed spanning up to 100 m. Arch structure of Cantina Fossoli has a span length of 22.1 m by 50.9 m (Fig. II.32).

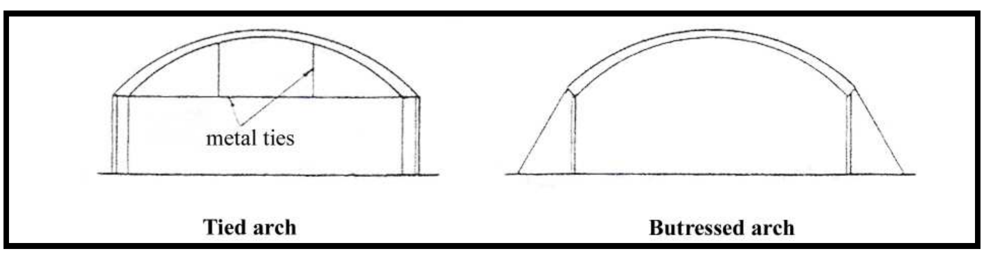


Figure II. 33 Two hinged arches are constructed in two ways (Hornbostel and Hornung, 1982:182).

In tied arches, the strings that take up horizontal thrust are composed of steel joints or rods (Karlsen, 1989:201). Shaeffer (1980:8) summarized that glued laminated arches have a span range between 20 m and 40 m; however trussed arches have a span length between 30 m and 70 m. Glued lamianted arches have an appearence of wood finish, but it is not easy to transport large pieces.

Hinged arches could be designed into two different types: two hinged and three hinged. a) Two hinged arches: Two hinged arches are statically indeterminate and generate outward reactions mostly on compression forces in the direction of the arch. Unsymmetrical or horizontal loads reason to bending, so that the amount of horizontal trust changes on the rise of the arch and its stiffness at the crown. The bending moment in the centre and the stiffness of splices determine the design. If the bearings are subject to lateral movement, the two hinged arch deforms and causes additional bending stresses. According to Hornbostel and Hornung (1982:182) two hinged arches are classified in two types: tied and butressed (Fig. II.33). Hornbostel and Hornung (1982:17) stated that two hinged arches are adaptable for wide spans up to 76.2 m and practical for even larger spans. However the most common and appropriate span range is between 9.15 m to 30.50 m. b) Three hinged arches: Three hinged arches are statically determinate. They withstand exterior loads mostly on through compression in the arch. The bearing reactions are inclined, the same as for three hinged frames or beams and the size of horizontal forces changes on the size of the arch. The design is determined by the compression and bending stresses from unsymmetrical or horizontal forces. Buckling out of the plane of the arch is also critical (Götz *et al.*, 1989:146). Hornbostel and Hornung classified the three hinged arches as tudor, gothic and parabolic (Fig. II.34). Three hinged arches can be jointed at the crown by side plates which resist the shearing force produced by the live load and ensure the lateral rigidity of the crown joint. Callender, (1982:2.61) claimed that two and three hinged arches have a span length between 12.2 m to 30.5 m.

Callender (1982:2.74) denoted that deciding the arch design depends upon soil conditions and other building requirements. Reactions to the ground should be faced by base bearings (Fig. II.35). In the buttressed arch type, horizontal and vertical reactions are taken through concrete abutments. In the tied arch horizontal reactions are taken by steel rods located at the ceiling height. Tied arches are usually placed on masonry walls or columns.

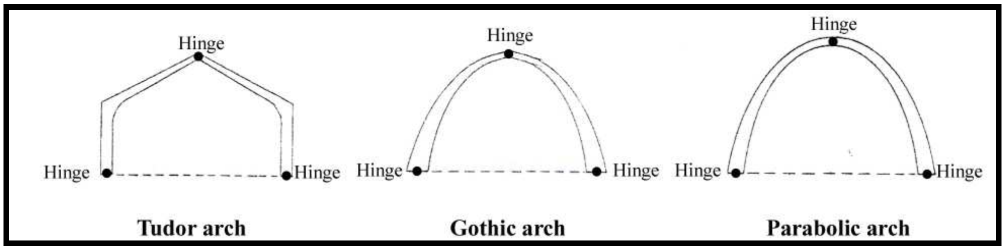


Figure II. 34 Three hinged arches have three general forms (Hornbostel and Hornung, 1982:183).

Steel arches may be three hinged, two hinged, one hinged, fixed or hingeless. Eggen *et al.*, suggested that steel arches have a span length between 25 m and 70 m, trussed arches have a span length between 40 m and 120 m (1995:105-107). The arched structure with three pin connections has the same capacity for lateral stability that a frame has. The arch can consist of steel sections or be made up of individual steel staves. Schodek, claimed that steel arches have a span length between 18 m and over 55 m (1980:523). According to Ellison (1987:221), steel arches may be used for spans exceeding 91.45 m.

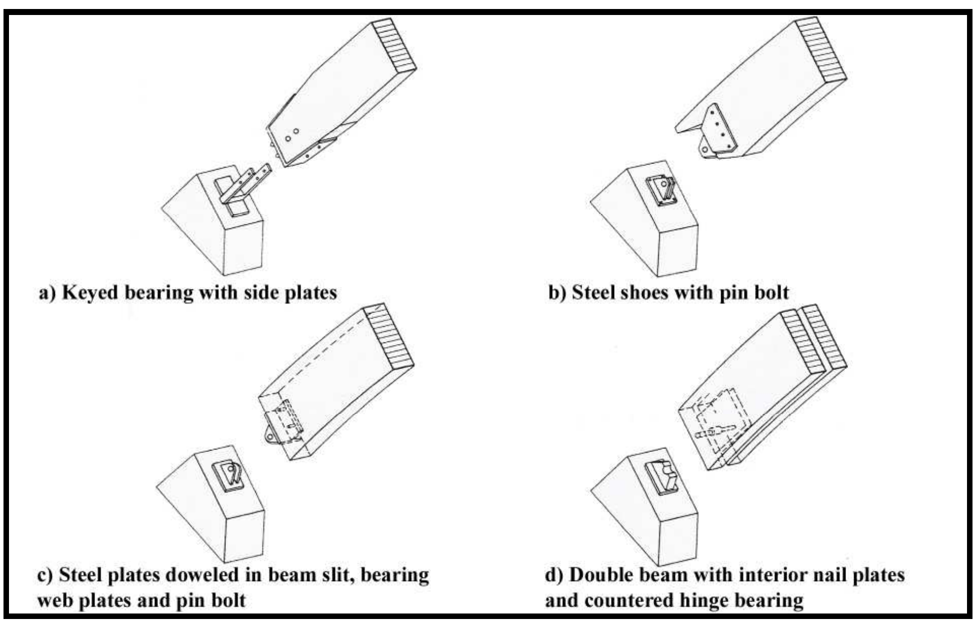


Figure II. 35 Base bearings of arches, the system can be also used in frames and beams (Götz *et al.*, 1989:147).

Merritt (1982:8.13) suggested that in steel arches hinge pins offers two advantages in long span frames, firstly they simplify design calculations, secondly they simplify erection. Fittings can be done and strong connections required to develop the needed strength at the ends of the arch can be made in the shop, instead of high above ground in the field. When the heavy members are raised in the field about their final position; holes in the pin plates line up exactly, the crown pin is adjusted and secured against falling out by the attachment of keeper plates. The arch is then ready to carry its loading.

The stresses in steel three hinged arches are not affected significantly by minor movements.Horizontal movements can be prevented by tying two abutments together with tie rods. The stresses in two hinged arches are unaffected by minor vertical movements of the abutments since the span remains constant. The stresses in fixed arches are affected by vertical or horizontal movements or rotation of the abutments. Generally three hinged arch is preferred. Various parts of an arch can be riveted or welded together. Ellison (1987:223), stated that since both chords or flanges of arches are in compression, they must be braced laterally to prevent buckling in that direction. This is usually acquired by using trussed purlins.

The researcher claimed that arch form is a suitable system for laminated sections, spans up to 40 m can be designed with arches. When the hinge is introduced to arch form, span distance increases, more than 70 m can be passed by hinged arches. Steel arches can be used up to 70 m, longer spans can be achieved by trussed arches, that the truss form increase span distance due to secondary members, and three dimensional stability of the truss form. Disadvantage of both timber and steel trussed arches is that they are not suitable for concentrated loads. Arch forms designed of glulam is a suitable way of medium and long span, because one factory finished form is erected in site, which has advantages in work man hours and workmanship.

II.5.4 Frames

In a truss design the connections are accepted as hinged joint theoretically however it is constructed as fixed joint because of practical reasons, which increases the stress about 15% on the truss. Introducing a hinge connection needs extra labor and detailing. Over large spans, deep trusses may result in excessive volume within the roof space of the building, furthermore an increase in the span cause the need of extra material to provide strength where the own dead weight increase. Frames can be designed as: 1) Rigid frames with fixed connections; 2) Hinge frames with pin connections.

The use of rigid frame construction overcome the problems of depth and extra material to a very large extent. The property of the rigid frame is continuity of the structure due to the stiff, restrained joints between the parts, and because of the nature of the stress distribution within such frames, less material is required at the center of the spanning elements than in a comparable simply supported beam (Foster, 1983:312-313). Vandenberg (1974:208), suggested that rigid frames transmit bending moments as well as forces to foundations. This is economical in frame itself, however it results worst loading conditions to foundations. Site conditions are not always appropriate for rigid frames. In the two hinged frames no moments are transmitted to foundations; therefore better loading conditions are acquired but maximum moment in frame is increased. In the three hinged frames maximum bending moment is increased in frame, they are more efficient in resisting differential deflection of foundations.

According to Callender, advantages of frames are: decrease in height in the building, increase in clear headroom, efficiency in erection and maintanence; on the other hand disadvantages are greater weight of structural material required and erectors may be unfamiliar in detailing and erection (1982:2.104). When a frame is built, it is not subject to only a single condition of loading. The self weight of the structure is constant, but the shape is distorted by axial strains, may be deformed by foundation movements. Imposed loads and live loads vary with the use of the structure. These variations in loading can be properly anticipated by choosing a shape which corresponds best to the loadings expected for most of the time and which responds safely to the loadings at any time. This form will require the least material and will usually not cause tensile stresses. Curved or polygonal forms that have large height to span ratios are often wasteful of volume and not cost efficient to construct (Callender, 1982:2.102).

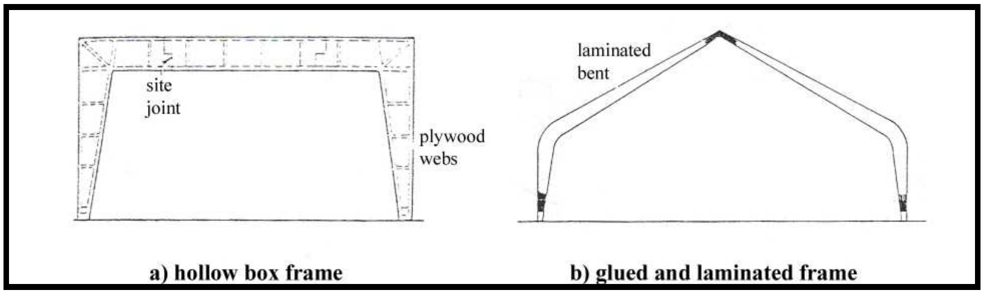


Figure II. 36 Frame structures are arranged two different ways (Foster, 1983:350).

Foster, classified timber frames as hollow and glued laminated (Fig. II.36). The forms of the spanning member may be horizontal, pitched or arched; connections of the vertical members may be restrained or hinged. A hinge may be designed in the middle of the spanning member and the structure itself may be solid or latticed (1983:315). Salvadori, claimed that frame structure is stronger than the post lintel structure against horizontal and vertical loads, moreover it behaves monolithically (1986:178).

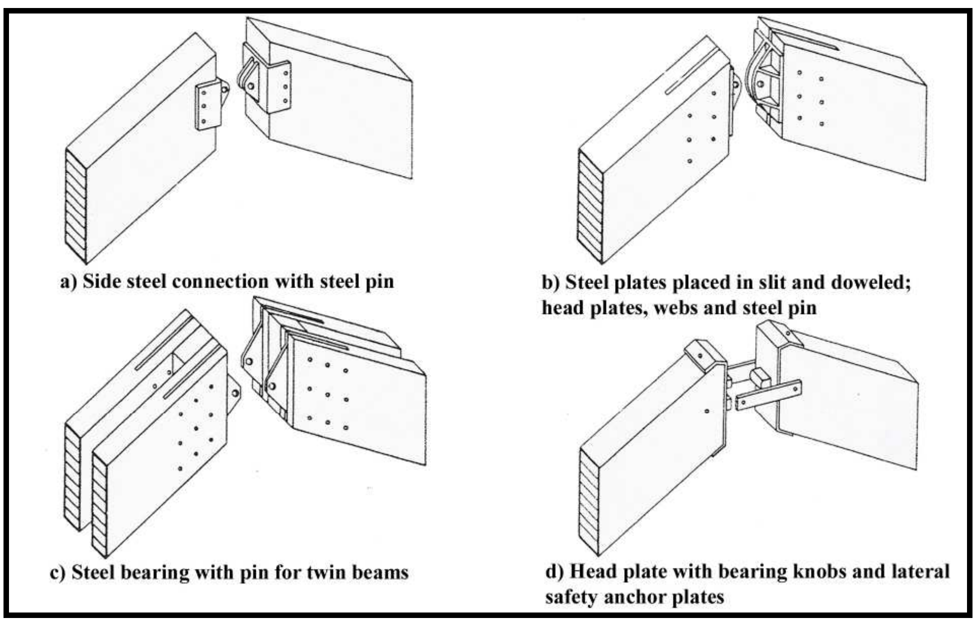


Figure II. 37 Crown hinge details for timber frames are composed of various arrangements (Götz *et al.*, 1989:119).

Specially designed hinged beams composed of statically determinate three hinged beams have a common crown joint (Fig. II.37), but arranged in plan and elevation in different ways. Despite a spacial set up, the load is supported through a planar arrangement in which the beam faces another one in the opposite direction (Götz *et al*., 1989:128). Various horizontal forces caused in each plane can be resisted by individual footings or can be balanced by ties connecting individual footings. In two and three hinged frames footing details vary, that the leg is composed of a compression and tension part (Fig. II.38). In the three hinged frames, hinge at the leg is composed of various jointings such as finger jointing, shear connectors and bolts (Fig. II.39). When crowns of three hinged frames do not meet in one point, the frame must be designed to maintain the stability of the structure (Götz *et al.,* 1989:128).

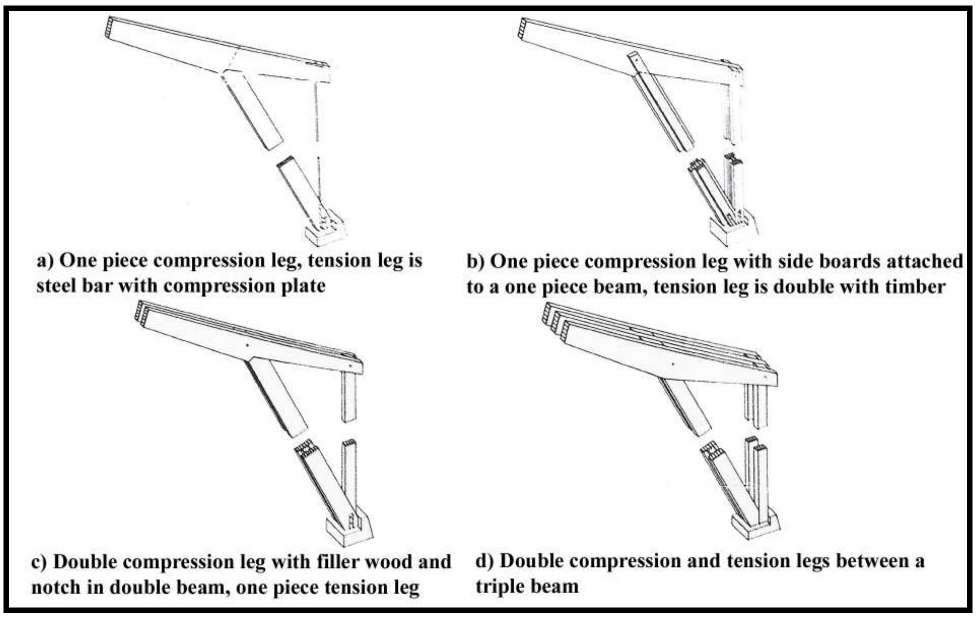


Figure II. 38 Frame leg details for two hinged frames (Götz *et al*., 1989:131).

Bent glued laminated frames are generally three hinged type, which is convenient in manifacture, transportation, and erection. The curvature of the rigid knee joints is effected by the laminations in a circular manner, during the manufacture period. The radius of the curvature is generally between 2-4 m. The thickness of the laminations should be between 1.6-2.5 cm. Bent glued laminated frames are more labour consuming and less economical of wood and glue than arches (Karlsen, 1989, 205).

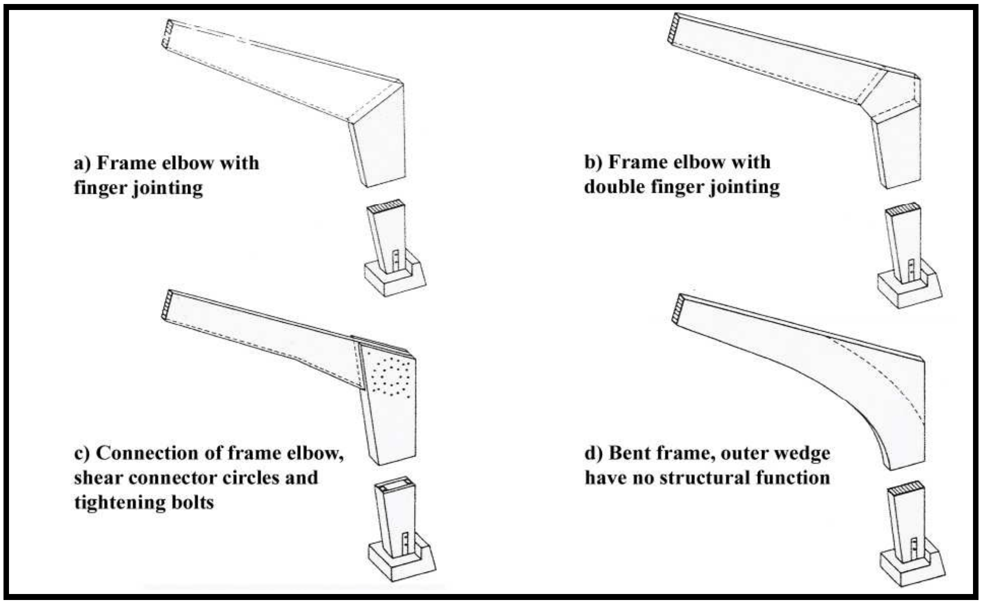


Figure II. 39 Frame leg details for three hinged frames (Götz *et al.,* 1989:133).

Figure II. 40 St Andrews Anglican Church[5]; Barry St, Gracemere, Central Queensland, Australia (Australian Government Forest and Wood Products Research and Development Corporation, document on-line).

[5] Owner: Anglican Church of Australia; Architect: Innovarchi Pty Ltd; Engineers: Timber structure; Date of Construction: 1995.

Figure II. 41 St Andrews Anglican Church (Australian Government Forest and Wood Products Research and Development Corporation, document on-line).

Figure II. 42 Roof structure of Caldonazzo; Caldonazzo, Italy (Holzbau Spa, document on-line).

Frames have a marked effect on the distribution of bending moments in the span. When the both sides of the frame are loaded equally, the bending moments are equal in the rigid knee joints, but may be considerable in magnitude that limits the span length between 18 m and 30 m. Timber lattice frames are built up with bolted or glued joints on the same lines as trusses and girders. The ratio of the radius of curvature to the thickness of the laminates must not be less than 100 and the ratio of 150 is normal in practice. The cost of fabrication rises as the laminates get thinner, therefore sharp curves should be avoided. Boxed plywood frames are used for spans up to about 18 m, glulam frames up to about 24 m and lattice and built up 'I' section frames up to about 45 m (Fig. II.40, Fig. II.41, Fig.42). Arch ribs are constructed in glulam up to spans of more than 60 m (Foster, 1983:348). According to Vandenberg (1974:208) three hinged frames may be designed from timber box section or laminated sections which span up to 50 m.

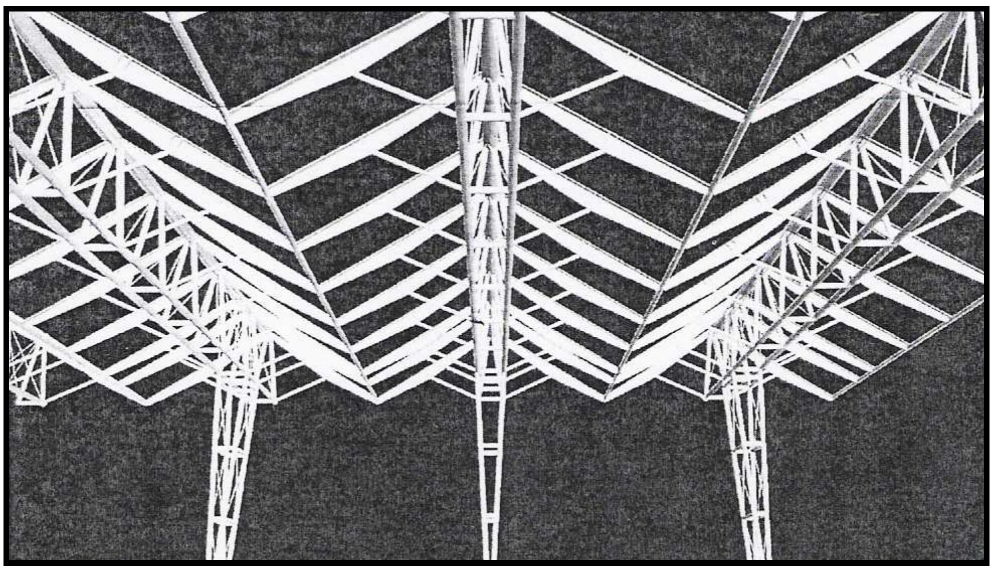


Figure II. 43 Steel framing 3D modeling of the project[6] for oil museum and boat terminal; Stavanger (Eggen *et al.,* 1995:103).

Steel rigid frames constructed in steelwork may be of welded solid web construction or of lattice construction (Fig. II.43). The lattice form is generally used in the construction of large span frames (Foster, 1983:349). Allen, mentioned that steel rigid frames are easily manufactured by welding together steel wide flange sections or plate girders. They may be set up in a row to roof a rectangular space (1999:395). Eggen *et al.* (1995:105), stated that steel frames with shaped sections have a span length between 5 m to 40 m. If the frame is combined of trusses the appropriate span is

[6] 3D modelling: Stein Erik Sandaker; Architect: Edwardsen, Hoglund, Witzoe; Engineer: B.N. Sandaker; Date: 1992.

between 8 m to 55 m. Ellison (1987:219), claimed that steel rigid frames are not efficient for spans of 12.2 m or less. They are efficient for spans up to 30.5 m and are used spans exceeding 71 m. For longer spans steel arches are used. Vandenberg, denoted that three hinged frames may have a span up to 80 m, and can be designed from solid web, castellated, lattice and hollow sections (1974:208).

Merritt (1982:8.149) suggested that, for steel design in small spans, it may be possible to place reinforcing bars in the floor slab or floor beams, by simply connecting them to the column bases. On larger spans, it is advisable to use tie rod, the latter affording the oppurtunity to prestress ties and thus compensate for elastic elongation of the rods when stressed. Prestressing rod during erection to 50% of its value is recommended. When using frame construction lateral bracing is important which is achieved by the joint action between the horizontal and vertical members. A structural frame can have a maximum of three pin connections, that are not moment connections. The frame can also be made of trusses. Frames have the capacity to withstand lateral forces acting on their own plane. Generally frames are fabricated in three pieces and site jointed at the points of least stress with splice plates and bolts. Effective lateral bracing is needed between adjacent frames, at the ridge and in the diagonal plane through plane the knee where the stiffeners are located. Such bracing is composed of light trussed members. Each frame and all its parts must be supported laterally to remain its true position.

According to the researcher timber, frames are suitable for medium span and long span, frame structure bears as a support and beam connected together. When the frame has an curved or arch form, span length increases. Frame span is up to 18 m for boxed beams, when laminated beams are used span distance increases to 24 m, ‘I’ section is used up to 45 m. Whe the arched ribs are used spans up to 60 m can be enclosed. In timber frames arch form increase the span distance; in steel frames truss form rise the span length. With shaped steel frames distances up to 40 m can be spanned, but when trussed frames are used 55 m can be enclosed. Castellated beams, plate girders and rigid frames in steel construction have the same characteristic that they must be braced laterally by purlins, decking or diagonal bracing to prevent from buckling. Frame forms should be designed by the supports, the structure behaves like a whole. Frame structures are proportional with the column height and span distance, when the span distance increases, height of the columns should be more; which is not generally preferred.

II.5.5 Space Frames

Although all structures apart from single layer flat grids are in fact space frames, the term space frame is usually used as a hollow section or three dimensional lattice beam (Foster, 1983:330). A space frame is composed of nodes that the members intersect, and lineer elements. Toydemir, classified the space frames as: a) Flat; b) Arch; c) Dome. Structurally space frames are divided into two basic types: four faced and diagonal triangular space frames (Fig. II.44).

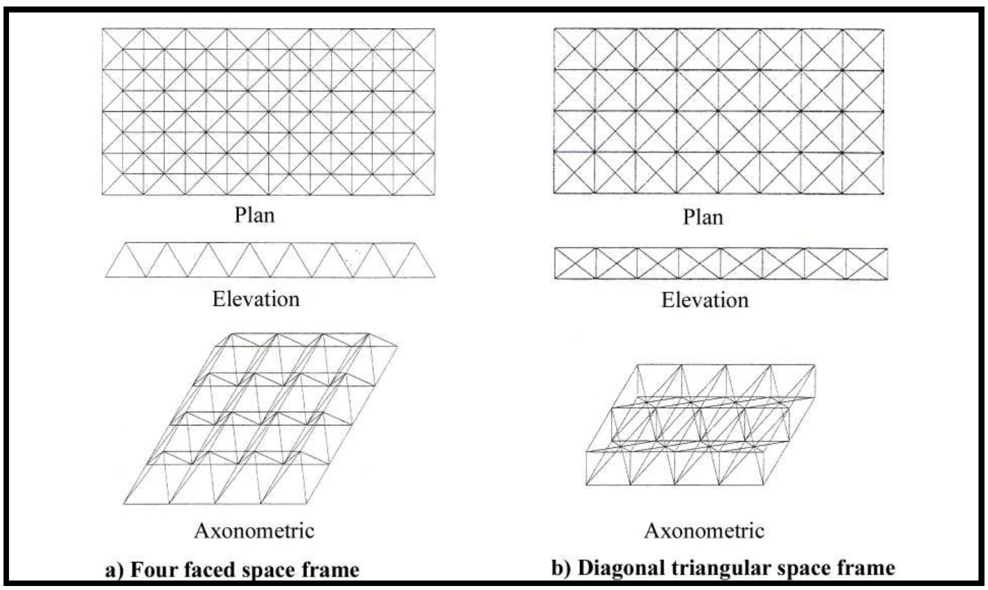


Figure II. 44 Types of space frames: a) four faced, b) diagonal triangular (Toydemir *et al.*, 2004:138-139).

Callender classified the space frames as rectangular, diagonal, triangular and mexagonal. According to Callender (1982:2.105) the analysis and fabrication costs would be less for the rectangular or diagonal arrangements than the other types. Diagonal system is more rigid than other systems, therefore it is usually preferred in the space frame design. The advantages of space frame are: reduction in required depth approximately 50% in height and decrease in the amount of structural material up to 25%, simplification of fabrication due to the repetition of members and better resistance to eartquake and other horizontal forces.

The main problem in constructing timber to space frame is the connection between timber members in several planes. The solution is to use three lattice girders and connect them by metal lugs at the nodes or using steel lugs bolted to the ends of each member by means of which they are fixed to pressed or cast metal multi-directional connectors.

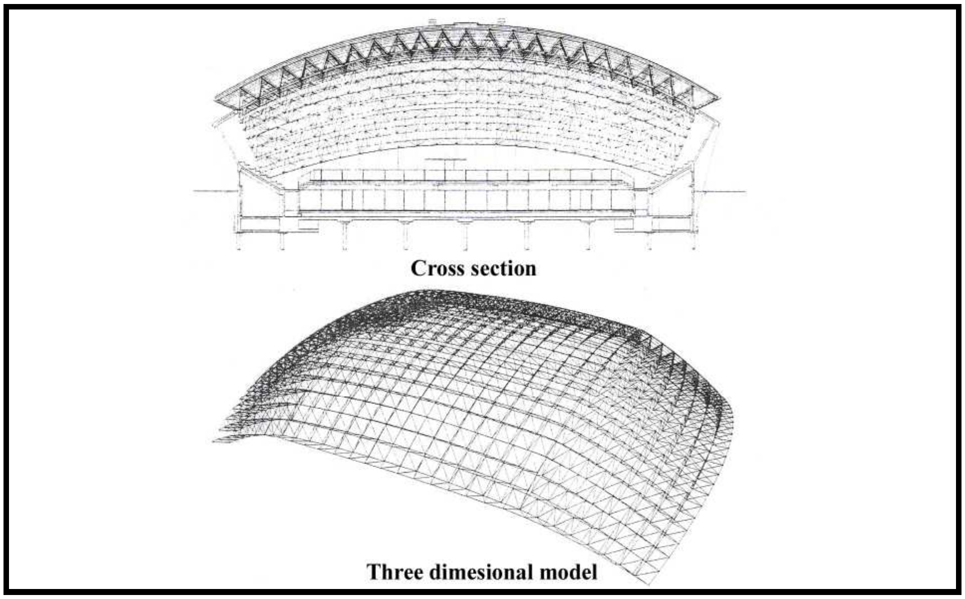


Figure II. 45 Roof structure of Oguni Dome[7], Japan (Chilton, 2000:76).

The gymnasium of Oguni is built with double curved, double layer space truss (Fig. II.45, Fig. II.46). Preservative treated cedar, *sugi* in Japanese, is used throughout the roof covers approximately 2835 m². Plan dimensions are 63 m by 47 m, additional stability is derived from the three dimensional form of the roof. Top chords are from 11x15 cm solid cedar and the bottom chords are 11x17 cm, with the web bracings of 9x12.5 cm (Chilton, 2000:75-77).

[7] Architect: Shoei Yoh, Yoh Design Office; Location: Kumamoto, Japan; Engineer: Gengo Matsui and Atelier Furai; Date:1988.

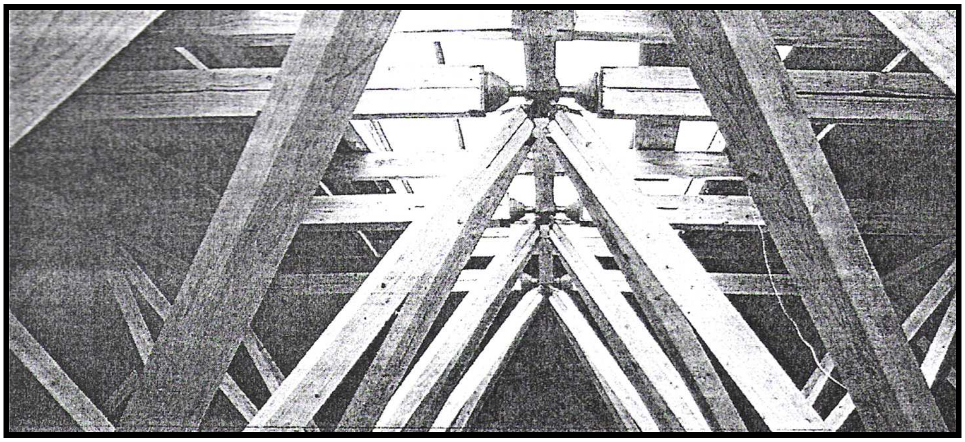


Figure II. 46 Solid cedar members of Oguni Dome (Chilton, 2000:77).

Steel space frame members may be applied by cold-rolled sections, angles or tubes riveted, bolted or welded together or to suitably shaped gusset plates or connectors. Tubes are the appropriate choice because they may be more easily joined at any angle, due to their better performance in compression will produce lighter structures especially for large spans (Foster, 1983:357). Steel members intersect at a node point, in different angles (Fig. II.47). According to Eggen *et al.* (1995:104), one way steel space frames have a span length between 10 m and 90 m, 3D space frames have a span between 20 m and 120 m. Schodeck, stated that column supported space frames have a span length between 9 m and 25 m, however wall supported space frames have a span length between 9 m and 40 m (1980:523). Shaeffer, denoted that steel space frames have a span range between 20 m and 80 m (1980:9). Callender (1982:2.106), claimed that in steel construction the members should have a ball and socket joints forming a spherical hinge, a most difficult condition to realize in practice. If all members can be made of uniform tubes of the same outside diameter, the wall thickness can be varied to maintain uniform stresses in the material. Othervise, majority of the members shall be designed oversize for the most heavily loaded member, not to be overstressed.

Toydemir (2004:136), suggested that space frames are suitable forms to be erected in site in short period of time. Toydemir claimed that a 2000 m² space frame can be erected in 5 days. Due to the three dimensionality of the form, it enables more efficiency and economy of the structural material when compared to two dimensional forms like beams or trusses.

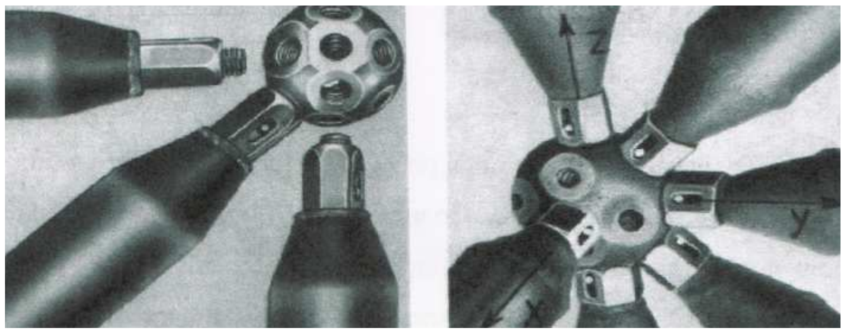


Figure II. 47 Node points in a space frame (Orbay and Savaşır, 2004:41).

According to the researcher, space frame is a form mainly suitable to steel. In timber construction due to the amount of connections, span sizes is less than steel. If joint problem is solved in timber construction, space frame can be feasible for timber. In order to simplify construction, architects and engineers tend to use members having uniform sizes and forms. Generally space frames designed with timber have a span maximum to 50 m; on the other hand when steel is used span distances up to 80 m is achieved. If the space frames are designed in three dimension, span sizes about 120 m is acquired.

II.5.6 Enhanced Roof Structures

Most timber structures only bear forces in two dimensions. Elements such as joists, trusses and rafters, act in isolation relative to each other and span in one direction, transmitting any load placed upon them to the two end supports. They need to be braced by other elements to prevent them from buckling. But for applications such as large spans, a much more efficient solution is a three dimensional structure, like a space frame or the beam grid of trusses. The disadvantages with such structures are their relatively high cost due to the complexity of the connections and increased labour time in fabrication and installation. Due to these factors, three dimensional structures are usually restricted to large spanning roof structures (Australian Government, Forest and Wood Products Research and Development Corporation, document on-line).

II.5.6.1 Folded Roofs

Karlsen defined the folded roofs as a construction consisting of thin, flat elements which are rigidly connected at right angles with one another forming a stiff cross section which is capable of carrying a load over a long span. According to Karlsen, folded plates may have triangulari trapezoidal or rectangular surface (1989:289). Benjamin, suggested that folded plates may be primarily in two types: prismatic and non prismatic (Fig. II.48). According to Benjamin, folded plates are suitable for timber and concrete but not suitable for steel. (1984:323-326). Toydemir (2004:131), mentioned that a plate is not appropriate for long spans due to its weight, but folding it increases the efficiency and divides the loads to pieces. If a flat slab is folded or bent it can behave as a beam spanning in the direction of the fold with a depth equal to the rise of the folded slab (Foster, 1983:327).

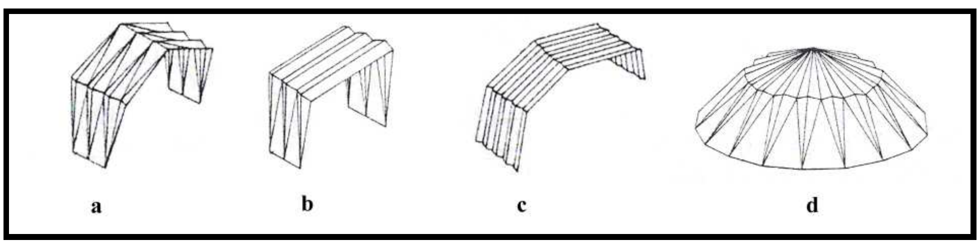


Figure II. 48 Folded plate types: a) three pin folded plate portal frame, b) two pin folded plate portal frame, c) folded plate arch, d) folded plate dome (Benjamin, 1984:326).

Gaylord and Gaylord (1990:10.24) considered that folded plate action recognizes the fact that any constituent plate is very rigid to its own plate and quite flexible perpendicular to its plane. The structure acts as each inclined or horizontal plane is made up of panels which span transversely between fold line members and are designed as simple spans.

Folded plates may be constructed in timber as framed panels, hollow stressed skin panels, laminated board panels (Foster, 1983:355). According to Schodek, plywood folded plates have a span between 11 m and 37 m and have a depth of L/4-L/6 (1980:513). Folded roof generally span 20 m to 25 m, the maximum span covered by folded system by glued pylwood construction is 30.4 m. The plates may be inclined between 20º and 45º. Ribbed folded roofs are constructed of timbers up to 15 cm deep to which sheets of plywood, particle board, or solid wood boards connected by nailing or adhesive bonds (Karlsen, 1989:289). The span and width of each bay dictates the overall depth of the structure. The depth should not be less than between one-tenth and one fifteenth of the span, or one tenth of the width, whichever is greater (Foster, 1983:327).

The researcher claimed that folded roofs are suitable for timber, but not for steel; because folded form defines a plate and steel is used in sections but not in plates. Folded plate is another form of stressed skin that the stiffness of the skin is used to distribute the loading to the supports. The folded form results in a great rigidity that the folded forms take most of their strength form their form. Folded plates designed of timber have a maxiumum span about 30 m, it is not a form that widely used. As the span increases, the depth of the folded section rise, that it is not appropriate for long spans due to increase in the depth of the structure.

II.5.6.2 Shells

Foster stated that the term ‘shell’ is generally used for to three dimensional structures constructed with a curved solid slab or membrane acting as a stressed skin , the stiffness of which is used to transfer loads to the supports (1983:319). Shells may be designed with a rib which supports a plate on it, or ribless that the whole plate is formed as a shell. Shells may be in the vaulted; hyperbolic, paroboloid, elliptical; pyramid shaped forms.

According to Callender (1982:2.109) in ribless steel shells which all forces are carried by steel plating, the danger of buckling is the major design problem. As a result the plating must be quite thick for example 1.6 cm plate at 11 kg per 0.09 m^2 of surface was needed for a 60.9 m diameter. The most heavily loaded member is of course the tension ring at the lower edge. Foster (1983:347) claimed that shells that are designed with steel can be fabricated from hot rolled sections which may be bolted or welded. Welded tubes have the advantage of lightness and stiffness. Steel shells are mainly designed with ribs which are composed of trusses as discussed previously.

a) Vaulted shells: Karlsen classified the vaulted shells as: smooth, ribbed, corrugated, modular, barrel, lamella (1989:267-272). Vaulted shells act primarily as the beam of the span of which is the length of the vault. The width of a long span barrel is usually not more than 12 m with a maximum practicable width of 15 m. The maximum economic span is about 30 m to 45 m. For small spans a radius of about 6 m and 7.5 m is used, for spans from 15 m to 30 m a radius of 9 m, and for spans over 30 m a radius of 12 m is appropriate (Foster, 1983:321). Karlsen, claimed that spans between 30 m and 60 m can be designed with glued plywood vaults, spans up to 100 m can be designed with lamella vaults (1989:266).

Figure II. 49 Roof structure of Sheffield Winter Garden (MiTek Solutions Catalogue, document on-line).

Shells in the shape of barrel arches may have a circular, parabolic, or elliptic cross section. The shape of the shell determines the design. Long shells can be designed by the beam analogy, but short shells must be designed by exact theories of anisotropic shells (Götz *et al.,* 1989:162). In long shells the load is mainly resisted by the longitudinal bending stiffness of the shell. In shorter shells the load is resisted by bending stiffness in both the longitudinal and the transverse directions. The design changes on the longitudinal and transverse bending stiffnesses. More over at the bearings the shell must be amply stiffened against shear in the transverse direction, the end of the shell must be stiffened by an arch, which may or may not be tied (Götz *et al*.,1989:162).

The Sheffield Winter Garden[8] (Fig. II.49, Fig. II.50, Fig. II.51) is nearly 70 m long and 22 m wide and rises from either end in a series of steps to a lofty 22 m over the three central bays. The single glazed building envelope is supported by a composite structure of laminated timber and stainless steel connectors in the form of primary arches and purlins. The arches and purlins are made from glue laminated arch. Glulam was chosen for its ability to be curved into the required geometry without the need for heavy plant; arch for its durability, minimal maintenance and maturing pale silvery-grey colour. The largest component was 24 m long and 90 cm deep (MiTek Solutions Catalogue, document on-line).

Figure II. 50 Roof structure of Sheffield Winter Garden (MiTek Solutions Catalogue, document on-line).

According to the researcher, vaulted shells are appropriate for timber, since their form, they act like a bent beam, and the form of the curvature is easier to construct with timber than steel. When compared with arches, vaults are more stable due to their

[8] The Sheffield Winter Garden is part of a larger redevelopment of Sheffield city centre. It is a grand urban space for people to walk through a covered park, designed to be energy efficient and to provide a model for sustainable urban development.

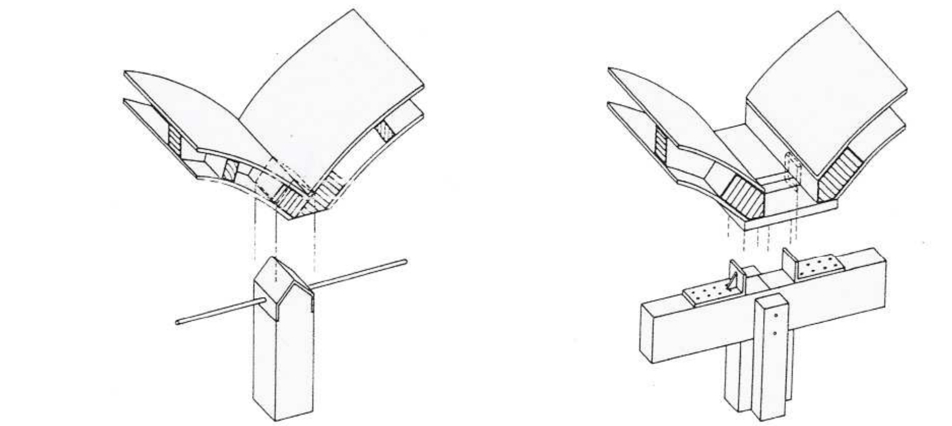
height to width ratio, because in the vaults less span is enclosed with heigher depth. With the same height an arch form has to span more distance than a vault, that the tension is more in an arch when compared to a vault structure.

Figure II. 51 Roof structure of Sheffield Winter Garden[9] (MiTek Solutions Catalogue, document on-line).

b) Hyperbolic, paraboloid, elliptical shells: A series of straight intersecting lines come together to form a hyperbolic shell, which has a saddle form. Concave and the convex shapes are the two translational surface of two parabolas. The load is carried to edge members by tensile stresses between two high points and compressive stresses between two low points. The edge members transmit the loads to bearings mostly by axial compression. The ties and the fixed bearings should absorb the horizontal

[9] Architects: Pringle Richards Sharratt Architects; Structural Engineers: Buro Happold; Completed: 2003; Area: 1,500m2; Location: Sheffield, South Yorkshire, UK.

components of bearing reactions (Fig. II.52). The design is determined by the axial loads in boards or edge members, and by shear or bending stresses generated by unsymmetrical load near the edges of the shell (Götz *et al.*, 1989:168).



a) Column with steel plate saddle bar tie, longitudinal rib of shell is notched out

b) Cross shaped column with wood tension tie, steel angles to transfer horizontal forces

Figure II. 52 Column connection for hyperbolic paraboloid shells (Götz *et al.,* 1989:167).

Benjamin (1984:334) stated that all the material in the shell structure stressed in compression or tension with little bending moment. According to Benjamin shells are divided to two catogories: singly curved and doubly curved. Hyperbolic, paraboloid shells consist of one, two, three or more hypars forming multi panel shells.These structures may roof over buildings which are square, rectangular, polygonal or curved in plan (Karlsen, 1989:325). Karlsen claimed that the mass of hyperbolic shells is reduced by nearly 30% as compared to elliptical shells of the same loading capacity.

Germany's World Expo building consists of ten identical, arched roof structures supported by columns (Fig. II.53, Fig. II.54). The theme of the World Expo was "Man-Nature-Technology" and timber, with its environmental properties, it was a rational choice for the structure.

Figure II. 53 Roof structure of the Hanover Expo Canopy[10]; Hanover, Germany (Timber Research and Development Association, document on-line).

Figure II. 54 Roof structure of the Hanover Expo Canopy; Hanover, Germany (Timber Research and Development Association, document on-line).

[10] Architects: Herzog + Partner BDA, Munich; Structural Engineers: IEZ Götz GmbH, Wiesenfelden; Completed: 2000; Area: 15.210 m².

Timber shell domes are generally designed with boards glued and nailed or screwed together, with laminated edge beams (Foster, 1983:353). Shell construction in laminated timber results a very light structure. A timber hyperbolic paraboloid shell 18 m² will be about 5 cm thick and weigth approxiametly 25 kg/m²; a comparable concrete shell would be 6.3 cm thick and weight about 150 kg/m² (Foster,1983:341).

c) Pyramid shaped shells: Trusses are usually thought of as two dimensional but the same principles can be employed to form four or more sided pyramids (Fig. II.55). Where the tension from the reactions at eaves can be accommodated in steel, timber, appropriate care should be taken to ensure that the centre lines of force are correctly appraised. A laminated structure exerting outward thrust on top of a support frame can result in eccentricity unless due care is paid. Ties need not be limited to acting around the eaves or directly across the void.

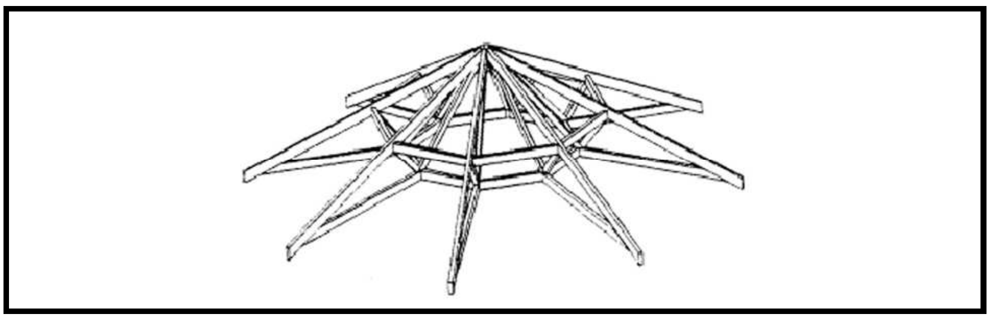


Figure II. 55 Illustration of pyramid shaped shell structure (Engineered Timber Catalogue, document on-line).

Eno is an old wood processing community in northern Karelia, in eastern Finland (Fig. II.56, Fig. II.57). It was natural that its local library should be built of timber. The load-bearing frame consists of arched glulam beams, which meet in the centre of the semicircular plan. The roof also comprises herring-bone finger-jointed pine panelling and the floor is finished with 2.8 cm thick finger jointed birch. Perforated birch ply on the walls provides the acoustic absorbency necessary for a properly functioning library space.

Figure II. 56 Roof structure of the Eno Library; Eno, Eastern Finland (Timber Research and Development Association, document on-line).

Figure II. 57 Roof structure of the Eno Library[11]; Eno, Eastern Finland (Timber Research and Development Association, document on-line).

[11] Architects: Arkkitehtitoimisto Antero Turkki; Structural Engineers: Insinööritoimisto Karrak Oy; Completed: 2000; Area: 600m².

The researcher claimed that shells are 3 dimensional designs and every design has its unique form. Design may include both concave and convax forms, every part should be specially calculated. Concave and convax parts do not have the same span and depth according to different response of timber and steel to compression and tension.

II.5.6.3 Domes

Domes are constructed either with curved members lying on a surface or of straight members with their connecting points lying on such a surface. The radial rib, a dome consists of curved rib members that extend from the base ring to a compression ring at the apex and with ring members that are at different elevations and extend from rib to rib circumferentially. The ring members may be curved or straight. The number of ribs are generally determined by the consideration of the circumference of the dome (Faherty, 1989: 9.44). Faherty classified the dome geometries in three basic types: a) Triax; b) Varax; c) Radial dome (Fig. II.58). Karlsen cliamed the domes as: a) Thin wall shell; b) Ribbed; c) Ribbed hooped, d) Meshed (1989:295-311). According to Gaylard (1990:16.50) domes may have spheroidal, elipsoidal or other shape spanning between 15.3 m and 106.7 m.

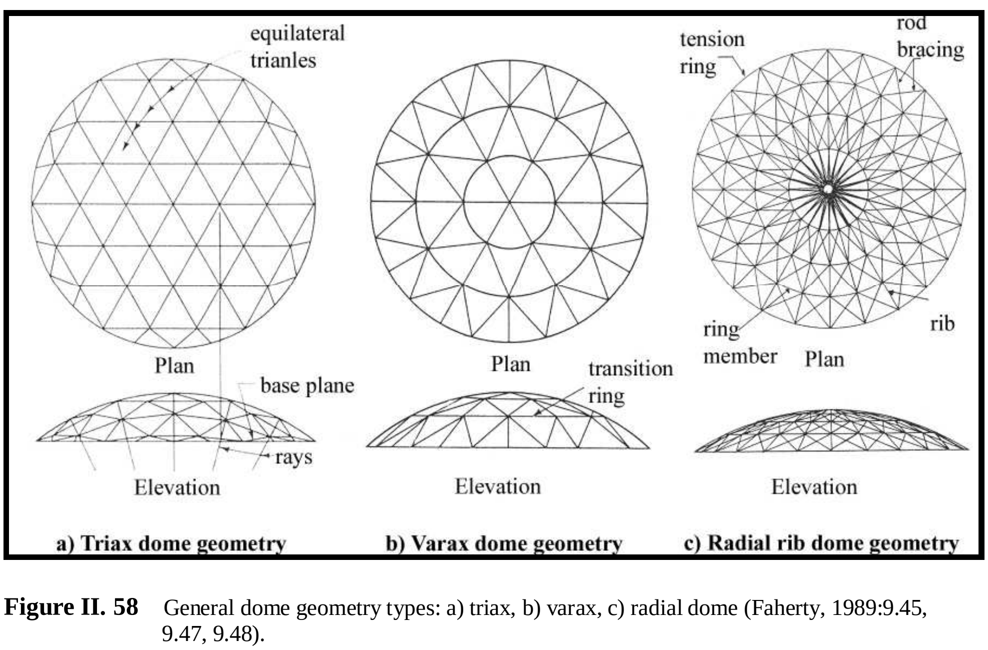


Figure II. 58 General dome geometry types: a) triax, b) varax, c) radial dome (Faherty, 1989:9.45, 9.47, 9.48).

Domes assembled from glued laminated members may have a span length up to 90 m in diameter. The structural analysis of domes depends on the type of shell, whether the imposed load is symmetrical or unsymetrical axially (Karlsen,1989:96). Tacoma Dome in the U.S.A. have a span length of 162 m and it is built by structural timber.

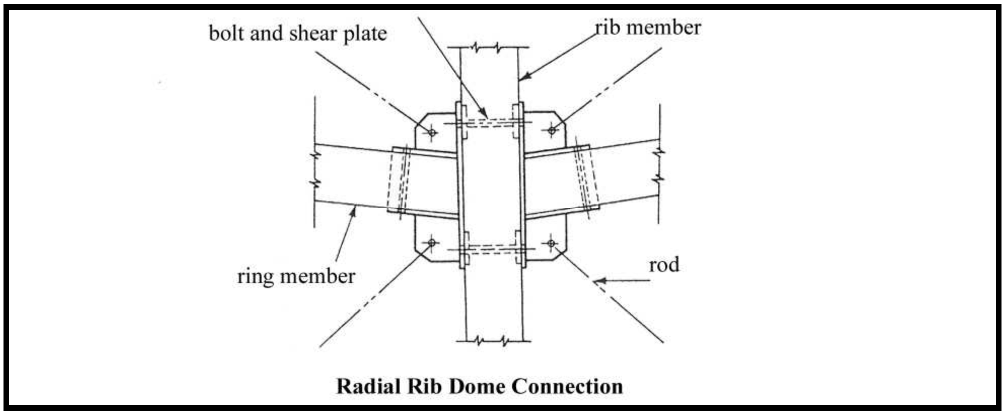


Figure II. 59 The connections of the rib members are critical. Radial rib dome conncetion is illustrated in the figure (Faherty, 1989: 9.46).

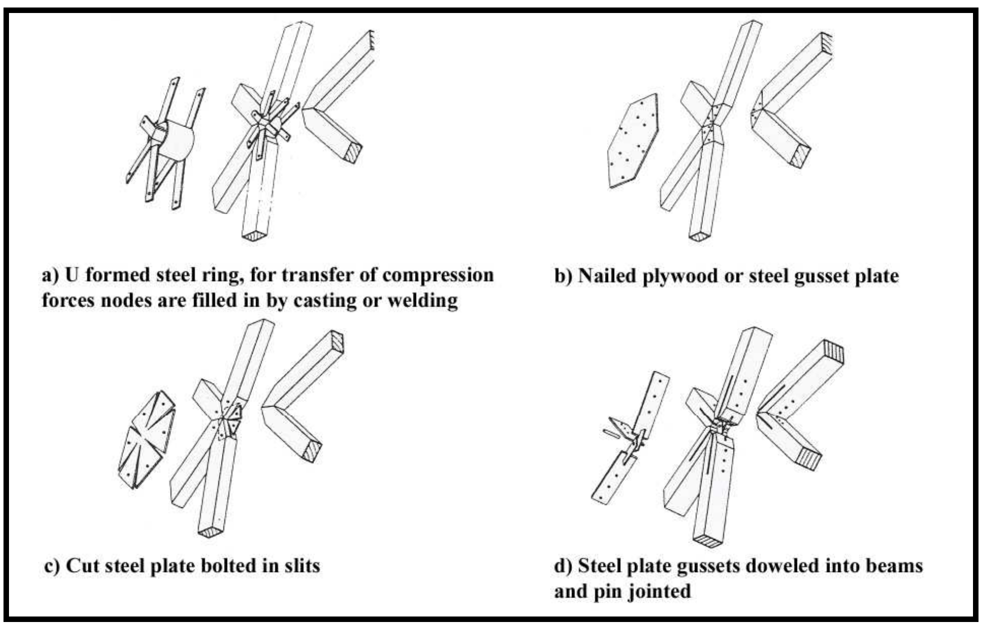


Figure II. 60 Various joint details in dome assembly (Götz *et al.,* 1989:165).

In dome design connections are very important due to members connecting from different axes. The cost of erecting the dome and X bracing can become quite high. The connections of the ring members are critical due to the magnitude of the axial compression in the ring members which may become large (Fig. II.59). Because of there is a component of force that causes shear between the connection and the side of the rib member, this force must be taken by the connectors (Faherty, 1989:9.46).

In dome design one of the major problems is that connection of elements in various directions. Timber members are connected to each other by steel joints which are composed of steel plates, rings, nails and bolts (Fig. II.60). Nailed connections do not bear forces as much as plates and rings.

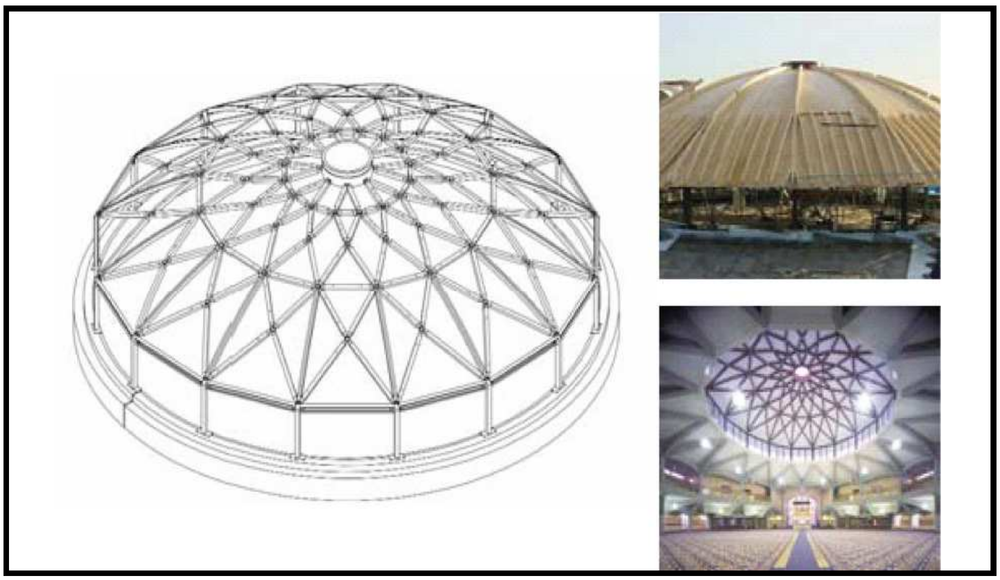


Figure II. 61 Roof structure of the Gurdwara Southall Temple[12]; London (Timber Research and Development Association, document on-line).

Gurdwara Southall Temple dome was originally conceived with the structural frame separated from the deck (Fig. II.61, Fig. II.62). The dome diameter is 18 m x 14 m radius. Decking panels are 450 wide ‘V’ groove jointed in single pieces from eaves to lantern. 176 pieces of curved pine glulam formed the ribs. The varax dome which

[12] Architects: Architects Co-Partnership; Project Engineers: Buro Happold; Completed: 2003.

spans 76 m, is home to the University of Portland Pilots (Fig. II.63). This multi-purpose facility gets heavy usage by the University as well as the community. The dome structure of Livorno Palasport has a diameter of 109 m span (Fig. II.64).

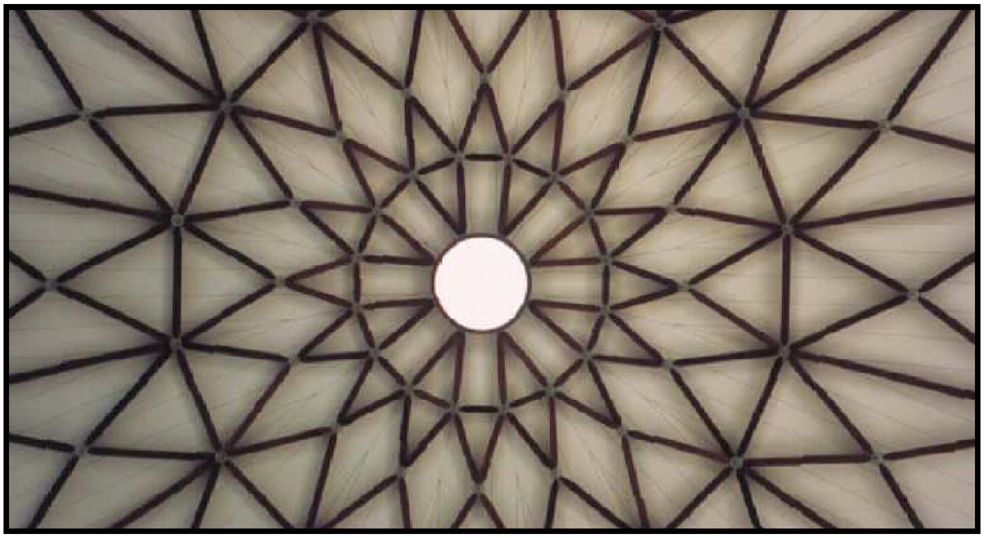


Figure II. 62 Roof structure of the Gurdwara Southall Temple (Timber Research and Development Association, document on-line).

Figure II. 63 Chiles Centre, University of Portland, Oregon, USA (Western Wood Structures, document on-line).

Schodeck stated that steel dome structures have a span length between 15 m and over 55 m (1980:523). The primary problem in using steel in the dome design is that using line elements to create a curved structure. According to Ellison (1987:225) diameters more than 121 m is constructed with steel. The members are generally designed with straight segments. The number of the radial ribs varies from 12 to 48, depending on the diameter of the dome and framing patterns. The steel ribs can be fabricated by welding or riveting. The ends of the ribs may be fixed or hinged at their junctions with the tension and compression rings. According to Callender (1982:2.109) the connections of the dome members to the piers are designed to permit a considerable amount of radial movement due to the temperature changes. This movement may be 1/500 of the span length.

Figure II. 64 Roof structure of Livorno Palasport[13]; Livorno, Italy (Holzbau Spa, document on-line).

In steel dome design one of the initial decisions is to have a form whether the dome should be a portion of a true sphere or a polyhedron. Rolled steel sections are most commonly used, since the depths of the sections can be easily found in the standard sizes. When straight members are used they form a polyhedron, if the members are closely spaced the visual effect will be that of a sphere. If curved members are desired light trusses can be fabricated to the correct radii (Callender, 1982:2.108).

The researcher suggested that, dome forms are appropriate for timber, but not for steel in long span; due to increasing height in steel, amount of connections and heat expansion of steel. Glulam ribs to form a dome is a suitable preference for dome design, that using rib form amount of connections and pieces are decreased, which end up in less workmanship, workhours; and a three dimensional stability is achieved by

[13] Architects: Antonino Valenti, Vittorio Legnani; Date:1990.

using ribs.Timber based dome forms may have a span more than 150 m, which is very suitable for long span roof structures. Steel is not preferred in long span dome design, especially over 60 m span, due to increase in height, amount of dead weight and connections of linear pieces.

CHAPTER III

PERFORMANCES OF TIMBER AND STEEL ROOF STRUCTURES

Performance of timber roof structures is greatly effected by a number factors, among which the most valid are the type of load bearing system, support locations and support types, kind of materials used for load carrying members, method of securing a structure against both local and global loss of stability, impact of joint slip, conditions of structure's performance with respect to the combined effect of humidity and duration of loading (Straka, 2000:1). Moreover transportation, connections, cost and maintanence are the performance criteria related to constructional items.

To a large degree, the load bearing capacity and primary structural response are influenced by the type of joints used in timber structures and their ability to transfer loads. This well known fact is even more emphasised in the case of space frames, since these structures require a large number of connections and members and hence their connectors transfer massive internal forces. A sufficient load carrying capacity of joints is, however, only one of the tasks which must be solved when designing a structural system. No less serious problem is posed by the construction of details which should meet two criteria: to be relatively easy to produce and to correspond to computational assumptions that should be in consistency with the calculations and design acceptance.

The dead weight to strength ratio of timber due to its weight and makes it the appropriate material for structures in which the load carrying capacity is determined by its bending moment. As a guide for structures, timber is the suitable material for in forms having a load intensity ratio square root of P/l ratio is less than 1.5, where P is the compressive load in Newtons, and l is the effective length of the member in milimeters (Foster, 1983:308).

According to Leatherbarrow, over the time of any architect's practice, the selection of materials can be more or less routine performance of previously successful habits or the more or less independent performance of design experiments. Selection in either case, can be indifferent to local possibilities of construction (1993:149). Leatherbarrow also stated that material choice depends on the local suppliers, local builders, current experiments, labor costs, owner's suggestions.

According to the researcher the performance criteria is set as following topics: a) Structural performance; b) Constructional performance; c) Material peformance. Structural performance criteria is composed of loads, duration of loads, stresses, strength, deflection and elasticity. Structural performance criteria is related with design procedure of the structure and to determine the selection of structural material that effect the form. Constructional performance is made up of site application, transportation, connection, cost and maintanence. Constructional performances are mainly related with erection of the structure. Material performance consist of fire resistance, decay, shrinking and swelling, environmental conciousness, resistance to chemicals, heat expansion and acoustical insulation. Structural performance and material performance is stable, they do not change by the fom of the structure; construction performance is related with the form, form and construction of the form should be decided together.

III.1 Structural Performance

Wood is one of the oldest building materials, which in the 16 th century Leanardo Da Vinci worked on an unusual design using wood as a loadbearing structural material. He was the first person, to discover the technique of cutting logs into tin planks, the edges of which were then notched and fitted one above the other to create combination sections that could be used as loadbearing elements to cover large spans.

Wood is a natural polymer and it retains most of its elasticity and it has small deformation in short term loading. When wood is subjected to long term constant load, deformation starts with time. If we a constant strain is applicated to a wooden element, the stress will decrease with time, although the deformation will remain unchanged. Immediately after the application of a constant load, wood shows elastic deformation (Karlsen, 1989:18). Structural performances are summarized as: a) loads; b) duration of load; c) stresses and strength; d) deflection and elasticity.

III.1.1 Loads

Loads that are carried on roof trusses are a combination of live and dead loads. Dead loads can be classified as the weight of the structural elements like truss and arch members, and the weight of any members that are carried by the trusses and arches such as, ceilings, joists, decking and the roofing material. The live loads are the snow and wind loads. On the steep pitched trusses, the snow loads are less than the roofs which have a low pitch and the wind load would be higher. Conversely, the wind load on a low pitched roof will be les than on a stock roof but the snow load will be greater because more of the snow will lie on the roof and will increase in the amount from snowfall to snowfall (Oberg, 1963:78-79).

Oberg, claimed that if there is a concentrated load at any point on a truss, or a load divided between two points, therefore the load can be computed by doubling the effective load on any truss and distributing this load over the full length of the truss. In a building if a truck rail is placed to roof and the block may be used at any part of the building all trusses should be designed as if each of them would support the load directly. On the other hand, if the load attached to a beam then the load would be carried between two trusses, and each truss would carry their proportional load due to their distance from the block. When trusses are ordered they are chopped from the tables that are produced by truss fabricators (1963:78).

III.1.1.1 Dead Loads

Forces acting vetically downward on a structure that are relatively fixed in character are called dead loads. The self weight of the structure itself is a dead load (Schodek, 1980:88). The dead load of any member is its self weight added by its share of the total weight of the rest of the structure. Binan (1990:9) stated the dead load on timber structures up to 20 m span is calculated as for 1 m^2 plan area: $P = 12 + 0.8L$ kg/m^2. Nash (1990:40,45) suggested that the weight density of steel is $72 kN/m^3$, about six times that of comercial softwoods. Wood generally has a weight density about 15 kN/m^3, timber is the lightest of the structural materials, the self weight of the structure makes a small contribution to the bending moments of spanning elements. For this reason steel is used in a solid form only for components whose cross sectional area is very small such as reinforcing bars, prestressing wire or ties in trussed members. In ‘I’ shaped cross sections there is more hallow space than solid steel.

Depending on specific loading conditions, a structural steel beam may be 20% heavier, and a reinforced concrete beam 600% heavier than an equivalent glulam beam of the same load-carrying capacity (Glued Laminated Timber Association, document on-line). According to Foster timber is a comperatively a light material and species used for structural purposes have dead load approximately 1/16 that of steel (1983:307). Foster also claimed that for spans up to 15 m the weight of cold formed steel construction per m^2 of area is about 40% to 60% less than that of similar hot rolled steel construction, greater savings can be achivied over the short spans (1983:341).

According to the researcher when timber and steel is compared in terms of dead load, it is obvious that steel is greately heavier than timber and wood products. Timbers gains its structural attraction from its low dead weight to strength ratio. Steel products are produced in forms like tubes, hallow sections and bars in order to decrease the dead weight, however timber always efficient when dead load is considered.

III.1.1.2 Live Loads

Schodek (1980:89) stated that live loads are the forces that may or may not be present and acting upon a structure at any given point of time. All live loads are characterized by their movability. Live loads are the loads that the specifically designed structure has to bear except of external factors such as wind, earthquake or snow. In a tower, live load may be very small but in a building live load can be every thing inside of it. Live loads may be either static or impact loads. Static load is the continual one, while impact load is the sudden or instantaneous force (Oberg, 1963:79).

The researcher claimed that live load is not a primary factor to decide to use timber or steel, that depending on the live load depth of the structure should be increased for both timber and steel design. Generally in roof structures live load is not a primary factor for the design calculations, that the workmans maintanence loads should be considered.

III.1.1.3 Wind and Snow Loads

A structure in the path of wind causes wind to be prevented, or in some cases stopped. The kinetic energy of the wind is transmitted into the potential energy of pressure or suction. The magnitude of the pressure of the wind depends on the velocity, mass density of the air, the geometrical shape, dimensions and orientation (Schodek, 1980:92). The wind loads create an effect on the building that would tend to push it over (Fig. III.1). However there is another force that should be considered: air passing over a roof causes an imbalance between air pressures on the top outside the building and inside of the roof. The same process occurs when the air passes over a wing, where a lifting action is created. If the air passes very fast on the roof, the serious difference in air pressures will lead to lift the roof from its supports. If the roof is not securely joined to the building this would mainly happen to the building.

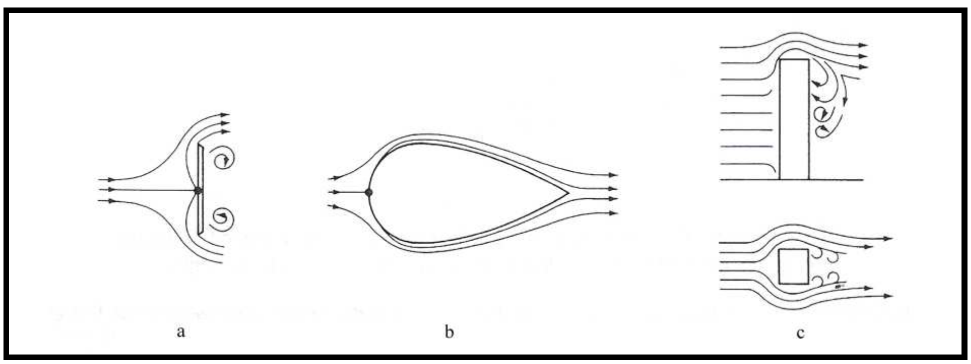


Figure III. 1 Wind flow about different shapes (Schodek, 1980:92).

Wind forces are very serious because their value of the implied force may be very high. Wind causes a horizontal push which is transferred to the whole structure. Roofs and other horizontal structures must withstand suction pressures from wind. A roof may treat like airfoil like the wing of an airlane. When air passes over a roof, a low pressure area is generated so that the higher air pressure over the roof pushes up against it. This effect may increases in the cases where the wind penetrates the structure and increases the internal pressure. Snow loads are easier to cope with because maximum amount of loading can be easily predicted and the direction of the load does not change, always downwards. Gusty winds cause impact load but snow load is a static load except for rare cases where a large amount of snow falls from one part to another part of the roof (Oberg, 1963).

Table III. 1 The amount of pressure caused by wind due to height (Binan, 1990:10).

Up to 7 m height	W=75 kg/m²
Up to 15 m height	W=100kg/m²
Up to 25 m height	W=125kg/m²
Over 25 m height	W=125 + 0.6h kg/m²

Wind loads differ due to the height of the roof structure. According to Binan (1990:10) wind comes to the roof vertically. The pressure caused by wind increase as the height rises (Table III.1). When the wind comes sloppy to the roof area wind pressure is calculated as $W=W.\text{Sin}\ \alpha^2$. If the wind pressure is calculated as W=125 kg/m², for different angles 'W'changes by 'α' (Table III.2).

Table III. 2 The amount of pressure caused by wind due to slope (Binan, 1990:10).

α	70º	65 º	60 º	55 º	50 º	45 º	40 º	35 º	30 º	25 º
kg/m²load	110	103	94	84	73	63	52	41	31	22

Snow loads on roofs depend on widely to elevation, latitude, wind frequency, duration of snow fall, site exposure, roof size, geometry, and inclination. Generally snow loads range from 958 N/m² to 2873 N/m² (Schodek, 1980:90). According to Binan (1990:10) snow loads change due to changing slope (Table III.3).

Table III. 3 The amount of pressure caused by snow due to slope (Binan, 1990:10).

α	25 º	30 º	35 º	40 º	45 º	50 º
kg/m²load	75	70	65	60	50	40

The researcher denoted that consideration of wing and snow loads are mainly related with the form of the structure, not the structural material. Sloped roofs are always have better performance for snow loads. In the areas that hurricanes occur, steel roof structures can be a better solution due to its high dead weight.

III.1.1.4 Earthquake Loads

Earthquakes are vibratory phenomena associated with sudden loadings on the earth's crust. The shock causes waves, and these waves cause buildings to vibrate. The magnitude of the vibrations depend on the mass of the building, rigidity of the structure, stiffness of the soil, type of the foundation, the magnitude of the vibratory motions (Scodeck, 1980:95). According to Oberg another type of loading is the stress resulting from the earthquake shocks when wood construction deals with very well when joints are well made and fasteners properly installed in a building that has been well designed (1963).

The researcher suggested that the movement results from an earthquake is claimed to be in a small period, high impact but rapid. Timber is a elastic material and it takes back the movement well where another structure would not absorb the shock and tend to continue in motion until the collapse occurs. Moreover when dead weight is considered, timber has less weight than steel, for the earthquake deformations.

III.1.2 Duration of Load

Timber and timber based materials deform under load rapidly more on the high stress levels than the low. The deformation's rate depends on the amount of moisture content and the dimension of cross section. A timber member can carry the design load indefinitely. However due to experiences designers should care two possibilities:
a) The long term deflection may increase, causing the pounding of water which increases the load and deflection; b) Long term deflection of heavily stressed members may cause load to be transferred to adjacent members.

Table III. 4 Load type duration adjustment of load factor (Breyer, 1998:132).

Dead Load	Permanent	0.90
Snow Load	2 Months	1.15
Construction	7 Days	1.25
Wind/Earthquake Load	10 Minutes	1.33
Impact	Impact	2.00

Faherty (1989:2.35) claimed that wood does not show any reduction in strength after 15 years of loading. National Design Specifications (NDS) recommends that when the duration of load is other than normal, the allowable stresses should be multiplied by specific factors (Table III.4). Wood has the ability of carrying a greater load for short durations than for long durations. This is particularly significant if an overload occurs, it is probably the result of a temporary load. The stresses occur in a structure are usually not the result of a single applied load, they are caused by a combination of loads that act simultaneously. The critical point is that which duration of load factor should be applied when checking a stress caused by a combination of loads (Breyer, 1998:132).

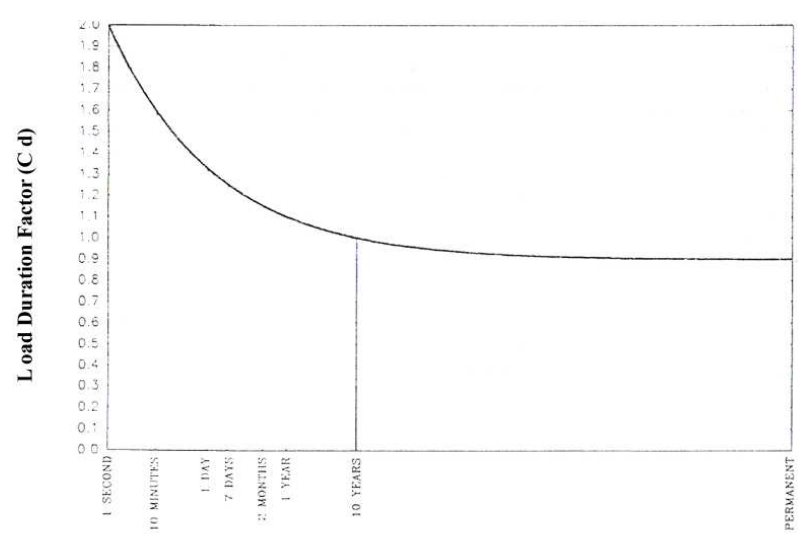


Figure III. 2 Duration of maximum load, load duration factors for various durations, decrease by the increasing time (Halperin, 1994:15).

According to Blyth, the designers should restrict the deflection less than the customary 3/1000 of the span moreover restrict the design stresses to less than the maximum permitted by the code (1989:168). The load carrying capacity is decreased independent from strength reducing characteristics, when loading at high levels and is continuous and over long periods. On the contrary, the load carrying capacity is more when loads are applied rapidly and of short duration. Test on wood specimens show that in a time of 100 years in service has no deterioration from such effects as decay, age does not have an important effect on the mechanical properties of wood (Faherty, 1989:27). Duration of maximum load factors depend by time, that structural timber can withstand higher loads in shorter time durations (Fig. III.2).

The modulus of rupture decrease in proportion, to the logorithm of time over which the load is applied. The suggestions indicate that timber beams which have to withstand a load for 50 years can be stressed to 50% of their ultimate short term strength (Illston, 1994:449).

According to the researcher timber can withstand impact loads that occur for a short duration which is meaningful when an overloading occurs, a temporary load that is not expected earlier. This is a distinctive factor when comparing timber and steel design, strength of timber does not decrease for a period of 15 years loading.

III.1.3 Stress and Strength

Stresses are the forces on a member that are caused by loads. There are five basic types of stresses: compression, tension, shear, bending moment and torsion (Oberg, 1963). When the stress-strain curves are indicated, incredibly high strength and modulus of elasticity (indicated by the slope of the curve) of steel relative to concrete and wood should be cared. Structural carbon steel, along with its high strength and modulus of elasticity, can be strained to a value 60 times greater than shown in both tension and compression, indicating a high degree of ductility. Wood has high tensile strength, but also fails in a brittle manner when stressed in tension; in compression, however, wood shows ductile behavior (Fig. III.3).

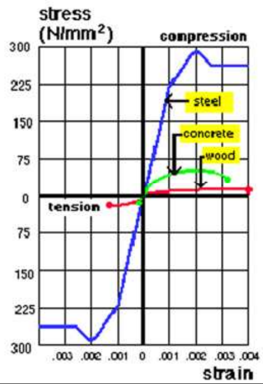


Figure III. 3 Stress strain diagram of steel and wood (Ocshorn, 1998, document on-line).

Stresses are summarized under five topics: a) compression, b) tension, c) shear, d) bending moment, e) torsion. Both timber and steel is effected from stresses and timber has various moisture content, specific gravity and strength value under these stresses. Timber has its highest strength value in bending. Compression strengths differ when the loads are applied parallel and perpendicular to the grain (Table III.5).

Table III. 5 Material properties of pine lumber used for model (Jeong-Moon Seo *et al.*, document on-line).

Property	**bending**	**compression**		**shear**	**tension**
		Parallel to grain	**Perpendicular to grain**		
Moisture content (%)	15.5	21.0	19.5	14.0	11.5
Specific gravity	0.55	0.44	0.46	0.57	0.48
Strength (N/mm²)	52.8	20.2	3.2	12.6	75.7

III.1.3.1 Compression Stress

Compression is the result of squeezing or crushing (Fig. III.4). It may be parallel or perpendicular to the grain. The most known example of compression parallel to the grain, is in post or a column. Compression that is perpendicular to the grain occurs in the bearing parts of the beam or in flooring. Oberg claimed that the most durable position for flooring and the bearing surfaces, is the vertical end grain (1963:80).

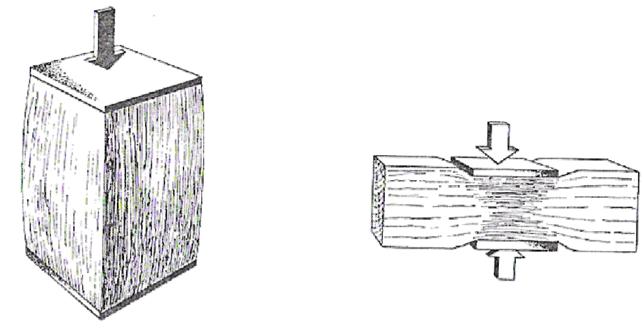


Figure III. 4 Compression of wood parallel and perpendicular to grain (Oberg, 1963:80).

Faherty, suggested that for long compressive members the wooden member may reach to its load bearing limit by buckling rather than by crushing. End to end bearing will change due to crushing strength only if a metal plate is inserted between two ends. If wood to wood end bearing occurs, latewood may bear on earlywood, moreover crushing may occur at stresses below the crushing strength of the wood. When wood is subjected to high loads perpendicular to grain , the cell cavities are eliminated and cell wall bears on cell wall (1989:13).

According to Karlsen (1989:23) standard specimens subjected to compression parallel to the grain have a crushing strength which is 1/2 to 1/2.5 of the tensile strength (Fig. III.5). The resulting average compressive strength of pine and spruce at 12% moisture content is 40 Mpa and the elastic modulus is same as in tension. The effect of wood defects is less crucial than tension. When the knots have an area about one third of the side area of compression member, the ranges of compressive strength is between 0.6 and 0.7 value of the clear specimen of the same size. In addition, compressive members are generally designed and proportioned of their ability to resist buckling rather than pure compression which results in a wider margin of safety.

Foster, claimed that the basic stresses in flexure and compression parallel to grain are 3-18 Nmm^2, depending upon the species and grade of the timber, which has a value about half as much as again in tension (1983:307).

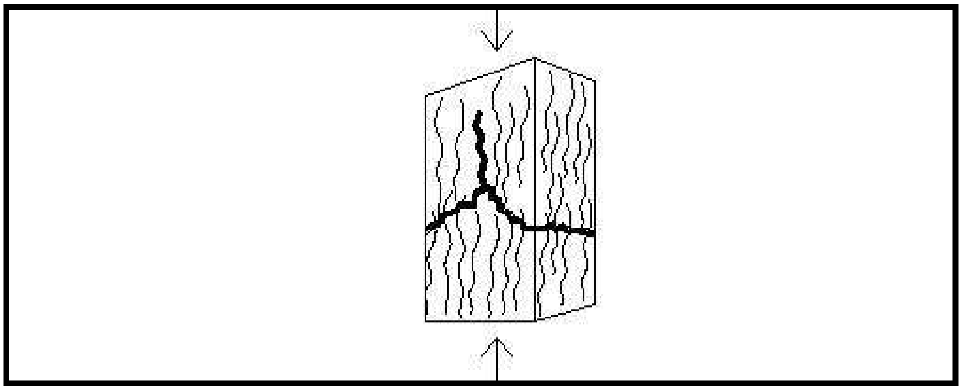


Figure III. 5 Breakdown of a test specimen under a compressive load applied parallel to the grain (Karlsen, 1989: 22).

III.1.3.2 Tensile Stress

The tensile strength of wood, parallel to the grain with 12% moisture contents is appreciable. For pine and spruce avarage value is about 100 Mpa and the modulus of elasticity is 11 and 14 Gpa. Knots and cross gained areas reduce the tensile strength of wood. Knots on the edges of structural elements are not applicable. If the knots gain the 1/4 of the member's side area strength of wood decreases by 27%. Wood's tensile strength across the grain is 1/12 to 1/17 when compared with along the grain (Fig. III.6). When the amount of cross grain area increases, the force component perpendicular to the grain increases, and the the strength of the element decreases. Cross grain is the secondary most important lumber defect and the amount in tensile members should be kept at minumum (Karlsen, 1989:22).

Tension stress is the result of pulling. It may be perpendicular or parallel to the grain. Tension parallel to the grain occurs in some members of a truss, like struts along with other stresses, in beams and girders. The ability of a member to resist tension stresses is called tensile strength. It is not possible to use all the tensile strength of a member because connectors and fasteners don't have an acceptable grip on a member. Tension perpendicular to the grain may cause splitting. It must be taken to care at the installation of fasteners. The most familiar of this case is seen in nailing (Oberg, 1963:81). Resistance to tension applied to parallel to grain is the highest strength property of wood. This resistance is however decreased when the load is applied at an angle to the grain, or when the cross section of the piece is reduced by knots or holes.

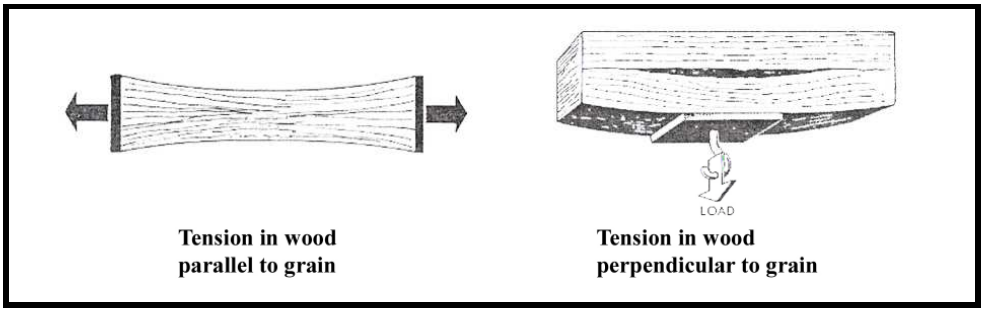


Figure III. 6 Tension in wood parallel and perpendicular to grain (Oberg, 1963:81).

III.1.3.3 Shear Stress

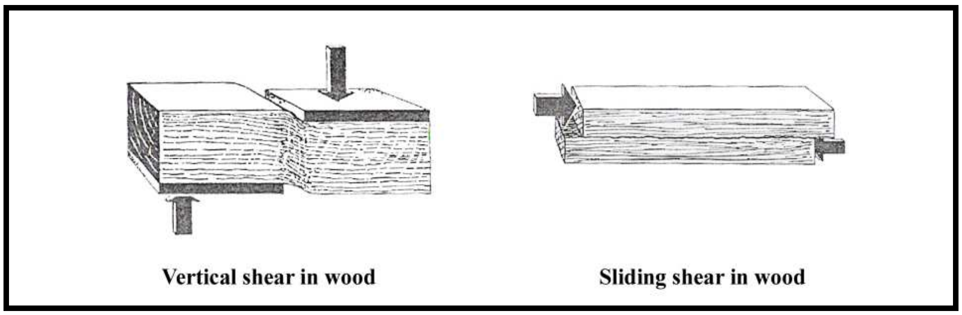


Figure III. 7 Vertical shear in wood is a shearing that is perpendicular to the grain. Sliding shear in wood is a shearing stress that is parallel to the grain (Oberg, 1963:82).

Shear is the result of opposing forces that almost meet head on which can be perpendicular or parallel to the grain (Fig. III.7). In timber construction when shear is perpendicular to the grain is called vertical shear and when it is parallel to the grain it is called sliding shear. Vertical shear occurs usually in a vertical direction near loaded bearing surfaces. Sliding shear is caused by loads which force sections of a member to slide across one another (Oberg, 1963:82).

The shear stress in a beam is often described as horizontal shear. Horizontal shear stres distribution on a wide flange steel beam and rectangular wooden beam differs

(Fig. III.8). The shear strength of wood parallel to the grain is much less than the shear across the grain, and in a wood beam the grain is parallel with the longitudinal axis. In the typical horizontal beam, then, it is the horizontal shear that is critical. The average shear stress calculation has reasonable results in typical steel beams, but it does not apply to rectangular wood beams. According to Breyer, the maximum shear in a rectangular beam is 1.5 times the avarage shear stress. The difference is significant and can not be disregarded (1988:184-185).

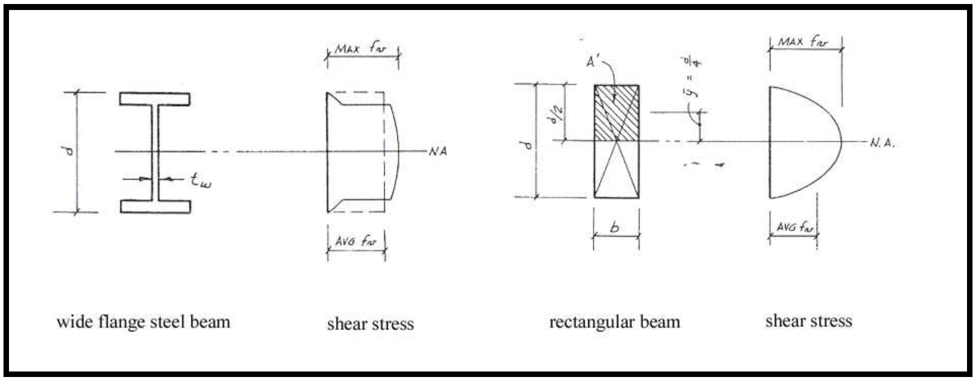


Figure III. 8 Horizontal shear stress distribution on a wide flange steel beam and rectangular wooden beam (Breyer, 1988:185).

III.1.3.4 Bending Moment

In bending of a member; compression occurs along the inner edge of the bend, tension occurs along the outer edge of the bend, and sliding shear occurs inside the bending member (Fig. III.9). The amount of bending that occurs in a member is called deflection. In beams, deflection limit is constrained about 1/360 of the span. Bending occurs due to impact or static loads. Under a static load as loading increases deflection rate increases. This proportion continues to a point called proportional limit or elastic limit. A member can be bent many times without damage as long as the proportional limit is not exceeded. Beyond this point a member can carry additional loads without failing, but it will be deformed. The next point after the proportional limit is breaking point when a member completely fails. The amount of maximum loading without any failure is called maximum loading. When an impact load is applied a member can support about twice as much deflection as it can under static loads. This property of resisting sudden loads is very serious and useful advantage of timber (Oberg, 1963:83).

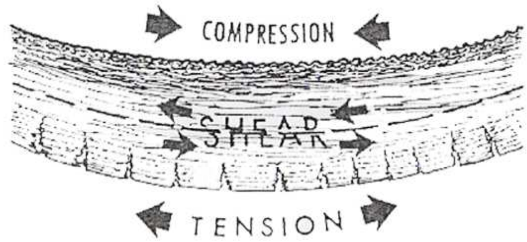


Figure III. 9 Bending a length of wood sets up multi-stresses: compression, shear and tension (Oberg, 1963:83).

The strength of wood in lateral bending is in between compressive and tensile strengths. The average bending strength of standard pine and spruce equals to 75 Mpa at a moisture content of 12%. The modulus of elasticity is almost the same as in compression and tension. Some portion of a bending member is always in tension, due to that knots and cross grain have a considerable effect on the strength. With the knots and cross grain having an area one third of side area of a bending member, the strength varies from 0.5 to 0.45 of the total value attained by straight-grained specimen. The bending strength of square timbers and especially logs have a range between 0.6 to 0.8 of the ultimate strength value (Karlsen, 1989:23).

The bending stength of wood changes according to the shape of the member cross section. For example being the section of modulus is the same, round members are stronger than rectangular members, which in turn stronger than 'I' section member. The increasing the depth of the cross section causes a decrease in the bending strength (Karlsen, 1989:24). Foster, stated that timber's bending strength changes about one 1/28 to 1/23 that of mild steel, due to species and to the presence of the knots and faults in timber (1983:142).

III.1.3.5 Torsion

Torsional stresses are the result of twisting. In the four types of the stresses torsion is the least important one in timber structures. It occurs but it the amount of it is negligible compared to other types of stresses. Large timbers that are twisted should be carefully placed. Cuts and joints should be done because timbers can be installed without forcing. If a twisted member is forcibly straightened and pulled into place, it will cause continual and possibly dangerous stresses on it's joints because it may return to it's original position (Oberg, 1963:86).

Shaeffer, basicly defined the strength as adequateness of materials selected to resist the stresses generated by loads and shapes of the structure (1980:3). A major consideration in designing a roof structure, is the most economical solution to span the roof and its dead load over the span of varying degrees. In all types of structures, it is possible to keep the dead weight to a minimum so that the imposed loads can be carried within a greates economy of material. The structural problem in designing a wide span roof structure is, therefore, acquiring a dead/live load ratio as low as possible while regarding all other factors relating to design as whole (Foster, 1983:162). In solving these problems there are two important factors: a) characteristic of the material used; b) the design of the roof structure.

Numerous tests show that strength of wood may show a considerable change even within a single specie. For example the latewood of conifers is three to five times strong than the earlywood. The density and strength of wood increases by increasing thickness of tracheid walls and percentage of latewood (Table III.6).

Holes, mortises, notches and cuts decrease the strength of wooden members. Actual strength of a weakened element turns out to be lower than the net section value because of the adverse effect of stress concentration at the sudden changes of cross section (Karlsen, 1989:22). Strength increases with density, decreases as moisture content rises. A 1ºC temperature rise reduces strength by about 0.3% (Everett, 1986:51). Taylor, (1991:16) stated that timber has an elastic modulus of 3-30 kN/mm², however steel has 210 kN/mm² for a given strain, stiff, materials having higher E become subject to higher stress.

Table III. 6 Mechanical properties of european lumber for 12% moisture content. //=parallel to grain ⊥=normal to grain (Götz *et al.,* 1989:31).

Type of wood	**Modulus of elasticity N/mm²**	**Compressive strength N/mm²**	**Tensile strength N/mm²**	**Bending Strength N/mm²**	**Shear Strength N/mm²**
Spruce //	11000	43	90	66	6.7
⊥	300	5.8	2.7	-	-
Pine //	12000	47	104	87	10
⊥	-	7.7	3	-	-
Larch //	13800	55	107	99	9
⊥	-	7.5	2.3	-	-
Beech //	16000	62	135	105	10
⊥	-	9	7	-	-
Oak //	13000	54	90	91	11
⊥	-	11	4	-	-

Wayne suggested that basicly glulam is not stronger than lumber, because gluing does not have an effect on strength, but wisely selected laminations the bending strength exceeds that of the lumber (Table III.7). High grade laminations are placed near the surface where the bending stresses are highest, low grade pieces are placed to the center near the neutral plane (1991:79). In curved timbers the bending decreases stress in each lamination, and lowers the strength of the timber as a whole. The effect is stronger as lamination thickness and curvature increase.

Table III. 7 Strength values of glulam and timber. The values are given in Mpa (Tokyay, 1995, document on-line).

	Bending	Compression	Shear
Glued laminated timber, class 12410	16.55	4.48	1.14
Massive structural timber, class 11030	9.31	4.31	0.59

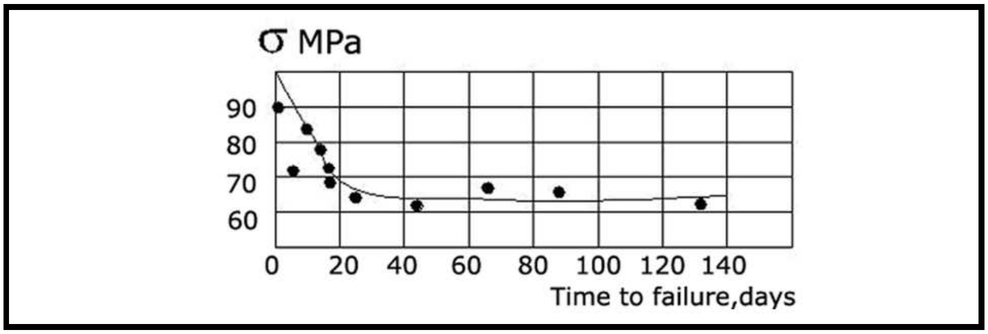


Figure III. 10 Long time strength curve for wood (Karlsen, 1989:20).

Timber's mechanical properties parallel to grain are different from the properties perpendicular to the grain. Compressive strength parallel to grain may be 5 to 10 times greater than the strength perpendicular to the grain. Modulus of elasticity parallel to grain is 10 to 25 times more than the value of perpendicular to the grain (Faherty, 1989:13). It has been experimentally found out that the strength of wood is directly proportonal to its density. Although it does decrease with increasing duration of load, the ultimate strength of wood does not fall without bound (Fig. III.10). Below the asymtote curve no failure will occur, irrespective to the duration of load. Above the asymtote the failure is inevitable, and the time to failure will decrease with increasing difference.

Connection details must effectively transfer loads, utilize durable materials, and be as free from maintenance as possible. Wood is an anisotropic material, it behaves different in x,y,z coordinates. The strength of wood is different in the parallel and perpendicular grain directions. Wood also has much less strength in tension perpendicular to grain than in compression perpendicular to grain. These facts influence design details. Vertical loads should be transferred so as to take advantage of the high compression perpendicular to grain strength of wood. For example, a beam should bear on the top of a column or wall or be seated in a shoe or hanger. Such a detail is preferred to the support of a beam by bolts at its end, particularly where there are large numbers of bolts.

Beams should be anchored at the ends in order to carry induced horizontal and vertical loads. Vertical loads may be either gravity loads or net uplift loads. The connections typically shown in this standard are primarily for vertical gravity loads. Provisions should be made to resist uplift or lateral loads as required. The bolts or fasteners at the beam ends must be located near the bottom bearing of the beam to minimize the effect of shrinkage of the wood member between the bottom of the beam and the fasteners. In many cases, individual details do not include all structural elements, such as lateral bracing ties to connect all of the components of the building together. Loads suspended from glulam timber beams or girders should preferably be suspended from the top of the member or above the neutral axis (American Institute of Timber Construction, document on-line).

Table III. 8 Material properties of carbon, low alloyed and cast steel (Eggen *et al.*, 1995:36).

	Carbon steel	**Low alloyed steel**	**Cast steel**
Carbon content weight %	0.15-0.20	0.20-0.25	0.50-1.50
Density kg/dm^3	7.8	7.8	7.8
Surface tension	230-300	340-450	200-400
Breaking tension	350-550	400-700	400-600
Temp. gives 50% reduction in firmness	500ºC	500ºC	500ºC

Steel has a high strength in both compression and tension, small amount of material is needed to carry large loads. Stresses on carbon steel, low alloyed steel and cast steel differ (Table III.8). Working stresses for mild steel of 165, 230 and 280 N/mm² for grades 43, 50 and 55 respectively are permitted for all normal structural members by BS 449 (Foster, 1983:305). According to Owens (1992: 199) steel can be supplied with stength levels from about 250 N/mm² up to about 2000 N/mm² for common structural purposes.

III.1.4 Deflection and Elasticity

All materials have deformation under external forces and temperature changes. In timber products deformations due to creep and moisture content, the latter in the form of swelling or shrinkage. Götz *et al.* (1989:71), stated that a deflection equal to 1/300 of span length is still acceptable. Under normal load all wood trusses have a deflection between the range of 1/500 and 1/1000 of the span length, the deflection at failure being is around 1/125 and 1/1200 of the span. Trusses having bottom chords of glued laminated timber with finger jointed laminations show smaller deflections under load (Karlsen, 1989:219). According to Breyer actual deflection is controled by the limit of the span divided by 240 (1988:193). When compared to other structural materials wood has a low modulus of elasticity. In long span members deflection can be the critical item. Obviously if this item is recognized, if more restrictive deflection criteria is used in design, the trial member size should be based on satisfiying deflection limits (Breyer, 1988:193).

According to Breyer for steel members K is taken as zero, it has no tendency to creep under normal temperature conditions. Tendency of wood to creep is affected by its moisture content. The dryer the member , the less deflection under sustained load. For seasoned lumber, K is taken as 0.5, for unseasoned wood K is taken as 1. Seasoned lumber is defined as having a moisture content less than 16 percent of the time of construction (Breyer, 1988:27).

Foster suggested that a structural material under stress should not stretch or contract to an excessive degree. This is important in horizontal members where large deflections due to loading must generally be avoided. The ratio of stress to resultant strain known as modulus of elasticity, indicates the extent to which the material will resist elastic deformation. If the resistance is high the material is stiff, the deformation under stress will be low, and the deflection of the beam will be small. Minumum depth of a section is designed by deflection rather than the strength of material, a high modulus of elasticity allows a shallower beam section, or a greater span. Steel has a high modulus of elasticty of 200 kN/mm^2 showing that it is a stiff material (Foster, 1983:305).

The strength of normal structural softwoods in bending is approximately 1/28-1/23 that of steel. The basic stresses in in flexure and compression parallel to the grain are 3-18 N/mm^2, depending upon the species and grades. The modulus of elasticity of normal structural softwoods is from 4 to 12 kN/mm^2. Some kinds of timbers like douglas fir have somewhat higher value, timber has a low modulus of elasticity when compared to other materials (Foster, 1883:307-308). Steel is a ductile material, it has a yield point of up to 350 N/mm², undergoes considerable strain after the elastic limit and before the ultimate failure (Foster,1983:305).

III.2 Constructional Performance

In order to decide a structural material, the decision lies behind constructional performance as well as structural and material performance. Construction technique, speed of erection, transporting the material, preserving the members, holding the elements, workability of the material, connecting the members, cost of the material and secondary costs, maintanence during erection and after erection greatly effect the decision of the structural material and design. These topics are summarized under five basic subjects such as: a) site applications, b) transportation, c) connections d) cost, e) maintanence.

III.2.1 Site Application

Structural members of wood, laminated wood and wood products should be built into structure with the moisture content that is expected to be during the use of the structure. Erection should be done by professional contractors who are experienced in erection and connection of timber. It should be based on appropriate construction sequence plans and on a knowledge of erection forces. The way of erection may effect the choice of structural system, design of columns, structural members and details. During the erection period, necessary bracings should be placed because the effectiveness of stiffening works only after all the members are placed (Götz *et al.*, 1989). Because of ease in working and handling and its comparative lightness in weight, erection time may be shorter than steel. Falsework may be easily removed by hand tools.

Burchell and Sunter, (1987:59) mentioned that the team of erectors should be equipped with claw hammers, boxed out nail pouches, measuring tapes, level, step ladders, full set drawings, specifications and adequate supply of nails. The fixing of nails should never done in straight angles, always at an angle, therefore staggered down the length of timber to prevent splitting. A pre-nailed temporary batten should be marked with the correct spacing.

Wood trusses are designed and fabricated based on exacting specifications. However, all this is at stake in the handling, erection, and most importantly, bracing stages of construction. Trusses must be erected properly to ensure they perform as expected, and for job site safety. Trusses may be erected manually, by fork lift, crane, depending on truss size, wall height, and job conditions (Fig. III.11).

Figure III. 11 25 m x 30 m roof structure over theatre cantilevered off 4 support points. Structure is hold by cranes (MiTek Solutions, document on-line).

Trusses should always be hoisted vertically to avoid lateral bending that could damage truss members or joints. All trusses must be securely braced. Trusses installed manually are slid into position over the sidewall and rotated into place using poles. Manufactured trusses are stored in a proper manner in order to prevent the damaging of trusses (Fig. III.12). When the span increases, more workers are needed to avoid excessive lateral strain on the trusses. Burchell and Sunter, (1987:57-58) claimed that roof trusses should always be carried, stored and hoisted in an upright position.The thickness of the trusses are grater on the points where metal connecting plates are placed, therefore it must be cared that when they are stored and transported together trusses may have distortion on the connection points. Merritt, (1982:7.87) pointed out that trusses and arches are generally shipped to site disassembled, they are assembled on the ground to decide the connection points before erection.

Figure III. 12 Manufactured trusses are stored on site in a proper manner (MiTek Solutions, document on-line).

Temporary bracing is used during erection to hold trusses in place until permanent bracing, purlins, sheathing, and ceilings (if used) are installed. Proper erection bracing will assure that trusses are installed properly, and create a safe working environment. Permanent bracing is used to make the truss component an integral part of the roof and building structure. Temporary and permanent bracing includes lateral, diagonal, and cross bracing (Fig. III.13). Building designers are responsible for the proper design of permanent bracing. Erection contractors are responsible for the proper installation of temporary and permanent bracing (Forest and Wood Products, Research and Development Corporation, document on-line). The bracing members must have their forces resolved. Trusses simply braced to each other may fall down as a unit. A compression member should be braced laterally in order to prevent buckling (Fig. III.14).

Figure III. 13 Glulam arch with metal connectors at end is being hosted for assembly (Timber Research and Development Association, document on-line).

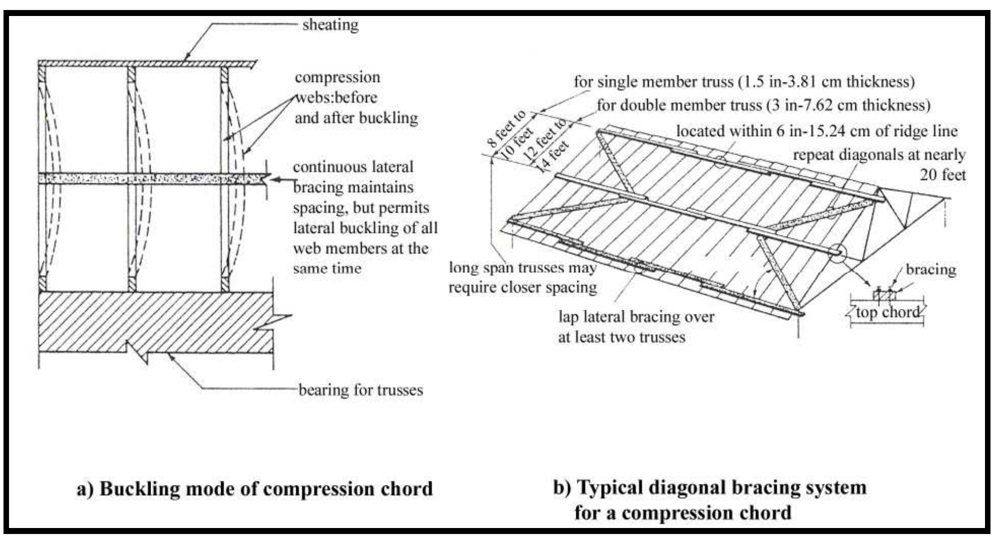


Figure III. 14 a) Buckling mode of a compression chord when lateral bracing is not tied to a braced system. b) Typical diagonal bracing for a compression chord (Faherty, 1989: 6.93, 6.94).

Burchell and Sunter, claimed that if covered for protection from rain and sun, enough space should be left for air circulation to permit timber to breathe (1987:57). It is crucial that hot sunshine can cause as much but not more damage to kiln dried timber than rain. Merritt, suggested that timber members should be seperated with strips so that air may circulate around all sides of each member (1982:7.84). A protective wrapping of water resistantant paper, the paper should remain intact until the roof covering is in place.

Steel construction needs an established sequence for completing the corrections. According to Merritt, (1982:8.99) during erection the number of bolts is kept to a minimum, just enough to draw joint up tight and take care of the stresses caused by dead weight, wing and erection forces. Permanent connections are made as soon as alignment is within tolerance limits. Some erectors prefer to used permanent high strength bolts for temporary fitting up. Because bolts used for fit up are not tightened to specified minimum tension, they may be left in place and later fastened as required for permanent installation. Merritt, claimed that it is standard practice to compansate in shop details for certain mill variations, adjustments are done in the site, usually with clearances and mills. Erected beams are considered level and aligned if the deviation does not exceed 1:500 (Merrit, 1982:8.102).

Erection of steel structures also requires a welding sequence, to control distortion due primarily to the effects of welding heat. In general, a large input of heat in a short time tends to produce the greatest distortion. Therefore it is advisable to weld in stages, with sufficient time between each stage to assure complete dispersal of heat. If beam flanges are to be field welded and the shear connection is a high strength bolted friction type joint, the holes should be made oversize, in order to provide some built in adjustment for site tolerances.

According to the researcher site application of timber needs care for moisture content of the members, temporary bracing is needed, elements should be protected from outside weather conditions. Air circulation should be provided between the members. Timber has advantages during erection when compared with steel, that it is easy to hold due to its less dead weight than steel, site falseworks are easy to correct with simple handtools, erection time is less than steel especially when one or two piece glulam members are used.

III.2.2 Transportation

Transportation is crucial in terms of moisture content, storage and cost. The moisture content of wood products should not basicly change during their transportation. Storage in the open air should not be allowed. The cost of transportation can have a substantial influence on the choice of the structural system (Fig. III.15). For example, two hinged arches must be constructed with field splices because individual arches can not be transported by train or truck due to their dimensions. Rail transportation requires a length of 2.5 m x 2.8 m dimensions of a horizontally lying girder. The length of a girder depending on the railway line can not be more than 50 m or 60 m. In special cases, the width of transportation can be 6 m if wide trucks or deep loaders are used. A length of 40 m can exceed only by special license in western Europe and USA. During the transportation period the timber girders should be well protected by cushioning and covers so that generally excellent protection is achieved against humidity and dirt (Götz *et al.*, 1989:79).

Trusses should be transported in the height limits of about 4 m, this means a pitched truss of 12 m span or more has to be disassembled (in part or completely) for transport. Trusses laid on factory nailed or dowelled connections are not appropriate for disassembly unless they are used in combination with bolted nodes. Complete disassembly can be an advantage in decreasing transportation costs especially to remote locations. When complete disassembly is required, the construction tolerances require that the members should be marked so that components from other adjacent trusses are not mixed during reassembly. Additionally, the finished truss should be protected during transportation and erection and may require water repellents, priming, or even protective wrapping (Australian Government Forest and Wood Products Research and Development Corporation, document on-line).

Figure III. 15 Structural timber and lumber is being transported by trucks (Alpine Engineered Products, Inc., document on-line).

According to the researcher, transportation of timber or steel members is mainly governed by the form, size and weight of the structural elements. Both in timber and steel structures, the whole structure may be produced and assembled at the job site, if it not able to be transported from the factory to the site. Generally whole structure is composed of from smaller and transportable pieces and they are bounded at the site. Since timber has less dead weight than steel, transportation costs are lower than steel members. During transportation placement of both timber and steel members is important, timber should be protected from water, vapor, insects and fungi, on the other hand, steel members must be kept away from chemicals and oxidation.

III.2.3 Connections

In the connections tension, compression and shear forces are transmitted through shear and bending stresses in connectors to bearing loads on the connected members. The development of timber structural systems has always been linked with the development of new types of connectors (Fig. III.16, Fig. III.17). Traditional mechanical methods of connecting, in which timber members are interconnected directly, are not optimal for creating spatial systems. Load carrying capacity of such connections is rather low in comparison with that of connected timber profiles. For that reason it is necessary to locate a large number of connectors in one joint. The analysis shows clearly that to the most advantageous types of connections belong those made by means of steel plates, or steel elements. Their major advantage lies in a significantly higher load-carrying capacity, the possibility of locating connectors in relatively small interfaces, the possibility of precise fabrication and simplicity of assembly (Straka, 2000, document on-line). Joints used in structural timber are nails, screws, bolts, toothplate, ring, shearplate, pressed metal and plate.

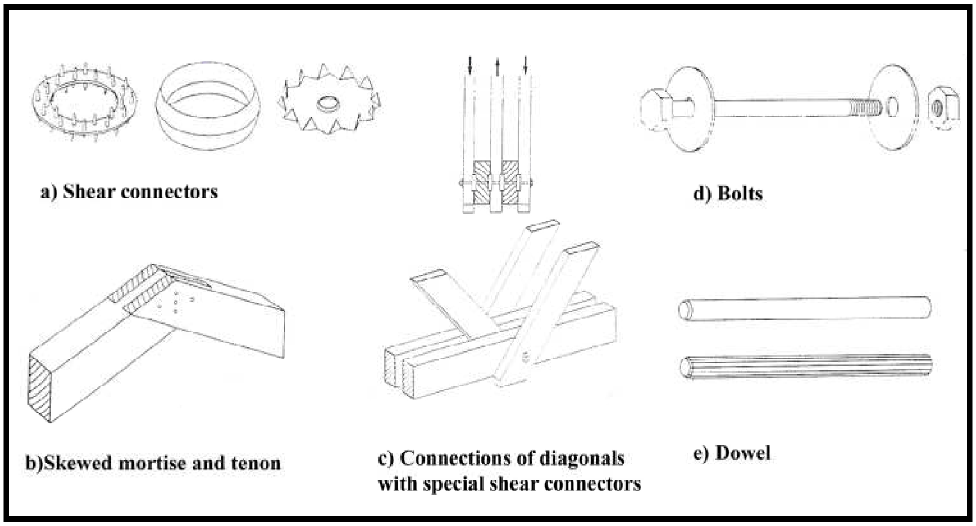


Figure III. 16 Typical timber connections used for jointing timber pieces (Götz *et al*., 1989:47).

Chilton, suggested that transfer of forces in timber structures between members at the joints is one of the design considerations. Because of the highly axial forces experienced by space grid members, individual members usually have metal inserts introduced at the ends so that forces can be transferred over a greater length of the member. These metal inserts are connected to metal nodes or directly to each other (2000:30). Merritt, (1982:7.32) claimed that fasteners subject to corrosion or chemical attack should be protected by painting, galvanizing or plating. In highly corrosive atmospheres such as in chemical plants, metal fasteners and connections should be galvanized or made of stainless steel. Connections may be covered with hot tar or pitch. In such extreme conditions lumber may be at or below equilibrium moisture content at fabrication to reduce shrinkage. Such shrinkage may open avenues of attack for the corrosive atmosphere.

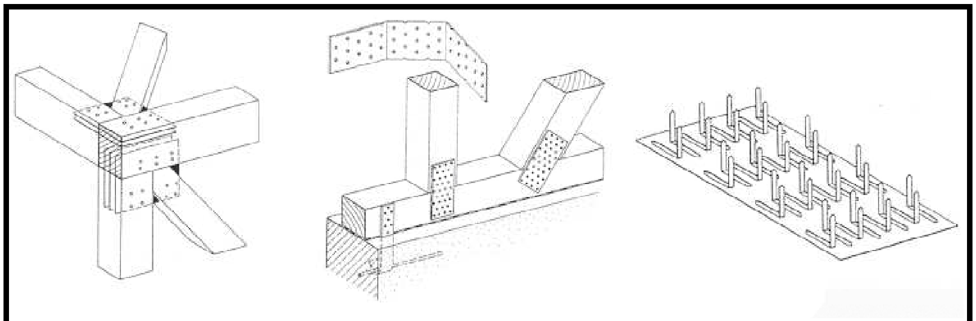


Figure III. 17 Various connections are used for jointing timber members (Götz *et al*,. 1989:52).

Connectors used for wood joints have some specific properties that effect the performance of the conncetion. Nails should not be fastened closer together than half their length, unless driven in prebored holes. Moreover nails should not be placed closer to an edge than one quarter their length. Wood screws should not be loaded in withdrawal from end grain. They should be inserted perpendicular to the grain by turning into predrilled holes. When properly designed metal side plates are used, allowable loads may be increased by 25% both for nails and wood screws. Bolts acting parallel to the grain, the distance from the center of a bolt to the edge of the wood should be at least 1.5 times the bolt diameter. For bolts bearing perpendicular to the grain, center to center spacing across the grain should be at least four times the bolt diameter if wood side plates are used. Split rings are the most efficient device for joining wood to wood. They are placed in circular grooves cut by a hand tool in the contact surfaces. Shear plates are useful for demountable structures, they may be installed in the members immediately after fabrication and held in position by nails. The role of the bolt through each plate is to prevent the components of the joint from seperating, loads are transmitted across the joint through the plates. To attach columns or arch bases to concrete bases to concrete foundations, anchor bolts are embeded in the concrete, with sufficient projection to permit placement of angles or shoes bolted to the wood (Merritt, 1982:7.32-7.40).

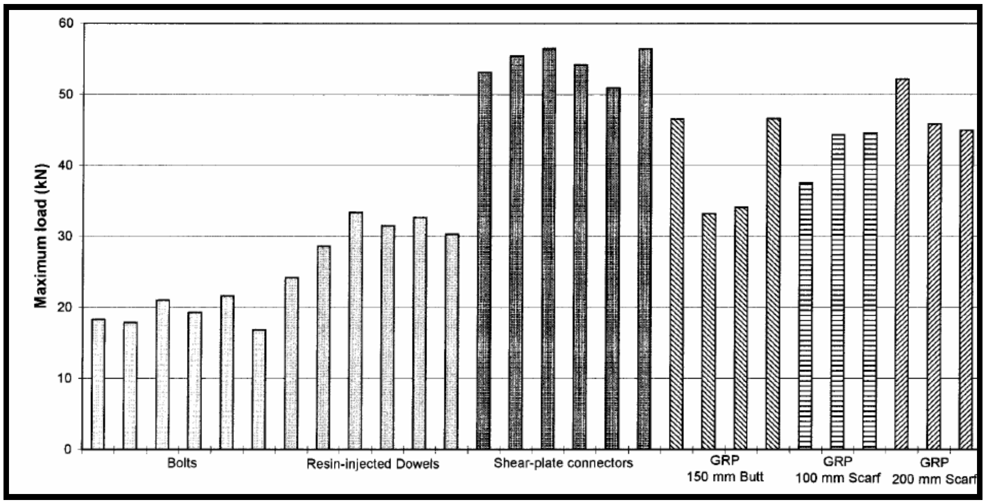


Figure III. 18 High performance jointing systems for timber. GRP: Glass Reinforced Plastic (Peter A. Claisseu, Tim J. Davis, 1998:415-425).

Table III. 9 Comparative strength of joints made with mechanical connectors loaded in shear parallel to grain (Blyth, 1989:163).

	Size mm	**Number required**	**Approx load kN**	**Notes**
Nails	.4.5	18	10	<50 mm point penetration
Screws	5.6	14	10	<40 mm point penetration
Bolts	19	2	10	Timber < 33 mm thick
Toothplate	63	2	10	Timber < 33 mm thick
Ring	64	1	10	Timber < 40 mm thick for two connectors
Shearplate	67	1	10	Timber < 40 mm thick for two connectors
Pressed metal plate	125x75	78 points	10	Same area and points to each side of joint

Cowan and Smith (1988:154) stated that a great advantage of timber is the ease with which it can be worked and jointed by hand, but the joints produced are much less efficient than those avaible with metals. All connections must be designed and detailed to transfer imposed loads from one structural member to another; without causing overloads and subsequent failure at the connection and then to the foundation (Table III.9). Connection designs include the use of nails, screws, bolts, specialty hangers, metal connector plates and nail gangs for trusses (Fig. III.19). The structural design, effect of moisture cycling and aesthetic features are important considerations when designing and specifying connection details. Timber with concrete connections are being developed and tested for large timber structures (Fig. III.20). Application areas are foreseen in bridges and repair of structures. The research and development project includes long-term testing of joints and earthquake resistance of timber concrete connections.

Figure III. 19 Nail gang connections on trusses (MiTek Solutions, document on-line).

Connection detailing becomes particularly important in a structure's resistance to lateral forces, such as those induced during a high wind or seismic load. Damage is greatly reduced when all framing elements are solidly tied together and then firmly anchored to the foundation. Notching can negatively affect connection strength and should be avoided. Whenever possible, it is important not to cut notches or holes near a connection. This could reduce the performance of the structural member, especially in header and beam applications. Most connections occur at the ends of members where wood end grain is exposed and susceptible to moisture movement. In any type of wood construction, it is important to maintain the desired appearance (Southern Pine Council, document on-line).

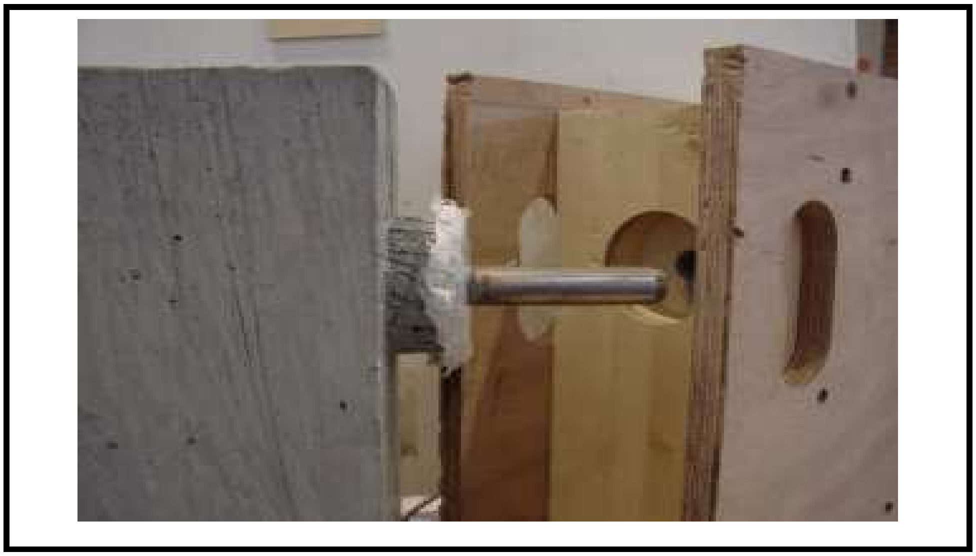


Figure III. 20 Timber with concrete connection is made with a 20 mm steel dowel (Kuilen, J-W.G., document on-line).

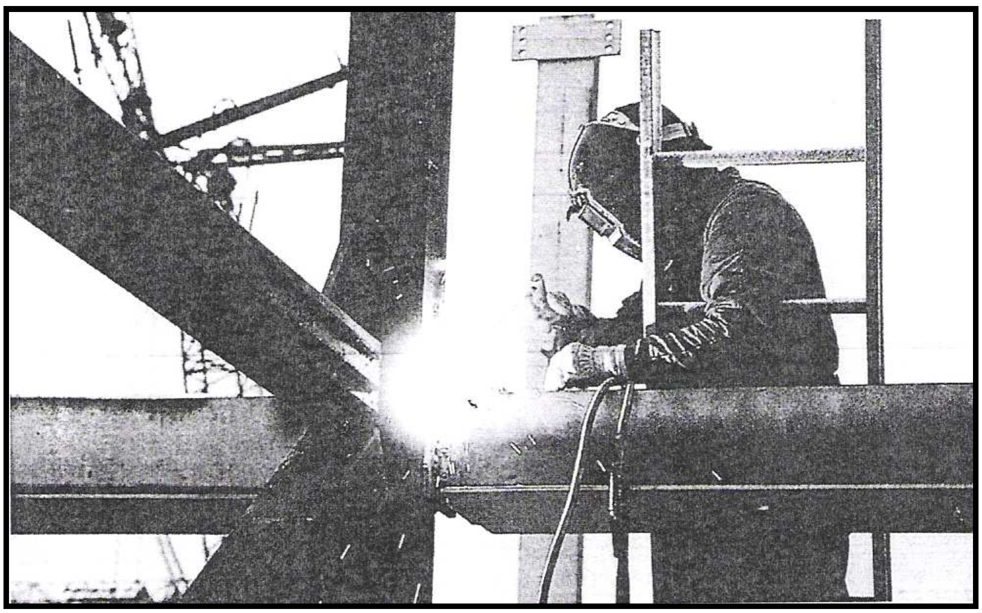


Figure III. 21 Pin welding at the job site (Eggen *et al.,* 1995:43).

Steel members are jointed either by welding, bolting or riveting. Deciding whether a connection should be welded or bolted depends on a number of factors such as project's structural concept, construction of the entire structure or some parts would be joined at shop or not. Welding can be done by gas, manually or automaticly. Welding is the most economical method of obtaining strong connections at the shop. At the building site, connections are usually done by bolting, because it is easier, quicker and cheaper. It is best to bolt the surface treated elements, welding the galvanized or painted members should be avaioded. On the other hand, both holes should be pre-drilled and the structural element should be galvanized before site assembly.

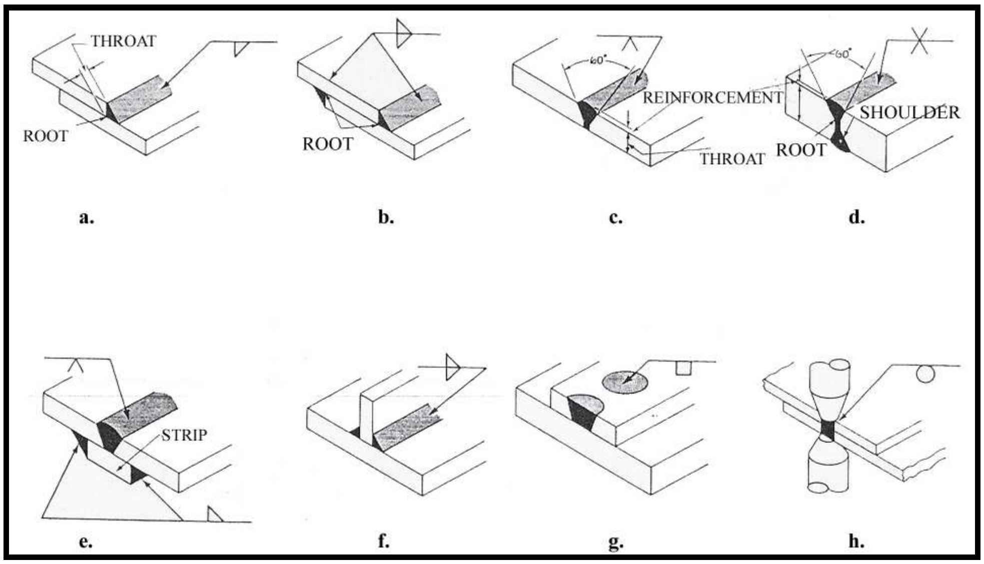


Figure III. 22 Types of welded joints: a) and b) Lap weld; c) and d) Groove weld; e) Single vee groove; f) Double fillet tee joint; g) Plug or rivet weld; h) Spot weld (Ellison, 1987:211).

Allen, (1999:358) claimed that welding offers a unique and valuable capability to the structural designer, it can join the members of a steel frame as if were a single piece (Fig. III.21). Welded connections are stronger than the members they join in resisting both shear and moment forces (Fig. III.22). Callender (1982:2.100) classified types of welding as fillet, groove and plug. Fillet welding is a triangular cross section joining two surfaces approximately at right angles to each other. The size of the fillet welding is determined by the length of the leg. The fillet weld is the most common type of weld used in structural work. Groove welding is made by depositing filler metal in a groove between two members to be jointed. Plug welding is made by a circular hole or elongated hole in one member of a lap joint, joining to the portion of the surface of the other member that is exposed through the hole.

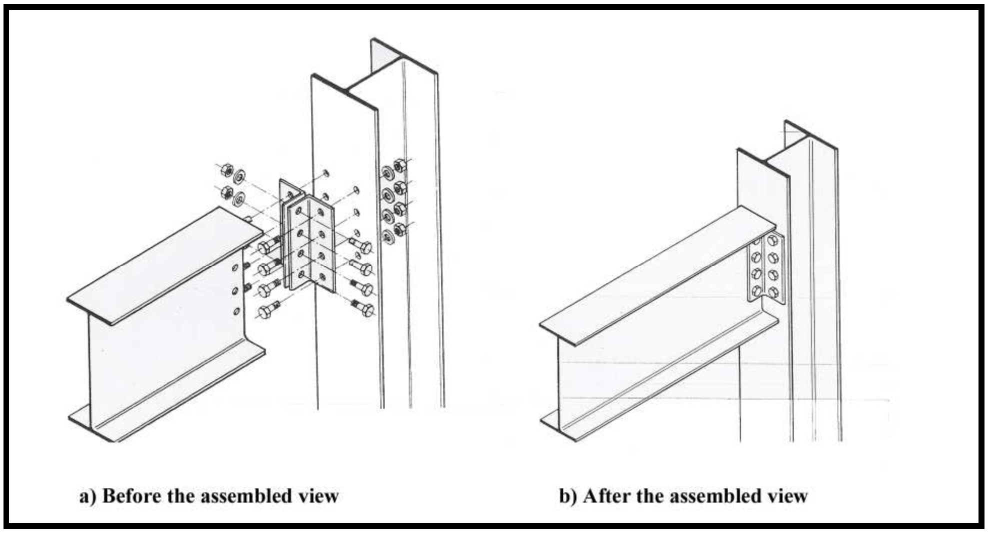


Figure III. 23 Exploded and assembled views of a framed, bolted beam to column flange connection, the size of the angles and the number of the bolts are decided by the magnitude of the load that the connection should transmit from beam to column (Allen, 1999:362).

Bolt connections requires bolts, nuts and washers and can be used in joining structural steel (Fig. III.23). Bolts are classified by strength grades (Eggen *et al.*,1995:43-44). Ellison (1987:212), suggested that joints provided by bolting or riveting plates or parts of members that lap over each other are called lap joints. Those formed by cutting the ends of two parts together and fastening them by bolts or rivets passing through a splice plate or connection plate are called butt joints. Allen (1999:357), suggested that bolts are classified as carbon steel bolts and high strength bolts. Carbon steel bolts have less installed cost than that of high strength bolts, carbon steel bolts are used in many structural joints where their lower strength is sufficient to carry necassary loads. High strength bolts are heat treated during manufacture to develop the necassary strength. They derive their connecting ability from their shear resistance.

In the connection of steel members the greatest problem is full contact bearing that, the stress on the contact area equals the stress in the member, thus full contanct is needed to transmit this stress from the member into the plate. For the tolerances of the connections, the area in contact may be significantly less than the area required to transmit the load (Owens, 1992:888). Ends have to be reasonably flat to enable the load to be transferred properly.

The researcher, claimed that connections both in timber and steel structures have a great influence on the efficiency and load bearing capacity of the whole structure. In connections combining two different materials, timber and steel can cause problems. Generally connections are made up of steel, and the connected pieces are made up of wood products. Since wood is an unisotrophic material, its resistance to loads in different directions should be calculated and placement of steel members should be done by increasing contact points in various directions. In timber construction, moisture content at erection is the same as that has to be reached in the service, and if the bolt holes permit the bolt movement, the tendency of timber structure to split will be reduced. Introduction of steel members in timber connections have increased the span lengths, load bearing capacity of the structure. Timber connections may be applied at the site, faults can be recovered with handwork. In steel structures connections also have influenced the efficiency of the structure. Generally steel connections need to be applied at factory, at site corrections may cause problems such as non full contact bearing, cutting or correcting the steel members by handwork is almost not efficient and applicable.

III.2.4 Cost

There are many factors to consider when comparing the budget of different construction systems including the complexity of the layout, site, workmanship experience, and relative material prices at the time of building. In the medium to long term, wood supply indicate a stable and growing supply in the western Europe, Australia and the U.S.A. . However, this price stability is questionable for materials such as steel, which consume considerable amounts of fossil fuels in their manufacture (Australian Government, Forest and Wood Products Research and Development Corporation, document on-line). The cost of a structure relatively to other building materials is probably one of the most difficult questions to answer. First, cost of materials, fabrication, erection are constantly changing and vary with geographic location. Second when one system is compared to another system, maybe the structural material cheaper may indicate great amount of volume unused, moreover to be heated and ventilated (Shaeffer, 1980:10). Wood is economical when compared to other materials because of it’s basic costs and the savings from other fields.

Transportation cost is lower and short erection time saves from labor (Oberg, 1963:94).

Foster, stated that costs of different forms of roof structure in different materials fulling the same requirements of the same span and loading, the difference is small. Cost of the structural frame may be as low as the 1/6 of the total cost, that the changing the material or system may have a very little effect on the total cost. For spans up to 30 m, the cheapest structure is truss and column frame. The space above eaves level is prevented by the trusses and over the wider spans of this type of structure encloses a considerable volume of space. For spans between 30 m and 45 m ‘umbrella’ construction or lattice girders can be used when economy is taken into consideration. Structures which are more closely related to the space which they enclose such as rigid frames or arch forms will be more expensive than deep beams, girders or trusses, but may compansate for this by reducing the volume of the building and the area of cladding and finishes. Rigid frames either square or arched although not always economical but because of the previous reason they are used. For spans between 45 m and 60 m, rigid steel frames with lattice structure are cheaper than solid web designed structures, for spans over 60 m or more, arch rib construction is likely to be the most economic choice. In suitable types of buildings advantage of timber with high strength to weight ratio can be taken into consideration. Bowstring trusses with laminated timber chords with a span to 70 m or more, lattice rigid frames over spans of 30 m, laminated timber bents froming rigid frames and laminated timber arches over spans of 60 m or more would be economical choices (1983:340-341). Shell and dome construction in laminated timber have comparatively very little weight when compared to reinforce concrete and steel.

In steel construction roof structures constructed from welded steel tubes are considerably lighter than similar structures constructed in ordinary steel sections and savings from cost is possible. The weight of a tubular framework per m² can be 25% to 40% less than that of a hot rolled section for spans up to 15 m (Foster, 1983:341).

Adrian (1983:544-545) claimed that steel is generally taken off and priced in units of weight. This is because a contractor purchases structural steel for a price per weight quantity. However weight is an indicative factor in deciding the cost, for purchasing labor and equipment cost which are mainly related with the number of steel sections and connections to be fabricated. Steel members are seldom custom designed or fabricated for a specific building, generally commonly available members are chosen. Unlike standard structural members which are purchased per weight, special designed members are priced on linear per linear foot basis.

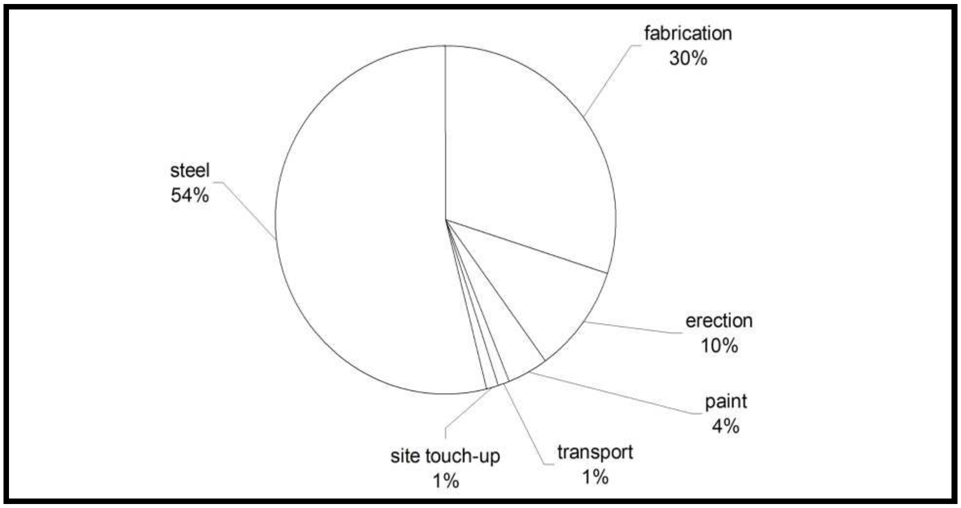


Figure III. 24 Cost breakdown of portal framed industrial steel structures (Owens 1992:913).

In order to minimize the cost in steel construction repetitions must be increased so that fabrication costs decrease (Owens,1992:912). The cost of steel is compound of raw steel, fabrication, painting, transport, erection and site painting (Fig. III.24, Fig. III.25). However in Turkey raw material costs are more than U.S.A., on the other hand workmanship costs are less than U.S.A.. In steel construction all steel kinds greater than A36 are more costly to buy from mills, but this not mean they are too expensive to use. Their advantage of greater strength and possibly simpler fabrication generally results in the most cost efficient construction. Nevertheless the demand for one grade of structural steel thatt is widely available and generally economical has made the A36 the most commonly used type (Merritt, 1982:8.3). When cost efficiency is considered, corrosion resistant steels A242 and A588 are not economical for ordinary usage. Their usage in buildings may be justified where steel is exposed or perhaps left unpainted.

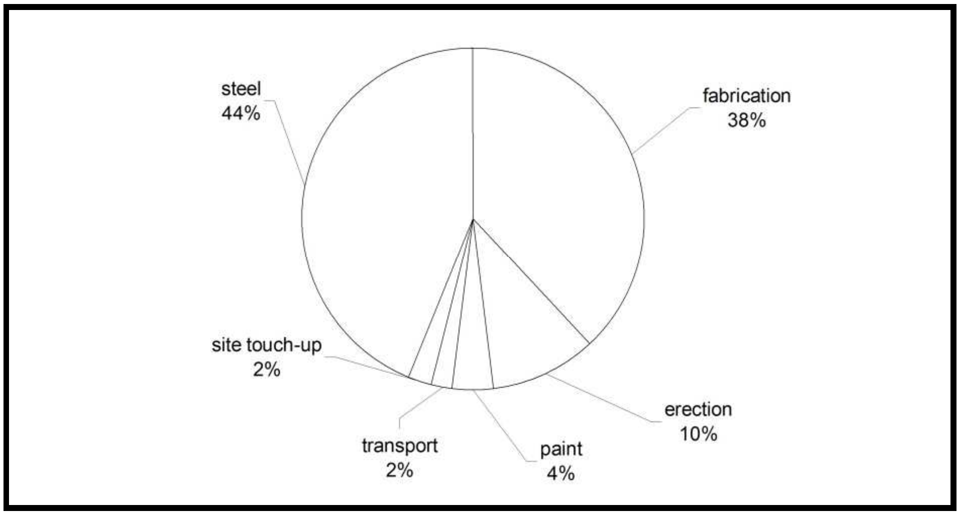


Figure III. 25 Cost breakdown of lattice steel structures in U.S.A. (Owens 1992:915).

According to the researcher, when timber and steel is compared, initial design and erection cost of timber is more than steel in Turkey; however this rate changes in the foreign countries acoording to the availibility of raw material and the amount of manufacturers. In Turkey since knowledge on structural timber is not wide; the demand for structural timber is less than steel. According to the demand, manufacturers and producers are not as much as steel producers; therefore cost rates are more than steel. Although timbers initial material cost is higher than steel; the cost of erection time, maintanence costs, transportation is less due to its low dead weight and ease in erection.

III.2.5 Maintanence

The choice of the structure is sometimes may be governed by the maintanence. Timber structures do not require protective coatings, particularly internally after erection and maintenance costs are therefore considerably less than for metal structures. The cost comprises the highest percent while the time of manufacture and assembly of the structure. After the erection, timber structures do not require further application or maintanence (Foster, 1983: 340). When lumber is used in exposed conditions it may attain a moisture content that may be open to decay and to insect attack. Moisture contents in excess of 20 % are recognized to make wood especially vulnerable to attack. Protective measures should be taken in order to give wood the necassary degree of resistance (Faherty, 1989: 2.38).

Steel lattice construction of all forms offers a large surface area for corrosion, and is difficult to paint. In highly corrosive atmospheres, it may be more economical to use alliminium alloy instead of steel in order to decrease maintanence costs, even though the cost of the structure itself may be higher than steel. Merritt claimed that ordinarily, steel corrodes in the rpesence of both oxygen and wateri but corrosion rarely takes place in the absence of either. For instance steel does not corrode in dry air, and corrosion may be negligible when the relative humudity is under 70 percent (1982:8.103). Galvanic corrosion takes place when dissimilar metal are connected together, noble metals such as copper and nickel should not be connected to steel with steel fasteners, since galvanic action destroyes the fasteners. When dissimilar metal are to be in contact, the surface should be insulated, painting is generally satisfactory.

According to Sperling, painting a timber structure with a couple of coats of aluminium wood primer followed by at least two coats of a modified oil paint containing zinc oxide can be much more satisfactory than treatment with clear varnish. It is essential to protect steel against corrosion in the humid tropics. Protection with a paint system based on red lead primer or by galvanising is adequate for structural components. Close to the sea a sprayed coating of aluminium itself protected by a paint system may be needed, but thick bituminous coating may be satisfactory and are much cheaper. Conditions in the dry tropics are rarely as corrosive as in the humid regions and lighter protective treatments would be suffice, but unprotected steel should not be used (Sperling, 17-19).

The researcher claimed that both timber and steel need maintanence. Steel oxides and annually needs painting, timber also requires painting and preservative treatment should be applicated. The main maintanence problem of timber is that, the humidity conditions should be controlled and should not be changed from design values.

III.3 Material Performance

Material performances are the characteristic properties of timber and steel, which do not depend on the form, structure or construction. These are constant properties, and valid at any part of the world. For example constructional performance may differ in some parts of the world depending on the avaibility of the material, application techniques, cost and transportation. Material performances are searched in the topics such as: a) environmental conciousness, b) fire resistance, c) moisture content, d) resistance to chemicals, e) heat expansion, f) acoustical insulation.

III.3.1 Environmental Conciousness

Of all building materials wood is unique because it is constantly being replenished by new growth. With an increasing awareness of the environmental impacts of human activities, homeowners are more often looking for building systems and designs which use little energy to manufacture and are built with sustainably produced materials. Timber has low embodied energy (the process energy requirement to produce the basic material) and wood is a net carbon absorber. Timber does grow on trees and well-managed plantations and forests can produce timber on a continuous basis, with minimal adverse effects on soil and water values. Wild life values of natural forests are protected by setting large areas of forests aside in reserves.

Table III. 10 The amount of energy to produce wood and steel (Design and Construction Guide, APA, The Enginereed Wood Association, document on-line).

	Percent of production	**Percent of energy used**
wood	47	4
steel	23	48

A direct comparison quoted in The Structural Engineer No 24, December 1989 was of a light gauge cold rolled steel purlin compared with a 30 x 5 cm rough sawn joist of the same stiffness which showed the steel member to have 19 times more energy cost. If the timber joist was planed and treated, the steel member still required 5 times the energy cost (Timber Research and Development Association, document on-line).

According to Taylor, constraints are likely to grow upon the materials from consideration energy input, raw material resources and environmental matters. Three very likely constraints are: a) Material conservation: there may be increased taxation on raw materials, like oil. b) Environmental matters: the levels of burning carbon dioxide being released by burning of fosil fuels. Taxation on such fuels would increase the use of low energy materials. c) Energy saving generally: if energy saving is seen as a world resource and taxed accordingly, low energy materials would be encouraged (1991:47). In the U.K. in the mid 1980s energy consumption to produce timber and steel is 4 and 35 GJ/tonne respectively (Table III.10).

Table III. 11 Primary energy to produce finished item (Timber Research and Development Association, document on-line).

	Primary energy used to produce finished item (kwh/m^3)
steel	1.02 x 1010
timber	695 (includes transport energy factor)

Taylor suggests that steel represents the main energy drain in construction industry, timber would be much more efficient, though there are resource implications. Timber with its low energy requirement and low density, would seem an attractive choice for the future (Table III.11). Supplies are currently being destroyed at an alarming rate, and only government action on a worldwide scale can change this trend (1991:48). Altenpohl, suggested that energy is an important cost factor in the total production cost of finished steel products (1980:42). The highest waste of energy in steel plants is in the coke making operation where red hot coke is quenched with water. It must be remembered that energy cost will continue to increase in every country, and therefore, will become a more serious problem as the time passes.

According to Altenpohl, air pollution by dust and fumes from steel plants is one of the most expensive environmental control problems. Coke ovens are the source of gaseous pollutants and dust. In mining problems concern mainly on disposal of solid waste, elimination of dust, reclamation of process water and recultivation of mined out areas (1980:43). The amount of energy to produce the finished item is compound of different steps, the amount of energy in the finished item is 20-25% (Table III.12).

Altenpohl (1980:115) considered that the harvesting of wood and the production of engineering materials from wood presents no difficult environmental problems, which also holds true for workers hygiene.

Table III. 12 Cost structure of finished steel products for the U.S.A. (Altenpohl, 1980:41).

	% of total cost
Ore	20
Energy (coal, fuel etc.9	20-25
Wages and salaries	35-45
Capital charges (inc. Depreciation)	6-10

According to the researcher timber has a positive value in environmental conciousness, that the energy needed to produce timber is 1:12 of steel. However timber is acquired from trees and cutting trees damages the natural balance; by tree farms the problem of yearly cut and renoval would be solved. Energy consumption is a very important topic due to world's natural energy resources will be exhausted in the future. Except for the energy consumption, steel production causes burning of oxygen and carbondioxide is acquired, that steel production pollutes environment. During production of timber no environmental pollution occurs, hence trees convert carbondioxide to oxygen too.

III.3.2 Fire Resistance

According to hazard characteristics wooden structures are often misregarded as being less safe than steel or reinforced concrete structures. Wood burns and steel or concrete does not. In fact during a fire unprotected steel or reinforced concrete structures rapidly lose their strength and collapse while wooden structures lose strength slowly (Fig. III.26). Due to wood's lower thermal conductivity, timber is weakened by the increasing temperature considerably lower than the steel member. In 20 minutes when the fire reaches 800ºC, the timber having the dimensions 5x10 cm in cross section obtains more than 40% of its original strength and the steel member, a mere 10%. When the size of the wooden element increases, its fire resistance rises (Karlsen, 1989:52).

Fire resistance[14] is defined as the ability of an element to carry on performing a building function in spite of being exposed to a fully developed fire. It is, thus a property of the elements of building construction, not materials. In consequence, to state that 12 mm plywood has 'x' minutes fire resistance is incorrect. It may contribute 'x' minutes to the fire resistance of a load bearing stud wall but several things, particularly its support and fixings, can alter this contribution.

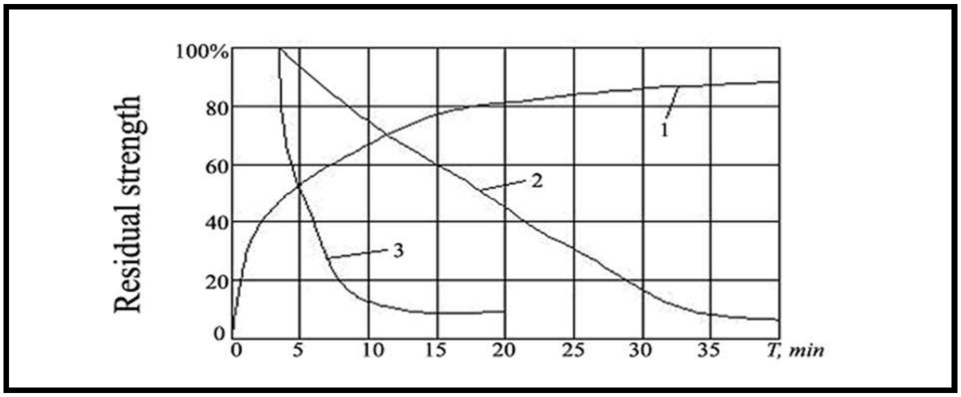


Figure III. 26 Strength vs fire temperature:1-standard curve for temperature variation during fire 2-residual strength of a 5x10 cm wooden member 3-residual strength of a steel member (Karlsen, 1989:51).

Glued laminated timber and solid logs are the most fire resistant (Fig. III.27). In a log converted to boards the burning surface heat one to another, so that an increase to focal points of combustion is acquired. The mutual heating is even more effective in a bundle of splinters or chips . When wood dust and air is mixed it is suggested as explosive (Karlsen, 1989:51).

Different timbers char at varying rates[15], largely as a function of their density with the higher density timbers charring more slowly. Certain of the denser hardwoods (>650kg/m³) used for structural purposes merit rates of 1.5 cm in 30 minutes, eg,

[14] The appropriate tests for fire resistance are contained in BS 476: Parts 20-24 : 1987 test methods and criteria for the fire resistance of elements of building construction and ISO Standard 834, the two being essentially harmonised. Test methods have been defined for most common elements - walls, floors, doors, glazing, beams, columns, etc. (Timber Research and Development Association, document on-line).

[15] For structural timbers listed in the code of practice for the design of structural timber, BS 5268-2, this rate of depletion is taken as 20 mm in 30 minutes from each exposed face.

keruing, teak, greenheart, jarrah. Timbers of lower density will char more quickly eg. Western red cedar is quoted as 2.5 cm in 30 minutes (Timber Research and Development Association, document on-line). Wood with a thickness of 9.5 mm ignites easily and burn, but timbers about 1.5 cm thick will char in depth and this prevents the rapid combustion of the wood beneath (Foster, 1983:376).

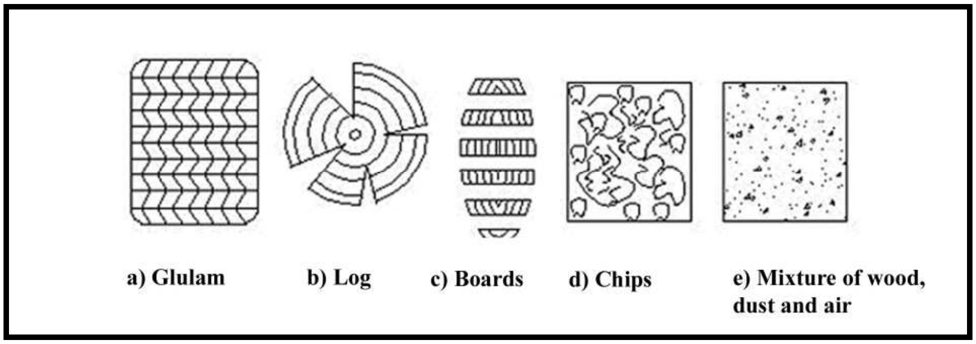


Figure III. 27 Fire resistance of various wood members and materials. Glulam and log are the most fire resistant, in boards and chips fire resistance decrease, the mixture of wood dust and air is regarded as explosive (Karlsen, 1989:51).

Glulam beams 9 cm thick and over usually resists a fire 30 minutes without modification (Fig. III.28). Longer periods will probably lead to an increase in thickness although this may be partially offset by a reduction in depth (Glued Laminated Timber Association, document on-line). Glulam has a high and predictable performance in fire because timber chars at a slow and known rate: 4 cm per hour for European white wood. More importantly, it retains its structural integrity. Class 0 and 1 face spread of flame, can usually be achieved for glulam members by the application of a proprietary treatment on site after the building is dry and watertight. Care needs to be taken to ensure compatibility between specified treatments.

The high thermal insulation characteristics of timber, and the charcoal layer that forms on it, both ensure that the interior of a fire exposed member remains cool and structurally sound over the design period. A glulam member also behaves as a single unit throughout its exposure to fire because of the high resistance of laminating adhesives to fire temperatures. This reliability of glulam's performance in fire means that it is possible to predict the inherent fire resistance of a particular component, or to design a component to resist fire for a specified period without the need of expensive testing (Glued Laminated Timber Association, document on-line).

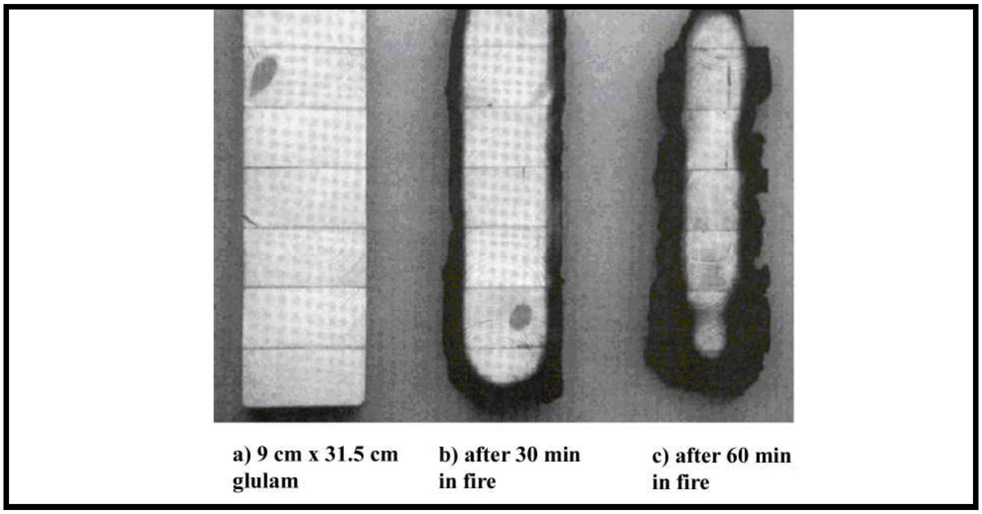


Figure III. 28 The effect of fire at 30 and 60 minutes on a glulam piece, dimensions of the beam is 9 cm by 31.5 cm before fire (Glued Laminated Timber Association, Technical Data Sheet, document on-line).

Combustion is the combination of the flammable components of wood with the oxygen of the air to produce heat, smoke and sometimes flame. Wood may ignite when heated for a short time to a temperature of about 250ºC. Initially the increasing temperature causes moisture to evaporate from the wood, if some moisture is left at wood temperature remains at 100ºC. When the temperature increases between 150ºC to 210ºC, wood dries, changes its colour, and chars. Combustion rate depends on the rate of feed and atmopsheric oxygen amounts on the surface and avaibility of the surfaces of adjacent wood members heat one another (Karlsen,1989:50).

There is negligible difference in the mechanical properties of the timber under charred layer. Any thermal expansion is compensated by drying shrinkage; that means strength and stiffness are substantially unimpaired and may be increased. Glued laminated timber had the same fire resistance as solid timber. These characteristics enable the fire resistance of larger sections (Blyth, 1989:169). Solid and laminated wood have classification of B1 after they have been chemically treated for fire protection

(Table III.13). Many timber buildings achive fire resistance classes F30 to F60[16] without any additional measures (Götz *et al.*,1989:42). According to Sperling, the strength of timber decreases by about 3 to 5 % for every 12.2 ºC (10ºF) rise in temperature (Sperling, undated, 18).

Table III. 13 Building materials are classified due to behaviour in fire (Götz *et al.,* 1989:42).

Class of Building Material	**Description**
A	Non-combustible Material
A1	
A2	
B	Combustible Material
B1	Materials of low combustibility
B2	Normally combustible materials
B3	Highly combustible materials

Wood is highly resistant to fire but in the end it does burn. Part of the strength of wood is lost in exposure to high temperatures, and the gone strength is not gained again. The loss from oxidation in metal is much more and the loss in strength of steel from rusting is permanent (Oberg, 1963:92). Because of wood burns very slowly in large cross sections, it is excellent against fire damage (Fig. III.29). Steel beams collapse in fire, but wood beams support not only their own weight but also other building material's weight such as steel. Steel's stiffness is reduced and this is calculated with a reduction factor (Fig. III.30). Steel collapses before wood is seriously damaged. It is estimated that steel loses half of its tensile strength at 1100º and it does not support the dead weight of the structure over 1700º. In flash fires that include gasoline, lacquers, or smilar products temperatures may rise immediately to 2000º -2500º F in that case metal may fail before the fire begins. Wood burns approximately in a rate of 2.5 cm in 33 minutes. In large sections a considerably time must pass to weaken the structure (Oberg, 1963:92).

[16] Fire resistance classes correspond to the duration of the functional capability of a structural members under fire, F30 do not mean resistant to fire, it is defined as only a retardant. As the F scale increases (Götz,1989:42):

F30	Fire-retardant
F60	Fire-resistant
F90	Fire-resistant
F120	
F180	

Figure III. 29 Glulam portal frames maintained their integrity in a magnesium-fed fire. The metal walls and roof of the building melted and collapsed (Glued Laminated Timber Association Technical Data Sheet, document on-line).

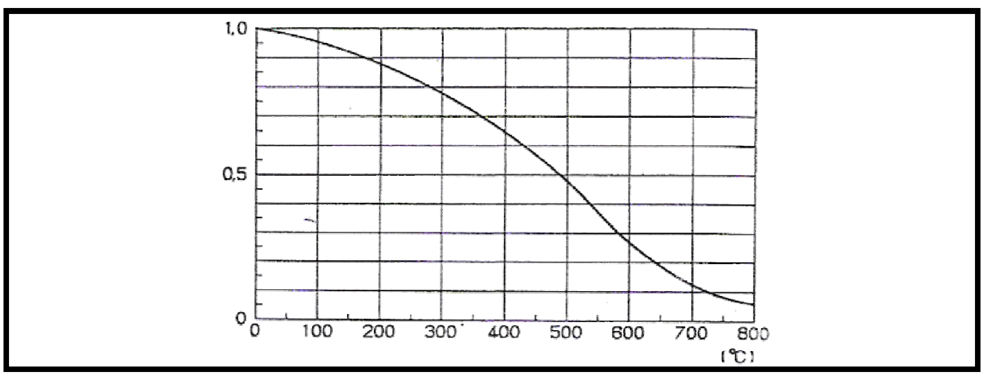


Figure III. 30 Reduction factor for steel's stifness at high temperatures (Eggen *et al.*, 1995:36).

Steel loses its strength and stiffness at high temperatures (Fig. III.31). At 400ºC / 725ºF steel gains its 2/3 of its original strength and rigidity and at 500ºC/900ºF it retains only about half. As steel structures are usually light, steel structures can be heated very quickly. The temperature of fire during its flame is normally 900ºC-1000ºC (Eggen *et al.*, 1995: 35-36). Steel loses its strength and rigidity above a temperature of 299ºC and at 427º-482ºC, a strength loss of 80% results. Together with the expansion of steel, in the early stages of fire unprotected steel will buckle, bend and expand (Foster, 1983:375).

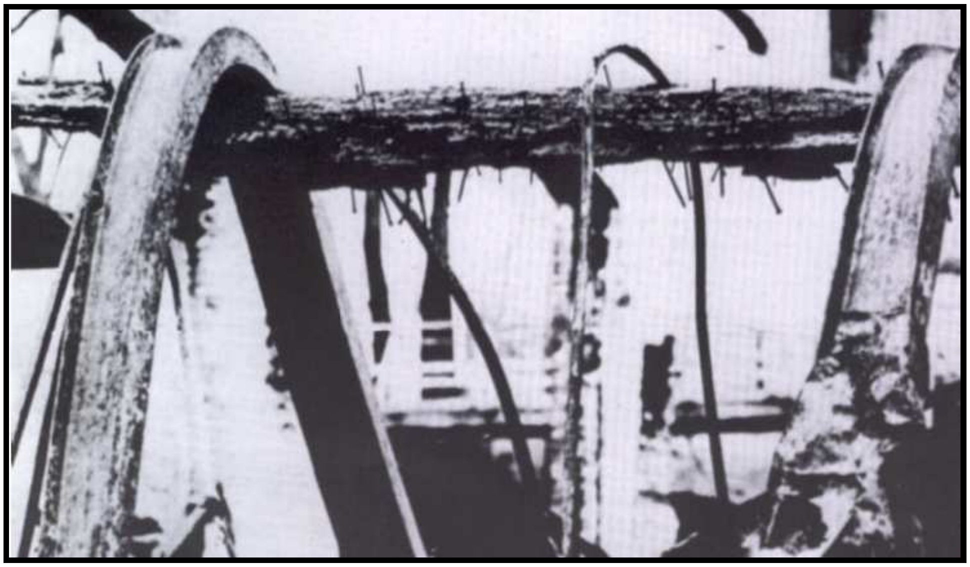


Figure III. 31 Steel beams have melted and collapsed over charred glulam beam which, despite heavy damage, remains in place (Glued Laminated Timber Association, document on-line).

Allen (1999:389) suggested that fireproofing of steel was done originally by brick, masonry or poured concrete. These heavy encasements were effective by absorbing the heat into their mass, but their weight added a considerable weigth to the structure. The search for lighter fireproofing materials continued with thin enclosure of metal lath and plaster around steel members. Contemporary way of fireproofing steel members is enclosures made up of boards or slabs of gypsium or other fire-resistive materials (Fig. III.32). These materials are fastened around steel shapes and in case of gypsium it can also serve as a finish material. These products are avaible in densities of 190 to 640 kg/m³. Lighter materials are considered as fragile and must be covered with a finish material. Denser materials are considered as durable and attractive. Spray on materials act primarily by insulating the steel from high temperatures for long periods of time and they are the most cost efficient form of fireproofing. Intumescent mastics and paints are the latest generation in fireproofing of steel members, they allow steel structural members to remain exposed to view in situations of low to moderate fire risk. They expand when exposed to fire to form a stable char that insulated steel from heat of fire for varying lengths of time, depending on the thickness of the coating.

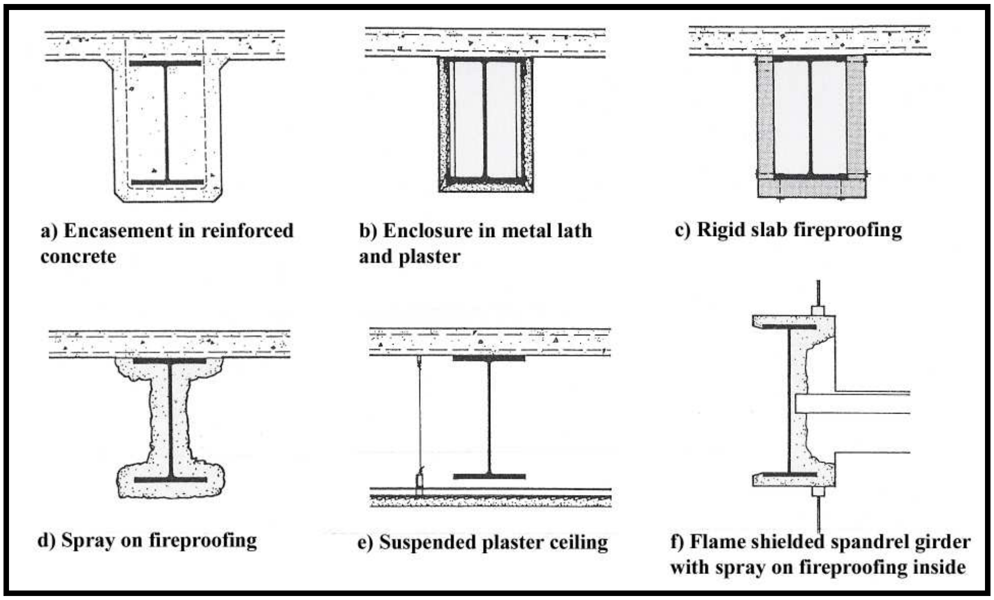


Figure III. 32 Some methods of fireproofing steel beams and girders (Allen, 1999:391).

The researcher suggested that both timber and steel is affected from fire, timber burns, steel melts. While timber burns, a layer of charchoal occurs which protects timber from rapid burn because it disables oxygen. Both two materials are affected from fire, but the critical point is that the necassary time for people to escape from fire should be acquired, which is mainly considered as half an hour. Building codes generally limit the exposed usage of steel, it should be insulated by fire proof materials which causes an additional cost, however timber structures can be used exposed without any extra encasement for fireproofing. Keeping material from fire by a preventive coat is the main point in the fire resistance. After the fire occurs, building would be damaged, the critical point is not to protect material from fire, the important point is that to keep the material from the effects of fire in order to get the time for people to escape.

III.3.3 Moisture Content

When wood is subjected to moisture, the cells absorb the water and the sizes of the dimensions can change. Unequal shrinkage of improperly seasoned wood can be the reason of unequal settlement in a wooden structure. Warping may cause problems in the structure and in the material before it is placed in the building. Careful storage and design to provide against unequal shrinkage and exposure may decrease the problems (Oberg, 1963). Swelling and shrinkage are reversible and change due to different directions of the yearly growth rings and the grain of wood. Dimensional changes are the mostly in tangentially to the yearly rings, they are half as large radially, and they are negligible in the direction of the grain longitudinally (Götz *et al.*,1989:26). Green wood has moisture both in the cell wall and in the cell cavity itself. The point where the cell cavity is empty but the walls are full is called the fiber saturation point. For most species this value is usually 30%. Change over moisture content over FSP has no critical effect on wood's properties, all shrinkage occurs as the wood dries from 30% to a lower MC[17] value (Halperin, 1994:12). According to BS 5268, moisture content recommendations vary between 14% and 24% (Table III.14).

Table III. 14 Moisture Content recommendations in BS 5268 Part 2 (Timber Research and Development Association, document on-line).

	Avarage moisture content in service (%)	**Maximum moisture content at time of erection (%)**
External uses, fully exposed	18 or more	
Covered and generally unheated	18	24
Covered and generally heated	16	21
Internal in continuously heated building	14	19

When wood is faced to moist conditions, it is under risk of attacks by fungis, molds, and insects. While these can be controlled to a certain extent, continued exposure

[17] The moisture content (MC) is measured as the percentage of water to the dry weight of wood:

$$MC = \frac{\text{moisture weight} - \text{dry weight}}{\text{dry weight}} \times 100 \text{ percent}$$

cannot be permitted without damage to the wood. Proper drainage around buildings and proper treatment of lumber is found quite effective (Oberg, 1963).

The moisture content in a living tree can be as high as 200 %. The avarage moisture content that lumber assumes in service is known as the equilibrium moisture content (EMC). Due to atmospheric conditions, the EMC of structural framing lumber ranges between 7% and 14 % (Breyer, 1988:109). Dry lumber is defined as lumber having a moisture of 19% or less. Lumber 20 % and over in moisture content is defined as unseasoned or green lumber. Approximate percentage increase (or decrease) in mechanical properties for 1% decrease (or increase) in moisture content. Existing standards require that the experiments should be determined at 12% moisture content. Swelling and shrinkage cause dimensional changes due to variations in moisture content. A rise in moisture content causes swelling, a fall causes shrinkage (Götz *et al.,* 1989:26).

When proper construction details are used and other good design and construction practices are followed, wood is a permanent construction material. Moisture barriers, flashings and other protective features should be used to prevent moisture or free water being trapped. Preservative treatments are recommended when wood is fully exposed to the weather without roof cover. Materials should be protected during construction. Arch and column bases should not be embedded below finished concrete floor levels and they should be elevated a minimum of 2.5 cm above the concrete floor level if there is potential for wetting of the floor (American Institute of Timber Construction, document on-line).

Especially in the design of connections, effort should be made to avoid splitting the member due to expansion and contraction of the wood. Consideration must be given to swelling and shrinking due to moisture content changes in service, similar to the consideration given to details in metal construction that must accommodate the expanding and contracting metal due to changes in temperature. Because wood swells and shrinks (primarily perpendicular to its grain) due to moisture content changes, connections should not restrain this movement. Even in covered structures, large laminated timbers may shrink after installation due to moisture loss from low relative humidity conditions. Long rows of bolts perpendicular to grain fastened to a single cover plate should be avoided. Although relatively dry at time of manufacture, glued laminated timber can still shrink to reach equilibrium moisture content in service. When possible, designers should avoid joint details that could loosen in service due to wood shrinking or that could cause problems when wood expands due to increased moisture content. Machine bolts should be used rather than lag bolts whenever possible. When lag bolts are used, correct lead hole sizes are important. Connections should be detailed to avoid loading lag bolts in withdrawal whenever possible (American Institute of Timber Construction, document on-line).

When the moisture content of wood, increases from 0 to fiber saturation point which is about 30%, its strength and long time strength falls, deformability rises, and its modulus of elasticity falls. The impact strength and the parallel to grain tensile strength of wood are the least affected properties of wood. Other strength properties are considerably sensitive changes, i.e. an increase of 1% in moisture content results in a decrease of 3 to 5 % in strength. When the moisture content rises beyond the fibre saturation point, no further fall in strength occurs (Karlsen,1989:32).

The researcher denoted that moisture content should be kept in a static level and it should be controlled according to the design service level. Timber decays due to moisture and organisms, steel does not have such a problem of decaying. In order to protect timber from decaying precautions should be taken by chemical treatments which results in an extra cost for the structure. In steel construction moisture also effects steel, steel rusts and decays at the end.

III.3.4 Resistance to Chemicals

Wood differently responds to chemical attacks. Hydrofluoric, prosphoric, and dilute hydrochloric acids have no effect on wood at atmospheric temperatures, however sulphuric acid, nitric acid, destroy wood even at low temperatures. At atmospheric temperatures organic acids like acetic, formic, citric will not weaken wood. On the other hand hot solutions of organic acids will destroy wood (Karlsen, 1989:16).

Timber generally has a remarkable resistance to chemical attack in polluted atmospheres and to contact with chemical solutions, glulam is also equally resistant. A recent application is in the construction of barns to store salt for deciding roads. The synthetic adhesives used in bonding glulam are equally resistant to most chemicals. However, glulam is not totally immune. Oxidising agents, sulphides and alkalis, for example, will exert a 'pulping' action on the timber and lead to loss of fibre and strength. But these are unusual agents in most service environments. Everett, considered that compared with metals, wood has good resistance to alkalis and weak acids (1986:51).

Wood should be protected by appropriate chemical treatments. Chemical protection should penetrate to the degree of the wood exposure. Its efficiency depend on the type of the treatment, the quality of the chemicals applied and their distribution within the wood. Protective treatment is essentially applied to wood after all other fabrication work has been completed. If later work such as drilling is necassary exposed surfaces have to be treated again.

In steel construction corrosion or rusting is an electromechanical process that deteriotes metals like nails, screws when is exposed to water and oxygen a thin layer of corrosion occurs on its surface. In the atmosphere practically no corrosion occurs when there is under 60% relative humidity. From a 70% relative humidity, corrosion occurs markedly with increasing humidity. A polluted industrial environment, and sea water also causes corrosion in steel (Eggen *et al.*,1995:35). The basic factors that effect the rate of corrosion in steel in air are: a) Type and amount of pollution; b) Time of wetness due to rainfall, condensation etc.; c) Temperature.

The researcher suggested that when comparing timber and steel in terms of chemical destruction, it is seen that timber is effected from less chemicals than steel. Many of the acids and chemicals effect steels body, however timber is effected from sulphuric acid, nitric acid and hot solutions of organic acids. Steel is corroded at a humidity level at about 70%, moreover atmospheric conditions destroy the body of steel.

III.3.5 Heat Expansion

Wood has a high resistance to heat flow. In wood thermal conductivity is same in radial or the tangential direction, but it is 2 or 3 times greater in the longitudinal direction. Thermal conductivity of wood is about one four-hundredth that of steel (Faherty, 1989: 8-9) (Table III.15). Thermal insulation is however a rarely factor that effects the choice of the roof type, because methods of providing the thermal insulation is applicaple to all forms of roof structures (Foster, 1983:164).

Table III. 15 Properties of timber and steel which affect fire performance (Taylor, 1991:36).

	Density **kg/m^3**	**Specific Heat** **J/kg°C**	**Thermal Conductivity** **W/m°C**	**Thermal Diffusivity** **mm^2/s**	**Cefficient of Themal Expansion** **/°Cx10^{-6}**
Wood	500	1200	0.14	0.23	3
Steel	7800	480	84	22	11

Specific heat capacity is the number of joules required to raise the temperature of 1 kg of the material by 1ºC. Thermal conductivity is the ability of the material to transmit heat by conduction. Diffusivity describes the ability of the material to transmit heat in fire, rather than become hot locally. For example steel beams heat up in fire rapidly rather than diffuse heat (1991:36-39).

The coefficient of thermal expansion parallel to the grain is a mere 1/7 to 1/10 of that across the grain and is 1/2 to 1/3 of that for steel. Little thermal expansion parallel to the grain leads to design without expansion joints in wooden structures (Karlsen, 1989:18). Moreover thermal coefficient of expansion for wood parallel to the grain may be as little as one 1/300 that for steel. Coefficients perpendicular to grain may be on the order of 5 to 10 times that parallel to the grain (Faherty, 1989:8).

Steel and concrete expands under different temperatures and this must be considered in measurements. On the contrary, wood has no significant change with temperature and this has no effect on the whole structure (Oberg, 1963). At sub-zero temperatures water contained in green wood becomes ice, and the compressive, bending, shearing increases. On the other hand, frozen wood is more brittle and less resistant to impact and dynamic bending (Karlsen, 1989:33).

The researcher mentioned that heat does not have a significant effect on timber, steel is mainly effected from heat changes. High heat levels like 100 ºC or more speeds timber to ignite, timber burns at 250 ºC or more. Heat speeds the burning period of timber, but timber do not expand like steel under heat changes. The changes occur due to moisture change, that it can be concluded that when the heat increases if the moisture contnent is more than the design level, the amount of moisture decreases in high heat temperatures. Long span dome structures that are fabricated with steel, cause a great change which cause noise due to the movement of the members.

III.3.6 Acoustical Insulation

Wood has an ability to dampen the vibrations. Therefore wood is preferred for structural components where sound vibration is undesirable (Faherty, 1989:10). Sound transmission ratings are closely aligned with fire endurance ratings for assemblies. This is because flame and sound penetrations follow similar paths of least resistance. Control of sound transmission is particularly important in post frame buildings used for commercial construction, such as offices or churches. Sound striking a wall or ceiling surface is transmitted through the air in the wall or ceiling cavity. It then strikes the opposite wall surface, causing it to vibrate and transmit the sound into the adjoining room. Sound is also transmitted through any openings into the room, such as air ducts, electrical outlets, window openings, and doors. This is airborne sound transmission. STC[18] method of rating airborne sound evaluates the comfort level of a particular living space. The higher the STC, the better the airborne noise control

[18] Sound Transmission Classes, adapted from Post Frame Construction Guide:

STC rating	Privacy Afforded
25	Normal speech easily understood
30	Normal speech audible but not intelligible
35	Loud speech audible and fairly understandable
40	Loud speech barely audible but not intelligible
45	Loud speech barely audible
50	Shouting barely audible
55	Shouting not audible

performance of the structure. An STC of 50 or above is generally considered a good airborne noise control rating.

Impact sound transmission occurs when a structural element is vibrated by direct impact, for example, by someone walking. The vibrating surface generates sound waves on both sides of the element. The Impact Insulation Class (IIC) rates the impact sound transmission performance of an assembly. Higher IIC values indicate better performance, with an IIC of 55 required for good impact noise control (Post Frame Construction Guide, document on-line).

According to the researcher, when the structure is exposed the interior sound may cause a disturbing effect. Mainly timber does not vibrate or causes echo effect; however steel reflects sound waves and steel elements which are used much can cause a disturbing effect. In that case acoustical dampers and insulation should be provided.

III.4 Timber Roof Structures in Turkey

Besides many advantages of timber, it is not widely used in Turkey due to high cost rates, manufacturing and the deficiencies in the production. People are not familiar with timber although it has a construtcion heritage. In the current situation of Turkey, because of the economic limitations, reinforced concrete is generally used without questioning its appropriateness. Reinforced concrete is easily obtained and cast by local contractors. Moreover when a wide span is needed steel is applicated as a structural material. Initial costs of the structural timber is higher than steel and concrete, when total workmanship, transportation, date of erection and dead weight compared, structural timber could be a wise decision. Especially after the 17 th August eathquake, timber buildings are being studied; moreover architects and engineers concentrate on to design lightweight structural systems. Besides many advantages of timber roof structures, there are deficiencies in the production, application and marketing. The greatest problem is the lack of information and knowledge in the market.

III.4.1 Manufacture

In Turkey, serious problems of producing high quality structural timber is within drying period of timber. New investments should be done in order to increase the quality of the material properties. Raw material is easier to be obtained than other structural materials, because the investment for the production period. In order to obtain high quality and cheap raw material, tree farms should be introduced. The responsibility is on the government, as soon as the necessary codes, laws and regulations should be formed. If better forest management, including more efficient logging operations, selection of trees for cutting, and the development of tree farms; the growth of new wood may come to exceed the yearly removal (Oberg, 1963:90). General trend in the western countries is the have tree farms, in order to obtain yearly removal of trees. On the contrary in Turkey, there is no such application, governmental forests can be hired in order to have tree farms.

During the research two companies[19] have been visited in Ankara. Although both of the firms have the technology for wide span roof structure systems, the greatest demand is for timber frame construction for domestic uses. Structural timber is covered with gypsium boards for fire precaution, timber is not used as exposed. One of the firms have a factory in Etimesgut, and a sample house have been built. The roof structure of the house is completely covered with gypsium board that disables to view the structural timber. Manufacture is so much bounded with demand, increasing demand would rise the quality of manufacturing and decrease costs.

III.4.2 Application and Marketing

There are improperties in the application and marketing of structural timber in Turkey. Application problems indicate designing stage, erection and transportation. The roof structure of the Darüşşafaka Çetin Berkmen Sports Center[20] is a space frame composed of structural timber. At an interview[21] it is declared that an engineering

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[19] Companies visited in Ankara are Nascor Turk in Etimesgut and Konkur in Sincan.
[20] Architect: Ersen Gürsel
[21] Günümüz Koşullarında Mimarın Strüktür Bilgisi (Architect's Structure Knowledge in Today's Conditions), *Mimarist*.

company was not found which can perform the analysis of space frame with structural timber, during the preliminary design calculations (2002:56-66). It is suggested that the engineering companies were mainly specialized on structural steel, the engineers refused to perform structural calculations of the space frame design with structural timber. Only one engineer, claimed that he could carry out the necassary calculations. The lack of knowledge to perform calculations for designing period is a critical point for applicating new structures with timber. Generally timber structures are designed and erected by professional firms which are the dealer of foreign firms that is specialized in structural timber design. The calculations are done by special softwares and still there is a deficiency in the calculations of the engineering firms due to lack of applications.

Figure III. 33 Elementary school project at Kocaeli[22] (Oran Mimarlık, document on-line).

There are a few firms in Turkey that use structural timber for spanning a medium or wide span roof structures: Oran Mimarlık, Hemel, Nascor, Konkur are the rare firms that manufacture structural timber and applicate. During the interviews with the firms, it has been seen that mainly application problems are caused by workmanship. Workers may sometimes do not applicate properly the erection manual of timber structure, sometimes in the connections necassary nail gangs or details are not fixed adequately.

[22] Architects: Köksal Anadol, Ersin Arıoğlu, Boğaçhan Dündaralp, Vedat Tokyay; Engineer: Tamer Ergenekon; Location: Adapazarı; Date:2000.

Figure III. 34 Elementary school project at Kocaeli (Oran Mimarlık, document on-line).

During marketing due to lack of information on timber, firms have problems to sell the end product. Tokyay (1998:119) pointed out that, if structural timber is indicated in the program of architecture and engineering faculties, young architects and engineeres would leave the negative prejudice on timber. Because timber has an image that it is a weak material for structural purposes and fire performance of timber is low or not known. Generally there is an attempt to design a medium or wide span by steel, due to its high strength property. If necassary precautions are taken, structural timber may be introduced in contemporary Turkish architecture. There are a few contemporary examples of structural timber that has been applicated in Turkey. The general trend is to have an assignment with a foreign firm, and have a vendor contract. Some of the rare examples will be introduced.

For the elemantary school project at Kocaeli structural timber is designed after 17 August earthquake. Ministry of Education requested a school project that was eqarthquake resistant and easily demountable (Fig. III.33, Fig. III.34). Anadol and Tokyay (2000) stated that the structural system was especially designed with timber due to its 45 min. fire performance, 7 days erection time, acoustical performance, energy efficiency and environmental conciousness. The structure is composed of glued laminated beams. Six school projects constructed in Kocaeli Değirmendere, Derince, Bekirpaşa, Adapazarı, Ferizli, Yalova.

Figure III. 35 Dome structure for the indoor swimming pool of Silyum Hotel[23] (Oran Mimarlık, document on-line).

The dome structure at Antalya Belek, requested for Silyum Hotel's indoor swimming pool, was designed with linear timber beams and the dome was covered with glass and the diameter of the dome is 22 m (Fig. III.35). The glued laminated timber truss at Antalya Lares Hotel for indoor swimming pool spans an area of 20 m x 15 m. Main span structure is designed with timber trusses, and secondary trapezoidal beams. Diagonal bracing is done with galvanized steel bars (Fig. III.36). The roof structure for the Aytek indoor swimming pool is composed of glued laminated vaults, with a span of 12 m and a length of 30 m (Fig. III.37).

The researcher claimed that the structural roof applications are limited in number and only large cities and resorts of Turkey. Roof structures with structural timber are generally designed for swimming halls and hotel shelters. Possible reasons are glulam's aesthetic appeal, its durability against chloride and chemicals at the swimming pools. Timber should also be considered as an alternative when a long span

[23] Architects: Oran Mimarlık (Oran Architecture Co.); Location: Antalya, Belek; Date:2000.

roof structure for sport center, concert hall or other type of buildings are needed. In the contemporary applications roofs spans range between 8 m and 30 m, longer spans have not been enclosed with structural timber. Due to the advantages and the few applicating fims are specialized in glulam, applications are assembled with glulam, solid timber has not been used as structural material in contemporary applications.

Figure III. 36 Roof structure for the indoor swimming pool of Lares Hotel[24] with timber trusses (Oran Mimarlık, document on-line).

Figure III. 37 Roof structure for the indoor swimming pool of Aytek indoor swimming pool[25] with timber vaults (Oran Mimarlık, document on-line).

[24] Architects: ibid; Location: ibid; Date:2002.
[25] Architects: Oran Mimarlık (Oran Architecture Co.); Location: İstanbul, Kumburgaz Date:2003.

Figure III. 38 Büyükada Rum Orphanage[26] is a building that have a totally timber structure (Adalar Governership, document on-line).

Figure III. 39 Kasaba Mahmutbey Mosque[27] has a timber structure without any nail connection (Tatil Dünyası, document on-line; Kastamonu Governership, document on-line).

However there are successfull examples of long span structural timber in the traditional Turkish architecture especially in the mosque design; timber tradition has not been continued in the contemporary examples. There are totally buildings constructed the whole structure with timber, Büyükada Rum Orphanage is the greatest timber building of Europe, has 6 floors with a 24 m height, 35 m width and 101 m length (Fig. III.38). The building includes a theather saloon, restaurant and other wide spaces; but it is left ruined. Since the tradition does not continue from the past to the future examples, timber structures would not develop. Taking the technology from foreign firms is not a rational choice to develop the technology. Mahmutbey Mosque is one of the oldest timber mosques in Turkey (Fig. III.39).

[26] Architect: Alexandre Vallaury, Date:1898-1899.
[27] Location: Kastamonu, Date:1336

However timber is an traditional material for Turkey, engineered timber products for medium and long spans is a new subject for architects and engineers. As seen from the examples in Turkey, in long span it has not been used. Possible reasons have been discussed in the introduction chapter, mainly due to lack of knowledge, cost and conciousness on environment. A few examples do not reflect the potential of timber, as discussed in the third chapter spans more than 100 m have been successfully spanned with structural timber. In order to have the same timber technology in Turkey, government should encourage tree farms, to have high quality and cheap raw material.

CHAPTER IV

CONCLUSION

Deciding the roof structure and the structural material of a medium or long span, has always been a critical point in the design period of a structure. Different materials and techniques being applicated for spanning of a space, but the search for the most efficient solution has always being paid attention. As being discussed in the chapters, there is not only one solution or choice for a roof structure. Various forms have been discussed, and different searchers answered the question of the most efficient structure and material for a specific span length.

The most important point is to decide the accurate structural system for a certain span length. As researched in the previous chapters, a structure may be very economical and efficient for a long span, like 60 m, but it may be so uneconomical and inappropriate for a span length like 10 m, due to need for high tecnology, workmanship, amount of material, number of connections, transportation or other factors. To decide the accurate structural system deals not only the span length, or type of the structure, also structural material is very important. The same type of a shell may give very economical results when dead weight, cost, workmanship or other factors considered, on the other hand the same design of the shell may become very unefficient when built with steel. Moreover visa verca, a beam design may be solved easily and economically with steel ‘I’ beams, but building it with solid timber may not be as efficient as steel.

Structural timber and steel in medium and long span roof structures are compared in four basic topics in the research: a) structural system of the roof, b) structural performance, c) contructional performance, d) material performance. In order to decide the structural material for a roof structure, it deals with these four main topics. Structural performance and material performance of timber and steel is valid, common and does not change with the project.

In structural performance timber is advantageous in dead weight to strength ratio than steel. However steel has a more strength than timber, timber's low dead weight decrease the depth of the structure. Mainly the unit strength of timber or steel is not an adequate data for the design, strength of the material should be calculated related with the section and dimensions. Manufacturers prepare strength tables for both timber and steel, and during the design period the decision is made related with the section dimensions.

When material properties of timber and steel is compared, both materials have advantages and disadvantages. If decay, shrinking and swelling is considered; steel is more advantageous than timber. Since timber is an organic material it is effected from living organisms, any moisture change may cause decay. During the moisture changes timber shrinks and swells, which is not desired for the stability and stiffness of the structure. If the necassary treatments are not done, timber should not be used for the circumferences that have the possibility of moisture change and living organism thread. The most important advantage of timber over steel is that environmental conciousness. In the contemporary material consuming, energy effciency is a very important topic, since the world's energy sources are limited. Steel requires energy about 12 times more than timber; which is a reason to design with timber more. Fire resistance is another advantage of timber over steel. Steel should be very well insulated for fire, in case it melts and steel structures collapse rapidly. Resistance to chemicals is a property that both timber and steel is effected. When the chemicals are considered, timber is resistant to much more acids than steel. Generally for the public buildings such as shopping malls, theatres, cinemas, culture centers there is not a acid problem. Heat expansion is a property that steel is greatly effected but timber shows no significant change. For the roof structures that are glazed and open to sun shine, steel expands. For acoustical insulation timber does not reflect sound waves as much as steel, however it is not a serious problem for roof structures. Keeping in mind the structural and material performances; a decision should be made for form and constructional performances.

When constructional performances are compared timber may have better performance depending on the local conditions. Site application of timber is easier than steel;

because the tools that are used to connect timber members are simple, due to low dead weight it can be handled easily, no complex aplications are needed at site, erection faults can be corrected rapidly. Site application of steel is more problematic than timber, due to its dead weight, welding precedures and the corrections are not easy, members should be cut at factory. In transporation costs are lower for timber than steel, because the unit weight is less. The only limitation is the length of the transportable member, according to the countries traffic limits. Connections that are needed at site for timber can be done by simple hand tools, however for steel factory connection is efficient especially for welding, at site bolting is preferred. The advantage of steel construction is connecting two steel members is more efficient than connecting timber and a steel member. If cost is compared timber has lower unit costs than steel in the western countries, because production of steel requires more service which is obvious in energy consumption. On the other hand, in Turkey due to low demand for timber; costs are higher than steel. Both timber and steel need maintanence, steel should be protected against corrosion, timber should be protected against moisture content change and living organisms.

Of course architectural considerations such as image of the building, aesthetic values, volume of the building would be determinative factors; but when making the decision from a structural point of view performances of both timber and steel should be considered for the specific form. Form of the structure greatly effects the performance of timber and steel. For instance, a 25 m long span can be designed both with a beam, truss, arch, frame, space frame, shell and dome. It is a common avarage span length for all forms. In order to decide which one to choose, efficiencies for both timber and steel should be considered. If the efficient span lengths are exceeded, a different alternative should be chosen. In an ascending order beam, truss frame and space frame can be the alternative for the previous. When a clear and huge roof height is needed the alternatives are arch, shell, prismatic forms and dome. They are in ascending order in terms of span, system requirements, amount of connections, labor, production time and dead load. Timber is advantageous in the forms that are bent, curved and can be produced with one unit, steel can have better results for the forms like space frame, where 3 dimensional stiffness is acquired and connections are in a complex form.

For a simple beam both steel and timber can be efficient for medium and long span, which is simple to produce, do not need much time for site erection and connections; however timber would have low dead weight, depth of the beam can be lower than steel. For example for the 25 m simple beam design; steel and timber would have section depths between L/18 and L/20, but dead weight occupied by steel is much more than timber, which brings extra load to the building. In a truss design if timber is used, timber is more efficient for medium spans, in the long span however it can be used, the amount of connections between the members decreases the system efficiency. For medium span when a truss used with steel design, the amount of dead weight is more which timber is preferable. In the long span however spans about 60 m can be achieved with steel, the amount of dead weight which causes high depths, is not preferable. In a frame design, timber is preferable for its low dead weight, which decreases the depth of the structure, but for longer spans more than 40 m, steel is efficient. Space frames are efficient for steel, in timber design amount of connections and stiffness is a problem to solve.

In arch design timber is preferable, because with hinge or hingeless one piece bent glued member can form an arch, in steel construction it is not preferable, one bent arch is not efficient to manufacture, transport and erect. If the arch form is produced from trusses, stiffness and efficiency of steel increase. A timber shell structure can be produced easier than steel, glued bent and shaped forms constitude a shell form. If steel is used for shell, it should be constructed from lineer members or trusses; amount of members increase the dead weight, production and erection time. Prismatic forms are more efficient with steel, because stiffness at the crown joint for timber is problematic. In dome design timber is more efficient than steel, because one piece bent forms come together to constitude a dome, and the dead weight is lower than steel dome. In steel design, lineer members come together for a dome structure, that to form a dome with lineer members is a complex precedure.

Every design and project should be decided by its own limits, local contractors, avaibility of the material, workmanship, transportation effect the decision on the structural system. Maybe a vaulted shell may be built with timber and give very economical results when structural properties are considered, but due to constructional criteria, the local contractors and avability of the material, it may have to be built with steel or another structural material. Due to discussions during the previous chapters the researcher has prepared tables (Table IV.1, Table IV.2, Table IV.3, Table IV.4) as a summary of the performances for both timber and steel. Table IV.1 refers to material properties of timber and steel, Table IV.2 summarizes the structural properties, Table IV.3 outlines the performance criteria for construction, Table IV.4 covers the structural systems.

Timber should be used more according to ts potentials as discussed before; in order to increase the amount of manufacture, design and construction necassarry regulations and standards should be harmonious with the contemporary regulations in European countries. Appropriate educations should be given by chamber of architects, chamber of contractors and ministry of cultivations. For future studies, increase in efficiency for manufacture and constuction should be researched including cost, construction period, transportation, connections, moreover problems of decay and shrinking should be solved.

Table IV. 1 Material properties of steel and timber.

Material Properties	TIMBER	STEEL
1) Environmental Conciousness	Requires 1/12 th energy of steel for production. Production period does not pollute environment.	Steel is not efficient for energy consumption, requires great amount of energy during production.
2) Fire Resistance	Holds the structure with a fire resistance up to 1 hour, while burning char layer prevents oxygen penetration. Most important feature of timber against steel.	Melts rapidly during fire, a steel structure collapses after 15 min of fire. Steel can not be used exposed, should be threated with a protective layer.
3) Moisture Content	Due to moisture changes timber changes its shape and length. Humidity changes and organic attacks cause decay. Living organisms should be prevented.	Steel rusts at high moisture levels. Needs protective coating against moisture.
4) Resistance to chemicals	Resistant many of the acids, it is not effected from cloride which is the main chemical at swimming pools.	Effected from acids, they detoriate the form and strength of steel.
5) Heat Expansion	Does not show a significant expansion against heat.	Expands rapidly when the heat increases which cause movement in the structures and during fire the form changes.
6) Acoustical Insulation	Does not reflect sound waves, appropriate for acoustical insulation.	Reflects sound waves, not appropriate for acoustical insulation. For the noisy places threatments should be taken.

Table IV. 2 Structural properties of timber and steel.

Structural Properties	**TIMBER**	**STEEL**
1) Loads	Has lower dead weight than steel, which decreases the section depths and prevents extra load to the building.	Has a higher dead load than timber, increases section depths, supports has to carry more dead loads. If the amount of members increase, total dead load creates problems.
2) Duration of Load	Withstand impact loads for a short duration of time, in long term loading capacity decreases. 100 years service without detoriation.	
3) Stresses and Strength	Strength in compression parallel to grain about 7 times perpendicular to grain. Elastic modulus between 3-30 kN/mm². Wood has high tensile strength, but also fails in a brittle manner when stressed in tension; in compression, however, wood shows ductile behavior. For unit test specimen has lower strength than steel, but dead weight to strength ratio is more than steel.	Steel has 210 kN/mm² for a given strain, can subject to higher stresses than timber. When the stress-strain curves are indicated, steel has incredibly high strength and modulus of elasticity. Has a high strength, can withstand loads about 7 times more than timber for a unit test specimen member.
4) Deflection and Elasticity	Timber has a modulus of elasticity between 4 and 12 kN/mm².	Steel has a high modulus of elasticty of 200 kN/mm² showing that it is a stiff material.

Table IV. 3 Constructional properties of timber and steel.

Constructional Properties	**TIMBER**	**STEEL**
1) Site Applications	Due to low dead weight easy to handle at site, site applications can be done with simple handtools, falsework can be corrected easily. Timber should be protected from sun, rain, humidity, storing needs care at site.	Due to high dead load, cranes needed to hold up and move, for site applications specific tools are needed, especially for welding, better to correct the falsework at factory. Members do not deform during storage, precautions should be taken.
2) Transportation	Transportation costs are low due to low dead weight, efficient long span glued laminated beams can be manufactured one piece but the limitation for length is according to traffic limits.	Transportation costs are high due to high dead weight, limitation is also for length according to traffic limits. Same truck with a specific volume can carry less steel than timber.
3) Connections	Steel connections increase the efficiency of timber, however problems occur due to two different type materials connection. Connections can be done with simple tools at site.	Steel is connected with steel connections or welding, which are efficient for strength. Steel to steel connection is efficient. Factory connection is more efficient than site connection.
4) Cost	Relatively low cost due to manufacturing do not require much energy, natural source easy to grow up, timber factories are not complex as steel factories. In Turkey costs are higher than steel due to low demand.	High costs, raw material is mined and manufacturing needs high technology factories.
5) Maintanence	Timber structures need maintanence against climatic conditions, insect attack and mould.	Steel needs paintings annually for corrosion. Costs increase to paint high level steel roof structures.

Table IV. 4 Structural systems according to limits and efficiency of timber and steel.

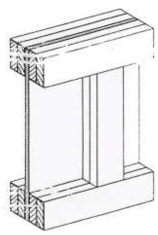
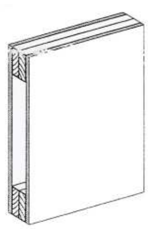
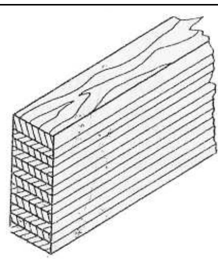
Structural Systems Performance	**TIMBER**	**STEEL**
1) Beam		
	Solid Beam: Max. span up to 7 m, depth depends on the solid piece, longer timbers can not be produced and the deflection effects the one piece solid beam. Most simple form of timber construction.	**Cold Formed Section:** Max. span up to 8 m, depth depends between L/15 and L/20. Most simple form of timber construction.
	**'I' Beam**: Max. span up to 35 m, depth between L/18 and L/20.	**Hot Rolled I Section:** Max. span up to 35 m, depth between L/18 and L/20.
	**Box Beam**: Max. span up to 28 m, depth between L/18 and L/20.	**Rolled Section:** Max. span up to 30 m, depth between L/18 and L/20.
	**Glued Laminated Beam**: Max. span up to 45 m, depth between L/18 and L/20, curved or tapered forms possible, most efficient type beam in timber construction.	**Castellated Beams:** Max. span up to 60 m, depth between L/15 and L/20, most efficient beam type in steel construction.
Comments:	Beam is the simple form of spanning a distance and smallest depth is acquired where small interior height is important. When the efficient limits are extended, truss system should be considered. Beam is erected in a short period of time, cost is lower than other systems. Transporting one piece long members can be a problem.	

Table IV. 4 (Continued) Structural systems according to limits and efficiency of timber and steel.

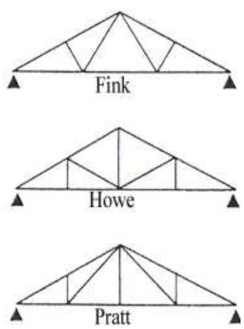
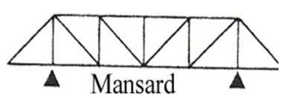
Structural Systems Performance	**TIMBER**	**STEEL**
2) Truss		
	Bowstring Truss: Span range between 12.2 m to 45.7 m, with glued laminated timber span range 30.4 m and 60.8 m, depth between L/4 and L/5.	**Bowstring Truss:** Span range between 20 m and 40 m, depth between L/6 and L/10.
	**Pratt, Howe and Fink Truss:** Span range between 12.2 m and 27.45 m, depth between L/4 and L/5.	**Pratt, Howe and Fink Truss:** Span range between 6 m and 12 m, depth between L/6 and L/10.
	**Mansard Truss:** Span range between 12 m and 33 m, depth between L/5 and L/10.	**Mansard Truss:** Span range between 15 m and 30 m, depth between L/5 and L/20.
	Special Shaped Trusses: Span of between 18 m and 46 m, depth between L/7 and L/10.	**Special Shaped Trusses:** Span between 23 m and 55 m, depth between L/5 and L/15.
Comments:	Generally truss is used for long span where distances longer than efficient beam span range is extended. The triangular form is used for a space where a high depth of a structure is needed. Truss erection time is longer than beam erection according to amount of members and connections. For medium spans trusses are manufactured at site, connections are fixed at factory, at site end connections are jointed.	

Table IV. 4 (Continued) Structural systems according to limits and efficiency of timber and steel.

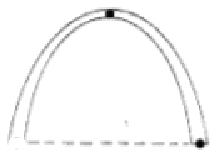
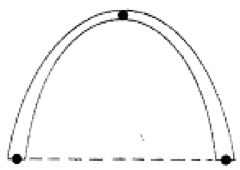
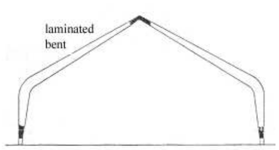
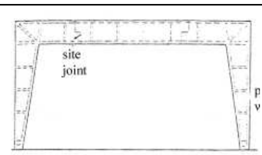
Structural Systems Performance	**TIMBER**	**STEEL**
3) Arch		
	**Two Hinged Arches:** Span range between 9.15 m to 30.50 m, depth between L/4 and L/6.	**Two Hinged Arches:** Span range between 18 m and 40 m, depth L/3 and L/5.
	**Three Hinged Arches:** Span range between 12.2 m to 30.5 m, depth between L/5 and L/7.	**Three Hinged Arches:** Span range between depth L/3 and L/5.
	Trussed Arches: Span range between 30 m and 70 m, depth is between L/7 and L/10.	**Trussed Arches:** Span range between 40 m and 120 m, depth is between L/6 and L/10.
Comments:	In arch forms laminated timber is the most efficient type, that is because one piece member is bent with a curvature. It is not easy to transport long members. Efficiency of steel arch increases with trussed members.	
4) Frame		
	**Glued Laminated Frame:** Span range between 5 m and 24 m.	**Shaped Section:** Span range between 5 m and 40 m.
	**Lattice Frame:** Span range between 10 m and 45 m.	**Trussed Frame:** Span range between 10 m and 55 m.
Comments:	Frame systems are stable and stiff due to they behave like one structure with the supports. If the building requirements are appropriate to design with supports, efficient in 3 dimension.	

Table IV. 4 (Continued) Structural systems according to limits and efficiency of timber and steel.

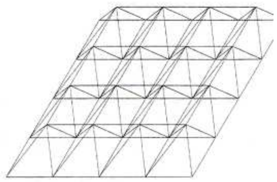
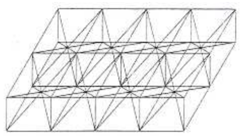
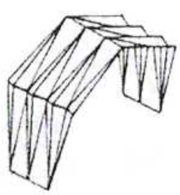
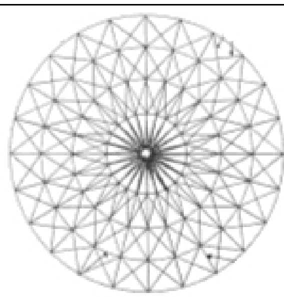
Structural Systems Performance	**TIMBER**	**STEEL**
5) Space Frame		
	**One Way Space Frame:** Span range between 20 m and 35 m, depth between L/10 and L/15.	**One Way Space Frame:** Span range between 20 m and 90 m, depth between L/10 and L/15.
	**Three Dimensional Space Frame:** Span range between 30 m and 50 m, depth L/10 and L/15.	**Three Dimensional Space Frame:** Span range between 30 m and 120m, depth between L/10 and L/15.
Comments:	The advantages of space frame are: reduction in required depth approximately 50% in height and decrease in the amount of structural material up to 25%, simplification of fabrication due to the repetition of members and better resistance to eartquake and other horizontal forces.	
6) Folded Plates		
	**Plywood Folded Plate:** Span range between 11m and 37 m, depth between L/4 and L/6.	**Rolled Sections Folded Plate:** Not a significant efficiency in folded plates.
Comments:	Folded plates are suitable for timber and concrete but not suitable for steel. A plate is not appropriate for long spans due to its weight, but folding it increases the efficiency and divides the loads to pieces. The folded form results in a great rigidity that the folded forms take most of their strength form their form. As the span increases, the depth of the folded section rise, that it is not appropriate for long spans due to increase in the depth of the structure.	

Table IV. 4 (Continued) Structural systems according to limits and efficiency of timber and steel.

Structural Systems Performance	**TIMBER**	**STEEL**
7) Shell		
	Ribbed Shells: Spans between 30 m and 45 m, depth depends on the form of the shell and should be specially calculated.	**Trussed Shells:** In the limits of trusses and space frames depending on the design, specially should be calculated.
Comments:	Shells are 3 dimensional designs and do not have a specific form. Design may include both concave and convax forms, every part should be specially calculated. Concave and convax parts do not have the same span and depth according to different response of timber and steel to compression and tension.	
8) Dome		
	**Ribbed Dome:** Span range between 15 m and 150 m, depth depending on the radius and form of the dome. Spherical domes have more depths.	**Ribbed Dome:** Span range between 15 m and 120 m. The primary problem in using steel in the dome design is that using line elements to create a curved structure.
Comments:	Dome forms are appropriate for timber, but not for steel in long span; due to increasing height in steel, amount of connections and heat expansion of steel. Steel is not preferred in long span dome design, especially over 60 m span, due to increase in height, amount of dead weight and connections of linear pieces.	

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GLOSSARY

Annual Ring (*yaş halkası*): The layer of wood growth added each growing season to the diameter of the tree. In the temperate zone the annual growth rings of many species such as oaks and pines are readily distinguished because of differences in the cells formed during the early and late parts of the seasons.

Axial Force (*eksensel kuvvet*): A push (compression) or pull (tension) acting along the length of a member.

Axial Stress (*eksensel gerilme*): The axial force acting along the length of a member, divided by the cross-sectional area of the member.

Battens (*kaplama tahtası*): Small timber members spanning over trusses to support tiles, slates etc.

Beam (*kiriş*): Normally a horizontal or sloping member that is designed to carry vertical loads:

Bearing (*mesnet*): Structural support of a truss, usually walls, hangers or posts.

Bending Moment (*eğilme momenti*):A measure of the bending effect on a member due to forces acting perpendicular to the length of the member. The bending moment at the given point along a member equals the sum of all perpendicular forces, either to the left or right of the point, times their corresponding distances from the point.

Bottom Chord (*alt başlık*): A horizontal or inclined (scissors truss) member that establishes the lower edge of a truss, usually carrying combined tension and bending stresses.

Box Beam (*kutu kiriş*): A built-up beam with solid wood flanges and plywood or woodbase panel product webs.

Cantilever (*konsol*): The part of a structural member that extends beyond its support.

Carbon Steel (*karbon çeliği*): A steel containing only residual quantities of elements other than carbon, except those added for deoxidization or to counter the deleterious effects of residual sulfur. Silicon is usually limited to about 0.60% and manganese to about 1,65%. Also termed plain carbon steel, ordinary steel, straight carbon steel.

Cast Iron (*dökme demir*): Iron containing more carbon than the solubility limit in austenite (about 2%).

Cast Steel (*dökme çelik*): Steel in the form of castings, usually containing less than 2% carbon.

Clear Span (*temiz açıklık*): Horizontal distance between interior edges of supports.
Cold Rolling (*soğuk haddeleme*): Rolling metal at a temperature below the softening point of the metal to create strain hardening (work-hardening). Same as cold reduction, except that the working method is limited to rolling. Cold rolling changes the mechanical properties of strip and produces certain useful combinations of hardness, strength, stiffness, ductility and other characteristics known as tempers.
Combined Stress (*bileşik gerilme*): The combination of axial and bending stresses acting on a member simultaneously, such as occurs in the top chord (compression + bending) or bottom chord (tension + bending) of a truss.
Concentrated Load (*noktasal yük*): An additional load centered at a given point. An example is a crane or hoist hanging from the bottom chord at a panel point or mechanical equipment supported by the top chord.
Dead Load (*hareketsiz yük*): Permanent loads that are constantly on the truss, ie: the weight of the truss itself, purlins, sheathing, roofing, ceiling, etc.
Deflection (*sehim*): The vertical displacement that occurs when a beam is loaded, generally measured at positions between supports or at the end of a cantilever.
Deflection Limit (*sehim limiti*): The maximum amount the beam is permitted to deflect under load. Different deflection limits are normally established for live load and total load.
Duration of Load Factor (*yük süresi faktörü*): An adjustment in the allowable stress in a wood member, based on the duration of the load causing the stress.
Equilibrium Moisture Content (*dengesel nem içeriği*): Any piece of wood will give off or take in moisture from the surrounding atmosphere until the moisture in the wood comes to equilibrium with that in the atmosphere. The moisture content of wood at the point of balance is called the equilibrium moisture content and is expressed as a percentage of the weight of the oven-dried wood.
Fire Resistance (*yanma direnci*): Relates to the period of time for which a structure (or component) will continue to perform its function under conditions of a fully developed fire.
Fire Retardant (*yangın geciktirici*): A chemical or preparation of chemicals used to reduce flammability or to retard the spread of a fire over the surtace

Flame Spread (*alev yayılımı*): Relates to the speed and extent of flame spread across the surface of a material. It is a property of wall and ceiling linings and can be modified by appropriate treatments.

Flame Spread Rating (*alev yayılımı oranı*): An index or classification indicating the extent of spread of flame on the surface of a material, or an assembly of materials, as determined in a standard fire test as prescribed in the building code.

Gauge (*mikrometre, kalibre*): The thickness of sheet steel. Better-quality steel has a consistent gauge to prevent weak spots or deformation.

Glued Laminated Timber, glulam (*tabakalı tutkallı ahşap*): Two or more boards glued together along their sides in order to acquire deep cross sections of beams or colums.

Header (*kasnak kirişi, başlık*): A beam which is used to support walls and/or floor and roof joists that run perpendicular to it.

Imposed Load (*özyük harici yükler*):The load produced by occupancy and use including storage, inhabitants, moveable partitions and snow but not wind. Can be long, medium or short term.

Joint, Butt (*düzyanaşma*): An end joint formed by abutting the squared ends of two pieces.

Joint, End: A joint made by bonding two pieces of wood together end to end, usually by finger or scarf joint.

Joint, Scarf (*düdükağzı geçme*): An end joint formed by joining with adhesive the ends of two pieces that have ben tapered or bevelled to form sloping plane surfaces

Joint, Tongue and Groove (*dil ve oyuk geçme*):

Joint Slip (*geçme kayması*): The failure of a connection due to movement of one piece inside the other causing slip and deformation in the joint.

Laminated Veneer Lumber (LVL) (*tabakalı kaplamalı kereste*): A structural lumber product manufactured from veneers laminated so that the grain of all veneers run parallel to the axis of a member.

Lamination (*tabakalama*): Individual pieces of lumber that are glued together end to end for use in the manufacture of glued laminated timber. These end-jointed laminations are then face bonded together to create the desired member shape and size.

Lateral Bracing (*yanal bağlama*): A member installed and connected at right angles to a chord or web member of a truss to resist lateral movement.
Light Framing (*hafif çerçeveleme*): The use of dimension lumber, trusses, and other small cross- section members to provide support and enclosure for a building.
Live Load (*hareketli yük*): Any load which is not of permanent nature, such as snow, wind, seismic, movable concentrated loads, furniture, etc. Live loads are generally of short duration.
Load Duration (*yük süresi*): The period of continuous application of a specified load, or the aggregate of periods of intermittent applications of the same load.
Log (*kütük)*: Trunk of the tree that have been cut down and freed from roots and braches.
Lumber (*kereste*): Debarked tree stems (rough timbers) are sawn into rectangular shapes of various sizes. It is a general term apllied to all cut pieces from a tree.
Lumber, Dressed Sized (*rendelenmiş kereste*) : The dimensions of lumber after being surfaced with a planing machine.
Lumber, Machine Stress-Rated (MSR) (*makinada gerilmesi ölçülmüş kereste*): Lumber which has been mechanically evaluated to determine its stiffness and bending strength.
Lumber, Nominal Size (*nominal boyut kereste*): The size of lumber after sawing and prior to surface finishing by planing.
Lumber, Patterned (*kalıplanmış kereste*): Lumber that is shaped to a pattern or to a moulded form in addition to being surface planed.
Lumber, Rough (*işlenmemiş kereste*): Lumber that has not been dressed (surfaced) but which has been sawed, edged, and trimmed.
Lumber, Visually Stress-Graded (*görsel olarak gerilmesi değerlendirilmiş kereste*): Lumber that has been graded for strength based on visual appearance, as opposed to MSR lumber which is evaluated mechanically and checked visually.
Medium Density Fibreboard (*MDF*): A panel product, widely used as a substitute for plywood, particleboard and solid lumber; manufactured in a process where wood fibres, resin and wax is compressed under high pressure to form a panel.
Moisture Content (*nem içeriği*): The amount of water contained in the wood, usually expressed as a percentage of the weight of oven-dry wood.
Node (*düğüm noktası*): Point on a truss where the members intersect.

Nominal Span (*nominal açıklık*): Horizontal distance between outside edges of the outermost supports.

Oriented Strandboard (*OSL*): A panel product, used for sheathing, made from strands with the face wafers orientation changes every layer to provide additional strength in that direction.

Parallel Strand Lumber (PSL) (*paralel lifli kereste*): A structural wood product made by gluing together long strands of wood oriented in the long direction of the panel which have been cut from softwood veneer.

Peak (*zirve*): Point on a truss where the sloped top chords meet.

Pitch Pocket (*aralık cebi*): An opening between growth rings which usually contains or has contained resin or bark or both.

Plywood (*kontrplak*): A glued wood panel made up of thin layers of veneer with the grain of adjacent layers at right angles, or of veneer in combination with a core of lumber or of reconstituted wood.

Plywood Stressed-Skin Panel: A form of construction in which sheating plywood, OSB etc. are applied over frame members to form a rigid structural panel.

Preservative (*koruyucu*): Any substance effective in preventing the development and action of wood-rotting fungi, borers of various kinds, and harmful insects that cause the deterioration of wood.

Pressure-Treating (*basınç uygulaması*): The process of impregnating wood with preservative or fire retardant chemicals by placing the wood and chemical in a pressure chamber.

Pressure-Treating, Empty-Cell Process (*basınç uygulaması, boş hücre işlemi*): Pressure treating process in which back pressure from air drives out part of the injected preservative or chemical to leave the cell walls coated but the cell cavity mostly devoid of chemical.

Pressure-Treating, Full-Cell Process (*basınç uygulaması, dolu hücre işlemi*): Pressure treating process in which a vacuum is drawn to remove air from the wood before admitting the preservation, resulting in a heavy absorption and retention of preservative due to the cells being almost filled.

Pulping (*kağıt hamuru oluşturma*): Action on wood that has been ground to a pulp; used in making cellulose products.

Purlin (*aşık*): A horizontal member in a roof perpendicular to the truss top chord used to support the decking.

Rafter (*mertek*):The uppermost member of a truss which normally carries the roof covering.

Resin (*reçine*): An ingredient of coatings which acts as a binder and gives the coating physical properties such as hardness and durability.

Round Timber (*yuvarlak ağaç*): Tree trunks cleared of bark and branches.

Sandwich Panel, Structural (*sandviç pano, yapısal*): Panels made of parallel framing members separated by expanded polystyrene which act as structural units in resisting horizontal or vertical loads.

Sapwood (*içkabuk*): The wood of pale colour near the outside of the log. Under most conditions sapwood is more susceptible to decay than heartwood.

Seasoning (*kurutma*): The process of drying lumber either naturally, or in a kiln, to a moisture content appropriate for the conditions and purposes for which it is to be used.

Shell (*kabuk*): A curved, stiff, surface which can carry normal forces by the normal components of tension, compression, or shear forces which can exist within the thickness of the shell.

Shrinkage (*büzülme*): The decrease in the dimension of wood resulting from a decrease of moisture content and generally occurring to the greatest extent between about 20 and 30 percent moisture content.

Top Chord (*üst başlık*): An inclined or horizontal member that establishes the upper edge of a truss, usually carrying combined compression and bending stresses.

Truss Plate : A light steel plate fastening, intended for use in structural lumber assemblies, that may have integral teeth of various shapes and configurations.

Web (*ağ*): Members that join the top and bottom chords to form the triangular patterns that give truss action, usually carrying tension or compression stresses (no bending).

Welding (*kaynaklama*): A process used to join metals by the application of heat. In pressure welding joining is accomplished by the use of heat and pressure without melting.

Wrought Iron (*dövme demir*): Iron containing only a very small amount of other elements, but containing 1-3% by weight of slag in the form of particles elongated in one direction, giving the iron a characteristic grain. Is more rust-resistant than steel and welds more easily.

Wood Preservative (*ahşap koruyucusu*): Means any suitable substance that is toxic to fungi, insects, borers, and other living wood-destroying organisms.

APPENDIX A.

STRUCTURE OF WOOD

A.1 Strength Properties of Timber:

The structural strength of wood is a measure of its ability to resist outside forces, such as compression, tension and shear. The density of wood is a reliable indicator of many of its structural and mechanical properties. There is a particularly strong relationship between density and compressive strength, bending strength and hardness, and a fairly reliable relationship between density and stiffness. Density ranges from an average of 160 kg/m3 for balsa to 1040 kg/m3 for greenheart, with most of the commonly used structural softwoods between 450 and 550 kg/m3.

The tensile strength of most timbers parallel to the grain is three to four times the compressive strength. There is a marked difference in all strength properties when measured parallel or perpendicular to the grain. The tensile strength parallel to grain can be thirty times as high as perpendicular to it, while for compressive strength, the ratio is of the order of six to one.
The range of strengths between species varies as much as their densities. The strongest hardwood species (eg greenheart) is almost eight times as strong in bending and almost six times as strong in compression as the weakest (eg balsa).

The small, defect-free specimens of wood used for basic strength testing give average values for all of these strength indicators but larger pieces of wood inevitably suffer from imperfections which weaken them. Features which affect the strength of timber and which are taken into account in grading systems are: a) Knots which reduce the tension, compression and bending strength. Size, frequency and position are the main considerations. b) Grain - diagonal, sloping, inclined or spiral grain, reduces the strength of the wood, particularly in bending and stiffness. c) Moisture content - as a general rule timber is more flexible when wet or ‘green’ but increases in strength as it dries.
d) Drying defects - distortion can occur due to drying stresses eg warping, cupping and twisting, and ruptures of the tissue eg checks, splits and shakes can also result. e) Biological degrade caused by fungal decay or insect attack. f) Natural defects resulting from the growth of the tree, such as bark and pitch pockets, compression fractures and brittleheart.

A.2 The Structure of Hardwoods:

In a wedge-shaped segment cut from the bole of a hardwood (Fig. A2.1); the outer layer, or bark, serves as protection for the delicate layers beneath, known as phloem. The phloem conducts food both to regions of active growth and to storage regions within the wood xylem. Between the phloem and the xylem, is a thin tissue called the cambium. This specialised, actively growing cell layer completely encloses the living parts of the tree. It produces bark towards the outside and wood towards the centre of the bole.

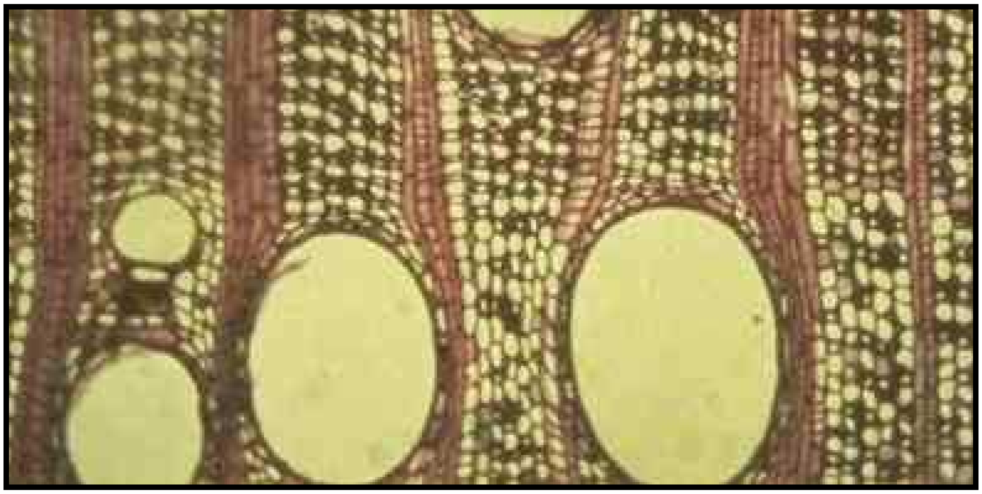

Figure A2. 1 Structure of hardwoods (Timber Research and Development Association, document on-line).

At the centre of the bole is the pith, the first year's growth of the sapling. This is sometimes difficult to see in a section of the mature tree. The division of the cambial cells lays down new wood on a core of existing material. Concentric layers of tissue known as growth rings can be seen in the wood of trees grown under seasonal conditions. These are actually layers extending for the full height of the tree. In temperate regions, and in some tropical situations, the alternation of a growing season and a resting season each year creates a distinctive pattern in the cross-section of the bole.

Wood laid down at the beginning of the growing season differs from that formed later in the year. Zones known respectively as early wood and late wood may be distinguished (Fig. A2.2). In temperate climates these may be referred to as 'springwood' and 'summerwood'. Early wood is softer and more porous than late wood. In some tropical regions or with certain species, growth rings may be indistinct, and may not necessarily be annual. Sap conduction is performed by the young, outermost layers of the bole. In many, but not all timbers, the sapwood is lighter in colour than the heartwood. The proportion of sapwood to heartwood varies, but very approximate figures are between one-quarter and one-third of the area of the cross-section of a mature bole. The change in colour which often occurs is related to the chemical changes which take place in the wood as the sapwood dies. Substances are laid down in the heartwood which frequently render it more durable and less permeable than sapwood.

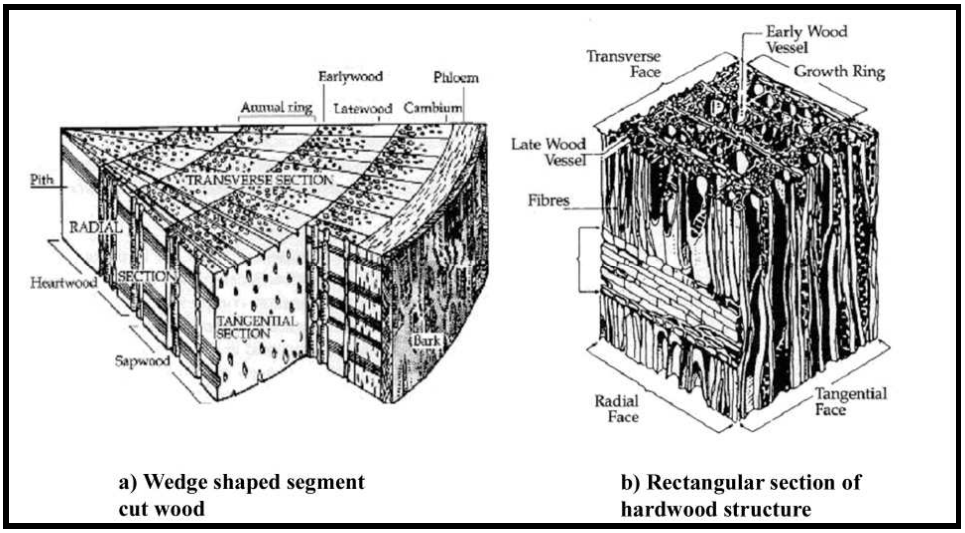

Figure A2. 2 Wedge-shaped segment cut from the bole of a hardwood and rectangular section of hardwood (Timber Research and Development Association, document on-line).

APPENDIX B.

EUROCODE 5

Eurocode No 5 ‘Common Unified Rules for Timber Structures’ will be one of the nine Eurocodes which are currently being prepared in association with the EEC Construction Products Directive. Eurocodes will provide common technical rules for structural design. They are intended to remove trade barriers arising from differing structural design rules. The Eurocodes are being published as European Pre-standards or prENs initially. They will circulate alongside national codes for an interim period of several years to allow designers in all member countries to become accustomed to the new code and design procedures and to propose amendments. Eurocode 5 sets out the basis of design in terms of principles which must be adhered to and application rules which present recommended ways of applying them. It is intended to ensure that, with an acceptable probability, a structure will remain fit for its intended use during its intended life.

The Eurocodes are a series of standards which establish common rules for the design of structures within the European Union. They provide the means for a designer to prove compliance with the requirements of the Construction Products Directive (CPD) and National Building Regulations. They also provide a framework for harmonized specifications for construction products. Ten Eurocodes are either published or are in preparation: EC0 Basis of design; EC1 Actions on structures; EC2 Design of concrete structures; EC3 Design of steel structures; EC4 Design of composite steel and concrete structures; EC5 Design of timber structures; EC6 Design of masonry structures; EC7 Geotechnical design; EC8 Design of structures for earthquake resistance; EC9 Design of aluminium structures.

Each Eurocode is initially published as a European pre-standard, or ENV to serve as an alternative to the existing national codes in each country. This ‘trial period’ allows time for experience in its use to be gained. The Eurocodes will then be revised before finally being published as definitive EN standards which will replace the national codes.

B1. Material Properties:

The Service Classes are the same as those adopted in the 1996 edition of BS 5268-2, whilst the definitions of load duration differ slightly between the two codes. The three Service Classes in EC5 approximate to average equilibrium moisture contents in most solid softwoods as given below. The NAD includes examples of the relevant environmental conditions: a) Service Class 1: Timber moisture content: 12% or less, corresponding to a temperature of 20° and a relative humidity of

65%. Environmental conditions: Timber in buildings with heating and protected from damp conditions eg internal walls, internal floors (other than ground floors) and warm roofs.
b) Service Class 2: Timber moisture content: 20% or less (20°, 85% relative humidity) Environmental conditions: Timber in covered buildings eg ground floor structure where no free moisture is present, cold roofs, the inner leaf of cavity walls and external single leaf walls with external cladding. Note:Service Class 2 is similar to the dry exposure condition formerly used in BS 5268. c) Service Class 3: Timber moisture content: above 20% ie conditions leading to a higher moisture content than in Service Class 1 or 2. Environmental conditions: Timber fully exposed to the weather eg exposed parts of open buildings and timber used in marine structures. (Note: The equilibrium values of panel products, such as particleboard will be lower than those achieved by softwood).

The five Load Duration Classes for actions in EC5 are: a) Permanent: more than 10 years, eg self weight; b) Long-term: 6 months - 10 years, eg storage; c) Medium-term: 1 week - 6 months, eg. imposed load; d) Short-term: less than 1 week, eg snow* and wind; e) Instantaneous: accidental impact.

* In areas which have a heavy snow load for a prolonged period of time, part of the load should be regarded as medium-term.The modification factors (kmod) for Service Class and Load Duration Class are tabulated for solid timber, glulam and wood-based board materials.

B2. Solid Timber and Glulam:

Solid timber must be strength graded, either visually or by machine. The requirements for strength grading are given by reference to BS EN 518 Requirements for visual strength grading standards or to BS EN 519. Requirements for machine strength graded timber and grading machines. The rules for visual stress grading of softwood laid down in BS 4978 and in BS 5756 for hardwoods meet the requirements of BS EN 518.

The characteristic values of structural timber are given in BS EN 338 Structural timber, strength classes. The way in which the species and grades of timber commonly available in the UK relate to these strength classes is given in the NAD and summarized in the TRADA WI Sheet European Strength Classes and Strength Grading. Timber sizes are given as target sizes in a national annex to BS EN 336 Structural timber. Coniferous and poplar -timber sizes - permissible deviations. The target size relates to a timber moisture content of 20%. Finger joints are required to comply with BS EN 385 Finger jointed structural timber. Performance requirements and minimum production requirements. Glued laminated timber is required to comply with BS EN 386 Glued laminated timber. Performance requirements and minimum production requirements.

B3. Durability:

Durability requirements are laid down for timber and wood-based materials and for metal fasteners and other structural connections. The resistance of timber to biological organisms is defined by reference to hazard classes set out in BS EN 335-1 to 3 Hazard classes of wood and wood-based products against biological attack and by reference to BS EN 460 Guide to the durability requirements for wood to be used in hazard classes. These requirements can be met by naturally durable timbers as defined in BS EN 350-2 Natural durability of solid wood or by preservatively treated wood to BS EN 351-1 Preservative treated solid wood. The specification of preservative treatment levels in the European standards is still under discussion but will differ from that traditionally used in the UK. Examples of minimum corrosion protection and specifications for metal fasteners are included in the Code.

B4.Serviceability Limit States:

This chapter deals with requirements for limiting deflection and vibration and gives the principles and equations for calculation. This is often regarded as one of the more complex areas of EC5 and so a number of guidance publications have been produced: WI Sheet 4 - 24 and TRADA Technology Eurocode 5 Guidance Documents 5 and 6.

B5.Ultimate Limit States:

This sets out the design procedure for members of solid timber or glued laminated timber. EC5 treats solid and glulam members in the same way. The chapter covers tapered, curved and pitched cambered beams, built-up components such as thin webbed and thin flanged beams and mechanically jointed beams. The design of assemblies such as trusses, wall diaphragms and plane frames is included. Aspects such as bracing of structures and load sharing are addressed.

B6.Connections:

The design of joints made with dowel-type fasteners ie laterally loaded nails, staples, screws, bolts, and steel dowels is covered. Axial capacities for such fasteners are also covered where appropriate. A design procedure for joints with connectors such as toothed plates, shear plates and split rings is being developed but is not included in the ENV edition of EC5. The NAD provides advice on design of such joints, based on the information given in BS 5268-2.
The design of joints using punched plate metal fasteners is covered separately in an Annex.

B7.Components and Assemblies:

Components, eg timber-based I-beams and assemblies, eg trusses, wall diaphragms, bracing systems, are covered in reasonable detail concerning modelling assumptions and calculation capacity.

B8.Structural Detailing and Control:

This gives requirements for materials, joints assembly, transportation and erection which are necessary to satisfy the assumptions made in the design process. It also provides guidance for a plan to cover the control of production and workmanship and of servicing and maintenance after the structure has been completed.

B9.Informative Annexes:

A series of annexed sections are included, which primarily focus on items of lesser relevance to mainstream timber design practice, such as the statistical treatment of data to derive characteristic properties.

APPENDIX C.

TURKISH STANDARDS FOR STRUCTURAL TIMBER

TS 1265 Coniferous sawn timber for building construction

TS 1485 Coniferous sawn timber, terms, definitions and methods of measurement

TS 2456 Rules for kiln drying of timber

TS 2472 Determination of density for physical and mechanical tests of wood

TS 2473 Testing of wood in compression perpendicular to grain

TS 2474 Determination of ultimate flexural strength of wood

TS 2475 Determination of ultimate tensile strength of wood parallel to grain

TS 2476 Determination of ultimate tensile strength of wood parallel to grain

TS 2477 Determination of impact resistance of wood in bending

TS 2478 Determination of static modulus of elasticity of wood in bending

TS 2595 Determination of compression strength of wood parallel to grain

TS 3459 Determination of shear strength of wood parallel to grain

TS 3842 Glued laminated timber structural members

TS 4085 Determination of volumetric shrinkage of wood

TS 4086 Determiantion of volumetric swelling of wood

TS 5497 Determination of physical and mechanical properties of solid timber in structural sizes