1. INTRODUCTION

The Japan Railway Construction, Transport and Technology Agency (JRTT) is a public corporation exclusively in charge of the construction of SHINKANSEN high speed rail projects in Japan. Among the newly developed SHINKANSEN lines, is the TOHOKU SHINKANSEN extension in the north part of Japan, which opened in December 2010 (Fig.1). Thanks to this extension the TOHOKU SHINKANSEN was completed 28 years after the opening of the first segment between Omiya and Morioka cities in 1982. Due to this extension, the maximum speed on the TOHOKU SHINKANSEN has been increased from the previous limit of 275 km/h or 170 mph to 320 km/h or 200 mph for the Series E5 train and the total travel time between Tokyo and Aomori city, at the north end of Honshu Island, is to be reduced from 4 hours to 3 hours. Subsequently, the Hokkaido SHINKANSEN, which will connect the TOHOKU SHINKANSEN with Hokkaido Island through the Seikan Tunnel, the world’s longest tunnel is under construction and is scheduled to be opened in 2015.
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As a feature of rail infrastructure design, it is important to not only support heavy trains, but also to provide enough rigidity for the structure as needed. Especially in high speed rail, the quantity of deflection must be strictly controlled within the limit of maintaining the safety and stability of the running train. Based on this situation, the Railway Technology Standards have designated a performance-oriented design for railway structures. Because the running safety and comfort are important elements of a railways performance, these standards are to be checked at the same level as a structures’ safety and serviceability.

While the restriction of deformation becomes more severe, the surrounding conditions around the construction site also become severe due to the limited flexibility of the route for high speed train operation, which needs a larger curve radius and a less steep gradient. This makes the span of a bridge longer and designers must consider the two contradicting conditions of less deformation and longer span simultaneously.

This paper presents the required performance characteristics for bridges on high speed rail lines and outlines the issues regarding long-spanned concrete bridges, by presenting the case study of the SANNAI-MARUYAMA Bridge as an example of a successful solution.

2. REQUIRED PERFORMANCE CHARACTERISTICS FOR HIGH SPEED RAIL BRIDGES

In the Railway Technology Standards, the required performance characteristics are sorted into three categories, safety, serviceability, and repairability. The Standards also show the items of performance which are required for satisfaction of the required performance characteristics and also the check indexes for the required items.

Table 1 shows the relation between the required performance characteristics, items of performance, and check indexes for bridges. Rupture, fatigue, and damage are common to all structures including railway bridges and these conditions are checked by common methods. On the other hand, running safety and comfort are unique and important for high speed rail bridges and need further explanation in this paper.

Running safety and comfort should be evaluated by the alteration of the axle load or vibration acceleration and finally by the trial run. However, in the phase of design, deformation or deflection of the bridge are used as the indexes for these checks.

The limit of deformation or deflection is set as the ratio with the span length. The longer the span, the larger the limit for deformation or deflection. In the longest bridges the limit sometimes exceeds 100 mm. Such a large value is doubtful as an acceptable index.

Furthermore, on the long span bridge there might be mutual interaction between the train and the rail track. In such a case it is impossible to calculate the deformation and deflection due to the dynamic effects of the train load by the static analysis.

The Railway Technical Research Institute, Japan has developed the dynamic analysis code for solving these issues. In this system the train and the whole of the structure are modeled by the multiple degree of freedom model and thereby a simulation of a trial run becomes available. This system can
also output the value of wheel-levitation which causes derailment and the train body acceleration which relates to the passengers’ comfort. These values are directly used for the check.

<table>
<thead>
<tr>
<th>Required performance</th>
<th>Items of performance</th>
<th>Check indexes</th>
</tr>
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<tbody>
<tr>
<td>Safety</td>
<td>Rupture</td>
<td>Member force at ultimate limit state</td>
</tr>
<tr>
<td></td>
<td>Fatigue</td>
<td>Variable stress of reinforcement</td>
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<tr>
<td></td>
<td>Running safety</td>
<td>Wheel levitation, Axle load</td>
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<tr>
<td></td>
<td></td>
<td>*Displacement, Deflection</td>
</tr>
<tr>
<td></td>
<td>Public safety</td>
<td>Durability of concrete</td>
</tr>
<tr>
<td>Serviceability</td>
<td>Comfort</td>
<td>Body acceleration</td>
</tr>
<tr>
<td></td>
<td></td>
<td>*Displacement, Deflection</td>
</tr>
<tr>
<td></td>
<td>Appearance</td>
<td>Crack width</td>
</tr>
<tr>
<td>Recoverability</td>
<td>Damage</td>
<td>Member force under over-load</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Member load or deflection under seismic load</td>
</tr>
</tbody>
</table>

*Indirect check index for conventional method

The limit of deformation and deflection for the check of running safety and comfort needs the precondition of the minimization of initial track irregularity which occurs before the passing of the train. Regular maintenance is needed to secure the precondition.

Initial track irregularity is the same as initial bridge irregularity because the slab track is directly connected to the structure. The causes of initial irregularity are temperature (sometimes partially by the sunshine) and shrinkage and creep of concrete itself. These are time- or season-related phenomenon. Although these phenomena have been known for a long time, recent longer spans make the absolute value larger and this results in negative effects to the running safety and comfort.

It is possible to check the running safety and comfort with the condition which is input to the dynamic analysis described above. However, because the deformation of the bridge is a time- or season-related phenomenon, the running safety and comfort are also time- or season-related. This is not preferable for reliability of the revenue from high speed rail operations. Therefore, in the high speed rail, we should make the best effort to reduce time- or season-related deformation and at the same time to control the deformation caused by passing trains.

The limit of initial track irregularity is 7mm in 40m-chord to keep a certain degree of comfort. Therefore, the time- or season-related deformation of bridges needs to be limited within the same value.

3. DESIGN OF SANNAI-MARUYAMA BRIDGE
3.1 General

The SANNAI-MARUYAMA Bridge on the TOHOKU SHINKANSENN extension was constructed.
to cross a broad highway, river, and reservoir near the famous Sannai-maruyama archeological site (Fig.2). It consists of two 75m spans and two 150m spans of extradosed bridges. The 150m span is the longest among the Shinkansen structures in Japan. The specifications of the SANNAI-MARUYAMA Bridge are shown below:

- Type of Structure: Extradosed Bridge with 4 continuous spans
- Type of Foundation: Pneumatic caisson
- Total length: 450m
- Span: 74.18m+150m+150m+74.18m
- Height of Pylon: 17.5m
- Material of stay cable: Epoxy coated strand
- Design load: Axle load 17 tons of Shinkansen rolling stock

Elevation of the bridge and P3 pier and pylon are shown Figure 3.
Due to the construction above the river and road, a pre-stressed concrete (PC) type of superstructure was adopted which can be installed by cantilever erection. But in the case of a longer spanned bridge, internal tendons make the girder too thick and heavy at the pier and this causes the aesthetic and seismic dysfunction for the bridge. A cable-stayed type of bridge with external cables (extradosed) solves this problem. The pylon (see Figure 3) is a double-pylon which is common in railway construction. However, these double pylons are not connected to each other by a cross beam from which snow and ice might fall on the train.

3.2 Methods for controlling initial deformation

On the SANNAI-MARUYAMA Bridge we applied the measures shown below for reducing time- or season-related deformation.

3.2.1 Sliding Elastic Bearing

Figure 4 (a) illustrates thermal deformations of a continuous rigid frame which is a common
type of construction for long spanned bridges in recent years, because of the high seismic performance and low maintenance costs. Expansion and contraction of the girder due to seasonal thermal fluctuation in the rigid frame structure are transferred to the piers. The displacements of the piers generate rotation of the top end of the piers and vertical displacement of the girder which is rigidly connected to the piers. The shrinkage and creep of concrete have the same effect as a reduction of temperature because they reduce the length of the girder.

The installation of sliding elastic bearings on four piers except the center pier (P3), which is the stationary point of this bridge, reduce the transfer of displacements into the piers and girder due to expansion and contraction of the girder caused by seasonal thermal fluctuation (Fig.4 (b)). This measure effectively reduced the vertical displacement of the girder.

Although initial deformations are restricted by sliding elastic bearings, deformations due to the train are better controlled in a rigid frame which fixes the girder and piers firmly. Bearings on P2 and P4 (see Figure 3) are arranged in two lines transversally to the bridge axis due to the weight of the superstructures. This “two-line-alignment” reduces the clear span of the girder, and also reduces deflection due to the train loads. The layout of the bearings on the pier is shown in Figure 5.

3.2.2 Lower Pylon

A cable stayed bridge with shallow deck creates an elegant structural form, however longer
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Stay cables from higher pylons under direct sun rays elongate and cause deflection of the stayed deck. The extradosed bridge with a lower pylon, shorter stay cables, and a deeper girder to secure stiffness against the train loads, reduces such thermal displacement of the cables and girder. Thus, this type of bridge is suitable for high speed railway bridges (Fig.6).

3.2.3 Insulation of stay cable from sunshine

A shorter pylon reduces the displacement of the girder due to thermal elongation of the stay cables. In addition, it was considered that thermal insulation of the stay cable is helpful to control the change in length of the cable due to the temperature rising.

A three-layered cable protection system comprising epoxy coated strands encased in high density polyethylene (HDPE) tube filled with cement grout works well as a thermal insulator. A light gray color for the epoxy coating helps to reduce the heating up of the stay surface (Fig.7). A prefabricated cable system with resin filler, that is increasingly adopted for stay cables in recent years, was not employed due to its insufficient thermal insulation.

The thermal insulation performance of this three layered cable protection system is verified in
an exposure test of the specimen (Fig.8). The temperature rise of the stay cable obtained from measurement of the completed bridge in summer is 6 degrees Celsius higher than the girder and the deflection at mid-span of a 150 m span is only 3mm (ratio to span length =2/100,000) (Fig.9).

3.3 Deformation due to train running

Additionally, because of the long span and the higher number of deflection modes for this statically indeterminate of this bridge, both the track safety (indicated by the axle load) and the ride comfort (indicated by the train’s vibration acceleration), were verified with a dynamic analysis which considered interactions between the train and the structure.
As shown in section 2, there might be mutual interaction between the train and the rail track on the long span bridge. Therefore, for the SANNAI-MARUYAMA Bridge, train running was simulated by a dynamic analysis with consideration of mutual interaction between the train and the bridge after setting the dimensions of the parts whose deformation or deflection satisfies the conventional limit on the basis of a static analysis. Running safety was checked for the required reduction of the axle load and comfort was checked for the resulting train body acceleration. For the initial analysis, the limit is within the axle load reduction ratio of 37% and the resulting body acceleration was 2.0m/s². The reduction of axle load was 9.0% and the body acceleration was 0.44m/s² after final analysis. The simulation using a dynamic analysis was checked visually by the animated image (Fig.10).

3.4 Other techniques

3.4.1 Verification of fatigue resistance of stay cable

The acceptable value of stay cables to resist repeated loading due to the live load is commonly controlled by limiting the maximum tension in the cable to 60% for road bridges. However, for high speed rail the condition is very severe due to the fact that heavy trains are passing frequently. Therefore the bridge was designed with an acceptable value of maximum tension in the stay cable of 40%. The calibration analysis for fatigue failure due to strain alteration accompanying train running with consideration of the effects of secondary bending due to sag in the cable and deformation of the girder, demonstrated that the ratio of strain amplitude/fatigue strength of the PC cable, is 0.58 during a 100 year design life, which was considered to be sufficiently safe.

3.4.2 Economical deviation saddle

The installation of an economical deviation saddle is a new technique which has been used for this bridge.

The conventional deviation saddle on the pylon enables the replacement of the stay cables by a dual tube system. In the case of this bridge, the replacement of stay cables is unnecessary as the limiting stay cable stress level, has been designed to be less than 40% of the tensile strength for the serviceability limit state and a fatigue performance service life of 100 years has been achieved in the design. An innovative simple and economical mono tube system deviation saddle has been newly developed in this project to replace the conventional dual tube system. Mono and dual tube deviation saddle systems are shown in Figure 11. The mono tube deviation system was verified under ultimate...
tension using a full-scale prototype test (Fig. 12)\(^2\).

![Figure 12: Prototype test of mono tube saddle system](image1)

![Figure 11: Mono and dual tube deviation saddles](image2)

FIGURE 11  Mono and dual tube deviation saddles

FIGURE 12  Prototype test of mono tube saddle system

4. Construction

4.1 Balanced cantilever erection

As the bridge spans the highway, river and reservoir, where falsework or scaffolding cannot be built on the ground, a balanced cantilever method is adopted to erect the girder. Figure 13 shows the process of erection. First, each head of the pier is completed. Then two formwork carriers are assembled on both ends to erect girders in balance. Temporary concrete supports are installed because the bearings on P2 and P4 are sliding and rotating. The stay cable is tentatively fixed to the girder which must not to be lifted to avoid creating an unbalanced situation. Figure 14 shows the form-traveller above the Aomori Ring Load.
4.2 Pylon

The main reinforcement at the Pylon is D51(SD490). The main reinforcement was joined by gas pressure welding, which required covering of the whole pylon to protect against the rain and wind. This cover was also convenient for the curing of concrete (Fig.15 (a)). All of the gas pressure joints were inspected by ultrasonic testing. For construction of the deviation saddle, it was divided into 4 prefabricated units and installed by the crane at the site (Fig.15 (b)).
4.3 Stay cable

Stay cables are fixed at the anchorages of the 5th to 15th girder segments and erected from pier 2 and pier 4, and at the anchorages of the 6th to 16th girder segments from pier 3. During the installation at the site, high density polyethylene (HDPE) tubes were assembled into the appropriate length for the cantilever and erected one by one. After the setting of the cover tubes, stay cables were inserted from the inner girder by the pushing machine (Fig.16).

It is important to control the initial tension according to the temperature of the stay cable because the difference of temperatures between the girder and the stay cables alters the level of the girder and the tension in the cables even during construction.

During construction we measured the temperatures of girder and a ‘dummy’ stay cable, which is equipped to control the initial tension.

Both sides of the stay cables were simultaneously tensioned by 4 jacks in order to avoid eccentric strain of the girder. After tensioning, measurement of the deck level was continued until
completion of the track structure.

5. Conclusion

In the design for the SANNAI-MARUYAMA Bridge, it was possible to attain a 150m span, the longest among the Shinkansen line structures, by reducing the time- or season-related deformation which is problematic for the long span concrete bridge construction. Although the accumulation of deflection due to creep of the concrete is not reduced, the creep is uni-directional and the speed of the creep is slow enough to allow maintenance work to be carried out.

The state-of-the-art technologies used in the construction of the SANNAI-MARUYAMA Bridge are proven to work effectively by measuring the long term deformation of both the bridge deck and the passing trains.

We sincerely hope that our experiences and knowledge based on the construction of the SANNAI-MARUYAMA Bridge can be a valuable reference to the future construction of large scale concrete bridges. In conclusion, we would like to report that this bridge was awarded the TANAKA Prize, which is given to the best designed infrastructure by the Japan Society of Civil Engineers.

References

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2) Sakamoto, Hasegawa, Tamai, Hashimoto: Verification of Mono Tube for Saddle of the Bridge, Journal for Prestressed Concrete Symposium, 2006