Design of the Second Jin-Do Bridge

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Summary

The 2nd Jin-Do Bridge is 3-span cable stayed bridge with main span 344m and composed of steel girder and steel pylons which crossing over the Ooldolmok-strait located between the Jin-Do Island and the mainland, where nation's fastest tidal flow, 13.2knots, occurs. A design criterion for this project has been studied mainly through the case studies, and the result from the study has been applied for the whole design procedure including the initial state analysis, erection stage analysis and so on. Wind-tunnel test, considering the existing bridge, apart only 22.25m center to center from the new one, has been done and small vanes on both sides of the stiffening girder showed good control over vortex shedding with satisfactory. To reduce the vibration amplitude due to the buffeting phenomenon during the erection stages, tuned-mass-damper has been proposed on the top of the pylon and the tip of the girder during cantilever erection, respectively. The dimple type cable sheath and the lead-core damper were also proposed to control wind vibration of the cable including the rain-wind vibration. The bridge construction has been processed since the end of 2001 with schedule of 5 year construction period.

1. Introduction

The existing Jin-do Bridge which was open to traffic in 1984 has been the only way linking Hae-Nam and Jin-do Island. Design live load, DB-18 (total weight 34tonf) at the time of designing the bridge means so called 2nd grade bridge according to current design category which means the bridge is not strong enough to take the normal live load. After it was decided that the truck weighing over the design load must not pass through the existing bridge from the close investigation including load and vibration test in 1996, civil petition calling for heavy truck passage for agricultural and marine products transportation has been arouse, therefore. Since Jin-do Island was getting more difficulties in water resource, this situation also required future water supply means into the Jin-do Island crossing the strait.

As the result from pre-feasibility study by Government (Ministry of Plan and Budget, 1999) considering the above mentioned situation, it has been decided that new 2-lanes 1st grade bridge to be built first and the future improvement of load carrying capacity of the existing bridge according to the traffic volume trend so that 4-lanes 1st grade bridge is to be operated, finally.

The bidding procedure for 2nd Jindo Bridge followed the Turn-Key bidding process (competition of both design and price). After selection of the contractor through the competition of both design and price based upon the basic design, detail design was carryied out in 2001. The bridge construction has been processed since the end of 2001 with schedule of 5 year construction period.

The 2nd Jin-do Bridge is the bridge linking the Jin-do Island and mainland crossing Ooldolmok (Myungryang) strait where Admiral Soon Shin Lee got the triumph of Myungryang during EemJin War against Japan Navy in 1592 during Chosun dynasty which is recorded in world history of marine war.

Following is the brief introduction of the 2nd Jin-do Bridge.

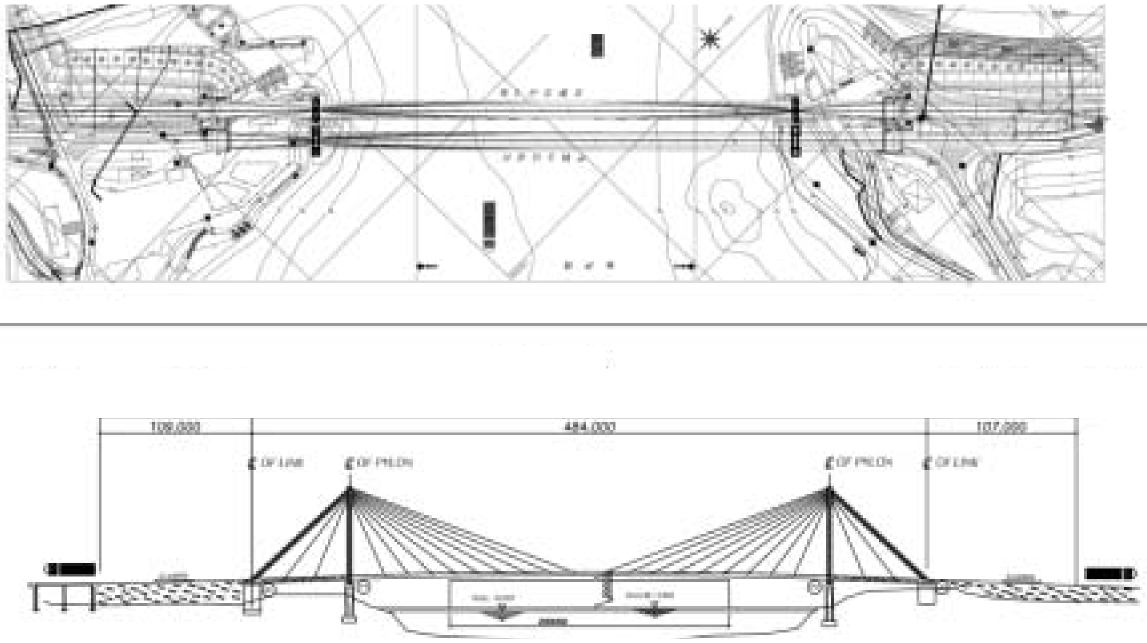


Fig. 1 Plan and Profile of the 2nd Jindo Bridge

- Location: Koon-nae myeon ~ Moon-nae myeon, Jin-do kun, Jeolla-nam province, Korea
- Length: 484m (70+344+70)
- Width: 12.55m (2-lanes)
- Height: EL+89.0m (top of pylon)
- Type and grade: 3 span continuous cable-stayed bridge (1st grade, DB-24/DL-24)
    - Stiffening girder : 1-cell streamlined steel box (orthotropic steel deck)
    - Pylon : A-shaped steel frame
    - Cable : NPWS cable (supplied by New Nippon Steel)
    - Footing : Spread footing on rocks
    - Construction cost : 47.3 billion Korean Won (yr. of 2001)
    - Client : Iksan regional office, Ministry of Transportation and Construction
    - Contractor : Hyundai Construction Company

2. Decision of the bridge location and type

Fig. 2 shows the various condition of the site based upon investigations.

There has been the navigational clearance requirement B x H = 200m x 20m which means long span cable supported bridge is required considering approaching roadway condition. Also tidal speed which is maximum up to 13knots imply that the foundation should be located in land not in the water since foundation in the water under this condition will cause tremendous difficulties in construction as well as can not assure the soundness as such a long span bridge foundation. In addition to the above, the aesthetic harmonization with the existing bridge was also most important since the new one is to be built near by the existing one, the surrounding area is a part of tourism belt zone of southern-west region of Korea and there has been public attention from the cultural and historical background around this area, so that the new bridge becomes a landmark and have symbolic image expression together with the existing one. There was of course important factor in deciding the bridge location such as living circumstances, existing roadway and future plans of the roadway network which also prefer the same solution from the above mentioned requirement.

As the result of the study of bridge location and type, it has been decided that 'twin bridge -parallel, adjacent and same shaped cable stayed bridge' was the best option, in detail no foundation in water, 22.25m center to center from the existing one to assure that there is no serious interruption during foundation construction.

Fig. 2 Site Conditions

3. Structural Components

3.1 Stiffening Girder

The shape of the stiffening girder has close relation with the wind dynamic characteristics and it is even more important especially in the case of two bridges closely located and parallel to each other. It was very important to select the shape of the stiffening girder which is sound not only statically but also wind dynamically, therefore.

Together with the wind dynamic, erection condition was also very important factor in selection of stiffening girder type. Erection condition of Ooldolmock straight, such as tidal speed 13knots, tidal water level difference 3meters and stable tide period 30 minutes, requires in-depth study of erection scheme and at the same time the shape of the stiffening girder which is to be affected by the erection scheme in some sense.

After study, 1-cell stream-lined steel box girder with orthotropic steel deck was considered most effective option because of its structural (statical) effectiveness, wind dynamic stability proven by several other long span bridges in the world, minimization of field fabrication work and good image expression as Twin Bridge.

Roadway arrangement, total 12.55meters, includes 1.5m sidewalk on one side (outer side of two bridges) for pedestrian considering twin bridge arrangement and pedestrian access and 0.75m sidewalk on the other side (inner side of two bridges) for the purpose of inspection and maintenance only of cable anchorage at all time (*Fig. 3*).

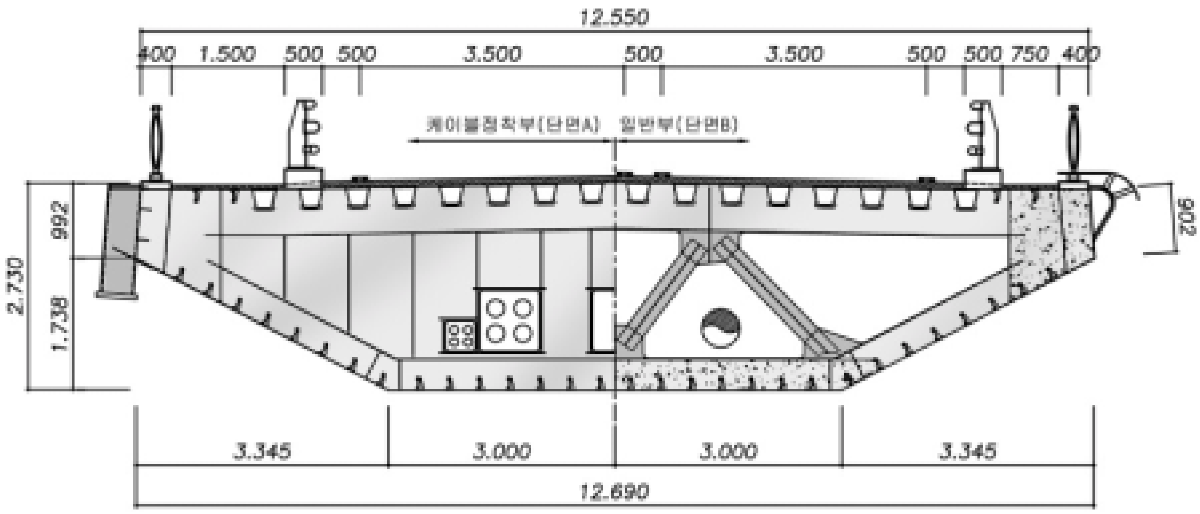


Fig. 3 Section of the Stiffening Girder

3.2 Stiffening Girder – side span

In cable stayed bridges, the more steep angle of backstays or anchor stays, that is, the shorter the side span length, the better control of vertical deflection in main span due to the increase of pylon stiffness. But, this situation inevitably causes more uplift reaction at anchor piers and/or abutments. (The span length ratio of 2nd Jinodo Bridge is ‘side span / main span = 70/344 = 0.2’ in contrast with about 0.4 in usual cable stayed bridges with no additional pier supports in side spans) In general, the uplift reaction which occurs in side span is to be controlled by link support or pull-down system using anchor piers, counter-weight balancing and so on. In case of pull-down system, when uplift force (reaction) increases it requires more space and size inside of the anchor pier or abutment and the substructure get more complicity accordingly. On the other hand, in case of counter weight balancing, due to the dead load increase in side span, the back stay cables could become bigger and heavier causing more cable materials.

In 2nd Jindo Bridge, to accommodate the smaller size of pull-down system and also to have a little increase in the backstay cable materials as possible, the scheme of partially fill-in concrete (concrete fill-in at bottom flange area and both corners of the steel box in side spans, *Fig. 4*) in side spans was applied.

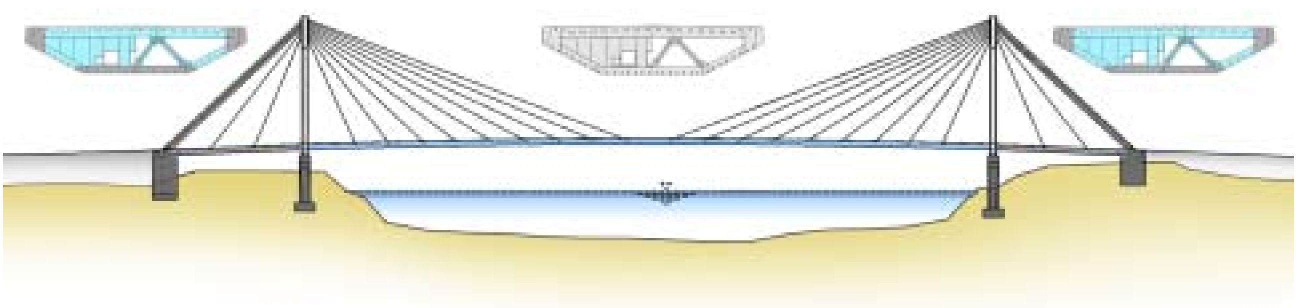


Fig. 4 Stiffening Girder at Side Span and Main Span

Through this application, there has not been any noticeable difference in cable arrangement and shape of the superstructure, and uplift reaction was reduced to 80% in comparison with without counter weight.

3.3 Pylons

3.3.1 Pylon type

To maintain the twin bridge image expression, basic shape of the new pylon followed A-shaped steel frame although in detail there are several differences to express the 'contrast and harmony' through aesthetic detail design. In fact, A-shaped frame need many cautions in design and construction since A-shape might cause construction difficulties of inclined members, lack of accessibilities into inside of the pylon members, complicate arrangement of cable anchorages in 3 dimensional ways and etc, even though its excellent structural characteristics in nature such as transverse and torsion stiffness and buckling strength.

3.3.2 Application of LP (longitudinally profiled) steel plates

In steel plate application for pylons, LP (longitudinally profiled) plates were used (someone says 'tapered plate'), which introduce effective application of steel materials, less number of welding joint at thickness transition locations, shorter fabrication period, cost reduction in fabrication and appearance enhancement at joints.

LP plate, in Korea, have been in market since 1999 though there are still some limitation in production Spec. for example the thickness transition ratio and in case of foreign country it has been used in some cable stayed bridge such as Meiko Nishi bridge in Japan and Rhein bridge in Germany.

The comparison study results between the case of using LP plates and using normal plates in 2nd Jindo Bridge are shown in table 1.

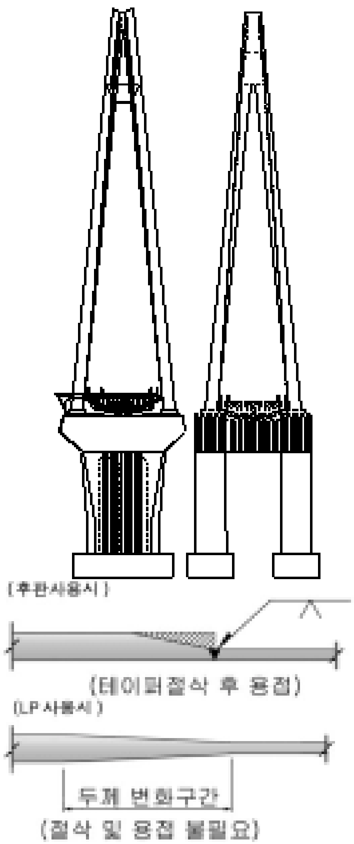


Fig. 5 LP .vs. Normal Plate

Table 1 Comparison Study Result of LP .vs. Normal Plate

Normal plates (welding location : 7)

length(m)	thickness(mm, flange/web))
6.537	30 / 30
7.040	28 / 27
10.057	26 / 24
8.549	29 / 27
6.537	32 / 30
15.086	38 / 36

LP plates (welding locations : **5**)

length(m)	length(mm, flange/web)
3.520	30
8.046	***26~30 / 24~30***
15.086	26 / 24
17.097	***38~26 / 36~24***
10.057	38 / 36

(Bold and Italic type print shows LP plate applications)

As shown in the table 1, by using LP plates steel weight and the number of welding could be saved 8.55% and 30%, respectively. In fact, the effect of LP application is even more enhanced when considering the save in fabrication cost and non-destruction test cost.

3.4 Cable and Cable Anchorage

In cable type selection, followings factors were considered

- arrangement conditions of cable anchorages
- appearance harmony with the cable in existing bridge (Locked Coiled Rope) especially in the size of outer diameter of cable
- trend of using high strength materials
- fatigue strength of cable anchorages
- quality control, delivery condition and field work

Through the investigation of above mentioned factors, NPWS type cable has been selected. (Technical information of NPWS cable should be referred to the supplier)

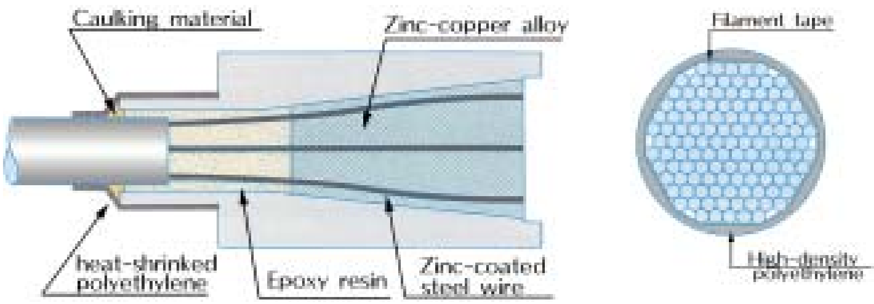


Fig. 6 Anchorage of NPWS Cable and Cable Section

Cable dimensions applied are shown in the *Table 2*. The number of cable dimensions are minimized (grouped together) as possible through the initial state analysis considering erection sequence and maximum tension from the load combinations.

Table 2 Cable Dimensions

No.	Wire dia.×No. of wires	Out dia.	Breaking load	Remarks
1	7mm×151	108mm	10,290N	Anchor Stay or Backstay
2	7mm×139	106mm	9,470N	Forestay
3	7mm×109	95mm	7,420N	Forestay
4	7mm×73	78mm	4,970N	Forestay

Even though, for cable tensioning work, it seems to be desirable to plan that the jacking end is to be at pylon side so that stressing equipments can be used for both backstay and forestay with small efforts, it has been considered to be inappropriate because pylon size, especially at the cable anchorage locations, would be much different from existing one in case of making the pylon size feasible enough to do tensioning work. Accordingly, it is planned that the cable tensioning work is to be done at girder side where working space security and accessibility during cable tension and future inspection are rather easy.

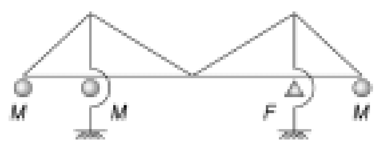
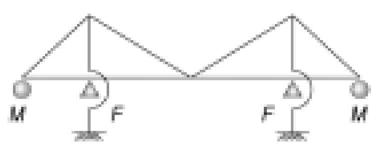
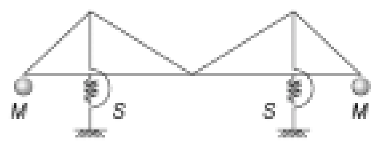
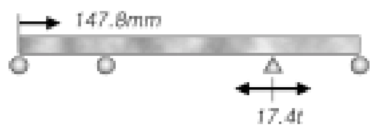
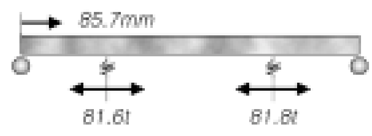
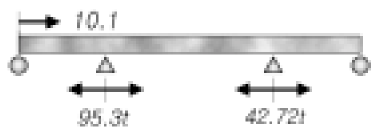
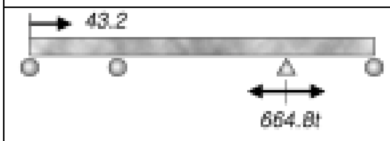
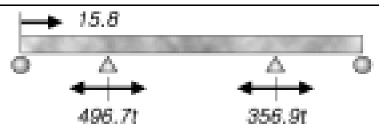
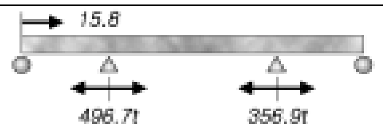
Shear bracket type anchorage was planned in girder side, which introduce minimum eccentricity of cable tension to girder and is profitable in fabrication accuracy control by independent fabrication process of anchorage bracket and also with good harmony with anchorage of the existing bridge which is apart only 10 meters. In case of anchorage in pylon side, considering A-shaped frame with double plane fan-type cable arrangement where cable entering angles are all different in 3 dimensional way, built-up anchorage was applied instead of pre-fabricated anchor-girder type.

3.5 Boundary Conditions and etc.

3.5.1 Boundary Conditions

In dynamic behavior of 2nd Jindo Bridge, because of rather short and massive pier supports, the inertia force concentration at those pier supports is dominant. And, it shows that the pylon mass imposes a burden on the boundary conditions of stiffening girder from the dynamic sensitivity study of pylon stiffness variation, To evaluate the most proper boundary condition system in 2nd Jindo Bridge, various options such as one fixed bearings, two fixed bearings and elastically fixed condition, were investigated. Table 3 shows the result of the investigation in comparative form.

Table 3 Boundary Condition Study

	One Fixed Bearing	Two Fixed Bearing	Elastically Fixed
			
Temp. Load			
Wind Load			
Seismic Load			
			

As shown, since one fixed bearing and two fixed bearing cause excessive loading at pier support / foundation, the increase of the foundation size seemed to be inevitable in compare with the elastically fixed condition. Since pylon foundations of 2nd Jindo Bridge are very closely located nearby sea shoreline, the decision of whether water construction or not is very sensitively dependent upon the size of the foundation planned. Therefore, elastically fixed condition which is certain to have no water construction due to the smaller size by less loading was considered to be most profitable solution.

3.5.2 Countermeasure of Uplift Reaction

There are always uplift reaction, maximum 2200tonf/abutment, regardless of the loading conditions due to the spanning, 2nd Jindo Bridge needs strong support mechanism against the uplift reaction together with anchor stays, therefore. Main considerations to decide the type of the mechanism are as follows;

- Enough stiffness against the uplift reaction existing during the bridge life
- Capacity for the longitudinal expansion (Max. 300mm)
- Capacity for the deflection induced rotational

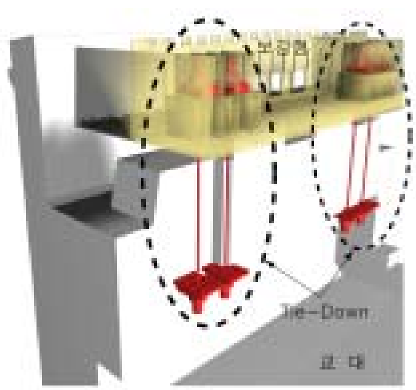


Fig. 7 Pull-down system

deformation
- Durability, easy of maintenance and future replacement

Two options, tie-down cable and link support made of steel structure, were investigated and Pull-down system (tie-down cable + pot bearing) was finally applied, which has features of redundant load path by 4 cables per each abutment, possibility of future replacement and effective construction due to same material as main cables.

4. Wind Tunnel Test

4.1 Introduction

As mentioned earlier, 2nd Jindo Bridge is to be built nearby and in parallel with the existing cable stayed bridge, wind dynamic characteristics of this twin bridge should be cautiously investigated especially in the mutual interaction or disturbance of wind flow, therefore.

From the experiences of other twin bridges, wind dynamic characteristics become more favorable in Meiko Nishi Bridge in Japan or unfavorable in Tsurumi Bridge in Japan depending upon various conditions given.

4.2 Wind Tunnel Test

Wind tunnel test for 2nd Jindo Bridge has been done in Test Laboratory in Hyundai Construction Research Institute, Korea. The tests include 2-dimensional section model test, 3-dimentional pylon model (free standing) test and 3-dimentional full bridge model test for two bridges independently, and 2-dimensional section test, free standing pylon, erection stage and completion stage of full model test for twin bridge situation, also.

In independent 2-dimensional section model test (model scale 1/36), vortex shedding phenomena which occurs in basic section has been disappear with small vane installation. (Vane type wind vibration control measure was considered as first option due to its similarity with that of existing one) For twin situation (*Fig.* 8), the section located at downstream side shows vortex shedding vibration of 40cm amplitude at 30m/sec wind speed without vane. With vane (*Fig.* 9), the vibration amplitude was decrease to 13 cm which is only 1/2600 of main span length. No flutter was occurred up to 64m/sec wind speed which is 1.2times of design wind speed in the bridge site location.

Fig. 9 Section Model Test Set-up

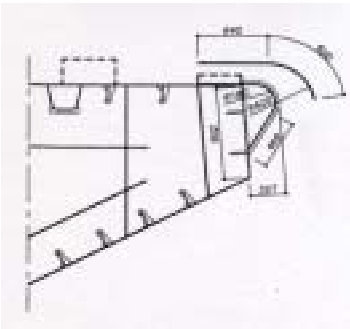


Fig. 8 Vane

In case of free standing tower (model scale 1/70), after the test of independent tower of new bridge to investigate its wind dynamic characteristics, test has been done for twin bridge situation again. From the test, at zero degree of horizontal angle of attack and with turbulent flow 44.6m/sec, buffeting phenomena occurs with 45cm amplitude.

Fig. 10 Free Standing Tower Test Set-up

In case of full bridge test (model scale 1/120), the test result showed that buffeting phenomena occur with 40cm amplitude at wind speed 60m/sec from the completion stage test and 150 cm horizontal displacement (sum of static and dynamic displacement) from erection stage test.

Fig. 12 Full Bridge Test Set-up

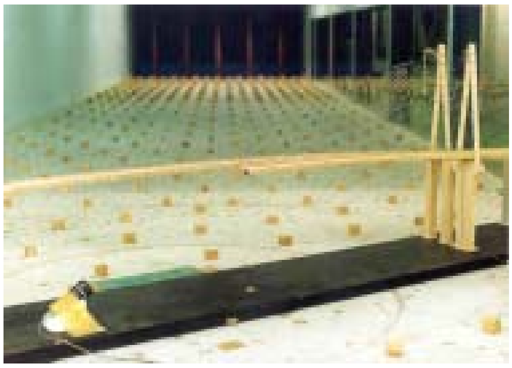


Fig. 11 Erection Stage Test Set-up

4.3 Countermeasure against wind vibrations

4.3.1 Girder and Pylon

As proven from the completion stage test, there seemed no wind vibration problems occur during bridge life when considering the vane installation and turbulent flow in real situation. In erection stage, on the other hand, it should be provided with some countermeasure against the buffeting phenomena during free standing tower situation and cantilever erection situation as described earlier.

To have control over the buffeting vibration during erection period, TMD (tuned mass damper) has been proposed. TMDs of pendulum type are to be installed at the top of pylon and tip of cantilever and are designed to have about 50% decrease of vibration amplitude through adjustment of mass and stroke of pendulum (1.4%~1.5% of the mass of the mentioned structure) At the same time, for pylon, sliding block was also proposed so that the contractor could decide which method to use depending upon the site conditions, during construction.

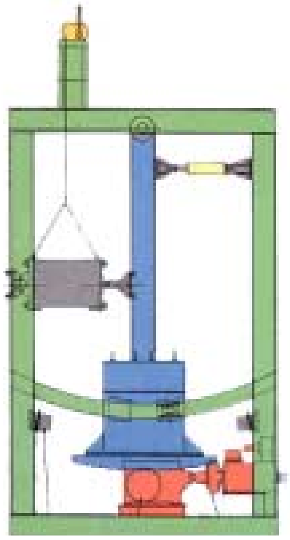


Fig. 13 TMD – pendulum type

4.3.2 Cable

Cable, in its nature, with virtually no bending stiffness, exposed to the ambient condition directly and becoming even more slender according to the development of high strength material needs more attention in providing control measure against wind vibration. Regulations or guidelines for wind vibration control of cable, unfortunately, have not gained official or public approval in engineering practice concern. *Table* 4 shows some guidelines for the wind vibration of cable from text and etc.

Table 4 Guidelines for Wind Vibration of Cable

<table>
<tr><td>Rain-wind Vibration</td><td>$Sc = m\xi / \rho D^2 > 10$</td><td rowspan="3">where, m = Cable mass per unit length
c = shape constant(40 for circle)
ξ = structural damping ($\delta / 2\pi$ = 0.2%)
δ = logarithmic damping ratio
D = Cable diameter
N = natural frequency of cable
ρ = air density
0.22 = Strouhal number
$Sc = m\xi / \rho D^2$ (Scruton number or mass damping parameter</td></tr>
<tr><td>Galloping</td><td>$U_{crit} = cND (m\xi / \rho D^2)^{1/2}$</td></tr>
<tr><td>Vortex induced Vibration</td><td>V = ND/0.22</td></tr>
<tr><td colspan="3">1) PTI Recommendation-Draft(1998) for circular shape with no surface treatment
2) Cable-Stayed Bridges - Past, Present and Future (IABSE Conference Proceedings, Malmo, 1999)</td></tr>
</table>

According to the guidelines shown above, cable vibration was examined. *Table* 5 shows the results.

Table 5 Result from the Cable Vibration Study

items	Study results	measures
Rain-wind Vibration	· S_c = 4.2~6.4 · required structural damping to satisfy S_c > 10: 0.3%~0.5%	Cable surface treatment
Galloping	· C_{urt} = 7.9m/s for maximum and/or minimum cable tension ※ There have been no galloping phenomena in real situation, with U_{crit}=3.62m/s, 7.76m/s in Normandy Bridge and Elorn Bridge respectively from investigation based upon same guidelines	Damper installation
Vortex induced Vibration	· wind speed which might cause vibration is no greater than 1m/s.	(measures given above)

As shown in the *Table* 5, it has been proposed to provide cable surface treatment (*Fig.* 15) and internal damper installation inside of the cable anchorage (*Fig.* 14) considering no clear and recognized guidelines up to now and sample guideline (ξ > 0.56%) from the experience of Soe-Hae Grand Bridge, Korea. But it was also notified that, during construction, whether damper application is necessary or not should be decided after field test of cable intrinsic damping.

Fig. 15 Cable Sheath (HDPE Tube) with Dimple type surface Treatment

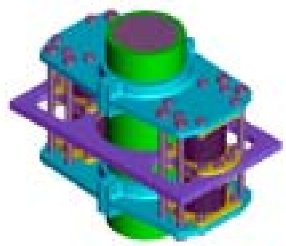


Fig. 14 Lead-Core Damper installed inside of the Cable

5. Structural Analysis

5.1 Design Criteria

Roadway Bridge Design Criteria in Korea is not supposed to be applied for bridges that span length is greater than 200meters, since it could cause excessively conservative design if someone used it without any filtering where it does not give good consideration regarding characteristics of long span cable supported bridge. In design of long span cable supported bridges, it is rather usual to have project specific design criteria through study and investigations as international engineering practice have shown so far.

In recent years, there have been several cable supported bridges designed and/or built or under construction in Korea and also there have been demands to built long span cable supported bridges which follows the plan of linking many islands located in southern west cost (an archipelago) and main land for the balanced development of the country. The development of design criteria or design guideline for the long span cable supported bridges is rather strong requirement in these days, therefore.

Design criteria should be developed based upon experience and study for a number of years, but unavoidably, to be in the project schedule, design criteria for the 2nd Jindo Bridge has been drawn through the case study and design examples in Korea and other international project and it meant if possible to provide basic material for the future study in Korea.

5.2 Analysis

Structural analysis for cable stayed bridge can be categorized as initial state analysis and complete state analysis and the initial state analysis includes the analysis for the erection stages.

The initial state analysis is the process to evaluate initial tension of cables which is in equilibrium with the dead load and satisfy the design profile of the bridge together with favorable member forces and reactions, thus, is described as optimization of the initial tensions. The initial state analysis becomes more practical when cable tension history considering fabrication camber and accumulated displacement following the erection process is investigated in addition to the analysis at completion stage.

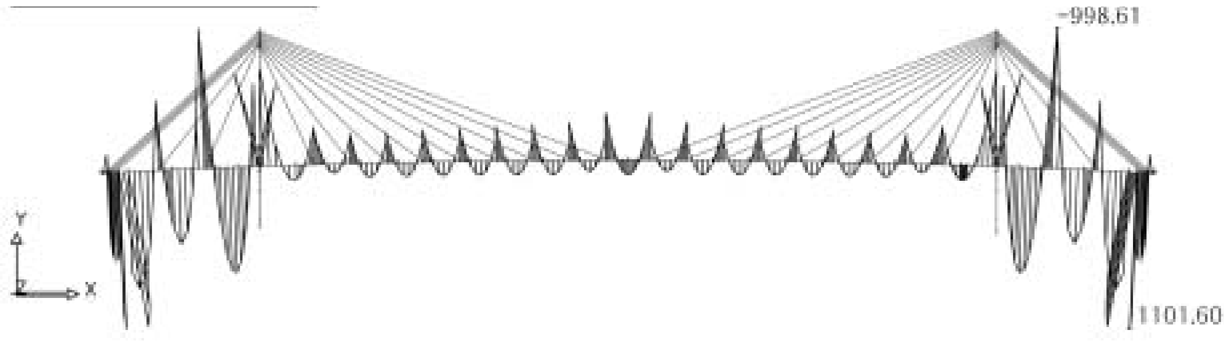
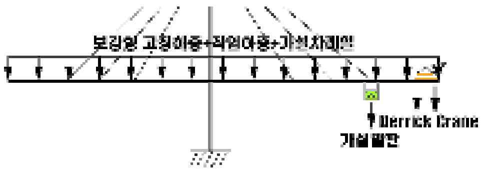


Fig. 16 BMD from Initial State Analysis at Completion Stage

Initial state analysis for the 2nd Jindo Bridge was performed in forward calculation manner provided by software RM-spaceframe based upon the above mentioned condition following erection stages. Erection stage analysis as part of the initial state analysis need inevitably some assumptions for the construction condition, for example weight of the construction equipment, erection sequences and so on. In real construction, the analysis has to be performed again based upon real condition which usually differs from the assumption in design stage slightly so that geometry control can be successfully achieved.

Table 6 show conditions for the erection stage analysis during design stage.

Table 6 Conditions for the Erection Stage Analysis in Design Stage

Dead Load during erection	· Dead Load (Segment Weight) · Derrick Crane Weight - 74tonf · Movable working platform - 10tonf · Crane Rail Weight - 0.5tonf/m · Miscellaneous - 1tonf/m 
Loading Conditions	· Dead Load · Temperature Load: ±15℃ · Wind Load based upon design wind speed v_{10} = 31.5m/s that estimated from the return period corresponding to the erection period (2 years) · Seismic Load based upon the Preservation of Functional Requirement Level (acceleration coeff. = 0.0399)

Erection stage analysis has been performed in forward calculation manner using the conditions given and *Table* 7 shows some samples of BMD, deformation and camber in each erection stages.

Table 7　BMD, Camber and Deflection in erection stages

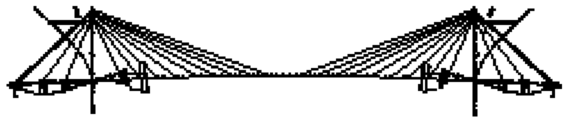
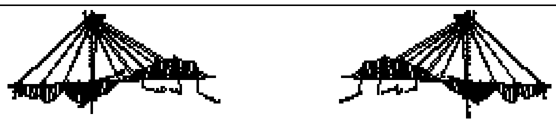
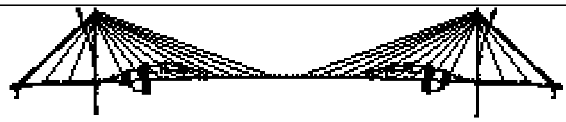
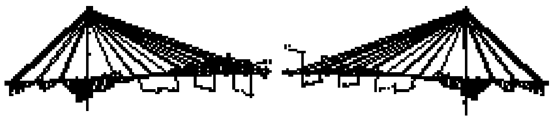
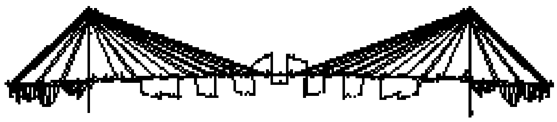
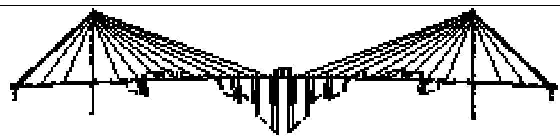
Stage	BMD	Camber and Deflection
10		
20		
30		
Key Seg. Closure		

Fig. 17　2nd Jindo Bridge and Existing Jindo Bridge (computer simulation)