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An analysis of frame type steel constructions

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Abstract
One of the most frequently used subassembly in constructions is the simple or multiple stage frame. It is this subassembly that makes the basis of most strength structures in steel constructions.
The basic structure's strength of a steel construction correlates with many factors. Welding mountings and the restrain type of structure are part of the most important elements that influence the strength of the compounding structures.
This paper generates a physico-mathematical example of steel frames strength. Starting from this example, a study of the influence of the welding mountings and of the couplings is being made.
This essay not only that analyses the sensitivity of the welding mountings unto the way of frame type steel construction loading, but also makes practical recommendations for the projection stage.
Key words: steel construction, strength, deformations, stiffness, moulding.

1. Introduction

The presence of a "cellule", that is more often than not called "frame", can be found in most study-cases based on steel constructions. The simplest schematization of a frame consists of one beam and two columns, as illustrated in figure 1. The complexity of studying this structure strength depends on the type of loading (point load - figure 1 a, distributed load - figure 1.b). and of the types of support (restrain fig. 1.a and 1.b, free bearing fig. 1.c, supported fig. 1.d).

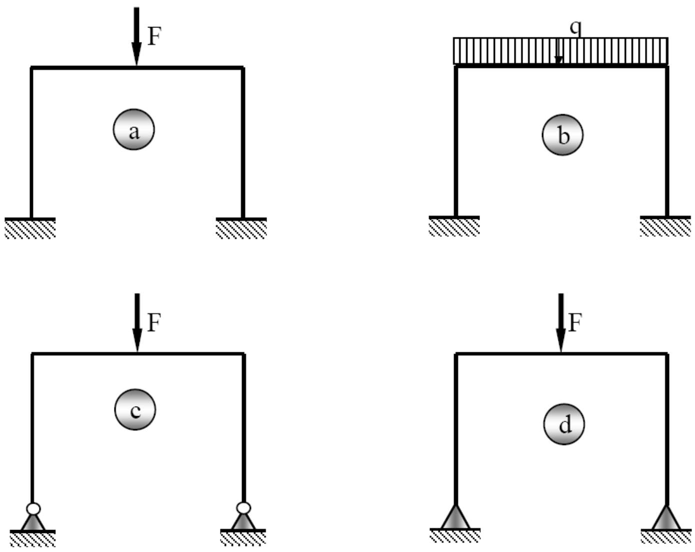

Fig.1

The bulk of computational process when establishing the structure's strength depends first and foremost on the support and load hypothesis. More often than not, the frame type resistance structure becomes more difficult to calculate, depending on the existence of various spans and levels. In order to make the analysis of real frame type steel constructions be more accessible, a simplification of diagrams is often used. The simplified forms of real frame type steel constructions may generate difficulties when the calculation includes the real geometry of the cross section of elements, more often than not the section being variable along the structure.

In order to simplify the moulding process, the real elements of the structure are excluded, the components of the steel construction being considered to have a constant section. This simplification hypothesis is considered to be correct by the specialists, the accuracy of the results being accessible (acceptable for strength calculation).

As a matter of fact, this hypothesis (constant section) is not frequently used when dealing with columns that have a stiffened base with gusset. Moreover, the superior flange of the columns is stiffened in the same manner as the base. The beams don't accomplish the hypothesis either, especially when the construction is superposed and the beams are supported at ends (on columns) through arms with high stiffness. We are going to analyze and quantisize the influence of the section variation along the building elements (a real situation) on the structure strength, comparing it to an ideal situation (the simplifier hypothesis of the constant section).

2. Design Assumption

2.1. Loadings

Two loading situations will be tackled upon:

- Hypothesis I1: single force "F" applied at the half of the spam with "l" length of the beam (fig. 1.a);
- Hypothesis I2: distributed force q=F/l on the whole length l of the beam (fig. 1.b).

2.2. The Geometry of the Structure

In order to easily emphasize the different influences, the dimensions of the frame type structure will be asserted according to the columns' height h (as illustrated in fig. 2), as it follows:

- the length of the beam: $L = 2l = 2\lambda H$; $L = 2l = 2\lambda h$

- the length of the variable thickness of the column: $h_0 = kH$, $k<1$; $h_0 = kh$ $k < 1$

- the length of the variable thickness of the column: $l_0 = kl = k\lambda H$, $\lambda>1$, λ and k are constructive parameters.

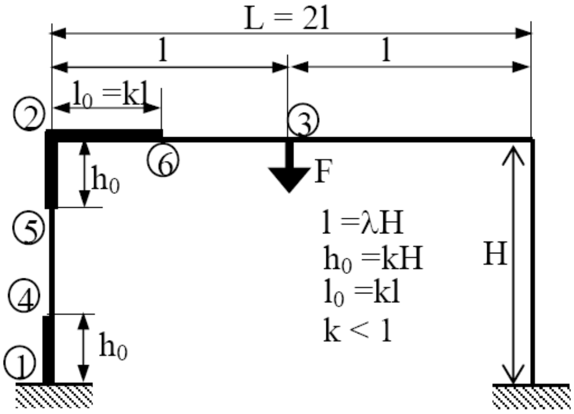

Fig.2 Building element

2.3. The Geometry of the section

The section has the following characteristics: the geometrical form, the area, the position of the centre of gravity, the moment of inertia and the axial section modulus. For a front analysis we will consider only the bending process; we will be interested only in the axial section modulus.

The modulation of the section may have various forms; the variation may be defined by imposed functions or my result from an analytical interpretation of a constructive solution. For want of space, we will focus only on the variation functions of geometrical parameters that are used for constructive defined sections, sections that are dealt with in the projection practice of steel constructions.

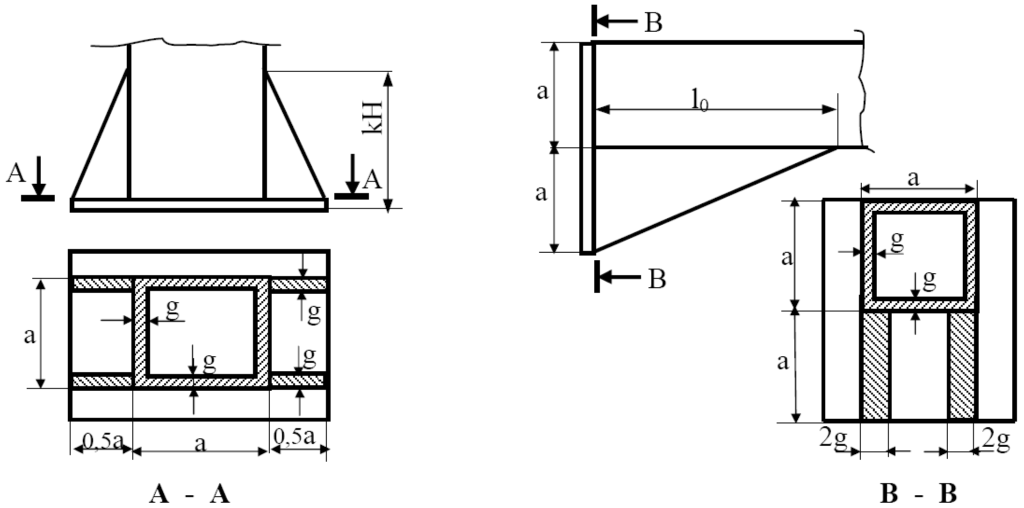

Fig.3 Schematization of the constructive solution caisson stiffened with gusset type

We frame the use of rolled stock of caisson type, stiffened with gusset (as illustrated in fig. 3). The geometrical characteristics of the sections (moment of inertia “I” and section modulus “W”) are calculated as it follows (adding the notations in fig.3):

- the beam and the column in the not stiffened area:

$$I = \frac{a^4}{12} - \frac{(a-2g)^4}{12}, \quad W = \frac{I}{0,5a} = \frac{a^3}{6} - \frac{(a-2g)^4}{6a}$$

- the beam in the maximum rigidity area:

$$z_c = \frac{a(4a-3g)}{2(2a-g)}, \quad h_m = \frac{a(a-g)}{2(2a-g)}, \quad d_1 = \frac{a^2}{2a-g}, \quad d_2 = \frac{a(a-g)}{2a-g},$$

$$I = \frac{a^4}{12} - \frac{(a-2g)^4}{12} + \frac{ga^3}{3} + \left[a^2 - (a-2g)^2\right]\cdot d_1^2 + 4ag \cdot d_2^2 =$$

$$= \frac{g}{3(2a-g)} \cdot \left(18a^4 - 27ga^3 + 22g^2a^2 - 16g^3a + 4g^4\right) = I_2,$$

$$W = \frac{I}{h_m} = \frac{2g}{3(4a-g)}\left(18a^4 - 27ga^3 + 22g^2a^2 - 16g^3a + 4g^4\right) = W_2;$$

- the column in the maximum rigidity area:

$$I = \frac{g}{6}\left(11a^3 - 12ga^2 + 16ag^2 - 8g^3\right) = I_1 = I_{2s} \ ,$$

$$W = \frac{I}{a} = \frac{g}{6a}\left(11a^3 - 12ga^2 + 16ag^2 - 8g^3\right) = W_1 = W_{2s} \ ,$$

- moment of inertia in the current section from the underside of the column:

$$I(x) = \frac{g}{24k^3h^3}\left(29k^3h^3a^3 - 48k^3h^3a^2g + 64k^3h^3ag^2 - 32k^3h^3g^3 - \right.$$
$$\left. - 27k^2h^2a^3x + 18k\,h\,a^3x^2 - 4a^3x^3\right) = I_{12}(x)$$

- moment of inertia in the current section from the topside of the column:
- (x=0, as illustrated in fig. 4)

$$I(x) = \frac{g}{6k^3h^3}\left(4k^3h^3a^3 - 12k^3h^3a^2g + 16k^3h^3ag^2 - 8k^3h^3g^3 + \right.$$
$$\left. + 3k^2h^2a^3x + 3k\,h\,a^3x^2 + a^3x^3\right) = I_{52}(x);$$

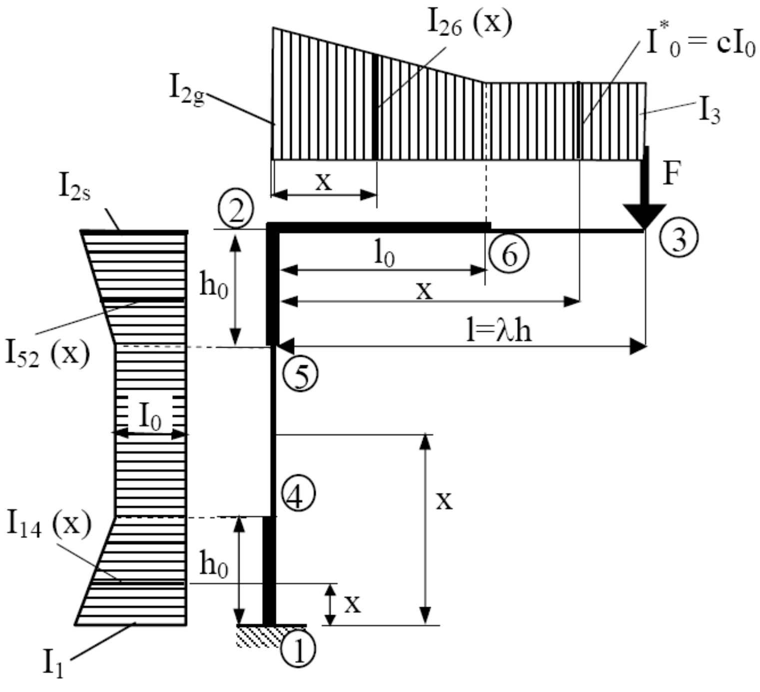

Fig.4 The assignation diagram of moments of inertia on the frame

- moment of inertia on the beam, in the stiffened area:

$$A_1 = 4g\,(a-g), \quad A_2 = \frac{4ag\,k\lambda\,h}{k\lambda\,h - x}, \quad y = \frac{k\lambda\,h\,a}{k\lambda\,h - x},$$

$$z_1 = 1{,}5a, \quad z_2 = \frac{a(k\lambda\,h - 2x)}{2(k\lambda\,h - x)}, \quad d_1 = z_1 - z_c, \quad d_2 = z_c - z_2 \ ,$$

$$z_c = \frac{a\left[3(a-g)x^2 - 2(4a-3g)k\lambda\,h\,x + (4a-3g)k^2\lambda^2\,h^2x^2\right]}{2\left[(2a-g)k\lambda\,h - (a-g)x\right](k\lambda\,h - x)},$$

$$I(x)=\frac{a^4}{12}-\frac{(a-2g)^4}{12}+\frac{ay^3}{3}+A_1d_1^2+A_2d_2^2=I_{26}(x)$$

$$\begin{aligned}I(x)=&\left[2\left(a^4g-4a^3g^2+7a^2g^3-12ag^4+4g^5\right)x^4+\right.\\&+\lambda k\,h\left(-13a^4g+41a^3g^2-64a^2g^3+52ag^4-16g^5\right)x^3+\\&+\lambda^2k^2\,h^2\left(33a^4g-81a^3g^2+108a^2g^3-84ag^4+24g^5\right)x^2+\\&+\lambda^3k^3\,h^3\left(-a^5-37a^4g+74a^3g^2-80a^2g^3+60ag^4-16g^5\right)x+\\&\left.+\lambda^4k^4\,h^4\left(2a^5+15a^4g-26a^3g^2+22a^2g^3-16ag^4+4g^5\right)\ \right]\bullet\\&\bullet\frac{1}{3(k\lambda\,h-x)^3\left[k\lambda\,h(2a-g)-(a-g)x\right]}=I_{26}(x);\end{aligned}$$

integration coefficients, on ranges:

$$c_{14}(x)=\frac{I_0}{I_{14}(x)}=\frac{16k^3h^3(a-g)\left(a^2-2ag+2g^2\right)}{\left(-4a^3\right)x^3+\left(18kha^3\right)x^2-\left(27k^3h^3a^3\right)x+k^3h^3\left(29a^3-48a^3g+64ag^2-32g^3\right)}$$

$$c_{52}(x)=\frac{I_0}{I_{52}(x)}=\frac{4k^3h^3(a-g)\left(a^2-2ag+2g^2\right)}{a^3x^3+\left(3k\,ha^3\right)x^2+\left(3k^2h^2a^3\right)x++4k^3h^3\left(a^3-3a^2g+4ag^2-2g^3\right)}$$

$$c_{26}(x)=\frac{I_0}{I_{26}(x)},\quad I_0=\frac{2}{3}g\,(a-g)\left(2g^2-2a\,g+a^2\right),\qquad c_{45}(x)=1$$

$$I_0^*=\frac{2}{3}g_1\,(a_1-g_1)\left(2g_1^2-2a_1\,g_1+a_1^2\right),\qquad c_{63}(x)=\frac{I_0}{I_0^*}.$$

a_1, g_1, a, g – geometrical characteristics of the beam section, and respectively of the column section.

3. Mechanical efforts

3.1. Built in structure

The diagram is represented in figure 7. The structure is twice statically undeterminated (unknown X_1 and X_2). In order to calculate the uncertainty, the method of efforts will be applied, considering only the bending strain.

The present equations resulted from the real and unit loadings are the following (as illustrated in figure 7):

- For real loadings:

$M_{140}=M_{450}=M_{520}=0,$

$M_{260}=M_{630}=V\cdot x=0{,}5Fx,$

- For unit load X_1=1:

$$m_{141}=m_{451}=m_{521}=-1,\qquad m_{261}=m_{631}=-1;$$

- For unit load X_2=1:

$$m_{142}=m_{452}=m_{522}=-x,\qquad m_{262}=m_{632}=-h.$$

When dealing with constructive defined stiffness (caisson type and I-strut), the current section on range 5 – 2 sets in relation to point 5 (as illustrated in fig. 5), the equation being modified as it follows:

$$m_{522} = -[(1-k)\cdot h + x]; \quad x \in [0, kh];$$

the defined limits of range are:

$$x_{14} \in [0, \quad h_0], \quad h_0 = kh, \qquad x_{45} \in [h_0, \quad h - h_0], \qquad x_{52} \in [h - h_0, \quad h],$$

$$x_{26} \in [0, \quad l_0], \quad l_0 = kl, \qquad x_{63} \in [l_0, \quad l].$$

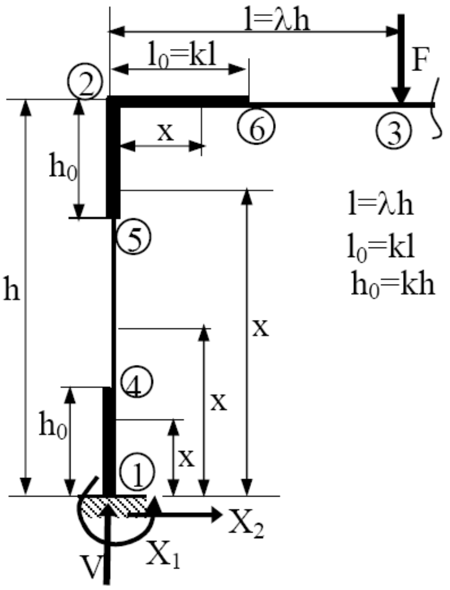

Fig. 5 The efforts diagram for built in structure

The canonical equations system is:

$$d_{11}X_1 + d_{12}X_2 + d_{10} = 0,$$

$$d_{21}X_1 + d_{22}X_2 + d_{20} = 0.$$

The system coefficients are calculated as it follows (using this kind of relations):

$$d_{ij} = \int_l \frac{I}{I_0} m_i\, m_j dx = \int_0^{h_0} c_{14}(x) m_{14i} m_{14j}\, dx + \int_{h_0}^{h-h_0} c_{45}(x) m_{14i} m_{14j} dx +$$

$$+ \int_{h-h_0}^{h} c_{52}(x) m_{14i} m_{52j} dx + \int_0^{l_0} c_{25}(x) m_{25i} m_{25j} dx + \int_{l_0}^{l} c_{53}(x) m_{25i} m_{25j} dx,$$

$$d_{i0} = \int_l \frac{I}{I_0} m_i \cdot M_0 dx = \int_0^{l_0} c_{26}(x) m_{26i} \cdot M_{260} dx + \int_{l_0}^{l} c_{63}(x) m_{26i} \cdot M_{260} dx.$$

The calculation of the coefficients relations results from the integration:

$$d_{11} = \frac{2k}{k_1 - 1} H \ln\left(k^2 k_1 H^2\right) + (1-k)H + \frac{k}{(k_2 - 1)c} H \ln k_2 + \frac{H}{c}(\lambda - k),$$

$$d_{12} = \frac{k}{k_1 - 1} H^2 \ln k_1 + \frac{k}{(k_2 - 1)c} H^2 \ln k_2 - 0{,}5(2k-1)H^2 + \frac{(\lambda - k)}{c} H^2,$$

$$d_{22} = \frac{kH^3}{(k_1 - 1)^3}\{k(1-k_1)(3k_1 k - 2k_1 - k + 2) + \ln k_1[(k_1 - 1)^2 + 2kk_1(kk_1 -$$

$$- k_1 + 1)]\} + \frac{(\lambda - k)H^3}{c} + \frac{k \ln k_2 H_3}{(k_2 - 1)c} + \frac{(1-2k)(k^2 - k + 1)H^3}{3},$$

$$d_{10} = -\frac{H^2F}{4c(k_2-1)^2}\left[\left(1-k_2^2+2k_2\ln k_2\right)k^2+(k_2-1)^2\lambda^2\right],$$
$$d_{20} = -\frac{H^3F}{4c(k_2-1)^2}\left[\left(1-k_2+2k_2\ln k_2\right)k^2+(k_2-1)^2\lambda^2\right]=Hd_{10}.$$

In order to solve the canonical system, the determinants are defined:

$$D_0 = \begin{vmatrix} d_{11} & d_{12} \\ d_{12} & d_{22} \end{vmatrix}, \quad D_1 = \begin{vmatrix} -d_{10} & d_{12} \\ -d_{20} & d_{22} \end{vmatrix}, \quad D_2 = \begin{vmatrix} d_{11} & -d_{10} \\ d_{12} & -d_{20} \end{vmatrix};$$

redundant reactions are:

$$X_1 = \frac{D_1}{D_0}, \qquad X_2 = \frac{D_2}{D_0}.$$

Real moment equations on ranges are:

$$M_{12} = m_{141}X_1 + m_{142}X_2, \qquad M_{23} = M_{260} + m_{261}X_1 + m_{262}X_2.$$

3.2. Swiveling Structure

The diagram is illustrated in fig.6. The structure is a redundant date. Moment equations in the basic system are:

$$M_{120} = 0, \quad m_{141} = m_{451} = m_{521} = -x,$$
$$M_{230} = 0{,}5F\,x, \quad m_{261} = m_{631} = -h.$$

The canonical equation is: $d_{11}X_1 + d_{10} = 0.$

Coefficients are calculated in a manner similar to the previous equations, as it follows:

$$d_{11} = \int_l \frac{I}{I_0}m^2dx = \frac{kH^3}{(k-1)^3}\left\{\ln k_1\left[2k_1k\left(k_1k-k_1+1\right)+(k_1-1)^2\right]-\right.$$
$$\left.-k(k_1-1)(3k_1k-2k_1-k+2)\right\}+\frac{k\ln k_2H^3}{(k_2-1)c}-(2k-1)\left(k^2-k+1\right)\frac{H^3}{3}-\frac{(k-\lambda)H^3}{c},$$
$$d_{10} = \int_l \frac{I}{I_0}m\cdot M_0\,dx = \frac{FH^3}{4c(k_2-1)^2}\left[k^2\left(k_2^2-k_2\ln k_2-1\right)-\lambda^2(k_2-1)^2\right].$$

The redundant reaction is: $X_1 = -\dfrac{d_{10}}{d_{11}},$

and real moment equations on the structure are:

$$M_{12} = m_{141}X_1, \quad M_{23} = M_{250} + m_{251}X_1.$$

3.3. Supported structure

The diagram is illustrated in fig. 6 (the structure is statically determinate). Flexure efforts vary according to the following relations: $M_{12} = 0$, $\quad M_{23} = 0{,}5F\,x$.

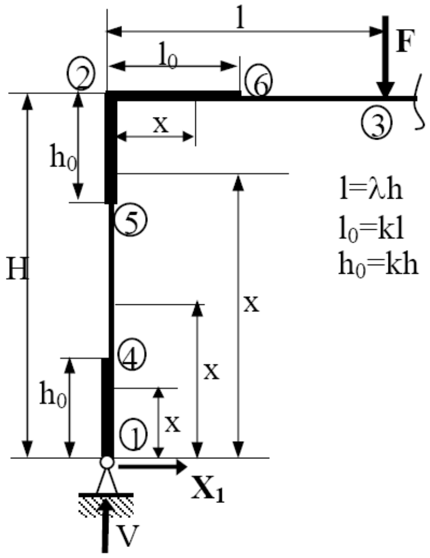

Fig.5 Efforts diagram in case of a swiveling structure

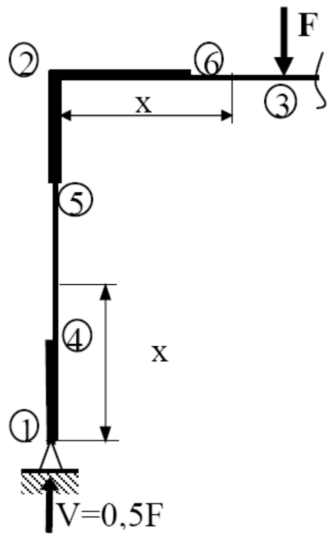

Fig.6 Efforts diagram in case of a supported structure

4. Initial Structure

We consider the constant section for the columns and also for the beam. In this case, we calculate the efforts and the sections for the two loading cases and under the three conditions of support. The analysis is similar to the one in chapter 3.

4.1. Built in structure

Moment equations in the basic system are:

$$M_{120} = 0, \quad m_{121} = -1, \quad m_{122} = -x,$$
$$M_{230} = 0{,}5Fx, \quad m_{123} = -1, \quad m_{232} = -H.$$

The coefficients of the canonical equation are:

$$d_{11} = \int_0^H m_{121}^2\,dx + \frac{1}{c}\int_0^l m_{231}^2\,dx = H\left(1 + \frac{\lambda}{c}\right),$$

$$d_{12} = \int_0^H m_{121}\cdot m_{122}\,dx + \frac{1}{c}\int_0^l m_{231}\cdot m_{232}\,dx = H^2\left(\frac{1}{2} + \frac{\lambda}{c}\right),$$

$$d_{22} = \int_0^H m_{122}^2\,dx + \frac{1}{c}\int_0^l m_{232}^2\,dx = H^3\left(\frac{1}{3} + \frac{\lambda}{c}\right),$$

$$d_{10} = \frac{1}{2}\int_0^l M_{230}\,m_{231}\,dx = -\frac{\lambda^2 H^2 F}{4c}, \qquad d_{20} = \frac{1}{2}\int_0^l M_{230}\,m_{232}\,dx = -\frac{\lambda^2 H^3 F}{4c}.$$

The redundant reactions are:

$$X_1 = -\frac{\lambda^2 H\,F}{2(c+4\lambda)}, \qquad X_2 = \frac{3\lambda^2 F}{2(c+4\lambda)}.$$

The moment equations are:

$$M_{12} = -\frac{\lambda^2 H\,F}{2(c+4\lambda)} - \frac{3\lambda^2 F}{2(c+4\lambda)}\cdot x, \quad M_{23} = 0{,}5F\,x + \frac{\lambda^2 H\,F}{2(c+4\lambda)} - \frac{3\lambda^2 FH}{2(c+4\lambda)}.$$

Maximum deflection is (m=0,5 x):

$$f_m = \int_0^{\lambda H} M_{23}\, m\, dx + \int_0^{H} M_{12} m\, dx = \frac{\lambda^3 H^3 F\left(2c+2\lambda-6\lambda^2 c+3\lambda c\right)}{12c(c+4\lambda)EI}.$$

4.2. Swiveling Structure

The structure is a redundant date; the parameters have the same significance and are calculated in the same manner as in chapter 3, as it follows:

$$d_{22} = H^3\left(\frac{1}{3}+\frac{\lambda}{c}\right), \quad d_{20} = -\frac{\lambda^2 H^3 F}{4c}. \quad X_2 = \frac{d_{20}}{d_{22}} = \frac{3\lambda^2 F}{4(c+3\lambda)}. \quad M_{12} = -\frac{3\lambda^2 F\cdot x}{4(c+3\lambda)},$$

$$M_{23} = 0{,}5F\,x + \frac{\lambda^2 H^2 F}{2(c+4\lambda)}. \qquad f_m = \frac{\lambda^3 H^3 F\left(2c+8\lambda+3\lambda\,H-6\lambda^2 c+3\lambda c\right)}{12c(c+4\lambda)EI}.$$

4.3. Supported Structure

The parameters previously mentioned are: $M_1 = 0$, $M_{23} = 0{,}5Fx$, $f_m = \dfrac{\lambda^3 H^3 F}{12c\;EI}$.

5. Longitudinal Seam Strength

5.1. Symmetric Section

The diagram of beads voltages is rendered in figure 7.
Local grip resistance in the current section is:

$$F(x) = \frac{T\cdot S(x)\cdot dx}{I(x)},$$

and the tangential stress in welds is:

$$\tau(x) = \frac{F(x)}{A_s} = \frac{1}{n\varepsilon}\,\frac{T(x)\cdot S(x)}{I(x)}.$$

The following notations are made:

- τ (x) – mechanical strain in the bead,
- T (x) – cutting force in the current section,
- S (x) – the statical moment of that part of the current section that tends to slide,
- I (x) – moment of inertia of the current section,
- n – number of beads,
- ε - weld width.

The static moment is: S(x) = A(x)·d(x);

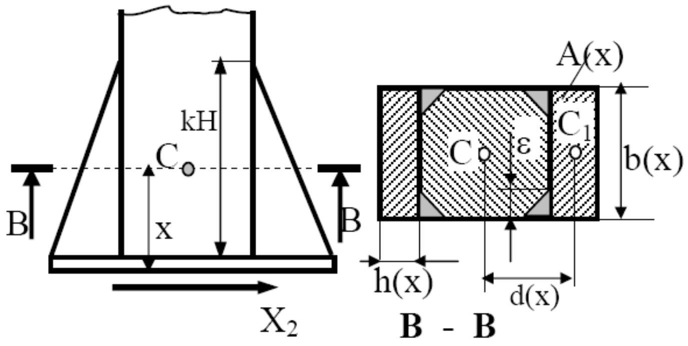

Fig. 7 Diagram of the beads produced in symmetric sections

The following notations have been used (according to fig. 7): A(x)– area of the section with longitudinal course gliding, in the current section, d(x)– distance between the C_1 centre of gravity of the element A (x) and the C centre of gravity of the entire section.

5.2. Asymmetrical Sections

The diagram is illustrated in fig. 8. Tangential stress in the beads is calculated using the relation from chapter 5.1, and the implicit parameters have the same significance, according to fig. 8.

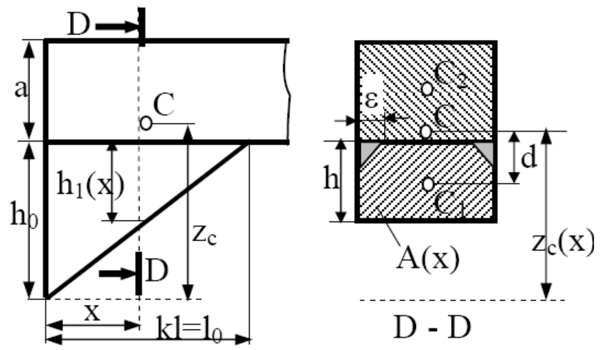

Fig. 8 Schematization of the beads produced in asymmetrical sections

5.3.Exemplifications

The focus is on the caisson section stiffened with symmetric gussets, as illustrated in fig. 9, with the implicit notations. The necessary auxiliary parameters are calculated as it follows (considering the notations in fig. 9):

$$h_1 = 0{,}5a\frac{kH - x}{kH}, \quad b = 2g, \quad d = 0{,}5a + 0{,}5h_1 = \frac{a}{4}\cdot\frac{kH - x}{kH}, \quad A = 2g\,h_1 = a\cdot g\frac{kH - x}{kH},$$

$$S(x) = a\cdot d = \frac{a^2 g}{4}\cdot\frac{(3kH - x)(kH - x)}{(kH)^2},$$

$$I(x) = \frac{a^4}{12} - \frac{(a - 2g)^4}{12} + \left[\frac{g\,h_1^3}{12} + g\,h_1(0{,}5\,a + 0{,}5h_1)^2\right].$$

Tangential stress in the weld, for the current section, is calculated using the relation in chapter 5.1.

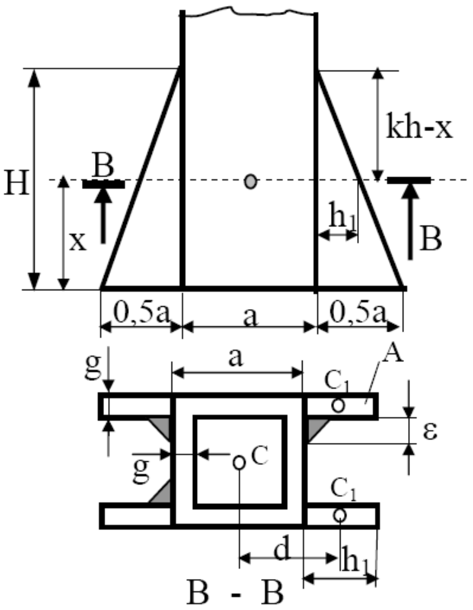

Fig.9 Diagram of the welds realized in symmetrical caisson section

Next we analyze the stress in the beads for the asymmetrically stiffened caisson section. The diagram of the beams voltages is illustrated in fig. 10. The necessary parametres in analyzing the current section are calculated as it follows:

$$h_2 = a\frac{l_0 - x}{l_0} = \frac{a(k\lambda H - x)}{k\lambda H}, \qquad z_c = \frac{1{,}5a\left[a^2 - (a-2g)^2\right] + 4g\,h_2\,(a - 0{,}5h_2)}{a^2 - (a-2g)^2 + 4g\,h_2},$$

$$A = 4g\,h_2, \qquad d = z_c - (a - 0{,}5h_2), \qquad S = A \cdot d, \qquad b = 4g,$$

$$I(x) = \frac{a^4}{12} - \frac{(a-2g)^4}{12} + \frac{g h_2^3}{3} + \left[a^2 - (a-2g_1)^2\right]\cdot(1{,}5a - z_c)^2 + 4gh_2 \cdot d^2 .$$

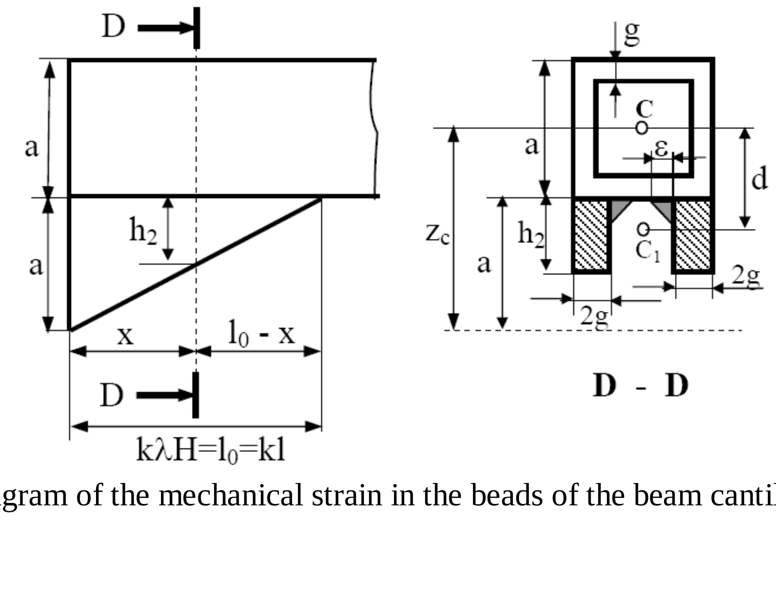

Fig. 10 Diagram of the mechanical strain in the beads of the beam cantilever

Tangential stress in the beads, in the current section, is calculated in the same manner as the symmetric section (the relation from chapter 5.1).

The use of the presented model is illustrated:

$$H = 3m, \quad \lambda = 1{,}5, \quad k = 0{,}5, \quad F = 100\ kN, \quad \sigma_a = 200\ MPa,$$

Due to the implementation of the conceived model, in the MathCad programming environment has resulted a means that allows the examination of a large number of constructive and loading versions. The three cases of support have been analyzed for the constructive caisson type version.

The calculation procedures suppose the following stages:

- defining the geometrical characteristics of the sections for each range, including their variation functions;
- calculating the uncertainty from the bearings using the efforts method;
- defining the variation functions of the bending stress and identifying the critical points and the maximum values;
- dimensioning the maximum loaded section on the beam and the standard section of the columns (sections 3 and 5);
- if the pillars are simple supported, their section is determined by the buckling criteria, using Euler relation and the buckling coefficient for a double checking;
- checking the stresses in all sections that are considered to be overstressed and resuming the entire calculation in case the resistance conditions are not proper (the calculation is iterative, in order to be finalized the resistance condition must be appropriate for all the required maximum sections);
- calculating the maximum axle on the beam, taking into account the variable sections and comparing it to the recommended axle;
- calculating the variation function of the tangential stress in the beads, not only on the beam, but also on the columns, by establishing the maximum values;
- the whole calculation procedure is repeated in order to be applied to a loaded frame that is supported in the same manner, but it is not reinforced (frame of reference).

Table 1 contains the results of using the automatical computational programme for the particular defined case. The significance of the notations in the table is:

a, g – the side and the thickness of the caisson of the beam;
a_1, g_1 – the side and the thickness of the caisson of the columns;
I_3, W_3 – moment of inertia, the axial section modulus of section 3;
I_4 – axial moment of inertia of section 4;
A_s, A_g – the area of the columns and the beam section;
X_1, X_2 – the statically undefined reactions resulted from the supporting frame;
M_3, M_2, M_1 – moments from sections 3, 2 and 1;

Table 1. Numerical results for F=100 kN, H=3 m, k=0.15, λ=1.5

The category of the parameters	The calculated parameter	Supporting case					
		Restrain		Swiveling		Bearing	
		Reinfor-ced frame	Frame of reference	Reinfor-ced frame	Frame of reference	Reinfor-ced frame	Frame of reference
Geome-trical characte-ristics of the sections	a (cm)	23	28	24	25	30	30
	g (cm)	1	1	1	1	1	1
	a_1 (cm)	25	25	26	26	16	16
	g_1 (cm)	0,5	0.5	0.5	0.5	0.4	0.4
	I_3 (cm^4)	7113	$1,3 \cdot 10^4$	8127	9232	16280	16280
	W_3 (cm^3)	598	939	639	738	1085	1085
	I_4 (cm^4)	4904	4908	5529	5529	-	-
	A_s (cm^2)	47	49	51	51	25	25
	A_g (cm^2)	88	108	92	92	116	116
	A_g/A_{go}	0,85	1	1	1	1	1
	A_s/A_{so}	0,96	1	1	1	1	1
Reactions and efforts	X_1(daNcm)	$6 \cdot 10^5$	$6,76 \cdot 10^5$	-	-	-	-
	X_2(daN)	5520	904	3238	2813	-	-
	M_3(daNcm)	$1,2 \cdot 10^6$	$1,8 \cdot 10^6$	$1,28 \cdot 10^6$	$1,4 \cdot 10^6$	$2,25 \cdot 10^6$	$2,25 \cdot 10^6$
	M_2(daNcm)	10^6	$4 \cdot 10^5$	$9.7 \cdot 10^5$	$8.4 \cdot 10^5$	0	0
	M_1(daNcm	$6 \cdot 10^5$	$6.7 \cdot 10^5$	0	0	0	0
	T_g (daN)	5000	5000	5000	5000	5000	5000
	T_s (daN)	5520	904	3238	2813	-	-
	τ_s(daN/cm^2)	1090	-	154	-	0	-
	τ_g(daN/cm^2)	110	-	110	-	87	-
Ceiling voltage (daN/cm^2)	σ_3	2000	1966	2000	1904	2073	2073
	σ_1	1070	720	0	0	0	0
	σ_{2s}	1880	1031	1600	1984	0	0
	σ_{2g}	934	430	788	1140	0	0
	σ_f	152	146	140	109	286	286
Arch (cm)	f	2,2	4.02	3.1	3.4	4.4	4.4
	f_o	1,8	1,8	1,8	1,8	1,8	1,8
	f/ f_o	1,2	2.2	1.7	1.9	2.47	2.47

T_g, T_s – the cutting force in the beam, and in the columns;
τ_s, τ_g – maximum tangential stress in the beads of the beam and columns;
σ_3, σ_1 – maximum stress in sections 3 and 1 (according to the diagrams);
σ_{2s}, σ_{2g} – maximum stress in section 2 on the columns and on the beam;
σ_f – buckling stress in the columns (checking with the buckling coefficient);
f, f_o – the maximum real arch and the maximum recommended arch on the beam.

6. Conclusions

The analysation of the loadings from welding joints of the frame components implies the determination of the efforts resulted from the structure. This operation is more often than not difficult because of the multiple statically undeterminables. The difficulties amplify if the variable thicknesses of the girders are taken into account. This is why the practice has established the reductionistic hypothesis regarding the continuous section.

If the real sections are used, the variation laws of the moment of inertia must be defined. The calculation becomes more difficult and complex due to the book keeping of the ranges and variation laws of the specific parameter. It has been acknowledged that for a single loading, the mathematical calculation is quite bulky; therefore the procedure is not a simple one. In order to make this method accessible for the engineer, automatic computer is used. However, this programme is hard to use if the operator doesn't make efforts to work with it.

The exemplification presented in this paper points out on the one hand the viability of this model, and on the other hand emphasizes the difficulty implied by its application. The determined parameters permit the quantification of the strength structure behavior when correlated with different constructive and functional aspects.

For instance, the dimensions of the sections (and also the quantity of building material that has been used) are highly influenced by the type of the supported structure. The same argument can be used for the structure's stiffness, the ground coupling being very important.

In order to practically apply the study, more loading hypothesis and multiple constructive solutions must be analyzed. It is only by comparing the various calculated parameters, that the positive and the negative influences can be detected. For lack of space, this brief study may seem less viable. However, the authors' concern with this issue is much larger. This is why they can assert the utility of simulations as the one described in this paper. Moreover, automatic programmes with wide usage for the engineers in welding assembled steel constructions can thus be generated.

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