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METAL STRUCTURES IN CONSTRUCTION

CONTENTS:

FOREWORD

This manual complies with the new syllabus “Metal structures. Metals and welding in construction” courses for students of specialities “Industrial and Civil Engineering” and “Highways and Aerodromes”.

This manual is based upon many-year experience of teaching this course by the group of lecturers of the department of computer technologies of construction at the Airport Institute of National Aviation University.

This training course consists in studying methods of designing and analysing metal engineering structures (of steel and aluminium).

The first two chapters are devoted to the fundamentals of designing and the theory of analyzing metal structures and structural members with account of the material working not only in the elastic, but also in the elastoplastic stage.

Chapter 3 describes various structural shapes and methods of joining structural elements together, the actual behaviour of the joints and their investigation, as well as certain industrial requirements, which the design of structures must meet.

In other chapters, the reader will find a detailed consideration of the principal elements of metal structures such as beams, girders, trusses and columns.

All the material conforms to the standards for designing steel structures and structures of aluminium alloys, as well as to the general building standards and regulations followed in Ukraine.

1. FUNDAMENTALS OF DESIGN OF METAL STRUCTURAL MEMBERS

1.1. Fields of application and nomenclature of metal structural members

Metal structural members are mainly designed for taking up the loads acting on a structure. The constructional shape of a structure is determined by the combination of the main constituent parts and elements such as beams, girders, trusses, columns or shells that form this structure. The best constructional shape of a structure and its elements is selected in the process of designing, which is a creative process wherein many solutions may be obtained. The duty of the designer is to find in each case the most appropriate and rational solution for his designing problems that will conform to the modern level of development of science and engineering.

Steel structural members have found a wide use in the following types of structures, which can be divided into two groups.

Framework or skeleton systems having as their main elements beams, girders, trusses, and columns, such as:

– the frameworks of industrial buildings and structures (mainly the shops of iron-and-steel and machine-building works) with their internal members such as crane girders, platforms, and so forth;

– railway, highway and urban large-span bridges;

– civil multistory buildings, exhibition pavilions, various vaults, roofs, floors, domes, etc.;

– special-purpose buildings such as hangars, shipbuilding slips, and so on;

– special structures, for example towers and masts, headwork's of mines, oil derricks, hydraulic engineering structures, trestles, cranes, etc.

Shell systems which are largely made up of plates or sheets and include, among others:

– gas holders and tanks for the storage and distribution of gases;

– tanks and reservoirs for the storage of liquids;

– bins and bunkers for the storage and handling of loose materials;

– special structures such as blast furnaces, air heaters, gas scrubbers;

– large-diameter piping employed at iron-and-steel, coke and by-product works, hydroelectric power plants, for oil and gas pipelines.

Owing to their low specific weight and high resistance to corrosion, structural members made of aluminium alloys have found use first of all in enclosing members such as roof cladding, wall panels, window sashes, etc., and also in the chemical and petroleum industries.

Aluminium members are employed in structures where the own weight of the structure is of considerable importance (transfer cranes, drawbridges, the large-span roofs of pavilions, structures erected in seismic areas and in regions difficult of access).

1.2. Advantages and disadvantages of metal members

The principal advantages of steel members are:

– the ability to resist high loads with comparatively light weight and small size of the members, due to the high strength of steel; regardless the high specific weight of steel (7.85 t/m^3) members made of this material have lower weight in comparison to members of other construction materials; owing to the high strength of the material, steel members are small in size, which makes them convenient for transportation;

– their gas and water tightness, which is due to steel high density;

– a long service life, determined by the high homogeneous strength and density properties of steel;

– the possibility of industrializing construction work, attained by the use of prefabricated members with mechanized erection thereof at the construction site;

– the possibility of readily disassembling or replacing steel members, which makes it easier to reinforce or replace parts of a structure.

The principal deficiency of steel members and elements is their susceptibility to corrosion, which necessitates their painting or the use of other methods for their protection.

Among the advantages of aluminium, alloys are their low specific weight (2.7 t/m^3) with comparatively high strength and their high resistance to corrosion.

The shortcomings of aluminium members include the relatively low value of the alloys elasticity modulus (E = 7100 kH/cm^2), which determines the increased deformability of there members, as well as their low heat resistance (the mechanical properties of the alloy begin to deteriorate at a temperature above 100 ^{o}C, while at the temperature over 200 ^{o}C creep begins).

1.3. Requirements for metal members to comply with

Metal members designed to take external loads must meet a number of requirements without which their normal service will be impossible, namely, a member must correspond to its application in service, it must have the required load-carrying capacity, i.e., it must be strong and stable with adequate rigidity.

The basic principles followed in designing structures are the development of structural members of minimum weight, with the lowest possible labor requirements for fabrication, and permitting the structures to be erected in the shortest possible time.

For this reason, designs of metal structural members must comply with the following requirements.

The shape of a member and its parts should be adapted to service conditions and be convenient for protection against corrosion, the accumulation of dust, dirt, etc.

The member must have the most advantageous dimensions making possible complete utilization of its material, the tendency having to make the greatest possible use of standardized structural elements.

The structure must be well proportioned and good-looking.

It is desirable that the greatest possible number of elements used in a structure be identical. An order placed for serial production of members with a minimum number of various types will cut the cost and labor required for fabrication of the elements or parts of the members by several times. The design must take into account the possibilities of fabricating plants (the use of jigs and various contrivances, characteristics of machine tools, capacity of the cranes, and welding equipment available etc.).

The elements should be simple in shape and consist of minimum possible number of pats, with a minimum of welds or rivets. The members should be easy to assemble.

A member or a part thereof shipped from a plant (the so-called *shipping element*) must have dimensions within the clearance gauge of the railway or other kind of transport that will deliver it to the erection site.

The tendency should be to use the minimum number of erection elements in a structure with the greatest possible interchangeability thereof. The connections of elements should be simple and convenient for assembly. Attention should be given to the possibility of using large prefabricated members in the erection of structures.

Members that meet all the above requirements can be called high quality and rational ones. The shapes of structural members are not fixed; they keep pace with the general development of science and engineering.

All terms, concept definitions and symbols presented in Appendix A of this manual are used in the Building Code ДБН В.2.6-163:2010–10, which came into force in 2011–12–01.

During any structure, designing it is necessary to insure its serviceability and operating safety predetermined in the design task.

To achieve that it is required to do the following:

− insure reliability of structures though observing requirements concerning their designing, calculation and material selection;

− make designing decisions insuring strength, rigidity, stability and spatial invariability of buildings, structures and their separate elements during their transportation, erection and operation;

− provide measures insuring durability of structures and their protection against corrosion, fire, heat, wear and abrasion.

1.4. Organization of designing process

Metal structural members are designed on the basis of the project requirements. Depending on their complexity, structures are designed within one or two stages:

– one stage — working project;

– two stages — project and working documentation.

The project assignment aims at determining whether complicated structures are feasible in technical and economic terms.

The working project of metal structural members consists of two stages. In the first stage, the final design of the structure is determined, all the members are analyzed and the sections of all their elements are selected, the general drawings and drawings of complicated assemblies are prepared, and all the members are coordinated with the technological, transportation, power supply, sanitary engineering, architectural, building and other parts of the project. Specification of the metal required is also drawn up at this stage. The working drawings at the second stage, which show all the elements of the members, are elaborated, as a rule, by the design offices of the fabricating plants on the basis of the first stage of the working project. They are drawings of separate member elements that will be shipped ready for erection from the plants (shipping elements, standard elements), as well as erection drawings with the corresponding marking.

1.5. Method of analyzing metal members

The designing process of metal structures begins with drawing up and comparing several alternatives of structure solutions meeting the stipulated service requirements. Then the design is selected that best complies with all the requirements relating to metal members.

The analysis of members is aimed at checking the strength, stability and rigidity of the preliminarily selected solution of the structure, which will make it possible to determine more precisely the dimensions and ensure construction reliability with the least metal consumption.

Structures and their members and elements are analyzed on the basis of the theory of materials strength and structural mechanics. The main purpose of this theory is to determine internal stresses that are induced in members under the action of applied loads.

With this aim in view there should be first drawn up what is called the general layout of a structure consisting of separate elements (beams, rods), temporarily disregarding the actual shape of their cross sections. The supporting connections of the elements are endowed here with certain theoretical properties (hinged, elastically restrained or fixed connections, etc.). After having determined the stresses in various elements according to the general layout adopted as the basis of designing, the element sections are selected, the load-carrying capacity is checked and the connection is designed to meet the stipulated conditions.

Sometimes more precise methods of determining stresses are necessary taking into account the development of plastic deformations. However the mathematical complexity of these methods, however, often necessary tats the use of a number of coefficients and factors in formulas whose values are given in various tables.

Previously steel structures were investigated by means of the method of elastic design, based on allowable stresses. At present a perfect method of analysis has been introduced, viz., the method of limit design, developed by Prof. N. Streletsky. There principles underlie the “Building Standards and Regulations”, Part 2 which is devoted to designing structural members and foundations.

In this method of design, each member is considered in its limiting state. The limiting state of a member denotes such a state wherein it does not any longer meet the service requirements, either it loses its ability to withstand external loads, or it acquires an inadmissible deformation or local damage.

Two limiting states have been established for metal members, namely:

– the first group of limiting states, also known as the condition of no destruction, determined by the load-carrying capacity (strength, stability or endurance);

– the second group of limiting states, determined by the development of excessive deformations (deflections and displacements); this state is used for checking members in which the magnitude of the deformations may limit the possibility of their service.

The first limiting state is expressed by the inequality

$$F \le C, \tag{1.1}$$

where F is the design force in a member caused by the combined action of all the design loads P in their most unfavorable combination; C is the load-carrying capacity of the member, which is a function of the geometrical dimensions of the member and the design strength of the material R.

The maximum values of the loads tolerated in the normal service of members, which are established by the building standards and regulations, are called the working or service loads P_n. The design loads P used in the analysis of members (for the condition of non-destruction) are taken somewhat greater than the service ones. The design load is determined as the product of the service load and coefficient of reliability on load γ_f (greater than unity), which takes into account the danger of an increase in load over its service value due to possible load changes

$$P = P_n\gamma_{f.}$$

Thus the behavior of members is considered subject to the action of the design, and not the service loads. The values of the design loads are used to determine for the members the design forces (axial force N, bending moment M, etc.), which are found in accordance with the general rules of the strength of materials and structural mechanics.

Design force F depends on response degree of a building and is multiplied by the reliability coefficient for usage — $\gamma_{n.}$

The right-hand term of the basic inequality (1.1), the load-carrying capacity of member *C* depends upon the ultimate strength of the material, its resistance to the action of various forces which is characterized by the mechanical properties of the material and is called the service resistance or strength R_n, as well as upon the geometrical characteristics of the cross section (section area *A*, section modulus *W*, etc.).

For structural steels the service tensile, compressive and bending strength is taken equal to the minimum value of the yield point guaranteed by the corresponding State Standards (GOST), i.e.,

$$R_{yn} = \sigma_y.$$

In some instances, when the service of members working under tension is still possible after the metal has reached the yield point (for example, cylindrical tanks, etc.), as well as reached for materials that do not have yielding properties (for example, high-strength wire, iron castings, etc.) the service tensile strength of the steel is taken equal to the minimum guaranteed value of the ultimate tensile strength, i.e.,

$$R_{un} = \sigma_u.$$

The service tensile, compressive and bending strength of aluminum alloy is taken equal to the smaller of the following two quantities:

– seven-tenths of the minimum guaranteed value of the ultimate tensile strength, i.e., 0.7 σ_u;

– the arbitrary yield point, also known as the offset stress $\sigma_{y0.2}$ corresponding to the stress producing a unit permanent elongation of 0.2 %.

The design strength R is obtained by multiplying the service strength by the coefficient of material γ_m (less than unity) that takes into account the danger of reduction in the strength of the material in comparison with its service strength due to possible variations in the mechanical properties of the material:

$$R = R_n \frac{1}{\gamma_m}.$$

Thus the design strength R is a stress equal to the lowest possible value of the yield point (or the ultimate tensile strength) of a material, which is taken as the limiting strength of the member.

To ensure safety of a structure it is necessary to take account of possible deviations from the normal service conditions, as well as special features of work of the member (for example, the particularly strenuous working conditions of girders supporting certain types of overhead cranes; a corroding medium causing an increased rate of metal corrosion; constant work of the members at the limit loads with only slight variation thereof; and where there is a high probability that the stresses produced by these loads coincide with the lowest value of the strength of the material). To take account of all these circumstances, there is introduced a safety factor known as the coefficient of working conditions reliability (γ_c) of structural members and their elements, which reduces the value of the design strength when necessary, and thus we have

$$R = R_n \gamma_c / \gamma_m.$$

Thus, the first limiting condition (1.1) for checking the strength can now be expressed as

$$N \le A_n R \quad \text{or} \quad M \le WR,$$

where N and M are the design axial forces and bending moments caused by the design loads.

When investigating the contemplated member first, the section of an element is selected and then the stress caused by the design forces is checked. This must never exceed the design strength (corrected by any or all of the safety factors indicated above that are essential for ensuring reliable service of the structure).

The basic formulas for checking the strength of members being designed will thereupon be as follows

$$\sigma = \frac{N}{A_n} \le R \quad \text{or} \quad \sigma = \frac{N}{W} \le R,$$

where σ is the design stresses in element (caused by design loads); A_n is the net area of section (holes subtracted); R is the design strength of material, taken from the Building Standards and Regulations; W is the section modulus;

The second limiting state of a member characterized by the appearance of excess deformations (deflections) requires that the member have sufficient stiffness. Under conditions of normal service the value of the unit strain $\left(\frac{f}{L} = \frac{\text{deflection}}{\text{span}} \text{ or } \frac{f}{h} = \frac{\text{deflection}}{\text{depth}}\right)$ must not exceed the value of the tolerated unit strain $\left(\frac{1}{n_0}\right)$ established by the standards for various members.

Thus the condition should be observed that $\frac{f}{L} \le \frac{1}{n_0}$.

When determining the deformations (deflections) the service load, and not the design one, is used, i.e., no attention is paid to the coefficient of reliability for load.

According to the Building Code /10/ all structures, depending on their purpose and possible effects which may occur when structures attain their boundary states, should be differentiated *by purpose into three categories of structures and their elements:*

A — structures and elements, whose boundary state when attained can result in full inoperability of either the whole building or structure or its larger part.

B — structures and elements, whose boundary state when attained can complicate the normal operation of buildings and structures as a result of unacceptable bending and displacements.

C — auxiliary structures and elements, whose boundary state when attained doesn't result in violation of operation requirements for load-carrying structures.

Depending on the possibility and reasons of attaining boundary state and conditions of fatigue or fracture failure, *all structures and their elements* should be differentiated *by stress state into three categories:*

I — structures and elements, whose boundary state can be attained as a result of direct action of dynamic motion or vibration loads;

II — structures and elements, whose boundary state can be attained only as a result of combining adverse factors (dynamic or vibration load, stress concentrators, areas of tension stresses, etc.)

III — structures and elements which cannot attain fatigue or fracture failure without presence or small influence of adverse factors;

The list of structures and elements containing indication of categories according to their purpose and stress state are provided in Appendix B, table B.1 /10/.

Elements and structures examined in the Building Code /10/ are divided into three classes depending on the type of stress-strain state of design section: the first class corresponds to such a stress-strain state when normal stresses in any part of design section are less than design strength of steel R_y and can attain it only in the most stressed part of the section (elastic behavior of the section (Fig. 1.1, *a*).

The second class corresponds to such a stress-strain state when normal stresses in some parts of design section are less than design strength of steel

R_y while in other parts — are equal to it (elastic and plastic behavior of the section (Fig. 1.1, *b*). The third class corresponds to such a stress-strain state when normal stresses σ in all parts of design section are equal to design strength of steel R_y (plasticization of the entire section with formation of plastic hinge (Fig. 1.1, *c*). The steel structures have to be calculated as a united spatial system taking into account all factors defining its stress-strain state and whenever necessary taking into account nonlinear property of the design model.

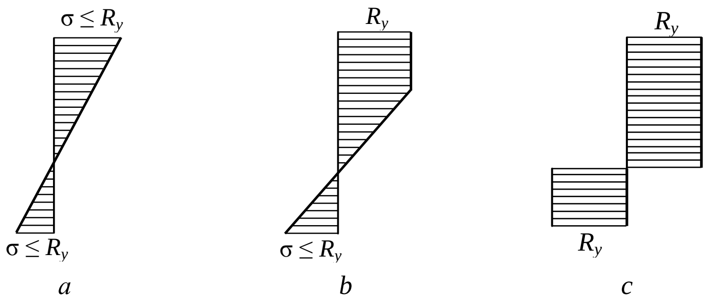


Fig. 1.1. Classification of sections by type stress-strain state

During calculations of structures and joints, it is necessary to take into account:

– reliability coefficient by the type of critical structures (critical coefficient) γ_n according to the requirements of the Building Code ДБН B. l.2–14;

– additional reliability coefficient by the type of material γ_u = 1.3 for elements of structures calculated for strength using design strength R_u, determined by temporary strength for tension;

– coefficients of operation conditions for structure elements γ_c,which are assumed according to the Code requirements within 0.75–1.20;

– coefficients of operation conditions for joints γ_b.

1.6. Loads and actions

The main purpose of any steel structure is to withstand various service (useful) loads. These loads include overhead and other cranes, equipment, etc., in industrial buildings; crowds of people, etc., in civil buildings; vehicles on bridges; water in hydraulic engineering structures; loose materials for bins and bunkers, and so on. Besides the service loads, structures are also subjected to other loads such as snow, wind, thermal effect, etc., as well as the own weight of the members.

According the duration of their action loads are divided into:

– permanent or dead loads — for example the own weight of the structural members, the weight of the floors, roofs, walls, the weight and pressure of soils, etc;

– temporary or live loads acting for a long period (long duration loads), known as movable loads — for example the weight of stationary equipment, the loads on the floors of stores and warehouses, libraries, theatres; the pressure of gases, liquids and loose materials in tanks and reservoirs; the continuous thermal action of equipment;

– live loads acting for a short time (transient load) called moving loads — for example cranes and other mechanical handling equipment, the load of the occupants of buildings, wind loads, temperature (climatic) action, erection and other loads;

– special loads — for example loads caused by earthquakes, accidents, the settlement of foundations, etc.

As a rule, not one, but various combinations of loads act on a structure. The probability of the simultaneous action of the maximum loads of all kinds on a structure is very small, and a structure designed for such a combination of loads would have an excessive margin of safety. The values of the most frequently encountered service loads and load factors, as well as the combination of loads that should be taken into account when designing structures, are established by the Building Standards and Regulations, which provide for the following two categories of load combinations:

– main combinations, consisting of the dead loads, movable loads and one, most important, moving load;

– special combinations, consisting of the dead loads, movable loads, possible moving loads and one of the special loads.

When these combinations are considered, the vertical and horizontal loads induced by overhead cranes are treated as one moving load.

In exceptional cases, the main combination also takes into consideration the combined action of a snow load and one or two overhead cranes (except cranes with light and medium duties). When designing members with regard to the main load combinations, the values of the design live loads (or of the stresses in the members corresponding to them) should be multiplied by the load combination factor $n_c = 0.9$, or while taking account of special combinations the factor $n_c = 0.8$ should be employed.

The own weight of the steel structural members is usually only a small fraction of the total load and can be preliminarily established on the basis of an approximate estimate, or from similar existing structures.

The service live loads are given in various Building Standards and codes. The wind load is considered not only as the result of active pressure on the windward wall, but also as being produced by the suction acting on the roof and the inner side of the opposite wall.

With dynamic loads that create impacts and cause vibrations of a structure (cranes, trains) the same service loads are used in analysis, but after being multiplied, besides the usual load factor, by the special dynamic load factor.

Methodical instructions to Chapter 1

Before starting the course "Metal constructions" students should have a clear idea of this subject significance in special training of engineers – builders of airports as well as this subject role in the field of technical construction science. It should be pointed out that designing civil and industrial buildings and structures is carried out by the research and design institute "UkrProjectSteelConstruction" (Kyiv).

Acquaintance of students with metal structures should start with getting knowledge about fundamentals of metal structural design — the field of application and nomenclature of steel and aluminium alloys structural members, requirements for metal members to meet, organization of designing.

It's necessary to master the basic method of analyzing metal members — the method of limiting states.

Questions and Tasks to Chapter 1

1. *Name the fields of application of load-bearing metal structural members in civil aviation buildings and constructions.*
2. *What are the main advantages and disadvantages of metal members?*
3. *What are the perspectives for using left-weight metal structures members?*
4. *What are the stages of structure design?*
5. *What state of a member is called the limiting state?*
6. *What groups of limiting states of a member do you know?*
7. *Note the analytical expression of providing load-carrying capacity of members.*

2. MATERIALS AND THEIR BEHAVIOUR IN MEMBERS

2.1. Materials used in metal members

General requirements. Steel building structures are produced of rolled steel (sheet steel, structural shapes, broad strip, and profiled iron), roll-formed shapes, pipes produced of low-carbon and low-alloy steel, and wire cables.

By the indexes of corrosion resistance, steel is divided into three groups:

• I group — steel of ordinary corrosion resistance which includes low-carbon steel of Ст3пс grade and low-alloy steels of such grades as 09Г2С, 10Г2С, 15Г2СФ and others;

• II group — steel of increased corrosion resistance of such grades as 09Г2Д, 09Г2СД, 14Г2Д, lОХСНД, 15ХСНД and others. Such steel grades are used for medium-, high- and heavy corrosive media;

• III group — weatherproof (economically alloyed) steel of 1ОХНДП, 14ХГНДЦ grades and others.

The main physical properties of materials for structures are determined according to Appendix Г /10/.

Sheet steel, structural shapes and broad strip are divided into:

– rolled steel of usual strength, made of low-carbon steel with flow limit R_{yn} up to 290 N/mm^2. By the deoxidation level such steel is divided into rimmer, balanced and finished steel. By the quantity of admixtures rolled steel has to contain: less than 0.22 % of carbon, less than 0.05 % of sulfur and less than 0.04 % of phosphorus. Temporary elongation of the circle shape sample with length equal to its five diameters has to be more than 18 %;

– rolled steel of increased strength, produced, as a rule, of lowalloyed steel with flow limit R_{yn} more than 290 N/mm^2 and up to 390 H/mm^2 inclusive when thickness of rolled metal doesn't exceed 50mm. Temporary elongation of the circle shape sample with length equal to its five diameters has to be more than 16 %;

– rolled steel of high strength, produced, as a rule, of treated steel with flow limit R_{yn} from 390 N/mm^2 to 590 H/mm^2. Temporary elongation of the circle shape sample with length equal to its five diameters has to be more than 14 %.

The requirements for low-alloyed steel used for steel structures are provided in table Д2 of appendix Д/10/. Requirements for using rolled steel depending on steel deoxidation level and elasticity factor impact are given in table Д3 of appendix Д/10/.

In general rolled steel has to meet the requirement concerning its impact elasticity according to table Д.1 of appendix Д/10/.

The rolled steel from low-alloyed steel for welded structures has to meet the requirement concerning its chemical composition and carbon content in accordance with table Д.2 of appendix Д/10/.

In selecting steel for any structure it's necessary to take into account the following:

– conformity class of buildings and structures according to the requirements of Code DBN V.1.2-14;

– purpose of structures and elements;

– possible effects of boundary state achieved;

– character of acting stresses (static, dynamic) and their level;

– type of stress state (uniaxial, plane, triaxial tension or compression);

– presence of welded joints (level of residual stresses and stress concentration, property of steel in the welding area);

– aggressiveness effect level;

– thickness of rolled steel;

– peculiarities of structural design and production technology (stress concentrators, guillotine cutting, mechanical hardening, etc);

– climatic zone of construction and specified temperature of operation.

Steel selection for groups of structures should be made in accordance with the requirements of table E1 of appendix E/10/.

Materials of mass usage. Rolled steel for metal structures manufacturing is used according to the corresponding state standards. So, standard GOST 27772 is applied to structures of structural shapes (angles, H beams and channels). The standard GOST 27772 and other operating normative documents are used for broad strip and roll-formed shapes. The standards GOST 19281 and GOST 535 are used for profiled iron (circle and square shaped, strip). The standard GOST 16523 is applied to sheet metal of carbon steel. The standard GOST 17066 is applied to increased strength steel. The standards GOST 10705 and GOST 10706 are applied to electric-welded pipes. The standard GOST 8731 is applied to hot-rolled pipes. As a rule, all steels should be used according to Appendix E/10/.

The materials for welding and welding technology shoud insure the value of temporary strength of joint weld's steel not less than strength R_{un} of welded steel elements, as well as such characteristics as impact elasticity and temporal elongation.

The steel bolts and screw-nut for bolted joints should meet the requirements of GOST 1759.0, GOST 1759.4, GOST 1759.5 and GOST 18123. High-strength bolts are applied according to table Ж3 of appendix Ж/10/.

Steel for foundation bolts should be used according to the standards of GOST 535 and GOST 24379.0 while design and dimentions of bolts, according to GOST 24379.1.

Screw-nuts for foundation and U-shaped bolts up to 48 mm in diameter should be used according to GOST 5915, while bolts of more than 48 mm in diameter, according to GOST 10605.

For friction joints of steel structure elements high strength bolts of steel 40X “select” should be used in accordance with the requirements of standard GOST 22356 and other normative documents, while their design and dimention shold be assumed according to GOST 22353. Screw-nuts and washers should be used according to GOST 22354 and GOST 22355.

For flanged joints of steel structure elements it’s recommended to use the high strength bolts of steel 40X “select” with temporal strength of more than 1100 N/mm^2 in accordance with GOST 22356 while corresponding screw-nuts and washers should be used according to standards of GOST 22354 and GOST 22355.

Carbon and Low-Alloy Steels. Depending upon its carbon content, carbon steel is classified as:

– low-carbon soft or mild steel containing from 0.09 to 0.22 %, carbon (used mainly in construction);

– medium-carbon or medium steel containing 0.25–0.5 % carbon (employed chiefly in machine-building);

– high-carbon or hard steel, with a carbon content varying from 0.6 to 1.2 %, used for making tools.

Steel always contains admixtures of manganese, silicon, phosphorus and sulphur whose total content does not generally exceed one per cent. Phosphorus and sulphur are harmful impurities, but they cannot be completely eliminated in the process of steel manufacturing. Phosphorus content of over 0.045 % renders the steel brittle at low temperatures (cold short). Sulphur content exceeding 0.055 % causes the steel to be red short or brittle when heated, i.e., facilitates the formation of cracks in a hot state.

The greater part of structural steel is made by the open-hearth process. Until recently converter (Bessemer and Thomas) steels ordinarily contained a somewhat larger amount of harmful impurities than their open-hearth counterparts. They sometimes contained dissolved gases (nitrogen, oxygen, etc.) which are also harmful admixtures.

At present, owing to the improvements in the process of producing converter steel (the top blowing of oxygen), its widespread use in construction has become possible.

By adding alloying elements such as manganese, nickel, chromium and others, it is possible to produce a high-strength alloy steel. In ordinary construction such steels are seldom used owing to their high cost. In many instances low-alloy steel is employed in the construction of large and critical structures.

The main structural steel is low-carbon steel, obtained by the open hearth or the new converter process, rimming, killed or semi-killed. Its distinguishing properties are poor hardening, high plasticity and good weld ability.

Killed steel advantageously differs from rimming steel in that the solidification of the steel in the ingot moulds takes place quietly, without violent evolution of the gases contained in the steel in large quantities. This is attained by introducing deoxidizing agents such as silicon, aluminium and others that bind the gases ("kill" the steel), forming slag. There is a less possibility of gas bubbles forming in the steel, around which there may concentrate nonmetallic inclusions, for example various sulphur compounds, that have a harmful effect on steel in rolling (exfoliation) and welding, especially in thick plates. Killed steel costs more than rimming steel, and for this reason it should be used only for certain kinds of critical structures, especially welded ones.

There also exists semi-killed steel that has not been completely deoxidized, obtainable in special ingot moulds.

These three kinds of steel used in welded members are as follows:

– killed steel (designated “cn”) — in members designed for a service temperature below — 30 ^{o}C, and also (regardless of the service temperature) in members working in especially strenuous conditions — under dynamic and vibrating loads;

– semi-killed steel (designated “nc”) — in the main load-carrying members of roofing’s and ceilings (trusses, frame collar-beams, beams);

– rimming steel (designated “kn”) — in the remaining instances.

Somewhat more relaxed conditions for selecting the kind of steel to be used are possible for structural members having no welded connections.

Iron and steel works give information on the chemical composition of each melt of steel, which is entered in a special certificate. The maximum contents of chemical elements in steel are established by the State Standard covering the delivery of ordinary quality carbon steel (GOST 380). There are various grades of steel in use, the grade depending upon the contents of various elements.

The main structural steel is grade Ст.3. This steel, owing to the relatively low content of carbon (as a rule less than 0.22 %) and silicon (less than 0.3 %) has good welding properties.

Steel of grade Ст.0. is not manufactured specially, but is obtained as the result of deflection from other grades of steel both for chemical and mechanical reasons, and therefore for it may be used only in the elements of members that are not analyzed.

Open-hearth steel is delivered according to GOST 380*, while converter steel — to GOST 9543, with division into 3 groups, group A having guaranteed mechanical properties, group B-guaranteed chemical composition, and group B with both mechanical properties and chemical composition guaranteed.

Seeing that the same guaranteed mechanical properties and design strength are established, as a rule, for definite groups of carbon steel grades, regardless of the method of manufacturing and the conditions of delivery thereof, these groups for purposes of simplification will be named “steel 3”, “steel 4” and “steel 5”, covering all the grades of steel respectively.

Since structural members require both definite guarantees in respect to mechanical properties, and compliance with limitations in respect to chemical composition, then steel of subgroup B will be the main kind used for them.

Low-alloy steels are delivered according to GOST 5058 and to special specifications and are used mainly in heavy members, as well as in members that find service at low temperatures.

The designation of low-alloy steel grades, consisting of letters and numbers, mainly characterizes the chemical composition of the steel. The left-hand figures indicate the average carbon content in hundredths per cent, the letters after these figures denote the constituents, viz., Г for manganese, С for silicon, Х for chromium, Н for nickel, Д for copper,
Т for titanium, М for molybdenum, etc. The numbers following these letters indicate the approximate percent content of the element (in whole numbers) exceeding one per cent. If the quantity of the constituents is less than 0.3 % they are not indicated in the grade designation.

Low-alloy steels of grades 15ХСНД and lОХСНД are used in construction.

Of the cheaper steels, containing no nickel, grades 14Г2, 15ГС; 10Г2С, 10Г2СД, 09Г2Т, 09Г2ДТ(М) and others are employed. The use of low-alloy steels leads to about 15% reduction in the weight of members. The cost of the members, however, remains practically the same. A considerably greater economy of steel in structural members can be obtained by using heat-treated low-alloy or low-carbon steel, in which the yield point and the ultimate strength are considerably higher with very slight reduction in plasticity. Heat treatment consists in heating the steel to a temperature of 900-930 deg C (above the upper critical point) and hardening (quenching) in water without or with the following tempering.

Aluminium alloys. Aluminium alloys containing various elements such as magnesium, manganese, silicon, copper and others are used in structural members. For brevity we shall call members made of aluminium alloys aluminium ones.

Aluminium alloys are divided into casting ones, used in machine building, and malleable or forging ones (working under pressure—in presses, by extrusion, rolling, stamping, etc.) used in construction.

The alloys are given the required strength not only by adding the appropriate components, but also by mechanical action consisting in cold deformation of the billets—cold working (cold hardening, drawing), while for some of the alloys heat treatment (hardening, ageing, etc.) is employed. The alloys are designated in accordance with the alloying elements as follows:

АМг denotes aluminium-magnesium alloys (in the grade designation АМг6, number 6 indicates that the alloy contains about six per cent of magnesium);

АМц stands for an aluminium-manganese alloy;

АВ (Avial) and АД, are alloys of aluminium with magnesium and silicon;

Д1, Дl6, etc. designate duralumin (the figure indicates the number of the alloy), the basic components of these alloys being aluminium, magnesium and copper;

B means high-strength alloys (B65, B92 and others) consisting mainly of aluminium, magnesium, copper and zinc. These alloys are more costly.

Alloys designated by the letters A, Д correspond to aluminium (A) malleable (Д) alloys of international standard. The figures following the letters show the number of the alloy (АД31, АД33).

The state in which the material is delivered is also designated by letters: M denotes annealed (soft) material, T — hardened and naturally aged (T1 — artificially aged), H — cold worked, П — semi cold worked.

The strength of the alloys can be increased by heat treatment, by 1.3 to 1.5 times, but this will be accompanied by a reduction in the unit elongation. Duralumin's, Avials and high-strength alloys are strengthened by heat treatment. Grade Д16-T duralumin is a strong alloy recommended for riveted structural members, but it is difficult to weld, since annealing of the weld zone takes place; this sometimes leads to the formation of cracks. Duralumin has lower resistance to corrosion.

The heat-treated alloys AB and АД, can be used for welded members provided that welding is followed by heat treatment; this is necessary for increasing the strength of the weld, since after welding this strength will be about 60% of that of the basic metal.

The alloys АМг and АМц are not strengthened by heat treatment. The alloys АМг6 and АД33, owing to their good weld ability and relatively good mechanical properties, have found the widest use in welded members; they have a comparatively high resistance to corrosion. The alloy АМц has low strength, and a high resistance to corrosion, it can be welded and is comparatively cheap. It can be employed for enclosures and guards.

2.2. Principal mechanical properties of steel. Behavior of steel under tension. Brittleness

The most important mechanical properties of steel are its strength, elasticity and plasticity, characterized by stresses and elongations, as well as its tendency to brittle destruction, which in an indirect way is determined by the toughness or impact strength of the material. Strength is determined by the resistance of the material to external loads and forces. Elasticity is the property of a material to restore its initial shape after removal of the external loads. Plasticity is the reverse of elasticity, i.e., the property of a material not to return to its initial dimensions after removal of the external loads or, in other words, the property of obtaining permanent deformation. Brittleness is characterized by destruction of the material upon slight deformation.

The relation between stresses and strains for various materials is established experimentally. The most simple and reliable test is the tension test of specimens, which is used to determine the strength of steel and its elastic and plastic properties. Other characteristics of steel are its notch toughness, as well as the angle when testing for bending in a cold state, which characterizes the state of the surface and is a simple field test of the plasticity and ductility of the material.

The design characteristics of materials and joints. The design strength of rolled steel and pipes should be determined by the formulas represented in table 1.3.1 /10/, where R_{yn} and R_{un} should be assumed equal the guaranteed value of flow limit and temporal strength predetermined by standards and technical specifications.

The reliability factors y_m for mass usage materials are available in table 1.3.2 /10/. The design strength of rolled sheet, wide-strip, structural shapes and pipes are provided in tables E2 and E3 of appendix E /10/.

The design strength of roll-formed shapes should be taken equal to the design strength of rolled sheet, which was used for this shape manufacture. Strengthening of roll-formed shape in the areas of its bending may be possible.

The design strength of circle-, square shape and strip rolled steel should be determined by the formulas represented in table 1.3.1/10/ where R_{yn} and R_{un} values are to be defined according to standards GOST 535 and GOST 19281.

The design strength of welded joints should be determined by the formulas given in table 1.3.3/10/.

The design strength of butt-welded joints, formed from steel components with different design strength, should be taken equal to the smallest design strength.

The strength characterics of weld metal R_{wun} and design strength of fillet weld metal R_{wj} are shown in table Ж2 of appendix Ж/10/.

The design strength of one-bolt joints should be determined by formulas given in table 1.3.4/10/. The design tensile and shearing strengths of bolts and bearing resistance of elements connected by bolts are shown in table Ж5 of appendix Ж/10/.

The desing tensile strengths of foundation bolts $R_{ьa}$ should be determined by formula $R_{ьa}$ = 0,80R_{yn}. The design tensile strengths of foundation bolts are shown in table Ж6 of appendix Ж/10/. The design tensile strengths of U-shaped bolts $R_{ьa}$ should be determined by formula $R_{ьu}$ = 0.85R_{yn} .

The design tensile strengths of bolts within friction joints should be determined by formula $R_{ьh}$ = 0.7$R_{ьun}$, where $R_{ьun}$ is the smallest temporal tear resistance of bolt assumed according to tables Ж4, Ж7 of appendix Ж/10/.

Behaviour of steel under tension. Should a specimen be subjected to tension by gradually increasing the load P, and should the resulting elongations ΔL be measured, the results can be used to plot an experimental tension diagram of elongation versus load. For convenience in comparison with similar charts such a diagram is usually plotted in stress against unit elongation (also called longitudinal strain)

$$\sigma = P/A \quad (\text{kN/cm}^2); \quad \varepsilon = \Delta L/L_0 * 100\ \%,$$

where σ is the normal stress; A is the original cross-sectional area of the specimen; ε is the longitudinal strain in per cent; L_0 is the original length of specimen.

The magnitude of the longitudinal strain depends upon the length and the cross-sectional area of the specimen. Two types of specimens, namely, long and short ones, are in use (GOST 1497).

An experimentally obtained tension diagram for carbon steel 3 is illustrated in Fig. 2.1, *a*.

First the relation between the stresses and the strains follows the law of a straight line, i.e., they are proportional to each other. This is expressed by the linear equation (Hooke's law)

Here E is a constant factor of proportionality, known as the modulus of elasticity in tension, the modulus of elasticity for axial loading or Young's modulus. For steel E = 21000 kH/cm^2.

The highest stress in a material, after which the relation between stress and strain no longer remains linear, is called the proportional limit y_{pr} (Fig. 2.1, *a*). Somewhat higher than this point is the elastic limit y_e corresponding to the maximum stress which can be developed in a material without causing a permanent set.

$$\sigma = E\varepsilon \ (\text{kH/cm}^2).$$

Here E is a constant factor of proportionality, known as the modulus of elasticity in tension, the modulus of elasticity for axial loading or Young's modulus. For steel E = 21000kH/cm^2.

The highest stress in a material, after which the relation between stress and strain no longer remains linear, is called the proportional limit y_{pr} (Fig. 2.1, *a*). Somewhat higher than this point is the elastic limit y_e corresponding to the maximum stress which can be developed in a material without causing a permanent set. When low-carbon steels are loaded above the proportional limit, the curve in the stress-strain diagram for tension moves away from the straight line and, after rising smoothly, makes a jump (forming at high loading speeds a characteristic “tooth”) and then with insignificant fluctuations runs parallel to the *x*-axis (Fig. 2.1, *a*). The specimen grows in length without any increase in load, the material flows or yields. That normal, practically constant stress, which causes an increase of strain without an increase in load, is called the yield point y_y.

The horizontal section of the diagram, called the yield area, is obtained in low-carbon steels for strains varying from 0.2 to 2.5 %. The existence of a yield area in the diagram of a material is a positive factor in the behavior of steel structural members.

In other steels, not low-carbon ones, the transition to the plastic stage takes place gradually, and thus the elastic limit and the yield point in these steels do not in essence differ from each other. In such steels the yield point, which in this instance is called the yield strength, is taken by convention equal to the stress corresponding to the permanent set of 0.2 %.

Upon removal of the load from the specimen after it has received plastic deformation, the unloading diagram follows the straight-line C-D parallel to the straight line of elastic loading (Fig. 2.l, *a*).

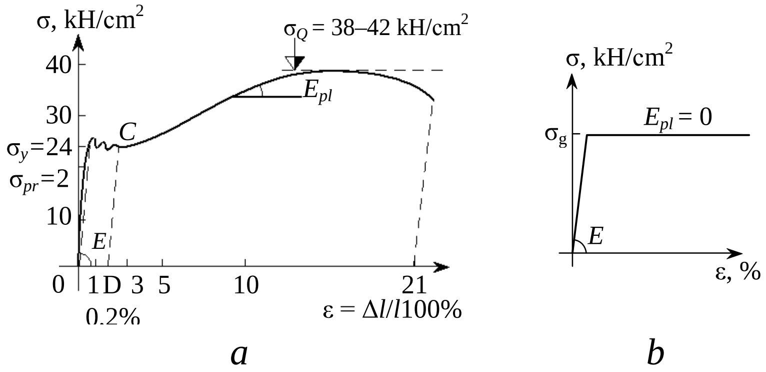


Fig. 2.1. Stress-strain diagram for steel 3

When the strain reaches a certain value (about 2.5 % for steel 3) the material stops yielding and again becomes capable of resisting loads. It behaves as if it has acquired new strength. Here the stress-strain relation will follow a curvilinear law, with a rapid increase of the strains. A neck forms in the specimen and, finally, it completely ruptures.

The maximum stress a material can withstand, which characterizes its strength, is determined by the maximum stress in the process of destruction (referred to the initial cross-sectional area of the specimen). This stress is known as the ultimate stress y_u (ultimate tensile strength).

The total strain, measured after destruction of the specimen, is a measure of the plasticity of the steel.

Since the modulus of plastic deformations E is small (Fig. 2.l, *a*), it is often taken equal to zero in theoretical investigations when considering the elastoplastic behavior of steel, an idealized tension diagram of an ideal elastoplastic material being adopted for all kinds of steel (Fig. 2.l, *b*, Prandtl's Diagram).

Thus the most important mechanical properties characterizing the behaviour of steel are the yield point, the ultimate tensile strength and the longitudinal strain. These properties together with the chemical composition are indicated in the certificates that are supplied with each shipment of metal.

Structure of steel and phenomenon of flow. Low-carbon steel is a homogeneous crystalline material consisting of fine crystals of ferrite, forming grains, and pearlite (a mixture of cementite Fe3C and ferrite) mainly located along the joints of the ferrite grains and forming a sort of a "grid" or disseminations between the grains. Pearlite is considerably harder than ferrite and is more brittle. In the course of elastic strain under the action of externally applied loads, the forces of interaction between the atoms of the crystals change, and as a result the shape of the crystals is distorted. After removal of the loads, the initial shape is restored. In the plastic strain of low-carbon steel tension specimens there can be noticed the appearance of characteristic lines known as flow lines (Chernov and Luders lines) that are directed at the angle of 45 ^{o}C to the line of action of the tensile forces. These lines, which are noticeable to the naked eye, are traces of plastic displacements of the layers of metal that remain after removal of the load. Their direction corresponds in the main to that of the maximum shear stresses.

There exists a hypothesis assuming that since in low-carbon steels the pearlite, which is located at the boundaries of the grains in a comparatively thin layer, is considerably harder than the ferrite; it will first hold back the plastic strain of the latter. However, at the spots subjected to the greatest pressure of the ferrite crystals when they move and turn under loads exceeding the elastic limit, local destruction of the brittle pearlite is possible. When this happens, the energy accumulated by the plastic ferrite will lead to its increased displacement. Upon displacement of a large number of the grains noticeable plastic displacements will be formed. This explains the appearance of a "tooth" and the presence of the yield area in the tension diagram. Thus the yield area is the consequence of the energy being released that had been accumulated in the grains of ferrite and restrained by the pearlite grid.

Owing to the presence of a large number of variously oriented crystals, steel may be considered as a homogeneous material, notwithstanding the heterogeneity of its microstructure.

Upon loading a specimen above its yield point, when the entire yield area is used up (i.e., when the energy accumulated in the ferrite grains is exhausted) the material acquires the ability of further load resistance, and the tension diagram becomes curved, reflecting the uniform development of plastic strains in the whole mass of the metal up to the moment of destruction.

Failure occurs as a result of the accumulation of large plastic shear strains and the development of local shearing stresses that disrupt the atomic bonds. A fine-grain crystal structure can be observed in the fracture.

In case a macro crystalline structure is formed (for instance when the metal is heated to a certain temperature) the yield area is reduced and the yield point lowers, which is explained by the reduction in the total length of contact along the boundaries of the grains with respect to that observed in a fine-grain structure.

In alloy steels and in steels containing over 0.3 % of carbon the pearlite inclusions have larger dimensions and do not form a grid or network surrounding the ferrite grains. For this reason deformation takes place more smoothly, there is no “tooth” in the tension diagram, and the yield area is reduced or completely disappears; the steel becomes harder. Thus a large yield area is characteristic of low-carbon steels with a carbon content varying from 0.1 to 0.3 %.

Behavior of steel subjected to combined stresses. Upon performance of a tensile test, the yield point σ_y is determined for the specimen when it is stressed only in one direction. If a combination of stresses is involved (for example, the combined action of normal and shear stresses in bending) the transition to the plastic state, in accordance with the maximum energy theory of strength, is characterized by the ultimate value of the unit work of deformation of a body (when only the shape, and not the volume, of the body changes under the influence of crystal displacement).

This transition to a plastic state is usually expressed through a reduced or equivalent stress by taking it equal to the yield point found in simple tension.

The unit energy in a combined stressed state of an elementary parallelepiped with dimensions equal to unity in a body, having no shear stresses in its principal planes, and with the principal normal stresses y_1, y_2 and y_3 applied to its sides, will be expressed as follows:

$$U_c = \frac{1}{2}\sigma_1\varepsilon_1 + \frac{1}{2}\sigma_2\varepsilon_2 + \frac{1}{2}\sigma_3\varepsilon_3,$$

in which ε_1, ε_2 and ε_3 are the respective strains.

The condition of plasticity in a combined stressed state, according to the maximum energy theory can be expressed in the form of reduced stress:

$$\sigma_{red} = \sqrt{\sigma_1^2 + \sigma_2^2 + \sigma_3^2 - (\sigma_1\sigma_2 + \sigma_1\sigma_3 + \sigma_2\sigma_3)} = \sigma_y .$$

Thus reduced stress is the stress in such uniaxial stressed state, which in respect to the condition of transition to a plastic state corresponds to the given combined stressed state; in other words, the reduced stress is a quantity equivalent to the yield point in the uniaxial stressed state.

For a plane or biaxial stressed state, for example in a cut out element of a shell, where the normal stresses σ_1 and σ_2 develop in two mutually perpendicular directions, the limiting state will be expressed through the following reduced stress ($\sigma_3 = 0$)

$$\sigma_{red} = \sqrt{\sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2} \le R .$$

In simple bending, when there are two components of the stressed state, namely, normal and shear stresses, for example in a simple beam having the stresses $\sigma_x = \frac{M_x}{W_x}$ and $\tau_{xy} = \frac{QS_x}{I_x t}$, the main stresses σ_1 and σ_2, as is known, will be $\sigma_1 = \frac{\sigma_x}{2} + \sqrt{\left(\frac{\sigma_x}{2}\right) + \tau_{xy}^2}$ and $\sigma_2 = \frac{\sigma_x}{2} - \sqrt{\left(\frac{\sigma_x}{2}\right) + \tau_{xy}^2}$.

Substitution of these expressions in formula (2.3) gives the reduced stress for bending:

$$\sigma_{red} = \sqrt{\sigma_x^2 + 3\tau_{xy}^2} \le R.$$

From this expression, it is possible to obtain the condition of yielding for the maximum possible values of the shear stresses in pure shear, i.e. when $\sigma_x = 0$ (assuming that $R = \sigma_y$): $\tau_y = \frac{\sigma_y}{\sqrt{3}} = 0.577\sigma_y \cong 0.6\sigma_y$.

The shear modulus of elasticity (or simply the shear modulus) G, is taken equal to 8400 kH/cm^2 for steel.

For a biaxial stressed state when three components of this stressed state are present (σ_x, σ_y and τ_{xy}), the reduced stress will be

$$\sigma_{red} = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x\sigma_v + 3\tau_{xy}^2} \le R_y\gamma_c.$$

2.3. Principal mechanical properties of aluminium alloys

The mechanical properties of aluminium alloys depend not only on their chemical composition, but also on the conditions of their machining and working. The principal difference between the behaviour of aluminium alloys and that of steel consists in their being more liable to deformation, i.e., less rigid. In aluminium alloys the modulus of elasticity in tension E = 7100 kH/cm^2, while the shear modulus of elasticity G = 2700 kH/cm^2, which is about one-third of the corresponding values for steel. For this reason, under equal stresses the deflections of aluminium members are three times greater. Poisson's ratio u = 0.3 for aluminium. In tension, diagrams of aluminium alloys there are no yield areas. The yield strength is by convention taken equal to the stress producing a strain of ε = 0.2 %. At temperatures above 100 ^{o}C a certain reduction of the strength characteristics is observed, while beginning from about 200 ^{o}C creep appears. The coefficient of thermal expansion of aluminium is 0.000023, which is twice that of steel. At reduced temperatures, all the mechanical properties of aluminium alloys improve. The notch toughness of the alloys at normal temperature is lower than that of steel and decreases very negligibly at temperatures below the freezing point.

The advantages of aluminium alloys include:

– a comparatively high strength with a low weight of the material itself, which has a great importance for large-span, mobile and enclosing members;

– high workability when subjected to pressing, rolling or forging, which makes it possible to manufacture articles of a complicated configuration;

– high resistance to corrosion;

– the retaining of its high mechanical characteristics at temperatures below the freezing point;

– the absence of sparks when struck with various articles.

The drawbacks of aluminium alloys are:

– a relatively low modulus of elasticity;

– a higher coefficient of thermal expansion;

– relative complexity of making connections;

– short supply and, as yet, high cost;

– comparatively low refractoriness.

2.4. Behavior of material in compression. Problem of stability. Behavior of steel in compression

Steel when subjected to compression in short elements behaves in the same way as in tension. The magnitudes of the yield point, the yield area and the modulus of elasticity are equal to those obtained in tension.

It is not possible, however, to destruct by compression short specimens made of plastic steel and thus determine the ultimate strength of the material, since the specimen flattens out.

An absolutely different picture will be observed in long compressed elements whose length is several times greater than the width of their cross-section (slender elements). In this case the element may lose its load-carrying capacity, i.e., the ability to withstand external forces, not as a result of failure of the material, but owing to the loss of stability (buckling).

Problem of stability. In steel structural members the problem of stability is of very great significance. Underestimation of this factor may lead to disastrous results.

If a straight rod is compressed by the axially applied force *P*, then the rod will initially remain straight, and this state of equilibrium will be stable. The stable state of equilibrium of an elastic rod is characterized by the rod returning to its original position after performing insignificant damped oscillations if it had been loaded and received an insignificant possible deflection due to any cause, after which the action of this cause stopped. This takes place because the external compressive force cannot overcome the resistance of the rod when it has been subjected to insignificant bending (deflection of its axis due to a small disturbing transverse force); the internal elastic work of deformation involved in the bending of the rod, resulting from deflection of its axis (the potential energy of bending ДV), is greater than the external work (ДW) done by the compressive force when the ends of the rod approach each other in bending: ДV > ДW (Fig. 2.2, *a*).

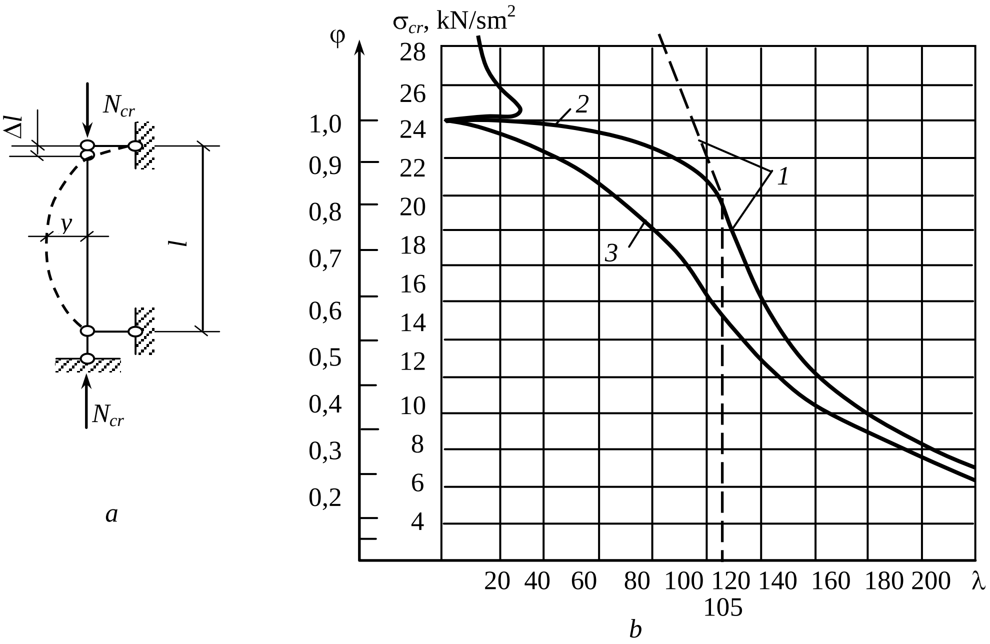


Fig. 2.2. Buckling of axially compressed rods:
a — basic case; *b* — curves of critical stresses and buckling factor tor steel 3;
1 — Euler's curve; *2* — curve of critical stresses with account taken of plastic behavior of material; *3* — curve of factor ц

Upon a further increase in the compressive force it may reach such a value that its work will be equal to the work of deformation in bending induced by any small disturbing factor. In this instance we have ДV = ДW and the compressive force reaches its critical value P_{cr}. Thus a straight rod when loaded with a force up to the critical state has a linear shape in the stable state of equilibrium. When the force reaches its critical value, the linear shape of equilibrium stops being stable, the rod may buckle in the plane of least rigity and it will now have a new curvilinear shape of stable equilibrium.

That magnitude of the force which causes the original stable form of equilibrium of the rod to become unstable is known as the critical force (or load).

If there is a slight initial curvature of the rod (or a slight eccentricity of the point of compressive force application), the rod upon an increase in the load will immediately deflect from a straight position. This deflection is first small, however, and only when the compressive force approaches its critical value (differing from it by not more than one per cent) do the deflections become considerable. This indicates a transition to an unstable state. Thus the unstable state of equilibrium is characterized by great displacements taking place even with a small increase of the load.

A further increase of the compressive force above the critical value calls forth ever-growing deflections, and the rod will lose its load-carrying capacity. It should be noted that different values of the critical load correspond to different end conditions of the rod.

For the axially compressed rod shown in Fig. 2.2, having hinged end constraints (the basic case), the critical load was determined by the renowned mathematician *L*. Euler in 1744 as follows:

$$N_{cr} = \frac{\pi^2}{L^2} El_{\min} .$$

The stress that is induced in the rod by the critical force is known as the critical stress:

$$\sigma_{cr} = \frac{P_{cr}}{A_{gr}} = \frac{\pi^2 El_{\min}}{L^2 A_{gr}} = \frac{\pi^2 Er_{\min}^2}{L^2} = \frac{\pi^2 E}{\frac{L^2}{i_{\min}{}^2}} = \frac{\pi^2 E}{\lambda^2}, \qquad (2.1)$$

where $i_{\min} = \sqrt{\frac{I_{\min}}{A_{gr}}}$ is the minimum radius of gyration; A_{gr} is the gross cross-sectional area of the rod; $\lambda = \frac{L}{i_{\min}}$ is the slenderness ratio of the rod, equal to the ratio between the length of the rod and the minimum radius of gyration of its cross-section.

A glance at expression (2.1) will show that the critical stress depends upon the slenderness ratio of the rod (since the numerator p^2E is a constant quantity), while the slenderness ratio is a quantity that depends only on the geometrical dimensions of the rod. Hence, the possibility of increasing the value of the critical stress by changing the slenderness ratio of the rod (mainly by increasing the radius of gyration of the cross section) is in the hands of the designer and he must use this possibility rationally. Thus the critical stress may also be considered as a parameter that shows whether the section of an element designed for service under compression has been advantageously selected.

The graph of Euler's formula (2.1) is a hyperbola (Fig. 2.2, curve 1).

The critical stresses determined from Euler's formula are true only for a constant modulus of elasticity E, i.e., within elastic limits (more exactly, within proportional limits), and this can take place only with high slenderness ratios (exceeding 105), which follows from expression (2.1)

$$\lambda = \sqrt{\frac{\pi^2 E}{\sigma_{pr}} = 3.14\sqrt{\frac{21000}{20}}} = 105 .$$

Here σ = 20 kH/cm^2 is the proportional limit for steel 3.

The critical stresses for rods (columns, struts) with a small (below 30) and medium (from 30 to 100) slenderness ratios are obtained above the proportional limit but, of course, below the yield point. The theoretical determination of the critical stresses for such rods is rather complicated owing to buckling taking place upon the partial development of plastic deformations and with a variable modulus of elasticity.

As a result of numerous experiments that have confirmed the correctness of the theoretical conclusions, critical stresses have been established for rods with small and medium slenderness ratios, their values being shown in the form of a curve in Fig. 2.2 (section 2).

The load-carrying capacity of a compressed member is also noticeably affected by the local stability of its elements, which depends on the slenderness ratio of the flanges, webs or other elements forming the cross section of the member. The slenderness ratios of these elements are determined by the ratio between their characteristically dimension (width of flanges or depth of section web) and their thickness, viz., b/t or h/t.

Thus, the load-carrying capacity of a compressed element may be exhausted owing to two causes:

– as the result of the stress in the member reaching the yield point (loss of strength);

– owing to the stress in the member reaching its critical value (loss of stability).

The limiting states of compressed rods in respect to strength and stability are expressed by the condition that the stresses in the member should be:

$$\sigma \le \sigma_y , \qquad \sigma \le \sigma_{cr},$$

in which y = stresses in member caused by design loads.

If the ratio between the two limiting stresses is denoted:

$$\varphi = \frac{\sigma_{cr}}{\sigma_y}, \qquad \text{where} \qquad \sigma_{cr} = \varphi \cdot \sigma_{cr}$$

then the second check for stability may be written as follows (the design strength R being taken as the lowest yield point)

$$\sigma \le \varphi R$$

or, in order to simplify the calculations and the comparison of their results, in the form of the working formula

$$\sigma = \frac{N}{\varphi A_{gr}} \le R_y \gamma_c .$$

The factor ц, which reduces the design strength to values ensuring stable equilibrium, is known as the buckling factor. The building standards and regulations establish the values of the buckling factor with regard to the influence of chance eccentricities

$$\varphi = \frac{\sigma_{cr}}{\sigma_y}\frac{\sigma_{cre}}{\sigma_{cr}} = \frac{\sigma_{crc}}{\sigma_y}.$$

In the above expression, y_{cre} is the critical stress of a rod compressed by a force that is applied with a possible chance eccentricity of «σ».

The buckling factor is of interest to designers only when it is less than unity, seeing that otherwise $\sigma_{cr} > \sigma_y$, and the case of the loss of load-carrying capacity due to the lack of strength now becomes dangerous. Since it characterizes the critical stresses, the buckling factor φ is a function of the slenderness ratio of a rod. Curve 3 in Fig. 2.2 illustrates the buckling factor.

Analysis of aluminum alloy members in compression is similar to that of their steel counterparts. The value of the critical stresses and, therefore, that of the buckling factor, depend to a very great extent upon the end conditions of the compressed member. The values of the buckling factor that are tabulated are determined for the basic end conditions of rods, viz., for rods with both ends pinned, (such ends are also called pivoted, hinged or round ends). For other end conditions, the shape of the buckling curve changes, but it can be reduced to the basic case by substituting for the actual length *L* the effective or equivalent length L_e, for which purpose the length of the rod should be multiplied by the length coefficient *k*. Hence, the slenderness ratio of a rod with any end conditions will be determined from the expression

$$\lambda = \frac{L_e}{i} = \frac{kL}{i}.$$

This method of analyzing buckling by using the effective lengths was proposed by prof. F. Yasinsky (1894) who devoted much time to the problems of buckling.

Central tension and compression analysis of steel structure elements. Calculation of solid cross-section elements. Strength analysis of steel elements with $R_{yn} \leq 440$ N/mm^2 for central tension and compression should be calculated by the formula

$$N/(A_n R_y \gamma_c) \leq 1.$$

Calculation for strength of tensile steel members with $R_u/\gamma_u > R_y$, whose operation is possible after the metal reaches its yield point as well as steel members of $R_{yn} > 400$ H/mm^2 should be made by the formula substituting R_y for R_u / γ_u.

2.5. Behavior of material in bending

Limiting state of steel beams in bending for strength analysis. When a beam is subjected to bending (Fig. 2.3, *a*) within elastic limits, a triangular diagram of normal stresses (Fig. 2.3, *b*) will be obtained in the cross sections of the beam. The maximum value of these stresses in the extreme fibres will be determined from the expression:

$$\sigma_x = \pm\frac{M}{W_x} \leq R_y\gamma_c . \tag{2.2}$$

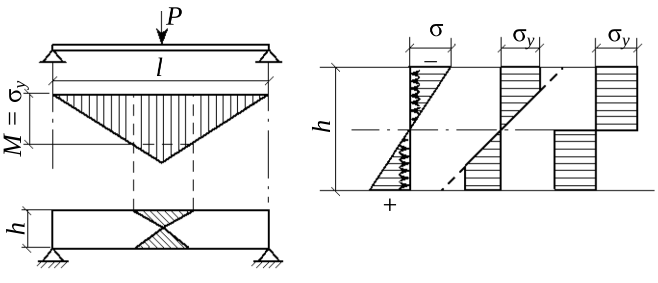


Fig. 2.3. Sequence of stress development upon plastic behavior of beam under bending

Upon an increase in the load the extreme fibres will be the first to reach the yield point, after which the growth of stresses in them will cease (with continuing increase in strains).

Yielding of the material spreads along the depth of the cross section, an elastic core remaining in the middle part of the section (Fig. 2.3, *c*) and the latter will be in an elastoplastic state. The load increase brings up to the yield point all the fibres in the heaviest loaded cross-section, characterized by a rectangular stress diagram (Fig. 2.3, *d*), and at the neutral axis, at the spot where the greatest moment is acting, a so-called plastic hinge will be formed. The spreading of yielding along the length of the beam is shown by the shaded area in Fig. 2.3. Under the influence of such action a large increase in the strains will take place at the location of the plastic hinge, the beam will sag, but will not fail. In this instance, the beam usually loses either its total stability, or local stability of its certain parts. The appearance of a plastic hinge transforms a simple beam into a changeable system. The maximum moment corresponding to the plastic hinge can be determined from the expression

$$M_p = \sigma_y \int_A y dA = \sigma_y 2S = \sigma_y W_p \ ,$$

where W_p is the plastic section modulus, equal to the double statical moment of half the cross-sectional area relative to the axis passing through the Centre of gravity.

The plastic section modulus W_p is somewhat greater than the ordinary (elastic) section modulus W_e. Thus, for a rectangular section and for rolled sections $W_p = cW_e$ (the values of c are given in table 2.1).

Table 2.1

Values of coefficient c for rolled sections

With bending in plane of	I-beams (GOST 8239-89)	Channels (GOST 8240-89)
Web	1.12	1.13
Flange	1.5	1.8

The Building Standards and Regulations allow account to be taken of the development of plastic strains for simple rolled beams constrained to prevent buckling and carrying a statically load. The values of the plastic section module in this instance are taken equal to W_p = 1.12W_e for bending in the plane of the web and W_p = 1.2W_e for bending parallel to the flanges (and not 1.5W_e or 1.8W_e as indicated in table 2.1) in order to reduce the edge strains.

The Building Standards and Regulations also permit account to be taken of the development of plastic strains for constant-section welded beams with a ratio between the width of the compressed flange overhang and the flange thickness of , and a ratio between the depth of the web and its thickness of (for steel 3).

At spots with the greatest bending moments, large shearing stresses are not tolerated. They should comply with the inequality.

When a long zone of pure bending is present, the corresponding section modulus, in order to avoid excessive strains, is taken equal to half the sum of the elastic and plastic moduli, viz., 0.5 (W_e + W_p).

In continuous beams, the formation of plastic hinges is taken as the limiting state, but on condition that the system remains unchanged.

In the analysis of continuous beams (rolled and welded) that comply with all the limitations indicated above for simple beams the Building Standards and Regulations permit the design bending moments to be determined on the basis of equalization of the support and the span moments (provided that adjacent spans differ by not more than 20 %).

In all cases when the design moments are determined on the assumption of developing plastic strains (equalization of the moments) the beam strength should be checked in accordance with formula (2.2) using the elastic section modulus. In analyzing beams made of aluminum alloys the development of plastic strains is not taken into consideration.

Limiting state strength analysis of steel. Beams in bending under simultaneous action of moments and shear forces. With simultaneous action in a beam subjected to bending of normal and shearing stresses
σ and ф, the transition of the beam to a plastic state is expressed through the equivalent or reduced stress, which establishes the relation between σ and ф at the moment when yielding reaches the extreme point, or any other point of the cross-section.

Until a plastic hinge is obtained, however, the beam being bent can still be additionally loaded. In this instance if large shearing stresses
($\tau > 0.4R$) are present yielding begins to spread over the cross section more rapidly than if only normal stresses were acting. Obviously, if the shearing stress ф grows, the normal stress should decrease, inasmuch as the equivalent stress must not be greater than the design strength R. The relation between y and ф, or between the corresponding values of M and Q acting together, at which a plastic hinge is obtained, has not been precisely determined for various cross-sections.

The ultimate equivalent or reduced stress in a beam corresponding to the moment when a plastic hinge is formed.

$$\sigma_{h.red} = \sqrt{\frac{3}{4}\sigma^2 + 3\tau^2\left(1 - \frac{1}{2}\frac{\sigma^2}{R^2}\right)} \le R_y\gamma_c\,, \qquad (2.3)$$

where, y is the maximum absolute value of the boundary stress in the web of the beam in elastic behavior of the material; $\tau = \dfrac{Q}{h_w t}$ is the mean shearing stress in the web of the beam.

Checking of the reduced stress in the webs of beams with the aid of expression (2.3) is required when τ is greater than 0.4 *R*.

2.6. Behaviour of material in eccentric tension and compression

Eccentric tension. If the axial tensile force F and the bending moment M act simultaneously on an element, or if it is placed in tension by a force applied eccentrically with respect to its axis, the strength in the elastic stage is checked by means of the expression

$$\sigma = \frac{N}{A_n} + \frac{M_x}{W_{n*x}} = \frac{N}{A_n}\left(1 + \frac{e_x}{\rho_x}\right) = \sigma_a(1 + m_x) \leq R_y\gamma_c\,,$$

where, A_n is the net cross-section, cm^2; W_{n*x} is the net section modulus, cm^3; $e_x = \frac{M_x}{N}$ is the eccentricity of force application, cm; $\rho_x = \frac{W_x}{A}$ is the core (kern) radius of cross-section, cm; $\sigma_a = \frac{N}{A_n}$ is the axial stress induced by longitudinal force; $m_x = \frac{e_x}{\rho_x} = \frac{M_x}{N}\frac{A}{W_x}$ is the relative or unite eccentricity.

When account is taken of the behavior of a material in the plastic stage (which is an owed by the Building Standards and Regulations for welded and rolled elements of steel members that are not directly subjected to dynamic loads), the load-carrying capacity of the element is higher than when only its elastic behavior is taken into consideration. The limiting state for an eccentrically tensioned element which the moment *M* and the force *N* are applied to can be determined in the same way as that for a beam in bending when the moment *M* and the force *Q* are applied to it. For this purpose it is necessary to determine the relation between *M* and *N* at which a plastic hinge will be obtained, and thus find the boundary lines separating the elastic, elastoplastic and plastic areas of behavior of the material in a member.

For I sections the boundary lines (surfaces) are nearer to the straight line (plane); for this reason the Building Standards and Regulations, to ensure a margin of safety, propose to compute the strength of solid-web eccentrically tensioned elements that are not subjected to the direct action of dynamic loads by the formula

$$\left(\frac{N}{A_n R}\right)^{3/2} + \frac{M_x}{W_{x*p} R} \leq 1.$$

Eccentrically compressed and bent elements. Upon the application of a compressive force with certain eccentricity, an element is subjected to eccentric compression. Bending also appears together with this compression and, consequently, deflection (Fig. 2.4, *a*). When a longitudinal axial force and a lateral load inducing bending are applied together, the element will be both compressed and bent (Fig. 2.4, *b*). The behavior of elements of these two types differs somewhat from each other, mainly in the limiting state at low slenderness ratios. Nevertheless (with an increase of the margin of safety) compressed and bent elements, when considering their critical states, are equaled to eccentrically compressed ones with the eccentricity of $e = M/P$.

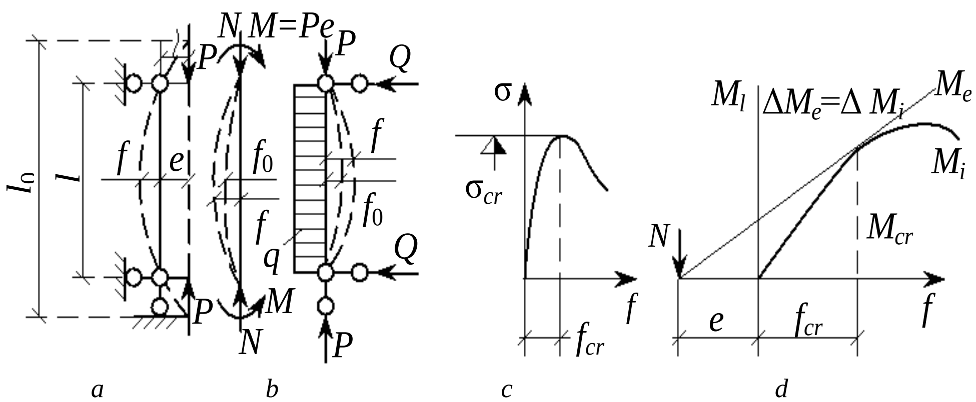


Fig. 2.4. Behaviour of eccentrically compressed element

Eccentrically compression, notwithstanding the existence of deflection from the very beginning of load application, also relates to the problem of stability, and not only to that of strength; moreover, axial compression is in essence a particular case of eccentrically compression with small chance eccentricity. Indeed, the load-carrying capacity of each element (it should not be forgotten that the loss of stability is always connected with the loss of load-carrying capacity) is characterized by only one value of the strain corresponding to each value of the load applied to the element. In the given instance with a growth of the force P (Fig. 2.4, *a*) the external moment will also increase owing to greater deflection, viz., $M_e = P(e + \delta)$. As soon as the stress in the extreme fibre of the bent element at one side of the section passes the yield point, then in accordance with the condition of equilibrium in elastoplastic bodies, upon an increase of the moment (and the deflection) the normal force should decrease, i.e., the value of the force as a function of the deflection will pass through its maximum and begin to diminish.

The stress-strain curve shown in Fig. 2.4, *c* has a rising line of stable equilibrium until the critical value (critical stress) is reached, after which the strains begin to grow without any increase of the load. Further retaining of equilibrium is possible only upon reduction of the load, and it is this that characterizes the loss of stable equilibrium

$$\frac{d\sigma}{d\delta} = 0 .$$

Upon the loss of stability, equilibrium is disturbed between the external force factors and the internal forces which develop inside the section of the element, the latter having a certain possible curvature.

The equation for checking the stability of an eccentrically compressed element can be written as follows:

$$\sigma = \frac{N}{\varphi_{ec} A} \le R_y \gamma_c ,$$

where, φ_{ec} is the factor that reduces the design strength for the given element to the value of the critical stress for the eccentrically compressed element.

The values of the factor φ_{ec} are given against the slenderness ratio of the element λ and the equivalent eccentricity

$$m_{ef} = \eta m = \eta \frac{e}{\rho} = \eta e \frac{A}{W}, \tag{2.4}$$

where $e = \frac{M}{N}$ is the eccentricity of force in the plane of bending, cm; M is the design moment, which is equal to the maximum moment within the limits of the column length for columns of frame systems with a constant section, to the maximum moment over the length of the part with a constant section for stepped columns, to the moment in the constraint for cantilevers; N is the longitudinal force in section under consideration; $\rho = \frac{W_{gr}}{A_{gr}}$ is the core radius of section, cm; W_{gr} is the section modulus for the most compressed fibre; $\lambda_{ef} = \lambda\sqrt{R/E}$ is the conditional slenderness ratio; η is the coefficient of depending of form cross-section; $m = eA / W_c$ is the relative eccentricity.

With values of $m > 4$ in addition to the standard formula (2.4), eccentrically compressed elements may be analyzed by means of Yasinsky`s binominal expression

$$\sigma = N/\varphi A_{gr} + M/W = N/A_{gr}\,(1/\varphi + e/\rho) \le R_y\gamma_c,$$

which doesn`t take complete account of the development of plastic strains and entirely omits the influence of section shape.

The inaccuracy of this expression is insignificant (5…8 %)

Overall stability analysis of solid cross-section elements of steel structures. Stability analysis of eccentrically compressed elements under the action of bending moment in one of the main planes should be carried out for both the plane form of buckling mode (in the plane of bending moment action) according to 1.6.2.2 i 1.6.2.3/10/ and the flexural-and-torsional form of buckling mode (out of the plane of bending moment action) according to1.6.2.6–1.6.2.8/10/.

Stability analysis of eccentrically compressed elements, having constant thickness of cross-section within the plane of bending moment action along the structure and when the plane of bending moment action coincide with its plane of symmetry, should be made by the formula:

$$\frac{N}{\varphi_e A R_y \gamma_c} \leq 1.$$

The stability factor for eccentrically compression should be defined by the table К.3 of appendix К/10/ depending on the values of conditional flexibility and reduced relative eccentricity m_{ef}, determined by the formula:

$$m_{ef} = \eta\, m,$$

where η is the section shape factor, defined by table К.2 of appendix К 10/; $m = eA / W_c$ is the relative eccentricity; $e = M/N$ is the eccentricity, where values of internal forces M and N should be taken according to the requirements 1.6.2.3/10/.; W_c is the moment of cross-section resistance, calculated for the most compressive fiber.

Methodical instructions to chapter 2

It is important to learn about the materials used in metal members – carbon and low-alloy steel, aluminum alloys.

It is necessary to learn chemical content, mechanical and technological characteristics of constructional materials.

Main attention should be paid to principal behavior of steel under tension, compression, bending, eccentric tension and compression.

Students should also learn the theoretical basis of calculation and how to write formulas. It is necessary to get acquainted with the structure of additional tables aimed at faster calculation in solving engineering problems.

Questions and Tasks to Chapter 2

1. *What chemical elements does steel consist of?*
2. *Enumerate mechanical and technological characteristics of steel.*
3. *What are the chemical content and symbols of carbon steel grades? How carbon is steel made?*
4. *What are the chemical content and symbols of low-alloy steel grades? How low-alloy is steel made?*
5. *Name the grades of aluminum alloys, which are used in constructions.*
6. *How do we calculate the total stability of elements in central compression according to Euler`s formula? In accordance with Building Standards?*
7. *What is the plastic hinge?*
8. *How do we calculate local stability of elements of cross-section members?*

3. STANDARD CLASSIFICATION OF ROLLED SECTIONS. CONNECTIONS OF ELEMENTS OF STEEL MEMBERS

Steel is employed in construction in the form of rolled stock delivered from iron and steel works and having various sections to meet construction requirements. There is distinguished sheet steel and section steel.

Many years of experience in the employment of various shapes have evolved products that give the best results from the viewpoint of erection and economy. These shapes are fit for different types of members in the diverse conditions of erection. They include angles, 1 section and channels. These three shapes together with sheets, plates and round bars are the main ones used in construction.

Catalogues are published by iron and steel works showing the standard structural steel products available.

3.1. Main sections and their applications

Angles. Two kinds of angles are in use, namely, equal-flange or equal angles (GOST 8509–93) and unequal-leg or unequal angles (GOST 8510–86*) with a ratio of their legs of about 1.0 to 1.6.

The great variety of applications of angles has resulted in a considerable development of their assortment, from the smallest and lightest sections with leg sizes of 20×20 mm and a thickness of 3 mm, designated 20×3, to the largest and heaviest section 250×30 in size.

To facilitate designing, connection and splicing of angles their legs have parallel faces.

The angle section should be selected with a view to the general design of the element section in which it is used.

In those cases when angles are to be subjected to axial forces as an independent section forming part of a truss or strutted member, it will be a good plan to select the thinnest angles, seeing that the comparative measure of the fitness of a compression element is its radius of gyration $i = \sqrt{\frac{I}{A}}$. The latter affects the buckling factor φ which, in turn, reduces the design strength. The greater the value of *i*, the more advantageous will be the section, and it must not be forgotten that the radius of gyration of thin angles is always greater than that of their thicker counterparts. When employing thin angles, local bulging of the legs should not be feared, since the maximum ratio of the width of a leg to its thickness (the slenderness ratio of the leg) used in standard sections, i. e., $b/t \le 15 \ldots 17$, will always ensure local stability of the legs. For the tension elements of welded members the thickness of the angles is less important, but in this instance thin angles are also generally used because of their high rigity, which simplifies transportation and erection.

The angles 50×5 and 63×40×5 are taken as the minimum sections for the main elements of load-carrying steel members.

The ordered (marketed) length of angles, depending upon the conditions of rolling and transportation, is taken equal to 6–9 metres for small sections and to 9–12 metres for their large counterparts. This length determines the location of the joints or splices.

I beams. I beams (GOST 8239–89) are mainly employed to resist bending, and this determines their configuration. I-shaped elements are also used in the built-up sections of columns**.**

Standard I beams are designated by numbers ranging from 10 to 60. The number of the I beam corresponds to its depth expressed in centimeters. Beginning with No. 18 and up to No. 30 inclusively, I beams are rolled with different widths and thicknesses of their flanges, the web remaining the same. I beams with thicker and wider flanges are distinguished by adding the letter “*a*” to their number. It will be more advantageous to use these shapes in beams subjected to bending.

I beams can be ordered in lengths of 6, 9 and 12 metres, but upon agreement with the manufacturers I beams from #30 can be delivered with the length of 15 metres and from #45 and above with the length of 15–18 metres.

Channels. A channel (GOST 8240–89) differs from an I beam in the web being shifted to the edge of the flanges, which is convenient for connection to the webs of other elements. Channels can be used as elements resisting bending and that is why they are used, for instance, as purlins in the roofings of industrial buildings. Owing to the absence of symmetry of the section with respect to the vertical axis *y*–*y*, a channel, as a rule, is subjected to torsion and additional stresses appear in it.

Channels are widely employed in members subjected to axial forces in the form of built-up sections of two elements connected by batten pla-tes or lattices, for example, in columns, the chords of trusses, and so on.

The standard channels are designated by numbers ranging from 5 to 40.

Channels #14–24 are rolled with different thicknesses and widths of their flanges, the web remaining the same. The letter “a” is used to designate the channels with the larger flanges. The allowances in depth are ±3 mm for channels from #20 to #30 and ± 3.5 mm for #33 and above.

The channels have a slope of 1 in 10 on the inside faces of the flanges. These shapes can be ordered in lengths of 6, 9 and 12 metres, while upon special agreement with the manufacturer this length can be increased to 18 metres.

Steel sheets and plates. Sheets and plates are widely employed in construction, often constituting from 40 to 60 % of the total weight of a structure, and about 100 % in special plate structures — shells. The cause of such widespread employment of sheets and plates even in framework or skeleton systems is the unlimited possibility of producing any sections of the required dimensions, thickness and configuration by welding or riveting together the separate plates.

Sheet steel is classified as follows:

– hot-rolled steel sheets (GOST 19903) having a thickness ranging from 0.4 to 4.05 mm, width from 600 to 1000–1400 mm (depending upon the thickness), a length from 1.2 to 2; 2.5; 4 metres. They are used in construction for stamped sections and roofing;

– steel strips (GOST 103) having thickness varying from 4 to 60 mm and width from 11 to 200 mm with 2 mm gradation. It is good policy to use this steel only when a big ordere is made;

– general purpose steel plates (GOST 82) having smooth and even edges. The plates have a width ranging from 200 to 1050 mm with 10–50 mm gradation, their thickness is 6 to 60 mm with 2–4mm gradation beginning from thickness of 12 mm. The usual length ordered vary from 6 to 12 metres. The general purpose steel plates having even edges, their employment will be very rational;

– steel plates (GOST 19903) having thickness ranging from 4 to 160 mm with 0.5 mm gradation for thickness from 4 to 6 mm, 1 mm gradation for thickness from 7 to 30 mm and 2 mm gradation for thickness from 31 to 40 mm. The width of steel plates varies from 600 to 2500 and even to 3000 mm with 100 mm gradations. It should be remembered that an agreement must be reached with the manufacturer before ordering plates over 1800 mm wide. The length of the plates, depending on their thickness, is 4 to 8 metres, and up to 12 metres upon agreement with the manufacturer.

3.2. Various sections employed in structures

Besides the principal sections listed above, a variety of other sections and shapes are employed in structures, namely:

– square steel bars (GOST 2591–88*) with straight and beveled edges and with sides ranging from 6 to 200 mm;

– round steel bars (GOST 2590–88*) with a diameter from 5 to 200 mm, employed for the elements of braces, stay bolts, etc., as well as for the reinforcement of concrete members;

– high-strength carbon steel wire for prestressed members (GOST 7348) with the diameter of 2.5 to 10 mm; rope wire, bare and galvanized (GOST 7372) with the diameter of 0.2 to 5 mm, and so forth;

– seamless steel thick-walled tubes (GOST 8732) with the diameter and thickness of 42×2.5 up to 377×20 and 500x15 mm;

– crane rails (GOST 4121) for crane tracks. The nominal width of the head of the rail in millimetres determines the number of the rail (from KP70 to KP140). The length of these rails ranges from 9 to 12 metres;

– fluted steel sheets (GOST 8568) are used for the landings and treads of staircases. The sheets (including the flute) of 6, 8 and 10 mm thickness, 500–1400 mm width and up to 6 metres lengths can be ordered;

– corrugated sheet steel (GOST 3685) is employed as cold roofing in hot shops. The thickness of this steel ranges from 1 to 1.75 mm, the width from 700 to 1000 mm, the length is 2,000 mm and the depth of the waves or corrugations is 30 and 35 mm;

– special sections of metal sashes for glazing industrial and certain civil buildings (GOST 7511).

In designing structures the trend should be to use the minimum guantity of various section numbers without permitting however, a noticeable increase in the weight of the members. Notwithstanding the relatively large list of standard rolled sections, the iron and steel industry does not always meet the requirements of construction. Hence it becomes necessary to weld or rivet shapes not available on the market from plates and other sections, which increases the cost of members. To meet more completely the requirements of construction, the following sections are produced, some of them already being made:

1. Wide flange (WF) beams with the depth up to 1000 mm and the ratio between the width of the flanges and the depth of the beam ranging from 1 : 1 to 1 : 2.5 (GOST 26020–83). There has been organized a specialized production of welded wide flange beams.

2. Thin-web I beams for light girder members resisting bending, with a minimum web thickness (TU 14-2-205). Such sections show 30–35 % economy over I beams of the normal types.

3. T beams with a long or short web and wide flanges, which can be employed in the capacity of independent elements of truss chords, in the

form of an *H* shape with one weld or as part of a welded I shape with a large depth having a plate inserted between two T beams.

4. Sections made of plates or sheets manufactured by bending on special bending machines or by cold rolling. It will be expedient to use these sections in the elements of light members, as well as in members that withstand insignificant loads, but are required to have high rigity. Such sections can have a great variety of shapes (GOST 8278 to 8282).

3.3. Aluminium alloy sections

Deformable aluminium alloys in the form of various sections or sheets are the main ones used in construction.

The sections are manufactured either by extrusion, or by bending or stamping rolled sheets.

Extrusion is performed in a hot state (about 400 ^{o}C) in hydraulic presses, in which a round ingot (with maximum dimensions of $d \times L$ = 345 × 1450 mm) is extruded through a die in which a hole of the required section has been made. In view of the size of the ingot indicated above, it should be possible to inscribe the cross-section of the shape in a circle with the diameter of 320 mm, and in some instances, when a powerful press is being used, in a circle with the diameter of 530 mm. When a set of dies with different configurations of the holes is available, a great variety of extruded sections can be obtained. The relatively small amount of aluminium members used does not make it possible to establish a catalogue of the most widely employed sections. Owing to the low modulus of elasticity of aluminium alloys and the small critical ratio between the width of the free overhang of angle legs and their thickness $\frac{b}{t} \le 7...12$), angle shapes have been proposed with bulbs on the ends of the legs.

This increases the $\frac{b}{t}$ ratio to 10...20 depending upon the shape of the section. The sections can have closed shapes, for example in the form of tubes.

The State Standards have been established for a number of small shapes with sectional dimensions up to 120 mm (GOST 8617–81* to 22233-93) and for pipes (GOST 18482).

After extrusion the sections are subjected to heat treatment such as hardening, ageing, annealing, etc.

Bent sections are manufactured mainly from thin sheets and strips up to 4 mm thick. The sheets and plates are rolled in cold (with the thickness up to 5 mm) or hot (with the thickness up to 80 mm) state with the width up to two metres and the length up to seven metres (GOST 21631). Sheets and plates of duralumin type aluminium alloys to be used in structural members are plated, i. e., coated with a thin layer of pure aluminium to prevent corrosion. Sheets and plates of alloys that are not strengthened by heat treatment, of alloys AB and B92, as well as sections made of alloys of all grades, may be used without plating.

Sheets employed for fencing or for decorative purposes may be given various colours with inorganic pigments by anodizing, which involves a number of electrochemical processes of treating the surface of the sheet.

3.4. Welding processes

Electric welding is the most widespread method of connecting the elements of steel members. There are distinguished manual, automatic and semi-automatic arc and gas-shielded arc welding.

Manual arc welding. Arc welding is based on the phenomenon of an electric arc bridging the gap between a steel rod (electrode) and the steel parts being welded when they are connected to a source of current. This arc melts and fuses the parent or base metal and the metal of the electrode and forms a weld that connects the original parts into one piece.

Welding with a bare electrode and an unshielded arc leads to the formation of low-quality welds, since oxygen and nitrogen get into the weld metal and form oxides and nitrides. This makes the weld brittle. In order to ensure the formation of a shielding gas medium and to improve ionization of the air gap use is made of coated electrodes. Part of the coating forms a flux on the molten metal that deoxidizes and purifies the latter. A slag crust is also formed that protects the hot surface of the weld against contact with the air and prevents oxidation. Initially the coatings of electrodes consisted of chalk dissolved in water glass (lime-or wash-coating) which was applied to the electrodes in a thin layer (thin- or lightly-coated electrodes). The quality of the welds, however, remained low. At present special high-quality coatings are in use that are applied to the welding wire (heavy-coated electrodes). These coatings contain various alloying additions that improve the mechanical properties of the welds.

The amount of carbon in the metal has a great influence on the weldability of structural steels. For welding steels containing over 0.22 % carbon, electrodes with special coatings should be used. Ordinary heavy-coated electrodes can be employed for welding the common grades of structural steel having a carbon content of less than 0.22 %.

Owing to the great variety of high-quality coatings, the electrodes are distinguished not by the composition of the coatings, but by the results of mechanical tests of the deposited metal and the welded connection.

The main types of electrodes employed in construction are as follows: type E42 for welding members made of steel 3 and type E42A for heavy-duty members; types E50A and E55, which ensure a high-quality weld, for welding members of low-alloy steel. The use of type E34 electrodes in elements and parts whose strength must be checked in designing is excluded because of the low plasticity and brittleness of the weld.

The strength of welded connections is appreciably affected by the structure of the weld, as well as by nonmetallic inclusions (entrapped slag or fine gas pockets that are formed when the weld cools). The presence of internal micropores leads to a dimensional concentration of stresses, thus increasing the brittleness of the weld.

Hot cracks sometimes appear during cooling of the weld within a temperature range of 1000 to 1350 ^{o}C. Cold cracks in the majority of cases are the result of tensile stresses induced in the welds by shrinkage.

The structure of a weld and the temperatures encountered over its cross-section. Here three zones are distinguished, namely, I — parent or base metal zone, II — transition zone (also known as dilution or fusion zone) and III — weld-metal (deposited metal) zone.

The base metal zone is that part of the base metal around the weld heated at most to the critical temperature (723 ^{o}C), wherein the metal retains its mechanical properties.

The fusion or heat-affected zone is located between the base and the weld metal. During welding a sharp change of temperature is observed in this zone, from 1500^{o} (melting point) to 723 ^{o}C. The structure of the metal in this zone is not uniform. In the part with temperature exceeding 1000–1100^{o} the crystals grow in size, forming a coarse-grain structure with poorer mechanical properties. The fusion zone is the weakest part of the weld.

In welding penetration of the weld metal into the base metal takes place, and the deeper this penetration, the higher the quality of the weld. The usual depth of penetration is 1.5–2 mm. Of special importance is deep penetration at the root of fillet welds in members taking alternation loads.

The existence of a gap caused by an unfinished plate edge creates a sharp change in shape that produces stress concentration and the formation of the most minute cracks. They gradually develop under an alternating load, and may be the cause of failure of the weld.

Deep penetration is achieved by means of automatic welding.

Automatic and semi-automatic welding. Automatic submerged or hidden arc welding is based on the principle of automatically feeding an electrode or welding wire into the welding zone with continuous unrolling of a reel of special welding wire. The welding wire is fed by an automatic head that performs the functions of the welder's hand in manual welding. Instead of the coating, use is made of loose material having a definite chemical composition (flux), which is spilled over the end of the electrode. The flux completely isolates the welding zone from the air, since it hides the arc. The result is a homogeneous compact weld with deep penetration that possesses high mechanical properties.

Owing to the heavy current used in automatic welding (600 to 3000 amperes) the output is from three to five times (and sometimes even scores of times) greater than that obtained in manual welding. When designing members that are to be connected by automatic welding, such layouts should be employed that will not hinder the movement of the welding head. Automatic welding can be performed on a stationary installation or with the aid of welding tractors. This method is generally used to make underhand welds, employing revolving jigs for the purpose.

To weld elements that are over 24 mm thick a method termed electroslag welding is employed, which can be used to weld parts with a great thickness (up to 200–300 mm). The current flows through the molten flux (slag) with the liberation of heat sufficient for melting the base metal and the electrode. This method is designed for making vertical welds.

Where it is difficult to employ automatic welding procedures, use can be made of semi-automatic submerged arc welding with a hose-fed wire. In this method, a thin welding wire two millimetres in diameter is fed to the welding zone mechanically along a flexible hose, while movement along the weld is performed manually. The flux is supplied directly from the hopper of the holder, the latter also carrying the control buttons.

The near future should see the introduction of semi-automatic “hose” welding where the filler metal electrode is replaced with a thin tube containing special powder (powder electrodes). The powder performs the functions of flux. This method of welding is especially convenient for field work.

Gas-shielded and welding. Among the various methods of arc welding increasing preference is being given to gas-shielded arc welding, using carbon dioxide and argon as the shielding gases.

Carbon dioxide is employed in welding low-carbon steel. The gas is fed through a special nozzle and flows around the melting electrode wire, which can be fed automatically.

The advantages of this method of welding are its high output and low cost, and also the deep penetration. Owing to the high temperature of the stream of shielding gas heated by the arc, the metal cools slowly, and a sound weld is obtained. The possibility of automation of the welding process without the use of fluxes makes this method a very promising one.

Argon is used mainly in welding aluminium members, as well as thin sheets of high-alloy stainless and heat-resisting steels.

Argon-shielded arc welding is performed with the aid of a nozzle containing a nonconsumable tungsten electrode that generates and maintains an electric arc. The shielding gas — argon, surrounds the lower end of the electrode. Filler metal in the form of wire is introduced into the weld.

When welding aluminium members it is also possible to make use of consumable electrodes.

Spot welding. At present spot welding is not employed to any noticeable extent for connecting steel structural members, it does not ensure stable strength of the weld when connecting thick parts.

Gas welding. The combustion of acetylene in the oxygen stream produces high temperature (3 200 ^{o}C) which causes melting and fusion of the base and filler metal, the latter having the form of wire that is introduced into the gas flame. Gas, or torch, welding yields a low output and is therefore seldom used, mainly for repair jobs.

Welding of aluminium members. For a long time aluminimum members did not lend themselves to welding, since the molten aluminium readly combines with oxygen and forms an oxide film on the drops of molten metal that hampers fusion. When welding is performed under a blanket of an inert gas, for example argon, such a film does not form. Welding can be carried on using a nonconsumable tungsten electrode with a filter of aluminium wire, which is rational for sheets from 1 to 6–8 mm thick. It will be good practice to use a consumable electrode in welding sheets and plates from 6 to 40 mm thick, or still better to weld automatically with an aluminium wire 1.5–3 mm in diameter. Manual arc welding is also employed with a special electrode coating that is able to combine chemically with the oxygen of the oxide film or dissolve it in molten salts.

Good weldability of aluminium members will be ensured by chemically pure elements and wire, as well as proper preparation of the surfaces to be welded (degreasing, pickling and cleaning). The welding of aluminium members requires strict adherence to the established technological conditions, since failure to do this will lead to reduction in the quality of the welds.

3.5. Design strengths of welds

The compressive and shear design strengths of butt welds made of type E42 and E42A electrodes in members of steels 3 and 4, and of type E50A and E55 electrodes in members of low-alloy steel, are taken equal to the design strengths of the base metal. Different values of the tensile design strength of these welds are taken depending upon the method of welding and the requirements which the quality of the electrodes, the weld and its control must meet.

If a butt weld is made by automatic welding, or by manual or semi-automatic welding with the use of high-quality electrodes, additional high-class methods being employed to control the quality of the welds (*x*-ray, gamma-ray,

electromagnetic methods, ultrasonic control, etc.), then the design tensile strengths are taken the same as for the base metal. Such connections requiring special attention may be made only in the critical parts of members. With ordinary methods of weld quality control (visual inspection, measurement of the welds, test drilling, hydraulic tests or testing with kerosene, and so on) the design tensile strengths for butt welds made by manual or semi-automatic welding are lower than those for the base metal.

Designing of steel structure joints. Welded joints. Designing of steel structures with welded joints needs observing the following requirements:

– minimizing the necessary quantity of welded seams and their lengths;

– insuring easy access to places of welding joints taking into account the type and welding technology.

The main types, structural components and dimentions of welded joints should be taken according to the standards: GOST 5264, GOST 8713, GOST 11533, ГGOST 11534, GOST 14771, GOST 14776, GOST 23518.

For selection of electrodes or welding wire it is recommended to take into account the groups of structures represented in appendix B/10/.

The dimention of fillet welds and design of joints have to correspond to the following requirements:

a) leg of fillet weld k_f has to insure the calculation requirements and be greater than values defined in table 1.12.1/10/; leg of fillet weld of *T*-joint, lap joint or angle joint may be less than specified in table 1.12.1 but greater then 4 mm taking into account that weld dimention has to insure its load-bearing capacity determined by the calculation. Absence of defects including technological crack has to be tested by production inspection group.

b) leg of angle joint k_f should not exceed 1.2t, where t is the smallest thickness of welded elements; leg of weld going along rounded edge of structural shapes should not exceed 0.9 t where t is the thickness of rolled steel;

c) calculated length of angle joint weld should be greater then 4 k_f but not less then 40 mm;

d) welding conditions should be taken so that to insure following relations: $b/h \geq 1.3$ for angle joint weld; $b/h \geq 1.5$ for butt one-pass weld;

e) calculated length of longitudinal fillet weld should be less then 0.85 $betta_f * k_f$ except for the weld where stresses acts along its hole length. $betta_f$ is a coefficient assumed according to table 1.12.2/10/;

f) overlapping should be more than 5 thicknesses of the thinnest element of welded joint;

g) legs relation of fillet weld should be assumed, as a rule, 1:1; when thickness of welded elements is different its possible to use welds with different legs; in such case the leg adjoining the thinner element should insure the requirements 1.12.1.5 б/10/, while the leg adjoining a thicker element should meet the requirements 1.12.1.5 a/10/;

h) when welded joints of elements are covered by plates, the side fillet welds should not come to the joint axis less than 25 мм;

i) the fillet welds for structures of 1 and 2 groups should be carried out, as a rule, so that to insure graded transition to the steel of welded element without its strengthening;

k) the distance between parallel welded joints of structural elements should be more than 10t or 100 mm, where is the thickness of element.

3.6. Types of welded joints

Investigation of joints for action of axial forces. Classification of welded joints and welds. In the design of welded joints there are distinguished butt joints, lap joints, composite joints, tee joints and corner joints.

Welds are classified according to a number of features, namely:

– with respect to their position on the members being connected — into underhand, vertical, horizontal and overhead welds;

– according to the design of the weld-into butt and fillet welds, the latter with respect to their arrangement relative to the acting forces being divided into side and end (edge) welds;

– according to the method of machining the edges of the plates — into single-V, double-V, double-level, X, K and U groove welds;

– with respect to length — into continuous and intermittent welds;

– as regards the number of layers applied during welding — into single-pass and multipass welds;

– according to their purpose — into working (force-transmitting) and constructive or connecting welds.

Generally the size of welds predominant, as well as the type of electrodes, is stated in the note located near the title block (name of drawing) in the lower right-hand corner of the sheet. Only those dimensions that differ from the predominant ones are shown directly in the drawing of the weld.

Butt joints. It will be the best practice to employ butt joints for connecting plates. These joints can also be used when connecting I beams, channels and angles. When designing butt joints care should be taken to make possible good penetration of the weld and create conditions ensuring free development of the welding deformations (shrinkage) upon cooling of the welds. For this purpose it is necessary in plates over 10 mm thick to machine one or both edges in order to make possible the inserting of the electrodes to a greater depth, which will ensure the required penetration, and also to leave a gap of a constant size between the elements being connected. After welding, the gap completely disappears, which is an indication of the considerable transverse shrinkage of the weld. Failure to provide such a gap leads to warping of the plates and large welding strains. When fastened plates are welded tensile stresses appear in the weld after cooling that lead to the formation of cracks and failure of the weld.

In automatic welding, owing to the high current intensity, the edges are not always machined, which reduces the amount of deposited metal. With thickness of steel parts up to 16mm automatic welding is performed from one side and in a single pass, with a square groove, leaving a gap between the parts. With thickness exceeding 16 mm the edges are bevelled at an angle of about 60^{o}. Automatic welding is performed on temporary (copper) or permanent (steel) backing strips, on a flux bed or with a preliminary back bead weld.

For automatic welding from both sides (t > 24 mm) double-V grooves are used with the length of the root constituting about one-third of the thickness of the plates being welded.

When welding aluminium plates up to 10 mm thick square grooves may be employed, but beginning with thickness of 12 mm bevelled edges must be the rule.

A gap of one millimetre is used between plates up to 10 mm thick, and one of 1.5–2 mm in plates up to 30 mm thick.

The strength of a butt weld subjected to axial forces is analyzed on the assumption that the stresses are uniformly distributed over the cross-section of the weld. When the weld is defective the force field inside the plate is not uniform and induces an additional bending moment and a dangerous concentration of stresses. To eliminate this a bead weld is applied after thorough cutting out of the slag inclusions at the root of the weld.

The section of a butt weld used for investigations, when the weld is located perpendicular to the centre line of the element (a square joint), is taken equal to the area, but without regard to the strengthening bead weld below and the deposited bead on top, i.e.,

$$A_w = l_w\, t,$$

where, t is the minimum thickness of elements being welded; l_w is the design length of weld, equal to the actual length less 10 mm (allowing for the formation of a crater and poor penetration at the ends of the weld). If the weld is extended to the backing strip these 10 mm may not be subtracted.

The stress in a weld is checked by means of the expression:

$$\sigma_w = N / A_w = N / l_w\, t \le R_w \gamma_c,$$

in which R_w is the design tensile or compressive strength of a butt weld.

The design tensile strengths of square-groove butt joint welds, made by manual or semi-automatic welding, and with ordinary methods used to control the quality of the welds, are lower than the strength of the base metal at the joint. For this reason the stresses in the base metal cannot be utilized completely.

The design of a skew joint has a strength equal to that of the base metal. The angle between the direction of the axial force and that of the skew weld should not exceed 65^o (generally the angle of 60^o is employed). With such an angle the strength of the joint does not have to be checked.

Lap joints. A lap joint can be made with or without straps, using fillet welds. Depending upon their arrangement relative to the forces acting on the elements, fillet welds can be side (located parallel to the forces) and end or edge ones (located perpendicular to the forces).

Lap joint with side welds. The force is transferred very unevenly from one element to the other along both the length of the weld and the cross-section of the joint. Upon statical loading, however, after stresses equal to the yield point have been reached in the extreme points of the welds, further loading will result in more uniform stress distribution over the length. The stress diagram will level out and failure will occur in the form of shearing along the surface. This line has curved sections at its beginning and end (at the most stressed points) and a straight section in the middle of the weld. Such a form of failure permits analysis to be based on the assumption that the shearing stresses are uniformly distributed over the minimum cross-sectional area of the weld, which passes through its theoretical throat. With equal weld legs this throat is equal to $0.7k_f$, where k_f is the size of the weld.

In view of the fact that in automatic and semi-automatic welding the penetration at the root of the weld is deeper than in manual welding, and that when the weld is resisting shearing stresses part of the base metal is involved in this resistance, the theoretical throat of the weld is generally taken equal to $\beta_f\, k_f$. Thus the shear area of side welds is found from the expression:

$$A_w = \beta_f k_f \sum l_w \quad \text{or} \quad A_w = \beta_z k_f \sum l_w \,.$$

Here, k_f is the size of fillet weld (leg of inscribed isosceles triangle); $\sum l_w$ is the sum of design lengths of fillet welds; β_f is the factor taken equal to 1.0 for single-pass automatic welding, to 0.8 for single-pass semi-automatic welding and to 0.7 for manual welding, as well as for multipass automatic and semi-automatic welding; β_z is the factor taken equal to 1.0.

Because of the poor penetration at the beginning and of the crater at the end of the weld, and also of the influence of chance inclusions, the design length of a fillet weld, according to the standards, must not be less than 40 mm or 4 k_f.

The strength of side welds also depends upon their size, the ultimate strength decreasing somewhat with an increase of the size. The weld size is stipulated in the standards and is taken not over $1.2t$ (t is the minimum thickness of the elements being connected). It will be good to avoid the use of welds over 20–25 mm in size. The minimum size of welds is taken equal to 4 mm. The recommended minimum sizes of fillet welds, depending upon the thickness of the elements being welded.

A lap joint with side welds subjected to an axial load, with regard to the assumption of uniform shearing stress distribution, is investigated by means of the expression:

$$\tau_w = N / A_w = N / \beta_f k_f \Sigma\, l_w \le R_{wf} \gamma_{wf} \gamma_c$$

$$\text{or}\ \ \tau_w = N / A_w = N / \beta_z k_f \Sigma\, l_w \le R_{wz} \gamma_{wz} \gamma_c,$$

where, N is the design axial load, R_{wf} is the design shearing strength on metal of fillet weld, R_{wz} is the design strength of metal of border zone of alloy; $\gamma_{wf} = 1{,}0$, $\gamma_{wz} = 0.85$ are the coefficients of reliability by conditions of weld working.

The size of the weld k_f is usually taken equal to the thickness of the elements being connected, or somewhat less. The required total length of the welds is found from the expression:

$$\Sigma\, l_w \ge N / \beta_f k_f R_{wf} \gamma_{wf} \gamma_c \quad \text{or} \quad \Sigma\, l_w \ge N / \beta_z k_f R_{wz} \gamma_{wz} \gamma_c.$$

The actual length of the weld should be greater than the design length by 10–20 mm.

When securing an unsymmetrical section, for example two angles, to a plate attention is paid to uneven distribution of the load between the welds transmitting the force field from the angles to the plate. The force N extending the angles can be considered as the resultant of a force field of normal stresses applied to the centre of gravity of the angle sections.

It is obvious that the force N is distributed in reverse proportion to the distances from the welds to the line of the centre of gravity of the angle. This line is situated at a distance of about one-third of the leg from the back. The force at the back is taken equal to $N_1 = 0.7\, N$ and at the tip equal to $N_2 = 0.3\, N$. Hence the formula for determining the design length $\Sigma\, l_1$ of the weld at the

$$\Sigma l_1 \geq 0{,}7\ N/\ \beta_f k_f R_{wf} \gamma_{wf} \gamma_c \quad \text{or} \quad \Sigma l_1 \geq 0{,}7\ N\ /\ \beta_z k_f R_{wz} \gamma_{wz} \gamma_c$$

in which k_{f1} is the size of the weld at the back of the angle.

Since the force at the tip of the angle leg is considerably smaller, the weld at this end can have a smaller size. It is not good to cut away the leg of the angle to reduce the length of this weld l_2. Angles (with rare exceptions) should be cut at right angles to their center line.

Lap joints with end welds. When the straps are arranged symmetrically, end welds give a sufficiently high strength. Owing to a sharp change in the direction of the forcelines, however, considerable stresses are concentrated at the root of the weld, and this leads to failure with small longitudinal strains (e = 4–6 %), i. e., brittle fracture is obtained with failure (in tension) along the plane of contact between the weld and the edge of the plate.

Owing to the combined stressed state and the extremely uneven distribution of the stresses, such joints are of lower quality. For this reason the design strength, regardless of the stresses acting on the weld (compression, tension, shear), is in all instances taken as equal to the design shear strength of a fillet weld.

Thus end welds are provisionally investigated for shear along the minimum section area, which is taken as the design area passing through the theoretical throat equal to $\beta_f\ k_f$. The expression used for analysis thus remains the same as for side welds.

The length of the strap is taken equal to at least $10t$, but not less than 80 mm.

It is not recommended to use an unsymmetrical connection with one strap, since the resulting eccentricity will create an additional moment.

The use of a lap joint with end welds is possible only when one of the elements is a sufficiently rigid member or, on the contrary, in slender (thin) sheet members. To reduce the influence of the additional bending moment in such joints the length of the lap should be at least 5*t*, where t is the thickness of the element being welded on.

Composite joints. A joint is called composite if it contains several different kinds of welds, namely, side, end or butt ones. The simplest example of such joints is a joint with welding done around the whole contour.

The actual behaviour of a composite joint depends on the distribution of the forces between the side and the end welds. It may be described as follows. During the first period after application of the load, the end welds, being more rigid, sustain greater forces, and only after a certain deformation of these welds do the less rigid side welds begin to take the load. Failure occurs in the plastic stage with relative equalizing of the stresses in all the welds.

It is not good practice to use a joint of this type, since it produces overloading of the end welds, especially when alternating loads are applied.

The force field is transmitted more uniformly in joints having straps of a rhombic type with acute angles of 35–45^{o} and cut off apexes. In this case, to avoid great concentration of stresses at the gap of the joint, the weld is not brought up to the middle of the joint, leaving a space of 25 mm at each side. Rhombic straps with uncut apexes are not recommended because of the formation of cracks at the apexes. To ensure smoother transition of the force field, the legs of the welds on the main plate should be extended.

The third type of composite joint used in tension elements of members can be made by butt welding of the elements with the additional use of rhombic straps. Such. a joint is more costly, since it requires processing of the edges, and removal of the excessive weld metal after butt welding.

The formula used for analyzing such a joint is

$$\sigma = \frac{N}{A_w + \sum A_s} \le R_{w.t},$$

where, A_w is the cross-sectional area of weld; A_s is the area of strap; $R_{w.t}$ is the design tensile strength of weld.

Upon comparing various methods of connecting plates the conclusion can be drawn that the best type of joint for plates subjected to axial loads is a square butt joint (or, in tension, a skew joint, which has equal strength). When straps are required in a square butt joint these should be of the rhombic type with cut off apexes. In critical members, however, it will be better to weld the plates without straps, using an automatic welding process (or manual or semi-automatic welding with high-class methods of weld quality control).

It is preferable to connect rigid shapes to plates by means of side welds, which are more plastic.

3.7. Bolted connections

General characteristic. Bolted connections are a very widespread method of securing members in erection and assembly, especially in industrial construction. In field conditions it is important to have a simple and sufficiently reliable way of fastening elements and members that does not require any special equipment consuming power.

Bolts are subjected to shear, crushing and tension. Compliance (deformability) of bolts is the result of a lower preliminary tension of the bolts (caused by tightening of the nuts), and also of the presence of clearances between the bolt and its hole.

When bolts are to work in tension, their preliminary tensioning is of great significance, and it should be greater than the external tension load.

Since it is difficult to ensure equal tightening of several bolts in a connection, and that they will be subjected to uneven stresses, lower design strengths are established for bolts.

Excessive tightening of bolts (which is possible when spanners or wrenches are used for assembly that has been extended by fitting lengths of tubes onto them) leads to plastic strains and improper behaviour of the connection.

The following types of bolts are employed in steel members:

– black or unfinished bolts with nuts;

– semifinished bolts with nuts;

– bright or finished (turned) bolts with nuts;

– high-strength bolts.

Unfinished bolts (class of accuracy C) are stamped from round steel bars, deviations from the nominal diameter of 0.75 to 1.00 mm being tolerated. Unfinished bolts may be fitted into holes with a clearance of 2–3 mm.

Semifinished bolts (class of accuracy B) with a hexagonal head are manufactured from calibrated round steel bars. For this reason the tolerance in the diameter is only a negative one ranging from 0.5 to 1.0 mm.

Bright or turned bolts (class of accuracy A) with a hexagonal head have a turned shank with a negative tolerance of up to 0.34 mm. They can be fitted into holes with a clearance of 0.3 to 0.5 mm, which means tight fitting. A bolt can be placed into such a hole only by means of light blows of a hammer.

Bolts are manufactured with a diameter of 10, 12, 14, 16, 18, 20, 22, 24, 27, 30, 36, 42 and 48 mm and a total length ranging from 40 to 200 mm.

Bolts are manufactured of grade ВСт3, ВСт5 carbon steels, grade 14Г2, 15ГС low-alloy steels, etc.

High-strength bolts are employed in field joints and transfer the load by means of friction. They are made from carbon or alloy steels (for example grade 40X chromium steel) with an ultimate strength after heat treatment of at least 100 kN/cm^2 for carbon and 130 kN/cm^2 for alloy steels.

Behaviour of bolted connection. When unfinished bolts are subjected to shearing stresses, the connection has a great deformability owing to the presence of clearances. Investigation carried out by G. Shapiro has shown that the shear of rivets at stresses near the yield point constitutes 0.24 mm on the average, while the shear of unfinished bolts with clearances from 1 to 4 mm varies correspondingly from 2.65 to 3 mm (i. e., about ten times greater). At the same time the statical strength of a bolted connection is only about 10 % less than that of a riveted one, which is explained by equalizing of the loads on the separate bolts when they are stressed above the yield point. The vibration strength of a bolted connection is considerably (about 50 %) less than that of its riveted counterpart. For this reason it will be good practice to use unfinished bolts when they are subjected to shear only under statical loads, where the principal requirement is strength, while the deformability of the connection is not of great significance. An effective but costly means of reducing the deformability of a bolted connection is the transition to bright bolts that will tightly fill their holes.

It is more rational to use bolts when they are subjected to tension. For reliable behaviour of a bolt, good tightening thereof is essential, since the external tensile force must first reduce the compression of the elements due to the initial tensioning. To reduce the deformability of a connection, either the stress in the bolts must not be allowed to reach the design strength, or the tightened nuts must be reliably secured with locknuts or by welding them to the bolts.

Analysis and design of bolted connections. The following expressions are used for analyzing bolted connections:

for shear $n \geq N / R_{bs} \gamma_b A_b n_s$;

for crushing $n \geq N / R_{bp} \gamma_b d_b \sum t_{\min}$;

for tension $n \geq N / R_{bt} \mathrm{A}_{bn}$;

in which N is the design load on the connection;

R_b is the design strength for shearing, crushing and tension of bolts;

A_b is the area of section of bolt.

The number of heavy bolts required in a connection (assuming uniform distribution of the load between them) will be

$$n \geq N / Q_{bh} k\gamma_c = N\gamma_h / R_{bh} \gamma A_{bh} \mu k \gamma_c,$$

where, N is the design load on the connection.

Bolts are positioned in accordance with the rules for the arrangement of rivets (except that the minimum distance between bolts must always equal 3.5d). Besides, it is necessary to take into account the possibility of screwing on the nuts with standard spanners or wrenches.

The bolts for bolted joints of steel structure elements should be used in accordance with appendix Ж/10/.

Bolts should be arranged according to the requirements represented in table 1.12.3/10/; taking into account that bolts obtained according to calculation are arranged by using minimal distances between them while the design bolts are arranged by using maximal distances between them.

Bolts of accuracy class A should be used for joints where holes are drilled according to the designed diameters in assembled elements or carried out by means of conductors in certain elements and parts or either drilled or perforated on smaler diameter in order to enlarge the hole to the designed diameter by drilling in assembled elements.

Bolts of accuracy classes B and C in multiple-bolted joints should be used for structures produced from steel with liquid limit up to 390 N/mm^2.

Methodical instructions to Chapter 3

All members of metal structures are made with csrcular sections. That is why, studying of standard classification of metal structures is of great importance.

Special attention should be paid to the methods of connecting elements of steel components. Without this knowledge you won`t be able to design metal structures.

Student should study the calculation of the welds, bolted connections with bolts of normal accuracy and high-strength bolts.

Questions and Tasks to Chapter 3

1. *Enumarate the main characteristics of sheat steel and section steel.*
2. *What types of cold rolling steel do you know?*
3. *What are the advantages of the cold rolled steel as compared with hot rolled steel?*
4. *What are the main types of connections of metal structural members?*
5. *What limit states are available for connection with the bolts of normal accuracy ?*
6. *How do we calculate the design strength perceived by the friction surface under one high-strength bolt?*

4. CONSTRUCTION MEMBERS OF BUILDINGS

4.1. Beams and beams grillages

4.1.1. General characteristics. Beam grillages. Types of beams

Metal beams and girders, which are subjected to bending, are either of the rolled or built up type. Preference should be given to the use of rolled beams, which will reduce labour consumptibu. Due to the limited number of standard sections, however, it becomes necessary to design heavy beams of a riveted or welded built up type.

Welded beams are made up of three plates, namely, a vertical one, called the web, and two horizontal ones, known as the flanges, which are welded to the web. If it is necessary to use heavy riveted beams, then horizontal plates are riveted to the legs of the angles to increase the section modulus.

Welded beams are more economical than their riveted counterparts, and this is why the latter are used only to a limited extent, mainly in heavy members, as well as in members that are subjected to large dynamic or vibration loads.

General Dimensions. The general dimensions of a beam are its effective span length and the depth of its section. The effective span length L of a beam is the distance between the centres of its supporting parts.

The actual length of a beam L_{y} is always somewhat greater than its effective span length. The length L_0 is called the unsupported or clear length. It is usually determined by the service conditions of the structure and is selected with a view to economical considerations.

The depth of the section h is selected with regard to ensuring the optimal relations between the section dimensions which will result in a minimum consumption of steel (if there are no special design requirements limiting the overall dimensions of the beam), as well as to providing the required rigity of the beam, which is determined by the ratio between its deflection and the span. The maximum values of the quantity are established by standards. The ratio between the deflection of a beam and its span directly depends on the ratio between the depth of the section and the span.

There fore the minimum depth of a beam can be determined for the given unit deflection

$$h_{\min} = (5R_y l \,/\, 24\,E)\ \ [l \,/\, f]\ \ q_n \,/\, q.$$

Beam grillages. When designing beam members it is essential, depending upon the designation of the beams, to draw up a diagram showing their arrangement, plan the general dimensions and determine the load to be carried by the beams. If it is necessary to cover a certain area, the beams supporting the member are generally arranged in two directions. Such a member, sometimes consisting of a whole system of intersecting beams, is referred to as a beam grid or grillage. The grillage is covered with a flooring in the form of a metal sheet (the working areas of shops, hydraulic engineering structures), reinforced concrete slabs, etc. A grillage consists of main beams bridging a large span (or bay), and secondary beams (or joists). The main beams rest on supports, while the secondary ones are supported by the main beams.

The following types of grillages are distinguished:

– with a tier arrangement of the secondary beams;

– with the secondary beams arranged on one level with the main ones;

– with the secondary beams located at a lower level;

– a beam grillage of a complicated type consisting of three systems of beams, namely, main, secondary and floor beams.

The type of grillage to be used is selected with a view to economical considerations, as well as to the established overall dimensions and clearances, which depend on the service conditions. The maximum height limiting the dimensions of a member is referred to as the construction height.

Determining load on beams. To determine the load on a beam (when the grillage is subjected to a uniformly distributed load) the tributary area of this beam must be found. The tributary area of a secondary beam has a width of b (beam spacing) and a length of L (span of beam). When investigating rolled beams the influence of their own weight may be neglected owing to its insignificant value.

Sheet flooring. A flat metal sheet flooring is placed on the flanges of the beams and welded to them. The thickness of the flooring is most frequently established depending upon the allowable deflection and for this case sheet flooring is analyzed in accordance with the loads stipulated in the building standards.

As regards the nature of its behaviour, sheet flooring occupies an intermediate position between a plate and a membrane. While a plate under a load is subjected only to bending, a membrane is subjected only to axial tension, which reguires fixed supports. Flooring may be subjected to both bending and axial tension, behaving as an elastic suspended member.

We shall consider only cases of sufficiently long flooring supported at two sides (with the ratio of the length to the span of the sheet exceeding 2). Axial tension may appear in such flooring only if the plates are fixed at their edges and the constraints are able to take up thrust loads *H*. This phenomenon is known as bending with membrane action. If there is no thrust load (membrane action) or if it is very small, the sheet may be considered to be subjected only to bending, as a plate.

4.1.2. Rolled steel beams

The rolled beams generally used are of an I or a channel shape. It is more rational to use an I shape owing to its symmetry. At the same time a channel shape behaves better in oblique bending.

The investigation of rolled beams consists in determining the required number (size) of the rolled shape, after which the strength, rigity and stability (resistance to buckling) of the beam are checked.

Analysis and selection of rolled beam. On having selected the beam type and having determined the design span and the design load acting on the beam, the maximum design bending moment *M* is found. The latter is used to compute the minimum section modulus required, i. e.,

$$W_{\mathrm{dem}} \geq M / R_y\gamma_c,$$

while when it is possible to take the development of plastic deformations into account

$$W_{\mathrm{dem}} \geq M/1{,}12\ R_y\gamma_c.$$

Having determined the required section modulus W, the nearest shape number with an actual section modulus W greater than or equal to the required one is selected from the catalogue of standard shapes.

Checking for adequate strength. When the section has been selected, the actual stress in the beam is determined, which must comply with the condition that

$$\sigma = M / \mathrm{W} \leq R_y\gamma_c.$$

Checking for rigity of a beam consists in determining the unit deflection, which should not exceed the design value

$$f / l = [f / l].$$

Checking for general stability. If the upper flange of a beam is not secured to prevent lateral buckling, then the beam may lose its general stability after the critical load has been reached.

The following expression is used for checking the general stability of a beam $\sigma = M / \varphi_b W \leq R_y \gamma_c$. The values of factor φ_b for rolled I beams are determined in the tables of design standards.

With low values of φ_b it is necessary to provide for horizontal ties that will strengthen the upper flange. When static load is transmitted through a solid rigid flooring continuously supported on a compression flange of a beam (reinforced concrete slabs, corrugated steel, etc.), no check of the general stability is required.

Calculation of steel structure elements under bending action. Classification of bending elements. Depending on the purpose, operation conditions, technical and economic feasibility studies, analysis of bending elements (beams) should be made either with taking into account the restricted plastic strain development or without it in accordance with the classification of element cross-sections in three classes 1.1.3.7./10/. It is assumed that beam class corresponds to the class of its design cross-section.

The 1class beams should be used for all types of actions and calculated for elastic deformations only, while beams of 2 and 3 classes are recommended to be applied under action of static loads and calculate taking into account the at development restricted plastic strains.

Bi-steel beams are recommended to class as 2-nd class and calculate taking into account the development of restricted plastic strains in the beam web when chords of the beam, produced from the stronger steel, attain its design strength R_{yf}.

The crane girders (single of bi-steel) for cranes of operation modes 1K-5K (GOST 25546) may be classed as 2-nd class for strength analysis.

Joints of rolled beams. Rolled beams can be connected together by welding or riveting. The simplest design of a welded joint (splice) is a square butt welding. Such a weld, however, can be located along the length of a beam only at places where the stresses in the beam do not exceed the allowable values for a weld $M = 0.85\ M_{\max.}$

If a welded joint is required in the middle of a beam, i. e., at the spot where the maximum moment is observed, it is strengthened with horizontal straps. The dimensions of these straps are determined from the condition that the stress in butt welds should not exceed the design strength R_{wt}.

Attention must be paid to the required sequence of welding a joint, which will noticeably affect its strength. Thus, for example, when joining I sections, first their webs and then their flanges should be welded, since otherwise great internal tensile stresses will appear in the web that often lead to the formation of cracks. It is also possible to use a connection that is simpler to make, wherein straps are used without butt welding of the I section. In this case the straps must transmit the moment M and the shear force Q acting at the joint.

Support connections of beams. When beams rest directly on masonry walls or concrete members, a sufficient bearing surface must be provided for transmitting the load to the support. Generally a bearing plate is welded to the beam. The dimensions of this plate should be sufficient to ensure that the pressure under the plate does not exceed the design compressive strength of the wall material.

The thickness of the bearing plate is found by analyzing its bending due to the action of forces on the plate from below. To avoid deformations, a long plate may be reinforced with stiffening ribs.

When simple beams rest on steel members, the following kinds of connections are possible:

– tier connection;

– bolted or riveted connection;

– seated connection.

Tier connection is the simplest kind and can be made both with bolts and by means of field welding.

The design of a connection using end angles joined by means of unfinished bolts or rivets is encountered quite frequently. The load is transmitted to the supports in this kind of connection through the angles. The latter are usually welded to the web of the I beam with jigs, seeing that deviation of the angles from a plumb or vertical line will necessitate costly field alteration jobs.

The behaviour of the weld should be checked for the action of the stress resulting from the common action of the forces in accordance with the conditional expression.

Beams can also be secured in place by directly connecting the web of the beam to ribs or other protruding parts of the member. Here it is necessary to cut away the flanges of the beam, doing this cutting when possible along the web, and not the flanges.

4.1.3. Built-up steel beams

General dimensions. As has been previously indicated, built-up beams are either of a welded (mainly) or riveted design. Fig. 4.1 is an example of a working drawing of a beam.

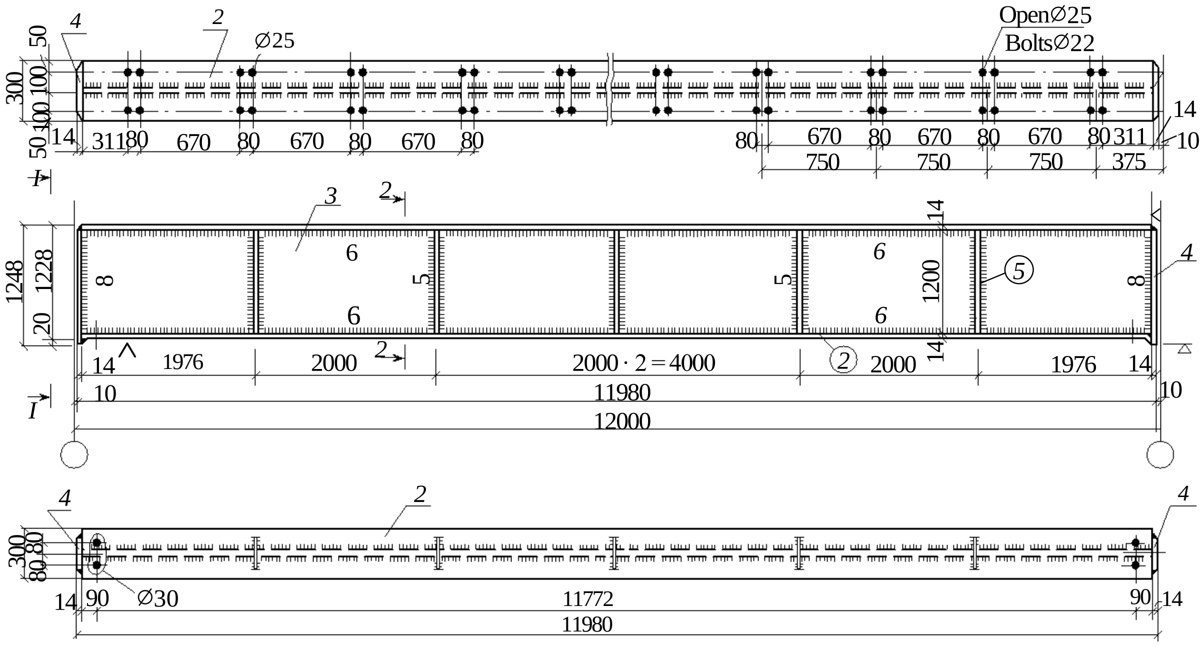


Fig. 4.1.Working drawing of a beam

The general dimensions — the span and the depth — are established with a view to the most advantageous (optimal) relations between the dimensions of the structure.

In industrial structures the length of the span is established depending upon the production process for which the structure is designed.

The minimum depth of a beam determined from the condition of is not, as a rule, optimal rigidity provided the material consumption. The determination of the most advantageous cross-section of a beam consists in finding the minimum area A of the section for the given section modulus $W = M / R_y$ and in the most efficient distribution of this sectional area between the web and the flanges depending on the depth h and the thickness of the web. Let us introduce the concepts of the web slenderness ratio (the ratio between the depth of the web and its thickness) $\lambda_w = h_w / t_w$ and the factor showing the distribution of materials over the beam section $\alpha = A_w / A$, we shall find the optimal depth of a symmetrical beam section

$$h_{\text{opt}} = 1,2\sqrt{W/t_w}.$$

Having selected the slenderness ratio of the web λ_w, and determined the optimal depth of the beam for this ratio, we have thus also established the best distribution of the material over the cross-section. In a symmetrical I beam having the optimal depth, the material is distributed equally between the web and the flanges ($\alpha = 0.5$).

In order to standardize members, it will be good practice to take the depth of a built-up beam in round numbers that are a multiple of 100 mm.

In individual projects, where the depth of the beams is not directly connected with insignificant deviations in the height of the structure, it will be better to round off the depth of the beam webs, which will simplify the work involved in the fabrication of the members.

Thus it can be said that the greater the web slenderness ratio $\lambda_w = h_w / t_w$ is, the more economical the beam is. In practice, however, the value of this ratio is limited by the necessity of ensuring stability of the web and its adequate resistance to shear. The selection of the web slenderness ratio is also noticeably affected by the minimum thickness of the web permitted by conditions of corrosion and the local pressure caused by a live load.

Generally the minimum thickness of the web is taken equal to

t_w = 8 mm, or sometimes 6 mm. The thickness of the web is increased in steps of 2 mm.

The thickness of the web can also be determined from an empirical equation that well reflects the increase in λ_w, with a growth of the depth of a beam, namely,

$$t_w = 7 + 3h,$$

where t_w is in millimetres and h in metres.

With large concentrated loads the minimum web thickness has to be checked for adequate strength on the basis of the maximum shear force Q (reaction of support)

$$t_{w,\,\min} \geq 1{,}5\ Q / h_w R_s \gamma_c,$$

where, R_s is the design shear strength of steel, $R_s = 0.58\ R_y$.

This expression has been obtained from the condition that only the web of the beam takes the shearing stresses.

It must be pointed out that from an economical viewpoint very great attention should be paid to selecting the thickness of the web, seeing that after the thickness t has been established there will only be one solution (with $\alpha = 0.5$) for the optimal depth and, therefore, the minimum sectional area of a beam. Indeed, assuming the depth of beams somewhat less than the optimal one, we obtain $h_{\text{opt}} = 1.15\ \sqrt{W_{br}} / t_w$.

It is obvious that the minimum possible web thickness complying with the conditions of sufficient strength and stability should be selected.

Selection of section for welded beams. When the required section modulus W_{br} is known and the optimal depth of the section and the thickness of the web have been determined, we find the most economical section of a welded I beam. For a beam that is symmetrical with respect to its neutral axis (with identical upper and lower flanges) the problem consists in finding such dimensions of the flanges that will give a total section modulus of the beam equal to the required W_{br}. The total section modulus of a beam is expressed approximately in terms of the section module of the web and the flanges $W_{br} = 0.5Ah - 0.3\ th^2$.

The ratio between the width and the thickness of a flange ensuring the required area must comply with certain design and production requirements. The wider a flange with the given area is, the more stable is the beam and the more convenient it is for transportation and erection. However, with a too great width and, therefore, a small thickness, the compression flange of a beam may lose stability under the action of normal stresses. Local stability of the overhanging part of the flange will be secured if this part does not exceed $15t_f$ for steel 3 and $15t_f\ \sqrt{E/R}$ for low-alloy steel. Consequently, the total width of a flange must comply with the following relations for steel 3 $b_f \leq 30\ t_f$ or for low-alloy steel $b_f \leq 30\ t_f\sqrt{E\ /\ R}$ in which R is the design strength of steel, kN/cm^2.

The minimum thickness of the flange should ensure general stability of the beam against buckling in a horizontal plane, when it will be unnecessary to introduce the buckling factor φ_b into the expression being used for analysis.

From considerations of design a flange width less than 180 mm should not be used.

The thickness of a flange is established within the limits of 8 to 40 mm, but not less than the thickness of the web and not over 2.5–3.0 times this thickness, since when thick plates are welded on, considerable residual tensile stresses due to shrinkage develop. The width and thickness of the flange should be selected in accordance with the standards for general-purpose steel sections, taking the following dimensions: for the thickness, in steps of 2 mm up to 22, and then thick-ness of 25, 28, 30, 33, 36 and 40 mm; for the width in steps of 20 mm up to 420, and then width of 450, 480, 500, 530, 560, 600 mm, etc.

After the section has been selected its actual section modulus W is calculated and the strength of the beam and the web is checked by means of the expressions

$$\sigma = M / W \leq R_y \gamma_c, \quad \tau = QS / I_x t_w \leq R_s \gamma_c,$$

where M is the design moment, Q is the shearing force, $W = 2I/h$ is the section modulus of beam; $S = A_f a + A_w h_w/8$ is the statical moment of half section with respect to neutral axis.

Simple welded beams with a constant cross-section that are stiffened to prevent the loss of general stability, and which carry a statical load, are investigated with regard to the development of plastic strains in them.

Besides checking the strength of the beam and the web, the stability of the web and the general stability of the beam are also investigated.

When a concentrated load is applied through the flange of a beam, at a spot not strengthened with a stiffening rib, the web of the beam must he checked for resistance to local pressure.

Variation of section along beam length. The bending moments near the supports are considerably less than the maximum ones, and therefore there is no necessity of selecting a constant section over the entire length of the beam on the basis of the maximum moment. There are two methods of reducing the cross-section and thus of lowering its section modulus.

The first of these methods, used most frequently, consists in reducing the section of the flanges. In welded beams this is done by reducing the width (or more seldom, the thickness) of the flange.

The second method consists in reducing the depth of the vertical web, as a result the beam will have a trapezoidal form. $x = L/6$ from the support. The moment M_1 acting at this point can be found graphically from a moment diagram or analytically from the equation $M_1 = qx(L - x)/2$.

The computed moment M_1 is used to find the required section modulus and select the new section of the flanges in the usual manner (retaining their thickness, as a rule). At the place where the section is changed, the flange plates are connected by means of a butt weld, and therefore the stress in the beam should not exceed the design tensile strength of a weld.

When the moment M_1 is known, the moment diagram can be used to find the point where the plates can be terminated. This point, called the place (line) of theoretical plate termination, can also be found analytically.

In practice, the horizontal plates should be extended further, beyond the theoretical termination line.

4.1.4. General and local stability of steel beams

General stability of beams. A long and narrow beam that is not stiffened in a lateral direction and that is loaded above a certain limit may lose its stability and buckle to a considerable extent in a horizontal direction.

This phenomenon is termed the loss of general stability of a beam, while the load and the stresses at which the loss of general stability commences are known as the critical ones.

Upon a loss of general stability, twisting of the beam cross-section commences (Fig. 4.1). This leads to displacement of the flanges laterally, and the beam, besides bending in a vertical plane, is also subjected to bending in a horizontal plane and torsion. It is obvious that the wider the flanges and the greater the value of I, the higher will be the critical stresses and the greater the stability of the beam. The critical stresses can also be considerably raised by fixing certain points of the upper flange of the beam in the span to prevent possible lateral displacement.

In welded beams withstanding a uniformly distributed load, the most advantageous place for changing the section is located at a distance.

The value of the critical stresses depends upon the constructive form or layout of the beams, and first of all upon the ratio between the span (and distance between the intermediate constraints of the compression flange) and the width of the flange L/b.

With high values of the L/b ratio the general stability of the beam should be checked by introducing the buckling factor φ_b used for this analysis $\sigma = M/\varphi_b W_{br} \leq R_y \gamma_{c}$.

For beams with a symmetrical I section the factor φ_b is the ratio between the critical stress resulting in the loss of stability and the yield point.

Local Stability. Local buckling of separate elements of members under the action of normal (compressive) or shearing stresses is known as the loss of local stability. In beams the loss of local stability of a flange or the web is frequently the main cause of a loss of load-carrying capacity. The web of a beam may lose its stability owing to the action of shearing or normal stresses, as well as a combination of these stresses.

Loss of stability of web due to shearing stresses. Near the support, the web of a beam is subjected to the action of shearing stresses which results in the web distortion, along the lines of the shortened diagonals the web is compressed, and along those of the extended diagonals it is stretched. Under the action of compression the web may buckle, forming waves inclined at an angle of about 45^{o} to the axis. To prevent the web buckling, vertical (lateral) stiffening ribs (stiffeners) are installed intersecting possible buckling waves. In this instance the web will be separated into rectangles limited on four sides by the flanges and the stiffeners.

According to the standards, the web of a beam in bending must be strengthened with stiffeners if hw / $t_w > 70 \sqrt{21/R}$.

The maximum distance between the stiffeners is taken equal to $a_{\max} = 2h$ for $h / t > 100$ and $a_{\max} = 2{,}5h$ for $h / t \leq 100 \sqrt{21/R}$.

In view of a certain constraining effect of the stiffeners and a number of simplifications, the standards permit doing without investigation of the beam web stability while should there be present local pressure between the stiffeners when $\lambda_w = h / t \leq 110 \sqrt{21/R}$ the values of the web slenderness ratio do not exceed $\lambda_w \leq 80 \sqrt{21/R}$.

Loss of stability of web and flange due to normal stresses. At a considerable distance from the supports, nearer to the centre of the beam, the influence of the shearing stresses on the web is not large, and the web at this part of the beam is subjected mainly to the action of normal stresses. These may also be the cause of a loss of stability.

The magnitude of the critical normal stresses depends upon the law of distribution of the normal stresses applied to the edges of a rectangular plate-web, which are characterized by the factor of stability.

In accordance with the standards the local stability of separate beam elements would be guaranteed by this conditions:

for web — $\lambda_w = h_w / t_w \leq 5{,}5 \sqrt{E/R_y}$,

for flage — $\lambda_f = b_{ef} / t_f \leq 0{,}5 \sqrt{E/R_y}$.

Loss of stability of web due to combined action of normal and shearing stresses. Upon bending of a beam a combined stressed state J appears in the web owing to the combined action of normal and shearing stresses, which may produce loss of local stability of the web.

As has already been indicated, the stability of the web must be checked if $h_w / t_w \geq 110 \sqrt{21/R_y}$. In this instance the I web must be strengthened with pairs of lateral stiffeners installed over its entire depth.

It is recommended practice to strengthen thin webs of deep beams, with additional longitudinal twin rib installed in the compression zone of the web, besides the lateral ribs, when the $h_w / t_w \geq 160 \sqrt{21/R_y}$.

The stability of a beam web is checked with the separate panels (rectangles) that are formed between the flanges of the beam and the stiffeners. By changing the distance between the stiffeners, such a ratio between the panel sides can be obtained that will ensure stability of the beam web.

On having planned the location of the stiffeners with the maximum possible distances between them, the stability of the web is investigated for combined action of normal and shearing stresses. Investigations carried out by S. Timoshenko, P. Papkovich, and later by B. Braude, have shown that the stability of a web will be ensured when the following condition is observed

$$\sqrt{(\sigma / \sigma_{cr})^2 + (\tau / \tau_{cr})^2} \leq \gamma_c,$$

where, $\sigma = M_y / I_x$; $\tau = Q / t_w h_w$; $\sigma_{cr} = C_{cr} R_y / \lambda_w^2$; $\tau_{cr} = 10.3(1 + 0.76 / \mu^2) R_s / \lambda_{ef}^2$.

4.2. Trusses

4.2.1. Roof trusses

General characteristics of trusses. A truss is a latticed member designed like a beam, to withstand mainly bending. Incontrast to a beam, truss is formed of separate straight bars or elements connected to each other at multiple joints (theoretically pinned or hinged) to form a geometrically unchangeable system. The load, as a rule, is applied to a truss only at the joints. Owing to such transmission of the load, the elements of a truss are subjected only to the axial action of tensile or compressive forces, which leads to better utilization of the material than in a solid beam. Trusses are especially advantageous in members where a great depth is required to satisfy the condition of adequate rigity. At large loads and relatively small spans, trusses become cumbersome and labour-copsuming for their fabrication. In this respect they are inferior to solid beams. The advantages of changing over from solid beams girders to trusses grow with an increase of the member span and a reduction of the load applied to it.

Trusses have a multitude of applications. They can be classified according to several aspects:

– application — bridge trusses, roof trusses crane trusses, the trusses used in the masts and towers of power transmission lines, etc;

– design — light single plane and heavy double plane trusses;

– direction of the support resdances and the design of the support members — girder trusses (simple, cantilever and continuous), strutted arch trusses, etc.

Besides, trusses can be of the plane or space types.

The present chapter is devoted to light single plane trusses used in roofs – roof trusses.

Elements of roof. Roof trusses serve to support the roof converting (cladding) and resist the loads acting on it. The main designation of the roof is to protect the premises against the weather (snow, rain, cold, etc.). Roof trusses are mainly rest on steel or reinforced concrete columns.

In industrial structures there are distinguished “warm” (heat-insulated) and “cold” (not insulated) roofings. Warm roofings consist of load-bearing slabs, heat-insulating material, asphalt binding course and a dampproof course made of roll material such as roofing felt or ruberoid.

Two types of roof are distinguished, namely, with or without purlins. In the first type the load-bearing elements may be standard precast reinforced concrete slabs or reinforced foam concrete and reinforced foam silicate ones (combining the functions of a load-bearing element and a heat-insulating material), etc. These slabs are laid on the purlins, the latter, in turn, being supported by the roof trusses at the joints (panel points), transferring the load to them. The most widely used length of a slab, which determines the size of the truss panels, is three metres.

The second, more widespread type of roofing (without purlins) consists of large-size standard reinforced concrete slabs with the following dimensions: 3×6 m (ПКЖ) with a depth of 300 mm, 1.5×12 m (ПНТП) and 3×12 m (ПКЖН) with a depth of 450 mm, that are directly supported on the top chords of the roof trusses. The slabs are secured to the truss chords by welding to the latter short angles embedded in the slab. Large-size slabs are also covered with a heat-insulating material (if the slabs do not simultaneously perform the functions of such material), a binding course and a layer of roofing felt.

When reinforced concrete slabs are supported on the truss angles, bending of the outstanding leg of the latter should not be feared, seeing that with the development of common deformation of the slab and the angle, the support pressure of the slab will move toward the back side of the angle. It is only essential to prevent shear of the angle material under the support pressure of the slab. For this reason with these truss chord angles ($t < 10$ mm for slabs 6 metres long and $t < 14$ mm for slabs 12 metres long) bearing plates 10–12 mm thick should be used on the angles.

Cold (without heat insulation) rooting is employed in hot shops and in not heated buildings. It is made of corrugated asbestos-cement sheets with a strengthened section or, in certain cases, of corrugated steel sheets or a solid sheet on purlins. Cold roofing can also be designed in the same way as its heat-insulated counterpart, using reinforced concrete slabs, but without heat insulation.

To ensure the shedding of water, roofs are generally given a slope or pitch that depends on the material of the roofing. Ordinarily the following pitches are used for roofs:

for roofing made of roll materials ....i = 1/8 to 1/12;

for asbestos-cement corrugated slab roofing ....i =1/3 to 1/4;

for corrugated steel sheet roofing .... i = 1/5 to 1/7.

Note. The above fractions denote a rise of 1 meter over the length shown in the denominator.

The pitch of a roof is created, as a rule, by designing the roof trusses with a sloping top chord.

There are distinguished single (also called lean-to, single-pitch or simple) roofs and ridge (also known as saddle or span) roofs.

The purlins are either solid (rolled) or latticed. Rolled purlins made of I sections or channels are heavier than latticed ones, but are considerably simpler and cheaper to fabricate, which explains their greater use. Rolled purlins are installed on the sloping top chords of trusses. Being located at an angle to the plane in which the load acts, they are subjected to unsymmetrical bending.

Rolled purlins are connected to trusses, as a rule, by means of unfinished bolts, and with the help of short angles welded to the chords of the trusses.

Latticed purlins are employed with a lattice made of round steel bars. Bar purlins can be installed vertically and inclined, i. e., perpendicular to the chords of the trusses (with roof pitches of 1 to 7). In all cases they must be braced with stays in the lateral direction (both the top and the bottom chords). The stays can be made of 6-mm wire. It is good policy to use latticed purlins with spans over 6 metres in length.

A shortcoming of large-size reinforced concrete roof slabs is their heavy weight, which leads to high consumption of metal for the load-carrying members. To reduce the weight of the roof, aluminium roofing plates, with or without heat insulation are used. The efficiency analysis of employing aluminium roofing in industrial construction has shown that the total saving in the cost of the structural part of the buildings due to the roof weight reduction is over 10 %.

Cold roofing (without heat insulation) can be made of corrugated aluminium sheets 0.6–0.8 mm thick, with a corrugation depth of 130 mm and length of 350 mm, the sheet being 6 metres long. These sheets, which are 1 150 mm wide (in projection) are placed on steel purlins and are secured to them by means of special cleats, or are designed in the form of special panels.

Warm (heat-insulated) roofing can be made with two plies or layers in the form of panels. The upper ply consists of a corrugate aluminium sheet, and the lower one is a sheet of asbestos plywood carrying sheets of a heat-insulating material such as slag wool or foamed plastics. A panel of 12 metres length must be strengthened with a strut.

It should be remembered that there must be no contact whatsoever, not at a single point, between aluminium sheets and steel elements. For this reason galvanized or cadmium-plated bolts are used for fastening the panels to the steel trusses.

Roof trusses sometimes carry skylights, i. e., members designed for lighting the premises from above and for natural ventilation (aeration).

Skylights can be arranged along a building (longitudinal skylights) and across a building (transverse skylights). Metal window sashes or shutters are suspended from the members of the skylight. The entire roofing load is transmitted to the roof trusses through the purlins and skylights, or directly through the large-size reinforced concrete slabs, if the latter are employed. It is assumed that the loads are applied strictly in the plane of the trusses. Actually this is not the case, and the load is applied with a certain eccentricity caused by the requirements of design. This circumstance, as well as the necessity of preventing buckling of the top compression truss chords, requires the installation of stays or ties outside the plane of the truss.

The ties used are horizontal ones in the plane of the top chord and vertical ones between the trusses, locating them at the ends of the building or of the temperature block.

4.2.2. Types of trusses. Determination of general dimensions. Spacing of trusses

Trusses are distinguished both with respect to the configuration of the chords and to the arrangement of their web members (lattice). As regards the configuration of their chords, trusses are classified into parallel-chord, single-pitch, trapezoidal and triangular trusses. The configuration of trusses is selected depending upon their designation, the material of the roofing, the system of water shedding, as well as with regard to factors of economy. Roof trusses with a trapezoidal configuration are the ones mainly used in industrial structures with roofing made of a roll material (roofing felt, etc.).

A permanent shape of a truss under any load is attained by using web members that form a system of triangles. The web or lattice of a truss is called a diagonal one (flat top Pratt or Howe trusses) if it is formed by a continuous zigzag arrangement of diagonals and verticals, all the diagonals of one half of the truss being directed to the same side. The lattice is referred to as a triangular one (single Warren trusses) if the zigzag is formed only by diagonals alternately directed to different sides. Most frequently use is made of a triangular lattice with additional verticals since the total length of its zigzag and the numbers of joints are less than in a diagonal lattice, while the additional verticals reduce the panel size in the top truss chord. In this system verticals are not required for creating a constant shape of the truss.

The general dimensions of a truss are its span and depth. For purposes of standardization, the spans of roof trusses in industrial buildings have been unified and are based on the module of 6 metres, i. e., they have the length of 18, 24, 30, 36 and 42 metres. For simplification of their fabrication and design, the unified standard steel trusses must have a standard geometrical layout for different spans. The length of the top chord panel in standard trusses is taken equal to 3 metres.

The optimal depth h in the middle of a trapezoidal truss span is determined by conditions of minimum weight and the required rigity (deflection). Here attention must also be paid to the possibility of rational transportation, with a view to the maximum concentration of fabrication work at plants. The minimum weight of such trusses is approximately obtained when the weight of the chords equals that of the lattice (including the gusset plates), which will be the case with a comparatively big truss depth-to-span ratio (h/L ~ 1/5). Such a big depth of a roof truss is not convenient from the viewpoint of transportation and erection, since in this instance the truss would have to be transported by separate elements with assembly in the field, which requires a lot of time and great financial outlay.

To make transportation possible by railway, the maximum overall dimensions of a member should not exceed (for the 1.524-mm gauge railways of Ukraine) 3.8 m in a vertical direction and 3.2 m in a horizontal one. The length of a four-wheel flat car is 9 metres and of an eight-wheel one is 13 metres. With a view to inscribing the overall dimensions of roof trusses within the limits of the railway clearance gauge so that only assembly of two halves of a truss would be required in the field, the depth of a truss to the middle of the span, as a rule, is taken not over 3.8 metres (between the extreme points of a member). Using a pitch of the top roof truss chord of 1 in 12, we obtain the depth at the support h_s, and thus all the overall dimensions of the trusses. It will be good practice to use the same depth of a truss at the supports h_s for trusses with different spans. This will permit designers to use a single standard geometrical layout and ensure standardization of the fastening elements. In standard trusses, the depth h_s is taken equal to 2.2 metres (between the backs of the angles).

A solution is possible, however, in which the depth of the truss in the middle exceeds 3.8 metres (with a top chord pitch of 1 to 8), while the truss well lends itself to fabrication and transportation at the expense of an insignificant increase in the number of field joints (three joints instead of two). For this end the bottom chord in the middle panel is shipped as an independent element, while the main part of the truss is transported upside down. In this instance the clearance depth of the truss is determined by the depth of the third vertical from the support.

The most advantageous angle between the diagonals and the bottom chord is 45–50^o in a triangular lattice and 35–45^o in a diagonal one.

The first, support, diagonal, whose direction determines the entire lattice system, may be directed either upward or downward. In the practice of designing industrial and residential buildings, the most frequent case is the use of support diagonals directed upward. With such solution the rigity of the shop or other premises is ensured more reliably when the truss performs the functions of a frame beam and the design of the support joint and the arrangement of the ties is better from a constructive viewpoint. Trusses with the support diagonal directed downward have an insignificant advantage in erection, consisting in that the point of support is located higher than the centre of gravity of the truss.

The distance between trusses (the spacing of the trusses) is established when solving the layout of the structure as a whole, with account taken of standardization of the members and parts of the structure.

As the result of investigations aimed at determining the optimal spacing of trusses, standardized spacings of 6 and 12 metres have found the widest use in designing practice.

4.2.3. Analysis of trusses

Determination of loads. Roof trusses are investigated for the f ollowing kinds of loads, transmitted to them as concentrated forces at the joints:

– dead load formed by the weight of the roofing and that of the members themselves;

– live load caused by snow, wind, suspended hoisting and conveying equipment, etc.

The majority of these loads are uniformly distributed ones.

First the uniformly distributed load per square metre is computed, next the tributary area per joint (panel point) is determined, and then the concentrated force acting on each joint of the truss is found

$$P = bd\,\Sigma q_i \gamma_{fi},$$

where q_i is the service (working) uniformly distributed load per square metre of horizontal projection; γ_{fi} is the factor of reliability on the corresponding load; d is the length of truss chord panel to which load is applied; b is the distance between trusses (truss spacing).

With steep roof pitches, the load induced by the weight of the roofing g_r should be taken equal to $q_r = g_r / \cos \alpha$, where α is the pitch of the roof in degrees.

The weight of glazing is taken equal to 0.35 kH per square metre of glazed area.

The snow load is taken in accordance with the prescriptions of the Building Standards and Regulations. In this instance the snow, as a rule, is considered to be distributed over the whole area of the roof. Sometimes when investigating roof trusses the possibility is considered of the snow being only on one side (on half of the truss span), which results in a greater load on the middle diagonals and may even cause a change in the sign of the force. However, seeing that the section of the middle diagonals, owing to the relatively small forces in them, is most frequently selected from considerations of design (with an adequate slenderness ratio), there will in most instances be no practical necessity of taking such a location of the snow load into account. The wind load is considered in trusses only when the top chord is inclined at an angle exceeding 30^o.

Determination of forces in elements of trusses. The main assumption made when investigating trusses is that all the bars are pin-connected at the joints. This assumption is generally possible with small ratios between the depth of the bar section h and its length L ($h/L \leq 1/10$).

The simplest way of determining the forces in the elements of roof trusses induced by the dead load is the graphical one, by plotting Cremona's force plans as for a statically determinate truss. It is also possible to compute the forces analytically.

When a roof is designed without purlins the top chord of a roof truss, besides compressive forces, may also resist local bending due to the supporting of a large-size slab at the middle of the panel (though, as has been mentioned above, in this instance, with respect to the consumption of steel, it is more advantageous to install additional struts).

4.2.4. Selection of truss element section

Determining area of element selection. The elements of trusses are mainly subjected to axial forces, namely, tension and compression. The most convenient section of roof truss elements from the design viewpoint, and therefore the most widespread one, is a section made up from two angles in the form of a *T* shape. Owing to the presence in the section of two identical elements, a member is obtained that is symmetrical with respect to the vertical plane .The joints of the trusses are formed with the aid of gusset plates (gussets), to both sides of which the chord and web elements are secured. A section of two angles with a clearance between them necessary for the gusset can be made up of equal leg and unequal leg angles placed with their wide or narrow legs outstanding. Other sections are also possible.

It is also possible to use thin-wall (lightweight) bent sections in the compression elements of trusses.

The required sectional area of a tension element is determined from expression:

$$A_{req.n} \geq N / R_y \gamma_c.$$

In tension elements, the sections should be of a rigid shape to prevent deformation of the truss in transportation and erection, and also bending of the element under its own weight. For this reason the Building Standards and Regulations do not permit the slenderness ratio of tension elements to exceed 400.

The required sectional area of a compression element is found from expression

$$A_{req.gr} \geq N / \varphi\ R_y \gamma_c.$$

For truss well (lattice) elements in winch the compressive force is relatively low, and small sections with low rigity are obtained, and which, therefore, may easily be deformed during fabrication and erection, a factor of reliability on condition of working $\gamma_c = 0.8$ should be introduced for reducing the design strength. At the same time, an increase in the eccentricity of force application in compression elements is dangerous.

Thus the main parameter influencing the selection of the type of section to be used for a compression element of a truss is factor φ.

The problem of determining the required sectional area of a compression element must be solved by the method of successive approximations, preliminarily assuming various values of factor φ. For the first approximation the following values of this factor can be taken: $\varphi = 0.70.8$ for the chords and $\varphi = 0.5–0.6$ for the web elements.

Effective length of compression roof truss elements. In the critical state the loss of stability (buckling) of a compression element is possible in any direction. Let`s consider two main directions — in the plane of the truss and perpendicular to this plane.

Deformation of the top chord of a truss upon the loss of stability in the plane of the truss may take place between the panel points of the truss. Such deformation corresponds to the main case of buckling with a length coefficient of $k = 1$. The effective length of a compression top chord in the plane of the truss is therefore taken equal to its geometrical length (between panel point centres)

$$L_e = L.$$

For the diagonals (except for the support one, which is considered as a continuation of the chord) and the verticals the effective length in the plane of the truss is taken equal to

$$L_e = 0.8\, L$$

in view of certain constraining of their ends due to the presence of tension elements connected to the gusset. Indeed, upon the loss of stability, a compression diagonal (or vertical) secured to the gusset tends to turn it, but the tension elements also connected to the gusset resist this turning, thus acting as a sort of constraint, which makes it possible to employ a length coefficient of $k = 0.8$.

The effective length of compression elements in buckling perpendicular to the plane of the truss is determined by the distance between their fixed points. It should be remembered that purlins are supporting points for the top chord only if they are restrained to prevent free displacement in a horizontal plane. After ties or stays have been installed in the plane of the top chord there will be ensured (within the limits of elastic behaviour of the bracing horizontal truss) fixation of the purlins restraining the points of the top compression chord in a lateral direction, and thus determining its effective length in buckling perpendicular to the plane of the truss. Generally the ties are located in such a way that there will be a joint fixed in place by these ties in every second panel (it is difficult to secure a tie in the middle of a purlin provided erection conditions). For this reason it is general practice, in determining the slenderness ratio for the compression chord of a roof truss in buckling perpendicular to the truss plane, to take the effective length of the chord equal to the double length of a panel ($L_y = 2d$), even when there are purlins at every joint (if only the purlin is not fastened to a tie). The effective length of compression diagonals and verticals in buckling at right angles to the plane of the truss is taken equal to their geometrical length.

In a roof without purlins, large-size slabs are welded to the top chords of the trusses. Thus they ensure lateral stability of the top chords. During erection, however (or when there is a skylight), ties may nevertheless be required.

Selecting type of sections. The type of angles to be used for the top compression chords of roof trusses is selected with a view to the minimum consumption of metal that will ensure equal stability of the chord in all directions, as well as to creating the rigity perpendicular to the plane of the truss necessary to ensure convenient transportation and erection.

Since the effective lengths of the chord in buckling in the plane and normal to the plane of the truss often differ considerably from each other ($L_y = 2L_x$), then to ensure equality of the slenderness ratios it is essential that the radii of gyration also differ from each other ($i_y = 2i_x$). This condition is met by installing unequal leg angles with their long legs outstanding from the plane of the truss.

If each joint of the top chord is fixed in some way by ties or roof slabs (i.e., $L_y = L_x$), equal stability of the chord is ensured by a section formed of unequal leg angles installed with their short legs outstanding from the plane of the truss, since in this case $i_x = i_y$. However, trusses with such a section of the top chord are not convenient for transportation and erection. They easily bend out of their plane, which leads to additional expenditure for straightening the members and delays in construction. For this reason it will not be good to use such a section for the top chord. Sections made of equal leg angles, from the viewpoint of the most advantageous ratios of the radii of gyration, are only slightly inferior to sections made of unequal leg angles. This is offset, however, by the variety of standard equal leg angles being considerably bigger than that of their unequal leg counterparts.

Thus it can be considered that the most rational shapes for the top chords of roof trusses are either section made of two unequal leg angles installed with their long legs facing outwards, or of equal leg angles.

The best practice will be to employ unequal leg angles placed with their long legs facing outwards for the support diagonals, dividing the effective length of the diagonal in the plane of the truss with the help of a strut. Such a design is advantageous not only from the viewpoint of economy in metal, but also because the strut strengthens the overhanging element of the top chord and prevents its bending during transportation of the trusses.

The remaining compression diagonals and verticals, with insigni-ficant difference (L_x = 0.8L, L_y = L), between their effective lengths are most frequently designed of equal leg angles, inasmuch as the ratio between the radii of gyration i_x and i_y in such a section approximately corresponds to the indicated ratio between the effective lengths. In all the sections considered above common stressing of the two shapes is ensured by the installation of small filler plates in the spaces between the gussets.

For tension elements, the type and arrangement of the angles is not so important, since here the determining factor is the net sectional area. This is why they are made both of equal leg angles and from unequal leg ones installed with their long legs facing outwards (if it is necessary to increase the rigidity in a lateral direction to ensure better erection conditions). In shops with heavy service conditions, the lateral slenderness ratio of the bottom chord should not exceed 250, which with a distance between ties along the bottom chord of 12 or 18 metres also often requires the use of unequal leg angles.

Selection of sections. When selecting the sections of truss elements, the tendency should be to use the smallest possible quantity of different numbers and sizes of angle shapes in order to simplify rolling and reduce transportation costs (since rolling at the mills is specialized by sections). It is usually found possible to select rational sections of roof truss elements by employing angles of 6–8 different standard sizes.

Selection of sections commences with the compression element, which resists the largest design force. Then the element is selected, which withstands the minimum force (most frequently with a view to the limiting slenderness ratio), and thus the range of angle sections is established. When selecting angle sections for compression elements, the tendency should be to use angles of the smallest possible thickness, since their radii of inertia have relatively hiah values.

To avoid the possibility of bending during transportation, a minimum angle section of 50×5 is taken. Of considerable importance for the sections selection is the restriction of the limiting slenderness ratio of the elements, necessitated by the tendency to prevent sagging, vibrations under a dynamic load, bonding in transportation of the truss elements, etc.

With small forces in compression elements, the section is selected with regard to the given limiting slenderness ratio λ_l. First the required radius of gyration is found $i_{req} = L_o / \lambda_l$, and in accordance with this radius corresponding angles are selected.

When selecting the sections of the truss chords the most rational section could be taken for each panel (in accordance with the change in the force), but this would lead to the necessity of having a large number of joints and a great diversity of sections. Such a design will considerably increase labour consumption for manufacturing, complicate the metal ordering, and almost entirely hinder the possibility of economy (owing to the necessity of making joints). For this reason it is usual practice to select chords of one section in trusses with a span up to 24 metres, this section being selected with a view to the maximum force. In roof trusses with spans of 24 metres and more, it is expedient to change the section of the chords along the length of the truss, the changes being made, as a rule, only in the width of the chords. The retaining of a single angle thickness will facilitate splicing with the help of splice angle straps.

Having selected the section of an element, the design stresses in it should be checked, compressed and flexural elements being investigated for stability not only in the plane of the truss, but also perpendicular to its plane by means of expression taking into account the influence of local bending.

The thickness of the gussets used to form the joints of trusses is selected depending on the value of the maximum force in the support diagonal, the same thickness of the gussets being used, as a rule, throughout the truss. In trusses with large spans, the support gussets can be two millimeters thicker than those of the intermediate joints. The results of calculation and sections selecting are generally provided in a tabular form. Fig. 4.2 is the example of working drawing of a roof truss.

4.2.5. Design of trusses. Details of joints

Centering of element. Configuration and fastening of gussets. The design of a truss begins with drawing of the centre lines forming the geometrical layout of the member. When doing this, strict attention must be paid to seeing that the centre lines of elements converging at a joint intersect in its center.

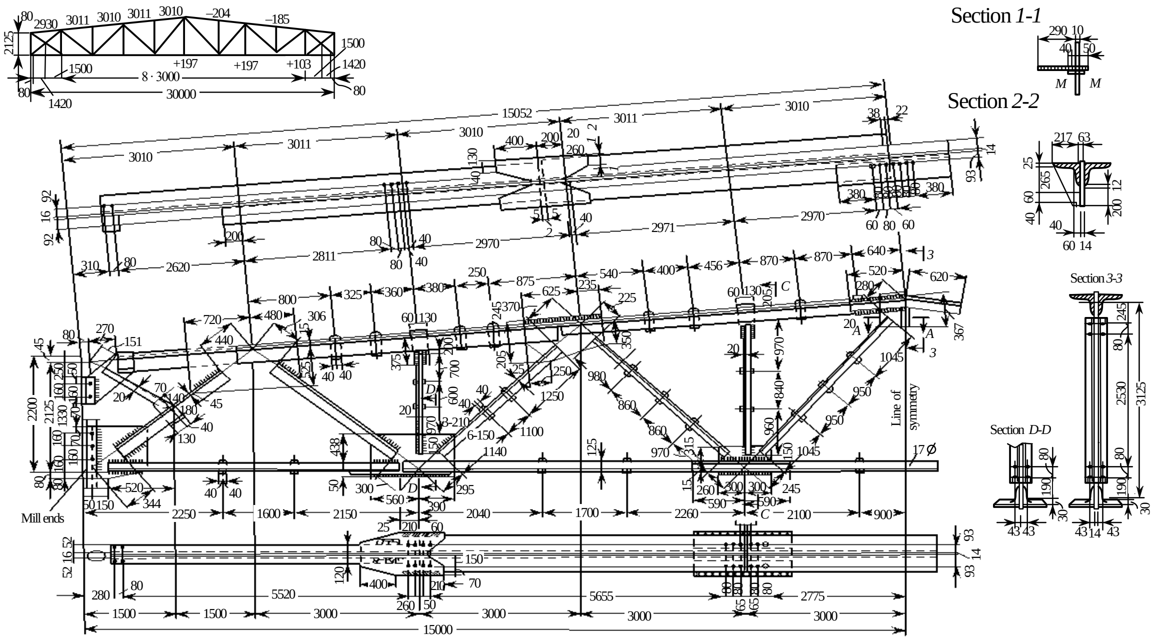


Fig. 4.2. Working drawing of truss

Notes: 1. Drill or punch all holes to diameter of 20 mm except where otherwise specified. 2. All cut off legs 40mm except where otherwise specified. 3. All welds with h_w= 6 mm, except where otherwise specified. 4. Use type 3 – 42 electrode for welding. Then the contour lines of the elements are added to the drawing to ensure the centre lines coinciding with the section gravity centre or being as close to it as possible. When doing this in welded trusses, the distance *z* from the gravity centre to the back end is rounded off to the nearest greater integer that is a multiple of five; in riveted trusses the angles are centred according to the rivet gauge lines. The angles of the web members, as a rule, should be cut off at right angles to the centre line, leaving a space of 30–50 mm between the ends of these elements and the chord. The configuration of the gussets at the joints, which is determined by the conditions of arranging the welds or rivets used for connecting the web elements, should be as simple as possible. Seeing that the gusset transmits the force from one element to another, each of its sections must be strong and capable of withstanding the corresponding force field.

It is better to secure gussets to chord angles on two sides of the latter- the back and the edge, since otherwise the chord angles may be easily bent back owing to chance causes (for example, during transportation). It is not always convenient from the view point of design, however, to let the gusset protrude beyond the surface of the chord, for instance, when installing purlins along the top chord that are connected to short angles. In this case an interval of 5 mm is left between the edges of the gusset and the back of the chord angle, and the gusset is secured only by means of the weld at its leg edge. It is desirable in this instance to weld up the space formed between the backs of the short angles and the gusset, but this weld cannot be considered as one subject to investigation, since it is difficult to ensure its good penetration (the weld is a caulking and not a penetration, one). Thus the main load-carrying welds to be investigated in this instance are the welds applied at the chord angle edge.

The force for which the connection of the gusset is investigated and which tends to displace it with respect to the chord, is the resultant of the forces in the web elements converging at the given joint. In a particular case, when there is no external load applied to the panel point of a straight chord, force Fg is equal to the difference between the forces in the adjacent panels of the chord $Fg = F_2 - F_1$.

Chord splices. Owing to the limited length of rolled shapes, as well as with a view to conditions of transportation, it becomes necessary to break up large-span trusses (over 18 metres long) into separate shipping elements, designing the field splices, as a rule, an the middle of the span.

Here the main rule of splicing must be observed that the sectional area of the splice plates or straps should never be less than that of the elements being spliced. Truss chord splices may be located both at a joint and in a panel. It will be more convenient to locate the chord splice at a joint, since part of the gusset is used as a splice plate. The simplest design of a splice is the covering of the chord angles with splice ones having the same section. In a welded splice the vertical legs of the splice angle are partly cut away in order to avoid concentration of the welds at the edges. To ensure more uniform transmission of the force, it is good to cut long angle legs obliquely. The splice of a top chord, usually located at the ridge of the truss, may be designed similar to the splice of a bottom chord, covering it with bent splice angles.

To increase the number of identical member elements used and therefore, to reduce the labour consumption and costs in fabricating members, the tendency should be to divide trusses into two absolutely identical shipping elements. An excellent solution here is a splice of the bottom chord for trusses with a span of 24 and 36 metres, in which the diagonals converge toward the middle of the chord. Here the gusset is split in the middle, while the splices of the chord angles are staggered. In this design two whole angle sections are obtained along the axis of the chord, while only one angle terminates at the splices, which is covered by the splice angle and the gussets. When angles with different leg thickness are employed in the chord of a truss, the chords can be spliced with the help of gussets and plate straps.

Support joints. Roof trusses can be carried on reinforced concrete columns, brick walls or steel frame (skeleton) elements of industrial and civil buildings, viz., steel columns or secondary trusses. The bearing plate of the truss, generally 16–20 mm thick, is secured to the column by means of anchor bolts with diameter of 22–24 mm. The dimensions of the plate are determined on the basis of the design compressive strength of support material.

To standardize the design of the trusses support joints resting on reinforced concrete and steel columns, as well as on secondary trusses, standard support joint can be used. Here the support load is transmitted through the milled edge of the support gusset, strengthened with side straps, onto special column caps.

The bearing plate of the column cap after installation is welded to the plate of the column head-piece. The trusses are connected to the column caps by means of unfinished bolts. For trusses with a span up to 42 metres there is generally no requirement that the support connections be movable (Fig. 4.2).

Details. As has already been indicated, compression elements of trusses consisting of two angles should be connected to each other in the intervals between the gussets by means of small connecting plates. If this is not done, the longitudinal compressive force F may cause each angle, which resists a force of $F/2$, to buckle independently of the other one, since in a single section the minimum radius of gyration is considerably smaller than that of the section made of two angles.

These plates are arranged along the length of the compression elements at distances of $L_1 = 40\ i$ from each other (where i is the radius of gyration of an angle with respect to the axis parallel to the plane which the plates are arranged in).

To secure better common stressing of both angles of tension elements in trusses, connecting plates are also installed, but spaced at distances of $L_1 \leq 80\ i$. The plates are usually made 60–100 mm wide.

4.3. Columns

4.3.1. Types of columns

Columns serve to transmit the load from members located above them through the footings to the soil. Depending on how the load is transmitted, axially loaded and eccentrically loaded columns are distinguished. Axially loaded columns resist a longitudinal force applied along the axis of the column and inducing uniform compression of its cross-section.

Eccentrically loaded columns, besides axial compression induced by the longitudinal force, also withstand bending originated by a moment.

Every column consists of a body or shaft, which is the main load-carrying element, a head-piece (capital) serving as a support for the member above the column and distributing the load over the section of the column, and a base (footing) that distributes the concentrated load from the column over the surface of the foundation and secures the column in the foundation.

Columns are divided with respect to type into columns with a constant and with a changing section in depth, as regards the design of the shaft section into solid and open-web columns, and with respect to the method of fabrication into welded and riveted ones.

4.3.2. Axially loaded columns

In axially loaded columns, the load is applied either directly to the centre of the column section (Fig. 4.3, *a*), or symmetrically with respect to the axis of the shaft (Fig. 4.3, *b*). When designing axially loaded columns, attention should be devoted to ensuring equal stability of the column, i.e., equal slenderness ratios of the column with respect to the principal axes of the section. The required sectional area of a column shaft is found from the basic expression for analyzing compression members

$$\sigma = F / \varphi A \le R_y \gamma_c. \qquad (4.1)$$

Namely, $A_{req} \ge F / \varphi R_y \gamma_c$.

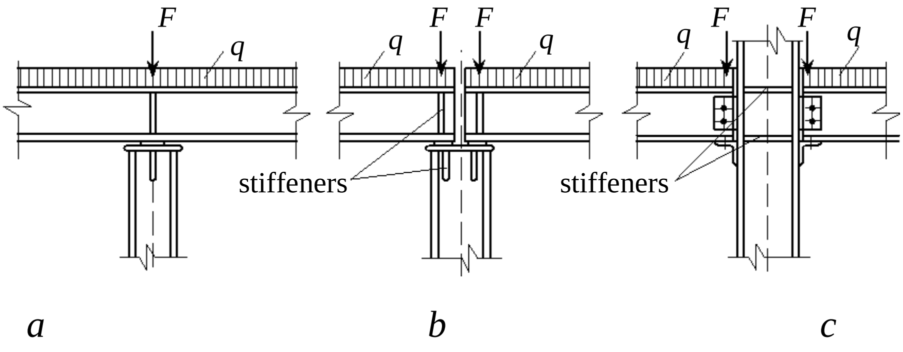


Fig. 4.3. Axially loaded columns

With given design load F acting on the column and design strength R, the minimum area A_{req} will be obtained in such a column whose buckling factor φ is the highest. The effective length of a column: $L_e = kL$, depends upon the character of the restraint of the column ends.

Let`s consider axially loaded solid and open-web columns.

Solid columns. Types of sections. A solid column shaft is formed of one or several rolled shapes or plates connected together by welding or riveting. The most rational section from the standpoint of material behaviour is a tubular one, which, however, is seldom used in practice. The principal section of solid axially loaded columns is a welded I section built up of three plates, although in such a section the condition of equal stability cannot be met. A single rolled I section is rarely employed as a compression element owing to considerable difference between the moments of inertia I_x and I_y. It may be used as an independent section only in columns that are braced in height perpendicular to the *y*-axis. Otherwise it requires strengthening with plates.

Welded I sections built up of three elements can be manufactured with the use of automatic welding; accessibility of all the surfaces of the column simplifies the design of connections with adjacent elements.

Use is sometimes made of sections consisting of three rolled shapes. Such sections, however, are heavier than the ordinary ones.

Solid riveted columns consist of plates and angles.

Analysis and design of solid column shaft. The analysis of a column commences with determination of the loads acting on it. Then for purposes of selecting the section, the required sectional area of the column shaft is computed from expression (4.1). For this end an approximate value of 0.75 to 0.85 is preliminarily assigned to the buckling factor φ.

The dimensions of the section are established with a view to the following considerations. Plates with thickness of t = 8 to 40 mm are used for the flanges, and with thickness of t = 6 to 16 mm for the web, depending on the size of the column. The depth of the column section h in structures of an ordinary type, i. e., with the height of the column H = 10 to 20 metres, is taken equal to at least h = (1/15 to 1/20) H.

The width of the flange plates should be selected so that the plate will not lose its stability under the action of the normal compressive stresses. This requirement is in essence similar to that indicated for selecting the section of beam compression flanges. In columns, however, it is obviously desirable to have the loss of local stability of the flanges occur with critical stresses somewhat higher than those of the column as a whole, and these stresses, as is known, are a function of the column slenderness ratio. For this reason the building standards give the maximum design width of overhang of the plate (flange) h_{ef} depending on the slenderness ratio of the column

$$h_{ef} \;/\; t_f \leq \lambda_{uf} \sqrt{\;} E \,/\, R_y.$$

In a similar way the slenderness ratio of the column web, i. e., the maximum ratio between the design depth of a solid column web and its thickness, depends on the degree of its restraint in the flanges. The latter, in turn, depends on the slenderness ratio of the column as a whole. For this reason the maximum slenderness ratio of a solid column web, as stipulated by the building standards, is determined from the equation

$$\lambda_w = h_w \,/\, t = \lambda_{uw} \sqrt{\;} E \,/\, R \tag{4.2}$$

but should not exceed 75.

With understressing of a column, the maximum values of the ratios b / 2t_{fl} and h_w / t can be increased $\sqrt{\;} R\varphi/\sigma$ times (here $\sigma = F/A$ and φ is the buckling factor). With ratios exceeding the values obtained from equation

(4.2), stability of the web is not ensured and it must be braced with a twin longitudinal stiffening rib. It is recommended to include the rib in the design area of the section. Besides, with h_w / t ≥ 70 transverse ribs are installed that are spaced along the column height at distances of at most (2.5–3) h_w.

The dimensions of transverse and longitudinal ribs in welded columns are established from considerations of design, namely, the width of a transverse rib $b_r \geq (h_w / 30) = 40$ mm, the thickness $t_r \geq b_r / 15$, the width of a longitudinal rib $b_r \geq 10\ t_w$, and the thickness $t_r \geq 0.75\ t_w$. The longitudinal ribs prevent wavy buckling of the web, while the transverse ones increase the rigidity of the section by bracing the flanges.

Having selected the dimensions of the section that comply with the design requirements, the actual slenderness ratio λ and the buckling factor φ corresponding to it are determined for the column, after which the stresses are checked by means of expression (4.1).

In welded columns the elements of the section are connected to each other by means of continuous welds, taking the size of the weld $h_{weld} \cong 0.5\ t_{web}$ (6 to 10 mm). When automatic welding is used, the flange welds should have a uniform size over the entire length of the column, whereas with manual welding it is recommended to increase the size of these welds at the connections of beams, girders and collar beams, as well as at the base (over lengths of about one metre).

Design lengths of colomns (uprights). The design lengths l_{ef} for colomns (uprights) with constant cros-section or for separate part of stepped colomns should be determined by the formula

$$l_{ef} = \mu\ l,$$

here l is the geometrical length of colomn or its sertain part or story hight;

μ is the coefficients of design lengths.

To determine the coefficient of design length, the values of longitudinal forces in the design model elements should be assumed, as a rule, for that combination of design loadings which is analysed to examine stability of colomns and their cross-section elements according to the requirements of paragraphs 1.4 i 1.6 of Code /10/.

When determining the design length coefficient of frame colomns (uprights) it's necessary to differ braced (unfree) and unbraced (free) frames. In the first case, the points of joining cross bars and colomns are braced to avoid horizontal displacement in the frame plane. In the second case, the points of joining cross bars and colomns aren't braced.

The design length coefficient for a colomn (upright) with constant cross-section along its axis should be determined depending on grip conditions of its ends and type of loading. The values of design length coefficient μ - for some kinds of grip conditions and types of loading are presented in table 1.9.7/10/.

Open-web columns. Types of sections and lattices. An open-web column shaft consists of two or more rolled sections connected to each other in the chord planes with batten plates or lattices.

The principal advantage of open-web columns is the possibility of ensuring equal stability in them.

Open-web columns are sufficiently economical as concerns the consumption of metal. At the same time more labour must be spent in their fabrication, since the multitude of short welds makes it difficult to employ automatic welding.

The section of an open-web column shaft is generally formed of two channels located with their flanges facing inside the section. The arrangement of the channels with their flanges facing outward, with the same overall dimensions of the section, is less advantageous from the viewpoint of metal consumption, and is employed only in riveted columns to make riveting more convenient. A section made of I shapes is used only with considerable loads that prevent the employment of channels.

A section built up of four angles is employed for compression elements having a large length (masts, crane booms, etc.), in which definite rigity is required in both directions. Such a section is sufficiently economical, and a comparatively light design is obtained, but the presence of lattices in four planes results in high labour consumption for fabrication.

The lattice of open-web columns is generally designed of single angles with the maximum slenderness ratio of an element of $\lambda = 150$. The lattice used is either of the simple triangular type or with braces or diagonals. The lattice is secured to the branches of the column either by welding or riveting, it being permitted to centre the angles on the outer edges of the branches. Columns with batten plates are simpler to fabricate, have no outstanding lattice angles, and their appearance is better; columns with lattices are considerably stiffer, especially against torsion.

Behaviour of open-web column shaft under load. The two branches of an open-web column shaft are connected by means of batten plates or lattices into a single member. If there were no such connection, each branch would be subjected under load to buckling with respect to its own axis (Fig. 4.4, axis *1–1*). When batten plates or lattices are installed, the rigity of the shaft as a whole grows considerably, with both branches resisting the load together, as a single section, and withstanding buckling with respect to axis *y–y* (Fig 4.4, *a*).

This axis, as distinguished from the material axis *x–x*, which intersects the body of the column, is known as the free axis. The slenderness ratio λ_x

of an open-web column shaft with respect to the material axis $x–x$ is equal to the slenderness ratio of one branch with respect to the same axis, since

$$r_x = \sqrt{I_x/A}.$$

On the other hand, the slenderness ratio with respect to the free axis $y–y$ depends on the distance between the branches (dimension $2a$) in fig. 4.4, *a*.

The moment of inertia I_y of a section made up of two branches is expressed by the equation

$$I_y = 2\,(I_{br} + a^2 A_{br}), \tag{4.3}$$

where, I_{br} is the moment of inertia of one branch with respect to its own axis *1–1*; A_{br} is the sectional area of one branch; a is the distance from the axis of branch *1–1* to the free axis of column $y–y$. It would seem that the slenderness ratio of the column shaft with respect to the free axis should be found from the equation

$$\lambda_y = L_y/i_y, \tag{4.4}$$

where, L_y is the effective length of the shaft in respect to the axis $y–y$.

In actual conditions, however, the slenderness ratio of the column with respect to the free axis is found to be greater owing to elastic yielding of the batten plates or lattices. This so-called equivalent or reduced slenderness ratio is equal to

$$\lambda_{red} = k\lambda_y,$$

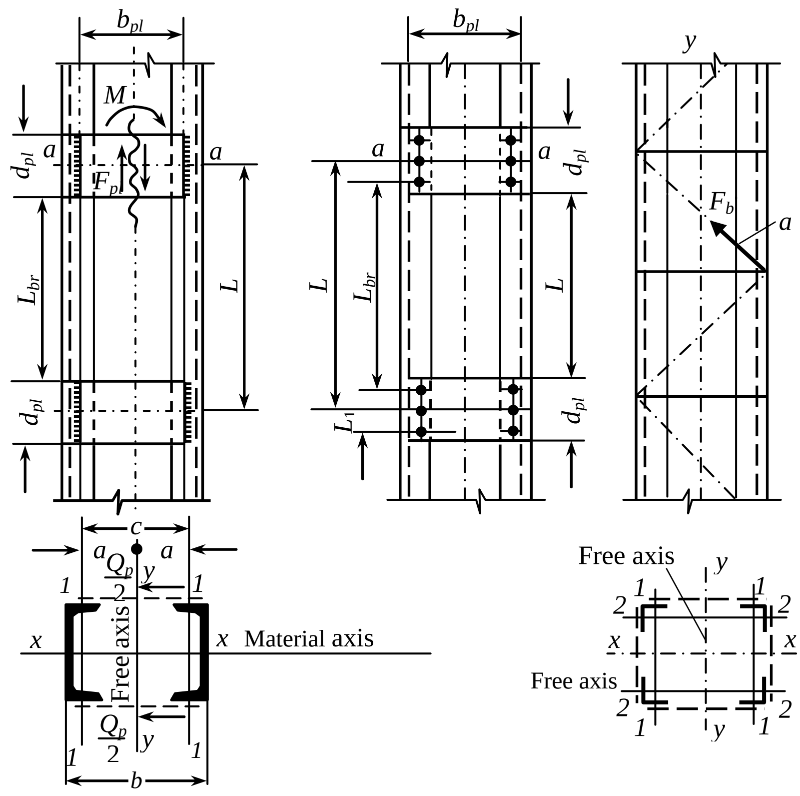
where, $k > 1$ is a reduction factor for a built-up column shaft depending upon the deformability (yielding) of the batten plates or lattices.

For columns with batten plates

$$k = \sqrt{1 + (\lambda_{br} / \lambda_y)^2}$$

and for columns with lattices

$$k = \sqrt{1 + (C_a A/\lambda_y^2 A_d)}.$$



a *b* *c*

Fig. 4.4. Analysis of open-web axially loaded columns

Thus the reduced slenderness ratio will be:

for columns with batten plates

$$\lambda_{red} = \sqrt{\lambda_y^2 + \lambda_{br}^2} \tag{4.5}$$

for columns with lattices

$$\lambda_{red} = \sqrt{\lambda_y^2 + C_a A/A_d}, \tag{4.6}$$

where, $\lambda_y = L_y / i_y$ is the slenderness ratio of the entire column shaft with respect to the free axis determined from equation (4.4); $\lambda_{br} = L_{br} / i_{br}$ is the slenderness ratio of the part of the branch between batten plates with respect to own axis; A is the sectional area of the entire shaft; A_d is the sectional area of two diagonals of a lattice (in two planes); C_a is the coefficient whose value depends upon the angle α between the diagonals of the lattice and the column branch:

with $\alpha = 30°$ $40°$ $45–60°$

$C_a = 45$ 31 27.

The second term in the radicand of equations (4.9) and (4.10) takes into consideration the slenderness ratio of the branches and the yielding of the batten plates or lattice, and thus determines the required location of the latter, since with a change in these quantities the reduced slenderness ratio also changes.

The design slenderness ratio used to determine the buckling factor φ will be greater of the two ratios λ_x or λ_{red}. By moving apart the branches (i. e., by increasing the distance a in Fig. 4.4, *a*) it is possible to lower the value of λ_{red} without any noticeable increase in the quantity of metal required, and thus meet the stipulation that $\lambda_{red} \leq \lambda_x$; accordingly, when selecting the section of a built-up column, the required slenderness ratio is generally determined with' respect to the material axis.

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For columns consisting of four branches (Fig. 4.4, *c*), the reduced slenderness ratio will be

$$\lambda_{req} = \sqrt{\lambda^2 + A\left(\frac{C_{a-1}}{A_{d-1}} + \frac{C_{a-2}}{A_{d-2}}\right)},$$

where, λ is the maximum slenderness ratio of the entire column with respect to the free axis; A_{d-1} and A_{d-2} are the sectional area of two diagonals of lattices located in planes that are respectively perpendicular to axes *1–1* and *2–2*; C_{a-1} and C_{a-2} are the the same as C_a for the respective planes.

In the branches of open-web columns, the slenderness ratios of separate branches should not exceed 40 in columns with batten plates and should not be more than the reduced slenderness ratio of the entire column in columns with lattices.

In axially loaded columns the connecting elements — the plates or lattices — are investigated for the lateral (shear) force that may appear upon bending induced by the critical force, which, as is known, depends for a given material only on the geometrical dimensions of the column shaft. In accordance with the building standards, the value of this fictitious shear force induced by buckling is determined, depending on the section of the shaft, from the equations for steel 3 $Q = 0.20\ A_{gr}$, kN, for steel 5 and low-alloy steel

$$Q = 0.40\ A_{gr},\ \text{kN}, \tag{4.7}$$

where, A_{gr} is the gross section of the column shaft, cm^2.

The shear force Q is taken constant over the length of the shaft and is equally distributed between the planes of the batten plates (lattices).

Analysis and design of open-web column shafts. As in solid columns, selection of the section of an open-web column shaft is commenced with calculation of the required sectional area on the basis of the design load and design strength of the material. For this purpose a value of 0.7 to 0.9 is preliminarily assigned to the buckling factor φ. After this, the required sectional area of one branch is found from the equation

$$A_{br.\ req} = F/\ 2\varphi R.$$

Taking into account this area, the nearest number of channel or I shape is selected from the catalogue of standard shapes, and its slenderness; ratio with respect to the material axis x–x is determined. Then expression (4.1) is used to check the design stress in the column for the selected section on the basis of the slenderness ratio with respect to the material axis x–x. The next step is to plan the layout of the section and check it with respect to the free axis. It is essential to so arrange the branches of the section and design the lattice as to comply with the condition $\lambda_{red} \leq \lambda_{x}$.

If for columns with batten plates, we assume as the first approximation that its branches are spaced at such a distance $2a$ from each other, whereupon

$$\lambda_{red} = \sqrt{\lambda_y^2 + \lambda_{br}^2} = \lambda_x$$

then in this instance the required slenderness ratio of the column with respect to the free axis will be

$$\lambda_{y.\,red} = \sqrt{\lambda_x^2 - \lambda_{br}^2}.$$

Ordinarily the slenderness ratio of one branch is taken within the limits of λ_{br} =30 to 40. Having found the value of $\lambda_{y.red}$, the required radius of gyration can be computed from equation (4.4), namely, $i_y = L_y/\lambda_{y.red}$, and the latter can be used to find the required moment of inertia I_y and the corresponding dimension a from equation (4.3).

The batten plates are so located in columns as to ensure compliance with the previously assumed slenderness ratio of the branch, i. e., $L_{br} = \lambda_{br}\ i_{br}$. Here the effective length of the branch is taken equal to the clear distance between the plates in welded columns (Fig. 4.4, *a*), and to the distance between the extreme rivets of adjacent plates in riveted columns (Fig. 4.4, *b*).

The length of a batten plate b_{pl} depends on the distance between the branches. In welded columns, overlapping of the plates onto the branches is about 40–50 mm. The width of the plate d_{pl} is established from the viewpoint of properly locating the welds or rivets used to secure the plate to the branch of the column. The thickness of a plate is taken from 6 to 12 mm, and in heavy columns must be checked . Besides, it should be seen that the ratio b_{pl} / t_{pl} doesn't exceed 50.

Batten plates and their connections to the branches of a column are investigated for the shear force $F_{sh.pl}$ and the moment M_{pl} acting in the plane of the plate and appearing in it as the result of the action of the fictitious shear force Q bending the column. Investigation is conducted with the aid of the equations

$$F_{sh.pl} = Q_p L / c, \; M_{pl} = Q_p L / 2,$$

where, L is the distance between batten plate centers; c is the distance between the axes of branches; $Q_p = Q / 2$ is the shear force acting on the system of plates located in one plane; Q is the shear force computed from equations (4.7).

The adequate strength of a plate is checked by means of the expression

$$\sigma = M_{pl} / W_{pl} \leq R_y \gamma_c.$$

The elements of a lattice are investigated for the axial forces that appear in them owing to the action of the shear force Q induced by buckling.

The compressive force in a diagonal (when the lattices are located in two parallel planes) is determined in the same way as in the elements of a truss lattice

$$F_d = Q / 2 \sin\alpha, \tag{4.8}$$

where, α is the angle between the diagonal and the branch.

As already indicated, for members made of steel 3 the fictitious shear force Q induced by buckling is taken equal to 0.20 A_{gr} kN (where A_{gr} is the sectional area of the entire column in cm^2). The stress in a diagonal should not exceed the design strength

$$\sigma = F_d / \varphi A_d \leq k_s R_y \gamma_c,$$

where, A_d is the sectional area of one lattice diagonal.

The minimum section of lattice elements employed in welded columns is an angle 45×4, while the minimum section of the lattice elements of a riveted column is dictated by the rivet diameter to be used. The maximum slenderness ratio of the lattice elements is taken equal to $\lambda = 150$.

To prevent twisting of open-web column shafts, transverse membranes are installed at about every four metres along the height of the shaft, regardless of the size of the lattice. They can be solid or built up of angles.

4.3.3. Eccentrically loaded columns

Eccentrically loaded columns are used to the greatest extent in the frameworks of industrial buildings, where they generally form part of the system of rigid transverse members of a shop (lateral frames).

Types of eccentrically loaded columns in industrial buildings. In accordance with the design of the column shaft three types of industrial building columns are distinguished.

Constant dimension columns. Such columns are generally employed in shops with overhead cranes having a lifting capacity of up to 10–15 tons. To save metal, such columns are being replaced at present, as a rule, with precast reinforced concrete ones.

Variable dimension (stepped) solid and open-web columns. These columns are the most widespread type used in industrial buildings and are fit for the heaviest loads. The lower part of the column having a length of L_1 is referred to as the crane runway part, while the upper part with a length of L_2 is termed the roof-supporting part. In outside columns, i.e., with the cranes located only on one side, the section consists of the inside crane runway branch directly resisting the crane load, and the outside, roof-supporting branch. In solid columns, both branches are connected by a solid plate, while in open-web ones use is made of lattices consisting of angles arranged in two planes.

Divided columns. It will be good policy to use such columns in shops with a heavy crane load (with cranes having a lifting capacity of over 150 tons) and a relatively small height (up to 15–20 metres). The crane runway upright of a divided column is connected to the roof-supporting column by means of a number of horizontal batten plates. Owing to the low rigity of these plates in the vertical plane, the crane runway upright withstands only axial compression induced by the crane load, without transmitting it to the roof-supporting branch.

Types and dimensions of eccentrically loaded column sections. If constant dimension columns are employed, the depth of the section h is usually taken equal to $h \cong L/15$ for columns of 10–12 metres high, $h \cong L/18$ for columns of 14–16 metres high and $h \cong L/20$ for columns of $L \geq 20$ metres high. As a rule, welded I sections are used in these columns.

In variable dimension columns, the depth of the section of the roof-supporting part is selected within the limits of 1/8 to 1/12 of its height L_2. This dimension with cranes of medium lifting capacity is generally taken equal to 500 mm, it being increased only with a large value of L_2 and with heavy cranes (Q exceeding 100 tons), as well as when it is necessary to design a passage through the web of a column. In such cases the depth of the column section is generally taken equal to 1000 mm. The axis of the roof-supporting part above the crane runway, as a rule coinciding with the middle of the section, also coincides with the marked out centre line of the side structure. The distance from this line to the outer surface of a column is taken equal to 250 or 500 mm.

The section of the roof-supporting part of stepped columns is usually taken in the form of a welded symmetrical I section.

The sections of the crane runway part of stepped columns can be solid and latticed. The sections of outside columns having one crane runway branch are unsymmetrical, while the sections of the middle columns in multispan buildings with cranes having an identical lifting capacity are symmetrical.

The crane runway branches of stepped columns, as a rule, are designed with an I section, whereas the outside (roof-supporting) branch generally has a channel section or is made of a plate with a smooth external surface. This is necessary for convenient connection of the wall panels. Both branches, if possible, should be designed of rolled shapes.

The depth of the section of a stepped column crane runway part is determined by the standard span of crane bridges, which are multiples of 0.5 metre, and the spans of the shop, which are multiples of 3 metres. The distance D_a between the axis of the crane track and the marking out axis (Fig. 4.5) is in the majority of cases taken equal to 0.75 or 1.0 metre, in view of the clearances that must be provided between the crane and column.

The depth of the column section h also depends on the height of the column H, since the latter determines the rigity of the structure characterized by horizontal deflections.

Solid columns have a somewhat greater rigity than their open-web counterparts, and are simpler to manufacture; however, when having a width of about 1.2–1.5 metres and above they are less economical.

Analysis and design of eccentrically loaded column shafts. **Solid columns**. When analyzing columns in which the compressive load is applied eccentrically with respect to the axis of the column, it is always possible to transfer the compressive load to the axis, adding at the same time a bending moment. By the axis of a column is meant the line connecting the centres of gravity of the column sections. In variable-dimension or stepped columns the axis is also stepped, which is taken into account when determining the moments acting on the column.

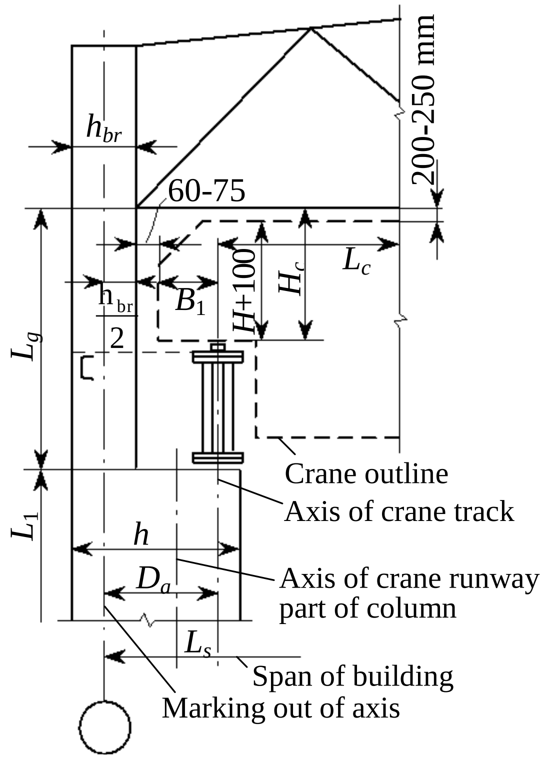


Fig. 4.5. Location of axes and crane clearance in stepped column

Thus the section of an eccentrically loaded solid column is selected and investigated for the action of the longitudinal force F applied along the axis and the moment M, whose values have been obtained by statical analysis of a frame or of a separately designed column.

The type of column to be used, as well as the type and depth of its section, is generally selected when designing the layout of a structure as a whole. When selecting a section, its area is planned in accordance with the selected depth.

The stability of an eccentrically compressed bar in the plane of action of a moment can be checked by means of the expression

$$\sigma = F / \varphi_{ec} A_{gr} \le R_y \gamma_c, \qquad (4.9)$$

where, φ_{ec} is the factor applied for reducing the design strength to the value of the critical stress of the given element. This factor is determined for columns with a solid section, depending on the slenderness ratio of the column λ and the equivalent eccentricity m_1.

The equivalent eccentricity m_1 is determined from the equation

$$m_1 = \eta m = \eta e / \rho = \eta MA / FW,$$

where, $e = M / F$ is the eccentricity of the longitudinal force in the plane of bending, the moment M in stepped columns being taken as the maximum one over the length of the part of the column having a constant section; $\rho = W_{gr}/A_{gr}$ is the core radius, cm; η is the coefficient allowing for the influence of the shape of the section, $\eta = 1.3…1.6$.

The slenderness ratio of the column can be taken within the limits of λ = 50–90 (on the average $\lambda = 70$).

With great eccentricities (m_1 exceeding 4) the influence of the normal force and the value of the column slenderness ratio are reduced, and therefore in this instance it is possible to use the two-term expression

$$\sigma = F/A\varphi + M/W \le R_y\gamma_c.$$

This expression can also be used to approximately plan the required sectional area A_{req}.

Assuming that $\varphi = 0.8$ and $\rho = 0.45\ h$, we obtain

$$A_{req} = F(1/\varphi + e/\rho)/R = F(1.25 + 2.2\ e/h)/R.$$

The slenderness ratio of the column λ, as has already been indicated, is determined from the equation

$$\lambda = L_e / i = kL / i.$$

The effective length, determined by the length coefficient k, is the basis of investigations of column shafts for stability and may be considered as the equivalent length of a pinned-end shaft of the same rigity.

When designing the section of a solid column the computed required area A_{red} should be distributed most advantageously, not forgetting to ensure local stability of the section separate elements.

The width of a branch (or of the column section) should be sufficient for ensuring general stability of the column in the direction perpendicular to the plane of the frame. This width is generally taken equal to b = (1/20…1/30) L_1 (the height of the crane runway part of the column). The other, less loaded branch is usually made of the same or nearly the same width, with a view to the convenience of securing the column base plate to the column shaft.

To ensure local stability of the section of the column branches it is essential, the same as in axially loaded columns, to take account of the maximum ratios between the overhangs of the plates and their thickness (b / t_{ch}), which are established by the building standards depending on the slenderness ratio of the column.

It is not good to design a web thinner than 8 mm. If adequate stability of the web is not ensured, it can be reinforced with a longitudinal twin rib over the whole height of the column, as indicated above for axially loaded columns. Since such a solution is connected with a considerable increase in the labour consumption for fabricating the column, it may be considered, when the stability of the web is not ensured, that the web is not subjected to any loads, except for its extreme parts adjoining the branches with dimensions of 15 t on each side. Here the column becomes a sort of an open-web one with the web playing the part of the lattice or lacing.

When the slenderness ratio of the web h_w / $t \geq 70$ transverse ribs must be installed at distances of (2.5 to 3) h_w from each other. These will bind the section into a whole, ensuring its high rigity against twisting of the column.

In heavy columns, membranes are installed over the width of the entire section at intervals of about 4 metres, and are also intended to ensure adequate rigity of the column.

Besides checking an eccentrically loaded column in the plane of bending, it must always be investigated for sufficient stability in a plane perpendicular to that which the moment acts in. This is performed by means of the expression

$$\sigma = \mathrm{F} / c\varphi_y A \le R_y\gamma_c,$$

where, φ_y is the buckling factor taken in accordance with the slenderness ratio λ_y in a direction perpendicular to the plane which the moment acts in; c is the factor accounting for the influence of the moment on the sta-bility of an eccentrically loaded element, provided the bending and twisting form of loss of stability, equal to

$$c = \beta / (1 + \alpha m_x),$$

where, $m_x = e_x / \rho_x$ and $e_x = M_x / F$.

Here M_x is taken equal to the maximum moment within the limits of the middle third of the length (but not less than half of the maximum moment over the length of the column) for columns with ends fixed against a displacement perpendicular to the plane which the moment acts in, and to the moment at the constraint for columns with free ends.

Open-web columns. The lower (crane runway) part of a column is in the majority of cases (when the width exceeds 1.2–1.5 metres) designed of the open-web type, consisting of two branches connected by lattices.

The analysis of open-web columns is based on the assumption that they behave in the same way as a truss with parallel chords. For this purpose the longitudinal force and the moment acting on the column are distributed among the branches, the forces in which are found from the equation

$$F_{br} = Fz / h\ M / h,$$

where, z is the distance from the centre of gravity of the column section to the axis of the branch opposite the one being investigated.

In a symmetrical section z = 0.5, whereas in an unsymmetrical one the distance from the centre of gravity to the branch resisting the greater load varies from 0.4 h to 0.5 h. Similar to a solid column, the section of the crane runway branch is designed of a rolled or (in very large columns) welded I shape. To ensure adequate general stability of the column in a direction perpendicular to the plane of the frame, the depth of the I shape is selected to be within the limits of (1/20…1/30) L_1, which corresponds to a slenderness ratio of $\lambda \cong 60…100$.

The roof-supporting or hip branch is generally designed of the channel type, with the same width as the crane runway branch. When no channels of the corresponding number are available, the channel section is made up of angles with batten plates or more frequently with a solid plate.

Both branches are laced together with lattices, usually of the triangular system, arranged in two planes.

The section of the branches is checked by means of expression (4.1) used for an axially loaded column

$$\sigma = F_{br} / \varphi A_{br} \leq R_y \gamma_c.$$

Here the buckling factor φ is taken equal to the smaller of two values, the first of which is determined from the slenderness ratio of the branch relative to the axis of the I shape (or channel) having the greater moment of inertia. Here the effective length is taken equal to the distance from the foundation to the crane girder. When finding the second value of φ, the effective length of the branch is taken equal to the distance between joints of the lattice, while the minimum radius of gyration of the branch section is used in the calculations.

The tendency should be to so design the connection of the crane girders to the branch of the column as to avoid eccentrical application of the bearing pressure of the girders with respect to the axis of the column, i. e., in a direction perpendicular to the plane of the frame. This is easily achieved with continuous girders by aligning the bearing (support) rib strictly along the axis of the column, as well as when the bearing member in light simple girders or beams is designed in the form of bearing plates. In heavy girders for cranes with a lifting capacity of 100 tons or more, when employing the bearing member, it is desirable to locate the bearing ribs of the girders closer to their ends to ensure the support reactions being within the limits of the column cross-section core. If this is not the case, the branch of the column must also be investigated for buckling in a direction perpendicular to the plane of the frame with a one-sided crane load.

In addition, open-web columns, especially narrow and high ones (whose section depth is less than l/15 of the length L_1) should be checked for stability on the assumption that the column branches are behaving as a single column with a built-up section. Such a column with lattices or batten plates located in the plane of bending is investigated with the aid of the same expression (4.9).

Each lattice of a column is designed of single angles, the diagonals being arranged at an angle of 45–50 deg to the horizontal. The force in the diagonal of a lattice is determined from equation (4.8) with a view to the maximum actual shear force in the column obtained when investigating the frame, or according to the fictitious shear force (if it is greater) computed from equations (4.7).

The procedure for investigating the diagonal connections and the design requirements are similar to those set forth above for constant dimension columns. The installation of braces reduces the effective length of the column branch. The force in a brace is determined on the basis of the fictitious shear force Q in the corresponding branch.

4.3.4. Splice and details of columns

Both shop and field column splices are used in practice. Shop splices are made owing to the limited length of rolled shapes. Field splices are made because of limitations in transportation possibilities (for example, a maximum length of 13 metres is permissible when transporting members by railway on single eight-wheel flat cars, and of 27 metres when groups of two flat cars coupled together are being used).

The shop splices of elements are ordinarily staggered, and not concentrated at one place, since the separate elements can be connected together before general assembly of the column. Examples of welded shop splices of separate column elements are pictured in Fig. 4.6.

The main condition for the formation of a strong splice is adequate provision for transferring the load from one element to the other. With butt welding this is ensured by selecting the corresponding length of the welds, while when straps are used to make the splices, care should also be exercised to ensure the required sectional area of the straps or splice plates. This must not be less than the sectional area of the main elements being spliced.

The simplest splice, and hence the one most recommended for use, is a straight splice with butt welding. Such a splice can be designed in all instances, since in centrically loaded columns it is always possible to find a section with low tensile stresses.

Field column splices are located at spots that are convenient for erection of the members. In variable dimension columns such a place is the step at the level of the crane girder bearings, where the section of the column changes.

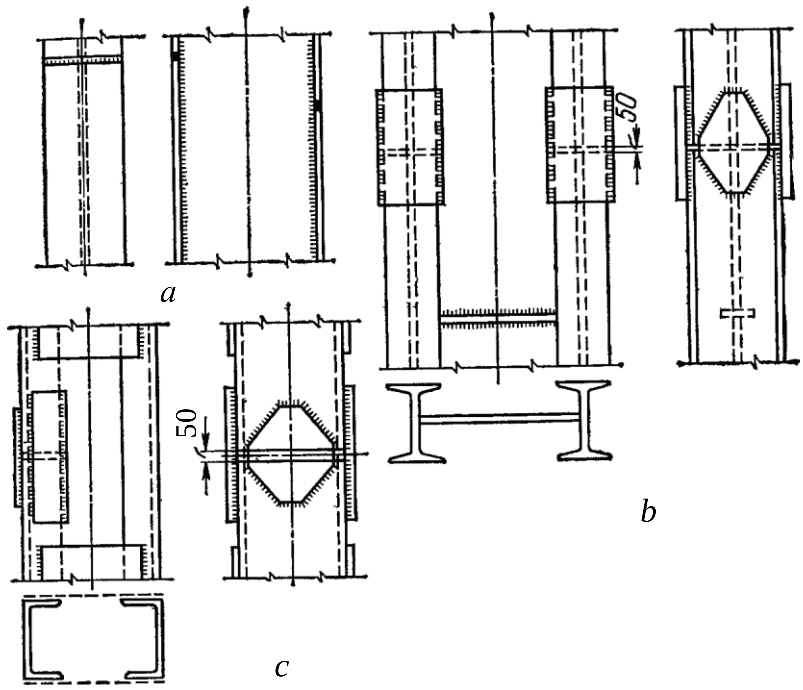


Fig. 4.6. Welded shop column splices:
a — splicing flanges of welded I section; *b* — splicing I section branches of solid column; *c* — splicing branches of open-web column with batten plates

In columns with a lower latticed part the upper part is connected with the help of a component called a cross-piece or traverse. The traverse is subjected to bending in the same way as a beam on two supports and must be checked for adequate strength. The traverse is connected to the branches of the column by means of continuous welds, and the connection is investigated for the support reaction of the traverse. Horizontal membranes or stiffeners are installed to ensure general rigity of the connection between the upper and the lower parts of the column.

Crane girders are seated on constant dimension columns (in shops with a light duty) by using a ledge made from a welded I section or of two channels. The ledge is investigated for the moment induced by the load of two cranes travelling over the crane girders and both operating in the vicinity of the support, namely, $M = Pe$, where e is the distance from the axis of the crane girder to the branch of the column.

4.3.5. Column bases

Types and designs of bases. The base (footing) of a column is designed to distribute the concentrated column load over a certain definite foundation area and to ensure connection of the lower column end to the foundation in accordance with the planned design.

Two basic types of bases are distinguished, namely, pinned and rigid ones.

The simplest kind of pinned base for axially loaded columns is a base consisting of a thick steel base plate on which the milled end of the column rests.

The usage of bases with transmission of the load through the milled end of the column is expedient for columns with a considerable load. For light columns (and also when no end-milling machines are available) bases are employed in which the entire load is transmitted to the plate through welds.

The load can also be transmitted from the column to the base plate with the help of a traverse, which serves for more or less uniform transmission of the force field from the column to the base plate. This, with respect to the nature of the load action, makes the design resemble a rigid “stamp” bearing on the foundation. The traverse simultaneously serves as a bearing member for the base plate, when it is subjected to bending induced by the reaction or upward pressure of the foundation. The traverse itself resists bending as a double-cantilever beam supported on the flanges or branches of the column and loaded with the upward pressure of the foundation.

In eccentrically loaded columns, the rule is to design rigid bases that can transmit bending moments. For this purpose it becomes necessary to develop the traverses in the direction of action of the moment. With relatively small moments at the bearings, the traverses are made from plates 10–12 mm thick or channels.

In columns with greater crane loads and with larger moments at the bearings, it becomes necessary to develop the bases and their traverses to a still greater extent.

In the open-web columns of industrial buildings, use is generally made of footings of the split type, consisting of two independent footings connected by means of angle ties. With a large distance between the column branches, these footings are more economical than their solid counterparts.

The footings are connected to the foundations with the help of anchor bolts (anchors) embedded in the foundation in concreting. In axially loaded columns the anchor bolts are not investigated, and their dimen-sions are established from considerations of design (d = 22–26 mm). In constrained columns subjected to bending the anchor bolts resist tension originated by the bending moment. Their diameter and length are established from the results of analysis.

The anchor holes should be laid out in a horizontal plane with the employment of rigid jigs, and checked by means of geodetic instruments. As a rule, the holes in a footing for the anchor bolts are made with a diameter greater than that of the bolts; they are closed with field washers, which are welded to the footing after the column has been installed in its proper position for erection. When the columns have been installed, the bases are grouted to prevent corrosion.

Analysis of base plate and traverse of axially loaded column. The dimensions of the base plate of an axially loaded column are determined in accordance with the design resistance of the foundation material to axial compression R_{fd}. The minimum area of the base plate will be

$$A_{p\mathrm{l}} \geq F / R_{fd},$$

where, F is the design force in the column.

The design compressive strength of a concrete foundation is found from the equation

$$R_{fd} = R_{con}\, \psi = R_{con}\, \sqrt[3]{A_{fd} / A_{p\mathrm{l}}}\,,$$

where, R_{con} is the design compressive strength of concrete (0,44 kN/cm^2 for grade 100 concrete and 0.65 kN/cm^2 for grade 150); A_{fd} is the area of foundation; $A_{p\mathrm{l}}$ is the area of base plate; ψ is the strength factor whose value should not exceed 2.

Having found the required base plate area, one may commence to design the footing, taking the width of the base plate B somewhat greater than that of the column.

The base plate resists bending induced by a uniformly distributed load (the upward pressure of the foundation)

$$q = \sigma_{con} = F / LB$$

and different parts of the base plate will be in different conditions of bending.

The first part 1 of the plate behaves and is investigated as a cantilever. For this purpose the moment in section l–l is found acting on a strip 1.0 cm wide

$$M = \sigma_{con} c^2 / 2. \tag{4.10}$$

The second part of the base plate 2 behaves as a plate supported on four sides and loaded from below with the same uniformly distributed load $q = \sigma_{con.}$ Such a rectangular plate with the maximum moment acting at its centre is investigated with the aid of tables compiled by Academician B. Galerkin, using the equations

$$M_a = \alpha_1 q b^2,\ \ M_b = \alpha_2 q b^2, \tag{4.11}$$

where, M_a and M_b are the moments calculated for strips 1.0 cm wide in the direction of the dimensions a and b; b is the length of short side of rectangle; α_1 and α_2 is the coefficients depending on the ratio between the longer side of the rectangle a and its shorter side b.

If a/b exceeds 2 the moment can be determined for a strip cut out along the short side, as in a single-span beam.

The third parts of the base plate behave as a plate supported on three sides. The critical point in such a plate is the middle of its free edge. The moment in this section is determined from the equation

$$M_3 = \alpha_3\, q\, d_1^2, \tag{4.12}$$

where, α_3 is the coefficient for analysis of the plate; d_1 is the length of the free edge of the plate.

When a_1 / d_1 is less then 0.5, the plate is analyzed as a cantilever.

The thickness of the plate is determined in accordance with the greatest of the moments computed from equations (4.10), (4.11) and (4.12). The plate should have a thickness adequate to ensure uniform transmission of the load to the concrete without bending, i. e., the footing should behave as a rigid stamp or punch.

The section modulus of a plate with a thickness of t and a width of 1.0 cm will equal W = 1.0 t_2 / 6.

Utilizing the total stress in the plate, equal to the design strength, we have $\sigma = M / W_{pl} = 6M / t_{pl}^2 = R_y$, whence

$$t_{pl} = \sqrt{6M/R}\,.$$

When designing the base of a column care should be taken to have the thickness of the different parts of the base plate determined from equations (4.10), (4.11) and (4.12) close to each other. This can be attained by changing the dimensions *a*, *b* and *c*.

The thickness of a base plate is generally taken within the limits of 16 to 40 mm (except for the plates of columns with milled ends, where the thickness can be greater if required by greater loads).

The height of the traverse is determined to make possible arrangement of the welds through which the loads will be transmitted from the column to the traverse.

Methodical instructions to Chapter 4

In this unit special attention should be paid to the constructions of one-storeyed buildings — beams, beams grillages, trusses and columns.

It`s important to know how to select beams grillages, analyze and check cross-sections of rolled and built-up steel beams, to design splices and connection of beams.

To design roof trusses it’s necessary to study types of cross-section elements of trusses, the methods of analyzing and rules of trusses construction.

While designing a framework of building its necessary to pay attention for selecting cross-sections of columns which is an element of skeleton system.

Questions and Tasks to Chapter 4

1. *What main types of beam grillages can you name?*
2. *What kind of limited state does the selection of cross-section of rolled beams depend on?*
3. *How can we choose cross-section of built-up beams?*
4. *How do we analyze cross-section of elements of roof trusses?*
5. *What types of one-storeyed industrial building columns are used? What is the choice of column construction based on?*
6. *How do we select and analyze cross-sections of an open-web column?*
7. *How do we analyze the cross-section of eccentrically loaded column shafts?*

5. STEEL FRAMES OF SINGLE-STOREY INDUSTRIAL BUILDINGS

5.1. General information

The steel skeleton or frame of an industrial building is the main load-carrying member supporting the roof and the walls, as well as the runways of overhead and other types of cranes serving the production process. Sometimes various technological equipment and working areas are supported directly on the frame.

Before designing an industrial building, the expediency of making the building of steel should be ascertained, since at present, due to the use prestressed reinforced concrete members, the possibility of using steel members, in industrial construction is continuously growing.

The fields of using steel and reinforced concrete in industrial construction are determined mainly by the “Rules for Economical Consumption of Metal, Lumber and Cement in Construction” (ТП 101-85*).

When commencing to design an industrial building, it is essential already when drawing up the preliminary project report to obtain some information both of technological and general construction nature concerning the contemplated service of the structure.

The technological information concerns data on the location of railway sidings in the shop, the arrangement of the crane runways and the lifting capacity of the cranes, on special overall dimensions of machines, on different live loads and their dynamic action, on underground facilities, special working and repair areas, passages and stairs, on the sequence of construction and the possibilities of development and expansion, on the location of service facilities, etc.

Information of a general construction includes the location of the shop on the general plan, appointment of the floor level elevation, data on the soils and groundwater level at the construction site, as well as the design strength of the soil to be used, data relating to local building materials, data on illumination, ventilation, heating and a number of other special requirements.

As has been indicated in the previous chapter, any structure being designed must comply with the service requirements, must be strong, stable and have the required space rigity. In every industrial building there should be created normal conditions of work for the employees (the shop should be light, warm or cool, as the case may be and well ventilated), and also normal conditions for operation of the shop (the required rigity of the crane runways, access for cleaning glazed surfaces, good drainage, reliable functioning of the gates, windows and doors, a proper selection of the floors, etc.).

Furthermore, a requirement that must be met in designing every structure is the greatest possible reduction of its weight, with minimum labour consumption for its erection. For this purpose a layout of the structure should be selected in which the force fields induced by the loads acting on the frame of the structure would pass into the ground along the most rational path. It is also essential that the material be utilized in the best possible manner which, in particular, leads to the requirement of a certain concentration thereof.

To ensure greater industrialization and reduce the, labour consumption for fabrication and erection of the members, care should be taken to ensure the greatest possible repetition of elements and their simple shape.

All industrial buildings can be divided into single-storey and multistorey ones. The buildings encountered most frequently are single-storey ones, which are divided in turn into single-span and multispan buildings.

The main elements of the load-carrying steel framework of an industrial building (Fig. 5.1), which resist almost all the loads acting on a shop, are the flat lateral frames formed by the columns and the roof trusses (collar beams). These frames are installed one after the other with a certain distance between them. The lateral frames carry the longitudinal elements of the framework such as the crane girders, the collar beams of the wall framework, the purlins of the roofing, and the skylights. The framework of a building must have space rigity, which is attained by designing braces and ties in longitudinal and lateral directions, as well as by rigid connection of the frame beam to the columns.

When it is necessary to space the columns far apart along the middle rows in multispan shops, the intermediate roof trusses are supported on secondary trusses installed along the longitudinal rows of columns.

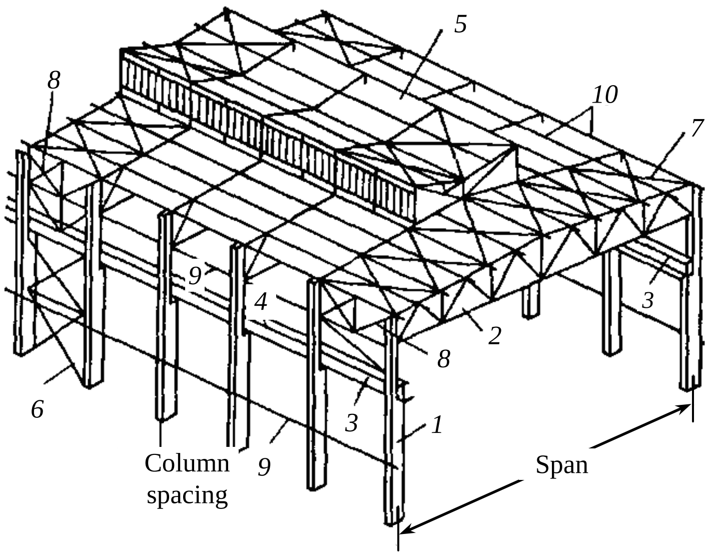


Fig. 5.1. Principal elements of steel framework of industrial building:
1 — column of frame; *2* — roof truss (truss of frame); *3* — crane girders;
4 — bracing beam; *5* — skylight; *6* — vertical ties between columns;
7 — horizontal ties of roof; *8* — vertical ties of roof;
9 — wall framework; *10* — purlins

The roofing, designed for protecting the building against the weather, together with the members supporting it, namely, the roof and secondary trusses, purlins, skylights, etc., is called the *roof*. The main design loads for the members of the roof are the snow load and the own weight of the roof. Members that reinforce the wall or support separate sections of the wall are referred to *as the wall framework.* The main loads for the elements of the wall framework are the weight of the wall in a vertical direction and the wind load in a horizontal one.

The main loads on a lateral frame are the load of the roof and the wall framework, as well as the action of the crane load, consisting, of the vertical pressure of the cranes and the horizontal braking forces acting in transverse and longitudinal directions. For resisting the lateral braking forces acting on the crane girders, horizontal bracing beams are installed, while the longitudinal braking forces are resisted by vertical braces or ties between the columns. An additional load on the frame is the wind load.

The general dimensions of a shop, namely, its span (Fig. 5.1), height to the elevation of the top of the crane runway rail h_1 (Fig. 5.2), as well as the total height of the shop up to the bottom of the frame collar beam H are established depending on the size of the equipment to be accommodated and the nature of the production process to be followed in the shop. In accordance with the general rules for standardization in buildings with traveling cranes, the height of the premises from floor level to the bottom of the load-carrying members of the roof should be a multiple of 1.8 (the width of a precast wall panel) as illustrated in Fig. 5.2. The value of the clearance for the crane above the top of the rails of the crane runway H_c is taken according to the Standard for overhead traveling cranes.

The total height of the shop to the bottom of the collar beam H (Fig. 5.2) which is a multiple of 1.8, is made up of the height from the floor level to the head of the crane runway rails h_1, and the height h_2 including the crane clearance H_c with additional 200–250 mm allowing for possible deflection of the roof trusses and braces along their bottom chords, and also the usual design of these braces with the angle legs outstanding downward.

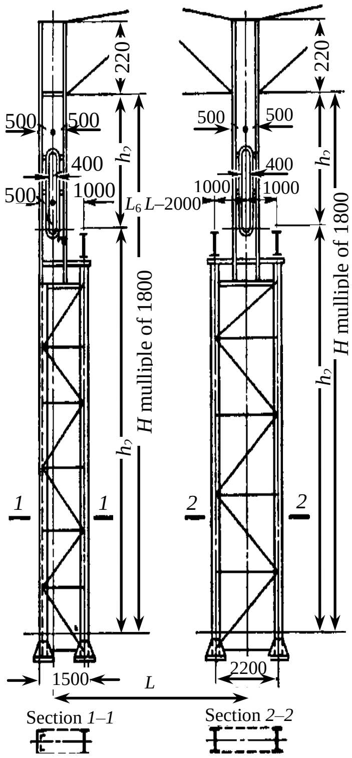


Fig. 5.2. General dimensions in height of shop

The selection of the design and layout of both single-span and multispan shops depends to a considerable extent on a number of factors, the most important of which are:

– the value of the crane loads, as well as the duty of the cranes and the operating conditions in the shop as a whole;

– the height of the shop.

– the roof load (the type of roofing, the magnitude of the snow load).

With a view to the above factors, all shops can be divided into three groups, viz., light, medium capacity and heavy shops.

Of special importance, mainly for iron and steel works, are the conditions of work of the shop. When these are heavy, there appear a number of additional requirements which the design of the structure must meet. Buildings with heavy service conditions include shops with continuous three-shift operation of cranes with very heavy continuous duties; the cranes and crane runways of such shops must be repaired without interrupting the production process.

These shops include, for example, the main buildings of steel foundries (open-hearth, etc.), the bedding plant, the ingot pit building and certain bays of rolling mills, ingot and pig iron stores.

As has already been mentioned, the steel members in shops with heavy service conditions must meet somewhat stricter design requirements.

This especially relates to fastenings and connections, which must have a higher reliability.

In shops with heavy service conditions, openings (400 mm wide) must be provided in the upper part of columns along the line of passages provided at the level of the crane girders (Fig. 5.2). These passages are guarded with railings on the crane side along the entire length of the crane runway.

The most stable index that can be used to judge whether the design of the steel framework of an industrial building is sufficiently ration is the amount of metal required per square meter of building area.

5.2. Column layout

Column Spacing and Span. When designing the steel framework of a shop, it is necessary first of all to plan the so-called column grid, i.e., the layout of the columns, which includes the spans (bays) and the spacing of the columns (the distance between the columns along the shop). To ensure the greatest possible repetition of members, a constant spacing of the columns should be employed which is multiple to a definite predetermined quantity known as *the module*.

The *modular system* provides for integer relations between the principal dimensions of structures and their elements and serves as the basis for unification and standardization of members.

The basic module used in single-storey industrial buildings is three meters. In this connection it is recommended to design spans up to 18 meters long with a length that is a multiple of three meters, and over 18 meters long — a multiple of six meters. It will be good practice to design the distances between column centers in a longitudinal direction (the column spacing) as multiples of six meters. Most frequently the spacing of columns is taken equal to 6 or 12 meters (if the technological conditions do not require a greater spacing). With an increase in the height of the building the optimal spacing grows.

The size of the spans and the spacing of the columns depend primarily on the production process of the shop, and on the required maneuverability of the cranes and other kinds of intra-shop transportation facilities serving this process. At present production processes are being rapidly perfected and often a corresponding reconstruction is required inside an already erected structure. This has led to the tendency of increasing the column spacing and spans, which ensures greater flexibility in meeting the demands of various production processes.

Of special importance in laying out a column grid is its coordination with the production process in the shop being designed, bearing in mind the necessity of retaining the selected span and spacing modules. Attention must be given to the location of the column foundations with respect to the underground facilities (foundations for equipment, furnaces, etc.), and also to the necessity of designing railway entrances that require definite clearances.

Expansion Joints. The total length and width of a shop are determined by the technological conditions in it. If a building has a considerable length or width, the danger appears of significant deformations of its separate elements owing to summer and winter temperature changes. The magnitude of the temperature strain is $\Delta = \alpha Lt$, where α is the coefficient of linear expansion of steel (α = 0.000012), L is the length and t is the temperature difference.

Long shops should be divided into separate sections (compartments or bays) with expansion joints between them. The distance between expansion joints in steel structures, for which the standards and regulations permit no account to be taken of the temperature stresses, should not exceed the values indicated in Table 5.1.

When within the limits of an expansion section of a building two vertical ties or braces are used between columns, the distance between their centers should not exceed 50 meters for buildings and 30 meters for open trestles.

Table 5.1

Maximum Permissible Dimensions of Expansion Sections of Buildings and Structures, m

Category of building or structure	Maximum distance from end of section to centre line of nearest vertical brace	Maximum length of section (along building)	Maximum width of section (building)
Heated buildings	90	230	150
Unheated buildings and hot shops	75	200	120
Open trestles	50	130	–

When precast reinforced concrete columns are used, the expansion joints are spaced not more than 60 meters apart, and with self-bearing brick walls

— 40 to 60 meters apart.

The best way of making the expansion joints in the framework of industrial buildings is to install double lateral frames (on a common foundation), i. e., double columns in each row and correspondingly two roof trusses, etc., in other words to design what may be called two separate building sections. In this case, in accordance with the basic rules for the unification of industrial building members, the centre line of the expansion joint is made to coincide with the marking out line, while the axes of the columns are displaced from the expansion joint centre line by 500 mm (Fig. 5.2). Such a design makes it possible to use walls made from standard precast wall panels. Sometimes, however, with a view to the arrangement of the equipment, the expansion joints may be designed with an additional short section inserted between the double frames. In this case the axes of the columns will coincide with the centre lines of the rows.

The design of expansion joints by employing connections of the member elements which permit movement in a longitudinal direction (for example, with the help of oval holes) has been proved by practice to be insufficiently reliable.

5.3. Lateral Frames

Single-Span Shops. Some examples of single-span shops are forge and smith's, press, heat treatment shops, ingot pit buildings, ingot mould yards, and mixer buildings. Generally these are shops that require a high lateral rigity owing to the necessity of installing crane runways which are rigid in a horizontal direction. Such a requirement will be best met by a lateral design of the shop structure in the form of a frame. The frames can be of three types, viz., two-hinged with the hinges at the corners (Fig. 5.3, *a* and *d*), two-hinged with hinges at the supports (Fig. 5.3, *b*) and fixed (Fig. 5.3, *c* and *e*). The fixed type of frame is the most rigid and economical one, and for this reason it is the basic type of lateral member used for the steel frameworks of single-span industrial buildings. The most widely used type of collar member is a truss, which is lighter and stiffer than a beam or girder.

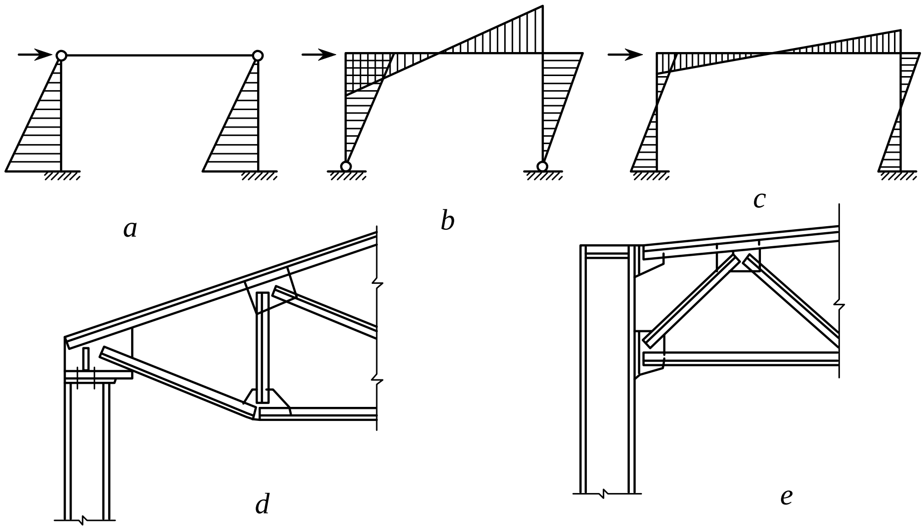


Fig. 5.3. Diagrams of lateral framts

The roof truss, which serves, as has been mentioned previously, as an element of the frame in industrial buildings with steel columns, where a high lateral rigidity of the shop is required, is rigidly connected to the columns. With such a design the truss will have fixed end conditions, which will result in a support moment M, appearing at the supports besides the support reaction (Fig. 5.4, *a*). Upon dividing the support moment M_s by the depth of the truss at the support h_s, we shall obtain the horizontal forces F_h (a pair of forces) which act both on the truss and on the columns (Fig. 5.4, *b*)

$$F_h = \frac{M_s}{h_s}.$$

The support moments are found when investigating the frame by the general methods of structural mechanics. Depending on the direction of the horizontal forces *F*, some of the elements of the truss will be additionally loaded by these forces (in addition to the vertical loads), while others will have their loads reduced.

Usually the forces in the elements are determined for the two combinations of the support moments creating the worst possible conditions for the behavior of the chords (Fig. 5.4) and for that of the diagonals (Fig. 5.4).

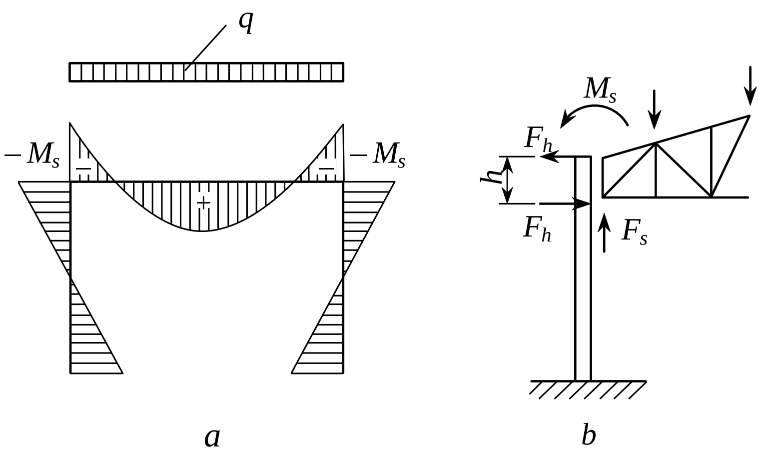


Fig. 5.4. Support moment in frame with collar truss and design combinations of moments in support sections of trusses

Plotting of Cremona's force diagram for the computed horizontal forces F_h, yields additional forces in the elements of the trusses.

The design force in each of the elements of a truss will be the sum of the force induced by the vertical loads found as for a statically determinate truss, and the force created by the support moment. When the support moment leads to relieving of the truss elements, this reduction in the forces is not ordinarily taken into account, since it is considered that the truss may behave as a statically determinate system, without relieving influence of the support moments (i.e., only subjected to vertical loads).

A truss is rigidly connected to a column in the way shown in Fig. 5.4, *a*. Here in the lower support joint the support pressures F_s, and the horizontal force F_h, (appearing as the result of the angular moment of the frame) are transmitted separately. The support gusset plate, made in the form of a welded T section, transmits the support pressure F_s, to the seat (via the milled surfaces) the bearing plate protruding by 10–20 mm to ensure better transmission. The thickness of the plate is taken equal to 16 mm. The seat, made from an angle stub or a thick plate (t = 30 mm) welded to the column, is designed somewhat wider than the bearing plate. Each of the vertical welds used to connect the seat to the column is computed for 2/3 F_s owing to possible unevenness in load transmission. The horizontal forces F_h may originate in the joint compression and tension (tearing away of the joint from the column).

The tension is resisted by unfinished bolts. The force F_h, applied at the level of the truss chord centre line does not pass, as a rule, through the centre of the bolted connection. It is conditionally assumed here that the resulting rotation of the joint takes place about a line passing through the axis of the bolts which are the remotest from the point of application of the force F, (about 40–80 mm below the top of the gusset). Hence the force applied to the most loaded bolt located at the bottom will be

$$F_{\max} = \frac{1}{2} \cdot \frac{F_h z L_1}{\sum L_i^2},$$

where z is the distance from the centre line of the bottom truss chord (line of application of force F_h) to the axis of the remotest bolts (conditional axis of rotation of the joint); L_1 is the distance between extreme bolts; $\sum L_i^2$ is the sum of squares of distances between axes of bolts and axis of rotation of joint in the example shown in we have $\sum L_i^2 = L_1^2 + L_2^2$ 1/2 is the factor indicating the presence of two bolts in each horizontal row of the connection.

The holes for the bolts in the bearing plate are made 3–4 millimeters bigger than the diameter of the bolts. This is done to prevent the unfinished bolts from taking the support reaction in case of slight deviations in the connection of the seat in height. These bolts are designed to withstand only tension.

Ties. In order to ensure space rigidity of a shop framework, as well as to ensure stability of the frame elements, ties or braces are installed between the frames. There are distinguished horizontal ties in the plane of the top and bottom truss chords, and vertical ones both between the trusses and between the columns. The main purpose in installing ties is to:

– ensure the absence of changes in the structure both in service and in the process of erection;

– ensure stability of the compression elements of the structure;

– resist and distribute all the horizontal loads (wind and inertial ones such as, for example, the braking forces of cranes).

The ties and braces ensure stability of the top chords of trusses in a direction perpendicular to their plane, contains an example of the location of ties along the top chords of trusses in a roof with purlins.

In roofs without purlins, where large-size reinforced concrete slabs are welded to the top chords of the trusses, the rigidity of the roof is so great that there seems to be no necessity of installing ties. Taking into account, however, the requirement of ensuring adequate rigidity of the members during installation of the slabs, it is necessary to provide ties along the top chords of the trusses at the edges of the expansion sections. Braces must also be installed at the ridge of the trusses, at the supports and sometimes under the extreme skylight verticals. These braces serve for connecting together the top chords of all the intermediate trusses. The slenderness ratio of the top chord between the points braced during the period of installation of the slabs should not exceed 220. The ties along the top chords of roof trusses are secured to the chords with unfinished bolts.

The horizontal ties along the bottom chords of the trusses are arranged both across the shop (lateral ties) and along the shop (longitudinal ties).

The lateral ties located at the ends of the shop are used in the capacity of wind trusses. They carry the struts of the end wall framework that take the pressure of the wind. The bottom chords of the roof trusses serve as the chords of the wind truss. The same kind of lateral ties along the bottom chords of the trusses are installed at the expansion joints (in order to form a rigid roof). With a large length of an expansion section the lateral ties are also installed at the middle part of the section to ensure that the distance between lateral ties does not exceed 50–60 meters. This is necessary because the ties are often connected to the trusses by means of unfinished bolts that permit considerable displacements, and therefore the influence of the ties does not spread over large distances.

With 12-metre truss spacing, the horizontal tie trusses are made with a width of 6 meters, arranging them on braces connected at the joints of the roof trusses. The horizontal longitudinal ties along the bottom chords of the trusses are chiefly designed to make the adjacent frames participate in space load resistance under the action of local loads, such as crane loads. This reduces the deformations of the frame and improves the lateral rigidity of the shop. Longitudinal ties are of special importance in shops with heavy service conditions, as well as with light and nonrigid roofing (of corrugated steel, asbestos-cement sheets, etc.).

In the absence of such ties in rigid roofing the horizontal crane loads are distributed by the large-size reinforced concrete slabs, which may lead to disarrangement of the roofing, and sometimes of the slabs themselves, since the latter are not designed to resist shearing forces. Besides, in this instance the welded connection of the slabs to the trusses may fail.

With two or more spans there is no necessity of installing longitudinal ties on the bottom chords of the trusses along the middle rows of columns in both spans. The first panel of the bottom chord, how ever, must be reinforced with braces to permit the chord to serve as; the support of the second corn pression diagonal of the trusses perpendicular to its plane. In buildings with heavy service conditions, the ties should be welded to the bottom chord (Fig. 5.5); unfinished bolts will be sufficient in all instances.

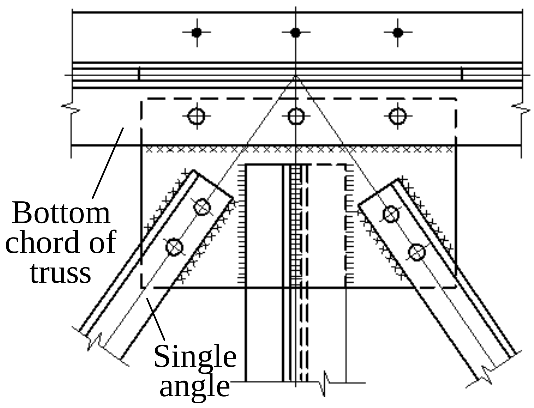


Fig. 5.5. Connection of ties to bottom cord of truss

When bracing trusses the rule is to use a cross lattice, it being assumed that loads acting on any side will be resisted only by the system of tension diagonals, while the other part of the diagonals: (the compression ones) will be idle. Such an assumption is true if the diagonals are slender (λ >200). For this reason the elements of cross ties, as a rule, are designed of single angles. When checking the slenderness ratio of cross lattice tension diagonals of ties made of single angles, the radius of gyration of the angle is taken with respect to the axis parallel to the leg. With a triangular lattice of the tie trusses, compressive stresses may appear in all the diagonals, and therefore they must be designed with a slenderness ratio of $\lambda \leq 200$, which is less economical.

In spans over 24 meters long, owing to limitation of the lateral slenderness ratio of the bottom truss chords, it often becomes necessary to install additional braces in the middle of the span. This eliminates vibration of the trusses during operation of the cranes.

The vertical ties between trusses are generally installed at the truss supports (between the columns) and at the middle of the span (or under the verticals of the skylight), locating them along the length of the shop in the rigid panels, i.e., where the horizontal lateral ties are installed along the chords of the trusses.

The principal purpose of installing vertical ties is to ensure a rigid unchangeable state of a space member consisting of two roof trusses and lateral ties along the top and bottom chords of the trusses. The vertical ties are designed in the form of a cross of single angles, always with a horizontal closing element, or in the form of a small truss with a triangular lattice. Unfinished bolts are used to connect the vertical ties to the roof trusses.

Vertical ties are installed along a shop between columns to ensure stability of the shop in a longitudinal direction, and also to resist the longitudinal braking forces and the pressure of the wind at the end of the building. In a lateral direction frames constrained in their foundations are an unchangeable system, but in a longitudinal direction a number of frames pin-connected by crane girders represent a changeable system, which in the absence of vertical ties between the columns may collapse (the supports of the columns should be considered as pinned ones in a longitudinal direction). For this reason the elements of the ties between columns (below the crane girders), which are quite important for the stability of the structure as a whole, are designed with adequate rigity to avoid their vibration. This is achieved by limiting the maximum slenderness ratio of such elements to $\lambda = 150$ for compression elements and to $\lambda = 300$ and 200 (the latter in buildings with heavy service conditions) for tension ones.

For other tension elements of the ties between columns, the slenderness ratio should not exceed $\lambda = 400$, and for compression elements $\lambda = 200$. The elements of cross ties between columns are generally made of angles. Especially heavy cross ties are made of pairs of channels connected by a lattice or batten plates.

Multispan Shops. The selection of the cross-section of multispan shops depends not only on the required clear dimensions of the shop and the size of the overhead cranes, but also on a number of general construction requirements, first of all on the method to be used for draining water from the roof and on the system of illumination of the middle bays. The water drainage system can be either an external or an internal one. External water drains are installed in narrow shops, as well as in unheated hot shops with cold roofs. The maximum width of a building with a ridge roof and with external water drainage is 60 meters for heated buildings and 100 meters for unheated ones.

In wide multispan buildings the drains are of the internal type (the water is drained on through pipes into the internal sewerage system).

Fig. 5.6 presents an example of a multispan building with internal water drainage and with skylights used for lighting.

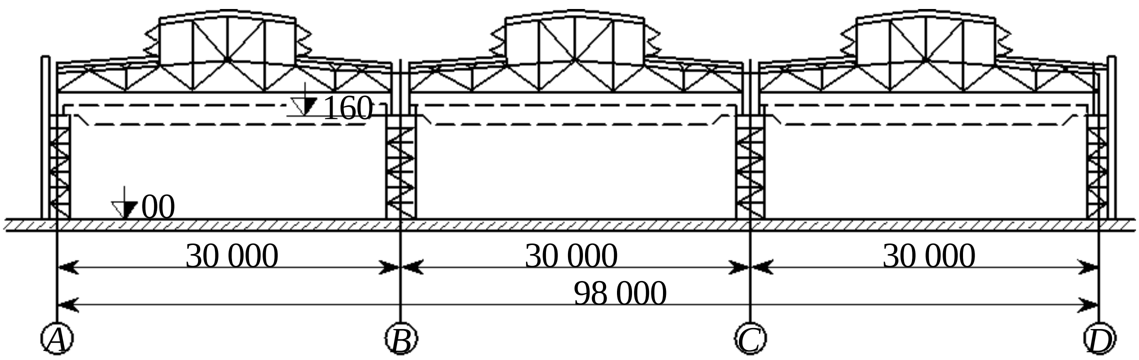


Fig. 5.6. Multispan shop

In multispan shops the simplest possible cross-section (without any changes in height) should be employed, and the maximum repetition of elements should be ensured. It is permitted, when necessary to design differences in height of at least 2 meters.

Heavy two-span shops, unlike light ones, require a high lateral rigidity, which is attained by designing stiff frame systems.

The various layouts of the lateral members of such shops can be divided into two main groups. Layouts of the first group consist of a number of parallel frames supporting the longitudinal members (Fig. 5.7, *a*). In two-span shops it is not always possible to arrange all the columns in one cross section of the shop, and therefore the lateral frames may have the form of inverted U- and L-shaped frames with intermediate supporting of the roof trusses on secondary ones (Fig. 5.7, *b*).

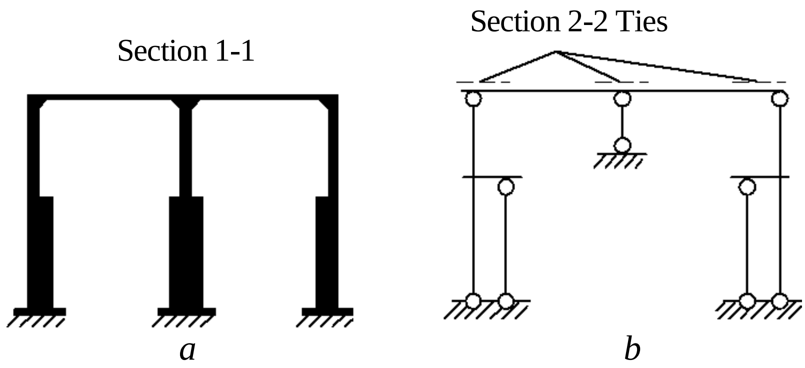


Fig. 5.7. Layouts of lateral members of heavy two-span shops

The layout under consideration can be solved in the form of separate plane frames functioning independently, i.e., resisting all the loads originating only on their own tributary area. Hence those columns whose number is less in a cross-section will have to be considerably heavier than the columns of the main frames. Such a design (in the form of separate plane frames) is not rational. With a different solution of this layout in the form of a group of frames the space behavior of the framework is taken into account. With the provision of horizontal ties and other longitudinal elements that can play a reliable part in the resistance of the framework to a local lateral load, the frames can be considered not as separate ones, but as the elements of a space system including both main and intermediate frames. In this case the displacements of the frames at the level of longitudinal members connecting them will not be independent of each other, but will be bound by definite relationships depending on the rigity of the longitudinal members.

Upon the provision between the bottom chords of wide horizontal ties that are reliably connected to the trusses, it can be assumed that these ties and the roof form a very rigid construction, so that the displacements of the main frame and the adjacent ones can be considered to be the same.

This determines the part played by each frame in the general functioning of the framework and, correspondingly, the section of the columns. Thus, when a-system of frames functions simultaneously, the load is distributed between the columns in proportion to their rigity, and they can be designed with a lighter weight. Such a solution (in the form of a system or group of frames) is more rational.

In the second group of layouts the members of the framework are divided with a view to their functional designation. The crane runway column branches, as well as the branches supporting the roof trusses, are designed as separate members and, all of them being Pin-connected, are loaded with an axial vertical load. To ensure rigidity of the system when subjected to displacements in a horizontal plane, as well as for resisting horizontal loads, use is made of longitudinal horizontal ties and wide crane runway bracing beams. These transmit all the horizontal forces to the main frames located at a considerable distance from each other and, therefore of a heavier design. Such a functional division of the members makes it possible to attain a high degree of element repetition and, consequently, a reduction in the number of different standard shapes required. With heavy cranes, layouts of the second group result in a somewhat lower consumption of steel than those of the first one.

In multispan shops horizontal rigidity is mainly ensured by the several rows of columns, and for this reason pinned connection of the collar trusses to the columns can be tolerated. This permits designers to make wide use of standard roof trusses.

The lateral frames of multispan heavy shops can sometimes be designed with fixed frame joints only along the extreme rows of columns, with pinned connections along the middle rows.

5.4. Special features of lateral frame analysis

Design Loads. The lateral frames of industrial buildings are investigated for the following loads:

– The dead load of the roofing and roof members.

– The snow load.

– The dead load of the walls (when supported on the framework).

– The vertical and horizontal crane loads.

– The wind load on the walls and skylights of the building.

The first four kinds of loads are included in the main combination of loads when cranes with heavy duties are to be installed. With light or medium duty cranes the main combination should include either the crane or the snow load.

The wind load forms part of the additional load combination.

The dead load of the roofing and roof members, together with the snow load, is transmitted to the column in the form of the support load of the truss P_T (Fig. 5.8), which is applied to the upper part of the column eccentrically. Besides, the upper part of the column also supports eccentrically part of the wall. The load originated by the latter is transmitted through the elements of the wall framework in the form of separate concentrated forces $P_{C.}$

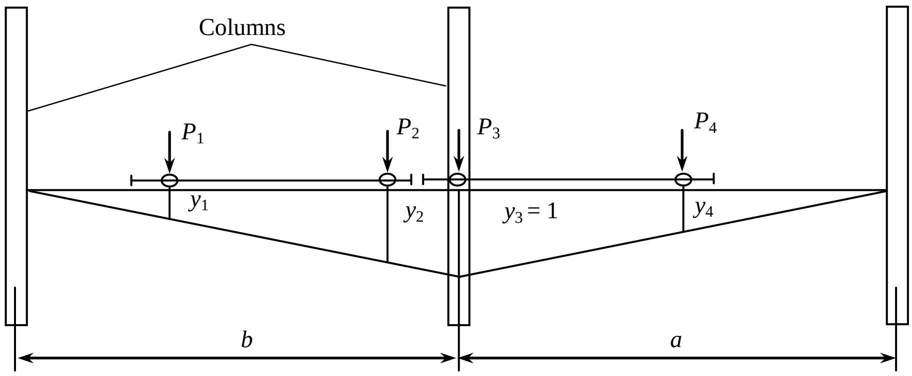


Fig. 5.8. Loads on lateral frame and influence line of crane load on columns

The design vertical load of cranes on one column is determined by adding to the corresponding load influence line the load caused by the crane wheels P, which is taken in accordance with the standards for cranes (Fig. 5.8). As previously indicated, the investigations are generally carried out for two cranes, and the maximum and minimum pressure on each column are determined

$$F_{\max} = \gamma_f \sum P_{\max} y, \quad F_{\min} = \gamma_f \sum P_{\min} y,$$

where λ_f =1.2 is the load factor for crane loads; y is the ordinates of influence line (Fig. 5.8); $P_{\max}$ is the maximum load on crane wheel (according to standards); $P_{\min}$ is the minimum load on crane wheel determined from the equation:

$$P_{\min} = \frac{Q+G}{0.5Ncw} - P_{\max},$$

where, Q is the lifting capacity of crane; G is the total weight of crane with truck; N_{cw} is the total number of crane wheels.

To determine how lateral braking of the cranes acts on the frame, t is first necessary to find the force applied to each wheel of the crane

$$F_{br \cdot cw} = \frac{Q+g}{20N_1},$$

where, g is the weight of a crane trolley, and N_1 is the number of wheels of each crane on one side.

The total action of the braking forces on the frame is computed in the same way as for the vertical load, and using the same line of influence

$$F_{br} = \gamma_f \sum F_{br \cdot cw} y.$$

The lateral braking force F_{br} is transmitted to only one of the frame columns and may be directed to any side. The longitudinal braking force transmitted to the vertical ties between the columns is determined from the equation

$$F_{br \cdot l} = 0.1 \gamma_f P_{\max} N_{br}$$

in which N_{br} is the number of braking wheels on one girder (equal to half the total number of wheels on the girder).

The wind load is taken as prescribed by the Building Standards and Regulations, assuming that it acts as a uniformly distributed load

$$q_w = \gamma_f q_r C_a b,$$

where $\gamma = 1.2$ = load factor for the wind load; q_r = rated velocity head; C_a = aerodynamical coefficient; b = spacing of frames.

The wind load acting on the skylight and on the part of the wall within the limits of the collar beam depth is transmitted to the frame in the form of a concentrated force P_w, applied at the level of the bottom chord of the collar beam. This force is $P_w = \gamma_f q_r C_a bh.$ (Fig. 5.8). For estimating the dead load or own weight of the steel members use can be made of the approximate data contained in table 5.2 which characterize the distribution of the metal between the separate elements of steel industrial building frameworks.

Special Features of Statical Analysis of Frames. The lateral frames of industrial buildings are investigated as plane statically indeterminate systems. In these investigations, a number of assumptions are generally introduced that simplify them without greatly influencing the final results. The main simplifications consist in the following:

Table 5.2

Approximate Weights of Elements of Industrial Building Steel Frameworks in kg per sq in of Building Area

Elements of steel framework	Group of shops		
	Light	Medium	Heavy
Roof:			
roof trusses	16–25	18–30	20–40
secondary trusses	0–6	4–7	8–20
purlins	10–12	12–18	12–16
skylights	0–10	8–12	8–12
ties	3–4	3–5	8–15
Total:			
Columns with ties and platforms	30–40	45–70	50–80
Crane girders with bracing	10–18	18–40	70–120
beams and repair platforms	0–14	14–40	50–150
Wall framework	0–3	5–14	12–20
Miscellaneous	–	0–10	3–12
Grand total	35–80	75–170	200–400

1. When investigating a frame with a collar truss for all the loads applied to the columns of the frame, the truss is assumed to have an infinitely high rigity ($I_{sb} = \infty$). This assumption permits the frame to be investigated according to the deflection method, only the horizontal displacements being unknown. By using specially prepared tables compiled for variable dimension columns constrained in the foundation and in the collar beam (at the level of the bottom chord), it is easy to investigate the frames of single-storey shops. Such tables are contained in various reference books and manuals.

2. Frames are analyzed for the vertical loads applied to the collar beam in accordance with the general rules of structural mechanics. Account is taken of the final rigity of the collar beam, but on the assumption of symmetrically arranged loads in symmetrical frames. This reduces the problem to the analysis of a frame with immovable joints (owing to symmetry) having one unknown-the angle of rotation of the collar beam (for a single-span frame).

In this instance a collar truss is conditionally replaced with an equivalent beam (with respect to the deflection or the angle of rotation on the supports), making the axis of the latter coincides with the bottom chord of the truss. The moment of inertia of an equivalent collar beam (with respect to the deflection) is approximately determined from the equation

$$I_{cb} = k_h \;\; A_l z_t^2 + A_b z_b^2 \;\; , \qquad (5.1)$$

where A_t and A_b = sectional areas of top and bottom truss chords; z_t and z_b = distances from the gravity centre (axis) of truss to the axes of top and bottom chords in the middle of the span, k_h = factor allowing for the variable depth of the truss, as well as for deformation of the web elements. For trusses with a pitch of the top chord of'1/8 to 1/12 this factor can be taken equal to 0.7–0.8.

The ratios between the moments of inertia of the separate parts of the columns, collar beam or truss, required for analysis purposes are taken on the basis of similar designs already completed. Generally these ratios are within the following limits (Fig. 5.9) $\frac{I_1}{I_2} = 5...12$; $\frac{I_3}{I_4} = 8...15$; $\frac{I_3}{I_1} = 1.2...4$; $\frac{I_3}{I_1} = 4...12$ (with double spacing between columns along the middle rows).

The smaller values given above relate to light shops, and the higher ones-to heavy shops. A change in the ratio between the moments of inertia within certain limits (about 30 %) only slightly affects the value of the design moments.

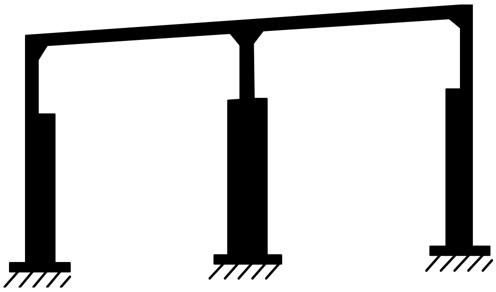


Fig. 5.9. Relation between inertia moments of frame elements

The introduction of these simplifications makes the analysis of frame much easier (5.1). For example, a single-span frame with rigid corners can be investigated for the loads applied to the columns in the following order:

1. One additional (extra) constraint is applied that prevents horizontal displacement; as a result the load will cause a reaction to appear in the constraint (basic system, Fig. 5.10, *a*).

2. The collar beam of the unloaded frame is given a displacement in the direction of this constraint equal to unity, i. e., Δ = l (Fig. 5.10, *b*). The total reaction in the constraint $\sum r_{11}$ caused by the displacement of all the columns over a distance of Δ = l, as well as the magnitudes of the moments at the ends of the columns (M_A, M_B, M_C etc.) are obtained from the appropriate tables. The reaction r_{11} *is* always in the direction of the displacement.

3. The reactions in the additional constraint induced by the external loads are determined for each load separately, since it is necessary to have total resultant moment diagrams for the columns. Thus, for example, when a crane load is applied to the frame there are simultaneously induced a maximum moment on one column and a minimum one on another column (Fig. 5.10, *c*). The total reaction in the redundant constraint will be

$\sum r_{1e} = r_{1e} - r_{1e}^{''}$. In a similar way, with the help of tables, moment diagrams are computed and plotted for each load in the fixed columns (*Ma,e*; *Mc,e*; M_{Be} etc.).

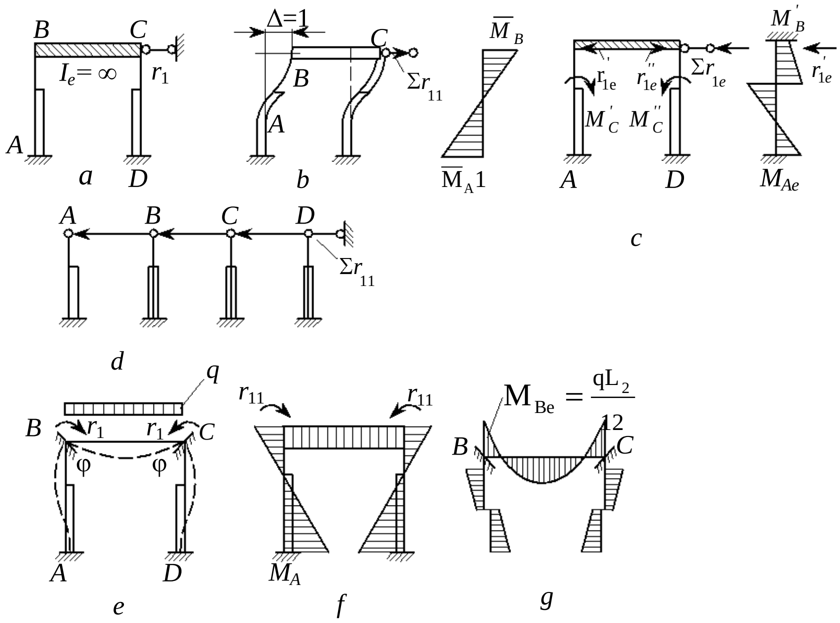


Fig. 5.10. Analysis of frames with equal spacing of columns

The actual displacement Δ_1 caused by each load in the given frame is found from the basic equation expressing the condition that the reaction in the additional constraint induced by each load is equal to zero

$$r_1 = \sum r_{11}\Delta_1 + \sum r_{1e} = 0, \text{ whence } \Delta_1 = -\frac{\sum r_{1e}}{\sum r_{11}}.$$

5. The final values of the moments in the relevant sections of the columns are found and the moment diagrams are plotted

$$M_A = M_{A \cdot e} + M_A \Delta_1; M_B = M_{B \cdot e} + M_B \Delta_1, \text{ etc.}$$

The values of the moments are entered in a table. In multispan frames the total reaction caused by a unit displacement will be (Fig. 5.10, *d*).

$$\sum r_{11} = r_{1B} + r_{1C} + r_{1D} + r_{1E}.$$

Should no tables be available for determining the reactions and the moments in constrained columns, they can be found by employing the general method of structural mechanics-the method of redundant reactions. When the number of spans exceeds three, the horizontal displacement may be disregarded, assuming that Δ_1 = 0.

A frame is investigated for vertical loads applied to the collar beam, taking into account the final rigity of the latter, in a similar way.

Owing to symmetry, we have only one unknown when the joints are restrained (Fig. 5.10, *e*), the angle of rotation γ_1 $\gamma_B = \gamma_C = \gamma$. Upon rotation of a joint through an angle of γ_1 = l the following reactive moments appear (Fig. 5.10, *f*)

$$\sum r_{11} = M_{B \cdot col} + M_{B \cdot cb} = M_{11}$$

in which the subscripts *col* and *cb* denote column and collar beam, respectively. When determining the reactive moments in the joints of a frame induced by an external load $\left(M'_{B \cdot e} = M_{C \cdot e}{}' = \frac{qL^2}{12} \right)$ account should also be taken of the moment resulting from the presence (Fig. 5.11) of a step in the axis of the column $M_{B \cdot e}$

$$\sum r_{1e} = \frac{qL^2}{12} + M''_{B \cdot e} = M_{1e} .$$

The actual angle of joint rotation is found from the equation

$$\sum r_{11}\gamma_1 + \sum r_{1e} = 0 , \text{ whence } \gamma_1 = -\frac{\sum r_{1e}}{\sum r_{11}} = \frac{M_{1e}}{M_{11}} .$$

As a result the final values of the moments are obtained, which are entered in the table. In frames with pinned beam-to-column connections the unknown force can be found quite easily without the use of tables, solving the problem by means of the method of redundant reactions.

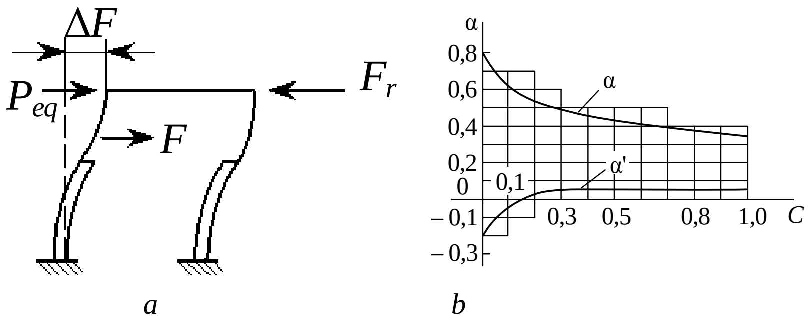


Fig. 5.11. Three-dimensional behavior of framework

When investigating plane frames for crane loads, the solution can be based on space behavior of the framework, taking into account that the adjacent frames will be involved in load resistance via the horizontal longitudinal ties along the bottom chords of the trusses. In this case, with equal column spacing along all the rows, a group of five to seven frames connected to each other by means of longitudinal ties is considered. The rigity of these ties is determined by their moment of inertia found from equation (5.2), it being assumed here that for ties welded to the bottom chord the factor k_h = 0.7, while when bolted tie connections are used k_h = 0.3.

Analysis consists in determining the value of the elastic reaction F_r considering the ties as a continuous beam on elastic supports. Hence the basic equation for determining the unknown displacement will be

$$\sum r_{11}\Delta_1 + \sum r_{1e} - F_r = 0\,.$$

For convenience of calculations it will be feasible to substitute for the displacement of the frames in a group induced by the applied loads a displacement caused by the equivalent load P applied at the level of the ties, and causing the same displacement of the frame as the initial loads, for example (Fig. 5.11, a)

$$P_{eq} = \frac{\Delta_j}{\delta}, \tag{5.2}$$

where Δ_j = displacement of collar beam caused by crane load, for example by force F; δ = unit displacement of collar beam (kN/cm) caused by unit force.

The reactive force F_r can be found from the equation

$$F_r = \alpha P_{eq} - \alpha' P'_{eq},$$

in which the coefficients α and α' establish the relation between the equivalent force P_{eq} and the elastic reaction, the latter depending on the height of the frame H, the ratio between the linear rigity of the collar beam and the columns, the spacing of the frames b and the moment of inertia of the horizontal ties $I_{h.t}$.

The coefficients α and α' are found from the chart contained in Fig. 5.11, *b* depending upon the quantity c'

$$c' = \frac{b^3 \sum I_{bot} i^{1/2}}{H^3 \sum I_{h \cdot t}},$$

In the above expression $i = \frac{I_{top}}{I_{bot}}$ is the ratio between the moments of inertia of the top part of the columns and of the bottom one.

The equivalent force P'_{eq} represents the influence of the displacement of two frames adjacent to the one under consideration and is determined from the force P_{eq} [see equation (5.1)]. The latter is multiplied by the ratio between the loads applied to the adjacent frames and to the one under consideration.

In buildings with a roof consisting of large-size reinforced concrete slabs without purlins the roof structure also leads to space behavior of the framework similar to the longitudinal ties. However, transferring of the distributing functions only to the roofing, as has been noted above, may lead to disarrangement of the slabs.

After diagrams of the moments in a frame have been plotted for each load separately, a table of the values of these moments for a number of column sections is compiled and the most disadvantageous combination of the total moment M and the longitudinal force F corresponding to it is established. Here account is taken both of the main and the additional combination of loads (including the wind load), for which a load combination factor of 0.9 is introduced (for all loads except the dead ones).

Testing of the actual behavior of steel frameworks of industrial buildings confirmed the appreciable influence of the space behavior of members. At the same time these tests proved that the weakest place in these frameworks are the connections of the elements of members and their components, especially the places where crane girders are connected to columns, which are the first to be subjected to the dynamic load of cranes (impacts). This explains why the building standards and regulations contain higher demands to the members of buildings and structures with heavy service conditions with respect to both the connections and the lateral rigity measured by horizontal deflections of the columns.

Methodical instructions to Chapter 5

A steel frame of anindustrial building is the main load carrying member supporting the roof and the walls. That is why the studying of steel frames component parts is of great importance.

Special attention should be paid to the ways of connecting steel frames component parts and arrangement of expantion joints as well as special features of steel frames analysis.

Students should study the procedure of design loads collection acting on the steel skeleton of an industrial building, features of statical analysis of frames.

Questions and Tasks to Chapter 5

1. *What is a steel skeleton of an industrial building?*
2. *Name the principal elements of a steel framework of an industrial building.*
3. *What factors are influencing the design and layout of both single-span and multispan shops?*
4. *What are expansion joints and conditions of their arrangement?*
5. *Speak on classification of ties used in industrial construction and ways of their arrangement.*
6. *What are types of design of loads lateral frames of an industrial building investigated for?*
7. *Describe specific features of statical analysis of frames.*

6. BEAMS OF EFFECTIVE FORMS

6.1. Crane girders with slender wall

Design and loads of crane girders. Crane beams and girders, which for convenience will all be termed herein “crane girders”, are members used as runways for overhead cranes (also called bridge cranes, traveling cranes, shop travelers, etc.) serving shops and other premises of industrial buildings. The crane girders are usually supported on columns. The cranes run along rails laid on the top flange of the girder.

Crane girders may be either solid (simple or continuous) or, less frequently, lattice ones.

The main features characterizing the behavior of crane girders are:

− withstanding of vertical live load of the crane, which has a dynamic action on the girder;

− the action of comparatively large concentrated loads resulting from the crane wheels and transmitted through the flange connections (welds or rivets) to the web of the girder, causing its crushing;

− presence of lateral braking forces that induce bending of the top beam flange in a horizontal plane.

An overhead crane consists of two main girders (trusses), along which the crane trolley with its load runs. The load being handled, as well as the weight of the crane and the trolley, are transmitted to the crane girders through the crane wheels.

Depending upon the location of the trolley, the crane wheel load may have a maximum or minimum value. The maximum service (working) loads of crane wheels, as well as the arrangement of the wheels with respect to each other, are specified in the standards relating to cranes. To obtain the design loads, the service ones should be multiplied by a reliability factor, which is taken equal to γ_f = 1.2 for cranes (except for cranes with a lifting capacity of less than 5 tons, for which a reliability factor of γ_f = 1.3 is taken). Besides, in view of the possibility of sharp changes in the speed of hoisting the load, unevenness of the crane runway and other reasons, the crane load is multiplied by the dynamic factor equal to 1.1. Thus the design crane wheel load will be

$$P = 1.1 \times 1.2 P_{\max} .$$

As regards the nature of their service conditions, cranes with light, medium, heavy, very heavy and very heavy continuous duties are distinguished. Cranes of light mode include cranes that are used seldom, for instance in the erection of equipment. Cranes of medium mode are, for example, the cranes of machine and assembly shops with a duty cycle factor ranging from 15 to 40 %. Heavy, very heavy and very heavy continuous mode cranes are characterized by a duty cycle factor varying from 40 to 60 and even to 80 %, with a high utilization factor both as regards lifting capacity and time.

Owing to braking of the trolley, lateral horizontal braking forces appear. A_s a result, a lateral braking force is transmitted to the crane bridge. It is determined from the equation

$$F_{br \cdot o} = \frac{1}{10}(Q+g)\frac{2}{4} = \frac{Q+g}{20} ,$$

where, Q is the lifting capacity of crane, tons; g is the weight of trolley, tons, taken in accordance with the standards for cranes; if no data are a vailable it may be assumed that g = 0.3Q; $\frac{1}{10}$ is the coefficient of friction; $\frac{2}{4}$ is the fraction whose numerator indicates the number of brake wheels and denominator—the total number of trolley wheels (since the force of friction appears only under those trolley wheels which are outfitted with brakes). The braking force $F_{br\text{-}o}$ is transmitted to one crane girder and is distributed uniformly between the wheels of the crane. The design braking forces, the same as the vertical loads of the wheels, are obtained by multiplying them by the same load factors. In buildings with heavy service conditions, the horizontal braking forces must be increased, in accordance with the Building Standards and Regulations, by multiplying them by the factor k_{br} (table 6.1) that allows for possible misalignments, impacts and other lateral forces, which are transmitted to the top flange of the girder.

It is also recommended to investigate the connections of the bracing members to the crane girders and columns in heavy and medium-duty cranes for lateral forces. These forces are computed by multiplying the horizontal braking forces by the factor $k_{br}/2$. When designing crane girders for cranes of a heavy, very heavy and very heavy continuous mode, a service condition factor of γ_s = 0.9 should be taken into account.

Solid Crane Girders. Types of Sections. Crane girders having a span of up to six meters, with a low lifting capacity of the cranes (up to 3–5 tons inclusively) are generally designed of a rolled I beam strengthened with a plate or angles (Fig. 6.1, *a* and *b*). Girders with a span of six meters in 5 to 30 ton light and medium duty cranes are, as a rule, of a built-up design with a solid unsymmetrical section (Fig. 6.1, *c*) having a developed top flange for resisting the lateral braking forces. In the remaining cases special horizontal bracing members-beams or trusses, are provided to withstand the lateral braking forces (Fig. 6.1, *d*).

Crane girders to be used outdoors or in unheated premises at a low temperature, as well as girders designed for heavy service conditions under heavy-duty cranes, must be made of killed steel of grade Bct.3 nc or of low-alloy steel.

Table 6.1

Factors k_{br} computing lateral forces caused by crane girders

Type of crane	Values of factor k_{br} for investigation	
	top flange of crane girders and bracing	connections of bracing members to crane girders and columns
Cranes with flexible hoist suspension (on ropes) with a lifting capacity, tons, of:		
5–10...	2.5	5
15–20..	2	4
30–150...	1.5	3
175–275..	1.3	2.6
300–350...	1.1	2.2
Cranes with rigid hoist suspension	1.5	3

Grade BCt.3 nc semi killed steel may be employed for crane girders used by medium or light-duty cranes. Welded I girders should be built up of three plates, and the tendency should be to secure the most economical distribution of the material between the flanges and the web of the girder (a material distribution factor of k_d = 0.5 with the optimal depth of the girder. The ratio between the thicknesses of the flange and the web (t_{fl}/t_w,) should not exceed 2.5 to 3. Besides, it will be good practice to limit the thickness of the flanges to ensure complete utilization of the design strength 40 mm for steel 3 and 32 mm for certain grades of low-alloy steel. As has been previously indicated, the width of the flange should be within the limit $b \le 30 t_{fl} \sqrt{21/r}$.

In large-span girders under heavy cranes, however, it will not be good to employ flange plates whose width exceeds 0.8–1 meter. In these cases the flanges may be designed in the form of a group of two horizontal plates symmetrically arranged with respect to the web.

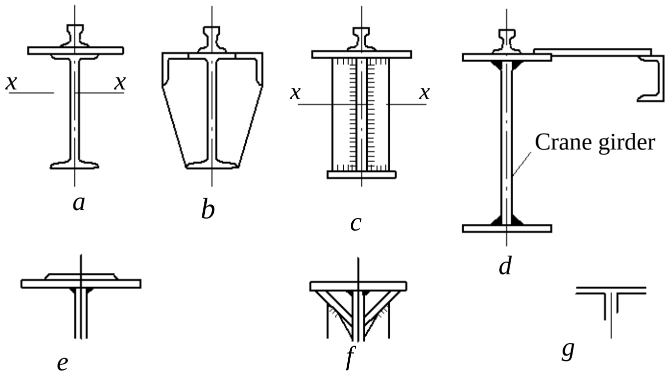


Fig. 6.1. Types of solid crane girders

The difference in the width of these plates must not be less than 40 mm (Fig. 6.1, *e*). It is also possible to develop the section of the top flange (in light or medium-mode cranes) by welding on two inclined plates arranged at an angle of 45 deg (Fig. 6.1, *f*).

Riveted girders are employed in shops with heavy service conditions, where they are subjected to heavy dynamic loads.

The flanges of such girders are ordinarily made from angles and horizontal plates (Fig. 6.1, *g*).

The sections of crane girders for heavy cranes (with a lifting capacity of 150 tons and more) are also sometimes designed of the riveted type, because of the difficulties encountered in welding heavy flange sections. The depth of such girders sometimes reaches 4–5 meters, and for this reason the vertical web is made up in depth of two plates connected together by means of a butt splice, with one or two rows of rivets (Fig. 6.1, *h*). The flanges are developed not only by means of horizontal plates, but also by using so-called lamellae secured to the vertical web.

It is desirable to use not more than three horizontal plates in the flanges of riveted girders, the area of the angles, as has already been mentioned, being taken not less than 30 % of the entire area of a flange.

Features of Solid Crane Girder Analysis. Determination of Design Forces. Solid crane girders are investigated in the same way as solid beams or girders carrying a statical load, but with regard to a number of special features.

The design moments and shear forces caused by the crane load may be computed either by using the influence lines plotted for two cranes installed on the runway, or by plotting a diagram of moments and shear forces for the most disadvantageous arrangement of the loads. For finding the maximum bending moment, the loads should be so arranged that the middle of the girder will be at equal distances from the resultant of all the loads and from the nearest load. Under the latter the maximum moment will be observed. To determine the maximum shear force (support reaction) it is necessary to place one of the loads above the support and the remaining ones to it as near as possible. The location of the cranes for determining the stresses induced by the vertical and horizontal forces should be identical.

The influence of the own weight of the crane girder and the live load on the bracing beam is allowed for by increasing the values of the bending moments and the shear forces caused by the cranes (M_c and Q_c). For this purpose they should be multiplied by the factors k_M and k_Q contained in table 6.2.

Table 6.2

Values of Factors K_M and k_Q

Factor	Span of girder in meters		
	6	12	18 and more
K_M (for M_c) k_Q (for Q)	1.03 1.02	1.05 1.04	1.08 1.07

Selection of Sections. The sections of a solid crane girder are selected in the same way as those of built-up beams designed for a statical load, with attention paid to the following features.

Owing to the presence of a moving concentrated load that acts on the web through the top flange at spots not reinforced with stiffeners, the web is subjected to a local load. The stress in the web σ_{toc} caused by this local load, when investigating for strength, should not exceed the design strength

$$\sigma_{toc} = \frac{n_1 P_1}{tz} \leq R_y \gamma_c . \tag{6.1}$$

Here P_1 is the of design concentrated load (without account of the dynamic factor 1.1, but with regard to the reliability factor); n_1 is the factor, taken for crane girders in buildings and structures with heavy service conditions equal to 1.5 for cranes with a rigid hoist suspension, 1.3 for cranes with a flexible suspension and 1.1 for other crane girders; t is the thickness of web; z is the conditional length of distribution of pressure induced by the concentrated load, determined from the equation

$$z = c_1 \sqrt[3]{\frac{I_{fl}}{t}} , \tag{6.2}$$

where c_1 is the f actor taken equal to 3.25 for welded and rolled girders and 3.75 for riveted ones; I_{fl} is the sum of moments of inertia of girder flange and crane rail with respect to their own axes; when the rail is welded to the flange by means of welds that ensure joint behavior of the rail and the girder, I_{fl} is the total moment of inertia of the flange and the rail.

The stress induced by the local load σ_{toc} must be taken into consideration when establishing the thickness of the crane girder web, as also the condition of ensuring adequate resistance of the web to shear.

From expressions (6.1) and (6.2), and assuming in the first approximation that I_{ft} *is* equal to the moment of inertia of only the crane rail (in cm^4), we shall obtain an equation for the minimum thickness of a crane girder web in which $P_1 = P'n$ is the design load of a crane wheel on the girder in tons, equal to the product of the service wheel load and the load factor n = 1.2.

$$t_{\min} = \frac{n_1 P_1}{3.25R}\sqrt{\frac{n_1 P_1}{3.25RI_{fl}}}\ \text{cm}. \tag{6.3}$$

The required section modulus (W_{gr} for welded girders and W_n for riveted ones) is determined on the basis of the design strength reduced by 1.5–2.5 kN/cm^2. This is done because in the top flange of the girder, which is simultaneously subjected to horizontal braking forces, additional stresses induced by these forces appear.

Having planned the cross-sectional dimensions of the crane girders and the bracing beams, the geometrical characteristics of the section (the moments of inertia, the section module, and so on) are determined with respect to the horizontal and the vertical axes, which is followed by investigation of the beam strength.

The strength of a solid crane girder is checked (when a solid bracing beam will be used) by means of the following expressions: for the top fiber of the girder

$$\sigma = \frac{M}{W_{n\cdot t}} + \frac{M_{br}}{W_{br}} \le R_y \gamma_c, \tag{6.4}$$

for the bottom fiber of the girder

$$\sigma = \frac{M}{W_{n\cdot b}} \le R_y \gamma_c. \tag{6.5}$$

Here M is the design moment induced by vertical crane load; M_{br} is the design moment caused by horizontal lateral braking forces; W_{nt} is the net section modulus for top fiber of girder (in welded girders; there are also sometimes encountered holes for the bolts securing the crane rail to the girder with the aid of claws); $W_{n.b}$ is the net section modulus for the bottom fiber of the girder (for welded girders W_{gr} is taken); W_{br} is the section modulus of the bracing beam comprising the top flange of the crane girder (6.5), a horizontal plate and an end framing flange (Fig. 6.2) with respect to the vertical axis *y–y*, while if there is no bracing beam-of only the top flange of the girder.

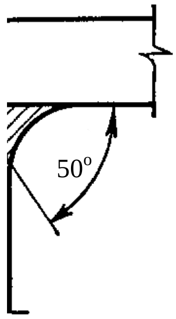


Fig. 6.2. Top flange weld of crane girder with penetra-tion throught the whole web thickness

The strength of a solid crane girder is checked (when a bracing member in the form of a truss is employed) as follows:

for the top fiber of the girder according to the expression

$$\sigma = \frac{M}{W_{n \cdot t}} + \frac{F_{br}}{\varphi A_{fl \cdot gr}} + \frac{M_{br \cdot toc}}{W_{fl}} \leq R_y \gamma_c , \qquad (6.6)$$

for the bottom fiber of the girder — by means of the expression (6.4).

Here $F_{br} = \dfrac{M_{br}}{h_{br}}$ is the design longitudinal force in top flange of a girder with a sectional area of $A_{fl.gr}$, as in the flange of a horizontal truss (with a depth of h_{br}) induced by the lateral braking forces; φ is the buckling factor in a horizontal plane for the length of the panel *d*; $M_{br \cdot loc} = \dfrac{F_{br} d}{5}$ is the local moment in the top flange of the girder in a horizontal plane, induced by the lateral braking force F_{br}; W_{fl} is the section modulus of top flange of crane girder with respect to its vertical axis.

The general stability of crane girders with a reinforced top flange (without a bracing beam) is investigated by means of the expression (6.6)

$$\sigma = \frac{M}{\varphi_b W_{gr}} \le R_y \gamma_c .$$

When determining the factor φ_b the width *b is* taken equal to that of the reinforced top flange.

The endurance of crane girders should be investigated for the service load of one crane from among those operating in the given span, without regard to the dynamic factor. The crane having the highest lifting capacity is taken for these calculations.

The endurance of simple crane girders with a solid section designed from grade ВСт.3 нс steel (welded) and grade ВСт.3 пс steel (riveted) or from low-alloy steels may be left unchecked on condition that the flange welds be made with full penetration (Fig. 6.2) or by means of automatic welding.

Additional Checking of Crane Girder. Having selected the section and investigated its strength, the following should be analyzed:

– the deflection of the girder;

– the strength of the web in local crushing induced by the load of the crane wheels [by means of equation (6.1)];

– he flange welds or rivets connecting the flanges of the girder to the web;

– the local stability of the girder web in accordance with the contemplated arrangement of the stiffeners.

The deflection of the girder can be checked by means of the equation

$$\delta = \frac{M_s L^2}{10EI},$$

in which M_s is the moment produced by the vertical service loads (without introducing the load and the dynamic factors).

This deflection should not exceed the allowable value.

The flange welds or rivets connecting the top flange of the crane girder to the web are analyzed for the action of not only the horizontally directed

shearing stresses due to bending τ_h but also of the local vertically directed stresses σ_{loc} [see expression (6.1)] caused by the concentrated load of the wheel. Here it is assumed that in welded girders the concentrated load *P* is resisted by the welds over a length of z, found from equation (6.2). The resultant of the shearing and local stresses, which is conditionally determined by geometrically adding them, must not exceed the design shear strength of the fillet welds,
whence

$$\tau = \sqrt{\tau_h^2 + \sigma_{loc}^2} = \frac{1}{2k_w h_w} \sqrt{\left(\frac{QS_{fl}}{I_{gr}}\right)^2 + \left(\frac{n_1 P_1}{z}\right)^2} \le R_{w \cdot f} .$$

In crane girders the welds securing the top flange to the web should be made with penetration through the whole thickness of the web. To ensure such a penetration, the edge of the girder web, when it has a thickness of 10 mm or above, is milled to a double-bevel shape (Fig. 6.2); with automatic welding edges are milled in this manner when their thickness is 14 mm and more. When welding is performed with penetration over the whole thickness of the web, the standards permit designers to consider that the weld has the same strength as the web.

In riveted crane girders it will be very good practice to mill the upper edge of the web flush with the back ends of the flange angles. For this purpose they are first made to protrude five millimeters beyond the angles. It is assumed here that the concentrated loads are partly transmitted directly to the web (60 %) and partly to the angles of the flange riveted to the web (40 %). When investigating the top flange rivets, it is assumed that the concentrated load *P* is uniformly distributed between the rivets located along the length *z* determined from equation (6.3). The pitch of the flange rivets in this case is determined from the expression

$$a \le \frac{F_r'}{\sqrt{\left(\frac{QS_{fl}}{I_{gr}}\right)^2 + \left(\frac{\alpha n_1 P_1}{z}\right)^2}}.$$

where $\alpha = 0.4$ if the upper edge of the web is milled flush with the back ends of the angles; $\alpha = 1$ if the web is not milled; F'_r is the maximum design force allowed per rivet in crushing or double shear.

The local stability of crane girder webs is checked if $\frac{h_w}{t} > 80\sqrt{\frac{21}{R}}$. To ensure stability of the web there are installed pairs of main stiffeners, whose spacing, as has already been indicated, should not exceed $2h$ when $\frac{h}{t} > 100$ and $2.5h$ when $\frac{h_w}{t} \le 100$. Having planned the arrangement of the stiffeners, the dimensions of the investigated panels between them are established. The stability of the web is checked for the combined action of the normal, shearing and local stresses σ, τ and σ_{loc}. When the web is being braced only with transverse stiffeners and when $\frac{a}{h_w} \le 0.8$ (a is the spacing of the stiffeners and h_w, is the design depth of the web), the stability is checked by means of the expression

$$\sqrt{\left(\frac{\sigma}{\sigma_{cr}} + \frac{\sigma_{loc}}{\sigma_{loc \cdot cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2} \le 0.9. \qquad (6.7)$$

The local stress σ_{loc} is determined from expression (6.3), but the factor n_1 is taken equal to 1.1 for all crane girders. The values of the critical stress σ_{cr} and τ_{cr} are taken in accordance with the equations:

$$\sigma_{cr} = k_g\left(100\frac{t}{h_0}\right)^2 \text{ and } \tau_{cr} = \left(12.5 - \frac{9.5}{\mu^2}\right)\left(\frac{100t}{d}\right)^2. \qquad (6.8)$$

Fig. 6.3 pictures the loss of stability of a web owing to local stresses. The critical local stress is found from the equation

$$\sigma_{loc\cdot cr} = \frac{k_g 10^4}{\lambda_{loc}^2} = k_g \left(\frac{100t}{a} \right)^2, \qquad (6.9)$$

where $\lambda_{loc} = \frac{a}{t}$ is the slenderness ratio of web between stiffeners; a is the spacing (distance between centre lines) of transverse stiffeners; k_g is the factor depending upon the depth of the web and the value of γ and taken in accordance with table 6.3.

If $a > 2h_w$, then in determining $\sigma_{loc\cdot cr}$ it is assumed that $a = 2h_w$. When $\frac{a}{h_w} > 0.8$ there may be two forms of buckling of the crane girder web, one with a ratio of the sides of a/h_w, having one half-wave over the length of the panel, and the other with a ratio of the sides of $\frac{0.5a}{h_w}$ having correspondingly two half-waves.

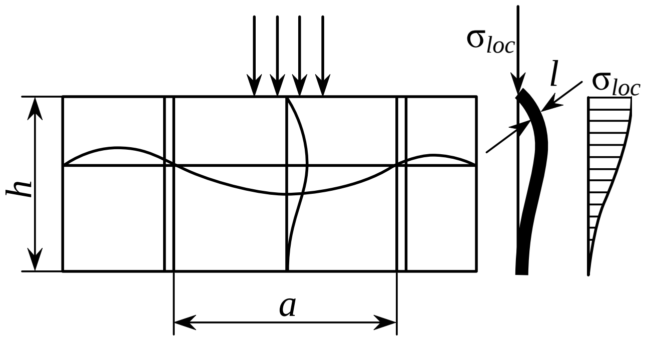


Fig. 6.3. Loss of stability of crane girder web due to local stresses

As has already been indicated, it will be good to install a longitudinal stiffener (Fig. 6.4) in heavily loaded girders with a large span, and having a web slenderness ratio of $\frac{h_w}{t} > 160\sqrt{\frac{21}{R}}$. Such a stiffener, generally located at a distance of $b_1 = (0.2 \text{ to } 0.25)h$ from the extreme compression fiber of the panel, divides it into two parts, namely, the top and the bottom ones.

The top panel, located between the compression flange and the longitudinal stiffener, is under conditions of non-uniform compression.

Table 6.3

Values of Factor k_g for Welded Girders

γ	Factor k_g for a/h_w equal to								
	<0.5	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0
< 0.8	2.04	2.39	3.03	3.67	4.11	4.67	5.32	6.04	6.88
1.0	2.07	2.44	3.13	3.80	4.29	4.90	5.61	6.39	7.29
2.0	2.13	2.54	3.33	4.17	4.85	5.67	6.59	7.59	8.72
4.0	2.17	2.60	3.47	4.46	5.33	6.38	7.58	8.89	10.35
6.0	2.18	2.62	3.52	4.58	5.55	6.73	8.07	9.54	11.19
10.0	2.19	2.64	3.57	4.68	5.76	7.07	8.56	10.21	12.06
> 30.0	2.21	2.67	3.65	4.86	6.15	7.74	9.56	11.58	13.86

It is checked by means of expression (6.9) obtained from consideration of the boundary curve separating the limiting areas of the load-carrying capacity of the panels

$$\frac{\sigma}{\sigma_{cr1}}+\frac{\sigma_{loc}}{\sigma_{loc\cdot cr1}}+\left(\frac{\tau}{\tau_{cr1}}\right)^2 \le 0.9 .$$

When $\sigma_{loc}=0$, the critical stress σ_{cr_1} is found from the equation

$$\sigma_{cr_1}=\frac{10}{1-\frac{b_1}{h_w}}\left(\frac{100t}{b_1}\right).$$

The bottom panel, located between the tension flange and the longitudinal stiffener, is in non-uniform tension. The stability of this part is higher than that of a girder web with a conditional depth of $h_w - 2b_1$, the latter being used for analysis. The stress in the extreme fiber of the conditionally separated girder (6.2) is equal to $\sigma_2=\sigma\left(1-\frac{2b_1}{h_w}\right)$.

The boundary curve of the stable and unstable areas in this case is assumed to have the form of

$$\sigma_{cr_2} = \frac{11.4}{\left(0.5 - \frac{b_1}{h_w}\right)^2} \left(\frac{100t}{h_w}\right)^2,$$

where $\sigma_{loc \cdot cr_2}$ is determined from equation (6.9) and table 2.3 for γ = 0.8, with substitution of the expression $\frac{a}{h_w - b_1}$ for $\frac{a}{h_w}$ when a load is applied to the compression flange, the value of σ_{loc} is taken equal to σ_{loc2} = 0.4 σ_{loc}.

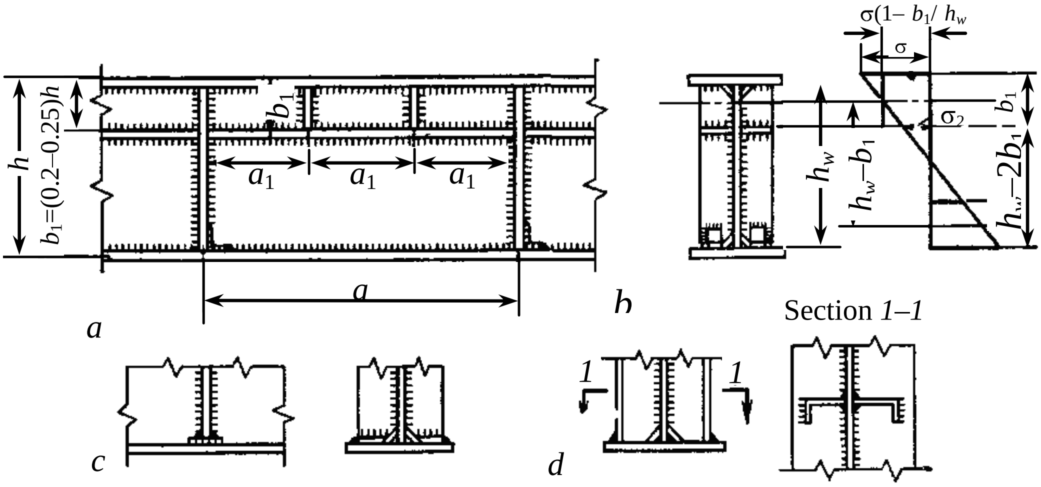


Fig. 6.4. Bracing of crane girder web with longitudinal stiffener, and details of ribs

In continuous girders, when the load is applied to the tension flange and the horizontal stiffener is correspondingly located in the bottom compression part of the girder, $\sigma_{loc2} = \sigma_{loc}$.

When a longitudinal stiffener is employed in a welded girder, the compression zone of the web may be braced with short stiffeners welded to the longitudinal one (Fig. 6.4, *a*).

The web stability of girders with an unsymmetrical section (with a more developed compression flange) is investigated for webs braced only with transverse stiffeners by means of expressions (6.7) and (6.8) in which h_w, is taken equal to the double distance from the neutral axis to the compression boundary of the panel.

Transverse stiffeners must be welded to the top flange of the girder by means of small-size welds (6–8 mm).

The transverse welds connecting the stiffeners to the bottom flange must also have a minimum size, since shrinkage of these transverse welds leads to the appearance of internal tensile stresses. In the present case the latter will be added to the tensile stresses induced by the external load.

It is not good policy to leave a space between the stiffeners and the bottom flange, since this will greatly reduce the rigity in twisting of the girder, and the bottom flange will be easily deformed. The stiffeners can be welded to the bottom flange with the aid of short plates or angles that are connected to the flange only with longitudinal welds (Fig. 6.4, *a* and *c*). For heavy crane girders in shops with heavy service conditions, stiffeners made of unequal leg angles welded to the web along their edge may be used (Fig. 6.4, *d*).

The stiffeners should be welded to the web by means of continuous welds of minimum size. When located near welded web splices, the stiffeners must be arranged at a distance of at least $10t_{web}$ from the welds. To reduce the influence of the heat-affected zone of the welds, the ends of vertical stiffeners should be bevelled 60 mm in depth and 40 mm in width. For the same reason at the intersections of girder web splice welds with stiffeners, the welds connecting the stiffeners to the web should be terminated at a distance of 40–50 mm before the splice weld.

The lower ends of supporting ribs can be designed with bevels 30 mm in depth and 20 mm in width, but with the weld connecting the stiffener to the flange terminating 40 mm before the web.

The dimensions of the main transverse stiffeners are established in the same way as in ordinary beams, i. e., the width of the stiffener $b_{rib} \geq \frac{h}{30} + 40$ mm and the thickness thereof $t_{rib} \geq \frac{1}{15} b_{rib}$.

Lattice Crane Girders (Trusses). With spans of 18 metres or more, and with cranes having a lifting capacity of Q = 10–20 tons, it will be good practice to employ lattice crane *g*irders (Fig. 6.5, *a*).

The top flange or chord of such girders is made of a rigid section (I beam) subjected not only to compression as part of the girder, but also to local bending under the load of the crane wheel *P*. The local bending moment can be found from the equation

$$M_b = \frac{Pd}{3},$$

in which *d is* the distance between panel points of the top chord.

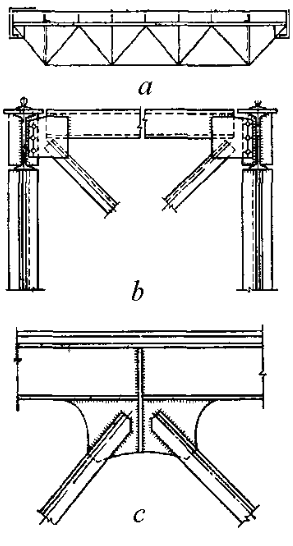


Fig. 6.5. Lattice crane girder

The remaining sections of the elements, as a rule, are designed of two angles. The elements of the lattice are ordinarily centered on the bottom edge of the chord.

All the bars of the lattice girder, except for the top chord, resist axial loads and are analyzed in the same way as the elements of trusses.

The expression used for investigating the top chord is

$$\sigma = \frac{F + F_{br}}{\varphi A_{gr}} + \frac{M_b}{W_x} + \frac{M_{br}}{W_y} \leq R,$$

where F is the design force induced by vertical loads; F_{br} and M_{br} are the design compressive force and local bending moment in horizontal plane induced by lateral braking forces (when a horizontal bracing truss is used); φ is the buckling factor with respect to the vertical axis (with respect to the horizontal axis when a solid bracing beam is used).

It is necessary to brace in a horizontal plane not only the top chord (Fig. 6.5, *b*), but also the bottom one, seeing that the bottom chord may be displaced sideways as a result of deflection of the girder owing to the influence of even the most insignificant eccentricity. For this reason the standards establish a limiting slenderness ratio for the bottom chord of lattice crane girders equal to $\lambda = 150$. For the same reason ribs should be installed on the gussets of the top chord (Fig. 6.5, *c*). The welds connecting the compression diagonals should be terminated at a distance of 40–50 mm before the nearest welds on the gusset.

Horizontal Bracing Beams. Bracing beams are mainly designed of corrugated steel sheets from 6 to 10 mm thick, with one flange made of a channel (Fig. 6.6) or an angle. The top flange of the crane girder serves as the second flange. With a girder span of 12 meters or more the outer flange of the bracing beam is usually suspended from members located above it. It will not be good practice to support it by means of a strut connected to the bottom flange of the girder, as this will result in oscillation in a horizontal plane. To provide a convenient passage, the width of the bracing beam should be at least 750 mm. With two crane girders, a bracing plate connects the top flanges of both girders.

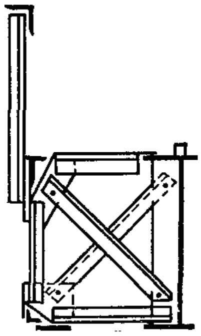


Fig. 6.6. Bracing beams

Crane Girder Column Connections. Crane girders are generally supported on columns by means of top-seated connections in accordance with one of the following two alternatives (Fig. 6.7):

– the milled edges of the end bearing plates of adjacent girders, which are bolted together, rest on the axis of the column (Fig. 6.7, *a*);

– the girders are supported on the crane I-shape branch of the column with the support stiffeners of the girders located opposite the flanges of the column branch (Fig. 6.7, *b*), or displaced a small distance inward toward the axis of the column.

For girders with a depth exceeding 1.2 meters it is recommended, while in shops with heavy service conditions it is obligatory, to install bearing membranes. In buildings with heavy service conditions, these membranes are connected by welding, using type Э52А electrodes or by means of rivets, the connection being investigated for the action of the force computed from the equation

$$R_c = \frac{Q_{br} h}{h - a},$$

where Q_{br} is the reaction of bracing beam multiplied by the factor k_{br} found in table 6.1; h is the depth of girder; a is the distance from the top of rail to the centre of riveted connection.

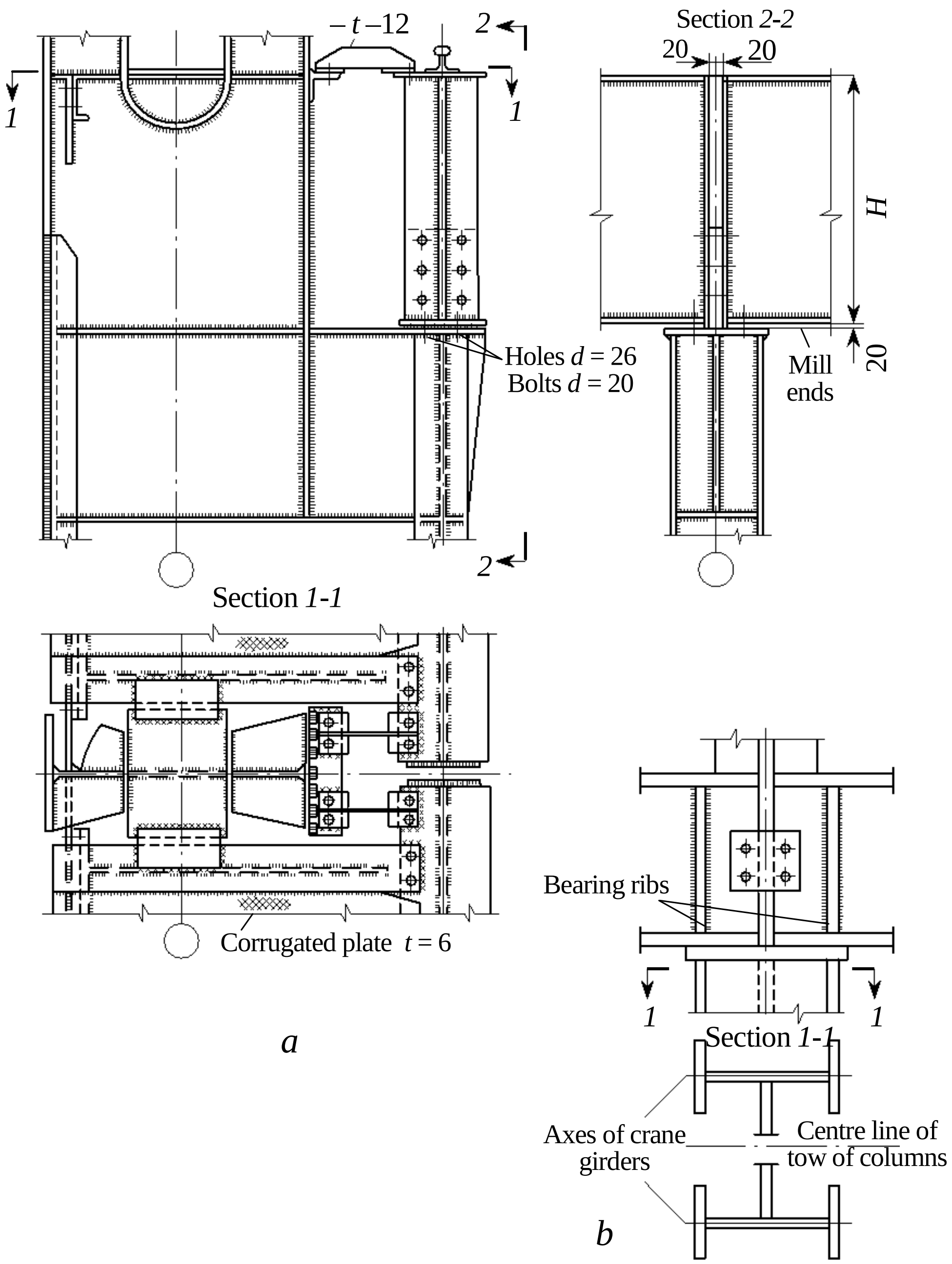


Fig. 6.7. Connection to columns of crane girders having the same depth

The vertical bolts connecting crane girders to columns are analyzed for shear induced by the force developed in longitudinal braking of the cranes, this force being

$$F_{br\cdot 1} = 0.1\sum P\,,$$

in which $\sum P$ is the sum of the loads of all the crane braking wheels on the girder (the number of braking wheels is generally equal to half of the total number of crane wheels).

The diameter of the bolt holes in the bearing plate is made 3–5 mm greater than the diameter of the bolts, while the washers are tightly fitted onto the bolts and welded in place during erection.

Types of Crane Rail Fastenings. The capacity of crane rails is filled by type *KP* rails having a special section to GOST 4121, by ordinary railway rails, or by rectangular bars. The type of rail and fastening to be employed is selected depending upon the capacity of the crane, its duty and the type of crane wheels (cylindrical or tapered).

The required rail head width is indicated in the standards for cranes and varies from 50 to 140 millimeters.

The fastening of the rails to the crane girders may be either rigid or movable, i. e., permitting lining (straightening) of the track.

Reliable fastening of the rails is very important for normal service. Rapid deterioration of the track and flange of the girder owing to poor initial lining and poor fastening of the rails is sometimes observed.

Fixed attachment, i. e., welding of the rail to the girder, is permitted only for cranes with a light duty, and in all other cases a movable fastening should be used. With a small width of the girder flange, railway rails can be secured in place with hooks (spaced 500–700 mm apart) directed to both sides. The special type KP crane rails are fastened in place by means of batten plates.

Rectangular-section rails (plates or bars) are fastened with the help of batten plates inserted into longitudinal milled grooves in the rail. Oval holes are made for the bolts in the batten plates, and round ones in the crane girders and baseplates.

A clearance of 10–20 mm is left between the baseplates and the sides of the rail to permit lining of the latter, while the thickness of the baseplates is made 2–3 mm less than the distance from the groove of the rail to its bottom, to ensure a spring action when the batten plates are bolted in place.

After installation of the batten plates into the grooves, they are welded to the baseplates. As a rule, each batten plate is bolted with at least two bolts.

When selecting the section of the crane girders, account should be taken of weakening of the section by the holes for these bolts.

6.2. Prestressed beams

Prestressing is employed in metal beams and girders in order to reduce the amount of metal required and the deflections of a member, and also to regulate the stresses in a member for obtaining the most rational shape. Prestressing can be achieved in various ways; the main ones are:

1. Tensioning with a high-strength element (a wire cable or a steel rope) located in the tension zone of the member.

2. Regulating the bending moments and deflections by changing the relative level of the supports in continuous beams, the tension of the overhanging ends in cantilever beams, and so on.

In the first case, owing to eccentric compression of the member stresses of the reverse sign to those which will be induced by the live load develop in it. When a live load is applied to the beam, the initial stresses caused by prestressing are first neutralized, and for this reason a higher total service load may be applied to the beam. Here the wire rope or cable located in the tension zone is additionally loaded (self-tensioned). Owing to the high strength of the wires, however, a comparatively small amount of metal is required for them. The total saving in the weight of metal in this case reaches 10–18 %, the saving in costs reaching 5–15 %.

In continuous beams it is also good practice to employ prestressing by locating the stressing elements above the supports in the tension zone. Each stressing element adds one redundant force to the scheme used for analysis.

With a large dead load on a beam, it is good policy to employ repeated prestressing by gradually loading the beam, bringing up the priestess to the maximum value each time. In the second case (regulation of moments) it is possible to reduce the bending moments in a span by increasing them on the supports, for example by tensioning the overhangs or arms, where, as a rule, it is easier to design sections with a large depth. This will also reduce the deflection at the middle of the span. It is good in such cases to make use of composite beams, in which reinforced concrete flooring included as part of the compression flange of the member takes the load together with the steel beams.

Let us consider the behavior of a beam that has been prestressed once. The high-strength element (generally known as a tie rod or cable) in the form of a bundle of wires with a diameter of 3–6 mm (or a rope) is located, as a rule, in the lower part near the bottom flange (better below it). It is freely passed through guide carriers having the form of half-rings welded to the flange at about 1.5–2 meters from each other. These carriers are necessary to prevent the bottom flange, which will be subjected to compression, from losing its stability in the process of prestressing of the cable. Ordinarily the cable does not occupy the whole length of the beam, and its ends are secured in special anchorages.

When analyzing a prestressed simple one-span beam, the sequence of its behavior step-by-step is taken into consideration. First, when the cable is prestressed with a force of F_{ps}, the beam is subjected to eccentrical compression, which induces in the top flange of the beam a tensile stress

$$\sigma_{t\cdot 1} = -\frac{n_2 F_{ps}}{A} + \frac{n_2 F_{ps} h_a}{W_t}$$

in the bottom flange – a compressive stress

$$\sigma_{b \cdot 1} = -\frac{n_2 F_{ps}}{A} - \frac{n_2 F_{ps} h_a}{W_b},$$

where n_2 = l.l is the load factor for the prestressing force, W_t and W_b, are the section modules of the beam for the top and bottom fibers, respect-tively.

The next stage, when the beam is subjected to a live service load, it begins to behave as a statically indeterminate trussed beam with one redundant element. We take as the unknown quantity the additional force in the cable X_1, which begins to develop under the action of the external load. This force, which can be termed a self-tensioning one, induces additional stresses in the beam.

The total stresses resulting from the combined action of the pre-stress and the external load will be:

in the top flange

$$\sigma_t = -\frac{n_1 F_{ps} + X_1}{A} - \frac{M_p - (n_1 F_{ps} + X_1) h_a}{W_t},$$

in the bottom flange

$$\sigma_t = -\frac{n_1 F_{ps} + X_1}{A} + \frac{M_p - (n_1 F_{ps} + X_1) h_a}{W_t}.$$

Here the load factor for the presstressing force F_{ps}, is taken equal to n = 0.9, i. e., less than unity, since in the present case a reduction of this force increases the stress.

The self-stressing force X_1 is determined from the general equation

$$X_1 = -\frac{\Delta_{1p}}{t_{11}} = \frac{\int_a^{L-a} \frac{M_1 M_P}{EI} dx}{\int_a^{L-a} \frac{M_1^2 dx}{EI} + \frac{L_a}{E_a A_a} + \frac{L_a}{EA}}.$$

Here M_p is the moment in the main system-beam induced by the external load; M_1 is the moment induced by the force X_1 = l; EI is the rigity of beam in bending; $E_a A_a$ is the rigity of tensioning element (cable); EA is the rigity of beam in longitudinal straining; a is the length of part of the beam having no prestressing cable.

Hence for a beam with a cable, the self-stressing force X_1 will be found as follows:

for a beam which a uniformly distributed load is applied to

$$X_1 = \frac{2M_p h_a}{3\left(h_a^2 + \frac{I}{A} + \frac{EI}{E_a A_a}\right)} \gamma_1$$

for a beam which a system of identical concentrated loads is applied to

$$X_1 = \frac{Ph_a\left[L \;\; x_1 + x_2 + ... + x_i \;\; - \;\; x_1^2 + x_2^2 + ... + x_i^2 \;\;\right]}{2L\left(h_a^2 + \frac{I}{A} + \frac{EI}{E_a A_a}\right)} \gamma^1 .$$

Here

$$\gamma_1 = \frac{L^3 - 6La^2 + 4a^3}{L(L-2a)} \cong \left(2 - \frac{L_a}{L}\right);$$

$$a = \frac{L - L_a}{2},$$

where h_a is the distance from neutral axis to axis of cable; P is the magnitude of concentrated load x_1, x_2, ..., x_i are the distances to 1st, 2nd, ..., i-th load from left-hand support.

The optimal section of the beam is obtained with such ratios between the sectional areas of the flanges and the web, with which the limiting stresses attained in the flanges of the beam are due not only to the load σ_b and σ_t , but also to presstressing in the bottom flange $\sigma_{b\cdot 1}$. It is this that limits the increase in the load-bearing capacity of prestressed beams. Thus, for the optimal section, the following conditions must be observed

$$\sigma_t = R; \quad \sigma_b = R; \quad \sigma_{b\cdot 1} = R .$$

It is obviously good practice to design beams with an unsymmetrical section, and to select such dimensions of the bottom flange that will just ensure its stability when prestressed.

If we keep the denotation for the geometrical characteristics, namely, the web slenderness ratio $\lambda = \frac{h_w}{t}$, the material distribution or shape factor $k_d = \frac{A_w}{A}$, i. e., the ratio of the web sectional area to that of the entire beam, and introduce the conception of factor of asymmetry

$$k_{as} = \frac{h_2}{h_1} = \frac{W_t}{W_b}$$

then the optimal geometrical characteristics of the section can be expressed as follows:

$$A_w = k_d A; \quad A_1 = A\left(\frac{k_{as}}{k_{as}+1} - \frac{k_d}{2}\right);$$

$$A = A_1 + A_2 + A_w. \tag{6.10}$$

By satisfying the optimal condition, it is possible to express the moment taken by the beam as a function of the geometrical parameters and the self-stressing factor $M = R\sqrt{A^3\lambda_w C}$,

where

$$C = (1-\beta)\sqrt{\frac{6k_{as}^3(1-k_{as})^2 \;\; k_{as}-(1+\beta)}{(k_{as}+1)^3 \;\; k_{as}(1-\beta)-(1+\beta) \;\; ^3}};$$

$\beta = \frac{n_1 F_{ps} + X_1}{n_1 F_{ps}}$ — self-stressing factor.

The quantity C can be called a parameter of the profitability of the section. For practical values of the self-stressing factor P within the limits of 1 to 3, the factor k changes very slightly, and can be assumed to be constant (k_d = 0.55) within a quite wide range of values of the asymmetry factor k_{as}.

For three characteristical types of loading indicated in table 6.4 it is possible to find the self-stressing force X_1 and establish the relation between k_{as} and C for the optimal conditions. Here it is assumed that the length of the stressing element is established so as to satisfy the condition of attaining the limiting stress (σ = R) in the beam at the beginning and end of the prestressing cable, while the sectional area of the stressing element is determined from the condition that the sum of the projections of all the forces on the horizontal axis $(A_1 - A_2)R = A_a R_a$ equals zero, by the equation

$$A_a = A\frac{R}{R_a}\cdot\frac{k_{as}-1}{k_{as}+1}. \tag{6.11}$$

Hence we directly obtain the required value of presstressing F_{ps} if the kind of loading of the beam is known, i. e., if the force X_1 is known

$$F_{ps}n_2 + X_1 = A_a R_a,$$

whence

$$F_{ps} = \frac{F_a R_a - X_1}{n_2}.$$

Table 6.4 gives the values of k_{as} and C depending on the selected ratio μ_a, between the unit strain of the beam and that of the presstressing cable materials, and on the kind of beam loading

$$\mu_a = \frac{\varepsilon}{\varepsilon_a} = \frac{RE_a}{R_a E}. \tag{6.12}$$

The load factors $n_1 = 0.9$ and $n_2 = 1.1$ are used with indirect control of the prestressing force F_{ps}, (by noting the deflection of the beam, the force used to tighten the bolts, driving in wedges, etc.).

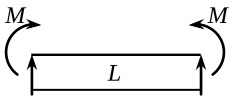
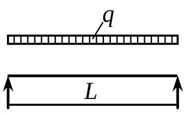
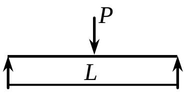
When determining the magnitude of the prestressing force by means of instruments (a pressure gauge in the hydraulic system, by measuring the stress with a tensometer, etc.) the values of the load factors may be taken equal to $n_1 = n_2 = 1$.

The values of the rational length of the stressing element for the various kinds of loading are indicated in the last column of table 6.4, where the coefficient η is equal to

$$\eta = 1 - \frac{\sqrt{0.55}}{C}\left[\frac{6k_{as} - 0.55(k_{as}+1)^2}{k_{as}(k_{as}+1)}\right].$$

Table 6.4

Values of quantities k_{as} and C

Kind of beam loading	V–a	$n_1 = 0.9$	$n_2 = 1.1$	$n_1 = 1$	$n_2 = 1$	Rational length of stressing element
		k_{as}	C	k_{as}.	C	
 	0.1 0.2 0.3 0.4	1.58 1.75 1.99 2.40	0.347 0.359 0.381 0.415	1.87 2.11 2.56 3.60	0.348 0.369 0.399 0.446	$L_a = L$
 	0.1 0.2 0.3 0.4	1.69 1.80 1.95 2.12	0.329 0.341 0.354 0.367	1.83 1.98 2.16 2.36	0.344 0.357 0.371 0.384	$L_a = L\sqrt{\eta}$
 	0.1 0.2 0.3 0.4	1.72 1.88 2.07 2.27	0.323 0.328 0.332 0.336	1.82 1.94 2.06 2.19	0.342 0.353 0.363 0.373	$L_a = \eta L$

Thus the optimal section of a beam or girder with a straight prestressing cable located at the level of the bottom flange can be selected in the following sequence:

1. The desired slenderness ratio of the web $\lambda_w = \dfrac{h_w}{t}$ is selected or the minimum possible web thickness t is determined, with a view to the web resisting the action of the shear force Q or the action of the local load from equation (6.3).

2. The material of the prestressed cable (a bundle of wires or a steel rope) is selected and the unit strain factor μ_a is found from equation (6.12).

3. The values of the asymmetry factor k and the parameter C are found in table 6.4 depending on the value of μ_g and the kind of beam loading.

4. In accordance with the values of M, C, R and λ_w, or t, the geometrical

characteristics of the beam are determined from equations (6.10) and (6.11), assuming that $k_d = 0.55$.

Methodical instructions to Chapter 6

In this unit special attention should be paid to the designing of crane beams with a slender wall.

To design crane beams it is necessary to study types of beams cross-section, the methods of analyzing and rules of solid crane girders and lattice crane girders construction.

It is important to know how to select beams section of different types and check the selected section.

Students should also learn theoretical bases of beams prestressing and ways of prestressed beams calculation.

Questions and Tasks to Chapter 6

1. *What crane beams and girders are used in an industrial building?*
2. *What are the main features characterizing the behavior of crane girders?*
3. *Name the types of crane girders sections.*
4. *What are the features of solid crane girders analysis?*
5. *Describe the features of design forces, acting on solid crane girders.*
6. *What are the application field of lattice crane girder and peculiarities of its designing?*
7. *Describe the types of crane rail fastening.*
8. *What are prestressed beams and the purpose of their usage?*

7. SPECIAL MEMBERS

7.1. Heavy Trusses of Large-Span Roofs

Heavy trusses is the name given to trusses whose elements have a double-plane section. The necessity of using such sections appears when the forces in the chords of the truss elements exceed 350 to 400 tons. Such great forces occur in trusses extending over large spans reaching 80 to 100 meters in length and subjected to heavy loads. Examples of such structures are the large-span trusses of aircraft assembly shops, hangars, ship-building slips, from which cranes are suspended, large-span crane trusses with heavy cranes, large-span trusses of public structures (exhibition pavilions, sports structures) and bridges.

The main trusses of large-span structures can be of a great variety both with respect to their type and their configuration. They are employed in the form of girder trusses or collars of trussed frames with parallel or polygonal chords, in the form of arch trusses, bowstring arches, girders strengthened with a parabolic arch or struts, etc.

Fig. 7.1 illustrates a frame with two pin connections at the supports, and at the top of the columns. The first type of frame is more advantageous, since the rigid corner of the frame facilitates relieving of the collar, reducing the stresses in it Fig. 7.l pictures a frame without pin connections, whose drawback the necessity is of is more sensitive to changes in temperature developing the foundation constraining the legs.

Besides, such a frame (Fig. 7.1) depicts an example of a hangar consisting of a main two-hinged frame with a span of 100 meters above the entrance gate, which supports roof trusses with a span of 60 meters and with a spacing equal to the panel of the main frame. On the other side, the roof trusses are supported on columns. Light traveling cranes (Q = 3 to 5, tons) are suspended from girders connected to the roof trusses at the joints. The entrance gates arranged along the frame consist of separate leaves mounted at the bottom on wheels. On top they rest horizontally against special guides, the latter, in turn, bearing against the ties along the bottom chords of the trusses.

The main frame has a thrust that in good soils can be resisted by the foundations and in poor soils by a steel tie bar subjected to tension, which is laid in a special box-shaped duct under the floor.

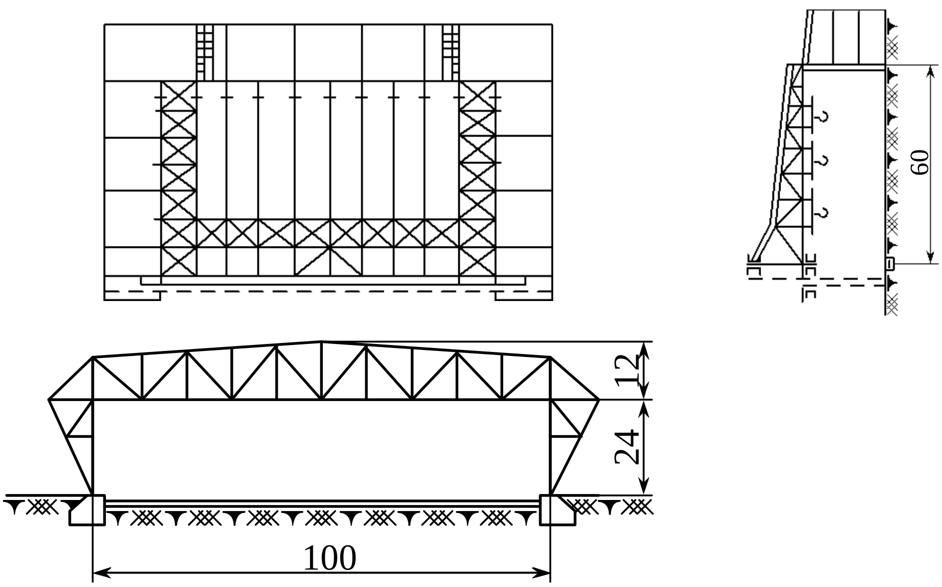


Fig. 7.1. Hangar (plan view and sections)

The depth of the main truss (collar of the frame) is appointed within the limits of 1/8 to 1/l0th of the span, depending on the conetruction reguirement to reduce building space size. Frame constructions make possible the wide use of prestressing, which reduces the weight of the members. In particular, in the frame pictured in Fig. 7.1 the extreme braces at the columns can be tensioned, relieving the collar to a still greater extent.

When designing a large-span structure, special attention should be devoted to arrangement of the ties ensuring stability of the top truss chords and rigidity of the structure as a whole as well as resisting the horizontal (wind) loads.

A distinguishing feature of large-span structures consists in the own weight of the roof being their dominant load. Hence it follows that in designing such structures, great importance should be attached to selecting the roof type. The use of an aluminum roof in these structures is usually justified from the economical viewpoint.

A double-pinned frame (Fig. 7.1) is investigated as a statically indeterminate system with one redundant member (the extra unknown force X_i is the thrust) with regard to deformation of the tie bar (if employed), i. e.,

$$X_1 = -\frac{\Delta_{1e}}{\delta_{11} + \dfrac{EL_{tb}}{A_{tb}E_{tb}}} = -\frac{\sum \dfrac{F_1 F_e}{A_i} L_i}{\sum \dfrac{F_1^2 L_i}{A_i} + \dfrac{EL_{tb}}{E_{tb}A_{tb}}} . \qquad (7.1)$$

Here F_1 = forces in the elements of the main system induced by force X_1 = l; F_e = forces in the elements of the main system induced by external loads; L and A_i = lengths and sectional areas of the elements of the truss chords and columns (the lattice may be omitted); L_{tb} and A_{tb} = length and sectional area of the tie bar.

The forces F in the elements will be

$$F = F_e + X_1 F_1 .$$

Seeing that the value A_i is found both in the numerator and the denominator of equation (7.1), then only the ratio $\frac{A_i}{A_{ref}}$ will be of importance, where A_{ref} is the conditional or reference sectional area, for example 1 or 100 cm^2. The investigations are carried out in tabular form.

When it is necessary to take into consideration the influence of the difference in ambient temperatures on a member, the corresponding thrust is determined from the equation

$$X_1 = \frac{E\Delta_{1t}}{\delta_{11} + \dfrac{EL_{tb}}{E_{tb}A_{tb}}}.$$

Here $$\Delta_{1t} = \alpha t \sum F_1 L_i ,$$

in which α = the coefficient of thermal expansion; t = the difference in temperatures.

Owing to large stresses, the elements are designed with double-plane sections, generally of the *H*-shaped, channel and other types.

The sections of heavy truss elements are first selected for the heaviest compression element of the chords, then for the lightest one, thus establishing a gradation of shapes and sections. The depth of the section h should not exceed one-tenth of the panel length, in accordance with the assumption that the joints in the truss layout used for purposes of analysis are pinned, and not rigid ones. It is desirable that compression elements have an approximately identical slenderness ratio in both directions, and symmetrical sections. The axes of the chord elements should not be displaced with respect to each other by more than 1.5 % of the section depth; otherwise account must be taken of the additional moment induced by the eccentrically applied axial forces.

The joints of truss elements having an H section are generally designed with the use of external cover gusset plates, ensuring the required transmission of the forces at the chord joint by means of additional straps. The horizontal plate of the H section, in view of the employment of flat jigs for drilling the holes, is sometimes left uncovered at the joint, transmitting the entire force field through the gusset plate. The batten plates or lattices connecting the branches of the section into a single unit are investigated similarly to axially loaded columns. Designers should make every effort to use the minimum number of elements, and also to give the gusset plates a simple shape, without any notches and internal angles. The joint connections may be riveted, on high-strength bolts, or welded. Fig. 7.2 shows a riveted joint of a heavy bridge-type truss having *H*-section elements. The rivets should be arranged symmetrically.

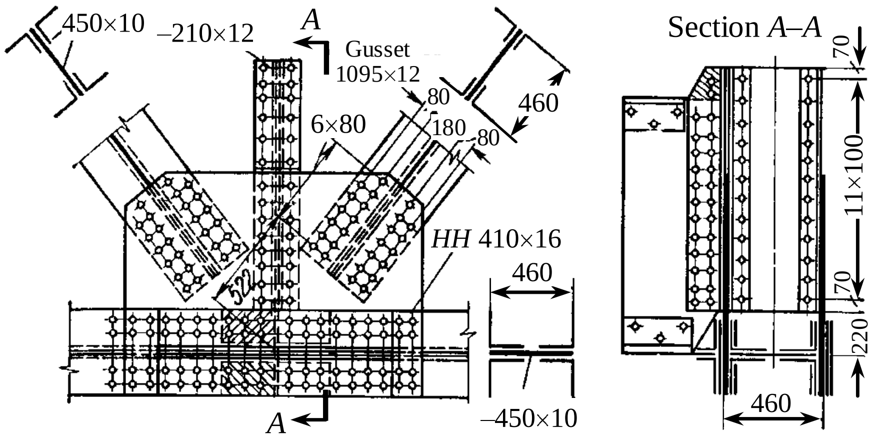


Fig. 7.2. Riveted joint of heavy truss

The welded connections of elements should be so designed as to ensure the lowest possible concentration of stresses in them. To this end it is essential to create a smooth transition of the force field, making coves in the gusset plates and processing the welds in the required manner (Fig 7.3). The bearing parts (supports) of heavy girder-type trusses and large-span frames, which resist a large load reaching several hundred tons, are so designed as to accurately transmit the support reactions at the designed spot.

The design must ensure that these reactions depend to a very low degree on changes in the distance between the points of support caused by deflection of the trusses, or by a change in the slope of the deflection curve. With a view to these requirements there are distinguished fixed and expansion bearings. Beams and girders having a span up to 40 meters are designed on rocker type or tangential bearings (Fig. 7.4, *a*).

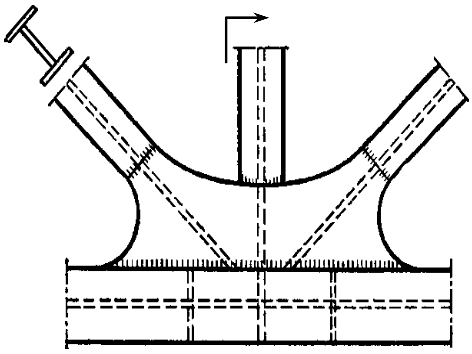


Fig. 7.3. Welded joint of heavy truss

With longer spans, the displacement of the bearing points caused by deflection of the trusses or by temperature changes becomes quite noticeable (Fig. 7.4, *b*).

For this reason the expansion pinned bearings of large-span members are designed on rollers (Fig. 7.4, *c*) Fixed supports are designed in the same way, but without rollers. Steel castings are used as the material for the bearings, while the rollers or the cylindrical bushes of the pinned connection are made of grade BCt. 5 steel.

Bearing rollers (Fig. 7.4, *d*) are investigated for conventional diametrical compression by means of the expression

$$\sigma = \frac{F_r}{NDL} \le R_{c\cdot r}, \qquad (7.2)$$

where F_r = the support reaction; N = the number of rollers; D = the diameter of roller; L = the length of roller; $R_{c\text{-}r}$ = the design strength of rollers in diametrical compression with free contact.

A cylindrical bush or journal with tight contact (a central angle of $\alpha \ge \frac{\pi}{4}$ Fig. 7.5, *a*) is investigated for local crushing by the formula

$$\sigma = \frac{F_r}{1.25rL} \le R_{cr\cdot l},$$

where r is the radius of the pin or journal; R_{crl} is the design crushing strengths for tight contact.

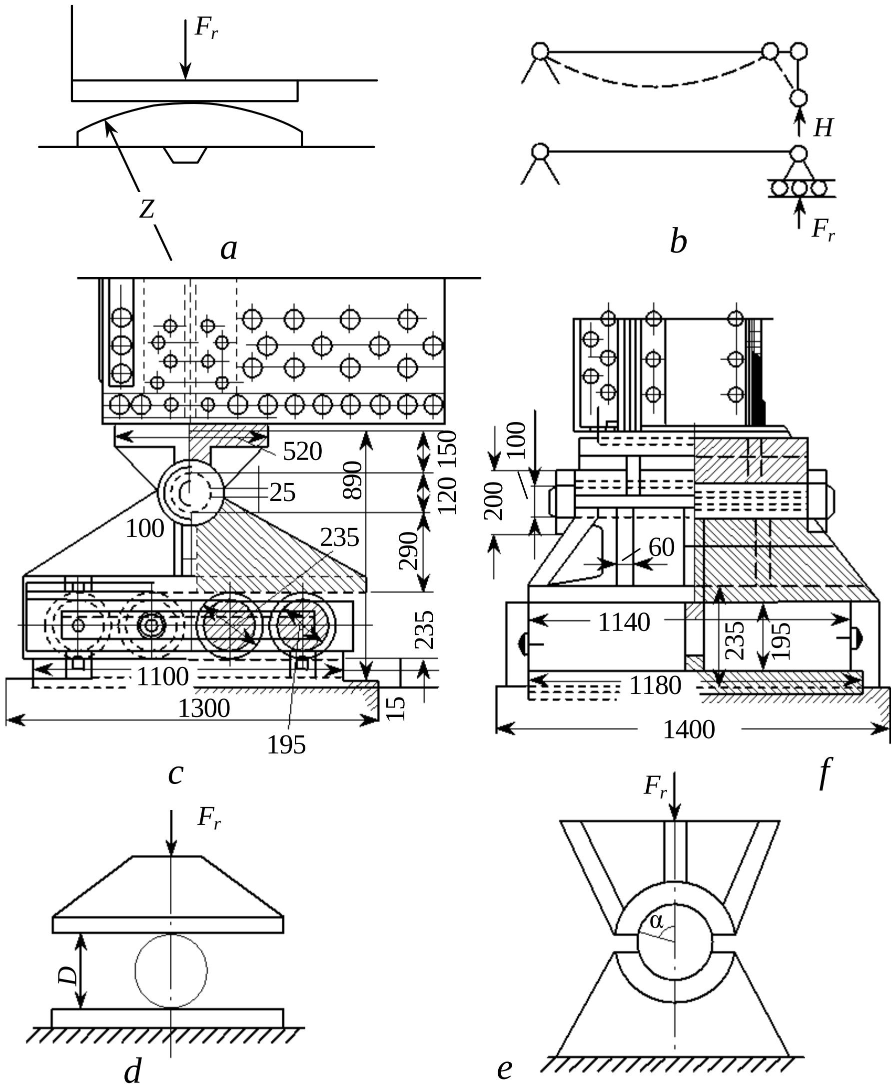


Fig. 7.4. Bearings of heavy trusses

Should the pinned bearings be designed with a spherical head, the crushing stress with tight contact is determined by the expression

$$\sigma = \frac{6F_r}{\pi D^2}\frac{1}{1-\cos^3\varphi} \leq R_{crl},$$

where D = the diameter of spherical surface; φ = the angle of contact of the head with the spherical equalizer.

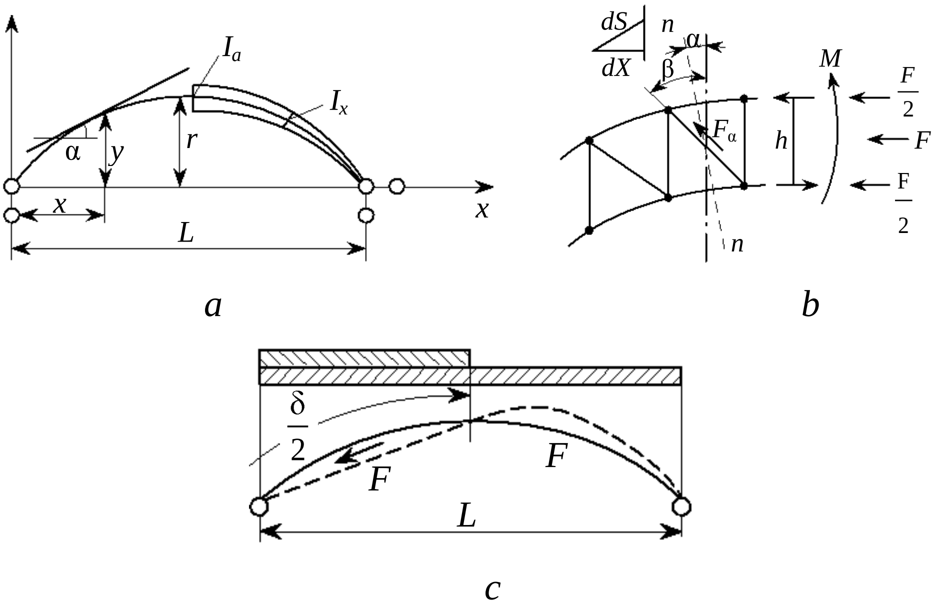


Fig. 7.5 Analysis of two-hinged arch

7.2. Arched, dome-shaped and suspended roofs

7.2.1. Arched roofs

It is rational to employ arched roofs for large spans exceeding 60 to 80 meters. Their principal merit is the member’s low weight, which is explained by the arches being a thrust system, with the section mainly withstanding compression induced by a normal force, and with insignificant bending. This is why they are sufficiently economical. In comparison with a frame member, an arch occupies more space. The hatched area shows the “dead” space that cannot always be utilized. An arched member is more liable to deformation than a frame, since the linear rigity of an arch i_a is less than that of a frame collar beam i_{cd}.

$$i_a = \frac{I_a}{S} < \frac{I_{cb}}{L_{cb}} = i_{cb}.$$

This explains the use of arched members in large-span roofs where there are no dynamic or large horizontal loads, in buildings designed for use as pavilions, markets, warehouses, etc.

Arches are divided with respect to design into *fixed, two-hinged* and *three-hinged* ones. The expediency of employing one of these systems is determined by the bearings. We gives a diagram of the moments induced by a uniformly distributed load for each of these types of arches. The greatest moment is obtained in a three-hinged arch at one fourth of the span, therefore such arches are the heaviest ones. Being statically determinate, however, they are not sensitive to settlement of the supports and changes in temperature.

The depth of an arch section is selected for small spans within the limits of $\frac{h}{L} \cong \frac{1}{30}$ to $\frac{1}{40}$, and for large ones $\frac{h}{L} \cong \frac{1}{40}$ to $\frac{1}{60}$.

The section of an arch may be solid (for small spans) or latticed (for large ones) with a diagonal or, less frequently, with a triangular lattice.

Investigation of arches is commenced with determination of the loads, which include the weight of the roof, snow (over the whole span and over half of its length), wind loads and the weight of the members. All the loads are determined as prescribed by the Building Standards and Regulations. The weight of the arch can be found from the equation

$$g = \frac{G}{S} \quad \text{kN/m},$$

in which g = weight of one linear metre of arch; S = length of arch; G = total weight of arch, equal to

$$G = \frac{2}{3} r_a L B C_g \quad \text{kN},$$

where B = spacing of arches, m; C_g = weight factor equal to 0.02...0.04.

The forces acting along the axis of an arch are found by the formulas

$$M_x = M_b - F_h; \qquad yF_x = Q_b \sin\alpha + F_h \cos\alpha;$$

$$Q_x = Q_b \cos\alpha - F_h \sin\alpha.$$

Here F_h = thrust; y = ordinate of arch axis ($y_{\max} = r_a$); α = angle between tangent to arch axis and the horizontal; M_b and Q_b = beam moment and shear force obtained when considering the arch as a beam with a span of L (Fig. 7.5, *a*).

A two-hinged arch is a statically indeterminate system with one redundant quantity. The thrust F_h is taken as the unknown force X_l which is found by the equation

$$F_h = X_1 = -\frac{\Delta_{1e}}{\delta_{11}}. \qquad (7.3)$$

In constant-dimension arches with a gentle slope and with a uniformly distributed load $F_h = \frac{qL^2}{8r_a}$.

The forces in open-web arches with parallel chords can be determined in the same way with the following resolution of the forces along the elements of the section (Fig. 7.5, *b*):

the force in the top chord

$$F_{ch \cdot t} = -\frac{M_x}{h} - \frac{F_x}{2} \ ,$$

the force in the bottom chord

$$F_{ch \cdot b} = \frac{M_x}{h} - \frac{F_x}{2} \ ,$$

the force in the diagonals

$$F_d = Q_x \frac{\cos\alpha}{\cos\beta} \ ,$$

the force in vertical

$$F_v = Q_x \cos\alpha.$$

When investigating open-web arches with a relatively large section, equation (7.3) should be employed for calculating the displa-cements used in determining the thrust, namely

$$E\delta_{11} = \sum_{i=1}^{n} \frac{F_1^{\,2} L_i}{A_i} \,,$$

$$E\Delta_{1e} = \sum_{i=1}^{n} \frac{F_1 \, F_e}{A_i} L_i \;.$$

Here it will be sufficient to sum the forces only along the chords, neglecting the forces in the lattice elements owing to their slight influence on the value of the thrust.

The sections of the elements of arch chords and lattices are generally taken of two angles and are designed as in ordinary trusses. With large forces in the chords, the same as in heavy trusses, a change-over to double-plane, mainly *H*, sections becomes necessary.

To ensure stability of the compression elements of the chords in a direction perpendicular to the plane of the arch, it is necessary to install horizontal ties, and also purlins or braces between the arches, similar to the arrangement of ties in ordinary trusses.

It is desirable that the distance between the purlins (braces) should not exceed 16 to 20 chord widths. When checking a compression element of an arch chord for buckling its effective length in a vertical plane (in the plane of the arch) is taken equal to the length of the panel and in a plane perpendicular to that of the arch to the distance between the braced points.

It is also necessary to check the general stability of the arch in a vertical plane. Since the most probable form of the loss of general stability of an arch in the vertical plane is S-shaped buckling with an inflexion point of the axis near the middle of the arch length (Fig. 7.5, *c*), the critical force can be approximately determined from the formula of Euler–Yasinski. The effective length is taken equal to half the length of the arch *S* multipled by the length coefficient *k* taken from table 7.1, i. e.,

$$F_{cr} = \frac{\pi^2 EI_x}{\left(k\frac{S}{2}\right)^2}.$$

Here the moment of inertia of the arch section I_x is taken at one-fourth of the span length. The following relation must be observed

$$\frac{F_{cr}}{F} > 1.2 \text{ to } 1.3,$$

where F is the force induced by the design loads.

Table 7.1

Length coefficient k for arches

	r_a/L			
Type of arch	1/20	1/5	1/3	1/2.5
Three-hinged	1.2	1.2	1.2	0.3
Two-hinged	1	1.1	1.2	0.3
Fixed	0.7	0.75	0.8	0.85

The depth of an arch section is sometimes determined by checking its general stability. To make possible the arrangement of arch-type members on walls or columns of various structures, bowstring arches are employed, the tie bar taking the thrust.

Fig. 7.6 depicts the bowstring arch used in the roof of the Palace of Sports in Luzhniki, Moscow.

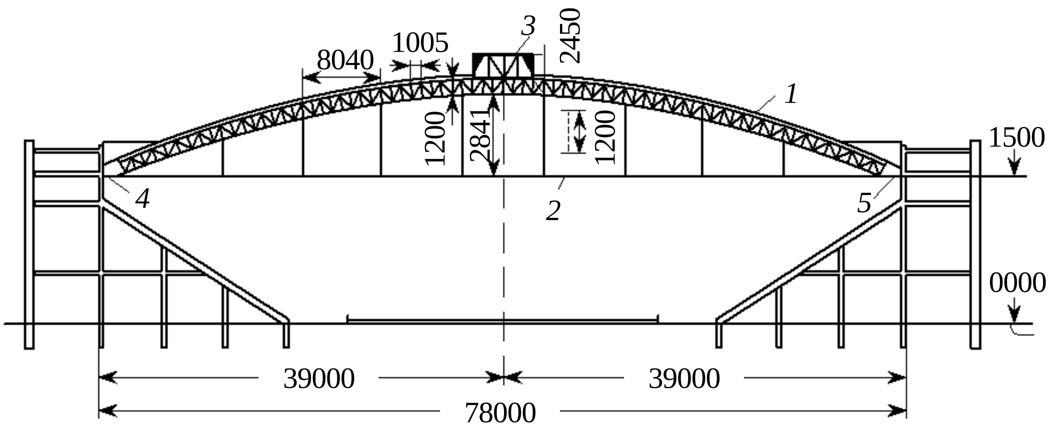


Fig. 7.6. Bowstring arch used in roof of Palace of Sports in Luzhniki (Moscow):
1 — arch; *2* — tie bar; *3* — aeration skylight;
4 — fixed support; *5* — pinned support

The thrust (force in the tie bar) is determined with account of the expansion of the tie bar (Fig. 7.7).

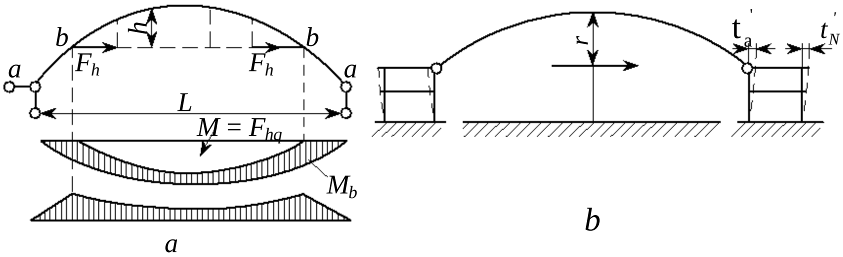


Fig. 7.7. Methods of increasing the inside clearance in arched structures

The height of the tie bar can be increased, arranging it above the supports and thus increasing the useful height of the premises. In this instance in parts *a*–*b* (Fig. 7.8, *a*) the arch functions in the same way as a simple beam subjected to bending, while the relieving influence of the force F_h in the tie covers only the part of the arch above the tie bar (with the ordinates *y*).

To increase the inside clearance of a structure and to employ an economical r_a/L ratio of about 1/5 to 1/6, arches can be located on auxiliary side structures (Fig. 7.8, *c*) or pilasters. Here the thrust is distributed according to equation (7.3), in whose denominator the displacement of the bearing member δ_{11} induced by a unit horizontal force should be added to the quantity δ_{11}.The bearings of arched members with large spans subjected to longitudinal forces of about 800 to 1,200 tons are generally made of steel castings with a cylindrical bush similar to bridge bearings (Fig. 7.8, *c* and 7.4, *b*). With lower forces, rocker type or tangential bearings are used (Fig. 7.8, *a* and 7.4, *a*). Such bearings can be investigated by means of the equation (7.2), taking $D = 2r$ (*r* is the radius of the bearer) and $N = 1$.

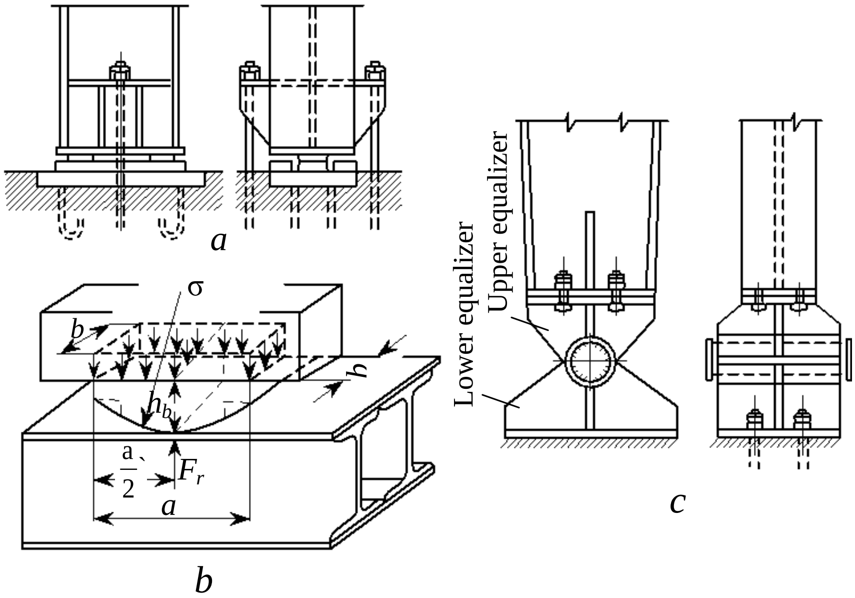


Fig. 7.8. Bearing of arched members

The required thickness of the bearer h_b is determined on the assumption of its bending along the section of contact with the bearing plate of the grillage and uniform distribution of the support reaction over the whole bearer (Fig. 7.8, *b*)

$$h_b \geq \sqrt{\frac{3F_r a}{2R_b b}},$$

where R_b = design bending strength of the bearer material, a and *b* = dimensions of the bearer (Fig. 7.8, *b*). The bearings should be anchored in the foundations.

7.2.2. Dome-Shaped Roofs

Dome-shaped roofs are very rational for use mainly in round buildings. Three main types of domes are distinguished, namely *ribbed, ribbed and ringed,* and *polygonal domes.*

Ribbed domes are in essence a system of two- or three-hinged bracing arches with bearings arranged along a circle (Fig. 7.9, *a*). Such systems can be considered as bar ones. In the intervals between the arches, purlins are installed for supporting the roof and the ties. The thrust of the arches F_h can be withstood either by the structure, which the dome is supported on, or by the bearing ring. In this instance the latter will serve as a sort of conditional tie bar for each arch.

Should the sectional area of a conditional tie bar A_{tb}, be so selected that its elastic deformations will equal those of a ring with axial-symmetrical loading induced by all the ribs, then an element of the dome can be analyzed as an arch located along the diameter with a conditional tie bar sectional area A_{tb}, and with the tributary area hatched in Fig. 7.9, *a*. If the bearing ring is designed in the form of a polygon located along the diameter with a conditional tie bar sectional area A_{tb}, and with the tributary area hatched in Fig. 7.9, *a*. If the bearing ring is designed in the form of a polygon (Fig. 7.9, *b*), then $A_{tb} = \frac{2LA_r}{L_r}\sin^2\frac{\varphi}{2}$, in which L_r = length of section of ring between ribs, φ = angle between ribs.

If the ring is a round one (Fig. 7.9, *c*), then $A_{tb} = \frac{2\pi A_r}{N}$, where N is the number of ribs (half-arches) in the dome.

The sectional area of the ring is found from the expression

$$A_r \geq \frac{F_r}{R},$$

in which F_r *is* the force in the bearing ring.

In a polygonal ring

$$F_r = \frac{F_h}{2\sin\frac{\varphi}{2}}$$

and in a circular one

$$F_r = \frac{F_h r}{L_r}.$$

The bearing ring is laid either on flat plates or slabs, or on tangential or roller bearings. Four longitudinally movable rollers are installed, the remaining bearings being spherical ones (Fig. 7.9, *a*).

The central ring at the apex of a dome formed by three-hinged arches resists mainly compression induced by the thrust forces F_h and should be checked for stability. It is essential that the stresses in it should not exceed the critical ones

$$\sigma \le \sigma_{cr} ,$$

where

$$\sigma = \frac{F_h r}{A_r L_r}, \qquad \sigma = \frac{3EI_r}{A_r r^2}$$

in which R = radius of central ring at apex; A_r = sectional area of this ring; L_r = distance between thrust forces applied to the ring at the apex of the dome; I_r = moment of inertia of ring section with respect to vertical axis.

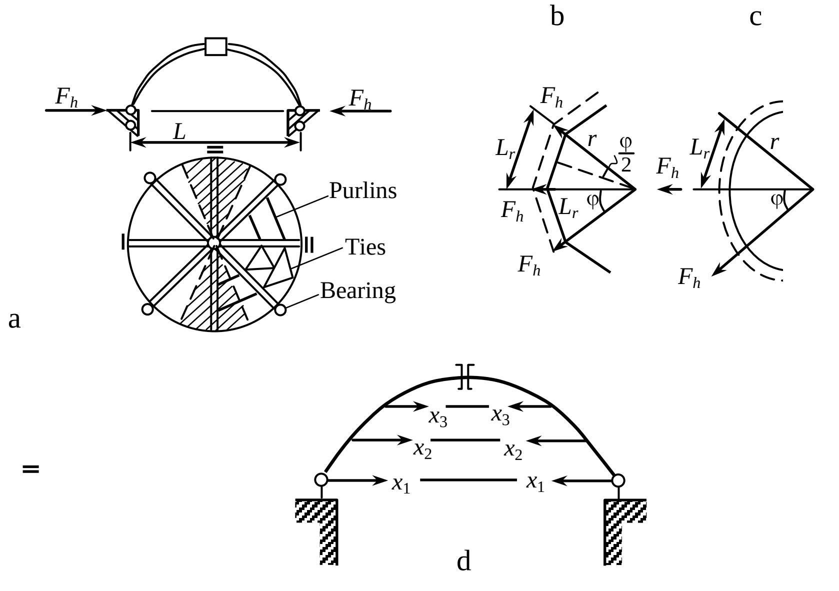


Fig. 7.9. Diagrammatic views of domes

With ribs in the form of two-hinged arches, the section of the central ring is subjected not only to compression, but also to bending, and therefore must be investigated for resistance to the total moment equal to the moment at the place of connection of the rib to the ring (Fig 7.10).

The section of the ring is designed either of the box type consisting of two channels or *I* shapes, or as an *H* shape made up of rolled elements.

Ribbed and Ringed Domes. If in a ribbed dome, which is in essence a bar system of arches connected only by means of a bearing ring, all the ring purlins are made to take the load, then we obtain a three-dimensional or space member with a number of rings serving as conditional tie bars for the separate arches. Such a dome is referred to as a ribbed and ringed one.

It is more rational and lighter than a ribbed dome, since almost all the elements are used to resist the loads. Fig. 7.10 depicts a ribbed and ringed dome with three rows of rings and, accordingly, with three redundant quantities X_1, X_2 and X_3. Each ring is selected with its own radius r and length L_r. Otherwise the investigations are similar to those used for a ribbed dome and are carried on as for an arched system with several conditional tie bars.

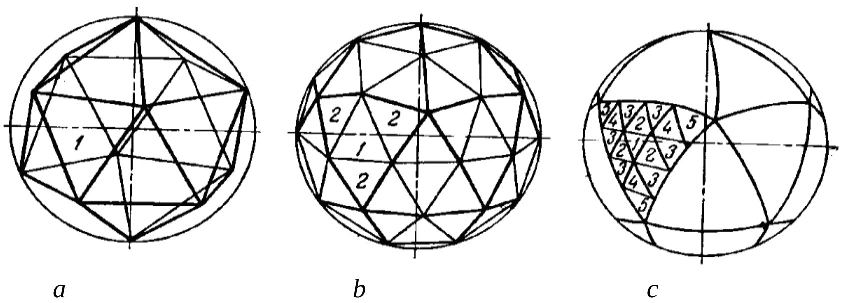


Fig. 7.10. Icosahedron and its development

Polygonal Domes. A polygonal dome may be designed in a number of ways. Thus, for example, a sphere may be divided by meridionally directed and annular ribs, and a diagonal installed in each of the rectangles obtained in this way. In such a design the lengths of the elements in each tier and the angles between them will be different, and this will result in large labour consumption for erecting the dome. To eliminate this great difference in element types, good practice dictates using the geometry of polyhedrons inscribed in a sphere. As is known from solid geometry, there are altogether five such polyhedrons, among which the one having the greatest number of identical faces (20) is the icosahedron (Fig. 7.10, *a*). When two different types of faces are used, an 80-face polyhedron is obtained (Fig. 7.10, *b*), and with five different types of faces-a 320-face one (Fig. 7.10, *c*), etc. This principle of construction has served as the basis for a large number of such geodetical domes.

A polygonal dome can be analyzed in the same way as a shell, by means of the membrane theory. The shell is assumed to be solid and axially symmetrical. The load of a dome generally consists of its weight (dead load), snow and wind loads.

In considering the equilibrium of a shell under a uniformly distributed load p kg/m^2 directed toward the centre, we shall obtain the basic equation for a spherical membrane shell

$$T_1 + T_2 = pR\,, \tag{7.4}$$

where T_1 = meridional force in shell, kN/m; T_3 = annular force, kN/m; R = radius of sphere.

Forces in Dome Induced by Dead Load of g kN/m^2. Let us denote by G the total weight of the part of the dome from its apex to the level y (or cut off by the radius R at an angle of φ as shown in Fig. 7.11, *a*, i.e.,

$$G = -g2nR(R - y).$$

This total weight will be balanced by the vertical reaction

$$T_1 2\pi r \sin\varphi = G\,.$$

Hence the meridional compressive force T_1 will be

$$T_1 = \frac{G}{2\pi r \sin\varphi} = -g\frac{R^2}{R+y} = -\frac{R}{1+\cos\varphi}\,.$$

The annular force T_2 is found from equation (7.4), substituting the pressure perpendicular to the surface of the shell with the load induced by the weight of the member's $p = -g\cos\varphi$:

$$-g\frac{R}{1+\cos\varphi} + T_2 = -g\cos\varphi R\,,$$

whence $$T_2 = -Rg\left(\cos\varphi - \frac{1}{1+\cos\varphi}\right),$$

or

$$T_2 = -Rg\left(\frac{y}{R} - \frac{1}{1+\dfrac{y}{R}}\right) = -g\frac{y^2 + yR - R^2}{y+R}\,.$$

Upon solving the equation $y^2 + yR - R^2 = 0$ we shall find the boundary line, where the annular force is equal to zero and a transmission from compression to tension takes place, namely,

$$y = 0.6187R \text{ and } \varphi = 51^{\circ}\,49'.$$

At the apex of the dome, with $y = R$, we have

$$T_1 = T_2 = g\frac{R}{2}$$

and for $y = 0$

$$T_1 = T_2 = gR\,.$$

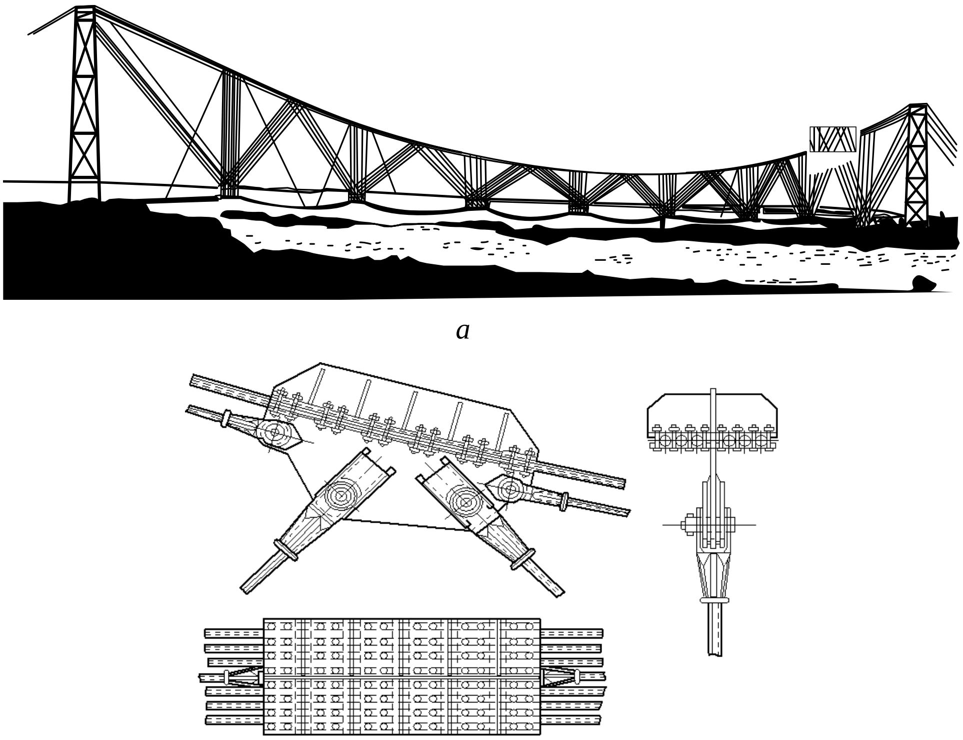
The corresponding force diarams are shown in Fig. 7.11, *a*.

Force in Dome Induced by Uniformly Distributed Load q, N/m². Similarly we obtain $G = -q\pi r^2$ and

$$T_1 = \frac{G}{2\pi r\sin\varphi} = -\frac{\pi r^2 q}{2\pi r\dfrac{r}{R}} = -\frac{qR}{2}\,.$$

By substituting in shell equation $p = -q\cos^{v}\varphi = -\dfrac{qy^2}{R^2}$, we find

$$T_2 = -\frac{qy^2}{R^2}R + \frac{qR}{2} = -\frac{q}{2R}\ 2y^2 - r^2\ = -\frac{qR}{2}\cos 2\varphi\,.$$



a

b

Fig. 7.11. Crossing of Volga River at Volgograd

Upon assuming that $2y^2 - R^2 = 0$, we find the value of $y = 0.707$ ($\varphi = 45$ deg) at which the annular force vanishes.

At the apex of the dome with $y = R$ we have

$$T_1 = T_2 = -\frac{qR}{2}$$

and for $y = 0$

$$T_1 = -\frac{qR}{2} \quad \text{and} \quad T_2 = +\frac{qR}{2}\,.$$

Forces in Dome Induced by Wind Load. The forces induced by the wind load are determined on the assumption that the pressure of the wind (normal to the surface) is equal

$$q_{\omega} = q_{\omega \cdot \upsilon}\sin\varphi\sin\theta\,,$$

in which $q_{\omega \cdot \upsilon}$ is the design wind load on a vertical plane.

For a dome in the form of the surface of a hemisphere, the meridional forces will be

$$T_1 = q_{\omega \cdot \upsilon} R \frac{\cos\varphi}{\sin^3\varphi}\left(\frac{2}{3} - \cos\varphi + \frac{1}{3}\cos^3\varphi\right)\sin\theta$$

and the annular forces will be

$$T_1 = q_{\omega \cdot \upsilon} R \left[\sin\varphi - \frac{\cos\varphi}{\sin^3\varphi}\left(\frac{2}{3} - \cos\varphi + \frac{1}{3}\cos^3\varphi\right)\right]\sin\theta\,.$$

For determining the forces in the bars of a dome, it is always possible to select a bar following a meridian which collects from a definite "force" area the force T_1. Thus, for instance, in the bar i_1, in section *1–1* is directed along the meridian. This bar collects the force from the hatched area having a width of a, i.e., the force in it is

$$F_1 = T_1 a\,.$$

In the same way the force in bar i_2, directed along the ring is determined. For this purpose we shall consider section *2–2* with a width of the "force" area equal to b. The force in the bar is

$$F_2 = T_2 b\,.$$

Since the forces T_1 and T_1, expressed in kN/m, can be determined at any point of the sphere, the forces in all the bars of the dome can be found.

Besides the axial forces, the bars, depending on the design of the roof, may be subjected to bending induced by a local load which must be taken into account when selecting their sections. It is also essential to ensure the required rigity of the bars in a vertical plane (to avoid the loss of stability) by meeting the condition that

$$I_d \ge \frac{T_1 R_a}{0.5E}\sqrt{\frac{T_1 R}{0.5E}}\,.$$

The elements of a dome are ordinarily made either with a tubular section, or of relatively small angle, channel or T shapes.

7.2.3. Suspension (Guy) Systems

In suspension (guy) systems the main bearing elements of the members are guys (cables, wire ropes).

Owing to the high strength of rope wire (from 120 to 210 kN/cm^2) and complete utilization of the sectional area of a rope in tension, suspension members have a light and economical design. It is advantageous to employ such structures where large spans are involved. The advantages of such structures also include their rapid erection.

The main shortcomings that lead to certain difficulties in solving the structural members are high deformability of suspension systems under the action of live loads and, in roofs, complications involved in draining away water.

Suspension systems are thrust ones. The members taking the thrust form a considerable part of the structure as regards the consumption of material. Guy systems may be divided into two types, namely, plane (guy-bar) and three-dimensional or space (in the form of membranes or polygonal members) systems.

The guy bridge trusses with a span of 874 metres used for the crossing of the Volga River at Volgograd (Fig. 7.11), designed of wire ropes according to a system proposed by engineers V. Vakhurkin and G. Popov in 1955, proved to be very rational. As the result of prestressing of the bottom chord, also made of ropes, all the truss elements were in tension. The roadway members were suspended at the joints. The compressive forces appearing in the elements of the truss under the action of the service loads do not exceed the prestressing forces, and thus, notwithstanding the presence of flexible elements, the truss behaves as a rigid system. Fig. 7.11, *b* pictures the design of a top truss chord joint.

Such members can be successfully employed for large-span roofs.

Space polygonal suspension members, depending on the external contour of the structure, may be of the barrel type (over a rectangular contour), spherical or in the form of other shells with a positive Gaussian curvature (covering an ellipsoidal contour, etc.), a negative Gaussian curvature, a double curvature (saddle-shaped), etc.

The design of barrel and round suspension shells can be considered from the viewpoint of analysis as a set of plane flexible strings. However, such strings are too deformable, and under the influence of uneven loading or gusts of wind and suction great displacements of the roof in different directions may appear. To eliminate these undesirable displacements, the practice is laying a flooring of reinforced concrete slabs on the ropes or cables, an additional load being applied during erection, after which all the joints are concreted in situ. After removal of the additional load and elastic rebounding of the ropes, the roof begins to function in the reverse direction as a shell and becomes sufficiently rigid. Shows diagrammatically such a roof with a span of 94 metres (stadium at Montevideo). Here the ropes are anchored in the external reinforced concrete compressed ring, while at the centre of the structure they are connected to a metal tensioned ring. Water is drained off along suspended pipes.

When light aluminium panels are employed, guy ropes must be installed.

Convex suspension roofs can be designed in the form of a two-chord system of ropes or cables. Several designs are possible, for example, in the form of a “bicycle wheel” with tensioned strings and a central drum (the USA pavilion at the Brussels World Fair, 1958, Fig. 7.12, *a*). Similar systems are possible without a central drum, with the installation between two ropes of compression posts (Fig. 7.12, *b*) or tension posts (Fig. 7.12). Semi-transparent plastics laid over the purlins can be used as the roof sheathing in such roofs.

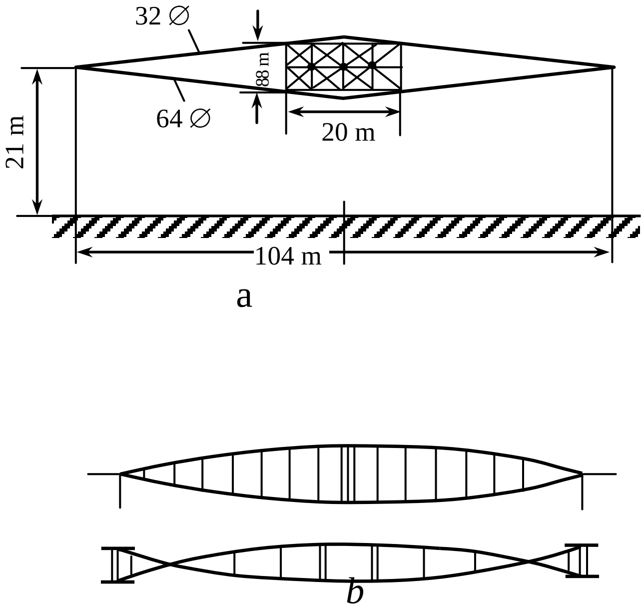


Fig. 7.12. Convex suspension roofs and roofs with tie rods

Polygonal roofs with a double curvature are a rational design of roofs. The bearing cables are arranged along a concave surface, while the tensioning ropes directed at right angles to them have a convexity facing upward. As a result, a saddle-shaped relatively rigid surface is obtained.

Cables and ropes are anchored in inclined arches whose weight facilitates their tensioning. The weight of the arches, however, is not sufficient, and the vertical external columns simultaneously serve as tie rods anchored in the foundation. It must be remembered that before installation the cables and ropes should be preliminarily tensioned with a force exceeding the design one by 20–25 %.

An approximate method can be used for investigating the strength of the ropes of roofs with a negative curvature supported on an elliptical contour if it is assumed that the loaded structure retains its parabolic configuration.

Let F_{h1}, and F_{h2} denote the forces in the bearing and the tensioning elements, b_1 and b_2 — the major and minor diameters of the ellipse; δ_1 — the deflection of the bearing rope; δ_2 — the deflection or rise of the tensioning rope; q — the intensity of the external uniformly distributed load (dead and live) in t/m, $q_1 = q_u b_{br}$ in t/m — the intensity of the live load, equal to the product of the load per square meter q_u and the distance between the bearing ropes b_{br}, q_{ps} — the intensity of prestressing transmitted from the tensioning ropes to the bearing ones.

The value of q_{ps} is found on the-assumption that the bearing ring will not take bending in a horizontal plane and that each tensioning rope transmits an identical load to the bearing rope, i.e., both ropes are under a uniformly distributed load and should therefore retain a parabolic configuration.

$$q_{ps} = \frac{q}{\frac{\delta_1}{\delta_2} - 1}.$$

The forces in the bearing rope and, accordingly, in the tensioning rope will be

$$F_{h_1} = \frac{(q + q_{ps})L_1^2}{8\delta_1} \quad \text{and} \quad F_{h_2} = \frac{q_{ps}L_2^2}{8\delta_2}$$

in which L_1 = length of bearing rope; L_2 = length of tensioning rope.

After the roof has been loaded with a live load q_1, the following force will remain in the tensioning rope

$$F_{h_2} = \frac{(q_{ps} - q_1)L_2^2}{8\delta_2}.$$

In these investigations the value of the modulus of elasticity of the rope (with a stiff core) is taken equal to $E \cong 16\times10^6$ t/m^2.

Ropes and cables should be well protected against the weather, for which purpose galvanized or plastic-coated ropes are used. The ropes are anchored by embedding them in steel sleeves with grade UAM9-1.5, UAM10-5 alloys, babbitt metal B-95, etc. The ropes are unbraided in the sleeve and the ends of the wires are bent down. It is also possible to wedge the ropes in a special casing. Below are considered the methods of analyzing flexible cable (ropes, guys).

Analysis of Flexible Non-Extensible Cable. Let us assume that we have a cable freely suspended in span *L* and loaded with a uniformly distributed load *q* (Fig. 7.13, *a*). Let us further assume that the length *S* of the cable has such a value that under the action of the load the deflections *y* (or their maximum value δ) are relatively small, within the limits of $\frac{\delta}{L} = \frac{1}{6}$ to $\frac{1}{20}$.

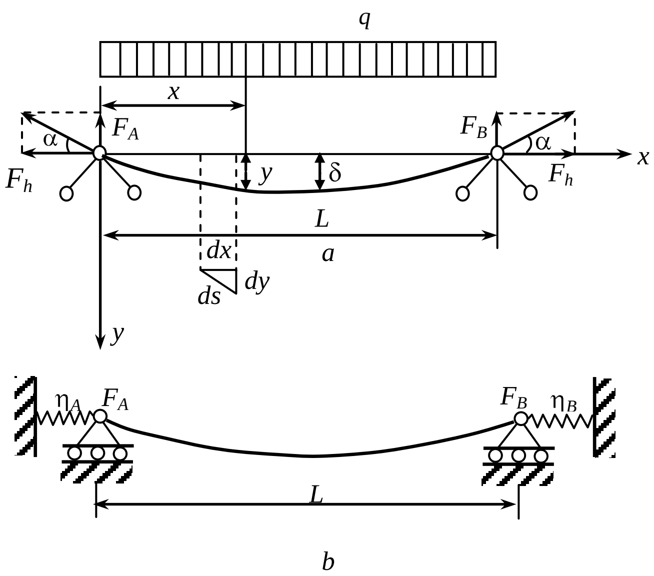


Fig. 7.13. To analysis of flexible cables

Then it can be assumed that the load is uniformly distributed over a horizontal line, while the equilibrium curve of the elastic line is a quadratic parabola.

The distinctive feature of flexible cable investigation consists in a different form of cable equilibrium corresponding to each new loading.

Seeing that in any section of the cable the moment is equal to zero, this condition can be written down for the section x whence.

$$M_x - F_h y = 0.$$

Here M_x = sum of moments of all the vertical forces acting on the left-hand part of the cable, which is equivalent to a bending moment in the section of a simple beam.

$$y = \frac{M_x}{F_h}.$$

Upon differentiating equation, we get

$$\frac{dy}{dx} = \frac{1}{F_h}\frac{dM_x}{dx} = \frac{Q_x}{F_h}, \tag{7.5}$$

where Q_x = shear force found in the same way as for a simple beam.

When Q and F_h are known, we can find the longitudinal force in a flexible cable

$$F_l = \sqrt{Q^2 + F_h^2}.$$

Thus, to determine the force in a cable, rope or guy it is necessary to know the thrust F_h, or the ordinate of the equilibrium curve y (or its maximum value-the deflection δ).

With a uniformly distributed load q we shall have

$$M_x = \frac{qL^2}{8}, \quad F_h = \frac{M}{\delta} = \frac{qL^2}{8\delta}, \quad \tan\alpha = \frac{Q}{F_h} = \frac{4\delta}{L}. \tag{7.6}$$

When determining the force in the cable, the length of the cable S, which is directly related to the deflection δ (Fig. 7.13, *a*), may be selected instead of the latter.

The length of the cable is determined from equation (7.6), in which relation (7.5) has been used and the radical has been replaced.

$$S=\int_0^L ds=\int_0^L\sqrt{1+\left(\frac{dy}{dx}\right)^2}\,dx\cong\int_0^L\left[1+\frac{1}{2}\left(\frac{Q_x}{F_h}\right)^2-\frac{1}{8}\left(\frac{Q_x}{F_h}\right)^4+\ldots\right]dx\cong$$

$$\cong L+\frac{1}{2F_h^2}\int_0^L Q_x^2\,dx\,.$$

The quantity $\int_0^L Q_x^2\,dx=D$ is a characteristic of the load.

Thus the magnitude of the thrust F_h may be expressed through the initial length of the cable (the pro-cut length) S

$$F_h=\sqrt{\frac{D}{2\;S-L}}\;.$$

A more precise value of the thrust can be obtained from one of the following formulas

$$F_h=\sqrt{\frac{LD}{S^2-L^2}}\quad\text{or}\quad F_h=\sqrt{\frac{3LD}{4\delta}}\,, \tag{7.7}$$

in which

$$D=\int_0^L Q_x\,dx\qquad\text{or}\qquad D=\int_0^L M_x q_x\,dx\,.$$

From formulas (7.7) it follows that

$$\sqrt{\frac{LD}{S^2-L^2}}=\sqrt{\frac{3LD}{4\delta}}\,,$$

whence we find the length of an arc for gentle curves (P. Chebyshev's formula)

$$S=\sqrt{L^2+\frac{16}{3}\left(\frac{\delta}{L}\right)^2}\cong L\left(1+\frac{8}{3}\frac{\delta^2}{L^2}\right).$$

Table 7.2 contains certain values of the quantity D calculated for different loads.

Analysis of Flexible Elastic Cable with Fixed Supports. If the deflection of the cable is less than $\frac{\delta}{L}=\frac{1}{r_0}<\frac{1}{20}$, then it should be investigated with account taken of elastic strains.

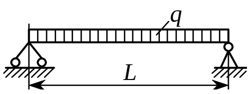
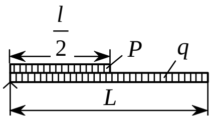
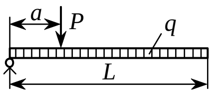
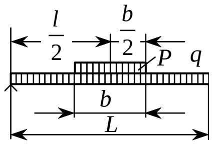
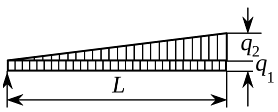
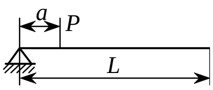
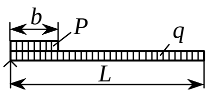
The thrust F_h is found from the equation

$$F_h^3 + \frac{8EA}{3r_0^2 r_s} H^2 = \frac{DEA}{2Lr_s^3} \ , \tag{7.8}$$

in which E = modulus of elasticity of the cable; A = cross-sectional area of the cable; $r_0 = \frac{L}{\delta}$ and $r_s = \frac{S}{L}$ quantities determined without account taken of elastic strains.

Table 7.2

Quantity D Characterizing Loads

Item №	Loading diagram	Value of D (in kg^2·m)	Notation
1		$\frac{q^2L^3}{12}$	–
2		$\frac{q^2L^3}{12}\left(1+\gamma+\frac{5}{16}\gamma^2\right)$	$\gamma = \frac{p}{q}$
3		$\frac{q^2L^3}{12}\left[1+(4-3\beta)\beta^3\gamma^2+(6-4\beta)\beta^2\gamma\right]$	$\beta = \frac{b}{L}$
4		$\frac{q^2L^3}{12}\left[1+(4-2\beta)\beta^2\gamma^2+\left(3-\beta^2\right)\beta\gamma\right]$	–
5		$\frac{q^2L^3}{12}+\frac{q_1q_2L^3}{12}+\frac{q_2^2L^3}{45}$	–
6		$\frac{p^2(L-a)a}{L}$	–
7		$\frac{q^2L^3}{12}\left[2\alpha\gamma_1(1-\alpha)+(1+\gamma_1)+1\right]$	$\gamma_1 = \frac{P}{qL}$ $\alpha = \frac{a}{L}$

Analysis of Flexible Elastic Cable with Yielding Supports. Let us assume that the supports have a yielding of η_A and η_B (measured in meters), which is characterized by their displacement under the action of a force equal to 1 ton.

The thrust F_{h1} in this instance is determined from the equation

$$F_{h_1}^4\left(\frac{4Lr_s^3}{EAF_h}+\frac{D}{F_h^4}\right)-F_{h_1}^3\left(\frac{4Lr_s^3}{EA}+4\eta\right)+D=0\,,$$

where F_h = thrust found from equation (7.8) $\eta = \eta_A + \eta_B$

Analysis of Flexible Elastic Cable Prestressed with Force F_{ps}. Let us assume that we have a cable whose length is somewhat less than the span L ($S < L$), the cable being loaded with a vertical load and subjected to prestressing with a force F_{ps}.

If we denote by F_h the total thrust appearing in the cable under the simultaneous action of the prestressing caused by the force F_{ps}, and the vertical load (i. e., the force F_{ps} will form part of the support reaction), then F_h will be found from the equation (7.8)

$$F_h^3-F_{ps}F_h^2=\frac{DEA}{2L}\,.$$

In a particular case, when the length of the cable is equal to that of the span and there is no prestressing, i.e., $S = L$, r_s =1 and $r_0 = \infty$ (analysis of a string), we have

$$F_h=\sqrt[3]{\frac{DEA}{2L}}=\sqrt[3]{\frac{q^2L^2EA}{24}}\,.$$

Methodical instructions to Chapter 7

In this unit special attention should be paid to designing special members.

It`s important to know how to design heavy trusses, as well as arched, dome-shaped and suspended roofs.

To design special constructions it is necessary to study the types of heavy trusses and their connection units, the methods of analyzing and rules of arched, dome-shaped and suspended roofs construction.

While designing a roof for large-span buildings it is necessary to pay attention to selecting cross-section of heavy trusses arched, dome-shaped and suspended special members.

Questions and Tasks to Chapter 7

1. *Give the definition of heavy trusses.*
2. *What is the distinguishing feature of large-span structures?*
3. *Give the definition of arches and their classification.*
4. *What indexes are influencing the weight of an arch?*
5. *Describe the classification of a dome-shaped roof.*
6. *How can we determine forces in the dome induced by dead and uniformly distributed loads?*
7. *What are suspension systems and peculiarities of their analysis?*

Appendix A

TERMS, DEFINITIONS, SYMBOLS

Terms and their definitions

Safety is the ability of construction object to confine consequences of its normal operation or emergency to tolerated risk level which is safe for human life activity, functioning of economy, social or nature environment.

Viscous failure is the type of failure accompanied with slow formation of plastic deformations.

Geometrical nonlinearity represents nonlinear or piecewise-linear dependence between deformations and displacements.

Flexibility of rod is relation of its design length to its cross-section radius of inertia.

Flexibility of plate (web, flange) is relation of the plate (depth web, width flange) to the thickness.

Deformed scheme is the design model, whose equilibrium equation takes into account the system displacements after its initial loading and change of these loads location as a result of the system deformation.

Deplanation of cross-section is such a displacement of a plane cross-section point which tranfors the plane into a surface or set of planes.

Durability is the ability of an object to maintain its operable state during long time in accordance with the predetermined system of its maintenance and specified procedure of repairs to carry out.

Survivability is the ability of an object to save its operable state (possibly with lowering its operation quality) even when its part is damaged.

Ideal system is a system without initial imperfections when critical load is applied in such a manner that the system loses its stability (buckling) and occurrence of qualitatively new displacements (bifurcation) is possible.

Qualitative change of configuration is such a state of an object when termination of its operation is necessary owing to excessive residual displacements including shearing in joints.

Constructive nonlinearity is a change of design model during its loading process.

Test assembling is assembling of shipping units in order to test geometrical parameters of the structure and competibility of its preassembly enlarging and erection joints.

Reliability is the ability of an object to perform specified functions during required time period.

Initial imperfections represent some set of unfavorable factors (deviation of a shape and dimensions from the nominal ones, departure from the design model, own initial stresses, etc) which may occur during production, transportation, and erection of a structure, lowering in such a manner its load-bearing capacity.

Load-bearing capacity is the ability of a structure or its separate elements to resist some type and level of loads and efforts.

General assembling is the assembling of shipping units of a structure or its part in order to test their assembling ability and designed geometrical parameters.

Interaction surface represents some surface of stresses or forces, whose points characterize the boundary or critical state of an element or system cross-section.

Boundary state represents some state of a structure or its separate part such as foundation when they don’t meet the specified requirements.

Reduced flexibility of through rod represents flexibility of a perfectly straight elastic rod with absolutely rigid connection elements when the value of its buckling load is equal to the buckling load of specified through rod with compliant connection elements.

Reduced stress represents such a value of stress under simple stretching or compressing which causes the same dangerous state of material as it is under its complicated stress state.

Design length is some conditional length of the rod, critical force of which under its hinged supporting is the same as for examined rod (CT C3B 3972).

Free torsion is such torsion of a thin wall rod when all its cross-sections are characterized by their equal warping and presence of shearing stresses only.

Complicated stress state is such a stress state when not less than two components of stresses occur in the examined points of cross-section (CT C3B 3972).

Fatigue fracture is such a type of fracture which is accompanied with formation and development of cracks owing to multiple repeated action of loads and efforts

Physical nonlinearity represents nonlinear or piecewise-linear dependence between stresses and strains caused by physical properties of materials used in the structure.

Brittle fracture is as a rule, sudden fracture, accompanied with the formation of small deformations due to the presence of stress concentrators, low temperatures, and impact efforts (actions).

Loading cycle represents a cycle of loading (stressing) caused by nonstatic load that is one-time change of load (stress) corresponding to its full period of chage.

Design history of load — (histogram) represents the appropriate record of load fluctuation versus time. It describes changes of stresses from maximum to minimum depending on load action.

Indexes in letter symbols and explanatory words

a —anchor;

a — axial;

b — beam;

b —bolt;

c — compression;

c —column;

c —chord of column;

c —condition;

d —design;

d —diagonal;

e —eccentricity;

h — highstrength;

I — flang;

F —force;

f —friction;

f —fillet weld;

i —inferior;

l —longitudinal;

m — middle;

m — moment;

m — material;

n —character;

n — net;

p — pressure;

r — rivet;

r — rib; .

s —shear;

s —super;

t —tension;

u —ultimate strength;

v — vibration;

w —web;

w — welding;

y —yield point;

z — zone;

ad — additional;

cr — critical;

fic — fictitious;

loc— local;

max — maximum;

min — minimum;

rel — relative;

tot — total;

Main symbols

A — gross area of cross-section;

A_a — design cross-section taking into account friction forces;

A_{bn} — net area of bolt cross-section;

Ad — cross-section area of diagonal;

A_f — flange cross-section area;

A_n — net cross-section area;

A_w — web cross-section area;

A_{wf}—cross-section area of fillet weld in the plane of welded metal;

A_w—cross-section area of fillet weld in the plane of welding border;

E — modulus of steel elasticity;

F — force;

G — shear modulus;

I — moment of inertia of a member cross-section; gross moment of inertia of cross-section;

I_b — moment of inertia of cross-section;

I_m, I_d — moment of inertia of flange and diagonals of truss;

I_r — cross-section moment of inertia of rib, strap;

I_{rl} —cross-section moment of inertia of longitudinal rib;

I_t —moment of inertia under free rotation;

I_x, I_y —gross cross-section moment of inertia with respect to axes $x–x$ and $y–y$;

I_{xn}, I_{yn}—net cross-section moment of inertia with respect to axes $x–x$ and $y–y$;

I_w —sectorial moment of inertia of cross-section;

M —bending moment;

M_x, M_y — bending moment with respect to axes $x–x$ and $y–y$;

N —longitudinal force;

n_{ad} —additional force;

N_{bm} —longitudinal force caused by bending moment acting in the column leg;

Q —shearing force;

Q_{fic} —conditional shearing force for joining elements;

Q_c—conditional shearing force of the set of straps located in the single plane;

R_{ba} — design tensile strength of foundation bolts;

R_{bt} —design tensile strength of high-strength bolts;

R_{bp}— design crumple strength of one-bolted joint;

R_{bs} — design shearing strength of one-bolted joint;

R_{bt}— design tensile strength of one-bolted joint;

R_{bun} —normal strength of bolt steel which is assumed equal to temporary strength of bolt steel in accordance with the state standards and technical specifications of bolts;

R_b — design tensile strength of *U*-bolts;

R_{byn} —normal strength of bolt steel which is assumed equal to the flow limit in accordance with the state standards and technical specifications of bolts;

R_{cd} —design strength on diametrical compression of rollers during their free contact with structures of limited mobility;

R_{dh} —design tensile strength of high-strength wire;

R_{lp} —design strength on local crumple in cylinder hinges under tight contact;

R_p—design strength of steel on crumple of frontal surface (when fitting was done);

R_s—design shearing strength of steel;

R_{th}— design tensile strength of steel in the direction of rolled steel thickness;

R_u—design tensile and bending strength of steel by the temporary strength;

R_{un}—temporary strength of steel which is assumed equal to its minimal value in accordance with the state standards and technical specifications of steel;

R_v — design fatigue strength of steel;

R_{wf}—design shearing strength of fillet welds in the plane of welded metal;

R_{wu}—design compression, tensile and bending strength of butt welds by the temporary strength;

R_{wun} — characteristic strength of welds by the temporary strength;

R_{ws}—design shearing strength of butt welds;

R_{wy}—design compression, tensile and shearing strength of butt welds outside the flow limit;

R_{wz}—design shearing strength of fillet welds in the plane of welding border;

R_y — design tensile, compression and bending strength of steel outside the flow limit;

R_{yf}—design tensile, compression and bending strength of flange outside the flow limit;

R_{yw}— те саме для стінки; design tensile, compression and bending or strength of web outside the flow limit;

R_{yn}—flow limit of steel which is assumed equal to flow limit in accordance with the state standards and technical specifications of steel;

S —statical moment of shearing part of cross-section (gross) with respect to its central axis;

W_x, W_y — modulus of section (gross) with respect to $x–x$ and $y–y$ axes;

W_c, W_t —modulus of section calculated for its most tensile and compressive fiber;

W_{xn}, W_{yn} — modulus of section (net) with respect to $x–x$ and $y–y$ axes;

b —width;

b_{ef}—design width;

b_f —width of flange;

b_r —width of rib projecture;

c_x, c_y— coefficients for calculation taking into account the development of plastic deformations under bending with respect to $x–x$ and $y–y$ axes;

d —hole diameter for bolt;

d_b—outer diameter of bolt rod;

e —eccentricity of force;

h —depth;

h_{ef}—design depth web;

h_w—depth of web;

I —radius of inertia of section;

$i_{\min}$—the smallest radius of inertia of cross-section;

i_x, i_y — radius of inertia of cross-section with respect to $x–x$ and $y–y$ axes;

k —leg of fillet weld;

l —length, span;

l_c —length of upright, colomn;

l_d — length of diagonals;

l_{ef}—design length;

l_m—length of panel of chords of truss or colomn;

l_s — length of strip;

l_w —design length of a weld;

l_x, l_y —design lengths of element in the plane perpendicular to $x–x$ and $y–y$ axes;

$m = cA /W_c$ — relative eccentricity;

R —radius;

t — thickness;

t_f — thickness of flange;

t_w —thickness of wall;

$a_f = A_f / A_w$ —cross-section area relation of flange and wall;

$в_f, в_z$ —conversion factor of fillet weld leg in the plane of welded metal into design width of its cross-section in the plane of welding border;

γ_b —coefficient of bolted joint work operation conditions;

γ_c —coefficient of work operation conditions;

γ_f —reliability coefficient by the load;

γ_n — reliability coefficient by the application;

γ_m—reliability coefficient by the material applied;

γ_u — reliability coefficient in calculations by the temporary strength;

η —coefficient of cross-section form effiency;

γ — flexibility ($\gamma = I_{ef} / i$);

$\bar{\lambda}$ — conditional flexibility $\bar{\lambda} = R_y / E$;

γ_{ef}—reduced flexibility of through cross—section rod;

$\bar{\lambda}_{ef}$ —conditional reduced flexibility of through cross-section rod;

$$\bar{\lambda}_{ef} = \lambda_{ef} \sqrt{R_y / E} ;$$

$\bar{\lambda}_f$ —conditional flexibility of flange out ($\bar{\lambda}_f = b_{ef} / t_f \; R_y / E$);

$\bar{\lambda}_{f,1}$— conditional flexibility of flange plate;

$\bar{\lambda}_w$— fictitious flexibility of web;

$\bar{\lambda}_{uf}$ —boundary conditional flexibility of flange plate;

λ_{uw}— boundary conditional flexibility of web;

λ_x, λ_y— design flexibility of element in the planes perpendicular to axis *x–x and y–y* respectively;

v —coefficient of lateral deformation of steel (Poisson’s ratio);

σ_{loc} — місцеві нормальні напруження; local normal stresses;

σ_x, σ_y — normal stresses parallel to axis *x–x* і *y–y* respectively;

τ — shearing stress;

φ, φ_x, φ_y— stability factor under central compression;

φ_b— stability factor under bending;

φ_e—stability factor under bending with compression;

φ_{exy}—stability factor under bending with compression in two principal planes.

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