FEASIBILTY STUDY ON SRC BRIDGES USING STEEL ROLLED H-SECTION

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1. INTRODUCTION

A new form of composite girder bridges using steel rolled-section is proposed in this paper. The super-structure is a continuous girder with steel rolled H-section and the substructure is reinforced concrete pier. The steel girder and the pier are rigidly connected at the pier top by reinforced concrete. This new bridge form is basically the multi-span rigid frame structure: the steel/concrete composite girder resists sectional forces at the span center, and the SRC girder resists at the rigid joints. Fig.1 shows a typical layout of SRC Girder Bridge (Nakamura, 2002).

The bridge using rolled H-section is expected to be economical compared with the welded plate girder bridges due to low material and fabrication cost. Besides, the rolled steel H-section is a compact section and has favorable bending characteristics and no need for stiffeners. The bridge with the rolled H-sections with a web height of 900mm has been used for spans up to 20m to 25m for simple and continuous spans, whereas the proposed SRC girder with the rolled H-section could extend the applicable span length up to 50m. When the structure has a continuous form, the area around the joint is always more critical than that of span center, which limits the span length. Hence the H-girders are strengthened around the joints by being covered with reinforced concrete, which forms SRC and increase the bending capacity of the section.

A rolled H-girder has high ultimate strength with good ductile property, attains full plastic moment and is regarded as the compact section. Therefore, the limit state design method is applied using the plastic moment as ultimate strength. The deflection of the SRC Bridge due to the design live load is expected to be small and satisfy the serviceability limit state. The super-structure is rigidly connected to the pier and seems to have better performance against earthquakes. This study shows that the proposed SRC Bridge using rolled H-section is feasible, economical and has good seismic resisting.

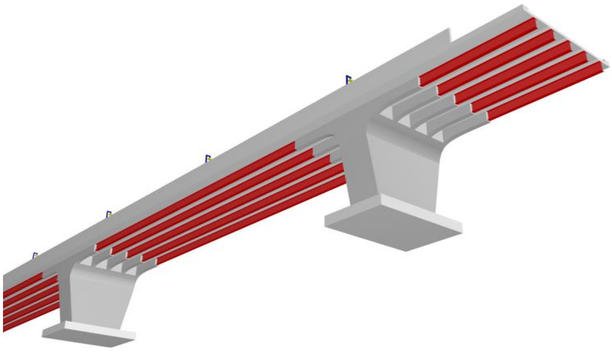

Fig.1 Typical layout of SRC Girder Bridge

2. STRUCTURAL FORM OF THE PROPOSED SRC BRIDGE

A two lane four-span continuous girder bridge with span length of 40+50+50+40m, with five girders of rolled H-section with a web height of 900mm is studied, as shown in Fig.2 and Fig.3. The concrete piers are 10m high and 3m wide, and are rigidly connected to girders. The steel /concrete composite girder is proposed for the sagging bending moment at the span center (Fig.4a), and the steel girder covered by reinforced concrete section is proposed for the hogging bending moment at the rigid joints near support (Fig.4b). The effective flange width is specified as per design code. About 15% of the span length near the joints are covered by RC section.

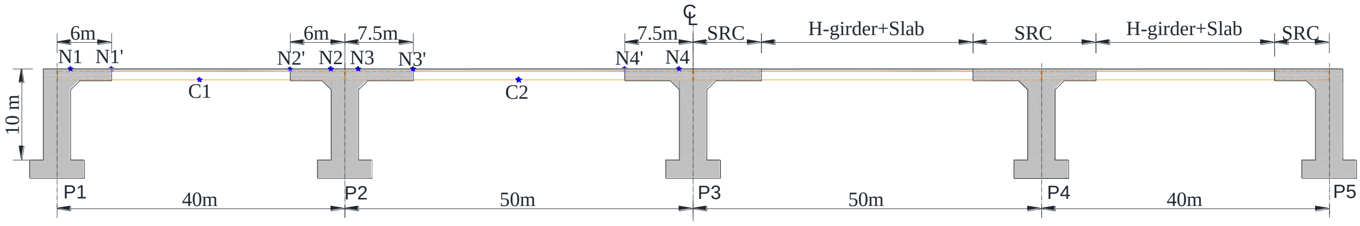

Fig.2 Proposed bridge longitudinal section

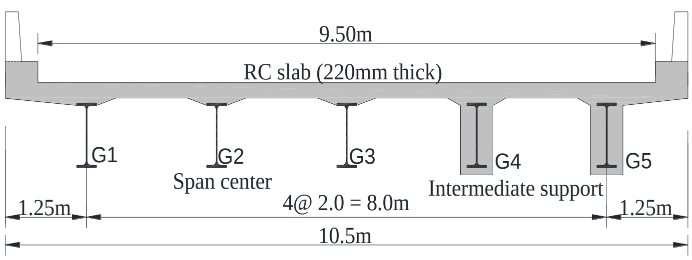

Fig.3 Cross-section of the proposed bridge

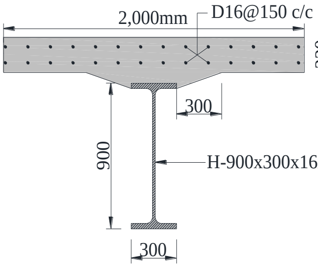

Fig.4a Section at span center

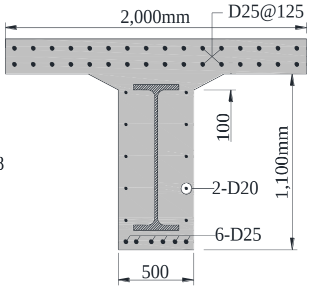

Fig.4b Section at joint

Keywords: New form of composite girder bridge, Steel rolled H-section, limit state design, experiment of bridge module
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3. TRIAL DESIGN

A trail design was carried out for the proposed multi-span SRC Bridge (Fig.2, 3). The static analysis of three dimensional module of the bridge with beam element was performed, using a structural analysis program (CSI SAP. v16). As for the material properties for trail design, SMY490 grade of steel was used for H-girders, and SD390 and SD490 for steel rebar with young's modulus of 205GPa. For the RC slab and piers a simplified model of concrete with the design compressive strength of 40MPa and the initial elastic modulus of 30GPa was used. The two girders, G1 and G3, were taken as the representative girders of the proposed bridge.

3.1 Design Loads

The two kinds of design dead loads (D) of super-structure are considered: pre-composite dead load (D1) due to self-weight of girders, fresh concrete of slab and form work, and the post-composite dead load (D2) due to surface wearing, railings and traffic barriers. The design live load (L) and the impact effect (I) is adopted from Japanese highway bridge specification (Japan Road Association, 2014). The B-live load consists of equivalent concentrated load (p1=10 kN/m^2) with the load length of 10m transversely and fully uniform distributed load of (p2=3.5 kN/m^2) with main lane width of 5.5m along the bridge. The live load impact factor is assumed as 20% based on the span length. Six live load cases, LC1 to LC6, were applied on the bridge model to obtain the maximum and minimum sectional force and deformation at node points, as shown in Fig-5.

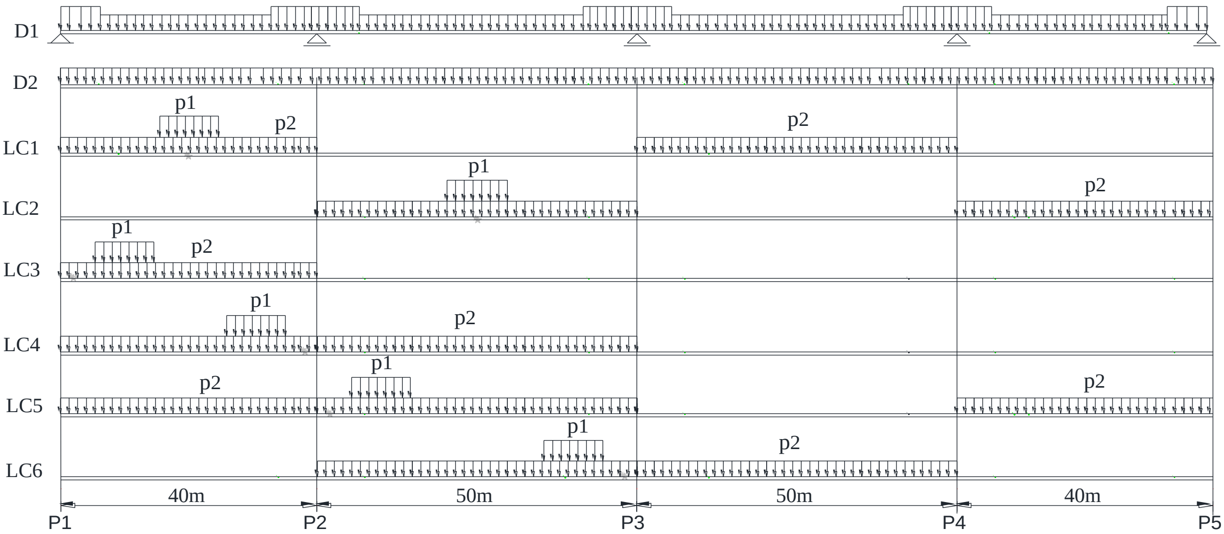

Fig.5 Design load cases

3.2 Sectional force and deformation

Structural analysis was conducted for pre-composite case with the continuous beam model, and for the post-composite cases with the 3D rigid frame model. The bending moment at span center and joints are illustrated in Fig.6: the bending moment diagram of the exterior girder (G1) due to pre-composite dead load (D1), post-composite dead load (D2) and live load (L) cases. The deflection due to live load cases are within the specified limit.

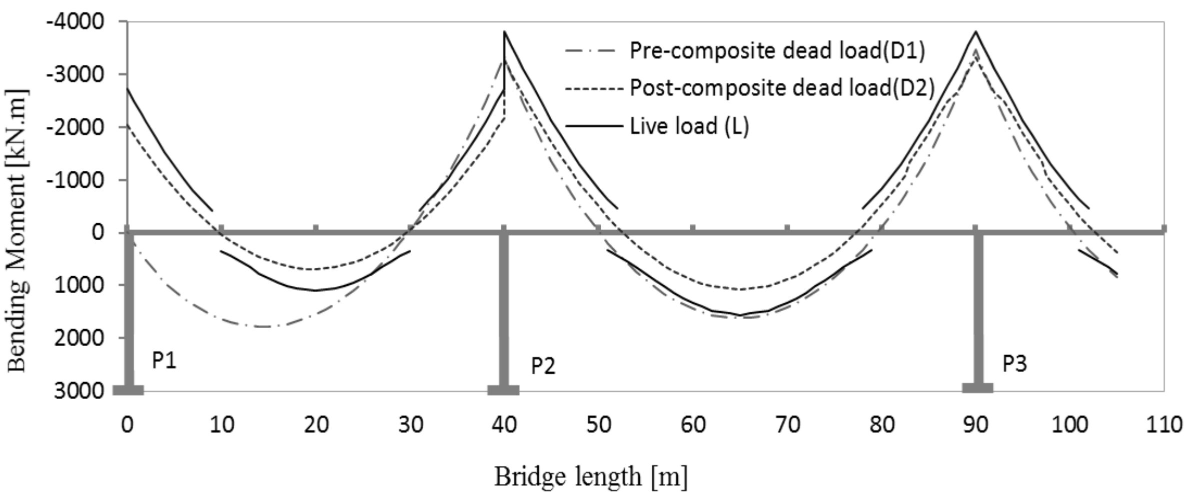

Fig.6 Bending moment diagram

3.3 Verification of Safety

Safety verification was carried out by the limit states design method to ensure load carrying capacity of the composite girders throughout the service life of the structure against possible actions. Verification for prevention of member failure of the composite girder is conducted according to the following equation:

$$\gamma_i \frac{S_d}{R_d} \leq 1.0 \tag{1}$$

where, S_d: design sectional force, R_d: design sectional resistance capacity, and γ_i: structural factor. Load factors and material factors are determined in accordance with Guidelines for performance verification of steel–concrete hybrid structures (Japanese Association of Civil Engineers, 2006). The fiber model is used for flexure behavior (M-ϕ) of composite and SRC sections in order to obtain the bending moment-curvature (M-ϕ) relation. The cross section is divided into small fibers and each fiber conforms to the constitutive law of steel girder, concrete or reinforcing bar. The non-linear uniaxial σ - ε relation of these materials constitute a key element of the model. Fig.7 and Fig.8 show the stress distribution of the sections at positive (span center) and negative (near support joints) bending moment regions respectively.

The design bending moment (Md) was determined by the critical load combinations. The safety of each sections are verified before and after formation of composite action in accordance to Guidelines for performance verification of steel–concrete hybrid structures. In the verification of sections before formation of composite action, the design bending moment due to pre-composite dead load (M_{d1}) is only taken by the plastic bending capacity of the steel H-girder (M_{sud}) while, in the verification of after formation of composite action the design bending moment (M_d) due to, D1+D2+L is resisted by the ultimate bending resisting capacities (M_{ud}) of the composite and SRC sections respectively, at the span center (positive B.M) and the joints (negative B.M).Table 1 and table 2 show that the verification of sections before and after formation of composite action at all anticipated points satisfied the criteria of design specification.

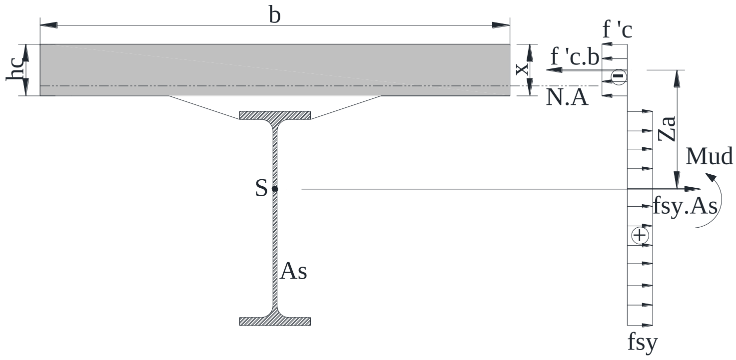

Fig.7 Stress distribution of span center

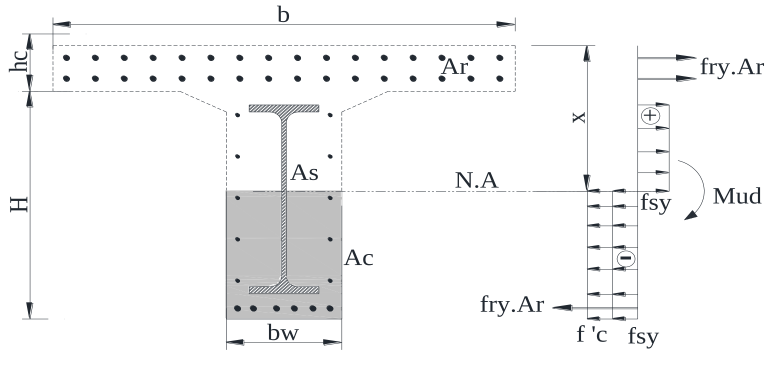

Fig.8**.** Stress distribution of support joint

Table 1 Verification of sections before composite action

anticipated point	*Design B.M* M_{d1} *(kN.m)*	*Resistance B.M* M_{sud} *(kN.m)*	$\Upsilon i\, M_{d1}/M_{sud}$
N1	0	3200	0.00
C1	1950	3200	0.67
N2	-2860	3200	0.98
N3	-2950	4100	0.79
C2	1780	4100	0.48
N4	-3080	4100	0.83

Table 2 Verification of sections after composite action

anticipated point	*Design B.M* M_d *(kN.m)*	*Resistance B.M* M_{ud} *(kN.m)*	$\Upsilon i\, M_d/M_{ud}$
N1	-6430	-11700	0.60
N1'	-2770	-4200	0.73
C1	4710	6000	0.86
N2	-9550	-13500	0.78
N2'	-3710	-4200	0.97
N3	-12780	-15100	0.93
N3'	-4500	-5250	0.94
C2	6140	7360	0.92
N4	-12960	-15100	0.94
N4'	-4530	-5250	0.95

4. EXPERIMENTS OF THE PARTIAL SRC BRIDGE MODEL

4.1 Experimental method

The bending experiment was conducted on partial SRC model to investigate the bending strength and fracture characteristic of the steel-concrete rigid joint, as shown in Fig.9 (Takagi, Nakamura & Muroi, 2003). The objective of this test was to confirm load transfer mechanism from the girder to the pier at the rigid joint. For the design of experimental module, the geometric dimension and material properties of the bridge is considered. The specimen is scale down to half of the actual bridge, using 440mm height of rolled H-section with approximately twice the embedded length of girder. The pier width is 1.0m which, makes half of the girder spacing. For the bearing surface of the steel girder and concrete to perform sufficient bond and prevent shear slip between these two surfaces, perforated bond rib (PBL) were used. As the neutral axis position seems to be away from the girder center of gravity and different sectional force generated in the upper and lower flanges thus, the arrangement of the BPL is used asymmetric in the upper and lower flanges instead of shear connector. Since it has become difficult to arrange the slab rebar in two layers similar to the actual bridge model hence, it has arranged in one layer at the slab center because of the slab thickness. The vertical stiffener is used at the loading point because of occurrence of stress concentration. The incremental load is applied and the vertical and horizontal displacement were recorded for each stages (e.g. cracking, yielding, and maximum) of any structure components. The geometrical dimension of the designed specimen is shown in Fig.10.

Fig.9 Test model

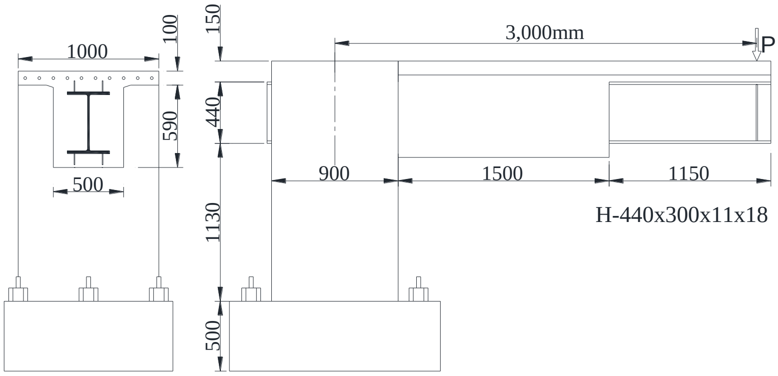

Fig.10 Dimension of test model

4.2 Experimental results

Fig.11 shows the relationship of applied load and vertical displacement of the testing specimen at the loading position and Fig.12 shows the horizontal displacement. The slab concrete cracks appeared at 80kN and the pier concrete started cracks at 131kN with the occurrence of the stiffness reduction and the gradient (slope deflection) changed. After that, the displacement began to increase with the load and it continued up to pier rebar yielding at 390kN. After yielding of pier rebar the rotation of pier increased by forming the plastic hinge, and together horizontal displacement of composite girder and vertical displacement directly under the loading point increased significantly. The load still increased even after pier rebar yielded, proceed the route and had been gradually increased, due to the limitation of the testing machine to end the test, vertical displacement is about 100mm with the corresponding load of 625kN. These test results show that the proposed SRC section has sufficient bending strength and good ductility.

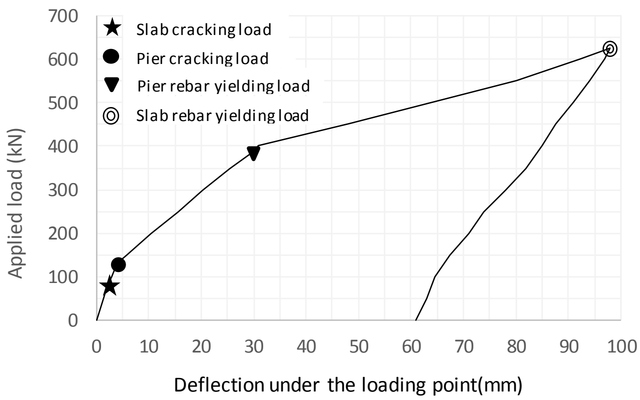

Fig.11 Load vs vertical displacement

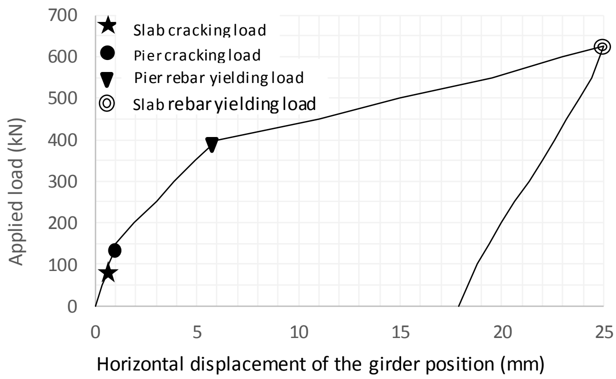

Fig.12 Load vs horizontal displacement

5. CONCLUSIONS

A new SRC Bridge was proposed and a trial designs was carried out, showing that the proposed SRC Bridge with rolled steel H-section can extend the maximum span length up to 50.0 m, which is more than double the existing similar type of bridges. It is also shown that this new bridge is very economical compared with the conventional built-up (welded) girder bridge. Bending tests were also conducted with the rolled steel H-section covered with RC section and RC slab, showing that the proposed structural form had sufficient bending strength and concrete cracks were within the acceptable limit. In conclusion, the proposed composite girder bridge using rolled steel H-section is feasible, economical and attractive.

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