Evaluation of the Real Loading Capacity Carried on Existing Plate Girder Bridges with RC Slab

Masaki YAMADA[1], Daisuke MITA[2] and Chitoshi MIKI[3]

Department of Civil Engineering, Tokyo Institute of Technology, 2-12-1 Oookayama, Meguro-ku Tokyo 152-8552, Japan

ABSTRACT

There are two types of plate girder bridge with RC slab. One is composite plate girder bridge, and the other is non-composite plate girder bridge. About 70% of Japanese road bridges has been designed as non-composite. But it is said that non-composite plate girder bridge behaves as composite plate girder bridge, and it has superior loading capacity. For these reasons, the object of this study is to proof the real loading capacity carried on non-composite plate girder bridge to measure by L25 load. We carried out proof loading tests on in use expressway bridges and FEM analysis on detailed models. And as the result, it became clear that plate girder bridge with RC slab designed by usual code has enough loading capacity to the increased loads.

INTRODUCTION

In recent years, Japanese bridges are applied heavier load than design load because of the increase of traffic volume and increase of heavy vehicles. And dead load is also increasing because of retrofitting or so on. For example, concrete layered on RC slab's surface against aging, inspection passageways or handrails for safety, soundproofed walls for surrounding environment. For that increase of dead load and traffic volume, retrofitting works are needed to many old bridges.

---

1) Doctoral Student
2) Graduate of Master Course
3) Professor

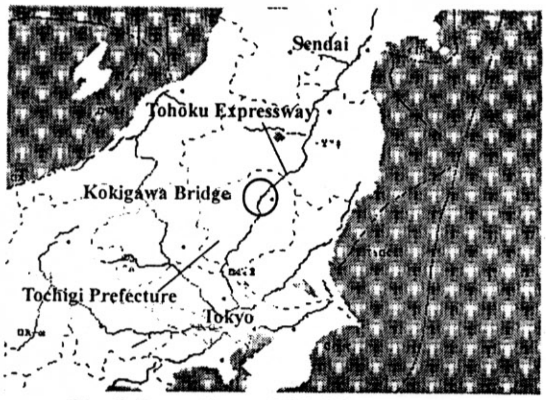

Fig.1 Location of Hokigawa Bridge

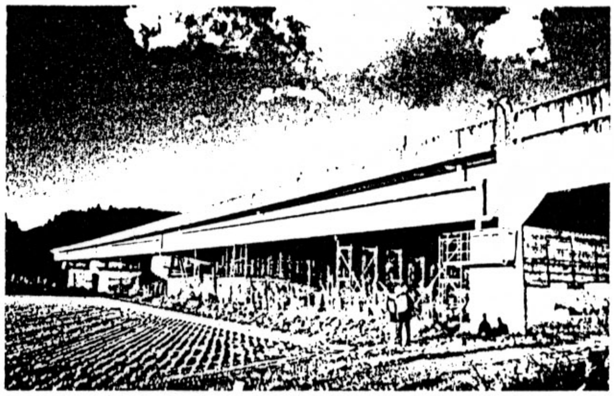

Fig.2 View of Hokigawa Bridge

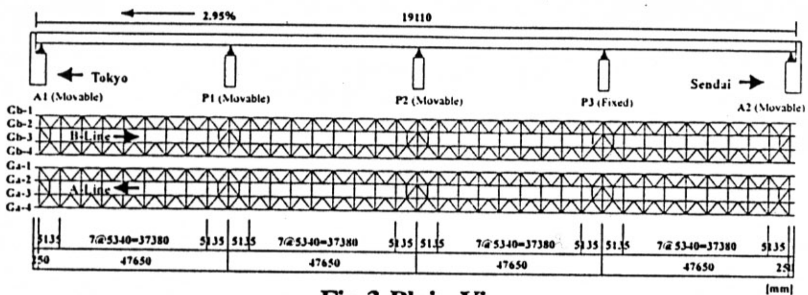

Fig.3 Plain View

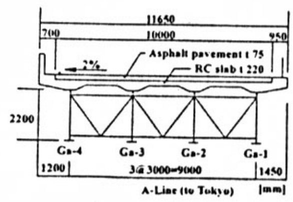

Fig.4 Cross-sectional View

About 70% of Japanese road bridges are designed as a non-composite bridges. In code, non-composite plate girder bridge is designed on the assumption that the load is carried on only plate girders. Because RC slab is regarded as not carrying load to longitudinal direction. RC slab's action is mainly to distribute load applied on its surface to each plate girders and it isn't assumed as structural part to longitudinal direction. But actually there are reports about composite work of slab and plate girders. There is possibility that the slab and girders work together and non-composite plate girder bridge has more loading capacity than its specification. The design load of Japanese road bridge code has been increased from TL20 to TL25 in 1992. If we can use a non-composite plate girder bridge as a composite plate girder bridge, it is adapted for new code. For these reasons, this study aims at evaluating the real loading capacity of existing non-composite plate girder bridges with RC slab. For these purposes I'll discuss real loading test and FEM analyses.

Table1 Specifications of Hokigawa Bridge

Type	Continious Non-composit Steel Plategirder Bridge
Total Length	191.10 m
Span Length	4 @ 47.65 = 190.60 m
With	10.00 m
Design Load	TL20
Pavement	Athpalt t = 75 mm
Slab	Reinforced Concrete t = 220 mm
Gladient	Lonitudinal 2.95% Closs-sectional 2%

THE HOKIGAWA BRIDGE

The objective bridge is the Hokigawa Bridge, A-Line (from Sendai to Tokyo) in Tohoku expressway. It stands at Nishi-Nasuno city ,Tochigi Prefecture crossing the Hoki River (**Fig.1**). It is RC slab, 4 span continuos non-composite steel plate girder bridge and a typical type of road bridge. It was designed by TL20 load and constructed in 1973. 25 years old and some fatigue clack were observed. The view and specifications are shown in **Table1** and **Fig.2,3,4**.

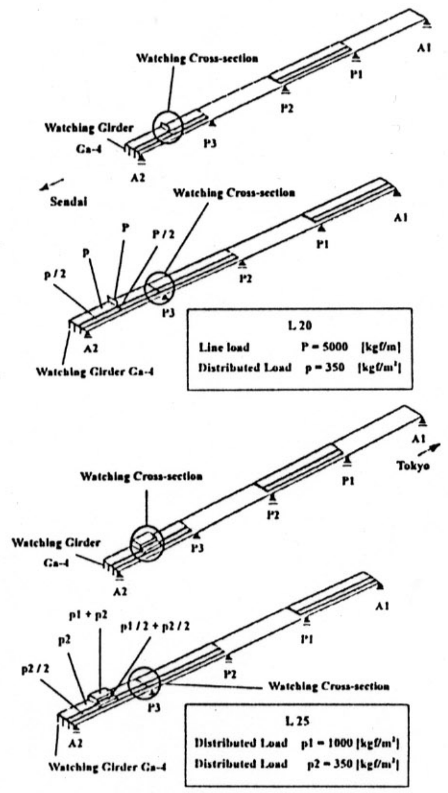

Fig.5 Loading Patterns of L20 &L25

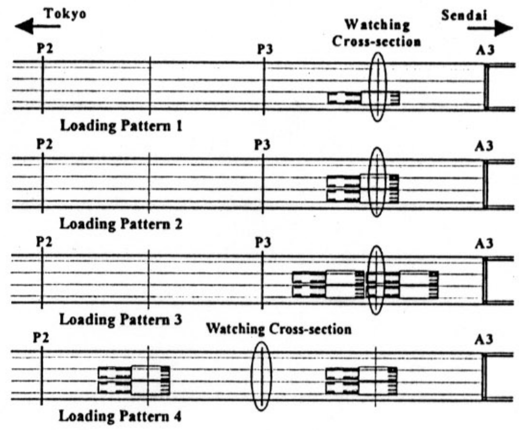

Fig.6 Loading Patterns of Loading Test

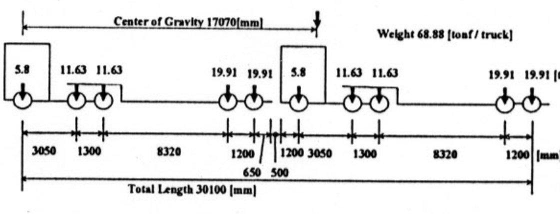

Fig.7 Axial Forces of Trailer-truck

COMPARISON L20, L25 LOAD AND REAL LOADING TEST PATTERNS

We planed for a real loading test to confirm the loading capacity of Hokigawa Bridge by loading L20 and L25 load. But it is difficult to apply L20 or L25 load to real bridge because of large distributed load, traffic volume and speed. For these reasons, we planed to use some heavy-duty trucks to make much the same bending moment as L20 and L25 load on watching cross-section. The trailer-truck weigh about 69 tons assuming over loaded vehicle. The maximum positive bending moment appears at the center of span, and the maximum negative bending moment appears at internal shoe. In this paper, we defined two cross-sections as watching cross-sections, one is the center of span P3-A2 and the other is the cross-section at shoe P3. And especially we pay attention the most outside plate girder Ga-4

There are two loading patterns to each L20 and L25 load. Each one makes the largest bending moment at center of span P3-A2 and at shoe P2. Each loading patterns of L20 and two L25 are shown in **Fig.5**. In each loading patterns, bending moment at Ga-4 in watching cross-section is maximum. And, I show the 4 loading patterns and amount of axial load of trailer-truck in **Fig.6, 7** and. Pattern 1,2 and 3 are to watch behaviors of central area, span A2-P3. Pattern 4 is to watch near shoe P3. The Hokigawa Bridge was designed by Guyon-Massonnet method and L20 load. We calculate the bending moment by Gyon-Massoment method in design specifications. Each load is distributed to 4 girders by inference lines in specification. And vertical displacement and moments are calculated by simple FEM which composed of 288 beam elements **(Fig.8)**. Specifications of steel plate girders are shown in **Table 2.**

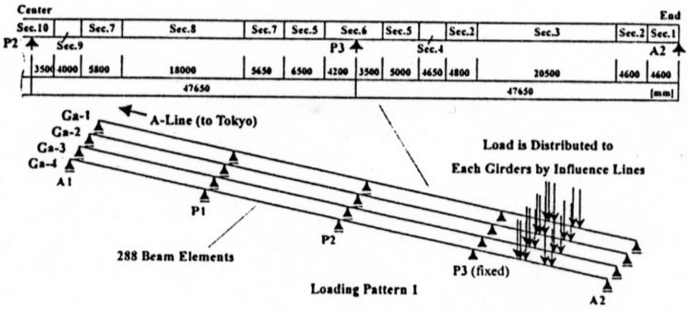

Fig.8 Guyon-Massonnet Method

Table2 Specifications of Girders

Ga-4	Sec.1	Sec.2	Sec.3	Sec.4	Sec.5	Sec.6	Sec.7	Sec.8	Sec.9	Sec.10
U.Flg	470*22	470*25	590*30	470*22	520*25	690*34	380*22	380*22	380*22	620*25
Web	2200*11 [mm]									
L.Flg	470*22	470*25	590*30	470*22	600*28	720*36	380*22	380*22	500*25	630*30
Ga-1,2,3										
U.Flg	470*19	470*22	570*28	470*19	540*22	650*32	380*19	380*19	380*22	620*22
Web	2200*11 [mm]									
L.Flg	470*19	470*22	570*28	470*19	570*28	720*34	380*19	380*19	460*25	630*28

Table3 Displ. & Moment by Dead Load

	Displ. at the Center of Span A3-P2 [mm]	Bending Moment at the Center of Span A3-P2 [tonf-	Bending Moment at the Shoe P2 [tonf-m]
Dead Load A*	-29.0	155.4	-259.1
Dead Load B**	-67.3	360.5	-599.6
Dead Load A+B	-96.3	515.9	-858.6

*Dead Load After Slab Hardening **Dead Load Before Slab Hardening

Table4 Displ. & Moment by Guyon-Massonnet Method

	Displ. at the Center of Span A3-P2 [mm]	Bending Moment* at the Center of Span A3-P2 [tonf-	Bending Moment* at the Shoe P2 [tonf-m]
L20 Loading	-64.5	838.8	-1177.5
L25 Loading	-85.1	940.6	-1239.3
Pattern 1	-50.1	766.4	
Pattern 2	-70.8	870.3	
Pattern 3	-119.9	1092.5	
Pattern 4	-39.7		-1282.6

*include Bending Moment made by sum of Dead Loads

Table5 Stress of Flange

	Stress of Lower Flange at the Center of Span A3-P2	Stress of Uppre Flange at the Shoe P2 [kgf/cm²]
Dead Load A*	327	-529
Dead Load B**	757	-1225
Dead Load A+B	1084	-1754

*Dead Load After Slab Hardening **Dead Load Before Slab Hardening

The results of calculations are shown in **Table 3** and **Table 4**. In this calculation the two types of dead loads (A and B in **Table3**) are dealt with respectively. Dead load are sorted out distributed load from not-distributed. Dead load made by pavement, wall, wheel guard and so on are constructed after slab hardening, and their loads are distributed to each plate girders. Vertical camber at the center of span P3-A2 is 95 mm, and that is almost equal to the calculated displacements 96.3 mm occurred by sum of both dead loads.

In the table you can see the L20 load's bending moment stands between pattern 1 and pattern 2. And L25 load's bending moment stands between pattern 2 and pattern 3. From these calculation, pattern 3 makes maximum displacement 119.9 mm at the center of the span A2-P3. This displacement is much larger than allowable displacement 95 mm. Pattern 4 makes almost the same moment as that made by L25 loading cross-section at shoe P3.

In the **Table 5**, you can see also stress at the plate girder's flanges made by dead loads. These stress were calculated by cross-sectional shape at watching points. Designed stress is 2100 kgf/cm^2. The stress made by dead loads is 1084 kgf/cm^2 at lower flange of the center of the span A2-P3 and -1754 kgf/cm^2 at upper flange of shoe P3. You can see the moment made by dead loads shear large amount of total moment. And it is important to circulate the effect of dead load for discussing limit state.

REAL LOADING TEST TO EXISTING NON-COMPOSITE PLATE GIRDER BRIDGE

In December 1997, We carried out real loading test to Hokigawa Bridge at Tohoku expressway(**Fig.9**). We measured stresses and displacements and observed under temporally traffic control. The real loading test was carried out quickly but very carefully attending to deformation or sound occurring at wrong part because the maximum bending moment was more than specification's load. But during the loading test, no dangerous sign was observed.

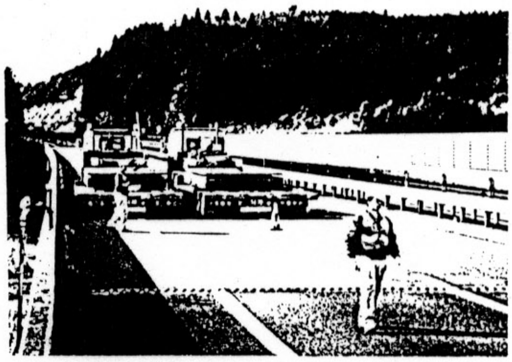

Fig.9 View of Real Loading Test

COMPRESSION RESULTS AND FEM MODELS

Here, the results of loading test and detailed FEM models are compared. The FEM models can expect the difference between the assumption in designing and the real behavior. One is the non-composite FEM model. And the other is composite model. In both model, RC slab and plate girders are connected by beam elements. It has no geometrical moment of area in non-composite FEM model, On the other hand, the element is rigid in composite FEM modal, and, RC slab and plate girder don't work together. The FEM model is shown in **Fig.10, 11.** It has over 14000 elements and sub structural parts such as sway bracing, lateral bracing and so on. Dead load is also dealt with sorted out. Result of loading test are shown in **Fig,12-15**. In them, the displacements at watching points calculated by composite FEM model, non-composite FEM model and Guyon-Massonnet method are also shown. You can see that observed displacements are less than all the calculated. Observed Displacements are close to composite FEM model's. The results from non-composite FEM model are almost the same as Guyon-Massonnet method's.

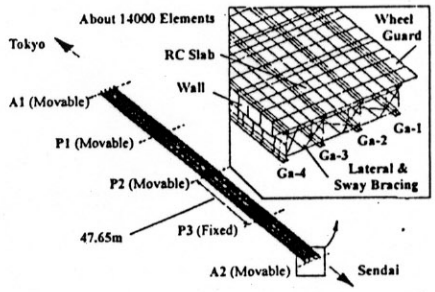

Fig.10 FEM Model

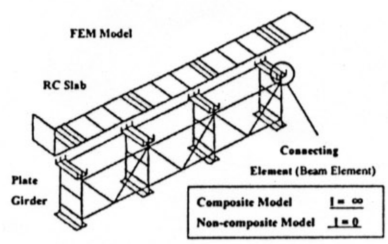

Fig.11 Shear Connector

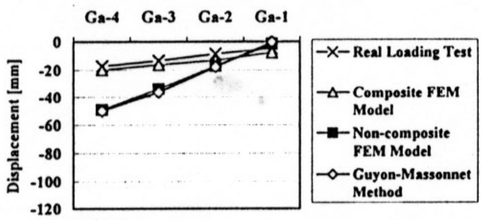

Fig.12 Vertical Displ. of Girders (Pattern 1)

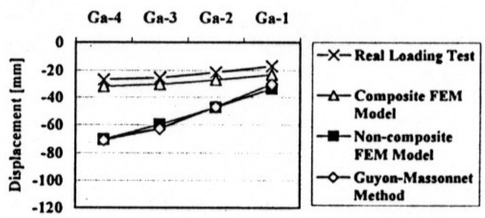

Fig.13 Vertical Displ. of Girders (Pattern 2)

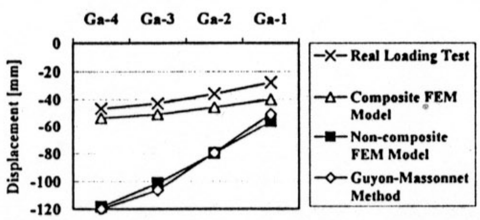

Fig.14 Vertical Displ. of Girders (Pattern 3)

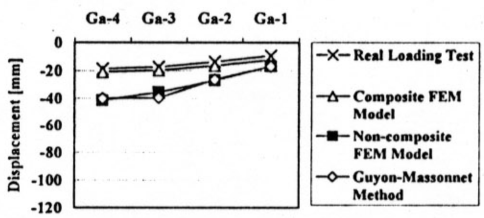

Fig.15 Vertical Displ. of Girders (Pattern 4)

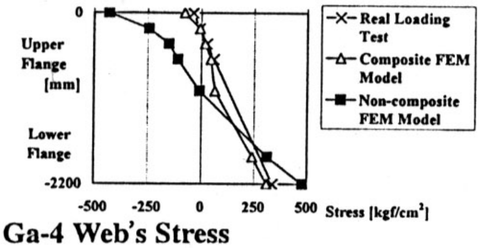

Fig.16 Ga-4 Web's Stress at the center of span P2-A3 (Pattern 1)

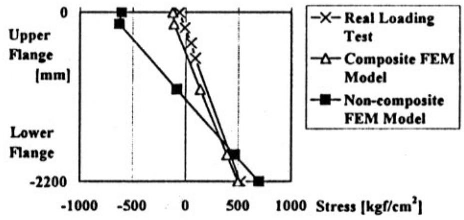

Fig.17 Ga-4 Web's Stress at the center of span P2-A3 (Pattern 2)

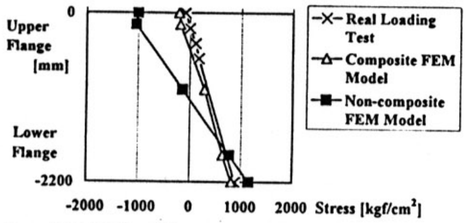

Fig.18 Ga-4 Web's Stress at the center of span P2-A3 (Pattern 3)

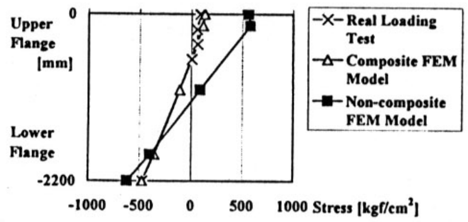

Fig.19 Ga-4 Web's Stress at shoe P2 (Pattern 4)

Table6 Stress of Ga-4 Flange

Stress of Lower Flange at the Center of Span A3-P2 [kgf/cm²]			
	Real Loading Test*	Composite FEM Model	Non-composite FEM Model
Pattern 1	1294	1266	1457
Pattern 2	1478	1457	1677
Pattern 3	1847	1780	2137
Stress of Uppre Flange at the Shoe P2 [kgf/cm²]			
Pattern 4	-1480	-1483	-1655

*include Stress made by dead load calculated by FEM

Ga-4 web's stress are plotted in **Fig.16-19.** From theses result it is clear that the bending center is about 200 mm from upper flange at the center of span A2-P3 and about 600 mm at shoe P3. The level of bending center at both cross-sections are fit for the calculations from composite FEM models. No relative displacement between upper flange and slab are measured in real loading test. It is clear that slab and upper flange works together. For these reasons the Hokigawa Bridge behaved as a composite plate girder bridge. Total stress including dead load's are shown in **Table 6.** All the stress is smaller than 2100 kgf/cm^2, allowable stress. Taking thought of safety factor 1.7, the Hokigawa Bridge has enough loading capacity to L25.

CONCLUSION

During the loading test, we could find no limit state, and after it, no residual displacement. So it is clear that the bridge made elastic deformation. From the comparison loading test's result and FEM model's, it is clear that RC slab and girder works together and the bridge has enough capacity to pattern 3 and pattern 4. So we proofed the Hokigawa Bridge has enough loading capacity to L25. But the composite mechanism of non-composite plate girder is not clear. And there are possibilities making sudden distraction of adhesion between slab and flange or something at the limit state. Next stage I'm going to discus to more heavier loading state.

REFERENCES

"Design specification of road bridges (I common, II steel bridge design)", Japan Road Association,(1990)
Takashima H., (1965), "Practical Method of Load Distribution for Road Bridges", Gendai-sha, Japan.
Takahashi S. et al., "Specification, analyses and test for PC slab 2 Plate Girder Bridge 'Horonai Bridge'" *Bridge and Foundation Engineering*, Vol.30, No.2, Kensetu-tosho, Japan
Takahata T., (1997), "On the Effect of the Existing Bridge-Deck Reinforcement by thickening Concrete Slab" *Chodai Technical Report*, No.6, Chodai, Japan