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Effect of Geometric Imperfections on the Design of Steel Box Bridges

Effets d'imperfections géométriques sur la conception des ponts en caisson

Auswirkung Massabweichungen auf die Bemessung von Kastenträgern aus Stahl

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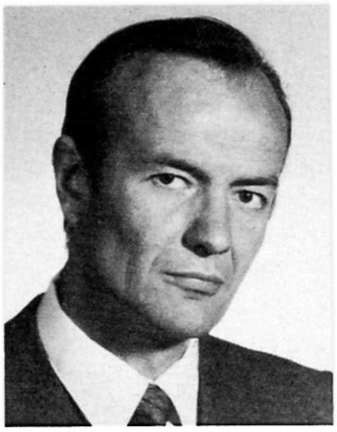

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SUMMARY
Geometric and structural imperfections have unfavorable effects on the ultimate strength of steel-plated structures. To define the magnitude of real imperfections of stiffened plates used in the fabrication of steel box-girder bridges in Canada, a statistical analysis is made of about 10,500 deflection measurements on six in-service bridges.

RESUME
Les imperfections géométriques et structurales ont des effets défavorables sur la résistance ultime des structures en tôles d'acier. Pour définir l'ampleur des véritables imperfections des tôles renforcées, utilisées dans la fabrication des ponts à caisson au Canada, une analyse est faite sur environ 10,500 mesures de déflection sur six ponts en service.

ZUSAMMENFASSUNG
Geometrische und strukturelle Abweichungen haben ungünstige Auswirkungen auf den Bruchwiderstand von Stahlkonstruktionen. Um die Grösse der wahren Messabweichungen von Stahlblechen, die in Kanada für die Herstellung von Hohlkastenträgern benützt werden, zu definieren, wurde eine statistische Analyse von 10'500 Durchbiegungsmessungen an sechs in Betrieb stehenden Brücken durchgeführt.

1. INTRODUCTION

The detrimental effect of structural and geometrical imperfections produced during the fabrication of welded stiffened plate structures on their buckling strength has for a long time been recognized. The measured magnitude of geometric imperfections, as well as those tolerances prescribed by codes, differ rather significantly from one country to another.

With the introduction of the limit states design philosophy which has now been adopted all over the world, it became essential to assess the actual ultimate strength of all elements and more specifically, those of the entire structure. The application of limit states in the design of plated structures implies the use of the semi-probabilistic theory in defining the magnitude of geometric imperfections as well as their correlation with the loss of strength of the structure.

Since 1979, Canada has introduced the limit states method in the design of highway bridges first in the province of Ontario [1] and tends to extend its application to the national level [2]. The uncertainty of the real magnitude of geometric imperfections and their influence on the buckling strength of compression flanges appears to be the main reason for restriction of their applicability to spans up to 50m for steel box bridges [1].

2. GEOMETRIC IMPERFECTIONS OF STEEL BOX BRIDGES IN CANADA

2.1 Measurement Program

To undertake the research program, six in-service steel box bridges across Canada have been chosen. It was anticipated that the average fabricating conditions or significant differences in the value of geometric imperfections from province to province would be evident. The pertinent features of these highway bridges are given in Table 1.

The above bridges were selected because the box girders are of large cross-section, are continuous over interior supports, and have spans longer than 50m. The negative moments which occur over the support regions induce significant compressive stresses in the bottom flanges. Since geometric imperfections are of concern in these areas, the major part of the measurements was concentrated here.

The geometric imperfections referred to in this paper include the out-of-flatness of the web subpanels and bottom flanges (f_1) and out-of-straightness of the longitudinal stiffeners (f_2).

To relate the measurements performed on in-service bridges, i.e., including the effect of their dead loads, with the imperfections produced during fabrication, a small correction should be applied to the former. It is claimed that this correction is likely to be less than 5% [3].

2.2 Results of the Measurements

To establish reliable values of imperfections, the statistical approach is recommended, and in this regard, a large number of measurements are needed. In the study performed, a number of 10,500 measurements were used to define the statistical parameters. Statistical analyses of the data were undertaken for each bridge and their aggregate [4] . As a representative magnitude of these imperfections the 95% fractile that has been extensively employed [3] was adopted. Table 2 defines the mean value, standard deviation and 95% fractile value obtained from the statistical analyses of aggregate data for the six bridges for the out-of-flatness subpanels and out-of-straightness longitudinal stiffeners.

		BRIDGE					
		Drinkwater	Glen Morris	Portage	Muskwa	Campbell	Mission
Spans (m)		69.56+85.34 +60.96	52.73+2@45.72 +59.43	36.58+85.34 +36.58	55.25+2@91.44 +54.86+36.88	2@55.17	88.39+134.11 +88.39
Length (m)		207.26	203.61	158.50	329.87	110.34	310.90
Type of Beams			C O N T I N U O U S				Cantilever
Number of Box Girders		4	2	5	2	3	1
Cross-Section		T R A P E Z O I D A L			R E C T A N G U L A R		
Dept. of Webs		C O N S T A N T		V A R I A B L E			
Width of (mm)	Bottom Flanges	1329	1219	Varies (1626-2388)	2591	1575	11252
	Panels	600	445	Varies (406-597)	520	560	675
Fabrication Year		1973	1972	1973	1975	1976	1974
Structural Steel	Specification	CSA G40.11 Grade B	CSA G40.11 Grade B	CSA G40.11 Grade B	CSA G40.21 Grade 50A	CSA G40.21 Grade 50A	CSA G40.8 Grade B
	Min.Yield Strength (MPQ)	250	250	250	350	350	350

Table 1

2.3 Comparison With Specifications

Fabrication tolerances prescribed in existing codes differ greatly from one country to another and they depend upon the experiences, technologies and traditions of the fabricators. Following the recommendations of a Task Group chaired by Professor Ch. Massonnet, a set of more realistic and easy to control tolerances has been proposed [3]. These tolerances which should be applied on unloaded bridges and in the opinion of the Task Group can generally be respected, are:

$f_1/200$ - for the deformation of plate panels, and

$f_2/500$ - for the deformation of longitudinal stiffeners.

It is evident from Table 2 that in the case of those steel box bridges studied in Canada, the measured imperfections of the bottom flanges meet neither the prescribed tolerances [5] accepted in Canada, nor the proposed tolerances mentioned above.

Comparing those values of geometric imperfections defined in Table 2 with those obtained in other countries [3],[10], one can note that they are comparable with those found in West Germany and Czechoslovakia and are larger than those established in Belgium and the United Kingdom. Even those values of geometric imperfections obtained in Canada, West Germany and Czechoslovakia should be decreased by 5% to take into account the influence of dead load; they remain larger than the prescribed tolerances for unloaded bridges. This increase represents about 70% in the case of out-of-flatness subpanels and 35% in the case of out-of-straightness longitudinal stiffeners for bottom flanges with thicknesses less than 30 mm.

3. EFFECT OF IMPERFECTIONS ON THE DESIGN OF STIFFENED COMPRESSION PLATES

3.1 General Considerations

The design of stiffened compression flanges by taking into account the influence of imperfections, can be conducted in one of the two ways developed during the last decade. First, is to treat the stiffened plate as a series of struts and to apply inelastic beam-column methods in assessing their strength. The second is to treat the discretely stiffened flange as a plate assemblage. In the last case, either the modified linear buckling theory or the non-linear post-buckling theory can be applied. [9] Although the second approach has been proposed by many authors, the only ones to appear in the codes under review are the inelastic strut approach and modified linear elastic buckling theory.[11],[12]

The use of more rational inelastic buckling methods in calculating the ultimate limit states could lead to greater consistencies in element strength and existing research information should be used in the new design codes. In the actual transition period from the older elastic based methods to the newer ultimate load ones, even the introduction of an inelastic strut approach in the design codes has to be welcomed. In this regard, the new British Standard [6] and American proposals [5] should be noted.

3.2 Magnitude of Imperfections to be Applied in the Design Analysis

The use of inelastic methods in the design analysis of stiffened compression plates implies the knowledge of magnitude of structural and geometrical imperfections. The values of imperfections to be used in the design codes should reflect the real study of the fabrication industry and they could differ from one country to another. As previously shown, the measured values of imperfections in Canada and in other countries are different from the tolerances prescribed by the codes. These differences are general due to the fact that the code requirements are not always in relation to the possibilities of the workshops. In these conditions the magnitude of those imperfections to be used in

Position of	Number of meas.	Thickness of the Plate (mm)	f_1/b - Absolute Values: Mean	Standard Deviation	95% Fractile	$1/f_{ip}$ (%)	Tolerance of code
a. Panel							f_{ip}/b
	2116	9.53	1/266	1/430	1/127	14.38	
	949	11.11	1/331	1/529	1/157	0.00	
WEB	148	12.70	1/446	1/630	1/191	0.00	1/150 [7]
	72	14.29	1/439	1/621	1/188	0.00	1/61 to
							1/160 [5]
AGGREGATE WEBS	3205	9.53 & 14.29	1/290	1/452	1/135	7.82	1/200 [3]
	400	9.53	1/248	1/348	1/106	32.99	
	594	12.70	1/263	1/262	1/88	26.87	
	828	15.87	1/331	1/491	1/149	17.47	
	919	17.46 & 19.05	1/466	1/664	1/201	4.74	1/200[5].[3]
	328	22.23	1/396	1/460	1/132	13.80	
BOTTOM	828	25.40	1/441	1/563	1/147	11.03	- [7]
FLANGE	520	28.10	1/334	1/267	1/111	18.84	
	156	38.10	1/729	1/1264	1/354	0.00	
	96	57.15	1/667	1/1126	1/325	0.02	
AGGREGATE							
BOTTOM FLANGES	4417	9.53 & 28.56	1/351	1/395	1/117	18.71	
	252	38.10 & 57.15	1/704	1/1147	1/336	0.01	
b. Stiffener			f_2/a - Absolute Values				f_{2p}/a
	387	9.53-16	1/674	1/785	1/338	-	
BOTTOM	1479	17.46-30	1/743	1/964	1/417	-	1/500[5].[3]
FLANGE	284	38.10-57.15	1/1871	1/1473	1/731	-	
aggregate	1866	9.53-30	1/709	1/893	1/366	-	- [7]

Table 2 Statistical results of measured deformations (f_1 and f_2)

design analysis should be the values defined for 95% fractile in a statistical analysis of measured imperfections and not those used for the prescribed tolerances. Because from the designer point of view it is easier to refer to prescribed fabrication tolerances, the magnitude of those imperfections to be applied in design analysis (f_{id}) could be expressed as follows:

$$f_{id} = \alpha_i . f_{ip} \tag{1}$$

where: $i = \begin{cases} 1 \text{ - for out-of-flatness subpanels} \\ 2 \text{ - for out-of-straightness longitudinal stiffeners} \end{cases}$

$$\alpha_i = \gamma_D . f_{i(95\%)} / f_{ip} \geq 1 \tag{2}$$

$$\gamma_D = \begin{cases} 0.95 \text{ - for measurements on in-service bridges} \\ 1.00 \text{ - for measurements on unloaded bridges.} \end{cases}$$

$$f_{ip} = \begin{cases} b/200 \text{ - for } i=1 \\ a/500 \text{ - for } i=2 \end{cases} \tag{3}$$

Referring to Table 2, the corresponding values to be used in Canada should be:

$$\alpha_1 = 1.60 \quad f_{id} = 1.60\ f_{1p} \tag{3.1}$$

and

$$\alpha_2 = 1.30 \quad f_{2d} = 1.30\ f_{2p} \tag{3.2}$$

Similar expressions have been found by using measurement results from West Germany and Czechoslovakia.

Only in cases where an agreement between the measured imperfections and prescribed tolerances has been established, as in the case of Belgium and the United Kingdom [3], can the latter be used in design analysis. In all other cases, Equation (1) should be applied.

3.3 Effect of Imperfections on Steel Box Bridges in Canada

Following the definition of the effect of geometrical imperfections on the buckling strength of compression flanges in six box bridges (Table 1), the inelastic strut approach and the well-known Perry strut equation have been applied. For each bridge the ultimate limit load has been defined as: a) the prescribed fabrication tolerances, b) the geometric imperfection defined by Equation (4.2), and c) for "ideal" flanges without imperfections. For the first two cases a) and b), where the imperfections are taken into account, the nominal value of 0.10 σ_y generally recommended for compressive residual stresses has been added.

The detrimental effect of structural and geometric imperfections on ultimate limit loads has been defined by the ratio between the difference in the limit loads of compression flanges with imperfections and those without imperfections with respect to the value of limit loads corresponding to "ideal" flanges. The decrease in the ultimate limit loads established for the six bridges are presented in Table 3.

In general one could note the influence of supplementary eccentricity due to the vertical curvature of the bottom flanges on ultimate limit loads. Referring to Table 3, it should be noted that it was the authors' intention to compare these results with those obtained by using some of the existing codes which have introduced the same approach in the design of compression flanges [5],[6]. It was found difficult to make useful comparisons because of differences in the specified tolerances and also in interpreting the intentions of the code writers.

It is hoped that the measurements of residual stresses, as well as the experimental test program in progress at this time, in Canada, will provide the necess-

Bridge	Magnitude of Geometric Imperfections	
	$f_{2p} = a/500$	$f_{2d} = a/385$
Drinkwater	4.23	5.19
Glen Morris	7.49	8.73
Portage	10.71	12.29
Muskwa	9.83	11.87
Campbell	8.65	10.43
Mission	10.07	13.35

Table 3 Decrease of ultimate limit loads due to imperfections (%)

ary information required for the introduction of inelastic buckling methods in the limit states design of steel box bridges and in this country, which for years has had some of the best design codes in the area of bridges.

4. CONCLUSIONS AND RECOMMENDATIONS

1. The weakening effects of manufacturing and erection procedures on the buckling strength of steel plated structures have been recognized for a long time. In the strength of the structure being affected by its imperfections, the design codes should take into account the influence of those tolerances prescribed by fabrication codes, in using a "performance coefficient - α_i". This coefficient should reflect the existing differences between the magnitude of real imperfections and prescribed tolerances. Some proposal regarding the introduction of larger fabrication tolerances should be carefully analysed by taking into account its implication on the cost and safety of the structure.

2. The prescribed tolerances used in the fabrication of steel-plated structures have to reflect the experience, technologies and traditions of those fabricators specific to each country. They generally have to be such that the workshops will be able to respect them by working well without applying specific procedures.

3. Even though the magnitude of actual measured imperfections are greater than those considered "reasonable" and are proposed [5],[3], or prescribed, it is expected that in the near future the existing gap between the two values will be substantially reduced. The understanding of those people involved in the fabrication and control of steel structures of imperfection importance on structureal strength will probably be the most important factor in reducing this gap.

4. Inelastic buckling methods tend to be adopted in more design codes in this transition period to ultimate limit load methods. It is expected that new methods will be introduced in the near future and in Canadian Design Bridge Codes, due to their consistencies, and in this case, they will take into account the specific fabrication conditions existing in this country, as well as Canadian research contribution in this area.

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NOTATIONS

- a - Plate panel or longitudinal stiffener length
- b - Plate panel width
- t - Plate thickness
- f_1 - Out-of-flatness subpanels
- f_2 - Out-of-straightness longitudinal stiffeners
- f_{1p}, f_{2p} - Prescribed tolerances referred to f_1 and f_2, respectively
- f_{1d}, f_{2d} - Magnitude of geometric imeprfections to be used in design analysis
- $f_{1(95\%)}$ - Magnitude of geometric imperfections defined for 95% fractile (1=1,2)
- α_i - Performance factor (1=1,2)
- γ_D - Imperfection correction coefficient due to the application of dead load

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