Saimaa University of Applied Sciences
Faculty of Technology, Lappeenranta
Double Degree Programme in Civil and Construction Engineering
Civil Engineering

Inna Bogun

Analysis of the steel structures connections in Finland and in Russia

Bachelor’s Thesis 2016

Abstract

Inna Bogun
Analysis of the steel structures connections in Finland and in Russia
47 pages
Saimaa University of Applied Sciences
Faculty of Technology, Lappeenranta
Double Degree Programme in Civil and Construction Engineering
Civil Engineering
Bachelor's Thesis 2016
Instructors: Petri Himmi, Saimaa University of Applied Sciences

The purpose of the thesis is to research modern joint connections of steel structures in Finland and in Russia. Thesis includes the review of historical development of steel structures and present types of connections in both countries and focuses on principles of their work.

The main methods of joint connection design according to Eurocode and SP (Russian norms) were considered.

Keywords: connection, steel structures, beam, column, welding connection, flanged joint, bolted joint

Table of contents

1 Introduction

The concept of "Metal structures" include their structural form, fabrication and erection processes technology. On the one hand, the level of development of metal structures is determined by the requirements for their national economy, and on the other - by the possibility of a technical basis for the development of metallurgy, metal-working, construction science and technology.

When choosing constructive solutions connections (joints and connections) must be consider factors such as the action of forces in the connected sections; section with the lowest resistance in the joints; shear strength of beams walls, chords and concluded between the part of the joint; eccentricity; concentration stress; deformability of the joint parts during manufacturing and operation.

Metal structures are widely used in the construction of various buildings and structures, because the quality that they acquire in the manufacturing process directly affects the longevity and reliability of the final product.

Installation of metal structures - is a very important and crucial step in the construction of buildings. And combination of these components is the most time-consuming process, specifically selects for each design, depending on its characteristics.

2 The history of metal construction

2.1 The first metal structures

The metal used for a long time from the XII century, in the unique for that time buildings (palaces, churches, and so on) in the form of fasteners and braces for masonry. The first such structure are fasteners of the Assumption Cathedral in Vladimir (1158). Pokrovsky Cathedral in Moscow - the first construction consisting of rods, the tensile, bending and compression. There fastening supporting floor and ceiling, strengthened to facilitate the work of bending braces. At that time designers already knew that for fastener working on a bend, it is necessary to apply the strip supplied to the edge and for the struts working in compression, it is better to do a square cross-section (Figure 1).

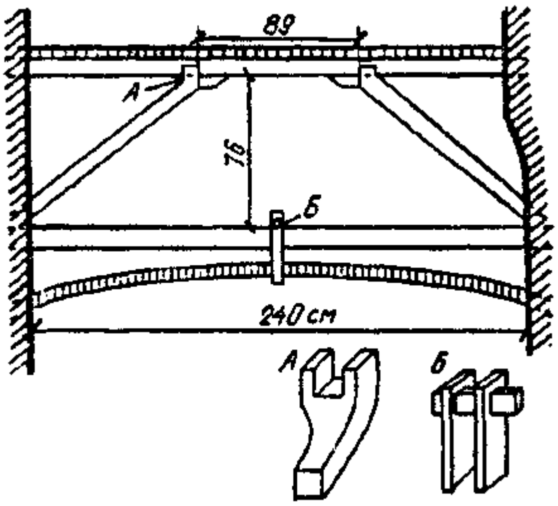

Figure 1. Overlapping corridor in Pokrovsky Cathedral (Moscow, 1560)

Since the beginning of the XVII century the metal used in spatial dome structures of the church leaders. The rods are made of forged constructions and bars connected to the castle and the mountain buckles welding. Such designs can be seen today in the refectory of the Trinity-Sergius Lavra in Sergiev Posad (1696-1698), the building of the Grand Kremlin Palace in Moscow (1640), the bell tower dome framework of Ivan the Great (1603), the framework of the Kazan Cathedral dome in St. Petersburg with a span of 15 m (1805) and others (1).

2.2 Cast iron construction

With the beginning of the XVIII began to master the founding process of cast iron rods and parts. Bridges are constructed of cast iron. Connections are made of cast-iron elements on the locks and bolts.

The first cast-iron construction in Russia is considered to cover the porch of the Nevyansk tower in the Ural (1725). In 1784 the first iron bridge is built in St. Petersburg. Unique cast iron design on the 40-s of the XIX century is the dome of St. Isaac's Cathedral, assembled from the individual stocks in the form of a continuous membrane (Figure 2).

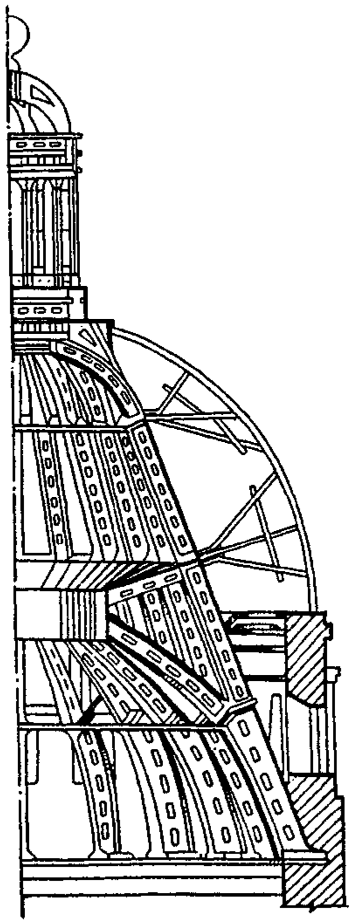

Figure 2. The dome of St. Isaac's Cathedral

The cast-iron arch span of 30 meters was used in the ceiling of the Alexandrinsky Theatre in St. Petersburg (1827-1832).

In the 50-s years of the XIX century in St. Petersburg the Nicholas bridge with eight arched spans from 33 to 47 m was built, it is the largest cast-iron bridge in the world (1).

2.3 The metal profile and sheet rolling technology

From the 30-s of the XIX century to the 20-s of the XX century - is rapid technological advances in metallurgy and metalworking, the riveted joints are appeared. In the 40-s of the XIX century is mastered the process of obtaining the metal profile and sheet rolling. Steel is almost completely displaced the cast iron building structures. All steel construction for the next hundred years performed riveted.

Cast iron structure in the second half of the XIX century were only used in the columns of high-rise buildings, railway stations ceilings landing stages, and so on. That is, where the resistance to compression of cast iron are better than steel.

By the end of the XIX century in Russian industrial and civil buildings were built mainly with brick walls and small spans, used to overlap the triangular metal farms (Figure 3). First, the farms did not have diagonal braces, they appeared at the end of the period under review.

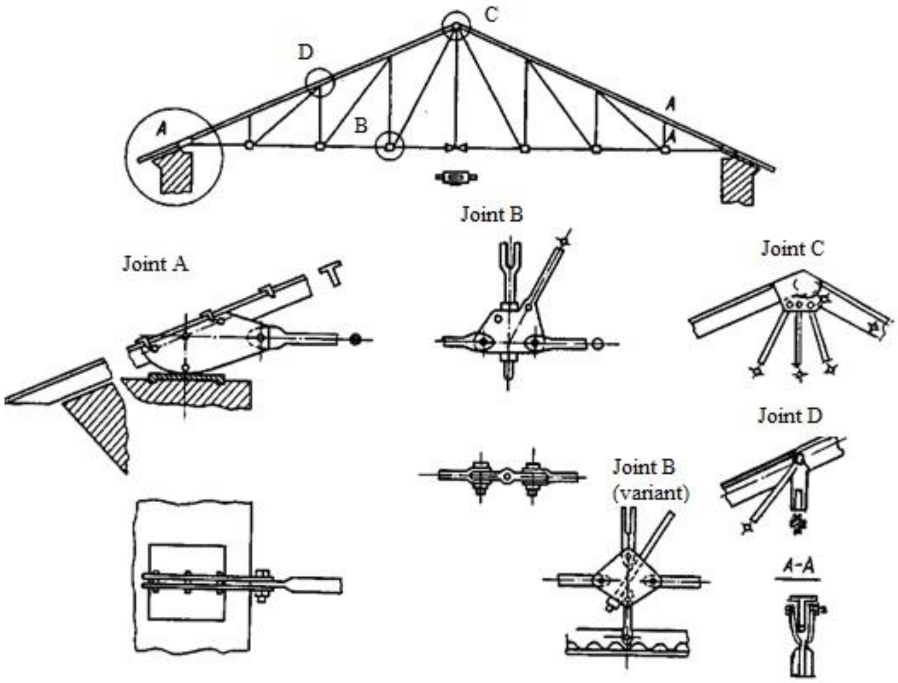

Figure 3. Trussed rafters (70s of the XIX century)

In the second half of the XIX century, the metal bridge building has received a significant development, where they began to apply the trusses with a triangular lattice truss, the metal assortment of rolled sections appeared.

In the beginning of the XX century industrial buildings were built with a metal frame that supports both the building envelope and the ways of overhead cranes. Bearing element of the frame was the transverse frame consisting of columns and beams (roof trusses). All steel structures are manufactured mostly riveted. Steel began to oust cast iron. By the end of the century form of farms being improved, there are diagonals, the connections instead of bolted on eyelets began to perform riveted with the gussets (1).

2.4 Applying of the lattice frame and arch designs

At the end of the XIX century the lattice frame and arch designs for buildings of significant overlap spans began to apply. For example, the Kiev railway station in Moscow on the project of Shukhov (1913–1914) (Figure 4). Metal bridge construction is developed (a bridge with lattice trusses through the Luga River, 1853). Professor L. D. Proskuryanov introduced in the bridge trusses triangular and truss bar lattice (across the Yenisei River Bridge).

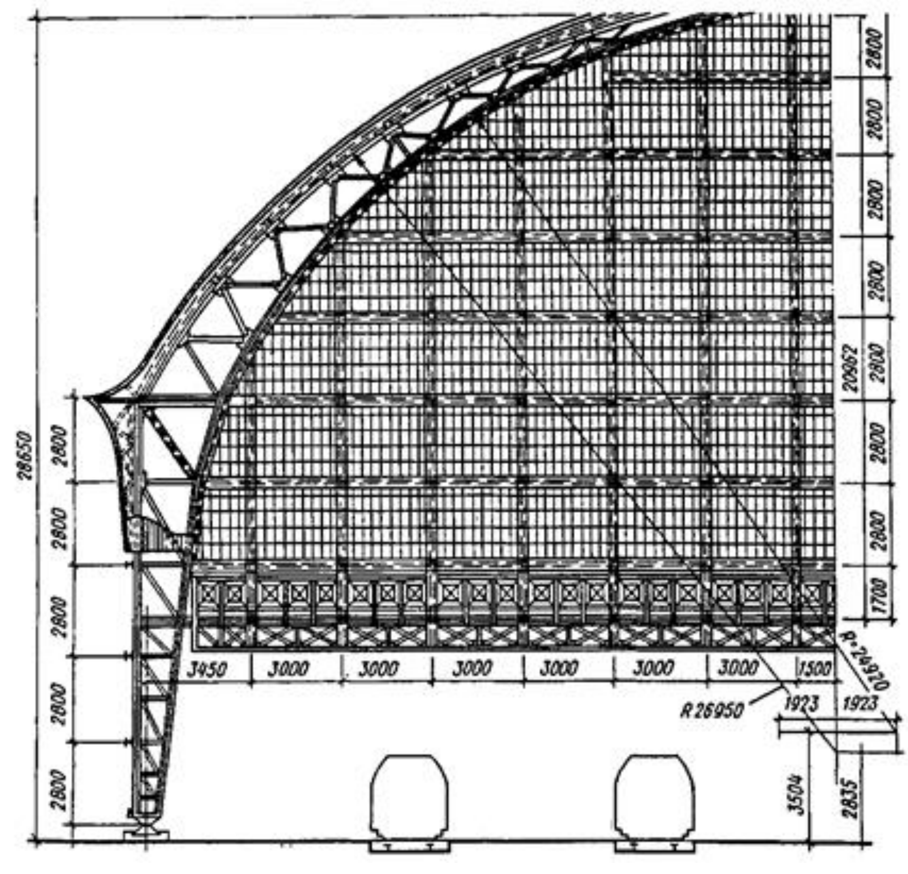

Figure 4. Overlapping of the Kiev railway station in Moscow

Further development of metallurgy, mechanical engineering and other industries requiring equipment of buildings with overhead cranes. First, they installed on the platforms, but with the increase in carrying capacity it was better to build a building with a metal frame that supports the ways of overhead cranes. Transverse frame became the main carrier element frame (Figure 5).

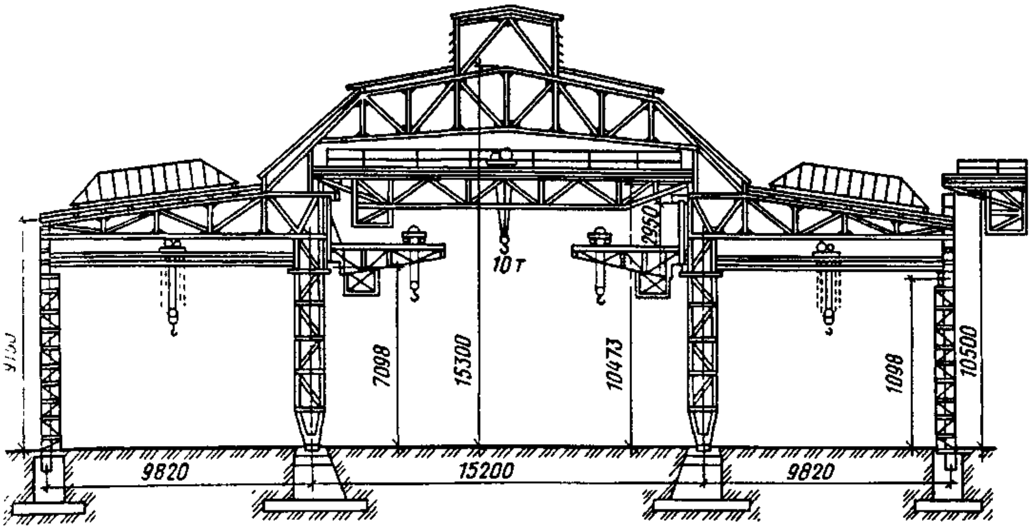

Figure 5. Industrial building frame (the beginning of the XX century)

Professor F. S. Yasinsky first projected multi-span industrial building. Academician V. G. Shukhov first in the world designed and built spatial lattice coatings structures and towers for various purposes (Television Tower, Figure 6).

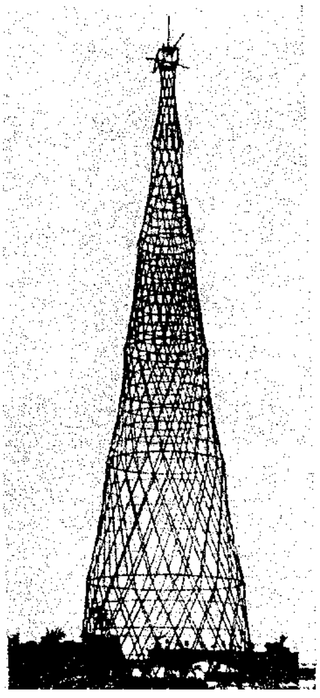

Figure 6. Shukhov Tower in Moscow

The structures he built sold the idea of pre-stressing structures and the construction of a suspension coating systems. Thus, he foresaw the future directions in the development of metal structures (1). Its significant work in the area of the reservoir structures, he developed new forms of reservoirs, their calculation and methods for finding optimal parameters (Figure 7).

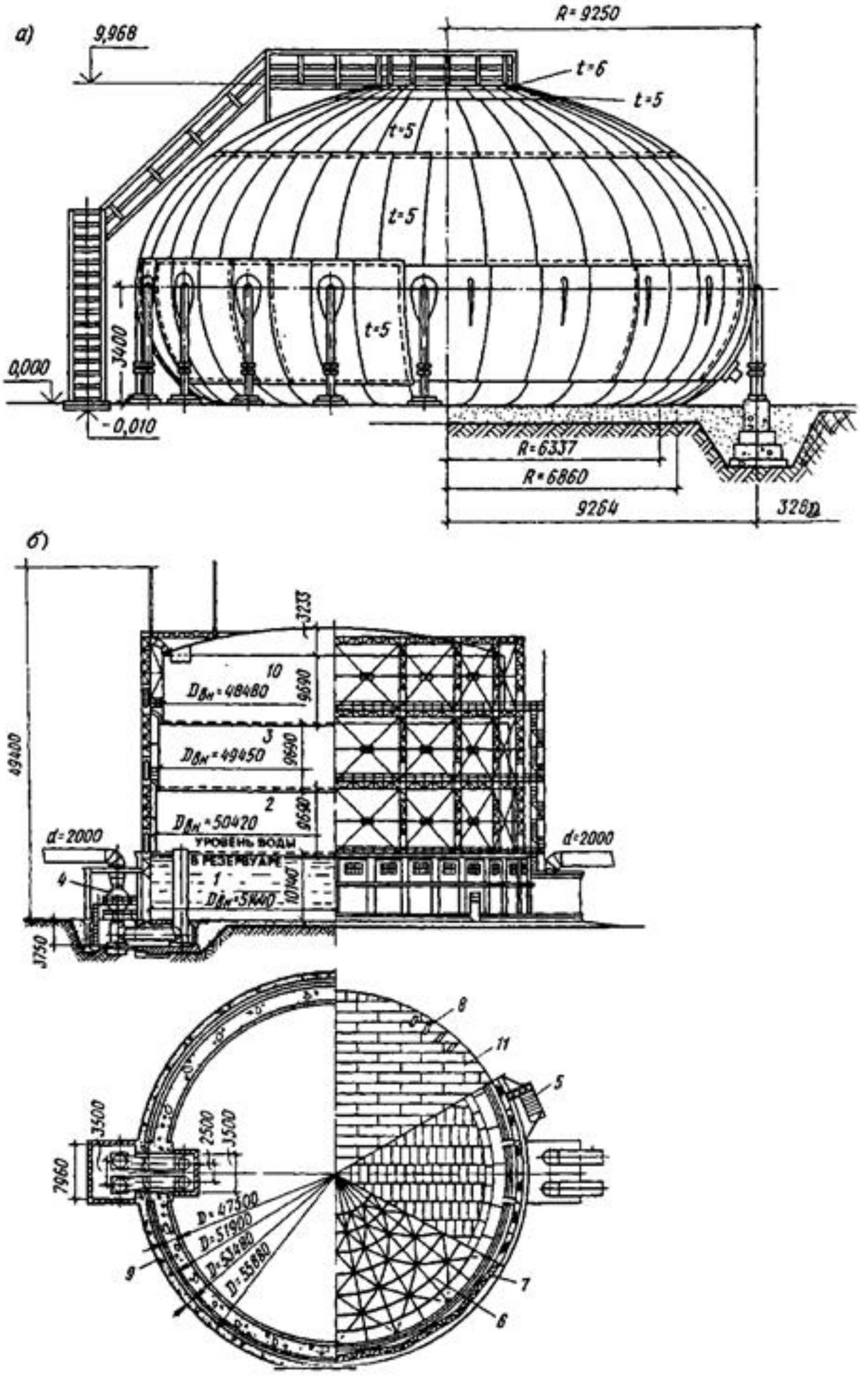

Figure 7. Sheet structures: a) guttate tank; b) the wet gasholder

2.5 Typification, unification and standardization

By the end of 40-s of the XX century riveted construction is almost entirely replaced by welded, more economical. There are low-alloy and high-strength steel. Besides steel, the aluminum alloys have begun to use, the density of which is almost three times less.

It expands the range of metal structures. Large and diverse tasks for the development of metal structures solved the efforts of design, research and production teams – “Proektstalkonstruktsiya” (ProjectSteelStructures), “Promstroiproekt” (IndustrialConstructionProject) and “TSNIPS” (CSRIIB - Central Scientific and Research Institute of Industrial Buildings), renamed later in “CNIISK” (CSRIBS - Central Scientific and Research Institute of Building Structures) and high school teams.

In this regard, first three-meter module bays introduced, which in the 50-s was replaced by a six-meter. Typification also cover the spans of bridges, reservoirs, gasholders, radio tower, radio mast.

Typification, unification and standardization - one of the main directions of metal structures development. This reduces the complexity of manufacture and assembly of structures, reduced steel consumption. From public buildings, it is possible to select the Cosmos pavilion at the All-Russian Exhibition Center (Moscow), overlap of the Palace of Sport in Luzhniki, unique large-span structures with metal supporting structure, built in Moscow for the Olympics-80.

Along with the improvement of structures forms and methods of calculation were developed. Until 1950 the calculation was based on the method of allowable stresses. This calculation is not fully reflected the actual construction work under load, resulting in a metal-expenditure, therefore was designed limit state method. There are computers that allows the designer to quickly find the optimum design solutions.

Success in the development of metal structures is obliged to Professor N. S. Streletskiy, that 50 years headed the metal construction school. He was one of the initiators of the transition from the calculation of allowable stresses for the calculation of the ultimate limit state. In the field of electric welding large contribution made by Professor E. O. Paton (1).

3 Elements of metal structures

Metal structures are used in all major engineering structures with the significant spans, heights and loads. Depending on the structural form and function metal structures can be divided into eight types:

1) Industrial buildings - all-metal or mixed frame (reinforced concrete columns). Full metal in buildings with large span, height and lifting capacity.
2) The large span covering of the buildings - sports facilities, markets, exhibition halls, theaters, hangars, etc. (spans up to 100-150 m).
3) Bridges, overpasses - bridges on railway and highways.
4) Sheet design - tanks, gasholders, bunkers, large diameter pipelines, and others.
5) The tower and mast - radio and television in the geodesic service, supports of the power transmission lines, oil rigs and other.
6) Frameworks of multi-storey buildings. They are used in multi-storey buildings in a dense housing in big cities.
7) Crane and other mobile structures - bridges, towers, cranes, excavators and other construction.
8) Additional designs for the use of nuclear energy for peaceful purposes, various designs of radio telescopes for space and radio communication platform for the exploration and production of oil and gas in the sea, etc.

4 Classification of steel structures connections

In the metal structures, different types of connections are used. In general, the choice of a particular type of connection depends primarily on existing efforts and constructive scheme of the junction elements.

In addition, the connection determined by the types of sections of elements that converge in it, the technological and economic needs, as well as their architectural expressiveness.

In terms of constructive work basically two types of connections are distinguished - rigid and hinged (Figure 8, 9) (2).

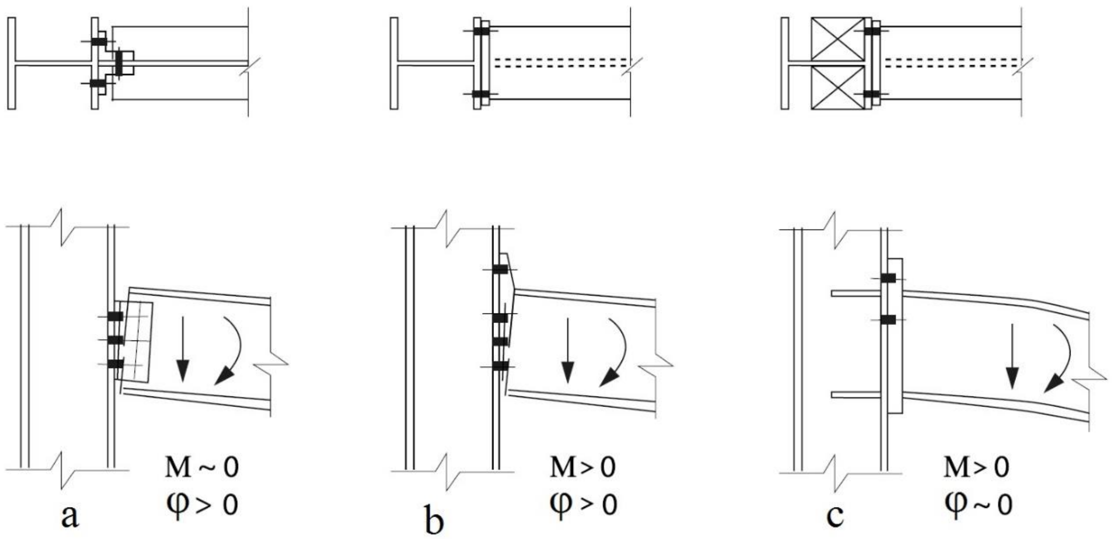

Figure 8. An example of the beams to the columns junction: a - the hinged; b - semi-rigid; c – rigid; M - bending moment; φ - sectional rotation angle

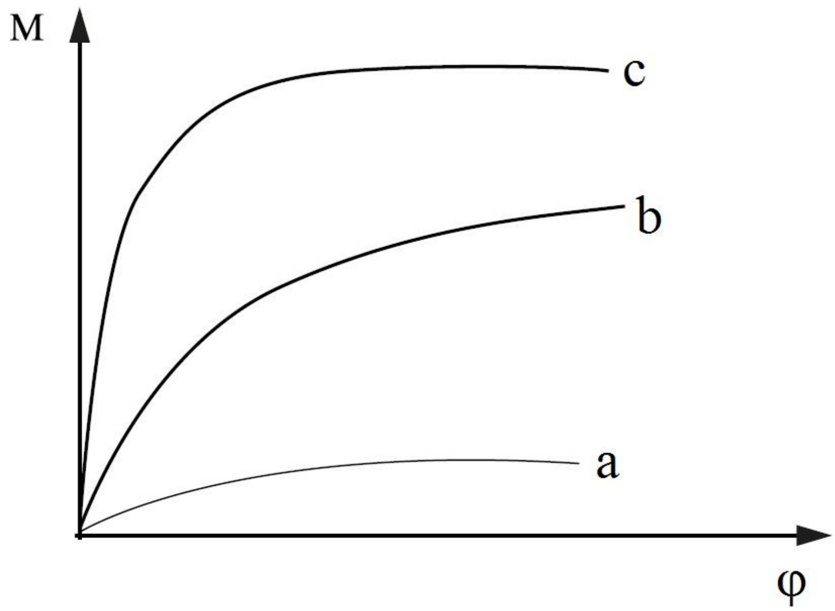

Figure 9. Dependence on the bending moment from the sectional rotation angle

In modern construction methods, structural elements that connected with each other, account for two basic types:

• welded connections;

• bolting connections.

Other types of connections - rivets, glue, etc. - currently used in the industry is very limited in special cases when it is justified.

5 Joints of the beams

There are two types of joints beams: the factory and the assembly (consolidated).

1) The factory joints

Performed at the factory, are the joints of the individual parts of a beam element (walls, chords). They are used when changing the section or insufficient length of an existing rolled. Location of joints is also caused by having a length of rolled and design considerations (wall junction should be different from the place of junction of auxiliary beams, stiffening ribs, etc.). To weakening section beams factory seam was not too large, the joints of individual elements are usually placed in different locations along the length of the beam, in the extension.

2) The assembly joints

Performed when installing, are used in cases where the weight or dimensions of the beam does not allow to carry and mount it in its entirety. Their arrangement should provide for the division of the beam into separate elements-departure, possibly the same (in the split beam butt joint arrange at the midspan or symmetrically with respect to the middle of the beam), satisfying the installation requirements, and transportation with the most common means.

In the mounting joints, it is convenient to connect all the elements of the beam in one section. This is called a universal joint.

Joints of rolling beams, factory and assembly, performed, as a rule, welded. Possible constructive solutions of are shown in the Figure 10.

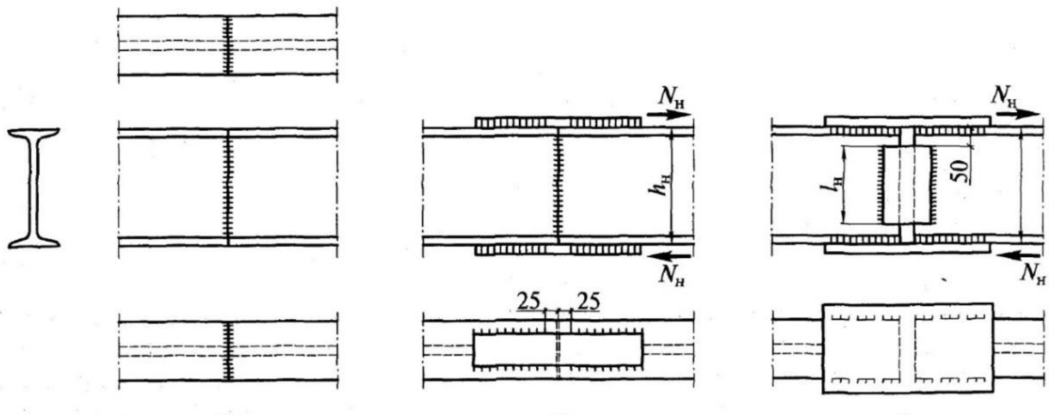

Figure 10. Joints of the rolling beams:

a) butt joint; b) butt joint with overlays; c) just overlays

The most simple and convenient is the direct connection of beams with the butt joint (Figure 10, a). The joint must be welded faster to reduce shrinkage welding stresses, so that cooling was going more evenly. Welding must be started with less rigid element - wall. However, manual welding of the junction with the use of conventional control methods of welding of the stretched beam chord in the junction will have a lower strength than the joint is, as the design resistance of the weld butt joint to the stretching R_{wy} is less than the calculated resistance of the base metal R_y:

$$M_{bj} = M * \left(\frac{R_{wy}}{R_y}\right) = 0.85 * M. \quad (4.1)$$

If necessary to make the joint in the section, which has a larger bending moment, a straight butt joint of beams is making and enhancing the shelf with the overlays (Figure 10, b). The bending moment in that junction is perceived by the butt seams and overlays:

$$M = W * R_{wy} + N_{ol} * h_{ol}. \quad (4.2)$$

where W – moment of resistance of the beam walls;

N_{ol} - force in the overlay;

h_{ol} - the distance between the overlays axes.

Hence, the design effort in the overlay is defined:

$$N_{ol} = \frac{M - W * R_{wy}}{h_{ol}}. \quad (4.3)$$

And the area of its cross-section:

$$A_{ol} = \frac{N_{ol}}{R_{wy}}. \quad (4.4)$$

The corner seams that attach the overlay to the beam should be calculated on the force in the overlay. To reduce welding stresses, these joints do not lead to the joint axis by 25 mm on each side.

In the manufacture of structures in the field workshops, when it is difficult to handle the ends of the beams for welding, it can be accomplished only by joint overlays (Figure 10, c). However, due to the high stress concentration in that junction, it can be used in constructions working only with static load and at positive temperatures.

Almost all of the bending moment at this junction is transmitted through the chords covers, and shear force is transmitted through a pair of covers on the wall. Therefore, conventionally and more in to safety margin an effort in the patch and the area of its cross-sectional area are defined by the formulas:

$$N_{ol} = \frac{M}{h_{ol}}. \quad (4.5)$$

And the area of its cross-section:

$$A_{ol} = \frac{N_{ol}}{R_y}. \quad (4.6)$$

Covers for wall are constructively taking 100-150 mm thickness, approximately equal to the wall thickness, and height is taking equal to the height of the straight wall portion (rounding up around shelves).

In a built-up member in which plates are connected by means of intermittent fillet welds, a continuous fillet weld should be provided on each side of the plate for a length at each end equal to at least three-quarters of the width of the narrower plate concerned (Figure 11).

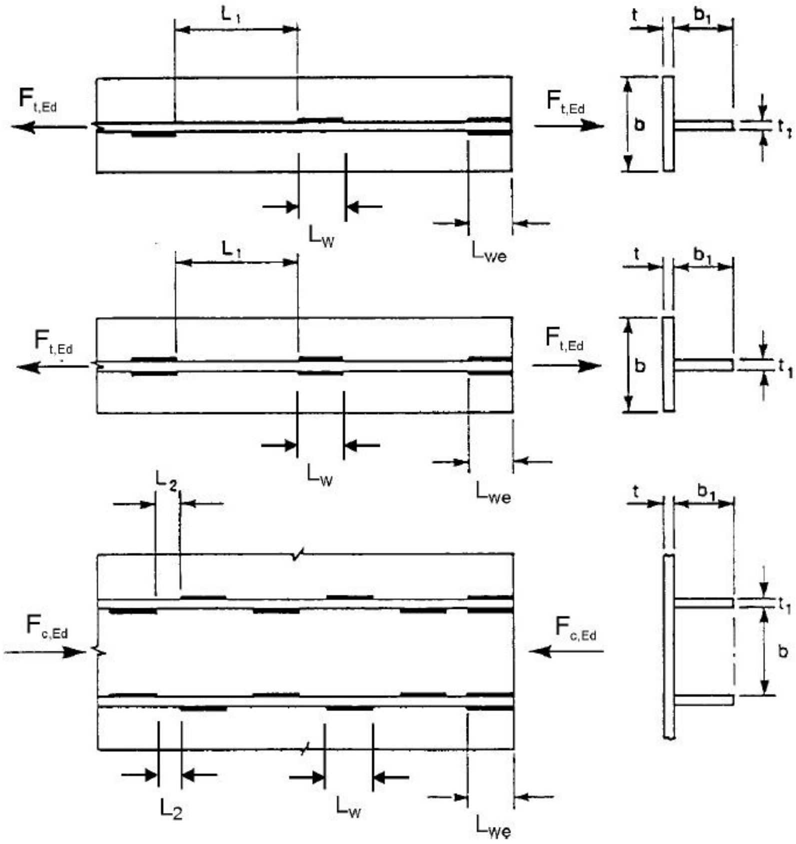

Figure 11. Intermittent fillet welds

The smaller of L_{we} ≥ 0,75 *b* and 0,75 b_1;
For build-up members in tension:
The smallest of L_1 ≤ 16 *t* and 16 t_1 and 200 mm;
For build-up members in compression or shear:
The smallest of L_2 ≤ 12 *t* and 12 t_1 and 0,25 *b* and 200 mm.

In the Russian and Finnish standards, such welding is carried out by the same rules. Fillet welds along the perimeter of holes or slits may be used in the lap welds joints in the cases provided to transmit forces in the plane of overlap, to prevent buckling of the elements of the overlapping or structural compounds of the elements.

The use of combined compounds in which part of the perceived frictional shear connection, and part shear welds is allowed providing that the welding must be performed after the final tightening of the bolts. The distribution of effort between the friction and welded joints may be taken in proportion to their load-bearing capacity or accept that the frictional connection receives a force from the constant load, and weld receives a force from temporary load. The use of other compounds in the combination of bolted connections is not allowed (Figure 12, Table 1) (6).

The difference between the Russian and Finnish standards is generally consists in difference between the reduction coefficients β, that in the Russian norms is varied from 0.7 to 1.9, while in the Finnish standards is varied from 0.8 to 1.0. In Finland, welding is not used when mounting structures. The priority is the use of bolting. One of the Finnish I-beam connection is shown in the Figure 13.

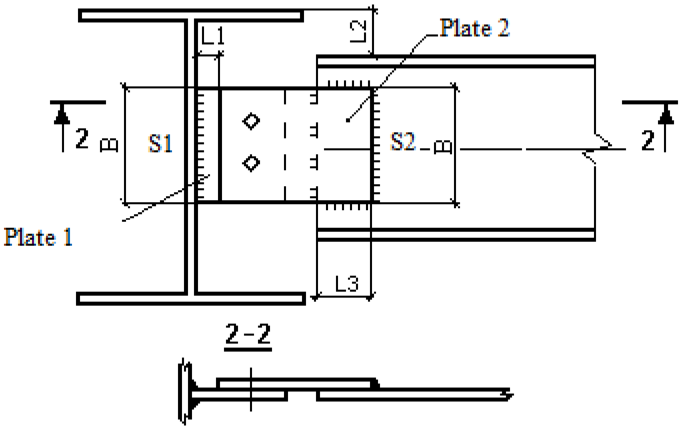

Figure 12. Beam - to - beam connection

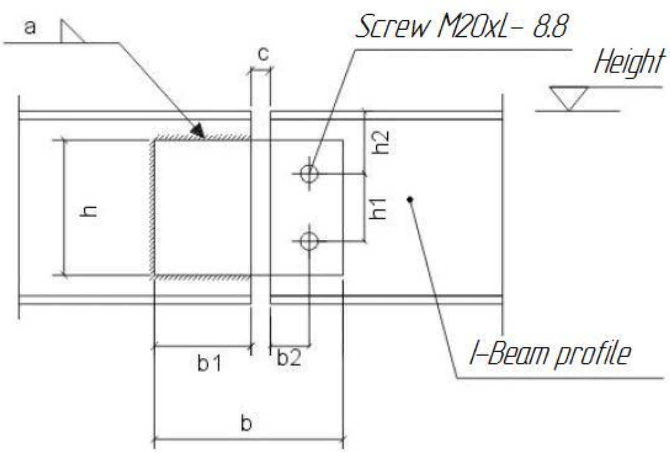

Figure 13. Beam - to - beam connection in Finland

Table 1: The results of verification

Parameter	Properties	Value	Percentage of use, %	Internal forces				
				N, kN	My, kN m	Qz, kN	Mz, kN m	Qy, kN
Seam S1	The cathe-tus	6.0 mm	45.7	200.0*	0.0	80.0*	0.0	0.0
Seam S2	The cathe-tus	7.0 mm	81.3	200.0*	0.0	80.0*	0.0	0.0
Plate 1	The thickness t1	6.0 mm	90.8	200.0*	0.0	80.0*	0.0	0.0
	The size B	375.0 mm						
	The size H	110.0 mm						
Plate 2	The thickness t2	10.0 mm	99.7	200.0*	0.0	80.0*	0.0	0.0
	The size B	205.0 mm						
	The size H	155.0 mm						
Bolts	The quan-tity	6	92,3	200.0*	0.0	80.0*	0.0	0.0
	The num-ber of ver-tical rows	2						
The size L1	-	10.0 mm	-	-	-	-	-	-
The size L2	-	0.0 mm	-	-	-	-	-	-
The size L3	-	40.0 mm	-	-	-	-	-	-

*the efforts involved in the selection or checking of the appropriate parameters

5.1 Joints of the composite welded beams

Factory joints chords and walls of the composite welded beams connects sheets to build them in the beam (Figure 14, a). The main type of welds sheets is a splice. Splice compressed zones made using welding materials with full penetration of connected elements (which are displayed on the ends of the seams special technological strips), are considered to have equal strength to the base metal, they cannot count.

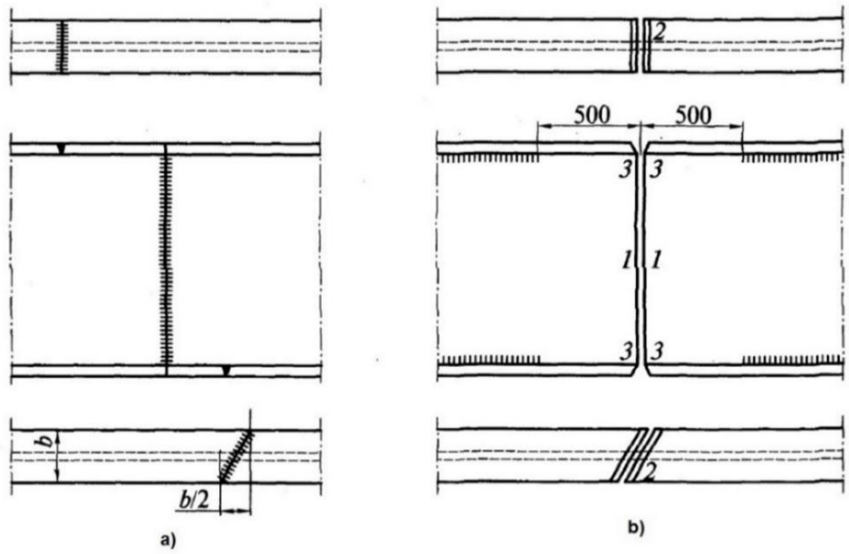

Figure 14. The factory (a) and the assembly (b) joints of composite beams

Joints stretched chords can also be considered as full-strength to the base metal only if the above requirements and additional testing of physical methods of control. In the absence of physical quality control of stretched seam, it can be considered equal strength to the base metal, but lengthening it and making an oblique at an angle of about 60° to the axis of the beam.

This complexity often makes production more appropriate the transfer of the direct factory stretched butt weld in place of the beam, where the tension in the chord does not exceed the calculated weld tensile strength. Factory joints compressed zone and beam web is always doing direct.

On assembling the compressed zone and the wall always connect straight seam butt joint and stretched chord oblique seam at an angle of 60°, as the installation of physical methods of control are difficult. Such joint full-strength to the main beam section and cannot be calculated.

Some overpotential of the wall, calculated against near the beam waist stretched, usually neglected since this portion is located between the two beams elastically working zones and it is impossible to ductile failure.

5.2 The joints of composite beams with high-strength bolts

Recently, assembly joints of welded beams are sometimes performed on high-strength bolts with overlays in order to avoid welding in the assembly. In such joints laths pressed strongly bolted to the abutting member and the force in the element of friction forces is transferred to the lining (Figure 15, 16).

Each chord of the beam is desirable to overlap with the three overlays on the both sides, and the wall of two vertical overlays, cross-sectional area of which should not be less than the cross-sectional area of the overlapped element.

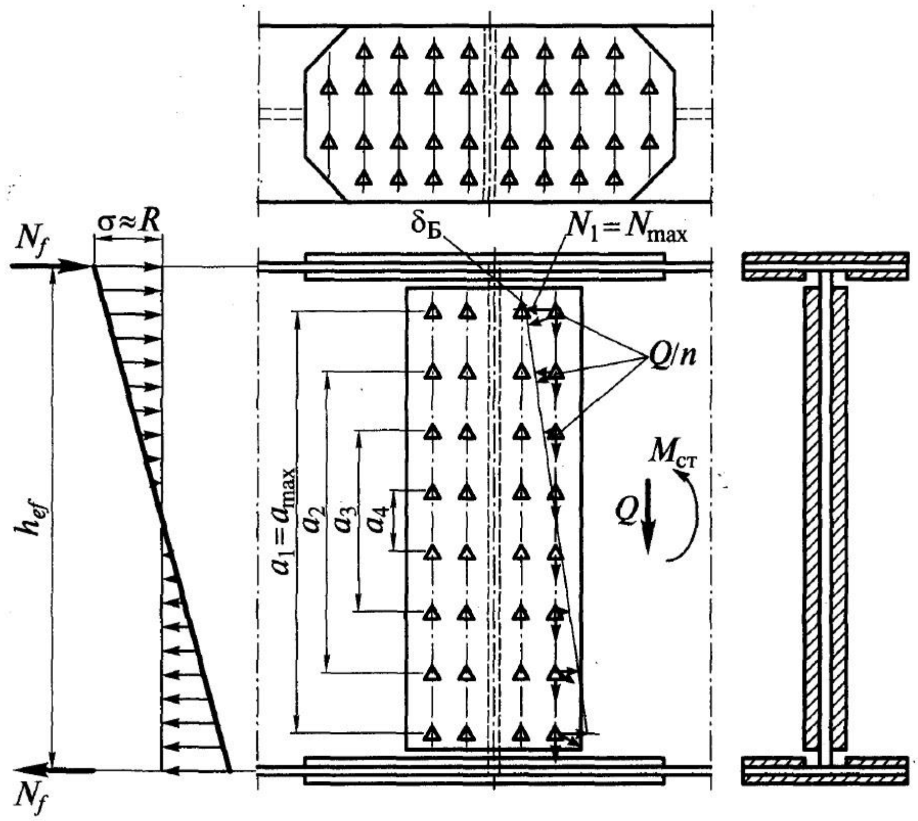

Figure 15. Assembly butt of the welded beams on the high-strength bolts

The weakening of the cross section of the beam zone accounted for as follows: static load, if the net section area A_n is less than 85% of the gross area, $A_n \leq 0.85 * A$, then the conditional cross-sectional area is taken as $A_c = 1.18 * A_n$. When the dynamic loads A_n is accepted regardless of the amount of attenuation. The weakening of the cross section can be adjusted with the corresponding arrangement of the bolts.

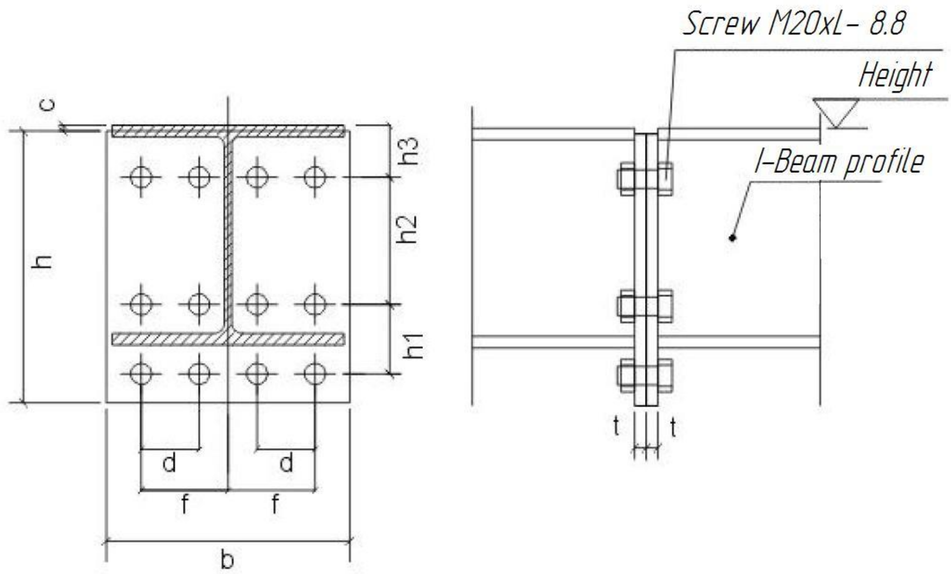

Figure 16. The Finnish I-beams joint with the screws

The bolts in the joint should be placed at a minimum distance from each other - 2.5 - 3 diameter of the holes for the bolts to reduce the size and weight of the cover plates. Calculation of the joint of each element of the beam are separated, and the bending moment is distributed between the belts and the wall in proportion to their rigidity. Then the design effort in the zone can be determined by the formulas:

$$M_f = M * (I_f / I). \quad (4.7)$$

$$N_f = (M_f / h_{ef}), \quad (4.8)$$

where M and I - respectively complete design bending moment and moment of inertia of the entire section in the place of the beam joint;

I_f- the moment of inertia of the beam chords;

$h_{ef} = h_w + t_f$ - estimated altitude of the chords.

The number the bolts to attach the cover plates to the chord of the beam:

$$n = (N_f / Q_{bh}), \quad (4.9)$$

where Q_{bh}- design shear force that can be perceived by a high-strength bolt.

Friction compounds in which the loads are transmitted through friction that occurs at the contacting surfaces of connected components due to high-tension bolts used:

1) in structures, directly perceiving moving, vibration and other dynamic forces;
2) in the multi-screws compounds, which are increased requirements for limited deformability.

Estimated effort, which can be perceived every plane of friction elements, strapped with one high strength bolt, determined by the formula (4):

$$Q_{bh} = (R_{bh} * A_{bn} * \mu)/\gamma_h, \quad (4.10)$$

where R_{bh}- calculated tensile strength of high-strength bolts;

A_{bn} - bolt-sectional area of the thread;

μ - slip factor (see Table 2);

γ_h - reliability coefficient.

Table 2: Slip factor, μ, for pre-loaded bolts (3, 4)

Class of friction surfaces in Finland and in Russia	Slip factor μ in Finland	Slip factor μ in Russia
A	0,5	0,58
B	0,4	0,42
C	0,3	0,35
D	0,2	0,25

The yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolt classes 3,6 – 12,9 are given in Table 3. These values should be adopted as characteristic values in design calculations.

Table 3: Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts in Finland and in Russia (3,4,5)

Bolt class		3,6		4,6		4,8		5,6		5,8	
Russian (R)	Finnish (F)	R	F	R	F	R	F	R	F	R	F
f_{yb} (N/mm^2)		180	-	240	240	320	320	300	300	400	400
f_{ub} (N/mm^2)		300	-	400	400	400	400	500	500	500	500
Bolt class		6,8		8,8		9,8		10,9		12,9	
Russian (R)	Finnish (F)	R	F	R	F	R	F	R	F	R	F
f_{yb} (N/mm^2)		480	480	640	640	720	-	900	900	1080	-
f_{ub} (N/mm^2)		600	600	800	800	900	-	1000	1000	1200	-

5.3 Flanged joints

The widespread use of high-strength bolts in the mounting connections led to the flanges in bending elements, especially in light metal structures and rigid connection of the beams to the columns. Flange connections have a number of advantages over offset-stable compounds. In the flange joints are reduced consumption of metal on the compound, the number of bolts and the complexity of the connection.

The flange joint construction is shown in Figure 17, 18. To the ends of the connected beams flanges are welded - thick sheets having holes for bolts. In the interconnected flanges, these holes are inserted in the installation of high-strength bolts and nuts are tightened, after which the joint is ready. Thus, the installation process is extremely simple and not time consuming.

The operation of this connection is determined by the fact that the compressed force in the junction zone beams are transmitted by direct contact of the flange with each other, and the force zone stretched beams transmitted through the flanges on the bolts, stretching them.

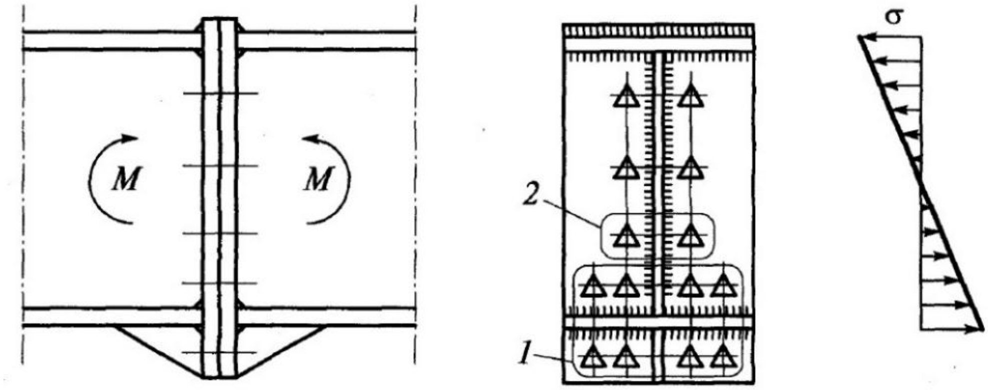

Figure 17. Flanged seam of the welded beam:

1 - bolts of the inner chord zone; 2 - bolts of the stretched wall zone

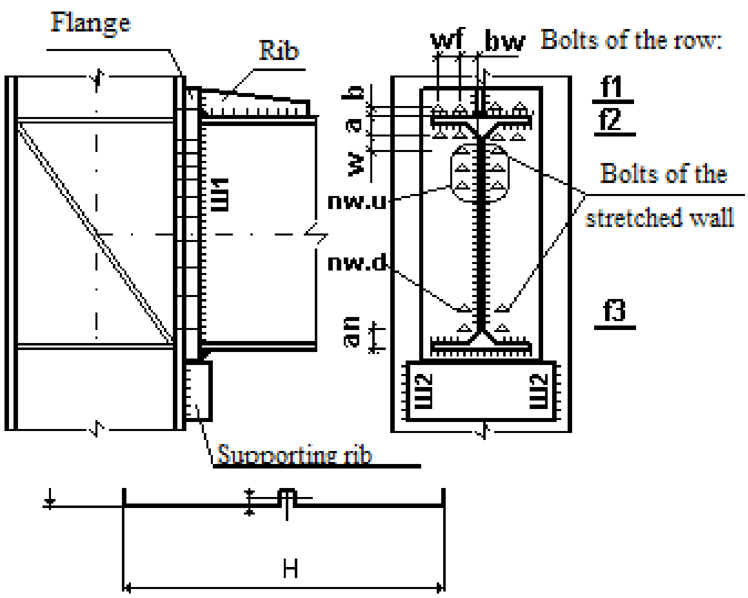

Figure 18. Flange beam to the column connection

In accordance with the specifics of the joint the majority of the bolts are concentrated in the stretched beam chord, sometimes being located in two or three rows, and in the area of compressed beams bolts are put on the maximum distance to save the connection density. The distribution of tensile forces between the bolts is very uneven and depends on the placing bolts and bending stiffness of the flange. Thus, the ratio of effort per one bolt of the inner and outer connection zones ranges from 1.1 at the thick flanges to 2.6 with thin flanges.

6 Supporting and connection of the beams

Connection beams with steel columns are the top bearing beams or beams abutting laterally applied to the column. Such an adjunction can be a swivel, transmitting only the support reaction of the beam, or rigid, transmitting to a column except the support of the reaction is also the moment of pinching beams in the column.

Swivel bearing is widely used in the majority of beam structures (Figure 19, Table 4) (6). Rigid connection is used in multi-storey buildings frameworks. Examples of the bearing beams on the top and side of the column are shown in Figures 20 and 21. End of the beam in its bearing spot on the support is bearing with the reinforcing ribs, while considering that all the supporting response is transmitted to the support beams through the stiffening ribs. Transversing of the compression on an unstiffened column is shown in Figure 23. In welded joints, the transverse stiffeners should be aligned with the corresponding beam flange. In bolted joints, the stiffener in the compression zone should be aligned with the centre of compression (3).

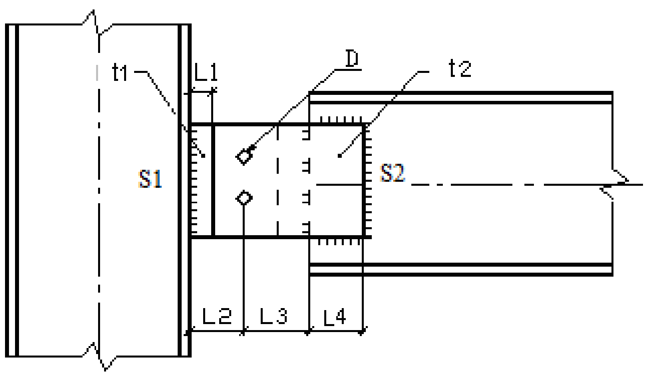

Figure 19. Swivel beam to column connection

Table 4: The results of verification

Parameter	Properties	Value	Percent-age of use, %	Internal forces				
				N, kN	My, kN m	Qz, kN	Mz, kN m	Qy, kN
Seam S1	Cathetus	20.0 mm	86.7	1000.0*	0.0	-250.0*	0.0	0.0*
Seam S2	Cathetus	8.0 mm	98.7	1000.0*	0.0	-250.0*	0.0	0.0*
Plate 1	Thickness t1	25.0 mm	98.5	1000.0*	0.0	-250.0*	0.0	0.0*
	Size B1	130.0 mm						
	Size H1	330.0 mm						
Plate 2	Thickness t2	40.0 mm	95.3	1000.000*	0.000	-250.000*	0.000	0.000*
	Size B2	260.0 mm						
	Size H2	245.0 mm						
Bolts	Quantity	12	92.9	1000.000*	0.000	-250.000*	0.000	0.000*
	Number of vertical rows	2						
Size L1	-	30.0 mm	-	-	-	-	-	-
Size L2	-	125.0 mm	-	-	-	-	-	-

*the efforts involved in the selection or checking of the appropriate parameters

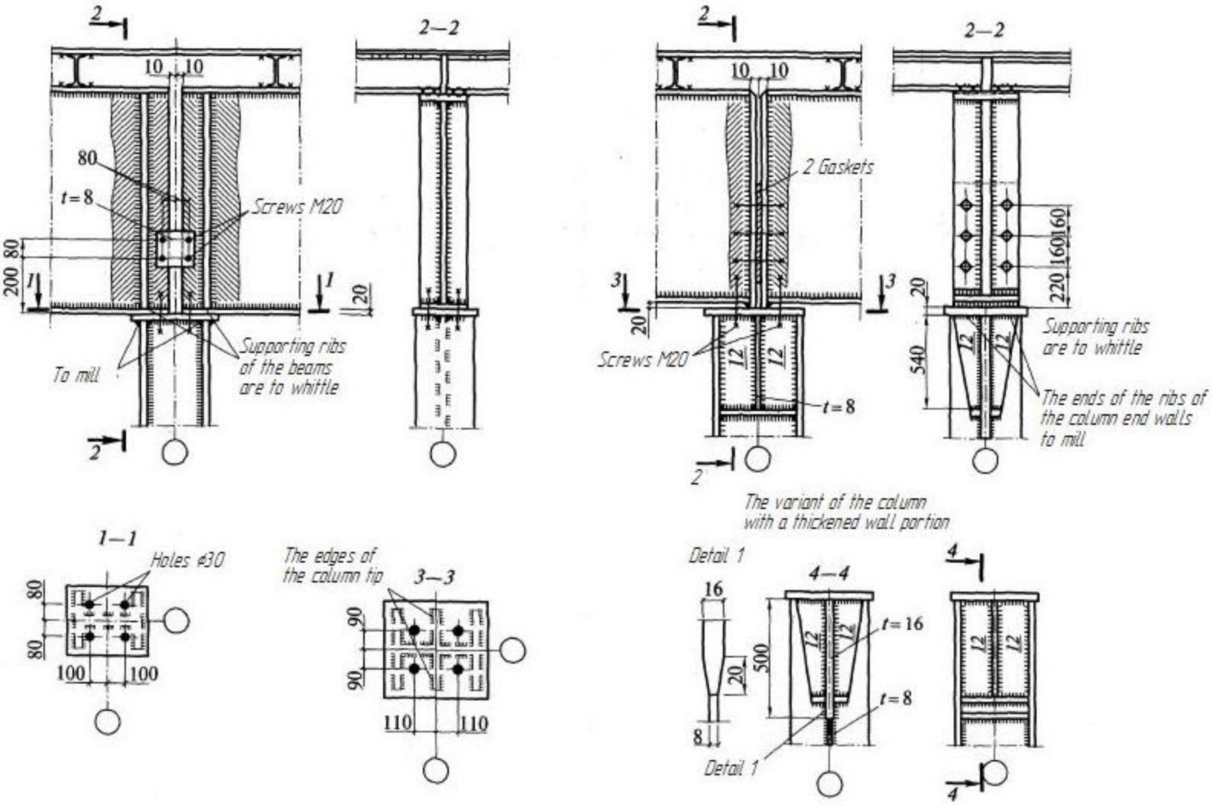

Figure 20. Supporting the beams on top of the column

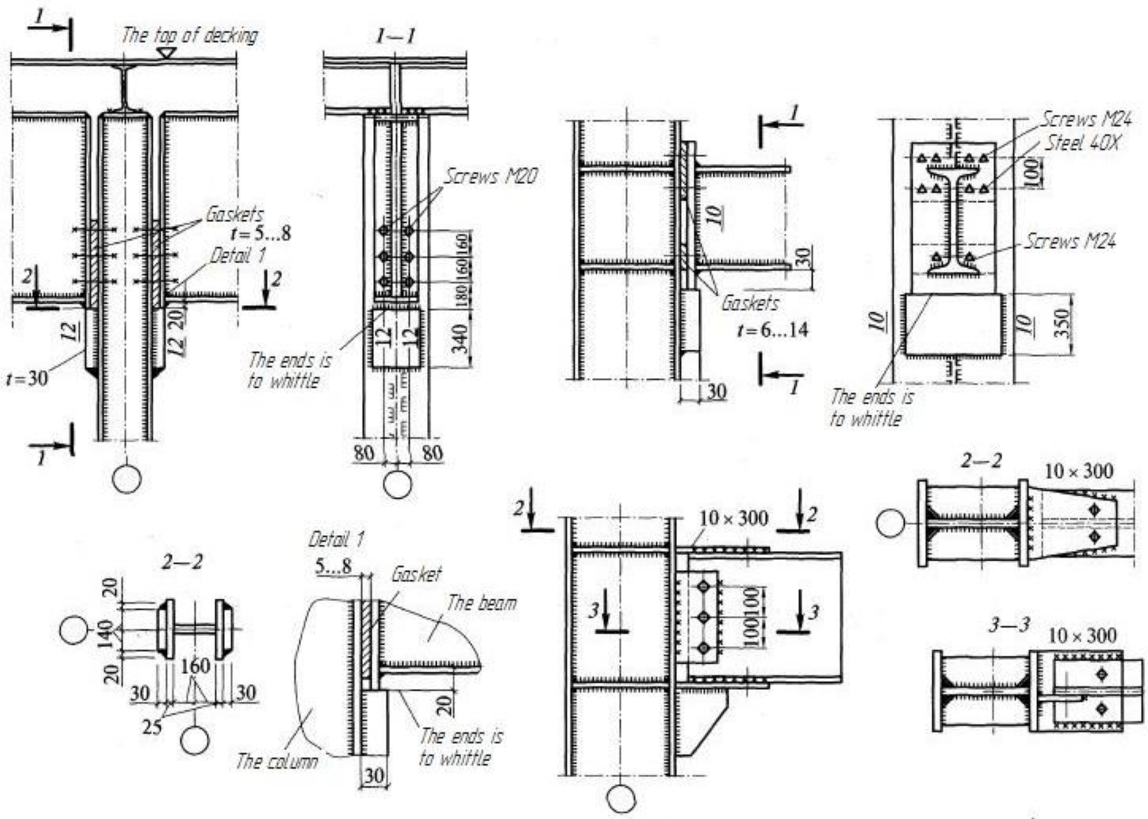

Figure 21. Supporting beams on the side of the column

In place of the transmission of the tangential stresses from the wall of the beam to the supporting rib, stress distribution law of Zhuravsky on height is broken, stresses are concentrated in the bottom wall, the degree of concentration depends on the ratio of wall thickness and the cross-sectional area of the rib. With relatively strong ribs and approval of local plastic deformation transfer of shear stresses to the supporting rib can be assumed uniform throughout the height of the wall. Stiffening rib for transmission support reaction is securely attached to the wall of the welds and end stiffeners tight-fit to the lower chord of the beam.

Dimensions of support ribs is usually determined based on crushing (pressure) of the ends of the ribs:

$$\sigma_p = \frac{F}{A_p} \leq R_p * \gamma_c, \qquad (5.1)$$

where F - support reaction of the beam;

A_p - pressure area of the support ribs, in welded beams shall be equal to the whole part of the rib area;

R_p - calculated resistance of steel collapse (pressure) of the end surface.

In addition to checking for crushing end support rib also checked the reference section of the beam on the stability of the plane of the beam as the conditional support rod, which includes the area of its section of the support ribs and part of the beam wall with the width about $0.65 * t_w * \sqrt{E/R_y}$ in each direction and a length equal to the height of the beam walls:

$$N/(\varphi * A_w) \leq R_y * \gamma_c, \qquad (5.2)$$

where φ - coefficient of buckling of the struts with flexibility $\lambda = h_w/i_z$, defined relative to the axis z- z, directed along the beam.

Attaching the supporting rib to the wall beam weld seams must be designed for full support reaction of the beams with the maximum working length of the weld. The hinge side abutment of beams in their constructive design, the work and the calculation is not different from the bearing top of beams.

In the rigid beam to a column connection, support reaction of the beam is transmitted to a column in the bolted version across the table or in a welded version - through a special edge welded to the column wall and beam and the reference point of the beam is passed through a similarly flanged joints and bolts stress on the lower column of the beam chord or horizontal plates connecting beams chord with a column (Figure 22, Table 5) (6).

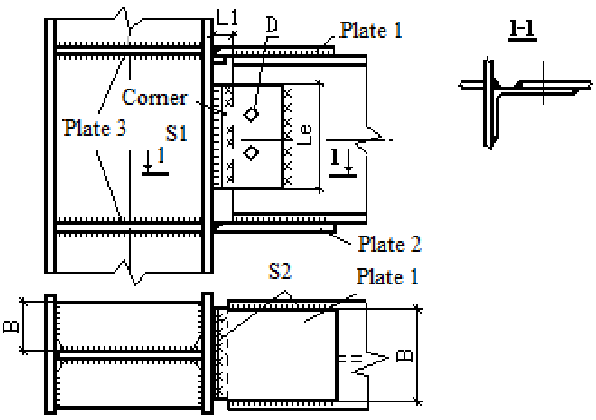

Figure 22. The rigid beam to a column connection

Table 5: The results of verification

Parameter	Properties	Value	Percentage of use, %	Internal forces					
				N, kN	My, kNm	Qz, kN	Mz, kNm	Qy, kN
Seam S1	The cathetus	5.0 mm	24.4	800.0	200.0	80.0*	0.0	0.0
Seam S2	The cathetus	10.0 mm	94.0	800.0*	200.0*	80.0	0.0*	0.0*
Plate 1	The thickness t1	10.0 mm	19.0	800.0	200.0	80.0*	0.0	0.0
	The size B	80.0 mm						
	The size H	270.0 mm						
Plate 2	The thickness t2	16.0 mm	34.3	800.0*	200.0*	80.0	0.0*	0.0*
	The size B	115.0 mm						
	The size H	330.0 mm						

Parameter	Properties	Value	Percentage of use, %	Internal forces				
				N, kN	My, kNm	Qz, kN	Mz, kNm	Qy, kN
Plate 3	The thickness t4	30.0 mm	78.5	800.0 *	200.0*	80.0	0.0*	0.0*
	The size B	195.0 mm						
	The size H	330.0 mm						
Beam	The wall thickness	6.2 mm	47.2	800.0	200.0	80.0*	0.0	0.0
The number of bolts	-	2	-	-	-	-	-	-
Size L1	-	20.0 mm	-	-	-	-	-	-

*the efforts involved in the selection or checking of the appropriate parameters

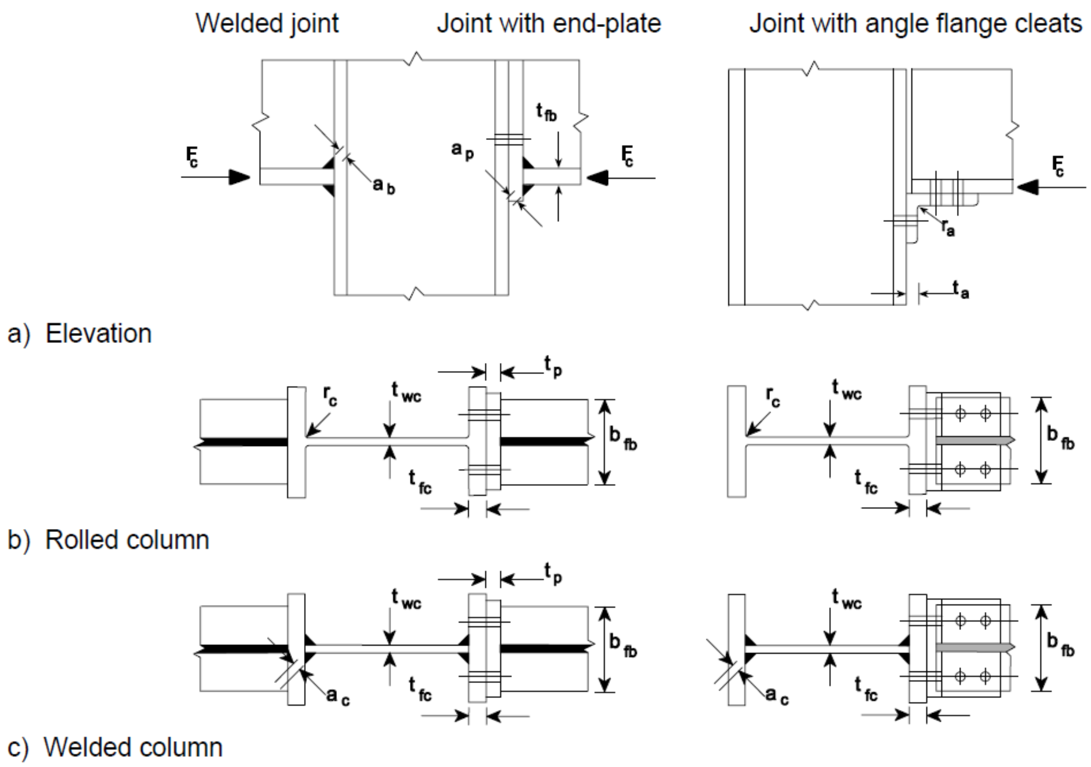

Figure 23. Transverse compression on an unstiffened column

6.1 Supporting of the beams on the walls and reinforced concrete lining

When leaning of beams on the stone walls and reinforced concrete lining is usually used special steel bearings, which are used for evenly pressure distribution of the beam over a larger area less durable than steel, bearing material (stone, concrete).

In addition, the support part needs to ensure freedom of deformation of the ends of the beam - rotating at the bending of the beam, the longitudinal displacement from temperature and strain of power; otherwise in the support arises undesired additional stresses.

In accordance with these requirements fixed and movable bearing parts are used with the following types (Figure 24): With the spans, up to 20 m - flat base plates (Figure 24, a and b.); up to 40 m - tangential support plates (. Figure 24, c); more than 40 m - rollers, bearing parts (Figure 24, d.). Bearing parts are made of cast or thick sheet steel.

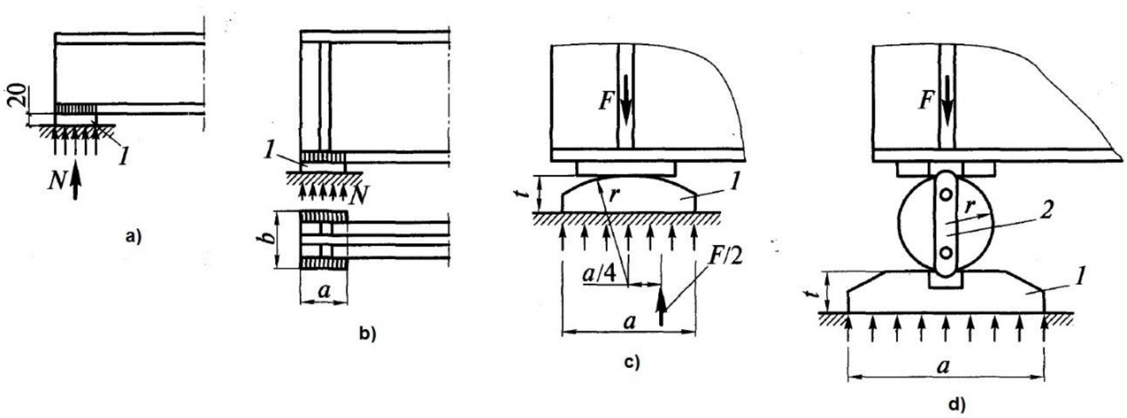

Figure 24. Supporting beams on concrete:
a) leaning of beams rolling on a flat base slab; b) the same, composite beams; c) tangential leaning of beams; d) leaning of beams to the rink; 1) the base slab; 2) anti-shift bracket

To ensure the correct location of the rink in the supporting parts to the sides attach anti-shift, and in the middle doing a flange that prevents the rink to move across.

Calculated bending moment in the middle section of the slab:

$$M = \frac{F}{2} * \frac{a}{4} = \frac{Fa}{8}, \tag{5.3}$$

where F - the design pressure of the beam to the support.
Moment of resistance of the slab section:

$$W_{sl} = b * \frac{t_{sl}^2}{6} = M/(R_y * \gamma_c). \tag{5.4}$$

To reduce rolling friction, roller diameter appoints approximate to the formula (not less), mm:

$$d = 2 * r \geq 130 + l/1000, \tag{5.5}$$

where l - span of the beam, checking on local collapse (pressure), according to:

$$r = \frac{F}{2 * l * R_{cd} * \gamma_c}, \tag{5.6}$$

where l - contact length of the cylindrical surface of the roller or tangential base plate from the top plate;
$R_{cd} = 0.025 * R_u$ - calculated resistance ”diametrical compression of the rollers” with free touch (4).

6.2 Connection of the beams

Connection of major and minor beams with each other are storey, in the same level of the upper chords and with the lower position of the upper chords of secondary beams (Figure 25). Storey connection (Figure 25, a) is the simplest, but because of the possible bending of the main beam belt it can only transfer small support reactions.
Connection in one level and in the decreased level are able to transfer large bearing reaction. The disadvantage of connection in the same level (Figure 25, b) is the need to notch the top part and the wall part of the auxiliary beam.
These inconveniences can be avoided by welding at the factory to the end of the auxiliary beam a short corner, and match it in the installation with bolts or welding to the rib of the main beam (Figure 25, c).

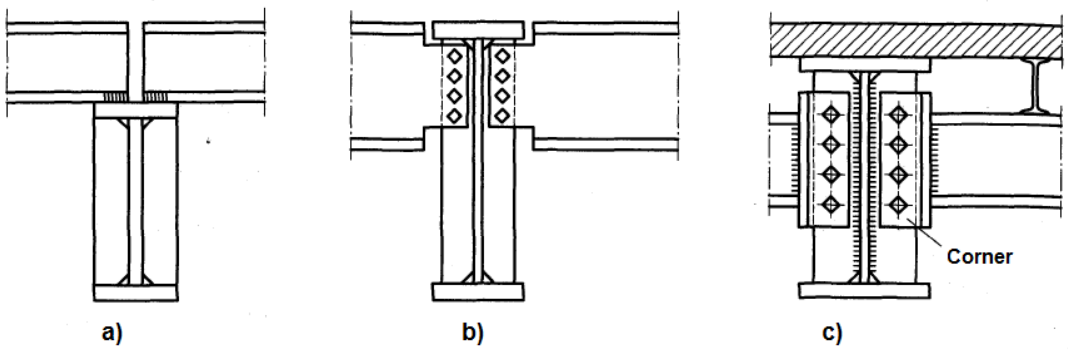

Figure 25. Hinged connection of beams:

a) per floor; b) in the same level; c) with the decreased level

All of the connection beams worked as hinged. If necessary of the rigid connection of beams (Figure 26) are introduced “fishes” (at the same height of beams) or “fishes" and a table (at a different height of beams).

In this connection arises not only the lateral force, transmitted to the screws that attach the wall of the auxiliary beam to the edge of the main beam, or directly on the table, but also the support moment that is transmitted through a special lining - fishes or the fish and table.

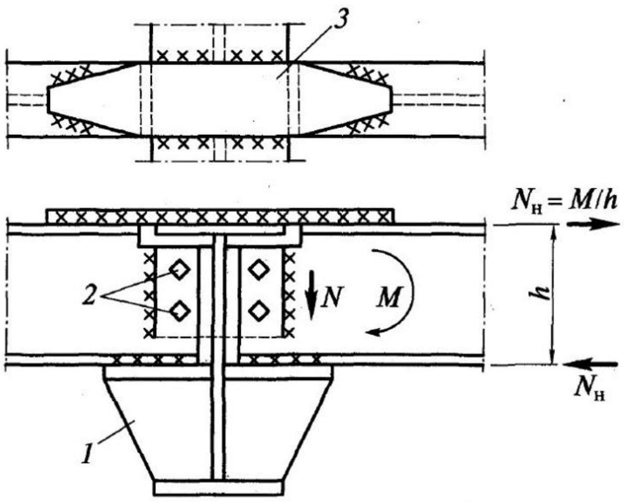

Figure 26. Rigid connection of the beams:

1) table; 2) assembly screws; 3) "fish"

The example of beams connection in Finland is shown in Figure 27.

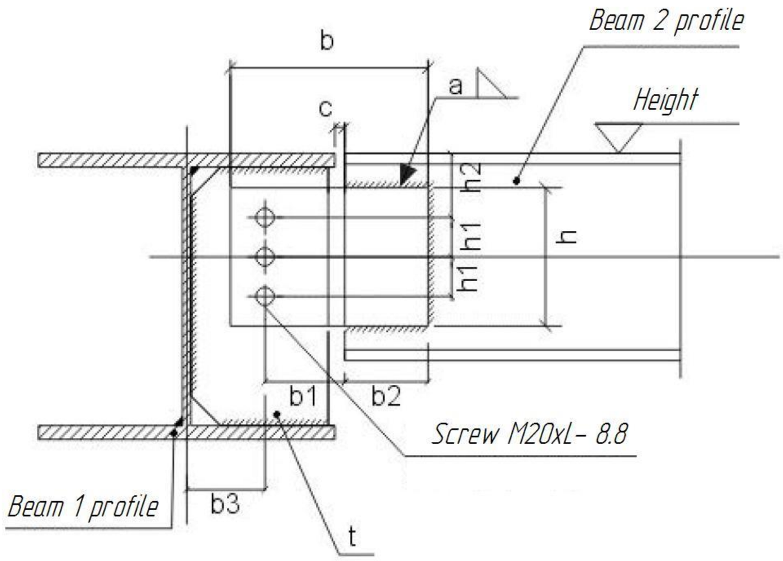

Figure 27. Connection of the beams in Finland

7 Bases of the columns

At the hinge connection, base, for random moments, should be able to rotate relative to a foundation, if the connection is rigid it is necessary to provide base connection with the foundation is not permitting rotation.

According to a constructive solution a base can be with crossarm (Figure 28, a) with a milled end (Figure 28, b) and with the hinge device in the form of a centering plate (Figure 28, c).

With a relatively small computational effort in columns (4000 - 5000 kN) base with traverses is applied. Traverse bears the load on the rod of the column and transmits it to the base plate.

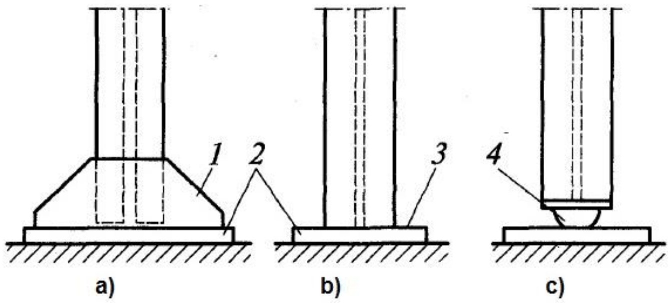

Figure 28. Types of the columns bases:

a) the presence of traverse; b) with milled end; c) tangential support; 1) traverse; 2) a plate; 3) milling; 4) centering plate

To improve the uniformity of pressure transfer from the plate to the foundation, the stiffness of the slab is increased by additional edges between the branches of the traverse (Figure 29, a). In the light columns, the role of traverses can perform the cantilever ribs, welded to the rod of the column and the support plate (Figure 29, b). In the columns with large calculated effort (6000-10000 kN and more) it is advisable to mill the end of the column and the plate surface.

At the hinged of the column connection with foundation anchor bolts are placed only for fixing the design position of the column and secure it in the installation process. In this case the anchors are attached directly to the base support plate (Figure 29, a and b). At rigid anchors connections are attached to the rod of the column via remote console and tightened with a voltage close to the calculated resistance, eliminating the possibility of turning of the column (Figure 29, c; Figure 30).

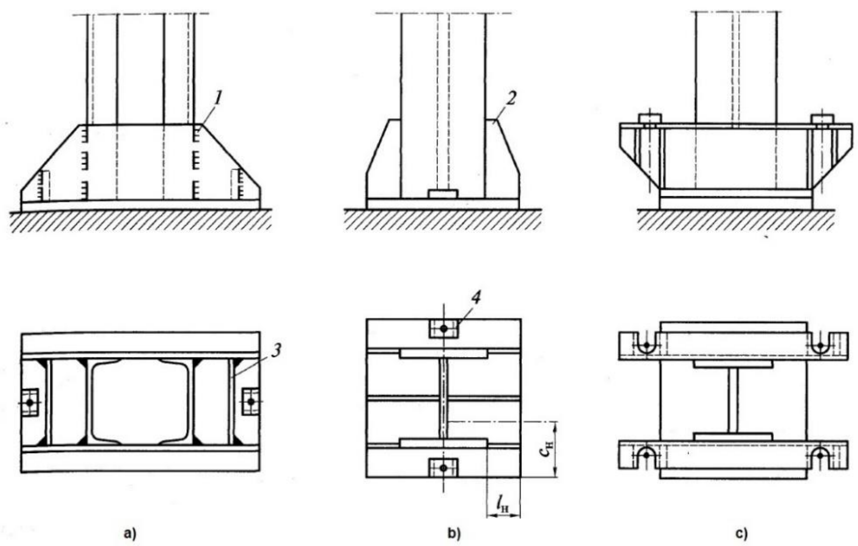

Figure 29. Bases of centrally compressed columns:

a) with the traverses; b) with the gussets; c) with a rigid support of the column to the foundation; 1) traverse; 2) the console rib; 3) the diaphragm; 4) the anchor washer

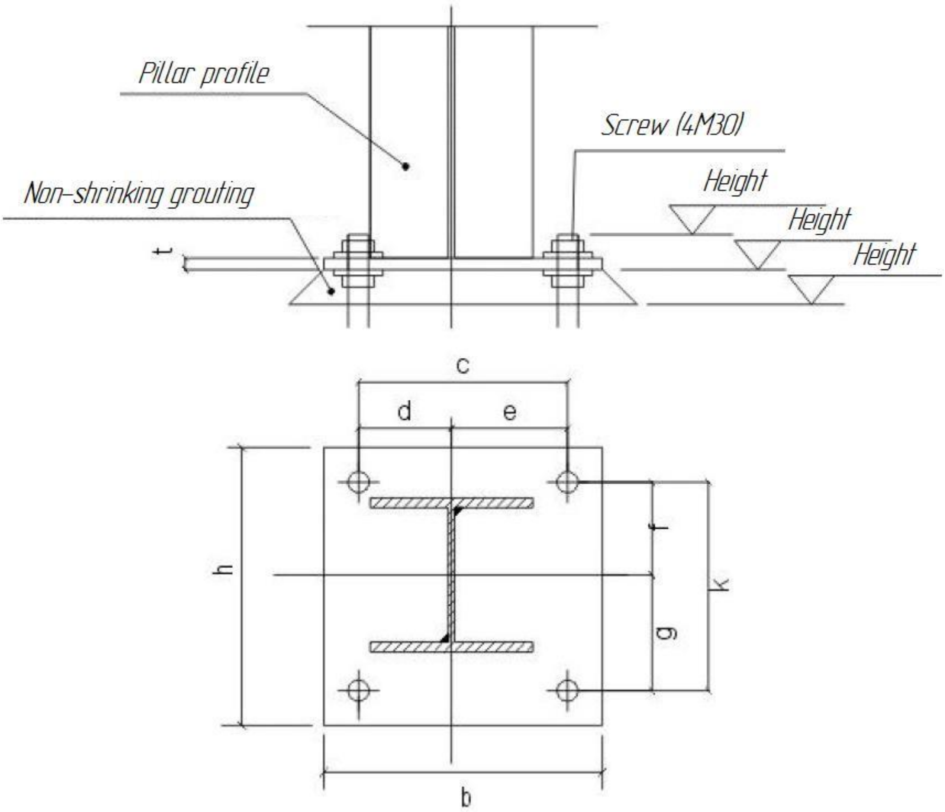

Figure 30. Connection of the pillar and the foundation in Finland

An example of the welded beam to the column connection is shown in Figure 31. The results of verification are shown at Table 6 (6).

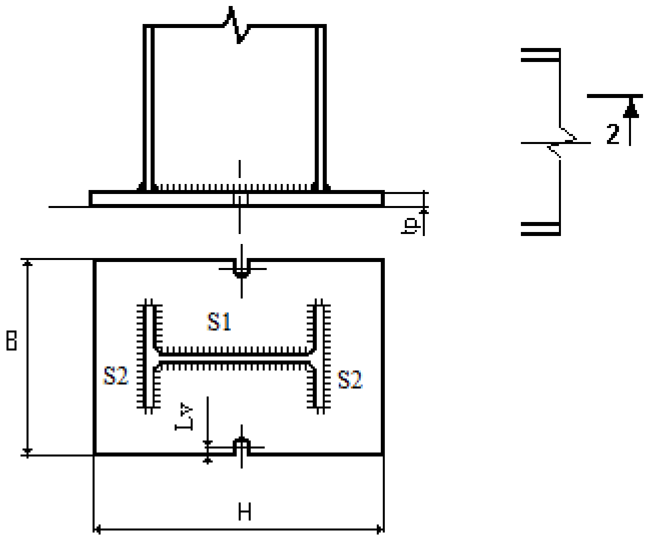

Figure 31. The welded beam to the column connection

Table 6: The results of verification

Parameter	Properties	Value	Percentage of use, %	Internal forces				
				N, kN	My, kNm	Qz, kN	Mz, kNm	Qy, kN
Seam S1	The cathetus	7.0 mm	34.4	-2500.0*	0.0	0.0	0.0	0.0
Seam S2	The cathetus	7.0 mm	34.4	-2500.0*	0.0	0.0	0.0	0.0
Plate 1	The thickness tp1	6.0 mm	99.0	-2500.0*	0.0*	0.0	0.0*	0.0
	The size B	375.0 mm						
	The size H	110.0 mm						

*the efforts involved in the selection or checking of the appropriate parameters

7.1 The bases with the traverses and cantilever ribs

The slab works as a plate on an elastic foundation, receives the pressure from the branches, traverses and ribs. Experiments have shown that the pressure on the foundation is unevenly distributed, with peaks at locations of the load transmission. However, for simplicity, the pressure under the plate is taken a uniformly distributed. The slab is calculated as a plate, loaded with uniformly distributed pressure from below and simply supported on the foundation section of the rod elements and the base of the column (branches, traverses, diaphragms, ribs, etc.).

In accordance with the base structure, the plate may have areas, simply supported on four edges - contour 1, on three edges - contour 3, and console - contour 2 (Figure 32).

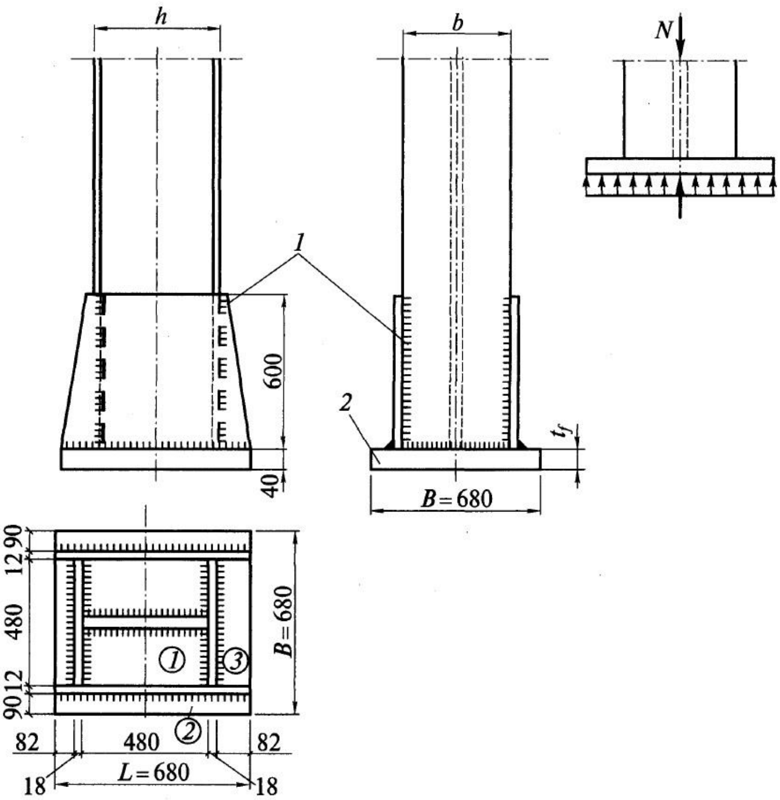

Figure 32. Calculation schemes of the column base (figures in circles – the contour number): 1) the traverses; 2) the slab base

7.2 The bases with the milled end of the column rod

At milled end of the column rod (Figure 33) a square plate is usually taken. As the slab overhangs are not strengthened, the slab sometimes turns of considerable thickness, thicker than a conventional rolled sheet (40- 50 mm). In this regard, perhaps the use of cast slabs.

For accurate fixing of a powerful column, baseplate conveniently installed separately using the three set screws (Figure 33, a).

After adjustment of the slab and filling it to the top edge of concrete, the rod of the column is set on it. The slab milled at the end of the column rod, works like a plate on an elastic foundation, perceiving pressure, concentrated in the area bounded by the rod (Figure 33, b).

The exact calculation that takes into account the actual spatial flexure of the slab for rectangular plate is very complicated, but it can be simplified by replacing the rectangular slab and the section of the column equal of them in the area of circles (Figure 33, c).

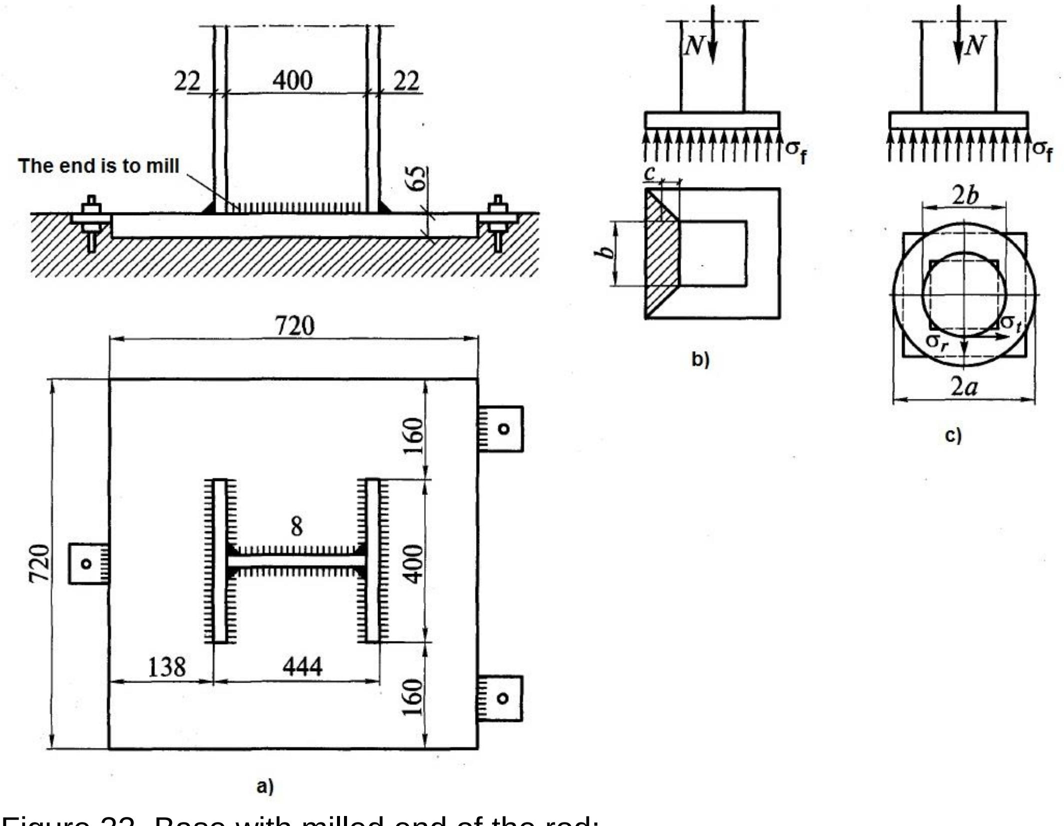

Figure 33. Base with milled end of the rod:

a) structural scheme; b, c)- calculation scheme of the slab

It possible to determine the bending moment in the plate on the column edge, considering the trapezoidal section of the console plate with the width b (at the connection with the column):

$$M = \sigma_f * A * c, \tag{6.1}$$

where σ_f - tension in the basement under the base plate; A - trapezoid area, shaded in Figure 33, b; c - the distance from the center of gravity of the trapezium to the edge of the column.

8 The farm connections of the paired parts

The farm with the rods of the two parts, made up of T-profile, the connections on the gussets design, which is placed between the corners. The rods are attached to the lattice gusset flag seams (Figure 34). The force in the element distributed between the seams on the butt and corner of feather is inversely proportional to their distance from the axis of the rod.

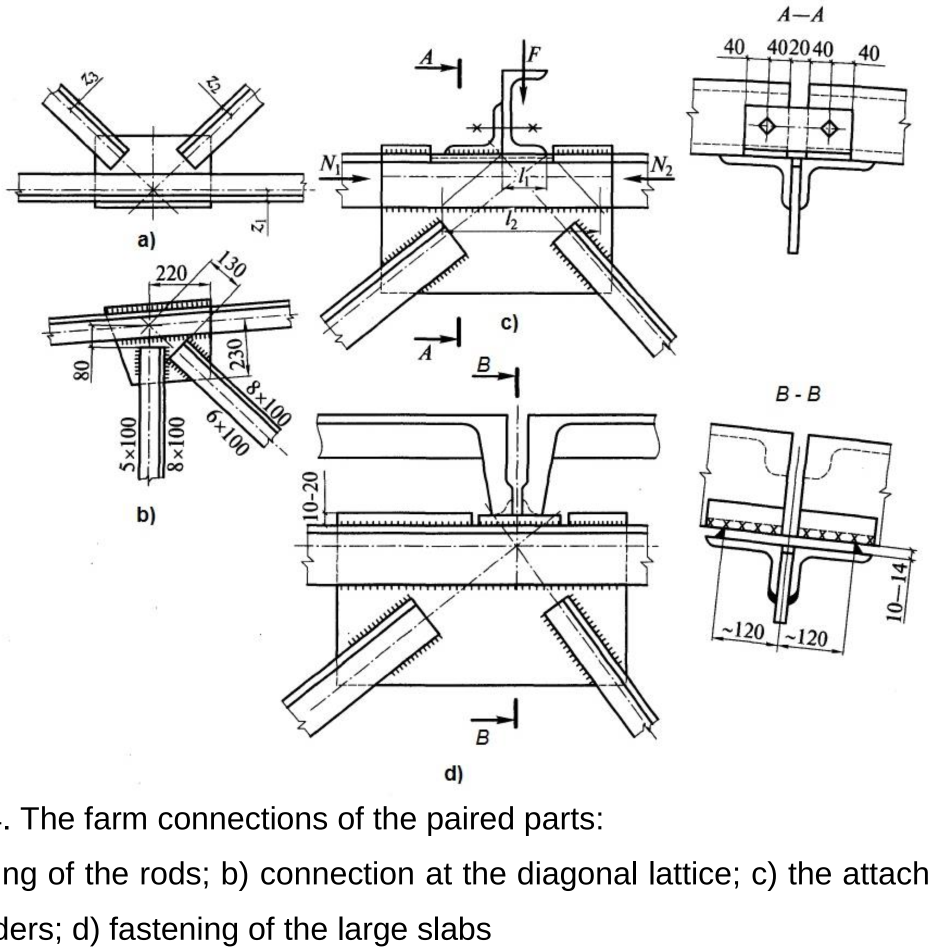

Figure 34. The farm connections of the paired parts:

a) centering of the rods; b) connection at the diagonal lattice; c) the attachment of the girders; d) fastening of the large slabs

The design of the support units depends on the type of supports (metal or concrete columns, brick walls, etc.) and the method of connection (hinged or rigid). When free support farms on the underlying structure, a possible solution for the support unit is shown in Figure 35. The farm pressure F_R through the plate is transferred to the support. The area of the slab A_{sl} defined by the bearing capacity of the support material.

The pressure from the farm to the support slab is transmitted through the gusset and support column, forming a rigid support of cross-section.

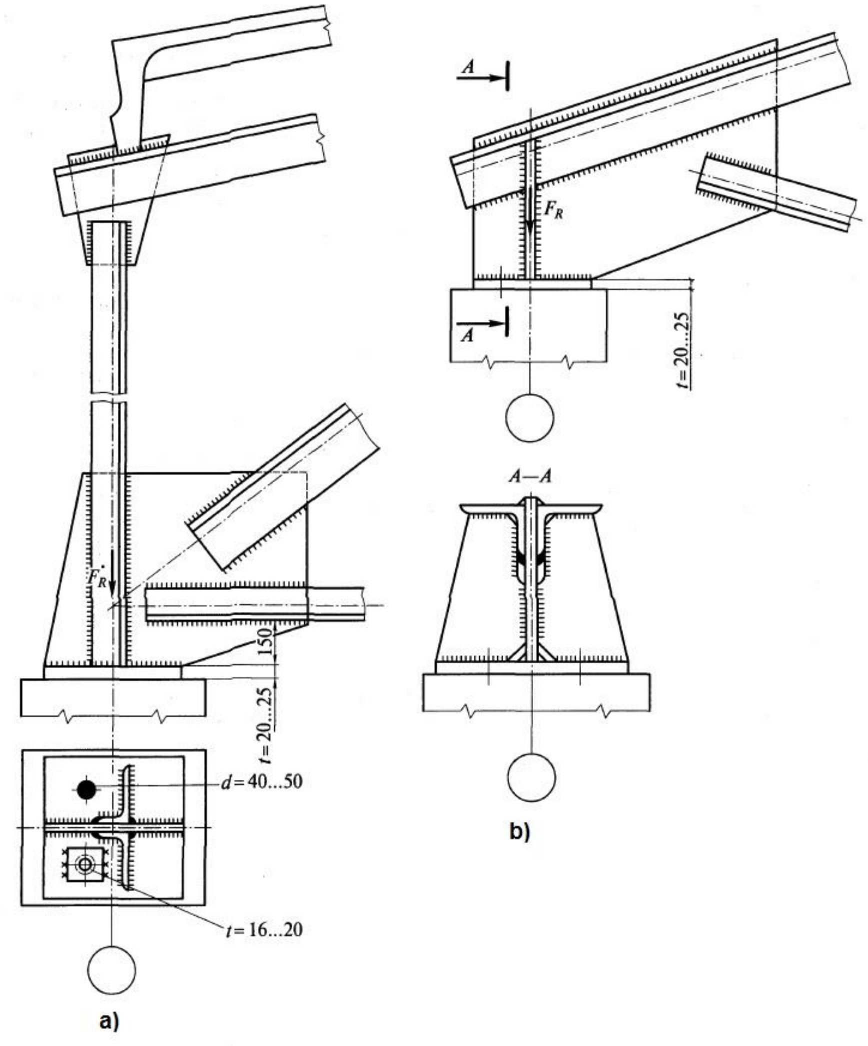

Figure 35. Supporting farm connections of the paired parts: a) bearing on the level of the lower chord; b) the same as the upper chord

Truss rigid support, typical for Finland is shown in Figure 36.

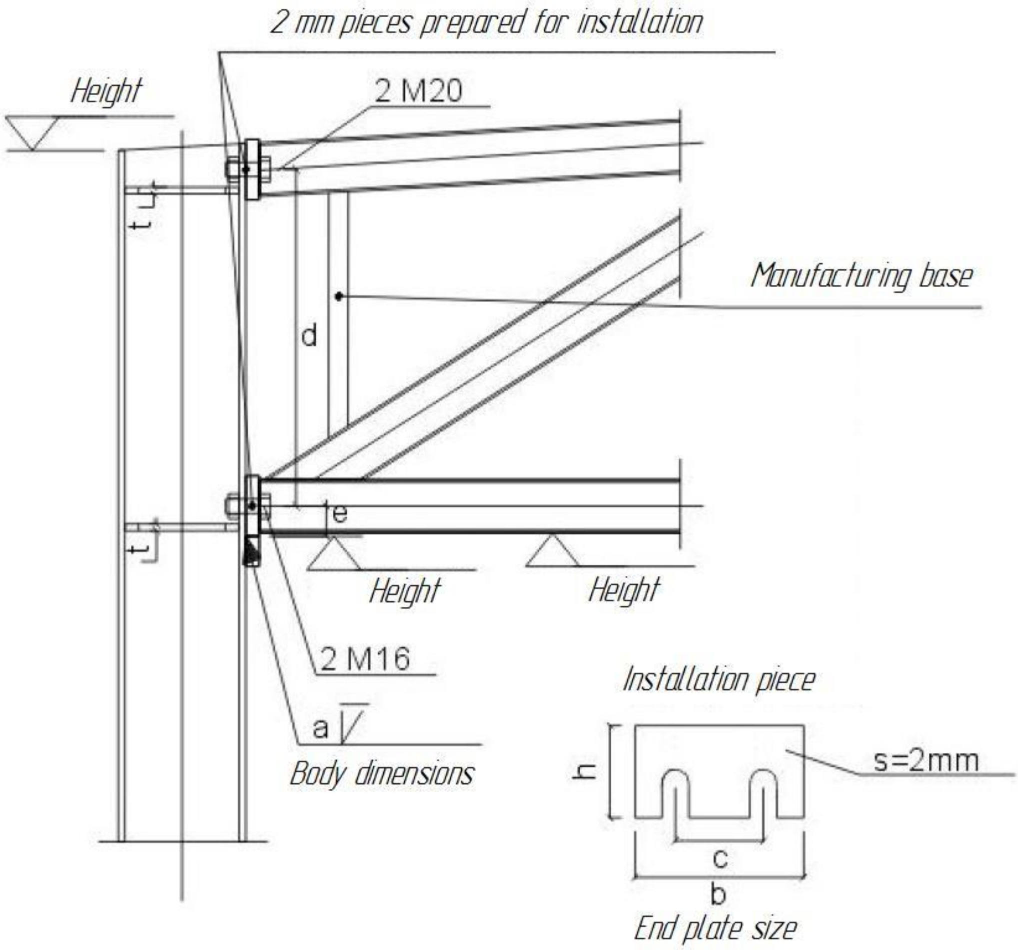

Figure 36. Truss rigid support

9 Bearing joints of crane beams and columns

In the columns of constant height section crane beams and other structures are based on the special console (Figure 37). When light-duty cranes are used console with one wall, welded to the rod of the column at the factory (if the dimensions allow transportation). Console and seams of its fastening to the column count on the bending moment M.

When transferring a great effort arrange a two-tier console (Figure 37, c).

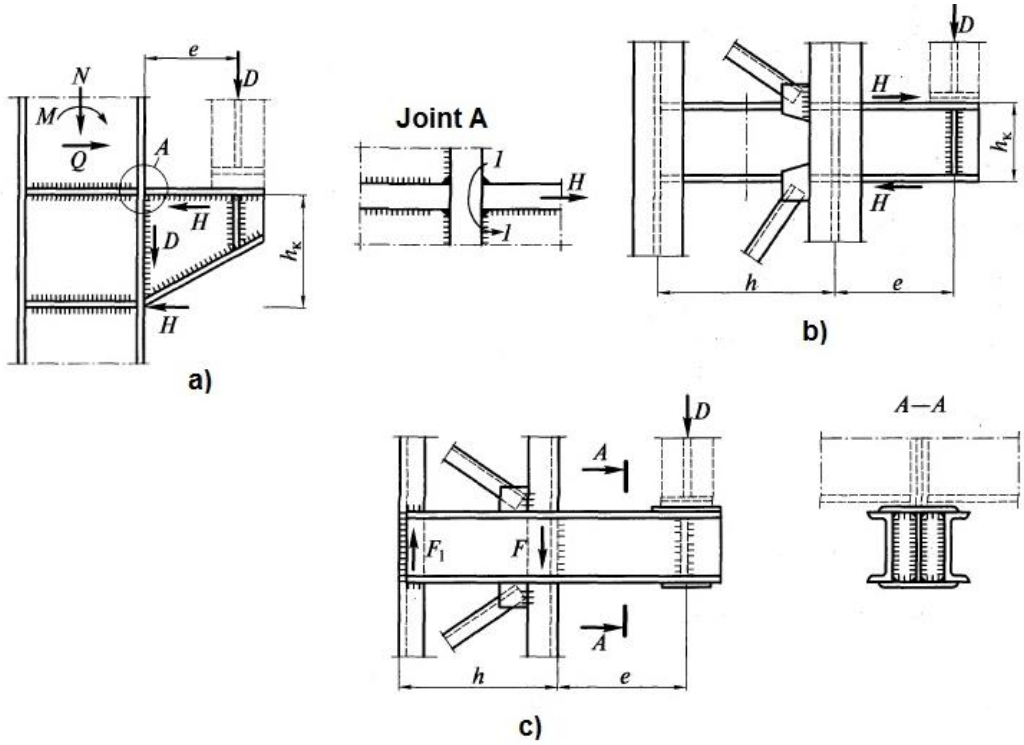

Figure 37. Crane consoles:

a) with the continuous columns; b) with the lattice columns; c) a two-tier console

In the stepped columns crane beams supported by the ledge of the column. To transfer the forces from the top of the column and of crane beams on the lower part in place of the ledge traverse is arranged (Figure 39). When transferring of force through the milled surface, the wall traverse is working to crumple.

Rail beam cantilever crane, typical for Finland is shown in Figure 38.

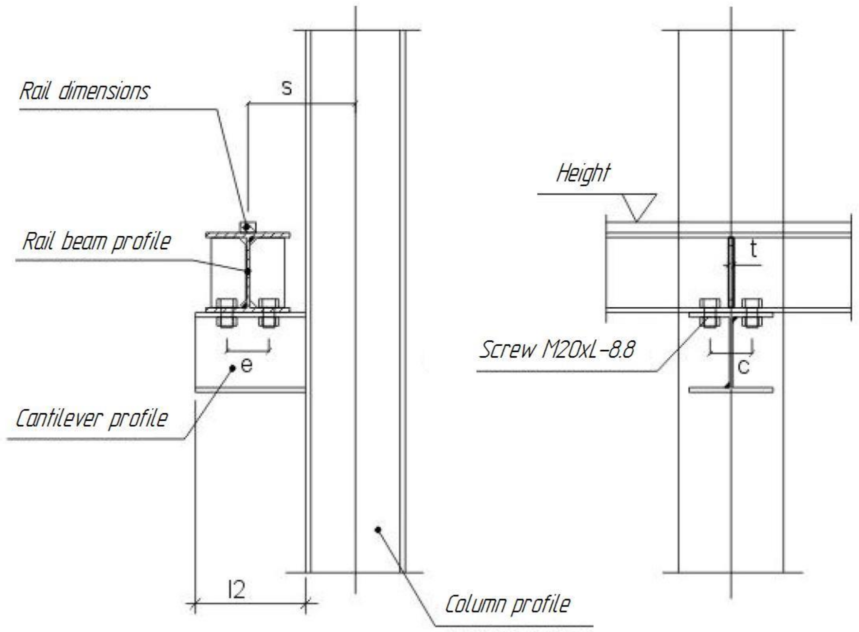

Figure 38. Rail beam cantilever crane joint

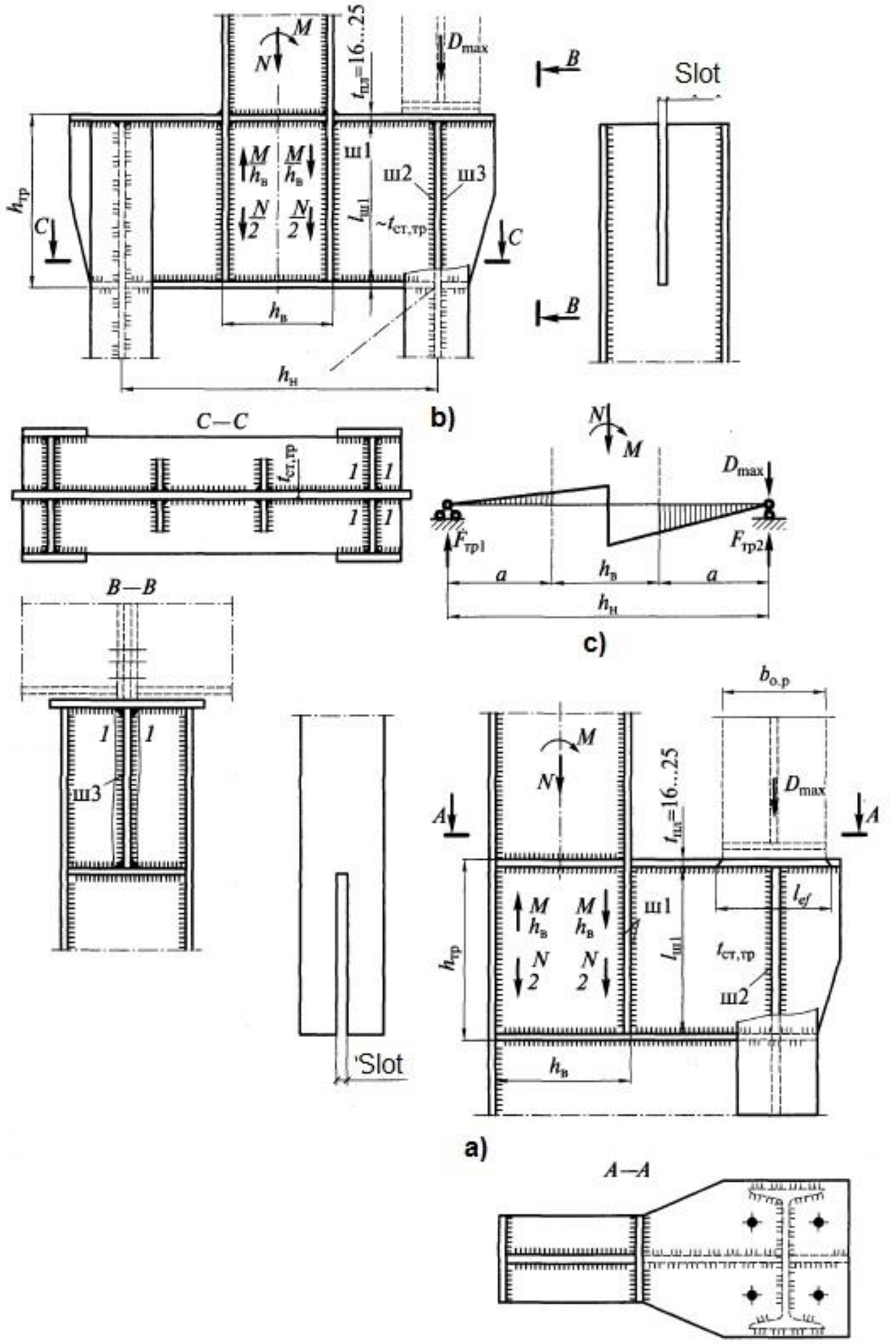

Figure 39. Connections of connection of upper and lower parts of columns: a) continuous; b) lattice; c) design scheme of the traverses

10 Conclusion

As a result of the thesis:

1. The history of development of steel structures in Russia and Finland was studied and history of standardization as well.
2. The modern methods of using steel in construction process were considered. Special attention was paid to the steel connections.
3. The main connection joints, which is often using in Finland and Russia, was demonstrated with the load and stress distribution on them.
4. The joints main design principles and methods according to actual European and Russian norms were considered.

Farm connections and bearing joints of crane beams and columns should be examined in more detail, but overall analysis indicated that the connections are almost identical. The main differences consists in the difference between the coefficients used in the calculations, as well as the classification of different ranges of auxiliary members. Another important difference is that in Finland welding and painting on-site is not applied, while it is very often used in Russia.

References

1. Y.I. Kudishin, E.I. Belenya, V.S. Ignateva and others. "Metal structures", 2007. 688 pp.
2. Mark Lawson, Artem Bilyk. “Steel structures in architecture”, 2015. 140 pp.
3. EN 1993 “Design of steel structures” Part 1-8: “Design of joints”
4. SP 16.13330.2011 “Steel structures”
5. SNiP 2-23-81 “Steel structures”
6. Lira soft - http://www.lira.com.ua/service/knots.php
7. EN 1990 “Basis of structural design”
8. http://gostrf.com/normadata/1/4293846/4293846982.pdf
9. http://eurocodes.jrc.ec.europa.eu/doc/2014_07_WS_Steel/presentations/07_Eurocodes_Steel_Workshop_WEYNAND.pdf