



SIMPLIFIED ENGINEERING METHOD OF SUSPENSION TWO-SPAN PEDESTRIAN STEEL BRIDGES WITH FLEXIBLE AND RIGID CABLES UNDER ACTION OF ASYMMETRICAL LOADS

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Abstract. The article deals with two spans steel split-type one band pedestrian suspension bridge structure. Structural behaviour of such suspension member subjected to asymmetrical load has been discussed under the condition of temporary load influence imposed on one span out of two. Two structural solutions of one band suspension bridges have been considered: with completely flexible retaining elements, and in case of bending rigid. The article provides analytical expressions for calculations of the displacements of these asymmetrically loaded suspended elements, thrust forces and bending moments. Simplified analytical method accuracy of one band bridges is illustrated on the basis of performed numerical experiment. Performed numerical experiment shows the resulting basis of simplified analytical method accuracy of one band bridges.

Keywords: suspension steel bridges, pedestrian bridges, steel stress-ribbon, asymmetric loading, kinematic displacements, elastic displacements, bending stiffness, cables rigidity.

1. Introduction

Suspension bridges, due to their technical performance and excellent architectural appearance are widely used both for large and small spans to overlap (Gimsing 1997; Ryall *et al.* 2000; Troyano 2003). One of the oldest and successfully used to this day pedestrian suspension bridges is a steel stress-ribbon bridge (Schlaich *et al.* 2005; Schlaich, Bergerman 1992; Troyano 2003). The main carrying elements of such modern steel bridges are steel bands or high-strength steel wire ropes (Juozapaitis *et al.* 2006; Michailov 2002; Strasky 2005). Construction depth of the above type buildings is the lowest one. According to the operational requirements sag values of such bridge suspension carrying elements are relatively mean (Schlaich *et al.* 2005). It induces high tensile forces formed inside load-carrying structures, requiring great steel rates and conditions the anchored foundation mass of such bridges (Katchurin 1969; Kulbach 2007). One of the most serious drawbacks of the suspension bridges is their excessive deformations caused by the impact of asymmetric loads (Juozapaitis, Norkus 2004; Katchurin 1969; Kulbach 1999; Moskalev 1981; Wollmann 2001). There are methods known to be applied in order to reduce the deformability of such sus-

pension structures. Commonly heavy reinforced concrete decks or prestressed concrete structures are used for such steel stress-ribbon (Caetano, Cunha 2004; Schlaich *et al.* 2005; Strasky 2005).

Recently the multi-span steel stress-ribbon pedestrian bridges are applied (Schlaich, Bergerman 1992; Troyano 2003). Various design solutions are known for structural suspension systems (Strasky 2005). Due to horizontal displacements of standing (medium) pier and wider range of situations to be applied for temporary load calculations, the behaviour of such structures becomes more complex. Without any doubts this fact complicates calculation of the structures facing geometrical non-linearities (Katchurin 1969; Tarvydaitė, Juozapaitis 2010).

It shall be noted that absolute flexibility of a suspension cable is a theoretical concept, since the above elements of real structures have a particular depth cross section and bending stiffness of a finite size (not equal to zero) (Fürst *et al.* 2001; Gimsing 1997; Katchurin 1969; Moskalev 1981; Wyatt 2004).

It is known that in order to reduce the displacements of suspension bridges induced by asymmetric and local loads the so-called “rigid” suspension elements shall be applied (Grigorjeva *et al.* 2010; Juozapaitis *et al.* 2006, 2010).

These retaining elements, combining tension and bending elements abilities, not just stabilize the initial form of the bridge effectively, but allow “to prevent” the application of expensive pretension or bulk reinforced concrete deck (Juozapaitis *et al.* 2008). These “rigid” structural elements are made of hot-rolled or welded steel cross sections. Due to the potential stress concentration it is recommended to rest such suspension bridge structures upon flexible, i.e. to design elements as split (Juozapaitis *et al.* 2006; Kala 2008; Prato, Ceballos 2003).

It shall be noted that behaviour of multi-span steel stress-ribbon bridges is not completely considered, particularly in view of supporting element bending stiffness.

The article describes pedestrian two-span suspension split-type steel stress-ribbon bridges with a bending stiffness, analyzes the behaviour of such structures under the asymmetric load. It deals with kinematic displacements of bearing suspension cable of such bridges, and provides them in the form of displacement calculation analytic expressions. The efficiency of steel stress-ribbon bridge displacement stabilization through the bending stiffness is being discussed. Methods of an engineering design of tension and displacement of the asymmetrically loaded suspension bridge steel structures, evaluating the impact of bending stiffness. Numerical experiments show the basis of the accuracy of the developed simplified analytical method.

2. Simplified engineering method for kinematic displacements of pedestrian two-span suspension split-type steel stress-ribbon bridge is

The main bearing element of a pedestrian two-span suspension split-type steel stress-ribbon bridge is a flexible cable which is calculated as a structure facing geometrical non-linearity. Supporting steel structure of suspension two-span bridge is being estimated. Suspension elements of this structure are flexibly and rigidly supported in the area of end piers. Standing pier of the structure is horizontally shiftable (Fig. 1).

When calculating the one band suspension bridge, it is assumed, that bearing suspension cable is absolutely flexible cable, i.e. it is free of bending stiffness EI . Under the own weight flexible cable takes shape close to the square parable. Cable stressed by concentrated or asymmetric loads will change its initial form. Such deformation is de-

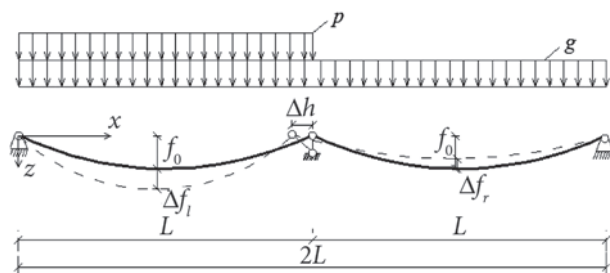


Fig. 1. Analytical model of the two-span split-type structure

termined by the kinematic origin shifts. It shall be noted that the increase rate of cable curvature induced by kinematic displacements exceeds the increase conditioned by the elastic deformation. This means that kinematic displacements can be more dangerous for cable deformability than vertical displacements caused by elastic deformations. Simplified engineering method (estimated) has been developed for the analysis of kinematic displacements of suspension split-type steel stress-ribbon bridge (Tarvydaitė, Juozapaitis 2010).

It is assumed, that axial stiffness of the cable is equal to $EA = \infty$ and the kinematic displacements caused by asymmetric load will be considered accordingly to the assumption. According to the ratio of the right and left suspension elements length $s_l = s_r$, the horizontal kinematic displacement of the standing pier can be calculated as follows (Tarvydaitė, Juozapaitis 2010):

$$\Delta h = \frac{4}{3L} [(f_0 + \Delta f_l)^2 - (f_0 + \Delta f_r)^2], \quad (1)$$

where L – span length of suspension cable, m; f_0 – suspension cable initial sag, m; Δf_l – vertical kinematic displacement in the middle of the left span, m; Δf_r – vertical kinematic displacement in the middle of the right span, m.

Then the thrust force equilibrium condition ($H_l = H_r$) is applied for calculation of left span vertical kinematic displacement:

$$\Delta f_l = -f_0 + \frac{(p+g)(f_0 + \Delta f_r)}{g}, \quad (2)$$

where g – constant load, kN/m; p – temporary load, kN/m.

With the help of geometric equations vertical kinematic displacement of the right span can be determined:

$$\Delta f_r = -f_0 + \frac{g\sqrt{f_0^2 + \frac{3}{8}L\Delta h}}{(p+g)}. \quad (3)$$

It shall be noted that the left kinematic displacement is downward, and the right – upward (Fig. 1).

3. Simplified engineering method for general displacements of pedestrian two-span suspension split-type steel stress-ribbon bridge is

It is known that general (total) displacements of the cable include of Kinematic and Elastic displacements. Elastic displacements are caused by cable elongation under the thrust (tension) force H . While calculating the above displacements it is considered that bearing cables of steel stress-ribbon bridge under the asymmetric loads experience, in particular, kinematic, and only then elastic displacements. In order to determine general (kinematic and elastic) displacements the well known equilibrium Eq of a deformed state shall be applied:

$$\Delta s_g - \Delta s_{el} = 0, \quad (4)$$

where:

$$\Delta s_g = s_1 - s_{k1}, \quad (5)$$

$$\Delta s_{el} = \frac{HL}{EA}, \quad (6)$$

where s_1 and s_{k1} – accordingly the cable length after the elastic deformation and before it, m; H – thrust force, kN; E – elastic modulus, MN/m²; A – cross-sectional area, m².

Length of the cable prior the elastic deformation s_{k1}

$$s_{k1} = L + \frac{8f_0^2}{3L}, \quad (7)$$

and after the elastic deformation s_1 :

$$s_1 = L - \Delta h + \frac{8(f_0 + \Delta f_{el})^2}{3(L - \Delta h)}, \quad (8)$$

where Δh – the horizontal displacement of a standing pier.

With the help of (4) – (8) expressions the equation for general cable displacement in the middle of the span will be as follows:

$$\Delta f_{el}^2 + 2f_0\Delta f_{el} - 0.375\Delta hL - \frac{HL^2}{EA} = 0. \quad (9)$$

Thrust forces of the right H_l and left H_r spans are as follows:

$$H_l = \frac{(g+p)L^2}{8(f_0 + \Delta f_l)}, \quad (10)$$

$$H_r = \frac{gL^2}{8(f_0 + \Delta f_r)}, \quad (11)$$

where Δf_l and Δf_r – accordingly general displacements of the left and right spans.

Solving the (9), (10) and (11) Eqs, the 3rd degree expressions will be delivered for the calculation of the left and right displacement values:

$$\Delta f_l^3 + 3f_0\Delta f_l^2 + 2f_0^2\Delta f_l - 0.375L\Delta h\Delta f_l - 0.375L\Delta hf_0 - \frac{3(g+p)(L-\Delta h)^2L^2}{64EA} = 0, \quad (12)$$

$$\Delta f_r^3 + 3f_0\Delta f_r^2 + 2f_0^2\Delta f_r + 0.375L\Delta h\Delta f_r + 0.375L\Delta hf_0 - \frac{3g(L+\Delta h)^2L^2}{64EA} = 0. \quad (13)$$

Horizontal displacement of the standing pier induced by the kinematic and elastic displacements can be calculated applying (14) and (15) Eqs:

$$s_0 = s_l - \Delta s_{el,l}, \quad (14)$$

$$s_0 = s_r - \Delta s_{el,r}. \quad (15)$$

Then:

$$\Delta h = \frac{4}{3L}((f_0 + \Delta f_l)^2 - (f_0 + \Delta f_r)^2) - \frac{(g+p)L^3}{16(f_0 + \Delta f_l)EA} + \frac{gL^3}{16(f_0 + \Delta f_r)EA}. \quad (16)$$

To solve the Eq of the 3rd degree with the help of modern mathematical methods, or mathematical programming operators (*Mathcad, Maple, Matlab, etc.*) is not difficult. However, the design of suspension bridges shall take into the consideration the operational requirements (for example, the threshold bridge displacements), and to determine parameters of the cable cross sections. In this case, the 3rd degree Eq is not helpful and the calculation will become quite complicated.

In order to obtain simplified engineering (estimated) expressions of suspension bridge displacements applying Eq (9) and taking into account Eq (10), the simplified formula to be applied for the calculation of the general displacement of the left cable will be as follows:

$$\Delta f_l = -f_0 + \sqrt{f_0^2 + 0.375L\Delta hf_0 + \frac{3(g+p)(L-\Delta h)^2L^2}{64EA}}. \quad (17)$$

This formula allows, at a known horizontal displacement of a standing pier to reduce the volume of the iterative calculation.

From the thrust force equilibrium condition ($H_l = H_r$), the right cable displacement can be calculated:

$$\Delta f_r = -f_0 + \frac{g(f_0 + \Delta f_l)(L + \Delta h)^2}{(p + g)(L - \Delta h)^2}. \quad (18)$$

It shall be noted, that in this case the horizontal displacement of the standing pier is calculated according to the Eq (16).

4. Numerical analysis of a general displacement of two-span suspension structure

In order to determine the accuracy of the developed engineering techniques the numerical experiment has been performed. For the numerical analysis the two-span split-type structure flexibly and rigidly supported in the area of end piers with spans equal to 40 m, and the total length of the suspension bridge structure equal to 80 m, has been selected. The initial sag values of the cable are accordingly –

$f_0 = \frac{L}{50} = 0.8$, $f_0 = \frac{L}{40} = 1.0$ and $f_0 = \frac{L}{32} = 1.25$. The analytical model of the retaining element is presented in Fig. 2.

Calculating with the program *Cosmos/m*, the each span retaining element was composed of 80 straight finite elements. Uniformly distributed loads have been replaced in points (nodes) by concentrated forces.

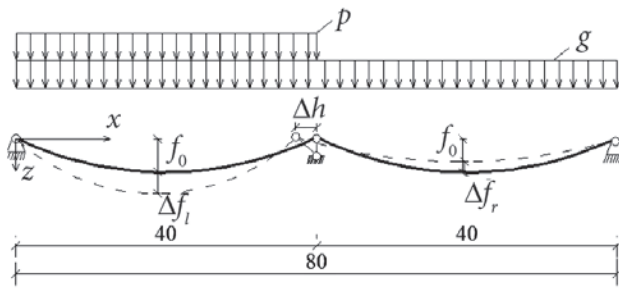


Fig. 2. Analytical model of the two-span split-type structure

During the numerical analysis of the corresponding bridge structure all values of evenly distributed asymmetric load are set in the context of temporary and permanent changes in the load ratio range. Table 1 provides the results of the corresponding structure.

Analysis of the results presented in Table 1, shows that general vertical cable displacements (at the middle of the left and right spans) and the horizontal displacement of the standing pier (Δh), calculated according to the 3rd degree and engineering formulas delivered by *Cosmos/m* program are almost coinciding. It shall be noted that displacement of the left cable at span quarters (i.e. if $x_l = \frac{L}{4}$ and $x_l = \frac{3L}{4}$) and general displacements are as well the same, and the greatest difference is less than 0.33%.

Determined that general displacements values of the particular structure calculated using the simplified engineering Eqs (12), (16) and (18), practically coincide with displacement values obtained with the help of *Cosmos/m* program, inaccuracies do not exceed 2.41%.

5. Engineering method for displacements of structure with bending stiffness

Bending stiffness structure is the structure which takes over loads both by stretching and bending. These retaining elements, combining tension and bending elements abilities, stabilize the initial geometric form (Juozapaitis et al. 2006).

Bending stiffness of the structure parameter is estimated by the plianthood kL . The greater plianthood parameter kL is, the greater flexibility of the structure, and vice versa, the less plianthood parameter value kL , the more rigid the structure is. It can be assumed that if the $kL \approx 1$, the structure is very rigid, and when the $kL \approx 10$ – the structure can be considered absolutely flexible.

Pliantness coefficient k is calculated as follows:

$$k = \sqrt{\frac{H}{EI}} \tag{19}$$

For the analysis of general displacements of the two-span suspension split-type steel stress-ribbon bridge the engineering formula (estimated) has been developed. The analytical model is presented in Fig. 1.

Table 1. General displacement values delivered by *Cosmos/M* (C) program and after the engineering calculation (A)

f_0	γ	Δf_l			Δh			Δf_r		
		C	A	%	C	A	%	C	A	%
0.8	0.5	-0.5035	-0.5017	0.36	-0.03128	-0.03116	0.39	-0.07148	-0.0705	1.39
	1	-0.6100	-0.6077	0.38	-0.04947	-0.04928	0.39	0.0924	0.0927	-0.32
	2	-0.7130	-0.7102	0.39	-0.06758	-0.06732	0.39	0.2931	0.2932	-0.03
	3	-0.7604	-0.7576	0.37	-0.07585	-0.07557	0.37	0.4077	0.4076	0.02
	4	-0.7869	-0.7840	0.37	-0.08035	-0.08006	0.36	0.4807	0.4807	0.00
1.0	5	-0.8036	-0.8007	0.36	-0.08311	-0.08281	0.36	0.5310	0.5310	0.00
	0.5	-0.4539	-0.4515	0.53	-0.03887	-0.03871	0.41	0.02791	0.0286	-2.41
	1	-0.5703	-0.5674	0.51	-0.06130	-0.06106	0.39	0.2113	0.2115	-0.09
	2	-0.6805	-0.6773	0.47	-0.08334	-0.08303	0.37	0.4363	0.4362	0.02
	3	-0.7300	-0.7266	0.47	-0.09320	-0.09286	0.37	0.5645	0.5643	0.04
1.25	4	-0.7570	-0.7535	0.46	-0.09847	-0.09811	0.37	0.6460	0.6458	0.03
	5	-0.7738	-0.7702	0.47	-0.10170	-0.10128	0.41	0.7021	0.7019	0.03
	0.5	-0.4219	-0.4192	0.64	-0.05130	-0.05109	0.41	0.1311	0.1315	-0.30
	1	-0.5530	-0.5498	0.58	-0.08071	-0.08042	0.36	0.3430	0.3428	0.06
	2	-0.6746	-0.6711	0.52	-0.1092	-0.10887	0.30	0.6032	0.6026	0.10
	3	-0.7275	-0.7240	0.48	-0.1217	-0.12134	0.30	0.7511	0.7505	0.08
	4	-0.7557	-0.7521	0.48	-0.1282	-0.12791	0.23	0.8450	0.8444	0.07
	5	-0.7728	-0.7692	0.47	-0.1321	-0.13179	0.24	0.9095	0.9090	0.06

Thrust forces, acting on the left and right spans, are equal to:

$$H_l = \frac{(g+p)(L-\Delta h)^2}{8} - \frac{48EI\Delta f_l}{5L^2}, \quad (20)$$

$$H_r = \frac{g(L+\Delta h)^2}{8} - \frac{48EI\Delta f_r}{5L^2}, \quad (21)$$

where I – cross-sectional moment of inertia, m^4 .

Similar to the flexible cable, when substituting expression (9) with (20) and (21) Eqs, the 3rd degree (cubic) equation needed for the calculation of the left and right cable displacements will be obtained:

$$\Delta f_l^3 + 3f_0\Delta f_l^2 + 2f_0^2\Delta f_l - 0.375L\Delta h\Delta f_l - 0.375L\Delta hf_0 - \frac{15(g+p)(L-\Delta h)^2L^2 - 1152EI\Delta f_l}{320EA} = 0, \quad (22)$$

$$\Delta f_r^3 + 3f_0\Delta f_r^2 + 2f_0^2\Delta f_r + 0.375L\Delta h\Delta f_r + 0.375L\Delta hf_0 - \frac{15g(L+\Delta h)^2L^2 - 1152EI\Delta f_r}{320EA} = 0. \quad (23)$$

Horizontal displacement of the standing pier, according to (14) and (15) Eqs will be equal to:

$$\Delta h = \frac{4}{3L}((f_0 + \Delta f_l)^2 - (f_0 + \Delta f_r)^2) - \frac{5(g+p)L^4 - 384EI\Delta f_l}{80L(f_0 + \Delta f_l)EA} + \frac{5gL^4 - 384EI\Delta f_r}{80L(f_0 + \Delta f_r)EA}. \quad (24)$$

In order to simplify the calculation it is proposed to calculate the general displacement of the right cable (in the middle of the second span) through a thrust force equality condition ($H_l = H_r$), as follows:

$$\Delta f_r = \frac{5g(f_0 + \Delta f_l)(L + \Delta h)^4 - 5(p + g)(L - \Delta h)^4 f_0}{384EI f_0 + 5(p + g)(L - \Delta h)^4} + \frac{384EI f_0 \Delta f_l}{384EI f_0 + 5(p + g)(L - \Delta h)^4}, \quad (25)$$

$$\Delta f_l = \frac{5(g + p)(f_0 + \Delta f_r)(L - \Delta h)^4 - 5g(L + \Delta h)^4 f_0}{384EI f_0 + 5g(L + \Delta h)^4} + \frac{384EI f_0 \Delta f_r}{384EI f_0 + 5g(L + \Delta h)^4}. \quad (26)$$

Eqs (22)–(26) show that displacement of suspension bridge cables depends not only on their cross-axial stiffness, but also on the bending stiffness.

According to the engineering calculation method (using the beam analogy) the bending moments of the left and right cables in the middle of the span are:

$$M_l \approx \frac{48}{5} \frac{EI\Delta f_l}{L^2}, \quad (27)$$

$$M_r \approx \frac{48}{5} \frac{EI\Delta f_r}{L^2}. \quad (28)$$

Inaccuracy of (27) and (28) expressions is high and does not exceed 8.86% (Fig. 3).

6. Numerical analysis of the suspension structure bending stiffness general displacements

For the numerical analysis the same type of the suspension bridge, as a two-span flexible cable structure has been selected. Pliantness values of the selected parameter vary from 2 to 10. The analytical model of the retaining element is presented in Fig. 2.

During the numerical analysis of the current structure the evenly distributed asymmetric load values are adopted in the context of temporary and permanent changes in the load ratio range γ .

The results of calculation are given in Table 2.

Figs 4-6 show stabilisation of displacements with the help of bending stiffness depending on the initial sag (f_0), when the temporary and permanent in the load ratio of 1.

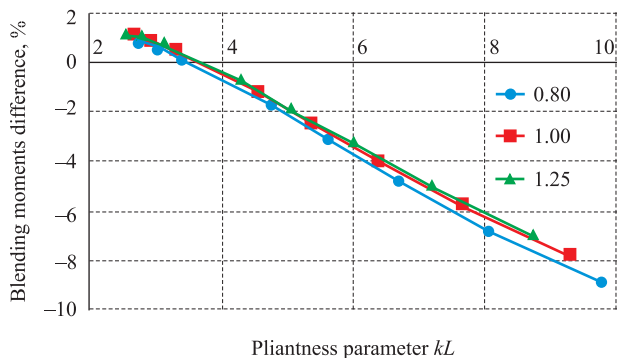


Fig. 3. Bending moments at the middle of the first span, differences of results calculated according to the engineering formulas and delivered by the program Cosmos/m

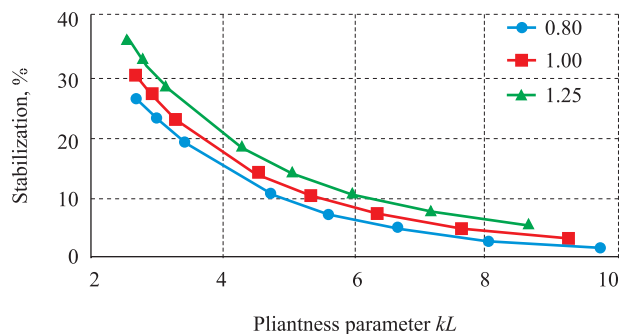


Fig. 4. Stabilization of general displacements, at the middle of the first span, by the bending stiffness

The results in Table 2 show that general displacements of the left and right cables and the horizontal displacements of the standing pier (Δh) calculated in accordance with the engineering calculation formulas and delivered by the program *Cosmos/m* practically coincide. The greatest inaccuracy of the general vertical displacement of the right unloaded by the asymmetric load mid-span in case of the directed downward elastic displacement is practically equal to the directed upwards kinematic displacement.

It shall be noted that general displacements at the quarter of the left loaded with asymmetric load span (when $x_l = \frac{L}{4}$ and $x_r = \frac{3L}{4}$) as well are practically the same.

The greatest difference between the results obtained by program *Cosmos/M* and after the engineering calculation is equal to 1.93%.

7. Conclusions

Presented simplified engineering method of suspension two-span pedestrian steel bridge under action of asymmetrical loads allows performing the relatively simple calculation of bearing suspension cable thrust forces, vertical and horizontal displacements and bending moments. Numerical analysis shows that proposed simplified engineering method of steel stress-ribbon bridge is sufficiently precise. The greatest inaccuracy of vertical displacement and thrust forces calculation does not exceed 1.37%, and the determination of bending

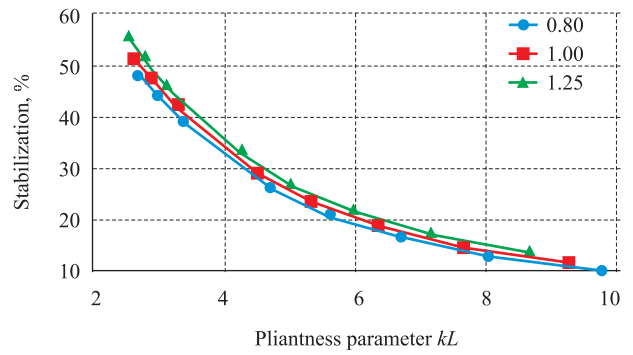


Fig. 5. Stabilization of standing pier horizontal displacements, by the bending stiffness

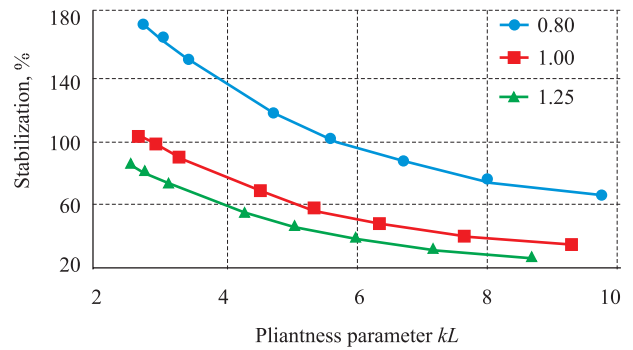


Fig. 6. Stabilization of general displacements, at the middle of the second span, by the bending stiffness

Table 2. *Cosmos/M* program (C) and the displacement values received after engineering calculation (A)

f_0	kL	Δf_l			Δh			Δf_r		
		C	A	%	C	A	%	C	A	%
0.8	9.76	-0.5985	-0.5858	2.17	-0.04442	-0.04418	0.54	0.03232	0.0322	0.37
	8.03	-0.5899	-0.5759	2.43	-0.04297	-0.04282	0.35	0.02291	0.0233	-1.67
	6.67	-0.5779	-0.5629	2.66	-0.04113	-0.04107	0.15	0.01123	0.0124	-9.44
	5.59	-0.5623	-0.5469	2.82	-0.03892	-0.03896	-0.10	-0.00229	-0.00272	-15.70
	4.71	-0.5425	-0.5271	2.92	-0.03630	-0.03643	-0.36	-0.01753	-0.0147	19.25
	3.37	-0.4901	-0.4762	2.92	-0.03003	-0.03033	-0.99	-0.04982	-0.0455	9.49
	2.97	-0.4658	-0.4528	2.87	-0.02739	-0.02773	-1.23	-0.06148	-0.0567	8.43
	2.70	-0.4465	-0.4344	2.79	-0.02539	-0.02577	-1.47	-0.06937	-0.0645	7.55
1.0	9.28	-0.5511	-0.5382	2.40	-0.05417	-0.05386	0.58	0.1396	0.1373	1.68
	7.64	-0.5413	-0.5273	2.66	-0.05219	-0.05199	0.38	0.1260	0.1242	1.45
	6.35	-0.5280	-0.5133	2.86	-0.04971	-0.04963	0.16	0.1091	0.1081	0.93
	5.33	-0.5111	-0.4963	2.98	-0.04676	-0.04682	-0.13	0.08952	0.0894	0.13
	4.50	-0.4902	-0.4757	3.05	-0.04333	-0.0435	-0.39	0.06748	0.0684	-1.35
	3.25	-0.4376	-0.4248	3.01	-0.03537	-0.03575	-1.06	0.0205	0.0234	-12.39
	2.87	-0.4143	-0.4024	2.96	-0.03211	-0.03254	-1.32	0.003103	0.00359	-13.57
	2.62	-0.3961	-0.3849	2.91	-0.02969	-0.03015	-1.53	-0.00889	-0.00809	9.94
1.25	8.68	-0.5223	-0.5086	2.69	-0.06986	-0.06943	0.62	0.2539	0.2485	2.17
	7.16	-0.5101	-0.4957	2.90	-0.06692	-0.06666	0.39	0.2345	0.2297	2.09
	5.96	-0.4939	-0.4792	3.07	-0.06327	-0.06319	0.13	0.2107	0.2068	1.89
	5.01	-0.4741	-0.4595	3.18	-0.05903	-0.05911	-0.14	0.1834	0.1807	1.49
	4.24	-0.4503	-0.4364	3.19	-0.05417	-0.05442	-0.46	0.1531	0.1518	0.86
	3.08	-0.3938	-0.3819	3.12	-0.04336	-0.04385	-1.12	0.08965	0.0910	-1.48
	2.73	-0.3699	-0.3590	3.04	-0.03908	-0.03964	-1.41	0.06633	0.0685	-3.17
	2.50	-0.3519	-0.3417	2.99	-0.03596	-0.03655	-1.61	0.05003	0.0527	-5.07

moments – 8.86%. It shall be noted that this method is suitable for preliminary design of such bridges.

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