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Artikel: Structural behaviour of hybrid plate girders in bending: application to actual bridges

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Structural Behaviour of Hybrid Plate Girders in Bending. Application to Actual Bridges

Comportement à la flexion de poutres à âme pleine hybrides.
Application aux ponts actuels

Biegeverhalten von hybriden Vollwandträgern.
Anwendung im Brückenbau

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I Introduction

At the IABSE London Colloquium in 1971, the ultimate strength of hybrid girders was discussed, and especially the fatigue study on thin-walled hybrid girders was encouraged.

The present study is intended to discuss the static and fatigue behavior of thin-walled, stiffened hybrid girders based on the results of tests carried out at Osaka University. Furthrmore, to apply the hybrid girder to long-span bridges, its design and economy are discussed with actual and design illustrations.

II Structural Behavior and Strength

II-1 Static Behavior and Strength

The overall flexural behavior of thin-walled hybrid girders consisting of WES-HW70, JIS-SM58 and JIS-SS41 in tension flange, compression flange and web, respectively, (1).is summarized in Fig. 1 in terms of the relation between applied bending moments and measured curvatures, to study on the influence of web slenderness ratios on the static flexural behavior. In Fig. 1, the calculated moment-curvature curves taking into account measured residual stresses and those neglecting the residual stresses are also given to show the effect of residual stresses. It can be seen in Fig. 1 that the girders with web slenderness ratios less than about 200 will behave like stocky beams owing to single horizontal stiffener located at one-fifth of the web depth, and that the test curves agree fairly well with the calculation curves with residual stresses. In the case of more slender girders of which web slenderness ratio exceeds about 250, however, the additional decrease of rigidity caused by web buckling diverts the test curves from the calculation curves.

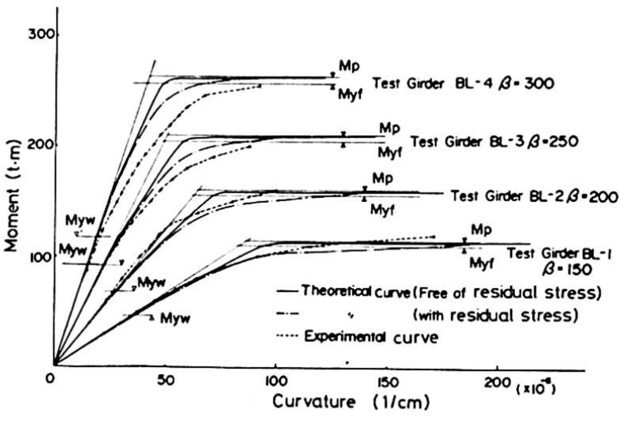

Fig. 1. Moment v.s. curvature curve

Table 1. Ultimate strength and reference loads

Girder	α	β	M_u/M_u^*	M_u/M_{yw}^{th}	M_u/M_{yf}^{th}	M_u/M_p^{th}
AL 1	0.5	141	1.18	3.42	1.18	1.16
BL 1	1.0		1.23	3.29	1.23	1.11
AL 2	0.5	188	1.10	3.09	1.10	1.08
BL 2	1.0		1.09	3.09	1.09	1.06
AL 3	0.5	232	1.15	3.14	1.15	1.12
BL 3	1.0		1.07	2.97	1.07	1.05
AL 4	0.5	281	1.05	2.72	1.01	0.99
BL 4	1.0		1.04	2.70	0.99	0.97

α : Aspect ratio of test panel, β: Web slenderness ratio

M_u^{th}: Experimental ultimate moment, M_u^*: Predicted ultimate moment

M_{yw}^{th}: Theoretical web yield moment,

M_{yf}^{th}: Theoretical flange yield moment

M_p^{th} ; Theoretical fully plastic moment

The experimental ultimate loads and the several reference loads calculated by the ordinary beam theory are shown in Table 1, which indicates that the limiting web slenderness ratio β with which a girder carries its theoretical flange yield moment is 323 and 289 for the aspect ratio α of 0.5 and 1.0 respectively. In Table 1, there are also the ultimate bending moment M_u^* predicted by a formula (1) which can estimate the decrease of resisting moment due to web buckling, and such prediction seems to be conservative.

According to the AASHTO Specifications (3), an allowable stress in a tension flange should be reduced depending upon the spreading of web yielding so that the slight decrease of rigidity of a girder due to web yielding can be estimated in design. Since such reduction, however, is derived from stress distribution on a cross section when its tension flange yields, yielded region in the web can be controlled relatively small to a primarily assumed extent in design. In Table 2, comparisons among the experimental ultimate moment (1), the design allowable moment and also the design ultimate moment to be examined at the assumed ultimate state in Load Factor Design (3) are shown. It is revealed that the experimental ultimate moment agrees well with the design ultimate moment and amounts to more than twice.as much as the design allowable moment.

II-2 Fatigue Behavior and Strength

Concerning static behavior and strength, it proved that a thin-walled hybrid girder with a web slenderness ratio less than about 290 could be dealt with as a homogeneous girder, only considering the slight decrease of rigidity due to web yielding. On structural behavior under repeated bending, however, the fatigue peculiar to a thin-walled plate girder that is caused by out-of-plane movement of a slender web, has been studied by Yen, Stallmeyer, Toprac and Maeda, etc.

Fig. 2 shows typical patterns of fatigue crack initiated in a hybrid girders subjected to pure bending. Among these types of crack, Type 1 cracks are the cracks above mentioned and were actually observed in the extensive fatigue tests of hybrid girders made by Toprac (8), who suggested that it is effective to use horizontal stiffeners and/or to limit the web slenderness ratio to be less than 200 for prevention of the initiation of Type 1 cracks.

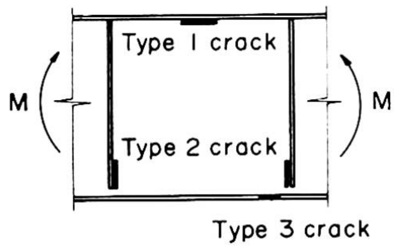

Fig. 2. Types of fatigue cracks under pure bending

In Table 3, the results of fatigue tests of six large-sized hybrid girders conducted by authors (13) are summarized together with parameters to evalulated the test results. All of the test girders failed due to Type 2 cracks initiated at the toe of vertical stiffener-to-web fillet welds. Type 1 crack and Type 3 crack were also observed in Girders B4-L1 and B3-L1, they were not a governing crack to cause the failure of the girders.

The regression analysis based on the method of least squares for the test results obtained by authors (10) and Toprac (8) with regard to the initiation of Type 2 cracks, gives the fatigue strength

Table 2. Comparisons of ultimate moment with design moment

Girder	M_u^{ex}	AASHTO				M_{yw}	M_u^{ex}/M_{all}	M_u^{ex}/M_u
		R	F_{all}	M_{all}	M_u			
AL 1	130	0.85	30.9	47.7	104	38	2.73	1.25
BL 1	125						2.62	1.20
AL 2	170	0.83	29.9	66.9	146	55	2.54	1.16
BL 2	170						2.54	1.16
AL 3	232	0.81	29.0	87.1	191	74	2.66	1.22
BL 3	220						2.52	1.15
AL 4	258	0.79	28.4	109.5	240	95	2.36	1.08
BL 4	257						2.35	1.07

M_u^{ex}: Experimental ultimate moment (t·m)
M_{all}: Allowable bending moment (service load design) (t·m)
M_u: Design ultimate moment (load factor design) (t·m)
R : Reduction factor
M_{yw}: Experimental web yield moment (t·m)
F_{all}: Allowable stress in tension flange (kg/mm^2)

Table 3. Test parameters and test results

Test Girder	β	γ/γ^*	S_r	R	Type of crack, N_c			Mode of failure
					Type.1	Type.2	Type.3	
B4L1	413	1	21.1	0.221	46.5	34.1	—	Type 2
B4L7		7	23.1	0.209	—	24.3	—	Type 2
B3L1	310	1	a) 11.7	0.543	—	218.0	218.0	Type2
B3L6		6	21.1	0.402	—	28.0	—	Type2
B2L0	206	—	17.6	0.439	—	104.0	—	Type2
B2L5		5	16.1	0.456	—	116.6	—	Type 2

β : Web slenderness ratio
γ/γ^*: Relative rigidity ratio of horizontal stiffener
S_r : Stress range in tension flange (kg/mm^2)
.R : Stress ratio, N_c: Number of cycles to crack (X10^4.)
a) Maxmum stress was increased up to 35.1 kg/mm after 2,10cycles

at 2×10^6 cycles in terms of stress range with 12.9 kg/mm^2 (mean value) and 10.7 kg/mm^2 (95% confidence limit) which is more than 25% above the allowable stress range of 8.4 kg/mm^2 (Stress Category: C) at the AASHTO Specifications. It is recognized that the fatigue strengths due to Type 2 crack and Type 3 crack can be respectively compared to those of a transverse non-load carrying fillet welded joint and a longitudinal fillet welded joint (4,7). Since the fatigue strength at 2×10^6 cycles of the latter joint is 30 to 50% higher than that of the former joint. According to a great number of fatigue tests (11), it may be concluded that Type 1 crack and Type 2 crack characterized the fatigue behavior of thin-walled hybrid girders with vertical stiffeners. As shown in Fig. 3, the test results of transverse non-load carrying fillet welded joints for several steel grades (12) agree well with those of Type 2 cracks at the girder tests (10), and the both fatigue strengths do not significantly differ from each other depending on the steel grades, covering 9 to 13 kg/mm^2. Consequently, it proves that a web in ordinary carbon steel is the most economical for a thin-walled hybrid girders as far as the fatigue strength is limited to the fatigue life of Type 2 crack.

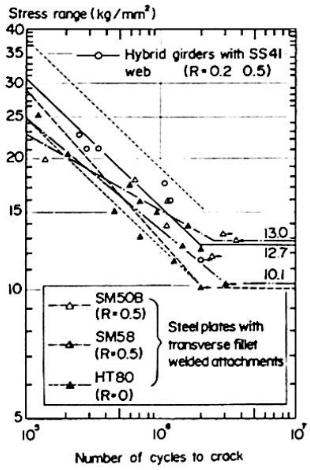

Fig. 3. S-N curves for Type 2 crack

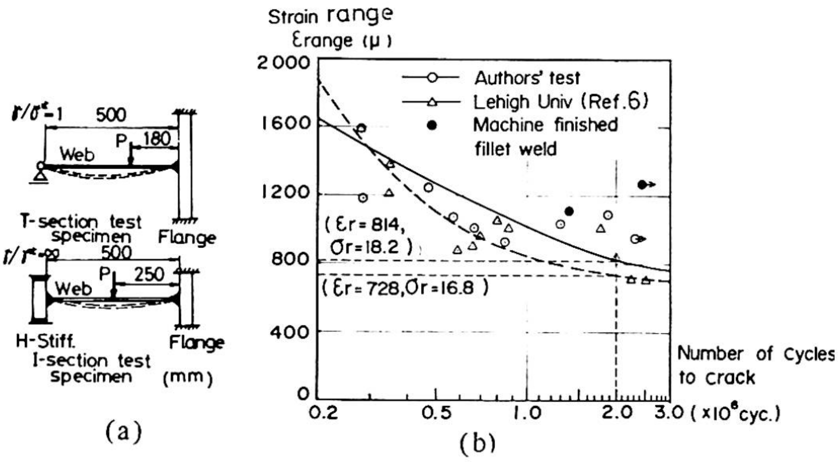

Fig. 4. Fatigue test results for Type 1 crack

On the other hand, few comparable tests for Type 1 crack have been carried out on model specimens of Type 1 crack, because of complicated cracking conditions. In Fig. 4, the relevent results obtained at fatigue tests made by Maeda (14) on model specimens of a web panel enclosed by a compression flange and a horizontal stiffener as shown in Fig. 4(a), are shown in terms of the number of cycles to crack initiation and the local strain range at the toe of fillet weld estimated by F.E.M. plate analysis combined with strain measurements. Available data of fatigue tests of large-sized plate girders conducted by Ostapenko (13) are also given, but they seem considerably dispersed, attributed to inevitable inherent indentations at the toe of fillet weld.

The fatigue strength at 2×10^6 cycles of the fillet weld under bending is estimated to be 814×10^{-6} in strain range or 18.2 kg/mm^2 in stress range with the regression analysis by the method of least squares. To examine the relation between the magnitude of out-of-plane deflection of a web and the fatigue lives of Type 1 crack, a non-dimensional parameter δ_r/h which represents the ratio of web deflection range to panel depth, is introduced. With this parameter the fatigue strength of Type 1 crack is rearranged graphically as shown in Fig. 5. The figure suggests that the magnitude of out-of-plane deflection of a web under a repeated live load should be limited to 1/350 of a web panel depth. To find out the governing

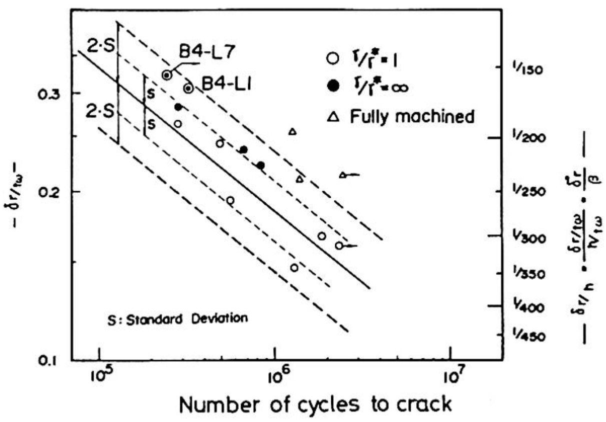

Fig. 5. Relation between magnitude of out-of-plane deflection of a web and fatigue lives of Type 1 crack

design criterion of Type 1 crack, it is desirable to develop the relevant fatigue data to induce the relation between the maximum initial web deflections, δ_0, and the out-of-plane movements, δ_r, under applied live loads in terms of $\delta_r/h = f(\sigma, \delta_0, \gamma/\gamma^*)$, where σ is the calculated bending stress at the extrem fiber of the web and γ/γ^* is the ratio of the relative rigidity ratio of the horizontal stiffener.

III Application of Hybrid Girders to Highway Bridges

Two composite hybrid girder bridges constructed in Japan are introduced in Table 4 as examples of actual highway bridges. Furthermore, including these bridges, the strength and economy of composite and non-composite hybrid girder bridges are discussed.

III-1 Design Problems

In design of hybrid girders, a reduction factor, R, specified at the AASHTO Specifications (3), was used to take into consideration the slight decrease of flexural rigidity due to web yielding; to prevent overstressing at erection, calculated dead load bending stresses at the extreme fiber of web, were limited to 80% of its specified yield stress; and web slenderness ratio were modified with a coefficient $\sqrt{R}$. It is found out that the reduction factor, R, is the most fundamental governing parameter to estimate a flange allowable stress, a web slenderness ratio and even economy of the hybrid girder, and that a further study on the reduction factor R is required.

Regarding fatigue design, it is proved that the initiation of Type 1 crack can be prevented as far as the web slenderness ratio discussed at the previous chapter, are used (1, 10), and Type 2 crack can be prevented if a calculated bending stress at the end of vertical stiffener does not exceed at an allowable fatigue stress at transverse non-load carrying fillet welds. In addition to these cracks, Type 3 crack will be kept away by well controlled welding operations.

III-2 Economy

In Japan, a number of structural steels in various grades can be put to practical use, and a choice of grade of steel, from JIS-SS41 to WES-HW80, is within a designer's choice.

Economy of hybrid girders was examined for hybrid composite girders and non-composite continuous girders in terms of overall costs for combination of different steel grades. The overall costs of the girders including material, fabrication and erection costs were calculated with the costs recommended at the Japan Bridge Construction Association (1972).

Table 4. Composite hybrid girder bridge in Japan

		ARAI Bridge	SORO Bridge
Span (m)		19.2	33.0
Steel sets	U.Flange	Center SM50	Center SM50Y
	Web	SS41	SS41
	L.Flange	HW70	SM58
Reduction factor		R = 0.721 (SS41 - HW70)	R = 0.877 (SS41- SM58)
Completed time		June 1974	April 1976

Table 5. Design requirements

Loading	TL- 20
Road width (m)	7.5 + 2X1.5 (footway)
Span (m)	26, 32, 36, 44
Web depth	$0.9h_w$, $1.0h_w$, $1.1h_w$
Steel sets	A, B, C, D, (Table 6)

h_w : Web depth of standard design

(1) Hybrid composite girders

Economical comparisons of hybrid composite girders with homogeneous ones were conducted under such design conditions as shown in Table 5. Structural proportions of homogeneous composite girders were designed based on the "Standard Design of Composite Girder Bridges (1972)", published at the Ministry of Construction in Japan.

The relationship between bridge spans and steel weights per unit area in various sets of steel with a girder depth of 1.0 h_w, are indicated in Fig. 6. A study of Fig. 6 reveals that the weight for all of the steel sets of the hybrid girders except 'A' are lighter than those of the homogeneous girders by the standard design. Especially, the weight of the 'D' set is the lightest among the steel sets for spans longer than 38 meters.

A comparison of overall costs of the hybrid girders with those of the homogeneous girders for various steel combination in Table 6, is shown in Fig. 7 with regard to bridge spans.

It can be said that the economy of hybrid girders is noticeable for the steel combination of 'D', 'C', 'B' and 'A' in this order, and 'D' of which cost ratio (=hybrid/homogeneous) is 0.93, will be the most economical.

Table 6. Steel sets for comparative design

	Standard design	A	B	C	D
Upper flange	SM50Y	SM50	SM50	SM50	SM58
Web	SM50Y	SS41	SS41	SM41	SM41
Lower flange	SM50Y	SM50	SM58	HW70	HW70
Web hight	h_w	$0.9h_w$, h_w, $1.1h_w$			h_w

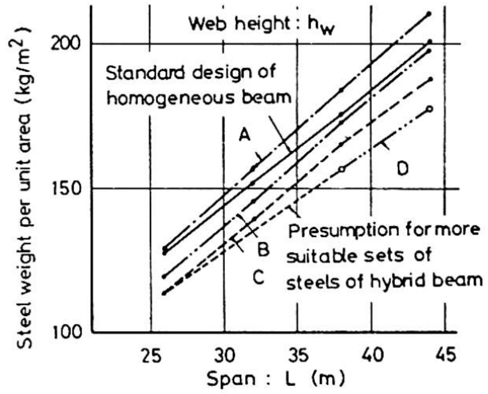

Fig. 6. Weight comparison of hybrid composite girders with homogeneous ones

(2) Three-span continuous non-composite girders

A cost comparison was carried out on three-span continuous non-composite girders with two kinds of equal span, namely 60 meters and 70 meters, under the same design requirements as the former examples. Girder depth in this comparison are fixed to one-twenty first of each span. Consequently, it is proved that hybrid girders can save 3% to 7% in overall cost more than homogeneous girders as shown in Table 7.

Table 7. Cost ratio (Hybrid/Homogeneous) for steel sets

		Homogeneous	Hybrid I	Hybrid II a)
Steel set	U. Flange Web L. Flange	SM50Y SM50Y SM50Y	SM58 SM41 SM58	HW70(SM58) SM41(SS41) HW70(SM58)
3×60 m 180 m	Total weight (t)	496	465	428
	Cost ratio	1.00	0.94	0.97
3×70 m 210 m	Total weight(t)	671	627	586
	Cost ratio	1.00	0.93	0.97

a) ( ) is for section at center span

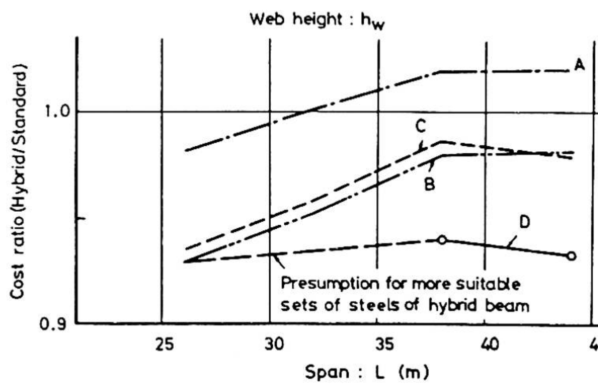

Fig. 7. Cost comparison of hybrid composite girders with homogeneous ones

IV Conclusion

It is concluded for the static behavior that the ultimate flexural strength of hybrid girders can be evaluated well by the reduction factor formula specified at the AASHTO Specifications and their flange yield moment can be carried with single horizontal stiffener up to their web slenderness ratio of about 320 and 290 for the aspect ratio of 0.5 and 1.0, respectively.

There is a possibility of initiation of Type 1 or Type 2 fatigue crack, but the former crack can be prevented for 2×10^6 cycles of loading by limiting out-of-plane movement of web, and the latter crack by controlling a tensile web stress below the fatigue strength of transverse non-load carrying fillet welded joints.

It has been found out that hybrid girder bridges are generally more economic than homogeneous girder bridges in terms of overall cost, particularly at composite girder bridges, but in long-span continuous plate girders it is required to do more detailed comparison between hybrid and homogeneous girder bridges.

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SUMMARY

For the purpose of optimum use of high-strength steels, static and fatigue behaviour of thin-walled, stiffened plate girders are studied. Design and economy of hybrid girders bridges are discussed with illustrations.

RESUME

Le comportement statique et de fatigue des poutres à âme pleine, mince et raidie est étudié en vue de l'utilisation optimum des aciers à haute résistance. Le calcul et l'économie de ponts à poutres hybrides sont discutés et illustrés.

ZUSAMMENFASSUNG

Zur optimalen Verwendung der Hochfestigkeitsstähle werden das statische Verhalten sowie Ermüdungsverhalten dünnwandiger und verfestigter Vollwandträger untersucht. Wirtschaftlichkeit der Vollwandträgerbrücken werden zusammen mit Bildern erörtert.