STRUCTURAL ANALYSIS OF TORISAKI RIVER PARK BRIDGE – AN INNOVATIVE PC BRIDGE WITH LARGE ECCENTRIC EXTERNAL TENDONS

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1. INTRODUCTION

The use of external tendons in PC bridges reduces the web-thickness of the girder, thus reducing the self-weight of the structure. Furthermore, the maintenance of PC tendons in such bridges is much easier compared to the conventional types. The above advantages have resulted in the increased use of externally prestressed bridges in Japan, in the recent years. A possible extension of external prestressing is to provide the tendons at large eccentricities, thus maximizing the performance of such bridges. The characteristic of this structure is that the external tendon layout takes the similar shape of the bending moment diagram as shown in **Fig. 1**. In this structure, the external tendons are placed below the girder in midspan region by means of steel struts and at the intermediate support region it is placed above the bridge deck. The tendon at the support region could be arranged within a concrete web member or a short tower similar to an extradosed bridge. The concrete girder bears the compressive forces and the external tendon takes the tensile forces, thus taking advantage of both materials effectively. As such, the girder height is considerably reduced, resulting in lightweight structures that have better performance under earthquake loading. Further, the external tendon layout could be arranged freely to suite the conditions at a particular site, based on the law of linear transformation [1]. It is believed that the structural performance of such structures can be improved, leading to aesthetically pleasing structures.

Considering the above concept, an innovative PC bridge with large eccentric external tendons was developed for a pedestrian bridge in Mori Town of Hokkaido, Japan. This bridge was constructed as a monument structure, crossing the Torisaki River as an approach road to a public park that is being created as part of the town improvement plan. The bridge was designed as a two span continuous girder with unsymmetrical span lengths. The external tendons are subtended by means of four steel struts in the midspan regions. At the center support region, the tendons are placed in a concrete web member in the form of a fin. The combination of fin-back member and subtended tendons makes this bridge a unique one, with aesthetically pleasing appearance. Due to the nature of this structure, several challenges were encountered during the design and construction stages. This paper highlights the FEM analysis that was carried out to check the stresses in some important structural elements of this innovative bridge.

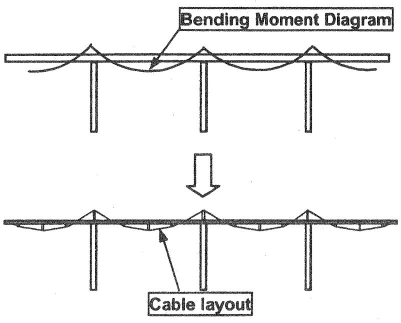
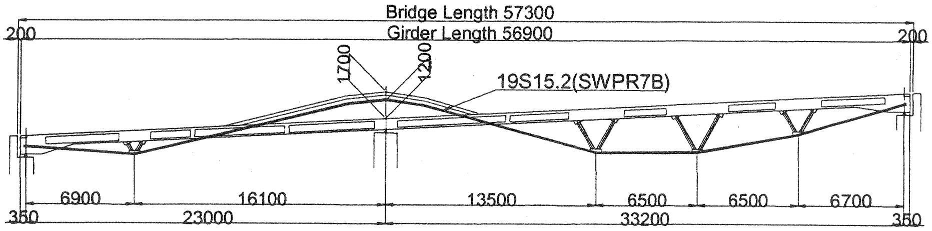
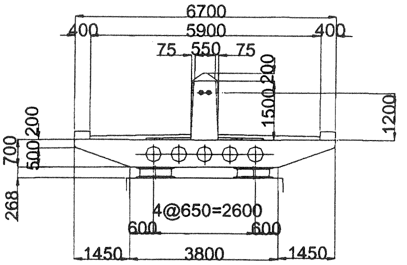


Fig. 1 Bending moments and cable layout in continuous span girder



Longitudinal Elevation



Cross Section

Fig. 2 Layout diagram and dimensions of the bridge

2. OUTLINE OF THE STRUCTURE

Based on the site conditions, the bridge was designed as a two span continuous girder having unsymmetrical span lengths as shown in **Fig. 2**. The external tendons were placed below the girder in the mid-span region by means of steel deviators. The amount of eccentricity of the subtended tendon at mid-span region was determined considering the high-water level expected at site. The eccentricity at center support was then decided by moving the cable layout above the girder by the law of linear transformation. In the center support region, the external tendons are placed above the bridge deck, covered by a concrete web in a fin-shaped manner. This fin-shaped web is to serve as a monument, since this bridge is built as an approach road to a public park. The effective width of the bridge varies from 3.0 m at the abutments to 6.0 m at the central pier, allowing for the fin-shaped web.

3. STRUCTURAL ANALYSIS USING 2D FRAME MODEL

The structural analysis was carried out considering the bridge as a 2D frame structure, using the analytical model shown in **Fig. 3**. The cross-sectional variation along the bridge axis was incorporated by varying the sectional properties of the respective elements. The strut was considered fully fixed with the main girder, while the external tendons were allowed to freely move in the horizontal direction at the deviators. The compressive deformation of the rubber bearings at the supports was taken into account by providing a spring element in the analysis with appropriate spring stiffness. The analysis under service load condition was based on the liner-elastic theory, using the software UC_BRIDGE and FRAME. The ultimate state analysis was carried out using DIANA FEM software, considering the non-linearity of geometry and materials [2].

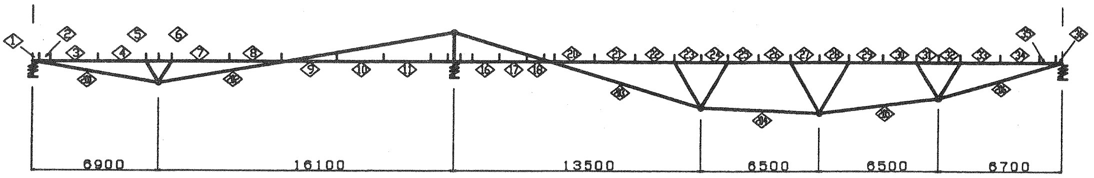


Fig. 3 Analytical model for 2D frame analysis

4. THREE DIMENSIONAL FEM ANALYSIS OF FULL BRIDGE

Considering the complexity of the structure, it was necessary to verify the validity of the assumptions made in the frame analysis. A detailed 3D FEM analysis was carried out to check stresses in the fin-shaped web and the effect of skew in the bridge. The details of this analysis will be discussed in this section.

4.1 FEM Model

The 3-D FEM model used for the analysis is shown in **Fig. 4**. The main girder and 'fin' section consisting of concrete were modeled using 8-node solid elements. The maximum element size was limited to 50 cm, while the average size of the solid elements was about 35 cm. The steel struts were modeled using 2-node beam elements. The strut is considered fixed to the bottom slab, since steel studs were used at this connection to make it rigid. The prestressing tendons were modeled using 2-node rod elements. The model consists of 11,320 elements and 16,401 nodes while the degrees of freedom being 49,005. The FEM modeling and analysis were carried out using the commercial software MSC/NASTRAN for Windows.

4.2 Materials Properties

The materials properties used in the analysis are given in **Table 1**. The Elastic modulus of concrete was selected based on the design compressive strength of concrete assuming sufficient strength is achieved when the bridge is under use.

Table 1 Material properties

	Concrete	Prestressing Cables	Steel Strut
Material used	Design strength = 40MPa	SWPR7B 12S15.2 (Int.) SWPR7B 19S15.2 (Ext.)	STK490 Ext. diameter ϕ 267 mm
Density (γ c)	2.50	7.85	7.98
Elastic Modulus Ec (GPa)	31	200	210
Poisson's Ratio (ν)	0.20	0.30	0.30

4.3 Loading Conditions

Two critical loading cases were considered; a) imposed load applied on both spans giving the maximum support moments (Case-1), and b) imposed load applied only on long span giving the maximum mid-span moment in the long span (Case-2). The value of imposed load was 3.5 kN/m^2. The self-weight of the girder was automatically calculated from the mass of each element. An average surface load of 2.875 kN/m^2 was applied for the surface finishes. The effect of handrail and related loads were incorporated by applying a load of 6.5 kN/m^2 on a strip of 400 mm, along the two edges of the bridge surface.

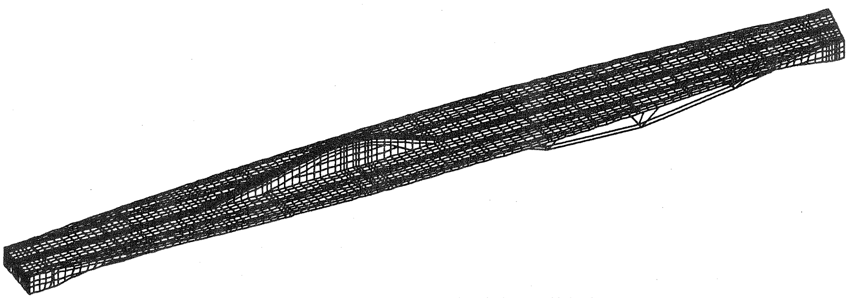


Fig. 4 3D FEM model of the full bridge

4.4 Amount of Prestress

The effective prestress at service load is applied as temperature load in the FEM model using appropriate coefficients. In case of external tendons, a value ranging from 810-950 Mpa was used considering the frictional effects at the deviators that was found to be significant due to large angle of deviation. This also incorporates the effect of bending stresses in the strut member due to differential tendon stress. For internal bonded tendons, the minimum value of 1050 Mpa was applied uniformly throughout the beam, so that the stresses will be estimated on the higher side.

4.5 Analytical Results and Discussion

The maximum and minimum compression stresses in the direction of the bridge axis for the two loading cases are tabulated in **Table 2** for the ‘fin’ shaped web member. It was found that stress levels in the fin portion were under compression for the critical loading cases under service loads. The maximum compressive stress was also within the allowable limits in the web section.

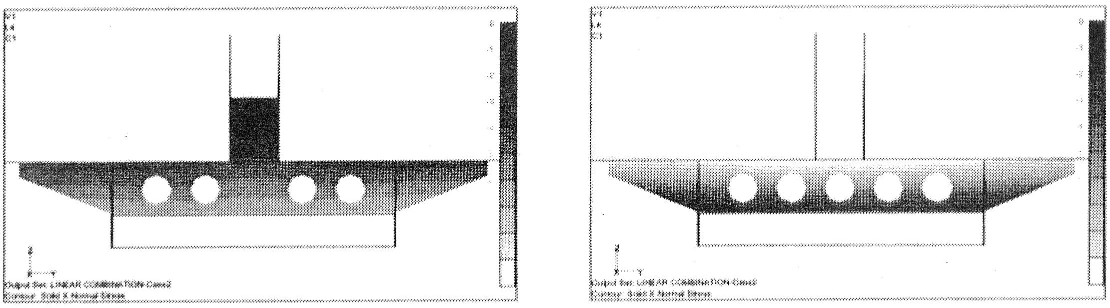
Table 2 Stresses in ‘fin’ shaped member

Load Case	Case-1	Case-2	Allowable
Maximum stress (Mpa)	5.67	6.92	14.0
Minimum stress (MPa)	0.94	0.74	-1.5

In addition to the above analysis, comparison was made between the simplified 2D frame analysis and rigorous 3D FEM analysis. The top and bottom fiber stresses in the critical sections are summarized in **Table 3**. The results agree well at the midspan section, but the discrepancy is more at the center support. This is probably due to the different method of analysis carried out. The contour diagrams at critical cross-sections shown in **Fig. 5** indicate that the variation of stress along the depth of the girder and across the width is fairly uniform. This confirms that the effect of skew present in this bridge is negligible from design point of view. Further, the influence of shear-lag due to the presence of fin-shaped web is also not significant. As such, it can be concluded that for practical design purposes, a simplified 2D frame analysis could be used with reasonably good accuracy.

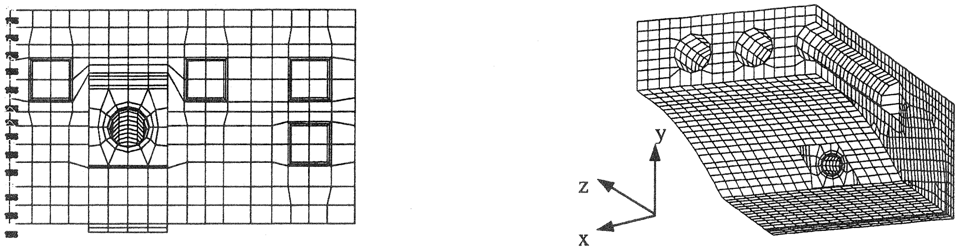
Table 3 Stresses in critical section

Section	FEM Analysis (Mpa)		FRAME Analysis (MPa)		Loading Case
	Top	Bottom	Top	Bottom	
Center support	2.43	4.18	1.63	4.71	Case-1
Mid of long span	10.14	1.42	10.20	1.34	Case-2



(a) Section at fin region　(b) Section at mid of long span

Fig. 5 Stress contours at critical cross-sections of the main girder (for loading Case-2)



(a) End view (b) Isometric view

Fig. 6 Model for end anchorage block

5. FEM ANALYSIS OF END ANCHORAGES

In addition to the analysis of the complete superstructure, it was also necessary to check the local stresses at the end anchorage zone, since there was high stress concentration in this region due to the presence of eight internal tendons and two large capacity external tendons in a relatively small bearing area. The analytical details are discussed in this section.

5.1 FEM Model

A preliminary analysis of a simplified model showed that the effect of external tendon duct is considerably significant. Considering the symmetry, a half model of the anchorage block was developed as shown in **Fig. 6**, with proper modeling of the external tendon duct. The ducts of internal tendons were neglected in the modeling, since its effect was not significant. The concrete block was modeled mainly using 8-node solid elements with some 6-node wedge elements. The steel bearing plates were modeled using 8-node solid elements and considered fixed to the concrete. This model consists of 6807 elements and 8346 nodes. Appropriate boundary conditions were applied to incorporate the symmetry of the structure. As for the material properties, a lower value of 28 GPa was considered for the elastic modulus of concrete, based on the concrete strength at the time of prestressing.

5.2 Loading Conditions

The effect of prestressing force was modeled as externally applied force on the bearing plate. Considering the sequence of prestressing, the total force for internal tendons was taken as the effective force after prestressing, while the maximum expected force during prestressing was considered for the external tendons. The applied forces considered in this analysis are summarized in **Table 4**.

Table 4 Loading due to prestress

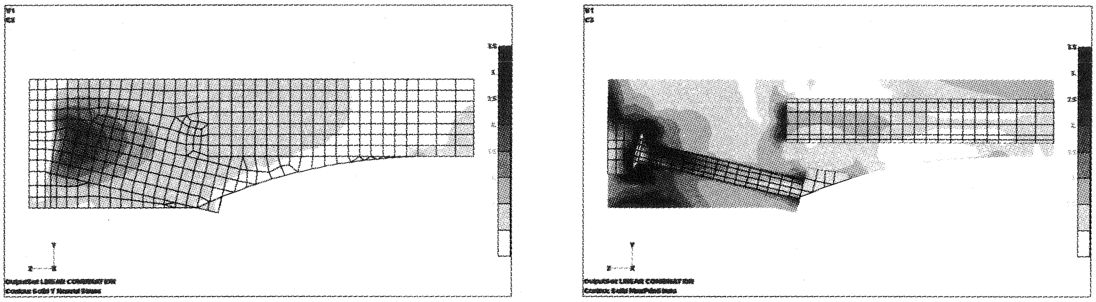
	Force per tendon (kN)	Type of loading
Internal Prestress	2016	Point load at center of plate
External Prestress	2530	Pressure load on a circular ring

5.3 Analytical Results and Discussion

The maximum tensile stresses at critical locations of the anchorage zone for the above combined loading are in **Table 5**. It was found that the maximum stress in the transverse direction of bridge axis (X-direction) was found at the bottom surface. The stress in the upward direction (Y-direction) was on the side face as shown in **Fig. 7a**. Though the values slightly exceeded the allowable tensile stress of concrete, the amount of design reinforcement was found to be sufficient to resist these tensile stresses. Further, high stress concentration was found around the

Table 5 Tensile stress in anchorage zone

	Stress (Mpa)	Location
X – direction	3.12	Bottom face, below the external tendon
Y – direction	2.01	Side face, near mid level
Max. principle stress	3.75	In the vicinity of external tendon duct



(a) Stress in Y direction (Side face)　　(b) Principle stress at cross-section

Fig. 7 Stress contours at the end anchorage zone

duct of the external tendon as shown in **Fig. 7b**. Spiral reinforcements were provided around the ducts to resist this stress. From the 3D analysis of anchorage block, an idea of the stress levels was obtained and it was made sure that the section is sufficiently reinforced to prevent any cracks during and after prestressing.

6. CONCLUDING REMARKS

An innovative bridge with continuous spans having large eccentric external tendons was developed for a pedestrian bridge. Due to the nature of this structure, several challenges were encountered in the analysis and design of this bridge. A 3D FEM analysis was carried out to check the stress levels in the fin-shaped web member and end anchorage zone. It was confirmed that these sections were safe under appropriate loading conditions. Further it was verified that a simplified 2D frame analysis could be used with good accuracy for practical design, with appropriate sectional properties incorporating the varying cross-section. Finally, the authors believe that the proposed bridge would pave way to a wider use of external prestressing technology in the construction industry, leading to improved structural performance as well as cost effective structures.

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