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1 Introduction

In recent years, efforts have been made to exploit important coal reserves in the Bukit Asam district of Sumatra in equatorial Indonesia. Mining and loading facilities have been erected, and a rail line with its terminus at the port of Panjang, Lampung, reaches as far as Muaraenim. With transport from loading base in the mine to the railhead in Muaraenim limited to overland trucks, efficiency has been severely hampered. To remedy this problem, the Indonesia State Railway PJKA (Persahaan Jawatan Kereta AP) proposed a railway extension into the Bukit District.

This project involved erection of a combined rail and highway bridge over the Enim River, a tributary of the Musi, at a site in an undeveloped wilderness area 200 km from Palembang, the capital of Sumatra Selam (**Fig. 1**). Quick erection of this bridge was considered a key to the early completion of the entire project.

Most bridge works in Indonesia involve the reconstruction of medium- and short-span bridges. For this reason, the large scale of the Enim River project posed a challenge, and techniques uncommon to Indonesia were required, including, in terms of design, partial composite girder construction as well as use of a canti-

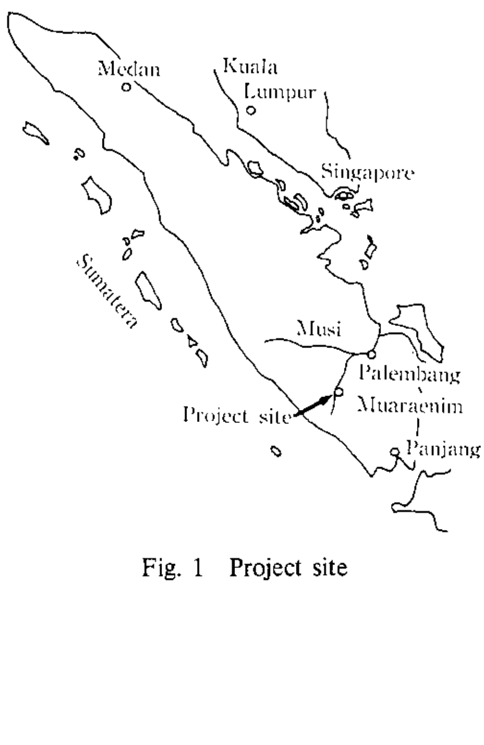

Fig. 1 Project site

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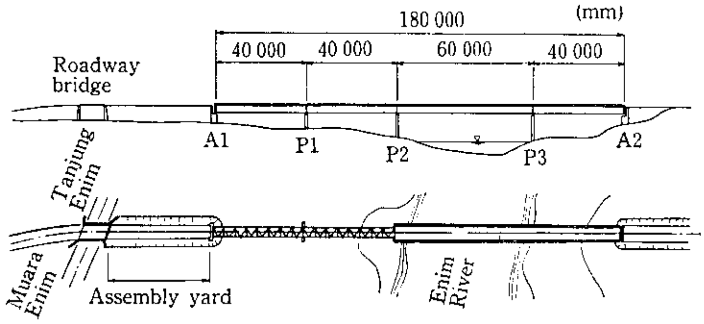


Fig. 2 General plan

lever launching method employing a repetitive erector in the actual work. Construction of the bridge substructure was a particular problem as part of the work was performed during rainy season. This resulted in a delay of about one year. Nevertheless, following the start of design work in 1983, the project was completed in December 1985.

This report describes the design of the Enim River Bridge and the techniques employed in its erection.

2 Outline of Work

An outline of this bridge is given below and the general plan is shown in **Fig. 2**.

Structure type: Four-span continuous partial composite girder type
Bridge length: 181.00 m
Span: 40.00 + 40.00 + 60.00 + 40.00 m
Width: 4.40-m one-lane roadway with single track railway
2 @ 0.8-m sidewalk (both sides)
Train load: PJKA 1st standard train (axial load: 18 t)
Highway load: BINA MARGA 70% loading
Longitudinal slope: Level
Crossfall: 3.0% linear
Deck slab: Reinforced concrete deck slab ($t = 31.0$ cm)
Pavement: Asphalt pavement (Roadway)
Concrete pavement (Track)
Earthquake coefficient: $K_H = 0.14$
Steel grade: SM53C, SM50YB and SM41A (JIS G3106)
Steel weight: 371 t

3 Design of Bridge Superstructure

3.1 Structure Type

Although this bridge was originally planned exclusively to pass coal freight cars from the coal shipping base, the plan was modified into a combination bridge of railway and highway using a single deck slab because there was no access road to the other side of the Enim River near the bridge site.

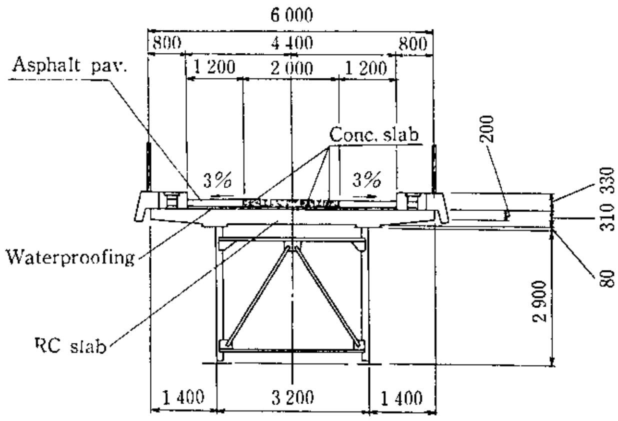


Fig. 3 Cross section of girder

As is apparent from **Fig. 3** showing the cross section of a girder, the track is directly fastened to the slab, and the road surface is paved with asphalt to the top level of rails. This results in an increase in the dead load. On the basis of the strong request from PJKA, which is the owner of the bridge, live-load-partial-composite-girders were adopted in consideration of the economy of the railway-highway combination bridge, the proof loading for a continuous girder, and smooth accommodation of railway and automotive loads.

3.2 Loads

3.2.1 Highway Load

The highway load conforms to 70% of the loading levels provided in the Loading Specification[1)] of the Ministry of Public Works of Indonesia (BINA MARGA). This load is composed of the concentrated load and uniform load given by the following equations and is characterized in that the way of giving loads is in line with the Standard Specification for Highway

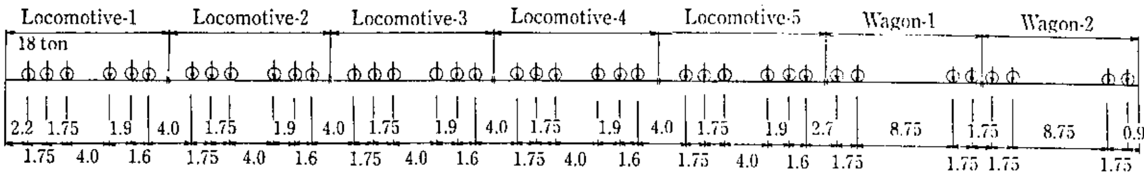


Fig. 4 PJKA first standard train loading

Bridges of the American Association of State Highway and Transportation Officials[2] and that the application of loading is in line with the Specifications for Highway Bridges of the Japan Road Associatrion[3].

Concentrated load: $P = 12$ t

$$\text{Uniform load (t/m): } P = 2.2 - \frac{1.1}{60}(L_1 - 30)$$

where L_1 is the span length (m).

With this 70% loading, the stress due to a concentrated load is about 30% of the TL-14 specified in the Specifications for Highway Bridges of the Japan Road Association and the stress due to a uniform load is about 1.5 times as high as TL-14. However, the stress due to this 70% loading is almost equivalent to TL-14 in total.

3.2.2 Train load

Since this bridge is provided with a track dedicated to coal transportation, the train load peculiar to the main track with an axial load of 18 t shown in **Fig. 4** was adopted.

3.2.3 Impact

The impact coefficient is given by the following equations for each load:

For the highway road

$$i = 20/(L + 50),$$

for the train load

$$i = 0.1 + 22.50/(L + 50),$$

where L is the length of the base line of influence line of the same symbol that generates a maximum stress in a member.

Figure 5 gives a comparison of the impact coefficient for the train load with that adopted by the defunct Japanese National Railways[4]. It is apparent that the two curves almost approximate to each other, showing similar values.

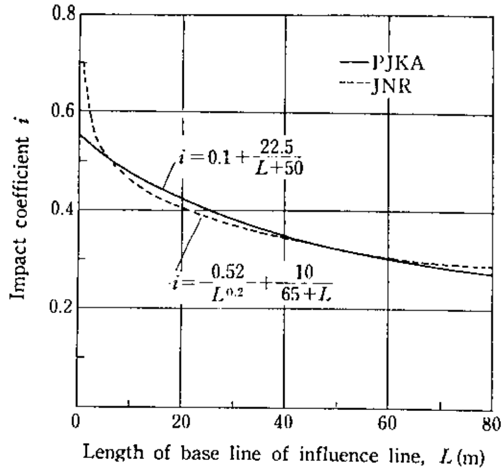


Fig. 5 Impact coefficient of train loading

3.3 Determination of Noncomposite Length

Because the bending behavior of partial composite girders has not yet been fully clarified, there are only a few cases where girders of this type were used in construction work. However, the structure type adopted in this project is based on a rational concept that the drawbacks of continuous composite girders without prestressing are effectively compensated for, as described below.

Nevertheless, partial composite girders suffer from the problem that the concentration of horizontal shearing forces on the boundary between the composite and noncomposite sections and a decrease in the proof stress occur with increasing noncomposite length.

The amount of reinforcement and noncomposite length ratio (ratio of the noncomposite area to the negative moment area) are important factors, as much as the negative moment area, for cracks in concrete floor slabs at the boundary between composite and noncomposite sections. In designing partial composite girders, therefore, it is necessary to pay attention to these factors along with the design method for girders of this type. A laboratory test[5] of cracks in concrete floor slabs of partial composite girders has revealed the following results concerning the amount of reinforcement and noncomposite length ratio:

(1) The amount of reinforcement has a great effect on the crack width. The larger the amount of reinforcement, the smaller the crack width.
(2) The noncomposite length affects the number of cracked floor slabs. The shorter the noncomposite length, the larger the number of cracked floor slabs.
(3) There is a relationship between the noncomposite length and the stress in floor slabs near the intermediate support. The stress in the distribution reinforcement decreases with increasing noncomposite length.

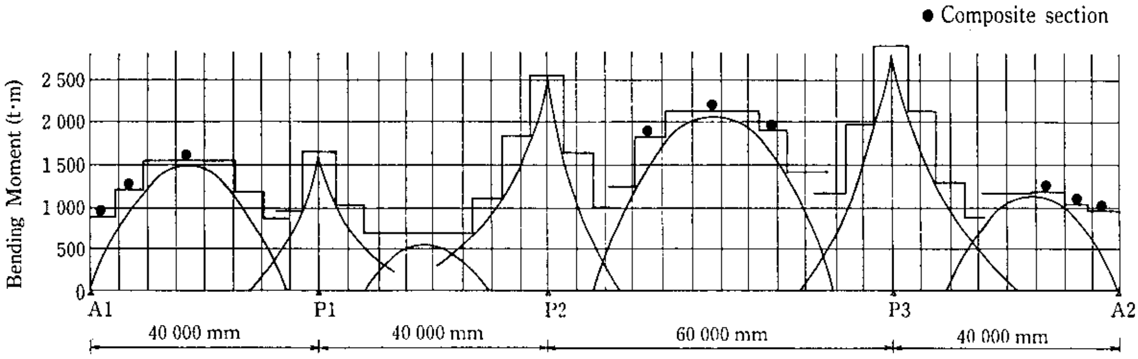


Fig. 6 Moment diagram and composite section

In view of the foregoing, the negative moment area was regarded as the noncomposite section in designing this bridge in order to minimize cracks formed in concrete floor slabs. Generally in such a case, the horizontal shearing force at the boundary between composite and noncomposite sections increases. Therefore, slab anchors were installed to prevent cracks at this boundary and to decrease abrupt changes in the section. This is because shear reinforcement does not contribute greatly to an increase in the proof stress.[6]

For this bridge, composite and noncomposite lengths were determined as shown in **Fig. 6** in consideration of changes in girder sections and the moment distribution.

3.4 Waterproofing Work

Cracks are apt to occur in concrete floor slabs at the boundary between composite and noncomposite sections and near the intermediate supporting point. However, a water-proofing layer (bituminous material thickness $t = 30$ mm) was spread to the full length of the bridge surface to prevent the corrosion of reinforcement due to cracks. This prevents the inflow of rainwater even if cracks are formed.

4 Selection of Erection Method

4.1 Geographical Conditions

The space below girders can be used under the existing land utilization condition and geology at the bridge site. However, the bridge erection was to be carried out in the rainy season and the ground condition was bad, thus making it difficult to carry in heavy equipment. For this reason, the construction method using mainly track cranes, etc. posed problems in safety as wells as in the execution of work. Furthermore, for the adjoining area in the axial direction of the bridge, a length of about 50 m can be secured between the abutment A1 and the approach road on the right riverside, whereas on the left riverside the girder transportation by trailer was impossible due to the narrow width of the access road. Therefore, the transportation of bridge members during the bridge erection is limited to one direction from the right riverside.

4.2 Erection Machinery and Materials

Basically, it was decided to locally procure erection machinery and materials from the viewpoint of economy. Depending on the distribution condition on the market, however, there are limits to capacities and quantities of procurable machinery and materials. Although the transportation route for carrying in machinery and materials has been established, the local procurement of special machinery and materials, such as heavy equipment with an ample capacity, erection trusses and cable cranes, is not economical in the light of the large scale execution of the work. Furthermore, the construction method requiring a large amount of timbering, such as bents and staging, is subject to limitations in the process. For this reason, it was necessary to adopt a construction method that can be carried out with small-scale equipment and materials capable of local procurement.

4.3 Selection of Erection Method

The ease of execution of work, safety and economy of various construction methods resulting from the abovementioned geographic conditions and the local availability or non-availability of erection machinery and materials were all taken into consideration in selecting the erection method. As a result, it was judged that the cantilever launching method, which requires no special equipment and only small-scale machinery and materials, is superior to other methods in all respects and is most suited to the present project. Because the assembly yard available was only about 50 m in length, the repetitive erector method was adopted in combination by which girder assembling stages are set beforehand and cantilever launching and assembling of girders are alternately conducted while a constant grasp of the shape of each girder in each stage is obtained in order to ensure smooth execution of the work.

5 Erection Work

5.1 Cantilever Launching Equipment

The equipment on each abutment and pier comprising the support of the girder is composed of rollers, a hydraulic jack and guide timbering. The rollers used were those with a maximum capacity of 60 t called Tirtank.

The use of this Tirtank permits the girder to be supported in a condition almost plane, thus relaxing the stress concentration on the web plate and increasing the safety against local buckling. Furthermore, no special working is required for the points where the girder section changes or field joints. Therefore, the efficiency of work is high. For these reasons, the safety and ease of work during cantilever launching are considerably improved.

The condition of the launching equipment at each support is shown in **Photo 1**.

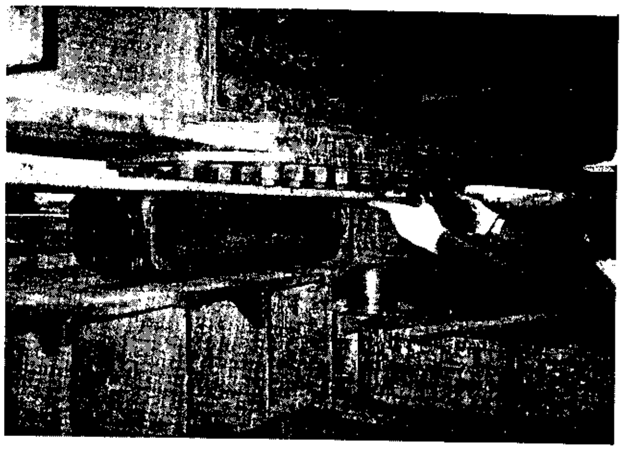


Photo 1 Equipment at each support

5.2 Procedure for Erection

As shown in **Fig. 7**, three or four girders are assembled as a unit one after another on the launching line and are launched after the fastening with high-strength bolts. After all girder blocks are assembled and launched to the prescribed positions, the girders are lowered and bearings are fixed. After that, the erection nose is removed, concrete is placed for the abutment parapet whose space has been left to ensure the safety in cantilever launching, and the earth at the back of the parapet is back-filled. How the cantilever is launched is shown in **Photo 2**.

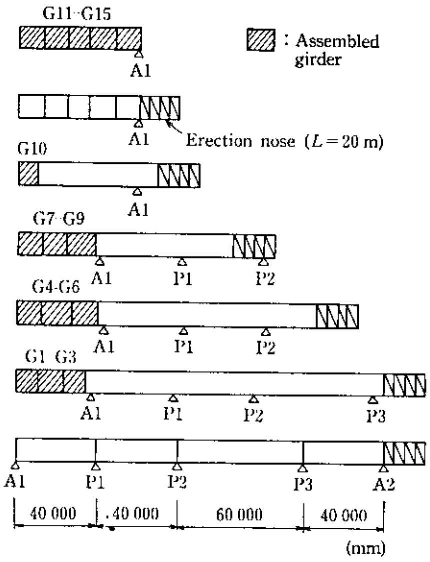


Fig. 7 Procedure of girder assembling and cantilever launching

5.3 Cantilever Launching Work

For the case where a point of change in the section of the lower flange and a field joint pass the support, the

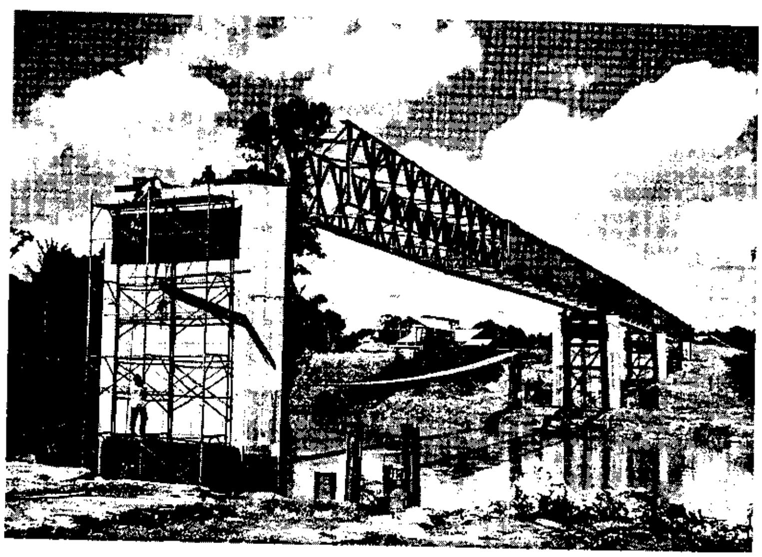


Photo 2
Cantilever launching

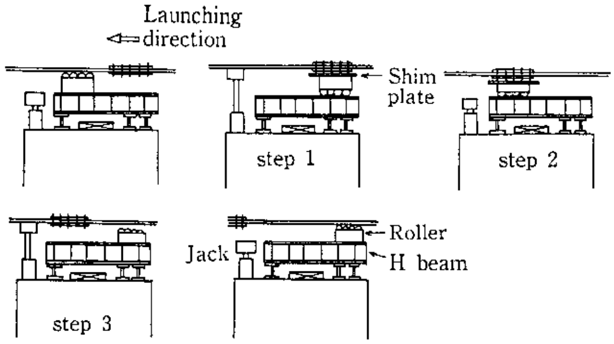
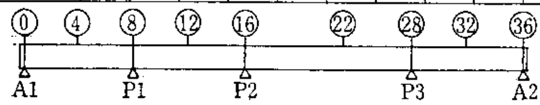


Fig. 8 Work flow at each support

Table 1 Camber

Measurement point		0	4	8	12	16	22	28	32	36
Shop Assembly	GA*	0	−1	+5	+6	+1	+7	+4	+4	0
	GB**	0	−2	+4	+5	+3	+7	+7	+4	0
Site Erection	GA*	−3	−7	+2	+5	+3	+9	+1	−1	0
	GB**	−3	−9	+2	+2	+2	+6	−1	−2	0



*GA: The upper stream girder
**GB: The lower stream girder

same system with the work procedure using launching equipment comprising a hydraulic jack and a sliding plate was adopted, and for other cases, the roller fixing system was used. The flow of work, when the field joint has reached the support in **Fig. 8**, is as follows:

(1) After jacking up the girder, the rollers are reversed and a shim plate is inserted in between the rollers and the lower flange (step 1).
(2) After ascertaining the roller arrangement, the cantilever is launched again (step 2).
(3) The instant the rollers have reached the edge of the guide timbering, the girder is jacked up and the rollers are returned to the initial position after reversing the rollers again (step 3)

This procedure aims at giving field workers a thorough understanding of instructions by simplifying work and, at the same time, at improving the work efficiency and simplifying the control of execution work.

Incidentally, the cantilever launching speed was 10 to 15 cm/min and the average length launched a day was about 10 m.

5.4 Camber Control

Because the camber shape during the assembling of the girders varies in each assembling stage, camber control was conducted by the repetitive erector method by which the camber shape during shop assembly is reproduced in the field based on the position of bolt holes. **Table 1** gives values of camber during shop assembly and upon completion of site erection.

This bridge was constructed based on the full-size drilling with satisfactory results.

5.5 Level Adjustment

The datum line of cantilever launching was set at a height of 650 mm from the levee crown of pier of each abutment, and the cantilever launching work was carried out along this datum line. The girder shape during launching was controlled by adjusting the level of the roller receiving point at each support with respect to the datum line depending on the amount of camber due to the dead load which changes every moment. Furthermore, when the changing point of the girder section and the field joint passed a support, level adjustments were made for other supports as required. The generation of unequal loads was prevented in this manner.

5.6 Adjustment of Direction of Launching

The direction of girder launching is determined by the installation accuracy of the rollers. The eccentricity perpendicular to the bridge axis during erection was linear and the whole girder tended to rotate around a certain center. This eccentricity was corrected by adjusting the launching direction of the rollers in the direction opposite to the direction of eccentricity at each support while constantly measuring the amount of eccentricity between the bridge axis line and the datum line. Incidentally, because there was a possibility of girder buckling due to a shift of the working point of the vertical load, the launching direction from the rollers was set at a ratio of 1:50 maximum with respect to the datum line and the strict measures were taken by establishing a severe measurement control system.

Amounts of eccentricity perpendicular to the bridge axis upon completion of the erection are given in **Table 2.**

6 Examination of Stresses during Erection

6.1 Design Conditions

The structure system during girder launching turns into that of a cantilever, differing from the completed structure system. For this reason, it is necessary to examine the working stress generated in the bridge due

Table 2 Alignment

(mm)

Measurement point	Alignment*
A1**	+50
P1	+ 5
P2	0
P3	− 1
A2	0

* Upperstream "−", downstream "+"
** Support free

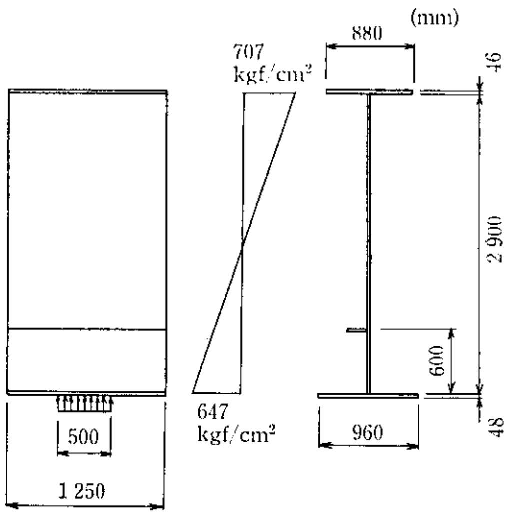


Fig. 9 Panel section and working stress

to the weights of the girders and temporary structures. Here is an example of checking in accordance with the Guideline 012 of DASt[7] with respect to a buckling analysis of the web plate of a plate girder subjected to local loads.

By paying attention to a case where the girder is subjected to a maximum reaction during the construction period of this work, a partial panel enclosed by vertical stiffeners is selected as the section to be checked for buckling. It is assumed that local loads are uniformly distributed at the middle of the bottom side of the panel. The section to be checked for buckling and standard stresses are shown in **Fig. 9**. The working stresses generated in this case are as follows:

Bending moment = 957.6 t · m
Shearing force = 48.6 t
Reaction force = 91.2 t

6.2 Stress Distribution and Safety Factor of Buckling

In the section shown in Fig. 9, a vertical stress $\sigma_y(Y)$

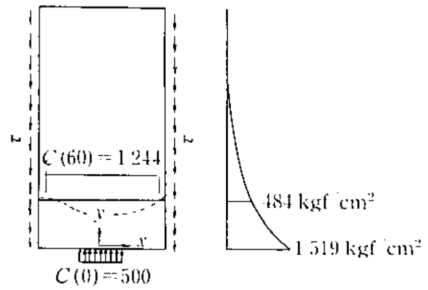


Fig. 10 Working stress due to vertical load

Table 3 Working stress

y (cm)	σ_x (kgf/cm²)	σ_y (kgf/cm²)	τ (kgf/cm²)	σ_v (kgf/cm²)
0	647	1 519	140	1 342
60	354	484	140	497
290	−707	0	140	747

is generated by the introduction of local loads. The stress distribution in a partial panel can be determined by calculating σ_v defined by the following equation in which this $\sigma_y(Y)$ is taken into consideration:

$$\sigma_v = \sigma_x^2 + \sigma_y^2 - \sigma_x\sigma_y + 3\tau^2$$

The stress diagram is shown in **Fig. 10** and the stress distribution is given in **Table 3**.

Based on the above-mentioned results of calculation, the safety factor of buckling is then found. In this bridge, the existing safety factor of buckling, vorh ν_B, at a web plate thickness of 12 mm were as follows:

vorh $\nu_B = 1.08$
erf $\nu_B = 1.49$

Similarly, the following results were obtained in a section with a web plate thickness of 16 mm:

vorh $\nu_B = 2.38$
erf $\nu_B = 1.48$

Accordingly, the relationship vorh $\nu_B >$ erf ν_B is satisfied (for a detailed calculation flow, refer to the literature[8]).

6.3 Stiffening of Web Plate

Based on the results of the checking for buckling during cantilever launching, the bridge was reinforced by increasing the web plate thickness in order to prevent the local buckling of the web plate at the supporting point of the launching equipment. **Fig. 11** shows places where the preliminary design section was increased in

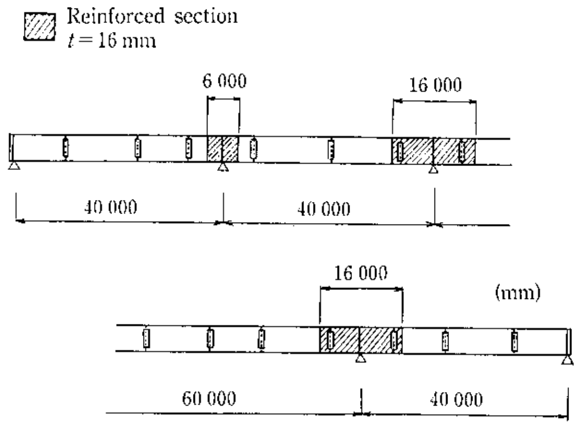


Fig. 11 Reinforced girder section

thickness t from 12 to 16 mm.

The buckling analysis in accordance with the Guideline 012 of DASt is not based on the use of computers and it seems that this analysis method is effective even when employed in the field.

7 Conclusions

Kawasaki Steel was awarded a contract for the erection of a bridge by Indonesia. This 181-m bridge, situated in the Bukit Asam district of Sumatra, was completed in December 1985. The present report has described the design and erection of this bridge. The following points were of note in connection with the design and erection work:

(1) A four-span continuous partial composite girder structure was adopted as the structure type. About 36% of the full girder length is of composite sections, in consideration of changes in girder section and the moment distribution.

(2) The cantilever launching method using a repetitive erector method was adopted in the erection of the bridge; cambers obtained in the shop assembly were reproduced with high accuracy on site.

(3) The eccentricity of the girders relative to the datum line of bridge axis in the course of bridge erection was easily corrected by adjusting the direction of installation of the rollers at each support in the direction opposite to eccentricity.

(4) It was considered that a Guideline 012 of DASt is effective with respect to the buckling analysis of girders during erection.

In this undertaking, Kawasaki Steel bore great responsibility as the opening of the bridge was a key to the early completion of a major project aimed at improving transportation facilities. Although for various reasons two years elapsed from the start of design and fabrication to completion, actual assembly of the girders on site required only two months. This work was the largest bridge erection project carried out by the cantilever launching method in Indonesia to date. The work was executed with a high degree of accuracy and without accident, in spite of the necessity of the use of machinery and materials locally procured. Furthermore, experience in the construction of this bridge should play a ground breaking role in design in Indonesia.

The authors hope that the Enim River Bridge will attain a place in the history of bridge erection work in Indonesia, and will serve as a friendship bridge between Indonesia and Japan.

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