IABSE YOUNG ENGINEERS COLLOQUIUM IN EAST ASIA

——PROCEEDINGS

Organized by

The Chinese Group of IABSE
The Japanese Group of IABSE
The Korean Group of IABSE
Tongji University
State Key Lab for Disaster Reduction in Civil Engineering

October 24-25, 2018
Shanghai, China

Chinese Group of IABSE

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October 24 - 25, 2018, Shanghai, China.

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Tongji University	Yokohama National University	CableBridge Co., Ltd

Message from the Organizers

Yaojun Ge

Shunichi Nakamura

Ho-Kyung Kim

The Chinese Group, the Japanese Group and the Korean Group of IABSE agreed to hold the IABSE Young Engineers Colloquium in East Asia in April 2018. Many young engineers want to participate in the IABSE congresses and symposiums where they can present their research and technical achievements and exchange information with engineers in other countries. However, IABSE activities are taken place world-wide and it is not easy for them to participate. Therefore, it is desirable to have regional colloquiums for young engineers in East Asia. On the other hand, the IABSE itself needs young engineers to be more actively involved in its activities. The organizer consisting of three national groups thinks that this event is important and useful, and has made great efforts to realize it as soon as possible. It took only six months to prepare and held this Colloquium.

In the technical sessions, a total of 22 young engineers present interesting research papers and technical reports of construction projects. Active discussions are expected between young engineers and senior structural engineers and researchers. They also exchange information on the latest technologies and knowhows with other engineers. Presentations are evaluated and the best speakers are granted the Outstanding Young Engineer Best Paper Award.

The organizer is grateful to Mr. Naeem Ullah Hussain, Director/Global Bridge Leader, Ove Arup & Partners Hong Kong Ltd., who kindly accepted to give a keynote lecture. He is a world-famous bridge engineer and has designed and constructed many long-span bridges including the Stone-cutter Bridge in Hong Kong. Young engineers surely learn a lot from his keynote lecture.

Technical tours are arranged in Shanghai by the Chinese Group. It is carefully planned and seems attractive including Wind Tunnel Lab of SLDRCE, Tongji University Museum, The Bund, and Bridges over Huangpu River through Boat Tour.

The organizer expects young engineers to enjoy and learn at not only technical sessions but also throughout the Colloquium. We also hope that this regional colloquium would stimulate other national groups of IABSE. The organizer thanks Tongji University and State Key Lab for Disaster Reduction in Civil Engineering for their support to the Colloquium. Finally, it is notified that the organizer has agreed to have the 2nd Colloquium at Tokyo in November, 2019.

Yaojun Ge
Shunichi Nakamura
Ho-Kyung Kim

Contents

Keynote Speaker: Naeem Hussain

Profession

Civil Engineer

Current Position

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Arup Fellow
Global Bridge Leader

Joined Arup

1969

Years of Experience

56

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BSc (Eng), The West Pakistan University of Engineering and Technology, 1962

Professorships

Adjunct Professor – Technological and Higher Education Institute, Hong Kong

Awards

The Scottish Engineering Hall of Fame – Inducted 2018

The Royal Academy of Engineering – Prince Philip Gold Medal 2012 for Exceptional Contribution to Engineering

Naeem's education in both engineering and architecture has enabled him to combine the best of both disciplines in winning international bridge design competitions and designing award winning bridges for both road and rail.
Naeem's success in bridge engineering can be illustrated by several award winning projects. Most recently, he was the Project Manager for the detailed design and construction supervision for the Hong Kong Stonecutters Bridge, a cable-stayed bridge with a main span of 1018m, the third longest cable-stay span in the world, which has won several awards including the IStructE Supreme Award 2010.

Naeem is Arup's global bridge leader and has over 56 years experience of structural civil and bridge engineering works around the world.

He was the concept and lead designer for the competition and award-winning Hulme Arch Bridge in Manchester which has been described as the "show-piece for the civil engineering profession" and was awarded the Millennium Product Status fully recognising its innovation, creativity and pioneering qualities within the bridge field.
Another notable project is the 10km combined rail and road Øresund bridge crossing between Denmark and Sweden in which Naeem was a key member of the original design team that won the international design competition for the bridge which has a double deck main cable stayed span of 490m, and approach spans of 140m. The Øresund bridge received the IABSE outstanding Structure Award.
Since February 1998 Naeem has been based in Hong Kong. He was the Engineering Manager for the detailed design of KCRC West Rail Yuen Long Section which comprises 9km of precast segmental post-tensioned viaducts with spans up to 90m, and three elevated stations each 400m long. The KCRC viaducts include innovative noise attenuation systems.
Naeem has also been involved in other major infrastructure projects in Hong Kong including Route 8 Cheung Sha Wan to Tsing Yi; Ground Transportation Centre at Chek Lap Kok Airport; MTRC Tsing Yi Railway Viaduct, Shenzhen Western Corridor and Deep Bay Link bridges, viaducts and cable-stay bridges on the 40km Hong Kong-Zhuhai-Macao Link.
He was the Main Bridge Design Leader for the unique Queensferry Crossing in Scotland which comprises of a 3-tower cable stay bridge with main spans of 650m and crossed cables and which was opened in September 2017.

Technical Session I

Field Investigation of Damaged Bridges in 2016 Kumamoto Earthquake

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Abstract

This paper presents a field investigation of damaged bridges in the 2016 Kumamoto earthquake. The 2016 Kumamoto Earthquake occurred in central Kyushu, Japan, on April 14th with Mw 6.2 followed by the Mw 7.0 main shock on April 16th. These earthquakes were mainly caused by the Futagawa fault and Hinagu fault where surface ruptures extended about 34 km long. The earthquakes killed 120 people and caused significant damage to buildings and infrastructure in Mashiki, Nishihara, and Minamiaso areas along these two faults. One of the important discoveries is the damage of relatively new bridges, designed by the bridge specifications after the 1995 Kobe earthquake, along Tawarayama bypass through Aso Mountains.

Keywords: Kumamoto Earthquake, Bridge Damage, Near-fault Earthquake

1 Introduction

The 2016 Kumamoto Earthquake occurred in central Kyushu, Japan, on April 14th with Mw 6.2 followed by the Mw 7.0 main shock on April 16th. These earthquakes were mainly caused by the Futagawa fault and Hinagu fault where surface ruptures extended about 34 km long. The earthquakes killed 120 people and caused significant damage to buildings and infrastructure in Mashiki, Nishihara, and Minamiaso areas along these two faults. The author investigated the damage of transportation facilities, especially bridges, from May 12 to 13, 2016 [1].

2 Damage of Bridges

2.1 Location of Bridges investigated

Figure 1 presents location of bridges investigated in Kumamoto and Mount-Aso area along with active faults based on the active fault database by AIST [2].

2.2 Kiyamagawa viaduct

Kiyamagawa viaduct is a highway bridge with 32 spans and its columns were retrofitted seismically with reinforced-concrete jacketing, as shown in Figure 2, after 1995 Kobe Earthquake. The superstructure was dislocated from the steel bearings as shown in Figure 3.

Figure 1 Kumamoto Area and Location of Bridges

Figure 2 Seismic Retrofit of Columns (Kiyamagawa Bridge)

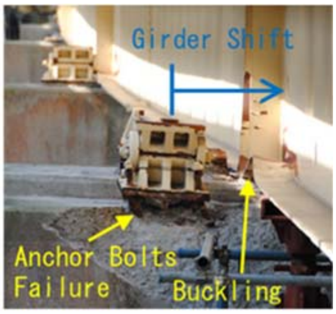

Figure 3 Shift of Superstructure (Kiyamagawa Bridge)

Figure 4 Shift of Superstructure (Okirihata Bridge)

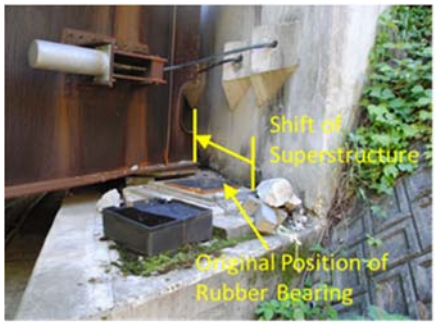

Figure 5 Shift of Superstructure (Okirihata Bridge)

Figure 6 Shift of Superstructure (Tawarayama Bridge)

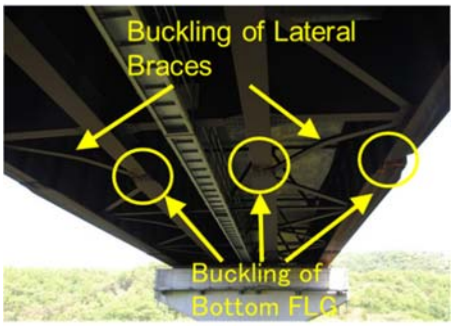

Figure 7 Shift of Superstructure (Tawarayama Bridge)

Figure 8 Shift of Superstructure (Tawarayama Bridge)

2.3 Okirihata Bridge

Okirihata Bridge is a 5 span continuous curved bridge with a skewed angle at A1 side as shown in Figure 4. This bridge was designed in accordance with Post-Kobe Earthquake design code. Figure 4 also shows the shift of superstructure with 0.9m from its original position. This shift came from the rupture of the rubber bearing as presented in Figure 5.

2.4 Tawarayama Bridge

Tawarayama Bridge is a 3 span continuous bridge designed in accordance with Pre-Kobe Earthquake design code. There was a collision between the abutment and superstructure as shown in Figure 6. The bottom flanges and lateral braces were buckled as in Figures 7 and 8.

3 Conclusions

The authors investigated the damage of bridges in Kumamoto and Mount-Aso area from May 12 to 13, 2016. One of the important discoveries is the damage of relatively new bridges, designed by the bridge specifications after the 1995 Kobe earthquake, along Tawarayama bypass through Aso Mountains.

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[2] AIST (National Institute of Advanced Industrial Science and Technology) [2016], Active Fault Database, https://gbank.gsj.jp/activefault/index_e_gmap.html.

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Bridge Engineer, Taisei Corporation, Tokyo, Japan, 1998 – 2003

Structural and Bridge Engineer, ARUP, Los Angeles, USA, 2008– 2016

Associate Professor, Utsunomiya University, Utsunomiya, Japan, 2016 – present

Remarks (Awards, Qualifications, etc.):

Awards

- 1995 Excellent research presentation award at 50th Annual Conference, JSCE
- 1998 Excellent research presentation award at 53th Annual Conference, JSCE

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- Professional Engineer, California, USA, 2009
- 1st Civil Works Execution Managing Engineer, Licensed by the Ministry of Education, Japan, 2003
- Junior Professional Engineer, Licensed by the Ministry of Education, Japan, 2003
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Evaluation of ERW Pipe Structural Performance Considering Manufacturing Process

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Abstract

With the development of the energy industry and the prolonged storage period, more economical and high performance steel pipe design is required. Typical examples are ERW pipes, which have been used as OCTG or recently expanded to offshore pipelines. The ERW steel pipe is produced by passing the initial plate through continuous rollers, and the repetitive loading and unloading during manufacturing process causes plastic deformation and changes the properties of the initial steel. In this study, the manufacturing process of ERW pipes was simulated using the finite element model, and the change of the mechanical properties of the ERW pipe during the manufacturing process was considered in consideration of the Bauschinger effect due to plastic deformation. Also, it was confirmed that the mechanical properties changed due to the manufacturing process lowered the structural performance of the ERW pipe.

Keywords: ERW pipe; Manufacturing process; Collapse pressure; Plastic deformation; Finite element analysis

1 Introduction

ERW pipes are made by passing the initial plate through continuous rollers and are generally used to make small diameter steel pipes. Since the manufacturing process is continuous, the thickness variation of the steel pipe in the circumferential direction is small, and it is more economical and more productive than other kinds of steel pipes such as UOE pipes or seamless pipes (Kyriakides and Corona, 2007). However, repetitive loading and unloading during the manufacture of ERW pipes causes plastic deformation, and it leads change of the yield strength and generation of residual stress due to the straining hardening and the Bauschinger effect. And the change of mechanical properties due to the manufacturing process lead to the deterioration of the structural performance of the steel pipe (Kyriakides and Corona, 2007). Therefore, it is necessary to understand the manufacturing process of the ERW steel pipe accurately and the influence of the manufacturing process on the structural performance.

In this study, finite element analysis is performed by simulating the forming and sizing process using ABAQUS, which cause a large deformation in the circumferential direction of the pipe. Also, the structural performance of the ERW pipes was evaluated by reflecting the forming and sizing results.

2 Change in mechanical properties of steel by manufacturing process

The target diameter and thickness of the ERW pipe are 12.7 in. and 14.7 mm, respectively. The type of steel is API 5L X70 with a yield strength of 566 MPa and it is simulated by the Modified Chaboche model (Zou et al., 2016). As shown in Fig. 1, Forming was simulated using 20 sets of rollers. The shape, position and speed of the rollers were adjusted to satisfy the target diameter and thickness of the steel pipe. And sizing was simulated using 4 sets of rollers and the position of the rollers was adjusted according to the sizing ratio.

Figure 1. Finite element model for forming analysis

In order to maintain continuity of analysis, residual stress, ALPHA, PEEQ and geometrical results after forming were mapped as initial conditions of sizing analysis model.

Table 1. Change of yield strength (MPa)

	Initial Steel	After forming	After sizing
Tensile	566.8	504.9	509.0
Compressive	566.8	585.4	591.0

As shown in table 1, tensile yield strength and compressive yield strength decreased by 10.2% and 4.3%, respectively, due to forming and sizing.

3 Evaluation of collapse performance considering manufacturing process

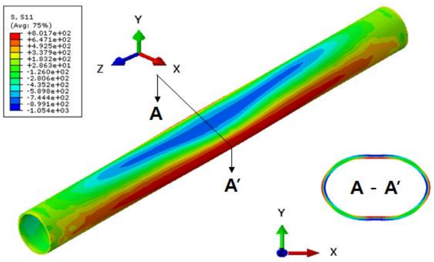

Figure 2. Finite element model for collapse pressure evaluation

The evaluation of collapse performance of the ERW pipe was simulated as shown in Fig. 2. Sizing results such as mechanical property change and geometrical imperfection were mapped to the collapse analysis model. A uniform external pressure is applied to the surface of the pipe with both ends fixed, and the length of the pipe is set to be 10 times the diameter to exclude the effect of both boundary conditions (Bastola et al., 2014). The ultimate pressure of the ERW pipe was evaluated using Modified Riks method, and it was reduced by 32% considering the manufacturing process. The deterioration of the structural performance of the ERW pipe affected the imperfect geometrical factors such as the roundness as well as the residual stress due to the manufacturing process.

4 Conclusions

In this study, two ERW manufacturing processes, which are forming and sizing, are simulated through the finite element model. And the effect of the manufacturing process on the collapse performance of the pipe was evaluated. Tensile and compressive yield strengths decreased by 10.2% and 4.3%, respectively, due to plastic deformation during the manufacturing process. The ultimate strength of the ERW pipe under external pressure was reduced by 32% due to the manufacturing process.

5 Acknowledgement

The first author of this study was supported by the BK21 PLUS research program of the National Research Foundation of Korea. And this research was supported by grants (Project no. 0583-20180036) from the POSCO.

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[3] Zou, T., Li, D., Wu, G., and Peng, Y. (2016). Yield strength development from high strength steel plate to UOE pipe. Materials & Design, 89, 1107-1122.

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1) 2017.06 - 2017.09 : Performance evaluation and enhancement approaches of ERW steel pipes considering manufacturing process
2) 2018.03 - 2018.07 : Analysis for structural performance evaluation of steel pipes considering manufacturing process
3) 2018.06 - 2018.10 : State-of-the-art for structural performance enhancement of steel pipes considering manufacturing process

Other project experience:

1) 2013.07 - 2014.05 : Sheikh Jaber Al-Ahmad Al-Sabah Causeway Project Main Link – Contract RA/140 (Free Standing Pylon Model Tests) (with HUYNDAI Engeneering & Construction)
2) 2014.06 - 2015.11 : Development of design criteria and program for improving performance of highway support structure (with HUYNDAI Engeneering & Construction)
3) 2016.09 - 2016.12 : Seoul-Sejong Expressway Anseong-Guri Construction Project (Section 14) – 3D full aeroelastic model tests (with Korea Expressway Corporation Research Institute)

Remarks (Awards, Qualifications, etc.):

1) Best paper award - Annual Conference of WEIK (2017)
2) Patent(10-2014-0079630): Adjustable Jig System for Two-Dimensional Wind Tunnel

Long term assessment and design of steel bridge – a copula approach

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Abstract

Maintaining deteriorated bridge structures has been a keen technical issue in developed countries and will surely be one in developing countries in the near future. An effective maintenance strategy is highly depending on timely inspections on the bridge health condition. This study is intended to investigate a way of considering temperature and vehicle weight as environmental factors for long-term bridge health monitoring by the use of copula theory. Long-term observed data for a seven-span plate-Gerber bridge is investigated. The autoregressive time series models of observed accelerations taken from the bridge are utilized for the computation of damage index for the bridge. The copula model is used to analyze the relationships between the data collected at different locations. The changes in the modal parameters with the temperature effect are identified by the copula statistical properties. Applicability of the proposed method is also further validated by a theoretical study.

Keywords: long term performance assessment; bridge structures; copula; structural health monitoring.

1 Introduction

Maintenance strategies and repair works are usually required in order to prevent the failure of such important infrastructure. However, developing an effective maintenance strategy is not easy which needs timely evaluations on the bridges health conditions. As most bridges have been functioned for tens of years, certain potential risks are expected due to the structural strength reduction. Therefore, an economical and efficient structural monitoring system for the bridges is highly demanding.

Structural health monitoring of bridges has became of great significance in civil engineering community for quite a few years. Many advanced techniques have been utilized and attempted to evaluate the structural health condition. Park et al. [1] have implemented the laser scanning technology to detect the displacement of the structure. The use of piezoceramic patches in the health monitoring of concrete bridges has been mentioned by Soh et al. [2]. Li et al. [3] have provided an overview of the development in the field of structural health monitoring by using the optical sensors. A key step in the damage identification process is to find anomalies out from the observations. Many works have been devoted to this task based on statistical properties of observed variables. However, most works are focusing on individual statistical proper-ties whilst the dependencies are simply ignored. An alternative way of improving the current identification techniques is to utilize the statistic-al properties of dependencies between the observed data. The canonical correlation analysis is perhaps one of these approaches that have drawn the most attention. However, all these approaches are having a general assumption that the data from different sources are linearly dependent. If the actual dependency is nonlinear, then the applied approach may only offer quite coarser approximations. Unfortunately, the nonlinear dependency is commonly existed between the sensor observations. The characterization of nonlinear dependency among the sensor network remains challenging and requires suitable multivariate models to detect structural damage from the available information.

2 Copula theory

To achieve a more efficient use of multivariate data, the concept of copula could be introduced in the structural health monitoring analysis. Copula is a model which could connect univariate marginal distributions to a multivariate distribution. This copula function can be expressed as

$$C: [0,1]^n \rightarrow [0,1]$$
$$and \quad H(x_1, \dots, x_n) = C\big(F_1(x_1), \dots, F_n(x_n)\big)$$

where $H(\cdot)$ is the cumulative joint distribution function and $F_i(\cdot)$ is the individual cumulative marginal distribution function for the *i*th variable. The structural damage can be detected by a change-point detection algorithm from the dependence structure in the multivariate series. For an *n*-dimensional data series $\boldsymbol{X} = (x_1, \cdots, x_n)^T$, the joint distribution of the multivariate variables at time *t* can be formulated by a copula function

$$H(\boldsymbol{X}_t) = C\big(u_t = (u_{1,t}, \dots, u_{n,t}) \,|\, \theta_t\big)$$

where u_i is the marginal probability vector of x_i, e.g. $u_i = F_i(x_i)$; $C(\cdot)$ is the copula function and ϑ_t stands for the copula parameter at time *t*.

3 Case study – Himeji Bridge

A steel plate-girder bridge with Gerber system which is located in Himeji, a city of Japan, is analyzed in this study. It has a width of 8 meters and length of 142 meters with seven spans. The bridge was constructed in 1960 and was operated since then. A picture of this bridge can be seen from Figure 1.

For a further detection of structural damage sign, the newly proposed copula based indicators are utilized. The indicator values for the whole period are calculated and plotted in Figures 2. In order to see the differences between the intact condition and the condition after the year 2010, the 95-percentile value of the data observed in intact condition is highlighted. For the copula based Z statistics, the critical Z statistic value corresponding to p-value of 0.05 is also highlighted.

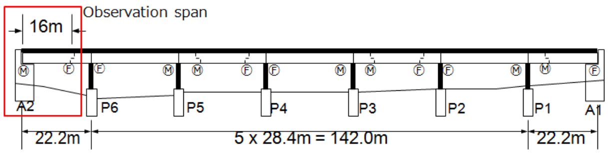

Figure 1. General picture of the bridge

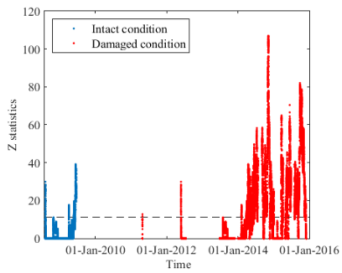

Figure 2. Calculated Z statistics

4 Conclusions

In this study, feasibility of using copula theory in the identification for health monitoring of an old deteriorated bridge structure is investigated through a real case study. This technique is applied to a real bridge located in Japan for detecting structural changes over eight years of time. The case study demonstrated that the changes in dominant frequency and damping constant of the bridge were identified using the measured data. Future work seems necessary to investigate the significance of this approach, including uncertainties such as environmental and operational influences during the long-term monitoring, on the health monitoring and safety assessment of deteriorated structures.

5 References

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[2] Soh, C.K., Tseng, K.K.H., Bhalla, S., Gupta, A. Performance of smart piezoceramic patches in health monitoring of a RC bridge. *Smart Materials & Structures*, 9(4): 533-542, 2000.

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Structural design of the New National Stadium in Japan

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Abstract

The New National Stadium is a facility that is also used as Tokyo Olympic Games and Paralympic Games in 2020. Based on the concept of "Stadium in Forest", the stadium is open to everyone. Blending with the forest of Meiji jingu, the new stadium will form a green network spreading from the inner Garden of Meiji Jing Shrine to the Imperial Palace, and it will become a "New center of sports cluster" where everyone can enjoy taking walks and doing various types of sports. Various unique structural ideas were applied for this stadium, for example the roof frames coordinated with the surrounding environment, the simple structure for the construction period, and the cantilever roof structure with the hybrid members using lumber and steel.

Keywords: simple structure; cantilever roof structure; hybrid member with wood and steel; soft first story response control system; adoption of precast and prefabricated products.

1 Introduction

In this paper, we will report the outline of the structural design which realizes the design concepts and which takes into account the construction sequence.

We proposed the structural design with thorough consideration for workability, especially how to build the roof.

Figure 1. Image of Aerial view from southeast

2 Concepts of structural design

The basic concepts of structural design are:

1) Simple stand structure which places the highest priority on constructability
2) Cantilever roof frame which can be built efficiently
3) Structural frame with wood material to create a very Japanese stadium
4) Response controlled structure by the Soft-First-Story system with high seismic performance.
5) Pursuit of constructability by using unitization and prefabrication.

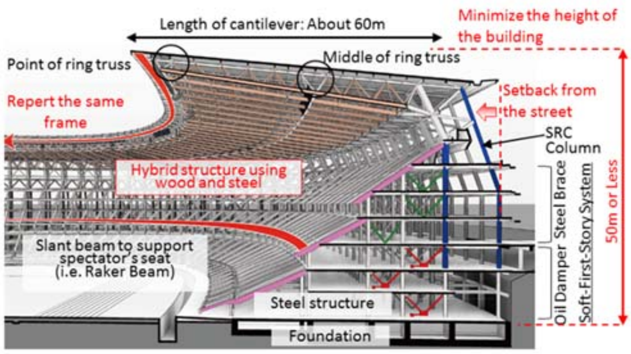

Figure 2. Outline of cross section

The maximum building height is 50m or less, and the low slope roof consists of a cantilever truss structure of about 60m length. The sense of oppression to the neighborhood is reduced through building setback from the street, with the inclined outer columns. The main structure of the stadium is a steel structure above ground. A steel reinforced concrete structure is adopted for the oblique beams (i.e. the raker beam), which supports spectator seats, and for the outer columns, which support the roof truss. Furthermore, the response controlled structure by Soft-First-Story system provides a high seismic performance to this stadium.

The hybrid structure of wood and steel materials for the lower chords (Japanese larch) and the lattice members (Japanese cedar), arranged in the three dimension of roof trusses, creates a space where "Japanese features " can be felt sufficiently. The wood is used as the member which controls the deflection of trusses caused by earthquakes and strong winds.

3 Simple stand structure taking constructability as top priority

By employing the same frame composition repeatedly in the circumferential direction for both roof and stand frame systems, productivity, transportability, efficiency of drawings production, and constructability are improved, which results in the thorough reduction of construction period and cost.

Further, considering the manpower shortage at current construction sites, and the efficiency of construction site work by unitization and promotion of factory making is planned. In particular, the unitization of the roof trusses, the precastization of the foundation and the outer SRC column, and the proactive use of a prefabrication products are planned.

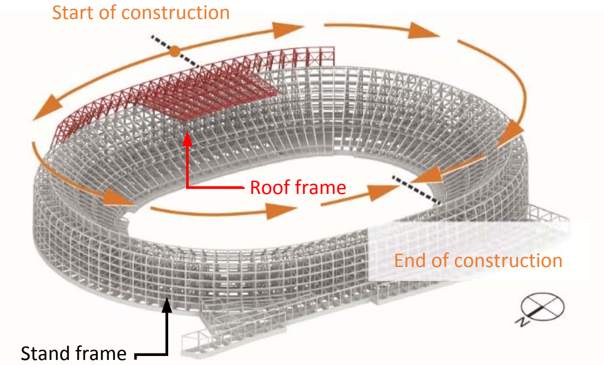

Figure 3. Simple structure plan

4 Cantilever roof frame which can be built efficiently

The roof truss is unitized to achieve efficient construction, and reduce the construction period. By creating a one frame free-standing structure of cantilevering truss, simultaneous construction of the stand, the field, and the roof potion can be achieved.

The roof frame adopts a frame system which has cantilever space trusses with triangular cross sections repeated in the circumference direction. Two upper chords and one lower chord are connected with lattice members in three dimensions. Lattice members contribute to avoid buckling of the upper chords and the lower chord and behave as horizontal braces in the roof surface. The roof is united by joining the web of the upper chords of each unit by high-tension bolts. Upper chord adopts channel steel.

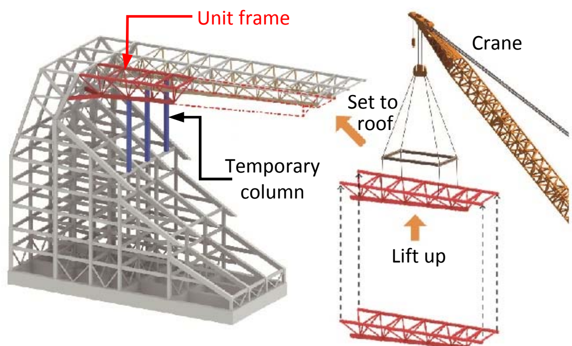

Figure 4. Images of roof construction

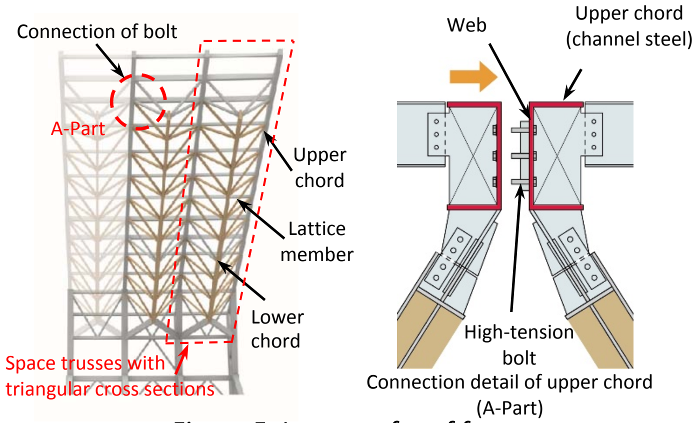

Figure 5. Images of roof frame

5 Conclusions

Structural planning was advanced with a top priority to creating the structural frame which keep the construction period, while achieving the basic concepts of the New National Stadium.

After completion, we hope that this stadium will be a wonderful space where the visitor will enjoy playing and feel Japanese features.

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Construction report of Long span suspension pedestrian bridge in Korea

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Abstract

Pedestrian bridges are becoming increasingly popular in South Korean tourist sites, providing pathway to get close to natural environment. This report will introduce and discuss about the design and construction of long-span suspension pedestrian bridges built in mountains and islands where transportation of construction equipment face difficulties.

Keywords: suspension; pedestrian; bridge; Construction;

1 Introduction

The debate between whether the suspension bridge (tower based) or Tibetan suspension bridge is more suitable solution to be used in mountains and islands, has been a recent topic discussed among the engineers. This paper will introduce the design and construction mechanics of both conventional and Tibetan suspension bridges.

2 Sogumsan Suspension Bridge

2.1 Design and Structure feature

Sogumsan Suspension Bridge is a typical example of Tibetan suspension bridge which has a longitudinal length of 200m, pathway width of 1.5m and whole girder width of 3.1m. Total of 8 cables support the bridge with 6 installed at the bottom of the girders while 2 cables are bound at the top of rail posts.

Figure 1. Sogumsan Suspension Bridge

2.2 Main cable

Main cable is Fully Locked Coil manufactured by Redaelli (Italy), having 40mm diameter with minimum breaking load of 1605kN and Zinc-Aluminium Alloy (Zn95/Al5) coated.

2.3 Girder

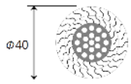
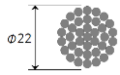
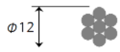
Girder consists rolled steel frame structure, with steel grating foothold to improve aerodynamic performance. Guardrail is made of stainless wire net provided by Hebei Martin Technology (china).

2.4 Construction Method

Initially, the main cables are installed then pre-assembled girders are mounted on the cables. Between the girder and cables, Polytetrafluoro-ethylene (PTFE) plates are placed in order to reduce friction and to prevent scratches on the girder body. By utilising cable way and winch, the girders were pushed to designated positions. After all girders have been connected, guardrails are installed.

Figure 2. Girder draw out

Table 1. Cable Cross-section

No.	Main Cable	Bottom Cable	Suspender
Section			
Type	19.342	1022.6	538.2
M.B.L	17.928	1161.5	580.6

3 Bangchuck-island Suspension Bridge

3.1 Design and Structure feature

Bangchuck-island Suspension Bridge has longitudinal distance of 83m with 1.5m pathway width. Main towers have X-shape and cables are installed to have 3-dimensional mechanical properties.

Figure 3. Bangchuck-island Suspension Bridge

3.2 Cables

Cables are distinguished as main cable, bottom cable and suspender. Fully locked coil is used as the main cables while open spiral rope is used for bottom cables and suspenders. Size of those cables are presented in Table 1.

3.3 Construction

Main and Bottom cables are primarily installed then suspenders are hanged on the main cables. Pre-assembled girders are mounted on the bottom cables, then the girder is dragged by a winch to dedicated positions through cable way. Once the girder is positioned, it is linked with suspenders and previously installed girder.

Figure 4. Girder Installation

4 References

[1] Sogumsan Suspension Bridge Design Report, 2017: 32-43.

[2] Bangchuck-island Suspension Bridge Design Report, 2017: 158-173.

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Design and construction of China-Maldives friendship bridge

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Abstract

China-Maldives friendship bridge which is located in Maldives is a large span sea-cross bridge at open ocean. This project is an exemplary project of one belt one road propose of China. In order to solve the problems caused by deep water, long period surge, coral reef geology and harsh corrosive environment, A series of innovative design and construction technologies used in the project were introduced.

Keywords: large span, bridge, continuous rigidity frame, V-Shaped pier, long period surge, coral reef

1 Introduction

The China-Maldives Friendship Bridge is located on Maldives North Male Reef, crossing the Gaadhoo Koa Strait and connecting Male Island, Airport Island and Hulu Male Island on the atoll. It is the most important islands connection project in Maldives and the first sea-cross bridge. The project is composed of the main bridge, the approach bridge, approach embankment with full length 2km.

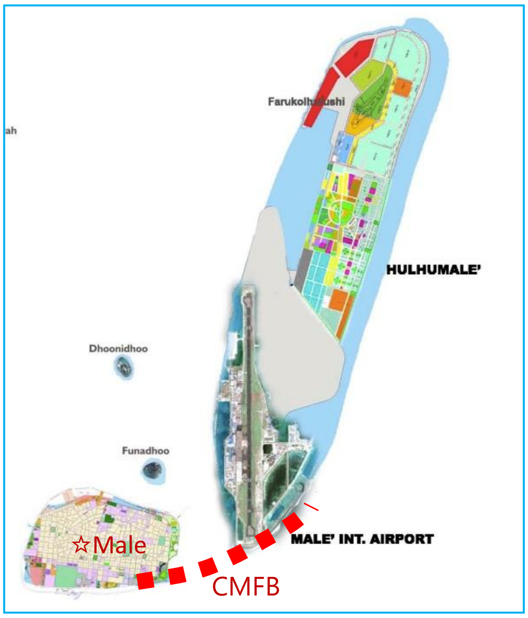

Figure 1. Location of CMFB

2 Engineering challenges

Since the bridge is a large span sea-cross bridge at open ocean, the hydrological condition is very challenging. The maximum depth of water at the piers of main bridge is up to 46m. the local swell period is 14–20s. The project located waters features low wind, high waves, and long wave period, and the project is mainly affected by the mixture of waves and swells. Working at sea must resist the strong wave especially monsoon season.

The geological conditions at the bridge site are typical coral reef geology. It has the characteristics of low density, large porosity, strong structural property, brittleness and strength anisotropy. The mechanical behavior is inhomogeneous and lacks engineering experience for reference.

Since the bridge is located in the low latitude area of India ocean, high temperature, high humidity and high salinity at the bridge location make the durability of the bridge face severe challenges.

3 Design

The main bridge is V-shaped pier six-span continuous rigid frame bridge, with largest span 180m and total length 760m. Both side approach bridge are I beams with span 30m. The span arrangement is shown in figure 2.

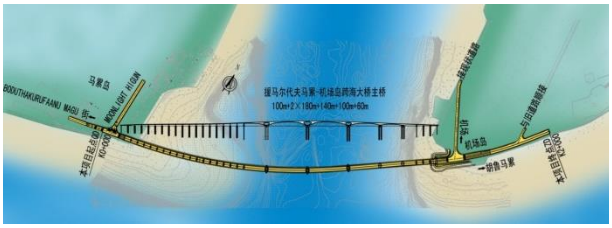

Figure 2. General arrangement of CMFB

Considering the geological characteristics of coral reefs, bored pile is used in the substructure of both main bridge and approach bridge. Large diameter steel-concrete composite piles and grouting at pile bottom for improving bearing capacity of pile tip are used in all the piles of this project. And quincunx layout of piles and shuttle-shaped pile cap is adopted to reduce the load caused by waves and flow.

The superstructure of the main bridge is hybrid girder combined by concrete girder and composite girder to reduce the dead load effect. UHPC-steel composite girder is used for the middle-span segment with various length from 22m to 50m. the superstructure box girder is the box section with single box and double-cell.

To ensure durability of the bridge, an anticorrosive system including marine concrete, epoxy steel bar, high nickel steel, anticorrosive coating for concrete and steel etc. is adopted in the project.

4 Construction of main bridge

4.1 steel pile construction

Because of steeply inclined seabed at two side pier of main bridge, fixed platforms for substructure construction were used in pile construction instead of self-jacking up platform. For resist the affection from long period surge at bridge site, two 1000t-grade Floating cranes was transported from China to Maldives for driving of steel tubular piles of fixed platforms. High rigidity guide frame with fusing position key was designed for pile run which occur frequently in procedure of pile driving. Hydraulic jumper hammer was used for steel pile sedimentation.

4.2 Construction of bored pile

Abrasion drilling was used in the large diameter bored pile. Due to uneven distribution of reef limestone and coral sand, drilling parameters need to be adjusted continuously during drilling. These key parameters were obtained through a series of process tests. In order to ensure the stability of the pile hole, environmental protection mud is also adopted.

4.3 Construction of superstructure

Cable stayed bracket was used in the construction of V-shaped pier. Hanging basket with length 8m was used in cantilever construction of prestressed concrete box girder.

To reduce the quantity of hoisting work from the sea, the steel box girder was fabricated in the factory in segments with length about 12m, and lifted up to the deck of PC girder at the pier top area by floating crane. An air pressure buffer system was used to avoid damage to the steel box girder in the hoisting procedure.

Then segments were welded in a integrated big block, and push forward to the closure using a sliding rail pushing system.

5 Conclusion

Construction of CMFB was start at 20 March 2016, and completed at 30 August 2018, in 29 months. A series of innovative design ideas and construction technologies make this project successful finally.

Figure 3. Completed bridge

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Structural characteristics of multi-span cable-stayed bridges with hybrid towers

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Abstract

Although multi-span cable stayed bridge is a new and elegant structure, its structural characteristics are not fully understood. The steel/concrete hybrid tower is proposed for multi-span cable stayed bridges. Then, its static and seismic behaviors are compared with the RC and the steel tower. The steel/concrete hybrid tower is a new structure consisting of a sandwich type double steel box section filled with concrete. The RC tower has a rectangular hollow section and the steel tower has a steel box section. First, static analysis is conducted with different live load patterns. Deformation and bending moment are studied. Second, elastic and plastic analysis using fiber elements is conducted for the medium strong and ultra-strong earthquakes according to Japanese Seismic Codes for Highway Bridges. Time history of deformation and sectional forces are obtained and compared. In summary, the hybrid tower showed good static features and energy dissipating behavior during earthquake. Steel tower had larger displacement and the RC tower had larger bending moments.

Keywords: Multi-span cable-stayed bridge; steel tower; RC tower; hybrid tower; seismic analysis.

1 Introduction

Multi-span cable stayed bridges are new structures and the Millau Bridge constructed in 2004 is a good example. Its structural form is complicated and the static and seismic characteristics are not fully clarified. Towers play an important role, in particular, for seismic behaviors. This study is conducted to clarify how three types of tower affect the seismic behavior of a multi-span cable stayed bridge. The steel/concrete hybrid tower, consisting of a sandwich type double steel box section filled with concrete, is proposed for this bridge, and compared with the RC tower and the steel tower.

2 Static analysis

The multi-span cable stayed bridge with 8 spans (100+6@200+100) and 7 towers is studied. The girder is a steel box girder with orthotropic deck with a width of 18.8m and a web height of 2.2 m. Cross-sections of steel, hybrid and RC tower are shown in Figure 1. The tower has an H-shape with two cable planes. Static analysis was carried out for the dead load (D) and the design live loads (L), and sectional forces and deformations were obtained. Two live load cases were considered: LC1 is the live load distributed in full spans and LC2 is that distributed in the alternate spans.

The displacement and bending moment of towers are minimized at the dead load stage, by incorporating optimum sets of cable pre-stress forces and counter weights at the side spans. Following this principle deformations and bending moments induced by self-weight of structure is kept to a minimum level. Figure 2 shows the deformed state of bridge due to D+LC2. The RC tower has smaller displacement at the tower top than the steel and the hybrid tower. Whereas, bending moment of the RC tower is the larger than others at P4 (Figure 3).

3 Seismic analysis

Seismic analysis was conducted for the ultra-strong earthquake wave, Level-2 earthquake (L2-EQ). Figure.3 illustrates the longitudinal displacement at the top of tower P4 due to L2-EQ for three types of tower. Displacement of RC tower is 567mm which is smaller than those of hybrid and steel towers with 843mm. And the bending moment at the base of tower P4. Whereas, bending moment of RC tower is 286MN.m, more than three times of steel tower with 79MN.m and more than twice of hybrid tower with 113MN.m.

4 Conclusions

In the process of designing a multi-span cable stayed bridge the choice of tower is important. Three types of tower are proved feasible for a multi-span cable stayed bridge from static and seismic aspects. In static analysis, RC tower showed triple less displacement and several times larger bending moment compared with other towers. Steel tower had the largest displacement but the least bending moment. Seismic behaviors show that; bending moment of RC tower is larger than those other towers when L1 or L2 hits the model. The seismic performance of hybrid is in between RC and steel tower but nearly similar to that of steel tower.

In summary, the proposed hybrid tower showed very good static features and energy dissipating behavior during earthquake.

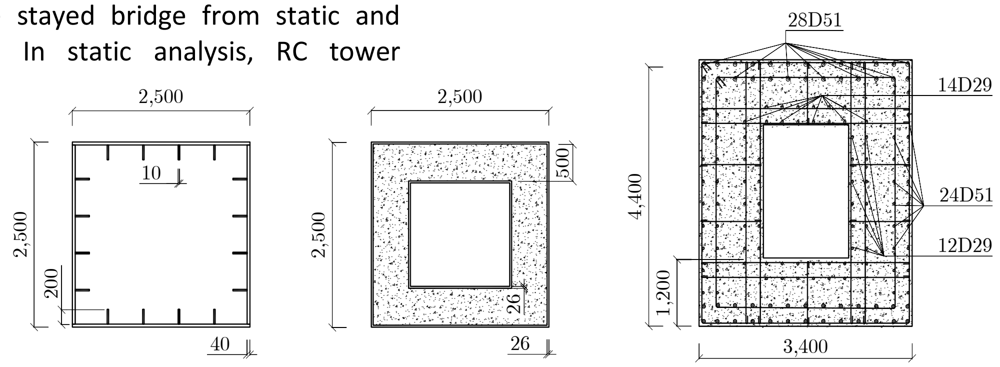

Figure 1. Cross-sections of towers (mm)

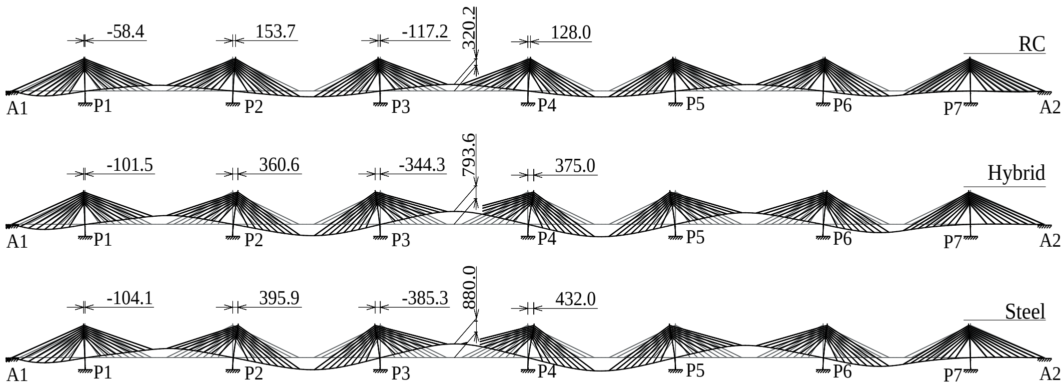

Figure 2. Displacement due to D+LC2

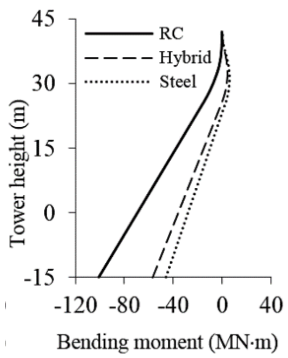

Figure 3. Bending moment due to D+LC2

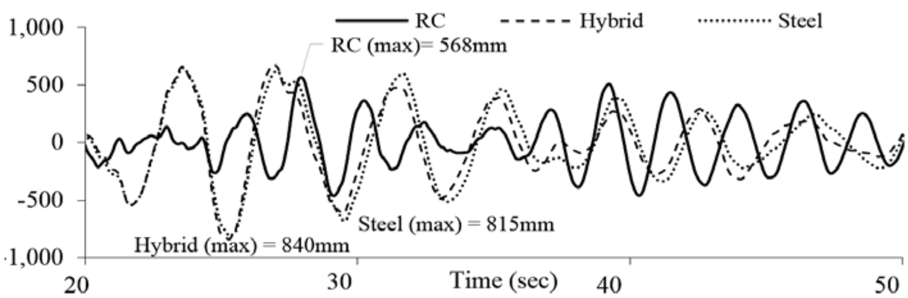

Figure 4. Longitudinal displacement at the top of tower P4 due to L2-EQ

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Control of Vortex-induced Vibration of Long-span Bridges by using the Tuned Mass Damper Inerter (TMDI)

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Abstract

A novel inerter-based system, namely the tuned mass damper inerter (TMDI), is proposed to control the VIV of the main deck of long-span bridges. An applicable layout of the TMDI inside the bridge deck is introduced, and the governing equations of the structure-TMDI system subjected to VIV are established. The optimization of the TMDI parameters with the consideration of nonlinear aeroelastic effect is derived. The control performance and robustness of the proposed system are investigated through an analytical case study in both the time and frequency domains. It is observed that the TMDI system can obviously reduce the VIV responses of the bridge deck. Moreover, compared to the conventional TMD system, the static stretching of the spring due to gravity and the oscillation amplitude of the mass block in the TMDI system are significantly reduced. These properties make the proposed TMDI system an attractive alternative for the VIV control of long-span bridges.

Keywords: Long-span bridges; vortex-induced vibration; control; TMDI; TMD

1 Introduction

With the ever growth of span length, bridge structures are becoming more flexible and therefore more susceptible to the wind effects. Vortex-induced vibration (VIV) may seriously influence the fatigue life and serviceability of bridge structures. It is therefore by all means necessary to take countermeasures to suppress the adverse VIV responses of bridge structures.

Among the VIV control countermeasures, the tuned mass damper (TMD) is most widely adopted (e.g. [1, 2]) due to its simplicity and effectiveness. However, the application of TMDs in the bridge structures also faces some practical challenges. The first challenge is the large static stretching of the spring due to gravity. The second challenge is the large stroke of the TMD mass block. These make it difficult to apply when the TMDs are to be installed inside a box girder.

This paper proposes using TMDI to control the VIV of long-span bridges. An applicable layout of the TMDI inside the bridge deck is introduced and the optimal TMDI parameters are derived. The influence of the nonlinear aeroelastic effect is considered in the optimization of the TMDI parameters. The control performance and robustness of the proposed system are investigated through an analytical case study in both the time and frequency domains.

2 Layout of the TMDI system

For the bridge girder as shown in Fig. 1, vortex shedding will result in vibrations in the vertical direction of the bridge girder. A conventional TMD is installed within the box cell of the main girder. One end of the inerter (with an inertance of b) is connected to the TMD and the other end is connected to the box cell. The inerter is a device that can generate an apparent mass that is much larger than its physical mass through transforming the linear motion into the high-speed rotational motion of the system.

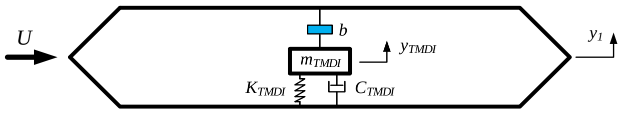

Figure 1. Bridge decks incorporating with a TMDI

3 Optimization of the TMDI System

To achieve the best control performance of the system, the TMDI parameters should be optimized. In this study, the mean square motion of the main deck is of interest. Thus, the optimization of the TMDI system is to obtain the minimum value of the following performance index:

$$J = \sigma_{\eta_i}^2 = \int_{-\infty}^{\infty} S_{\eta_i}(f)df \qquad (1)$$

in which, $S_{\eta_i}(f)$ is the response spectrum of the main deck.

4 Effectiveness of the TMDI

Fig. 2 shows the dimensionless bridge deck response, i.e. the RMS of the deck responses with respect to the deck height. It can be seen that at the most unfavorable wind velocity, the dimensionless response of the bridge deck reduces significantly from 0.085 to around 0.0025 when TMDI is installed, with a reduction ratio of 97%. These results clearly demonstrate the effectiveness of applying TMDI for bridge deck VIV control.

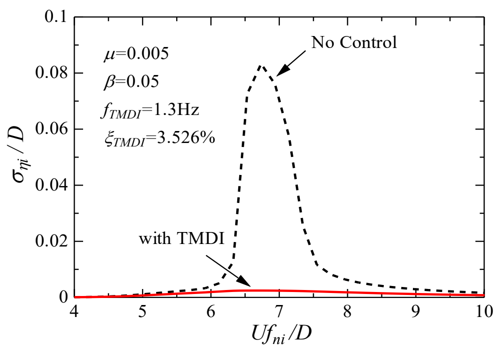

Figure 2. RMS of the main deck responses

5 Comparison with TMD

Fig. 3 shows the optimal frequency of TMDI system. It is seen that the optimal frequency increases with the increment of the inertance ratio. In this sense, the involving of the inerter device tends to amplify the optimal frequency as compared to the conventional TMD system, which is always around the structural natural frequency to make the TMD resonate with the primary structure. For this case study, the static deformation of the spring is 1.64m and 0.148m respectively if TMD and TMDI systems are installed. This property is actually beneficial for the design as discussed above, since the larger frequency can result in a smaller static deformation of the spring in the TMD/TMDI system.

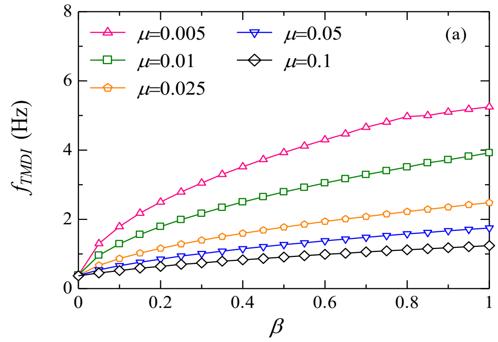

Figure 3. Optimal frequency of TMDI

6 Conclusions

This paper focuses on using TMDI to control the VIV of the main deck of long-span bridges. The results indicate that the TMDI system can significantly reduce the VIV of the main deck. Compared to the TMD system, the TMDI system can significantly reduce the static deformation and oscillation of the mass block in the system. These properties make TMDI an attractive alternative for the VIV control of long-span bridges, especially for large-span bridges with box girders, where inside dimension of the deck is limited.

7 References

[1] Fujino Y, Yoshida Y. Wind-Induced Vibration and Control of Trans-Tokyo Bay Crossing Bridge. *Journal of Structural Engineering*. 2002;128:1012-25.

[2] Battista RC, Pfeil MS. Reduction of vortex-induced oscillations of Rio–Niterói bridge by dynamic control devices. *Journal of Wind Engineering & Industrial Aerodynamics*. 2000;84:273-88.

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Technical Session Ⅱ

Seismic Retrofitting of Rectangular Bridge Piers Using Ultra-High Performance Fiber Reinforced Concrete Jacket

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Abstract

An innovative seismic retrofitting method for existing “as-built” reinforced concrete (RC) bridge piers was proposed, by using ultra-high performance fiber reinforced concrete (UHPFRC) jacket. One “as-built” and two retrofitted rectangular cross-section RC piers, subject to combined axial loading and lateral cyclic loading, were fabricated and tested to investigate the enhancement of UHPFRC jacket on seismic behaviors. Damage evolutions, hysteretic loops, skeleton curves, strength and stiffness degradations, ductility and energy dissipation (ED) capacity were derived and analyzed. Both two different height UHPFRC jackets boosted overall integrity and resilience of the “as-built” pier, by eliminating concrete damage and decreasing residual drift. The 850 mm-height jacket significantly increased the strength, but ductility and total ED capacity. On the other hand, the 400 mm-height jacket exhibited superior performance in ductility and total ED capacity. Three different strengthening mechanisms are derived for UHPFRC jacket, and they are enlarging cross-section effect, gap-opening effect, and passive confinement effect. Among them, passive confinement effect on rectangular RC shaft is analytically derived. The fiber-based finite element (FE) model for the UHPFRC jacket-retrofitted RC pier is developed within OpenSees framework. The simulation agrees well with the test and can benefit further seismic research of the UHPFRC jacket-retrofitted pier.

Keywords: Retrofitting; UHPFRC jacket; Cyclic loading; Fiber-based finite element.

1 Introduction

Considering that conventional retrofitting methods, e.g. RC jacket, steel jacket, and FRP jacket, are less effective in rectangular piers, the rectangular cross-section pier was deliberatively selected. In this study, one “as-built” RC pier and two UHPFRC jacket-retrofitted rectangular piers were fabricated, all of them were with relatively lower longitudinal reinforcement ratio, to represent existing old RC bridge piers in China, and were designed to exhibit flexural-dominated failure mode. Low cyclic loading tests were performed in order to investigated the effectiveness of UHPFRC jacket on seismic behaviors of retrofitted piers, including strength, stiffness, energy dissipation, self-centering, and damage, etc. Passive confinement effect from UHPFRC jacket is analytically evaluated. The fiber-based FE model using the OpenSees platform is developed for UHPFRC jacket-retrofitted RC piers, based on the three strengthening effects observed in the test.

2 Experimental Study

Three 1:2 scaled “as-built” RC bridge piers were firstly fabricated in the lab. After 28-day curing, two “as-built” RC piers were “retrofitted” with UHPFRC jackets. Pier UR denotes the un-retrofitted (“as-built”) pier, owning a 0.45 × 0.50 × 2.30 m rectangular RC shaft, a 0.90 × 0.70 × 1.30 m RC footing and a 0.70 × 0.80 × 0.70 m RC cap, see Fig. 1. Horizontal reversed load was exerted at the midpoint of the RC cap, through a 6000kN MTS electrohydraulic servo machine. The height-to-depth ratio for pier UR was around 5.89, and the flexural-dominated failure mode was deliberatively expected. Two of the “as-built” bridge piers were subsequently “retrofitted”, with the hollow rectangular cross-section (0.50×0.55 m, thickness: 0.05 m) UHPFRC jackets, Fig. 1.

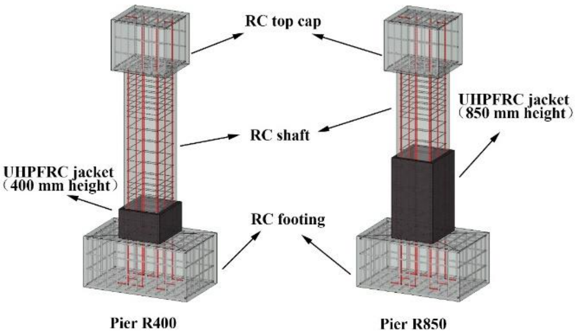

Fig. 1. Location of the UHPFRC jacket.

Test setup for cyclically loading is shown in the left figure of Fig. 2. During the test, constant vertical axial load $N = 0.08 f_c' A_g$ (about 628 kN) was maintained through hydraulic jacks and pre-tensioned thread bars, which represented the self-weight from the superstructure. Under large lateral drift, the two thread bars inclined and produced a trace of horizontal restoring force. The displacement-control loading scheme is illustrated in the right figure of Fig. 2. Three repeated cycles were performed at each loading phase. 5 mm increment was set for loading phases with ≤50 mm amplitude, and subsequently amplified to 10 mm until the termination of the test.

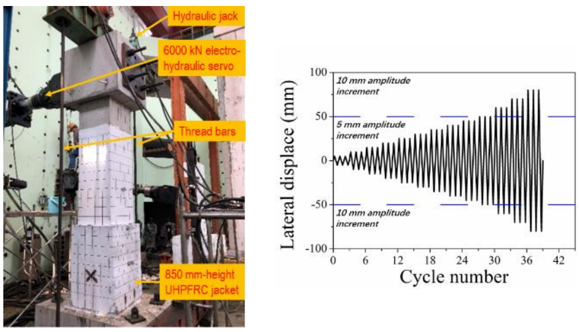

Fig.2. Test setup for pier R850 and loading scheme.

3 Conclusions

An exploratory test was performed to illustrate the high potential of UHPFRC jacket in retrofitting existing bridge piers with relatively lower longitudinal reinforcement ratio. One “as-built” RC pier and two retrofitted RC piers were tested under constant axial load and incrementally increasing lateral displacement cycles.

The tests presented herein provide a useful contribution to seismic retrofitting of existing RC bridge piers, using the novel UHPFRC jacket. This retrofitting method was effective for rectangular cross-section RC pier, which was dilemma with conventional RC, steel, or FRP jacket. Furthermore, UHPFRC jacket owns wider range of application, as it avoid the durability and fire-resistant problems for steel and FRP jacket. The fiber-based FE model is also proposed for UHPFRC jacket-retrofitted RC piers. The test is limited due to the quantity of rehabilitated bridge piers. More test are needed to further determined the mechanism and optimum design of thickness and height of UHPFRC jacket.

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Construction of the stayed cable bridge, Samdo Bridge

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Abstract

Samdo Bridge in Korea was constructed by changing segment length and form traveler type due to difference between original design and site condition. The segment length changed to 4.5m from 9.0m, the form traveler type also changed to the above type. The construction stages and cable tensioning force were modified to control the concrete girder stress during construction.

Keywords: Stayed cable bridge, Independent design check, Geometry control, Camber

1 Introduction

Samdo bridge consists of 130m + 290m + 130m span with the concrete edge girder section. This bridge is located at between Haui island and Sinui island in Korea. It was constructed by free cantilever method. The main services of COWI Korea were the independent design check and construction engineering during erection phase.

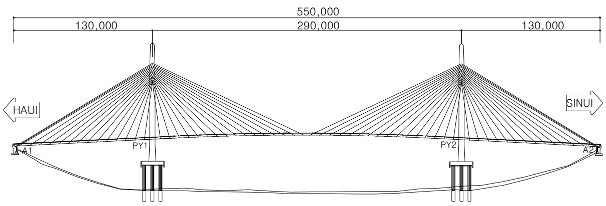

Figure 1. General view of Samdo bridge

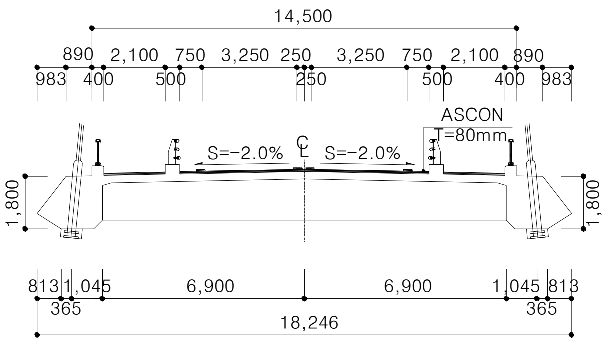

Figure 2. Cross section of Samdo bridge

2 Independent design check

Independent design check is performed in order to check the stability and reflect the construction site conditions before erection.

2.1 Findings

There were some problems in adapting the original design to construction site. The first problem was the concrete supplement. The construction plan of the original design was constructing a segment of 9m length with a below type of form traveler. However, due to the bridge being on the island, the concrete was supplied from the land. So the amount of one segment could not be transported in one day.

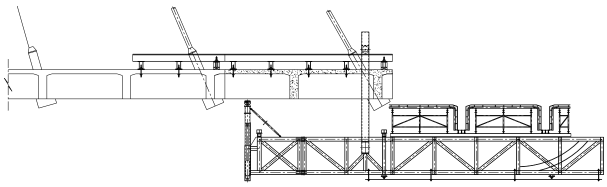

Figure 3. Below type F/T for 9m segment

The second problem was that the transverse crack on the girder is expected during construction. Since the original design allows the cracks during construction based on the design criteria, the stiffness of the girder is not matched to the actual case when the elastic analysis is performed. This is not a problem in the design, but it will occur a problem in the geometry control that the displacement does not match with the actual deflection.

2.2 Solutions

2.2.1 Change segment length

To solve the problem of supplying concrete the segment length is changed to 4.5m instead of 9.0m. Furthermore above type of form traveler is applied to the segment erection in order to increase constructability.

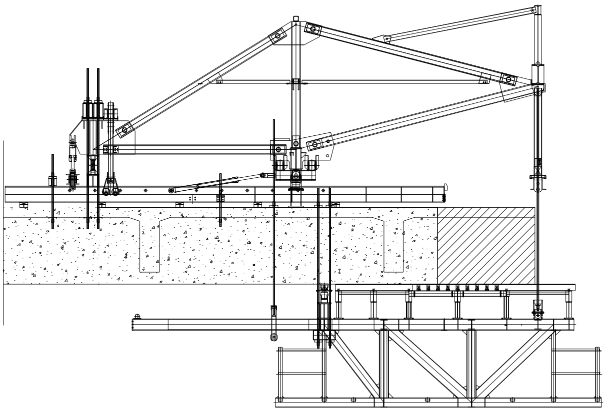

Figure 4. Above type of F/T for 4.5m segment

2.2.2 Temporary prestressing bar

To reduce the crack width and tensile stress of the girder, temporary prestressing bar is applied to the girder. 4 prestressing bars are installed on the girder top surface. These bars were removed after critical construction stage.

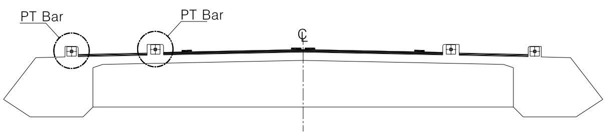

Figure 5. Temporary prestressing bars

3 Geometry control

Geometry control is the process of calculating and applying the camber to the site to meet the designed level after bridge completion.

3.1 Analysis

The bridge model for geometry control is developed using RM Bridge V8i. The dead loads were recalculated and applied to the geometry control model in comparison with the drawing and actual construction conditions.

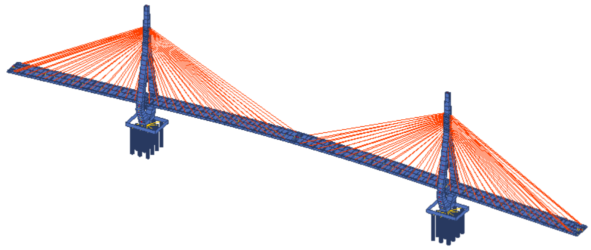

Figure 6. Analysis model for geometry control

3.2 Camber control

In order to construct the bridge with elevation consistent with the plan, the following camber was calculated and applied during construction. The target day of camber was 10000 days.

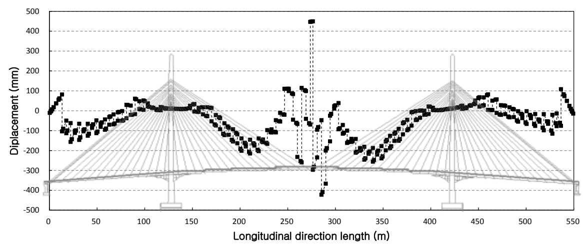

Figure 6. Bridge camber

3.3 Cable tensioning

There were 2 steps for the one cable tensioning. First step is for the segment with cable and second step is for the segment without cable. This is for the control the stress of concrete girder due to the concrete pouring load.

4 Conclusions

Due to the differences between the original design and the site condition the segment length and form traveler are changed. Samdo bridge is completed in May 2017 successfully.

Figure 7. Samdo bridge

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Impact resistant behavior of steel fiber reinforced PFC beam strengthened with AFRP sheet

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Abstract

To investigate impact resistant behavior of Porosity Free Concrete (PFC) beam strengthened with Aramid Fiber Reinforced Polymer (AFRP) sheet, impact loading test for the beams were conducted varying steel fiber mixing ratio and falling height of weight. Here, PFC is newly developed fiber reinforced cementitious composite which compressive strength is over 300 MPa. The primal results obtained from this test are as follows: 1) increasing the fiber mixing ratio, bridging effects of the fiber could be occurred after cracking, and tensile stress could be reduced; 2) bonding performance between PFC and AFRP sheet could be kept enough in spite of after sheet breaking.

Keywords: Porosity Free Concrete (PFC), steel fiber, AFRP sheet strengthening, impact resistance

1 Introduction

In order to investigate impact resistance behavior of Porosity Free Concrete (PFC) beam strengthened with Aramid Fiber Reinforced Polymer (AFRP) sheet, impact loading tests were conducted taking fiber mixing ratio and falling height of weight as variables. Here, PFC is the newly developed concrete that has the world's highest compressive strength over 300 MPa.

2 Experimental overview

Figure 1 shows the geometry of the specimen. The specimen used in this study is a PFC beam reinforced with AFRP sheet which dimension is 100 x 25 x 500 mm.

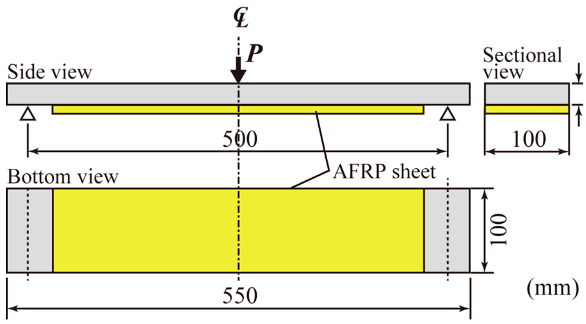


Figure 1. Geometry of specimen

Table 1 shows a list of specimens used in this experiment. The number of test specimens is 8 in total with varying steel fiber mixing ratio and weight falling height. Table 2 shows a list of calculated load-carrying capacities. In this calculations, the PFC and AFRP sheets were assumed to be bonded perfectly up to reaching the ultimate state of the beam.

Impact loading tests were carried out based on a single loading method in which a steel weight with a mass of 20 kg and a tip diameter of 60 mm is freely dropped once. In addition, loading point is limited to the center portion of the beam span.

Table 1. List of specimens

Specimen	Fiber mixing ratio *Vf* (%)	Falling height of weight *H* (mm)
PFC0-H150/300	0	150, 300
PFC1-H300/450/600	1	300, 450, 600
PFC2-H300/450/600	2	300, 450, 600

Table 2. Calculated capacity of each beam

Spec-imen	Strength of PFC (MPa)	Bending Capacity *Pu* (kN)	Shear Capacity *Vu* (kN)	Shear bending capacity ratio
PFC0	327	7.52	16.3	2.16
PFC1	351	8.45	35.1	4.15
PFC2	336	9.32	53.1	5.70

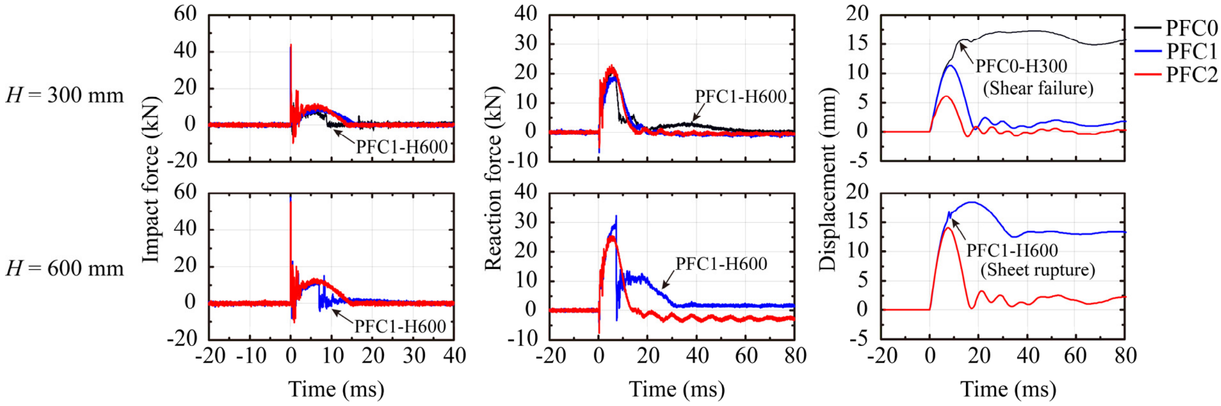


Figure 2. Time history of each response wave

3 Experimental results

3.1 Time history of each response wave

Figure 2 shows the time histories of response waveforms. From this figure, it is seen that the impact force excites as a waveform with long duration and large amplitude at the time of weight collision, and then a main wave with duration of about 15 ms and amplitude of about 10 kN is generated except for PFC0-H300 and PFC1-H600. In the cases of PFC0-H300 and PFC1-H600, the weight impact force sharply decreased at elapsed time of about 10 ms. This is probably due to the disappearance of the resistance of the beam caused by the shear failure of the PFC beam or the rupture of the AFRP sheet.

The reaction force shows that the main wave with the duration of about 15 ms and the maximum amplitude of 20 - 30 kN is excited except for PFC0-H300 and PFC1-H600. In the cases of the PFC0-H300 and PFC1-H600, the reaction force drastically decreased at about 10 ms as in the case of the weight impact force.

As for the displacement wave, except for the PFC0-H300 and PFC1-H600, a half-sine wave with duration of about 15 - 20 ms is excited and the amplitude tends to increase with increase in the falling height *H*. Also, in the case of *H* = 300 mm, it is seen that the beam was damaged severely because the deflection significantly remained.

3.2 Failure conditions

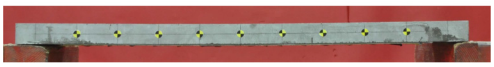
Figure 3 shows the failure behavior of the PFC0-H300, PFC1-H600, and PFC2-H600.

In the case of the PFC0-H300, diagonal cracks occur, and the upper surface of PFC beam are widely exfoliated. In the case of the PFC1-H600, symmetrical bending deformation remains, and in the vicinity of the loading point there are crack openings on the beam side surface and rupture in the bottom sheet. However, diagonal cracks and exfoliation on the upper surface cannot be observed because of the bridging effect of the fiber.

In the case of the PFC2-H600, remarkable residual deformation and sheet breakage, which were observed in the PFC1-H600, were not occurred. This result suggests that the bridging effect of steel fiber could be improved by increasing the mixing ratio of the fiber, so that the crack openings were restrained and the tensile stress subjected to the AFRP sheet was reduced.

(a) PFC0-H300 (Shear failure)

(b) PFC1-H600 (Sheet rupture)



(c) PFC2-H600 (No sheet rupture)

Figure 3. Failure conditions of beams

4 Conclusion

1) By applying the steel fibers, its bridging effect after cracking of PFC is exerted, so that the shear resistance of the beam is improved and the tensile stress subjected to the AFRP sheet is also reduced.

2) The adhesion performance of AFRP sheet and PFC is sufficiently secured until sheet breakage.

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Vibration Testing and Damping Identification of a Long-Span Suspended Footbridge

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Abstract

This study focuses on operational modal analysis (OMA)-based damping estimation of a long-span suspended footbridge for a serviceability assesment. Data acquired from vibration tests performed on the Majang Lake Suspended Bridge, the longest suspended footbridge in Korea, were used for the analysis. Using frequency domain decomposition (FDD) and natural excitation technique (NExT) with eigensystem realization algorithm (ERA), higher modes and their damping ratios were investigated.

Keywords: Pedestrian bridges; Monitoring; System Identification; Damping; Serviceability;

1 Introduction

In recent years, there has been a surge in the number of suspended footbridges in Korea. Where there was only one suspended footbridge in Korea before 2000, that number has risen to 10 from 2000 to 2010 and has since then rapidly increased to 49 in as of 2017. [1] However, this type of structure is relatively new and inherently sensitive to pedestrian loading by design. Thus, it is important to properly consider the unique structural dynamic characteristics of these suspended footbridges.

Because of its flexibility and light weight, damping ratio is an important indicator for long-span footbridges when assessing vibration serviceability. Using various output-only modal analysis techniques such as frequency domain decomposition (FDD), natural excitation technique (NExT) and eigensystem realization algorithm (ERA) will allow us to investigate the complex dynamic properties of the structure.

2 Bridge and Test Procedure

2.1 Majang Lake Suspended Bridge

Majang Lake Suspended Bridge is a suspended bridge without main towers located in Paju City, Gyeonggi-do, Korea. With a span length of 220 m, it is the longest suspended footbridge in Korea. Its deck is comprised mainly of wood and steel, with cables at the sides to support the structure.

Figure 1. Majang Lake Suspended Bridge

2.2 Vibration Test Procedure

On February 23rd, 2018, 12 accelerometers were installed to the bridge at 1/4, 3/8, 1/2 and 3/4 points of the span. One laser displacement transducer was installed at the center of the span.

In order to perform a variety of analyses, 13 forced vibration tests (FVT) and 22 pedestrian vibration tests (PVT) were performed with a varying number of people, loading point and frequency of excitation.

3 Damping Identification

3.1 Frequency Domain Decomposition

FDD is an output-only modal analysis technique that uses singular value decomposition to identify modes from multiple sensor channels. One of FDD's advantage is its ability to identify closely situated modes that may cause uncertainty for peak-picking based techniques. By applying FDD to a case where random pedestrian loading was applied, 12 structural modes, some with mixed dominant modes were identified. Because the results are not presented in a typical nth mode format, we use T to denote the identified modes.

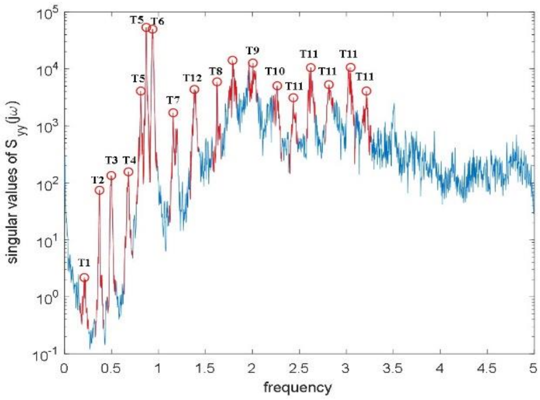

Figure 2. Power Spectral Density from FDD

3.2 Natural Excitation Technique and Eigensystem Realization Algorithm

Another output-only modal analysis technique used in this study, NExT, allows us to obtain an impulse response function (IRF) from ambient vibration with the assumption that the loading is a stationary white-noise excitation. In conjunction with ERA, which allows us to obtain damping ratios from the IRF, damping ratios of the modes identified with FDD were estimated. However, some of the modes, especially T7, showed fluctuations in its damping ratio depending on the length of the IRF. Therefore, acquiring longer datasets through continuous monitoring is required to improve reliability of the estimated damping ratios.

Table 1. Results of Damping Estimation

Mode Freq. (Hz)	Mode	FVT-10	FVT-13	PVT-17/18
0.484	T3	1.2~1.4%	-	-
0.967	T5	1.00%	-	-
1.149	T7	-	0.2~1.4%	-
1.353	T12	0.7~1.1%	-	-
1.637	T8	0.8~2.0%	1.0~1.4%	-
2.264	T10	-	**0.5~0.7%**	-
2.912	T11	-	**0.6~0.9%**	**0.6%**
3.541	T11	-	-	**0.4~0.6%**

4 Conclusion

In this study, we can observe that FDD is effective for output-only modal analysis for a long-span suspended footbridge. Estimation of higher modes reveal that modes that are higher than 2 Hz, which are susceptible to excitation by pedestrians, have damping ratios ranging from 0.4~0.5%. Further studies must be done for a longer dataset to continuously monitor and refine our results.

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6 Acknowledgement

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Experimental Study of Wave Loads on Elevated Pile Cap of Pile Group Foundation for Sea-crossing Bridges

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Abstract

Elevated pile cap, which transits the load from sea-crossing bridge pylon to pile group, is always close to the sea level and suffering from extreme wave loadings. This paper describes a wave flume experimental program to investigate wave loads on the elevated pile cap of pile foundation for sea-crossing bridges. Two specimen configurations including isolated cap and cap with pile group were set up with different clearance heights. Wave forces on the cap, including horizontal and vertical force and moment, were measured simultaneously under five nonlinear regular wave trains. The effects of pile group, clearance height and wave height on wave forces on the cap were investigated experimentally. Clearance height and wave height affect wave forces significantly. The largest horizontal force occurs when the cap is fully submerged, while the largest vertical force occurs when the bottom of the cap locates at still water level (SWL). As wave height increases, horizontal forces increase except for fully submerged case and vertical forces increase firstly but decrease when the wave height exceeds 0.18 m. Pile group makes little influence on the horizontal forces but increases the vertical uplift forces slightly when the bottom of the cap locates above SWL.

Keywords: Wave force; Elevated pile cap; Sea-crossing bridges; Wave flume experiment; Pile group.

1 Introduction

Pile group foundation is widely used to support contemporary long-span sea-crossing bridges due to its good applicability in marine environment with deep water and complex seabed terrain. However, the elevated pile cap, which transits the load from bridge pylon to pile group, is always close to the sea level and suffering from extreme wave loadings.

Although the first order diffraction theory is commonly used to assess the wave force on the large-scale marine structures, such as elevated pile cap, it is not sufficient to deal with waves with nonlinear kinematics and caps locates above the mean sea level (Molin, 2002; Ti et al., 2018). Moreover, the blocking and shading effect of pile group on the wave force of the pile cap has yet been well understood.

This paper presents a series of experiments to investigate the wave forces acting on the elevated cap of pile foundation for sea-crossing bridges. In the experiments, 5th-order Strokes waves with five different heights and associated lowest periods according to breaking limits were used to simulate extreme waves, and the specimen was under submerged and elevated conditions with consideration of five clearance heights, and two configurations of the specimen including isolated cap and cap with pile group were set up. Accordingly, the following objectives are set: (1) to discern the characteristics of the time history of wave forces under different clearance conditions and wave conditions; (2) to understand effects of clearance height and wave height on the wave loads acting on the pile cap; (3) to investigate the effect of pile group on the wave loads acting on the pile cap.

2 Experimental setup

Four wave gauges were installed in the flume to measure the water surface elevations. The F/T (force and torque) transducer was used to measure both force and torque acting on the specimen along X-, Y- and Z-directions. The reduced scale tested specimen using a 1:90 geometric scaling, was composed of two parts, including a group of 12 circular piles and a rectangular pile cap. Two configurations of tested specimen, (1) the isolated pile cap and (2) the pile cap with pile group, were set up. Five different clearance heights were investigated experimentally for both configurations of tested specimen.

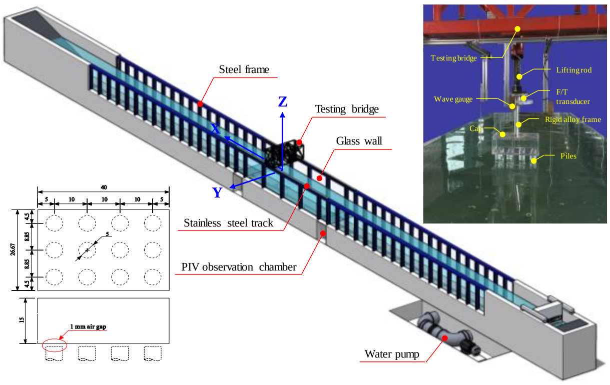
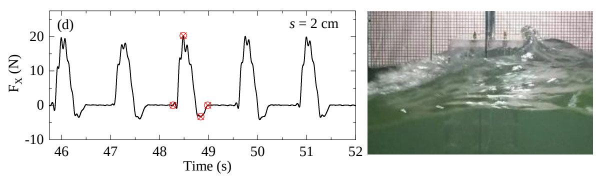


Fig. 1 The testing facility and specimen.

3 Results and discussions

The horizontal wave force arises abruptly reaching to the crest in a short time, then diminishes below zero, and finally returns to zero. The vertical wave forces generally go through three phases. Initially, slamming or impulsive force occurs with considerable magnitude but exceptionally short duration, followed by quasi-static or pulsating small-scale uplift force dwindling down in relatively long duration. Finally, vertical downward pulling force occurs and turns to zero when waves separate from the cap.



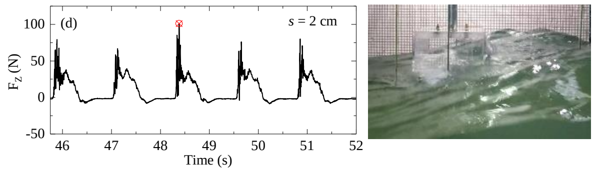


Fig. 3 Time series of horizontal and vertical fore.

And effects of pile group, clearance height and wave height on the wave loads acting on the cap are investigated based on the experimental data.

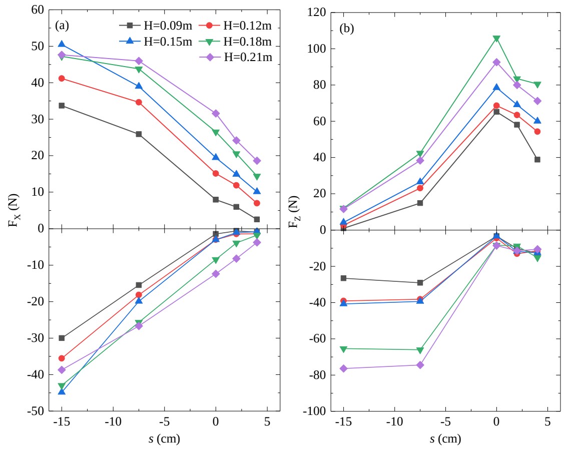


Fig. 4 Representative wave forces versus clearance heights under different wave height.

4 Conclusions

The wave force on the cap is significantly related to the effects of clearance height and wave height. The largest horizontal wave force occurs when the cap is fully submerged, while the largest vertical maximum force occurs when the bottom of the cap locates at SWL. The increase of wave height induces a growth in horizontal wave force. The vertical wave force increases firstly but exhibits a decreasing tendency when the wave height exceeds 0.18 m due to overtopping waves.

5 References

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[2] Ti, Z.L., Wei, K., Qin, S.Q., Mei, D.P., Li, Y.L., 2018. Assessment of random wave pressure on the construction cofferdam for sea-crossing bridges under tropical cyclone. Ocean Engineering, 160, 335-345.

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Push-out tests of perfobond strip with steel fiber reinforced mortar

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Abstract

Perfobond strip is widely used as a shear connector in various steel-concrete hybrid members and structures. Nowadays, it has been also applied to connections between two precast concrete members. In these connections, it is effective to combine the perfobond strip with steel fiber reinforced mortar (SFRM) in order to enhance the restraint effect without reinforcements around the perfobond strip. In this study, the push-out tests of perfobond strip with SFRM were carried out, and its shear resistance and shear force-relative slip relations were investigated with various experimental parameters.

Keywords: Push-out tests, perfobond strip, SFRM, shear resistance, connections between precast concrete members

1 Introduction

Fig.1 shows an application of perfobond strip to the connection between two precast PC slabs [1]. A half part of the perfobond steel plate is fixed to the base precast slab by the penetrating rebars inside perforations, and the other half part is arranged in the connection between two precast PC slabs. Therefore, two precast PC slabs are integrated after casting concrete in the connection. The similar method is also considered for application to the other structures such as the connections between precast concrete railings of bridges, or precast concrete walls of buildings.

In above connections, it is expected to shorten the construction period and to design the connection more compact by reducing the arrangement work of loop reinforcements such as the conventional method. In this method, since the reinforcements are not arranged in the connection, it is important to enhance the restraint effect around the perfobond strip to exert sufficient the shear resistance of the perfobond strip. Therefore, it is effective to combine the perfobond strip with steel fiber reinforced mortar (SFRM).

In this study, the push-out tests of perfobond strip with SFRM were carried out, and the influences of the compressive strength of SFRM, the perforation diameter and the block dimensions of specimen were discussed focusing on its shear resistance and shear force-relative slip relations.

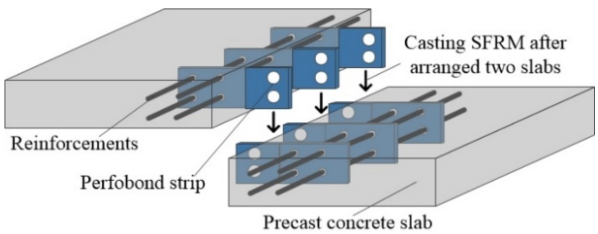

Figure 1. Connection between two precast PC slabs with perfobond strip

2 Push-out tests and results

The push-out specimen is shown in Fig.2 and the parameters of specimens are shown in Tab.1. The experimental parameters are, the perforation diameter, dimensions of the mortar block, compressive strength of SFRM which depends on its age. The specimen name in Tab.1 are represented by the perforation diameter D, age P of SFRM, and S, M and L denotes the dimensions of mortar block. Note, SFRM for specimens is composed of a cement, water in nearly 20℃ and steel fibers. The diameter and the length of the

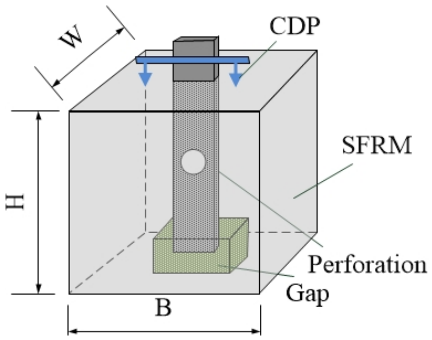

Figure 2. Push-out specimen

Table 1. List of push-out specimens

Name	D(mm)	f'$_c$ (N/mm^2)	H-B-W (mm)	Number
D35P1S	35	59.7	150-155-125	3
D35P3S		76.7		3
D35P28S		94.4		2
D35P3M		85.6	150-300-200	3
D35P28M		88.8		3
D35P3L		91.3	200-300-300	3
D35P28L		102.1		3
D45P28L	45	101.5	200-300-300	2

steel fiber are 0.2mm and 15mm respectively, and the mixing ratio of steel fibers in SFRM is 9%.

An example of shear force-relative slip relations of the perfobond strip with SFRM is shown in Fig.3. The specimens in this figure have the same perforation diameter (35mm), the same age of SFRM (28days) and, the different block dimensions is distinguished by the color of lines. It is confirmed that the relative slip is smaller than 0.1mm when the shear force reaches nearly 80kN, and the relative slip at the shear resistance are from 0.5 to 1.3mm. From these relations, it can be seen that the sudden decrease of shear force is not observed after the shear fracture of the perfobond strip, and the shear force at the relative slip of 5mm is nearly 90% of the shear resistance.

From the shear resistance of all specimens in Tab.1, relations between the shear resistance of specimens and its parameters are shown in Fig.4. It is confirmed that the shear resistance (Q_u) of the perfobond strip with SFRM has a good correlation with the value of $Af'_cV_b^{0.24}$. It is said that the shear resistance of the perfobond strip with SFRM increases with the perforation area (A), SFRM compressive strength (f'_c) and the volume (V_b) of mortar block of specimen, and its shear resistance can be evaluated by considering the relative relation of these parameters.

3 Conclusions

The shear resistance of the perfobond strip with SFRM increases with the perforation area, SFRM compressive strength and the block dimensions of specimen, and its shear resistance can be evaluated by considering the relative relation of these parameters. In addition, steel fibers in SFRM contributes to prevent the sudden decrease of shear force after the shear fracture of the perfobond strip.

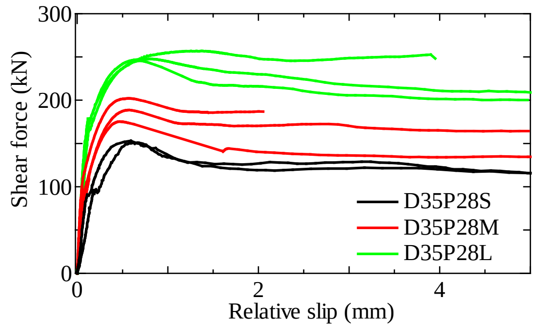

Figure 3. Shear force-relative slip relations

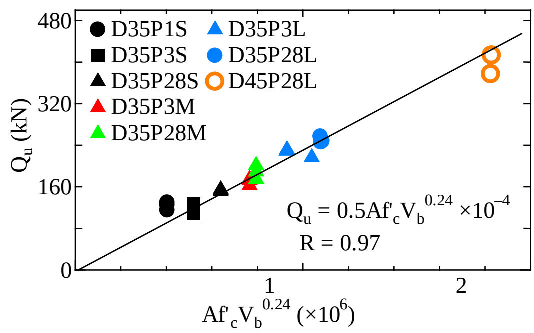

Figure 4. Relations between shear resistance and experimental parameters

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Experimental study on cable force measurement by cable frequency method in cable workshop

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Abstract

Frequency method has been widely used in cable force detection because of its simple operation and economical and practical application. After the cable is installed with damper, the cable boundary conditions become complex, and the natural frequency of vibration will change, which will affect the accuracy of measuring cable force by frequency method. In this paper, the long cable is used to test the cable force in the tension groove. The relationship between frequency and cable force is established by using the theoretical string formula, dimensionless parameter method and Mode shape and Modal frequency-based method using multiple vibration measurements. The results show that the virtual cable length method can obtain high accuracy without the damper parameters.

Keywords: cable force; frequency; force experiment, virtual cable length method, damper.

1 Introduction

The cable-supported bridge with a span of 200m or more is very competitive and has been used more and more. The cable is one of the core load-bearing components of the cable supported bridge. Frequency method has been widely used in cable force detection because of its simple operation and economical and practical application. The key of measuring cable force by frequency method is the relationship between cable force and frequency under different boundary conditions. There is a lot of literature on this issue. This paper aims at cable force identification problem after installing damper. A LZM7-85 parallel wire cable is designed, which is 167.85m long. The cable force test was carried out by frequency method in the tension groove of the company's cable workshop. The test results show that for the length of the cable, the formulas for the string can achieve good accuracy, dimensionless parameter method presented good fitting linear relationship, Mode shape and Modal frequency-based method using multiple vibration measurements appearing in the treatment of before and after installing damper cable force identification has a certain advantage.

2 Experiment Scheme

2.1 Cable parameter

The test cable is a complete set of LZM7-85 wire cable system of 167.85m, including anchor, beam end casing, connecting device and other parts. The cable section is shown in figure 1, and the cable parameters are shown in table 1.

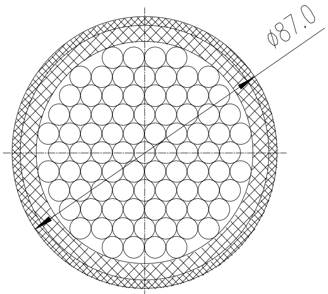

fig 1.cable section

2.2 Cable installation

The cable installation is shown in figure 3.

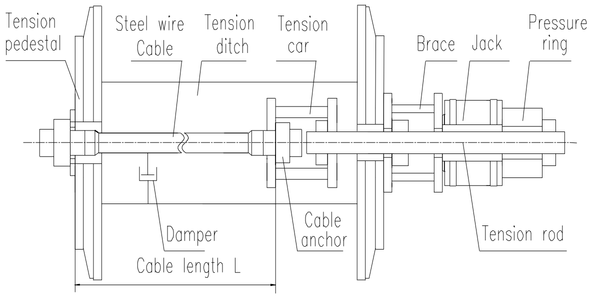

fig 3.cable installation

3 Data Analysis

3.1 Theoretical string formula

The test cable is a long cable. Without considering the influence of the bending stiffness of the cable, when there is no damper, the cable vibration frequency is calculated by sinusoidal theoretical formula. As shown in table 2 below, the 5th order frequency is used to calculate the tension.

3.2 Dimensionless parameter method

The cable's dimensionless parameter model is established by using the cable force -- frequency. As shown in formula (1), the dimensionless parameter model is studied to calculate the cable's force precision. The linear model is established by using the cable force -- frequency of level 2 and level 4. Then 1st, 3rd and 5th frequencies are substituted into the linear model to calculate the cable force, the result when there is no damper is shown in table 4.

$$T = Af_n^2 + B \qquad (1)$$

3.3 Mode shape and Modal frequency-based method using multiple vibration measurements

In the case of no damping, the characteristic system realization method (ERA) is used to identify the frequency and mode of the cable, and the frequency and mode of some vibration modes of the cable are obtained, as shown in table 6 below.

4 Conclusions

Frequency acquisition was carried out in the 167.85m length of the tension groove designed in the cable production workshop. The relationship between frequency and cable force before and after the installation of the damper was compared and analyzed by using the theoretical string formula method, dimensionless parameter method and the virtual cable length method. The results show that the cable force error is less than 1% without damper installation and 7% with damper installation. Before and after the damper installation, the dimensionless parameter method was used to calculate the cable force error less than 1%, that is, there is a good linear relationship between the cable force and frequency square. The Mode shape and Modal frequency-based method using multiple vibration measurements is not necessary to consider the damper parameters and other information. The cable force error before and after installing the damper by virtual cable length method is 2%. And then it has high application value in real bridge.

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Technical Session III

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Reliability-Based Fatigue Life Evaluation of Deteriorated Steel Structural Members

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Abstract

Due to social and economic needs for their use continued, it is indispensable to assess current reliability level of several bridges in the world have been built a long time ago. This study presents a fatigue life evaluation procedures that can be used to perform reasonable fatigue life evaluations of deteriorated steel structural members using inspection results. At this time, the corrosion nucleation time and the initial crack depth are set to random variable by using MCS because there are strong uncertainties. To verify the effectiveness of the developed procedure, a case study was conducted using monitoring data from an existing bridge.

Keywords: Fatigue life evaluation, deterioration, corrosion, steel member, reliability index

1 Introduction

Several bridges in the world have been built a long time ago. Due to social and economic needs for their use continued, it is indispensable to assess their current level of reliability. This study presents a fatigue life evaluation procedures that can be used to perform reasonable fatigue life evaluations of deteriorated steel structural members using inspection results. The reliability index (β) and probability of failure (P_f), calculated based on the procedure, are used to secure the engineering basis of the procedure and standard. For deteriorated steel members, fatigue life evaluation methods should differ depending on the cause of the fatigue failure, such as like cracks and corrosion. Therefore, the suggested technique is to undertake a fatigue life evaluation upon the detection of cracks or corrosion on deteriorated steel members.

2 Deterioration Scenarios and Definition of the Fatigue Limit State

When inspecting deteriorated steel structural members, the definition of the limit state for performing a reliability-based fatigue life evaluation depends on the type of damage found.

2.1 Scenario I: only a fatigue crack is found

In this case, the linear elastic fracture mechanics (LEFM) is used to set the limit state of scenario I (g_I).

$$g_I = \int_{a_0}^{a_c} \frac{da}{C\cdot\left(\Delta K\right)^m \cdot 365 \cdot ADSC} - t_N \tag{1}$$

Where a_c and a_0 are critical crack depth and initial crack depth. C and m are material constants, and ΔK is the stress intensity factor range at the crack tip. Here, the initial crack depth (a_0) has a strong uncertainty. Inspection results like inspection time (t_{ins}) and inspected crack depth (a_{ins}) can be utilized to refine the variable. Monte-Carlo Simulation (MCS) and Akaike Information Criterion (AIC) is used to set the new initial crack depth.

$$\begin{gathered} t_{ins} = \int_{a_0}^{a_{ins}} \frac{da}{C\cdot\Delta K^m \cdot 365 \cdot ADSC} \\ \Downarrow \\ a_0 = f\left(t_{ins}, a_{ins}, C, m, S_r, ADSC\right) \end{gathered} \tag{2}$$

2.2 Scenario II: Corrosion is found

A fatigue crack can occur at the corrosion site as the corrosion progresses due to the stress concentration and micro-cracks. Therefore, it is necessary to set a limit state that can reflect the general phenomena of corrosion nucleation, corrosion growth, the transition from corrosion to a fatigue crack, crack propagation, and a fatigue failure. The LEFM based approach is used to calculate a pit depth threshold (d_{th}) which means that the pit depth where a transition occurs from corrosion to a crack.

$$g_{II} = t_0 + \left(\frac{d_{th}}{p}\right)^{\frac{1}{q}} + \int_{d_{th}}^{a_c} \frac{da}{C \cdot \Delta K^m \cdot 365 \cdot ADSC} - t_N \quad (3)$$

Where t_0 is the corrosion nucleation time. p and q are environmental coefficients. In this scenario, the corrosion nucleation time (t_0) can be reproduced by using the inspection results. This case also utilized MCS and AIC.

$$d_{ins} - d_0 = p \cdot (t_{ins} - t_0)^q \Rightarrow t_0 = t_{ins} - \left(\frac{d_{ins} - d_0}{p}\right)^{\frac{1}{q}} \quad (4)$$

3 Case Study

A case study is conducted using existing steel bridge monitoring data from the US to confirm the effectiveness of the proposed procedure. Improved first order reliability method (iFORM) was used to the reliability analysis. Based on the Eq. (2) and (4), new random variables for a_0 an t_0 are generated. The sample of the results for MCS and data fitting is shown in Figure 1.

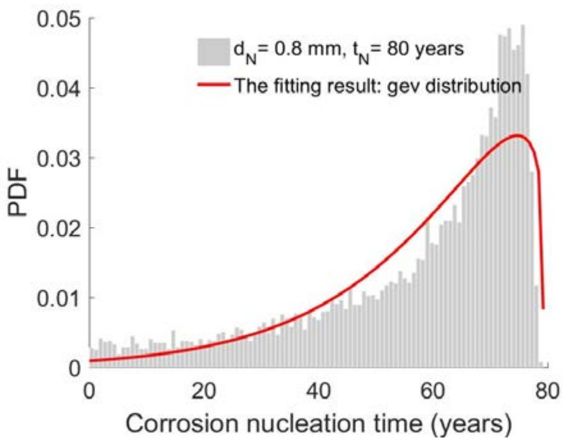

Figure 1. The sample of the results for MCS and data fitting; Scenario II, d_{ins}=0.8 mm

With the new random variables, iFORM was conducted and one of the results is shown in Figure 2. The case is for the top surface of the bottom flange corresponding to the scenario II. the reliability index decreases as the size of the corrosion pit found during the inspection increases.

4 Conclusions

The objective of this study was to develop a framework for reliability-based fatigue life evaluation of deteriorated steel structural members considering corrosion and fatigue crack as the major defects of steel members. At this time, the corrosion nucleation time and the initial crack depth are set to random variable by using MCS because there are strong uncertainties. To verify the effectiveness of the developed procedure, a case study was conducted using monitoring data from an existing bridge.

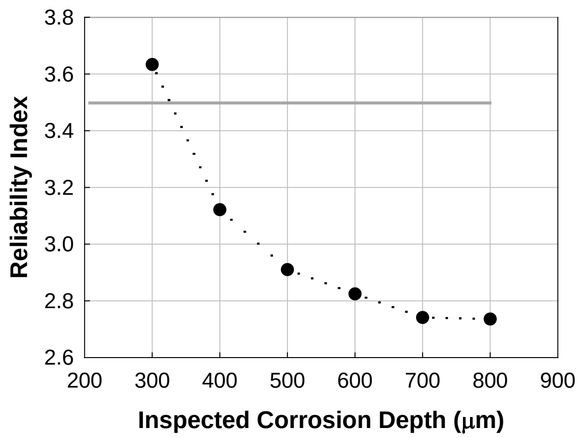

Figure 2. Results of fatigue reliability evaluation for cracking at the weld toe of bottom flange

5 Acknowledgement

This work was supported by Smart Civil Infrastructure Research Program (18SCIP-B128568-01) funded by Ministry of Land, Infrastructure and Transportation of Korean government.

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  I aim to conduct following research to participate in the research project on "Research on Evaluation of Performance and Repair-Strengthening for Deteriorated Short-medium Span Bridges Based on Full-Scale Experimental Verifications" supported by a grant (17SCIP-B128568-01 2/ 18SCIP-B128568-01) from Smart Civil Infrastructure Research Program funded by Ministry of Land, Infrastructure and Transport of Korean government.

  - Development of reliability-based fatigue life evaluation procedure of deteriorated steel structural member
  - Improvement of criteria to identify pit corrosion-to-fatigue crack transitions on steel member
  - Reliability-based improvement of the Korean condition evaluation standard of deteriorated steel member

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Simulation of Flow over Complex Terrain by Coupling of WRF and LES

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Abstract

Simulation of atmospheric boundary layer wind attracts interest from both meteorology and civil engineering. The coupling method of Weather Research and Forecasting mesoscale meteorological model and Large-eddy Simulation microscale computational fluid dynamics model satisfies both requirements of real-time wind speed prediction and turbulence simulation. A WRF study of complex terrain wind is carried out in the beginning, by which WRF's accuracy in surface wind speed prediction is improved. Next, by adding Random Flow Generation after interpolation of time-averaged variables in the downscale process from WRF to LES, the difficulty in artificial turbulence production is overcome. Finally, the turbulent wind and temperature field is applied as the inflow condition of LES domain and the flow over complex terrain is simulated. Generally, the coupling method of WRF and LES has provided better mean and fluctuating wind prediction than WRF. Further enhancement of high-frequency wind component requires higher resolution of elevation.

Keywords: Complex terrain wind; Numerical simulation; WRF+LES; Thermal effect.

1 Introduction

Atmospheric boundary layer wind is important to wind resistance of bridge and structures, wind energy exploitation and pollutant control. The Weather Research and Forecasting (WRF) mesoscale meteorological model provides wind and temperature real-time predictions without inflow assumption or measurement[1]. Large-eddy simulation (LES), as a kind of Computational Fluid Dynamic method, is good at simulating mean flow and turbulence in bluff body aerodynamics and complex terrain wind simulation. The coupling method linking mesoscale WRF with microscale LES (WRF+LES) satisfies requirements of both real-time wind prediction and surface turbulence simulation.

2 Method

The coupling method of WRF and LES can be divided into three parts. Firstly, high-resolution WRF study of complex terrain wind is presented, where modifications of surface roughness and elevation resolution have improved wind speed accuracy[2]. Secondly, a Random Flow Generation[3] step is added after interpolation of time-averaged variables like velocities and temperature, introducing artificial turbulence in the downscale process from WRF to LES, which overcomes the difficulty of turbulence simulation due to the grid space gap and turbulence model difference, as is shown in Fig. 1. Thirdly, by modification of the incompressible N-S equations using Boussinesq approximation, thermal effect on flow is simulated

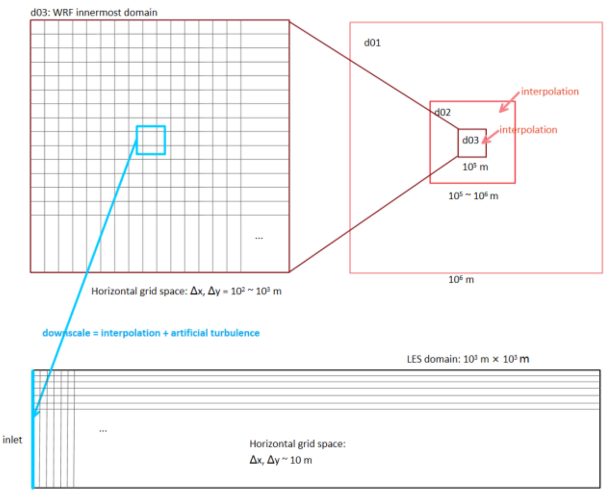

Figure 1. Downscale process from WRF to LES

in LES, which is essential since our previous work[4] has proved that a temperature inversion has direct influence on wind speed. Velocity and temperature results from the downscale process is defined as the inlet boundary condition of LES.

3 Results

Coupling of WRF and LES has noticeably outperformed WRF in both mean and fluctuating wind speed. Fig. 2 has compared WRF and WRF+LES wind speed history with observation. Obviously, WRF+LES has produced nice mean velocity results and generated significant fluctuations. Fig. 3 furtherly diagram the power distribution of wind speed using WRF and WRF+LES. WRF barely produces any high-frequency wind component, while WRF+LES method is much better at generating turbulence, which is also shown in Fig. 4, though there's still a gap between WRF+LES and observed/Karman spectrum. A further enhancement of high-frequency wind component requires higher resolution of elevation and denser grids. Also, Fig. 4 has shown that WRF+LES method has successfully simulated terrain effect on mean wind speed profile.

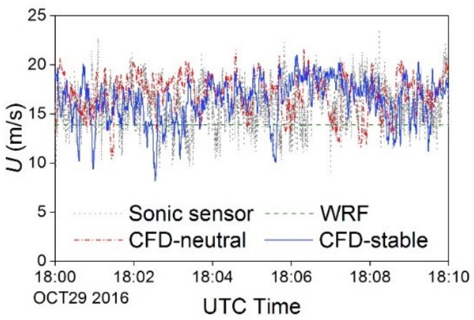

Figure 2. Comparison of wind speed history

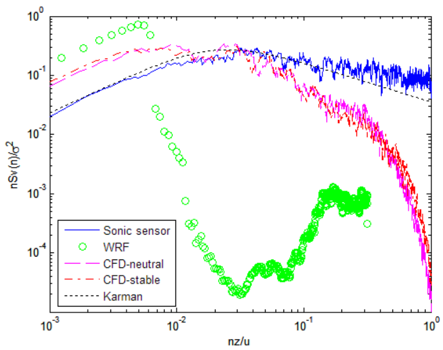

Figure 3. Comparison of wind speed spectrum

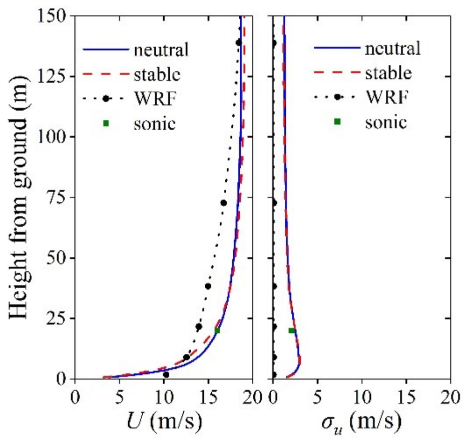

Figure 4. Mean and fluctuated wind speed profiles

4 Conclusions

Simulation of flow over complex terrain is accomplished by coupling of mesoscale WRF model and microscale LES model. WRF+LES method has noticeably outperformed WRF in both mean and fluctuating wind speed history and vertical profiles, proving its qualification in simulation of terrain effect and turbulence. Since WRF+LES predicts turbulent wind speed without any inflow assumptions, it has broad prospects in wind disaster prediction and pollution control.

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Basic study on torsional behavior of the box girder bridges subjected to thermal history due to fire

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Abstract

Recently, the fire attack for bridges due to a fire from accidents including rollover and collisions, fire from burned fields and so on trends to increase in Japan and in foreign countries. One example of such an accident occurred in Osaka, Japan, where the Osaka Port line of the Hanshin Expressway at the beginning of February 2006. In this case, the maximum temperature was above 400°C due to incendiary fire. The part of the lower flange in this box girder bridge was deformed owing to accident. Therefore, it is necessary to clarify the behavior of the box girder bridge subjected to thermal history. The authors carried out the torsional analysis focused on torsional behavior subjected to thermal history at the lower flange. In this paper, the results of the torsional moment and angle, shear stress of the steel web plate due to eccentric loading are reported and discussed.

Keywords: Fire; Box girder bridge; Torsional behavior; Eccentric loading.

1 Introduction

Recently, the fire attack for bridges due to a fire from accidents including rollover and collisions, burned fields and so on trends to increase in Japan and in foreign countries [1]. At present, the influence of the box girder bridges damaged by fire have not been clarified yet in Japan. Therefore, the authors carried out the torsional analysis of the box girder bridges after fire accidents. The purpose of this study is to clarify the torsional behavior subjected to thermal history at the lower flange.

2 Outline of the analysis

The authors carried out the analysis of the torsional behavior subjected to thermal history.

We noticed that the lower flange of the steel box girder bridge is deformed when the fire occurred under the bridge [2]. In our research, we assumed that the lower flange is only deformed due to fire under box girder bridge.

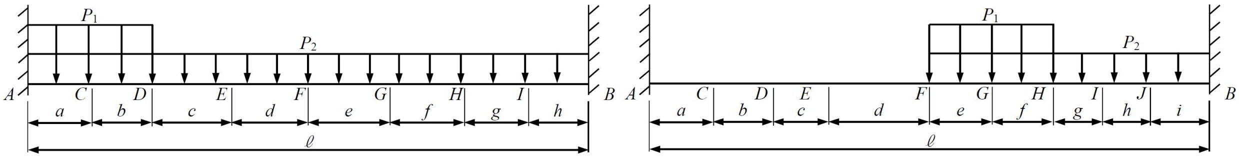
In this analysis, we simulated the deformed cross section of the lower flange. Table 1 shows the relationship between the deformation of the lower flange and the resistance cross section [3]. From this table, *w* means the interval of the ribs.

We set the parameters of the fire scale which is small scale (maximum temperature: about 700°C) and large scale (maximum temperature: about 1100°C). In addition, we changed the width of the lower flange from *b* to 0.25*b*, 0.50*b* and 0.75*b* as a damaged area due to fire, respectively.

The elastic modulus of the lower flange is obtained by multiplying subjected to thermal history to be shown steel reduction factor *k* at temperature [4].

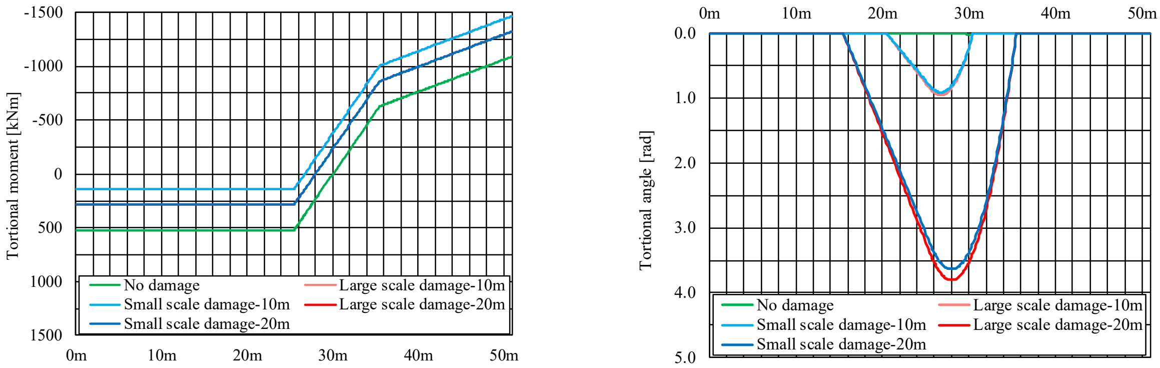
Table 1. Resistance cross section

Deformation	Method of calculation
$\delta < w/150$	Closed cross sections
$\delta > w/150$	Open cross sections



(a) Focus point A *(b) Focus point F*

Figure 1. Loading model



(a) Torsional moment *(b) Torsional angle*

Figure 2. Calculation results in focus point F

Table 2. Shear stress of the web (Unit: N/mm²*)*

Damage / Point	No damage	Fire area	
		10m	20m
A	142.1	140.3	141.0
F	43.9	41.5	42.4
B	-144.8	-146.7	-146.0

3 Torsional analysis Studies

In this analysis, we selected a 3 spans continuous steel box girder bridge with a center span of 51.0m. The steel material in center of span is SM490Y (near intermediate support) and SM400 (other area).

An example of the calculation results in focused point F is shown in Figure 2. We assumed that fire scale (small and large) and fire areas (10m and 20m from the center of the span). From this figure, we found that the fire scale unrelated the torsional moment and the torsional angle. Then, we calculated the shear stress of the steel web plate by applying the large scale damage. The calculation results of the shear stress of the steel web is shown in Table 2. From this table, we found that the shear stress of the web is changed according to increasing of fire area. However, there is no difference of length in this area.

4 Conclusions

The authors carried out analysis to clarify the torsional behavior subjected to thermal history at lower flange only.

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Training General Bridge Damage Detection Deep Net using Inspection Data Base

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Abstract

With the successful application of Convolutional Neural Network (Deep Net) based image processing algorithms, Deep Learning, in many fields such as image classification and object detections, the possibility of using this technology to develop a revolutionary automatic bridge inspection approach becomes more and more possible. In this study, the bridge damage photo reported in recent bridge inspection documents are collected to build a data base for training damage recognition machines. As training Convolutional Neural Network models need large amounts of pictures which are not available in short time for bridge damage detection mission, some data argumentation and data compensation efforts are conducted in this study to train a Deep Net with small amounts of pictures. The accuracy of the damage detection machines using different data base are evaluated. Finally, a picture of UAV inspection was used to test the availability of application of UAV and AI based bridge damage detection.

Keywords: Deep Net; Convolutional Neural Network; Bridge Damage; Image Classification; Damage Detection.

1 Introduction

For bridge structures, the inspection and health state evaluation are the essential issues for long term use and the most effective method to resist the deterioration in service or damage in earthquake or tsunami. However, the detailed close-access based vision investigation and hammer sound inspection are expensive in human cost, road occupation time, and rental cost of special inspection vehicles.

The technology to quickly collect image data, such as UAVs, are available to inspect the status of structure members in some difficult to access locations. It can dramatically fasten the progress of bridge status diagnose.

With the successful application of Convolutional Neural Network (Deep Net) based image processing algorithms, Deep Learning, in many fields such as image classification and object detections[1], the possibility of using this technology to develop a revolutionary automatic bridge inspection approach becomes more and more possible.

In this study, the bridge damage photo reported in recent bridge inspection documents are collected to build a data base for training damage recognition machines. As training Convolutional Neural Network models need large amounts of pictures which are not available in short time for bridge damage detection mission, some data argumentation and data compensation efforts are conducted in this study to train a Deep Net with small amounts of pictures. The accuracy of the damage detection machines using different data base are evaluated. Finally, a picture of UAV inspection was used to test the availability of application of UAV and AI based bridge damage detection.

2 Deep Net for Damage Detection

Here, a well-known model, GoogLeNet[1] which won the competition of large scale image recognition (ILSVRC) in 2015. GoogLeNet has a structure in which modules called Inception are stacked. In Inception, convolution is performed in parallel, and each obtained feature map is combined. The input size is 256 * 256 pixels, and the output size is 5 classes.

3 Bridge Damage Image Data Base

In this research, damaged images were extracted from bridge inspection reports with label of damage type, as shown in Fig.1. Though many images were found in the reports, a lot of them were taken from a distance and difficult to see the damage clearly.

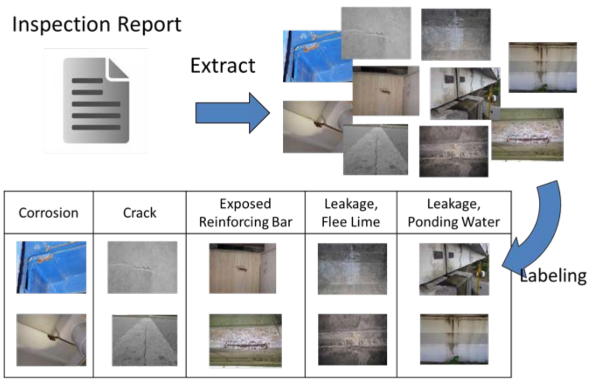

Figure 1. Structural Damage Detection Data-Base

4 Data Argumentation and Compensation

As a common limitation for structural engineers, data for real damage is very rare. For image classification, a few image process methods, commonly called as data argumentation, can be used to improve this situation of using small size of training data [2]. For data argumentation it can flip, rotate, or change contrast of the original data pictures to generate new pictures.

Images which are just notating the location of damages were deleted and damage area in the picture was cut out and resized (Data Compensation). By data argumentation and refinement, the accuracy for damage detection was increased from 50% to 90%.

5 Damage Detection for UAV Image

The 4K picture taken by UAV were separated to 20*10 small areas. As can be seen from Fig.2, corrosion areas were successfully recognized. However, some areas of background buildings were also mis-recognized as corrosion.

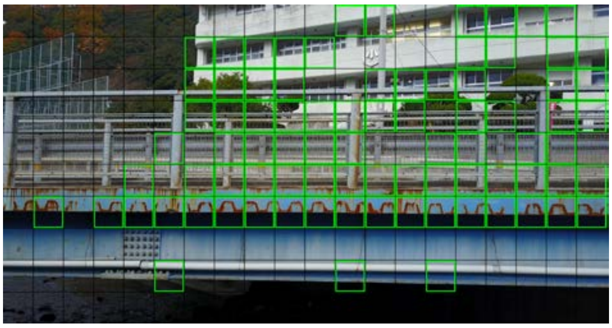

Figure 2. Damage Detection from UAV Picture

6 Conclusions

In this study, Deep Net trained by inspection report images can only detect structural deterioration damage in about 50% accuracy due to small number of the clear pictures. Data argumentation and refinement were found as useful method to improve the accuracy to 90%. By using the trained model to detect the damage in a UAV 4K photo shows a lot of mis-recognition of corrosions. More real word UAV pictures should be used in training the Damage Detection AI.

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Numerical Simulation of Mechanical Behaviour of Post-tension Prestressed Concrete Beam with Various Bond Condition of PC Bar

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Abstract

In this study, results of the loading tests of post-tension prestressed concrete (PC) beams with different bond condition of PC bar were selected as a research object. The structural performance of the PC beam was evaluated with Finite Element (FE) analysis. PC bar was simply expressed by using two kinds of line elements in analysis. As a result, numerical results were able to simulate the loading tests of PC beam by using appropriate constitutive model.

Keywords: Finite Element analysis; Post-tension; bond condition of PC bar; line element

1 Introduction

Some of Post-tension PC bridges which were constructed in 1960's and 1970's in Japan don't have grout between concrete and PC bar. To evaluate the structural performance of these structures accurately, the influence of bond condition of PC bar must be considered. However, little research work has been done about bond condition of PC bar. In this study, results of the loading tests of post-tension PC beam with different bond condition of PC bar [1] were selected as a research object. FE analysis was conducted to evaluate the structural performance of the PC beam.

2 Results of loading tests of PC beam

Table 1 shows the specifications of PC beam, *Figure 1* shows loading condition and bond condition of PC bar. In loading tests, diameter, bond condition of PC bar and span of beam were selected as the test factor (Case 1: bond, Case 2: unbond).

Figure 2 shows Load - center span deflection relationship on Series A. Focusing on the maximum load, Case 2 is about 25% lower than Case 1. Therefore, it was found that the structural performance of PC beam was affected by bond condition of PC bar.

Table 1. Specification of PC beams

Series No.	Dia. of PC bar (mm)	Width (mm)	Height (mm)	Eccentric distance (mm)	Span (mm)
A	11	150	200	30	2200
B	11	150	200	30	1300
C	17	200	250	40	1600
D	23	200	300	50	1600

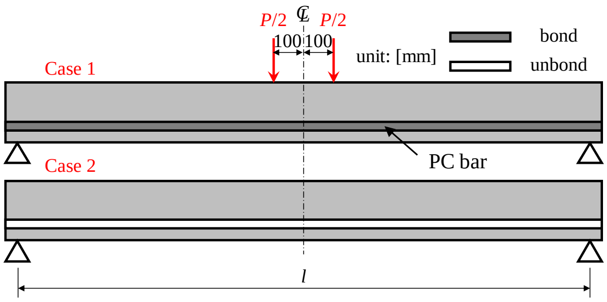

Figure 1. Loading condition and bond condition of PC bar

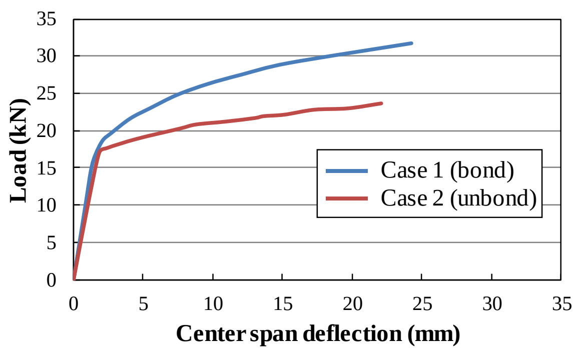

Figure 2. Load - center span deflection relationship on Series A

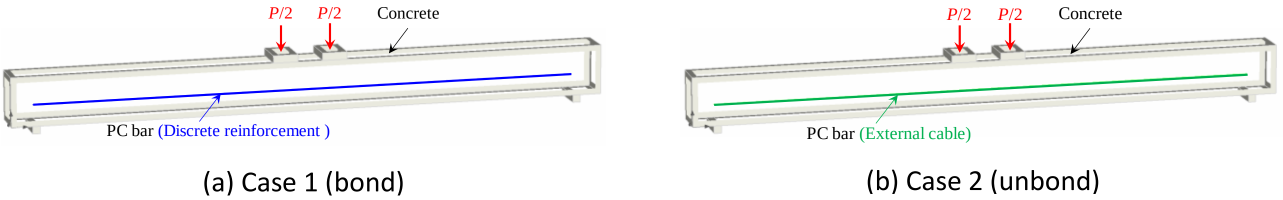

Figure 3. Modeling of PC bar in FE analysis

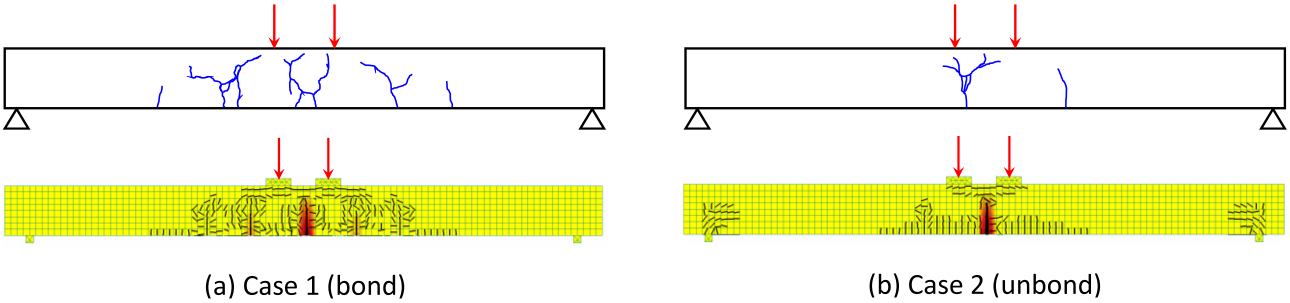

Figure 4. Comparison between analysis and experiment in crack patterns

3 Numerical simulation

3.1 Method of analysis

We have tried to simulate the structural performance of Series A. ATENA was used to conduct FE analysis. PC bar was expressed by using two kinds of line element as shown in *Figure 3*. By using these line elements, the influence of bond condition of PC bar was simply considered.

As to stress-strain relationship of concrete, CEB-FIP Model code 1990 is applied in compressive model. Linear model is used in compressive softening while exponential curve is used in tension softening.

Stress-strain of PC bar was adopted bilinear model. Prestress was directly expressed by introducing strain in line element.

3.2 Results of analysis

Figure 4 shows comparison between analysis and experiment in crack patterns, Figure 5 shows comparison analysis and experiment in load-center span deflection relationship. Crack patterns and maximum load estimated by FE analysis coincides with experimental results. Hence, it was found that the tendency of structural performance simulates if the bond condition of PC bar was expressed by using these line elements in analysis.

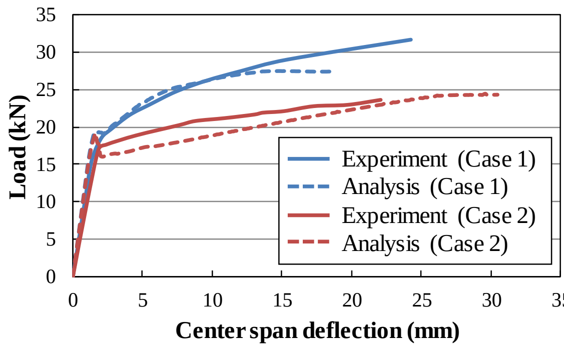

Figure 5. Comparison analysis and experiment in load - center span deflection relationship

4 Conclusions

If the bond condition of PC bar was used two kinds of line element, FE analysis succeeds to simulate experimental results of loading test of PC beam.

5 References

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Doctor Bridge 4.0: Advances in component-oriented analysis software for bridge structures

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Abstract

Doctor Bridge is an analysis software for bridge structures which is developed in China. The latest version released in September 2018, was a collection of innovative ideas and technologies, such as the component-oriented modeling technique, the cross-section data processing and the proposed application of standard library. Doctor Bridge 4.0 made breakthroughs in the graphic platform, the modeling technique, the computational core for mechanics, the distinctive solutions to bridge-specific mechanical problems and the support for current design specifications. Thus, it could reduce the difficulty in structural analysis, improve the efficiency of analysis for bridge structures and show a significant extensibility.

Keywords: Doctor Bridge; component-oriented; analysis for bridge structures; finite element method.

1 Introduction

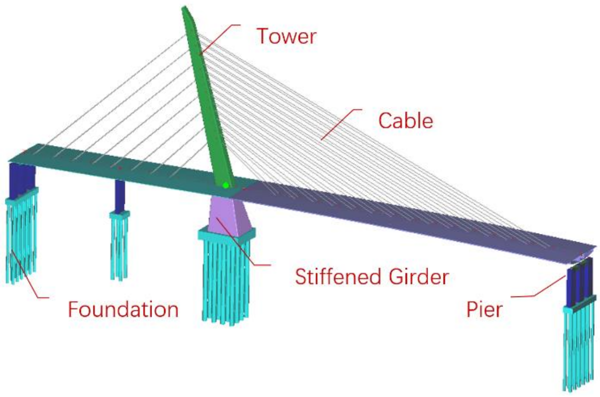
Compared with general finite element softwares, a specialized analysis software for bridge structures would give more consideration to professional experience and methods in bridge engineering so that it could analyze problems from the perspectives of engineers. This would bring greater convenience to the design and construction of bridges. In order to meet the needs of bridge design, the analysis of bridge structures during the construction process and the calculation of effects such as creep and live loads should be supported by the specialized analysis software for the bridge structure.

In September 2018, the latest released version of Doctor Bridge made breakthroughs in the graphic platform, the modeling technique, the computational core for mechanics, the distinctive solutions to bridge-specific mechanical problems and the support for current design specifications. The innovative technologies and ideas implemented in this domestically developed software include the component-oriented modeling technique, the cross-section data processing and the proposed application of standard library. In this article, we would like to give a brief on the core features of Doctor Bridge 4.0.

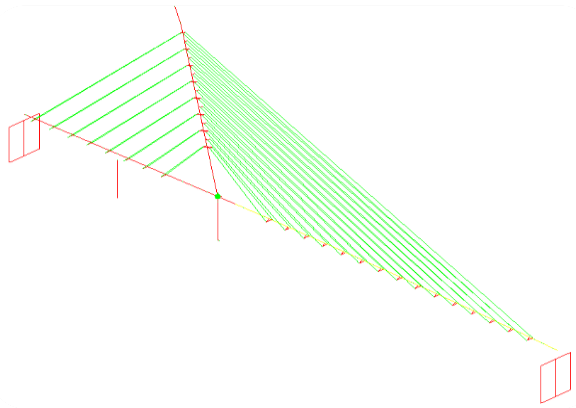
2 Modelling Technique

From the perspective of engineering practice, a modern specialized software for bridge structures should establish a set of methods including pre-processing, capacity checking, post-processing and results querying based on component-oriented modeling technique. Considering the characteristics of bridge structures, structural geometrical axes, sections, reinforcement details, as well as the construction process are set as objects in the program.

With the component-oriented modeling technique, users are able to set various attributes for structural components. Components can be independent, correlated, and even reused. According to the differences in material, functionality and location, a complete bridge can be divided into several separate components. These components are the basic elements of an entire bridge. They determine the function of the bridge all together. For example, a typical cable-stayed bridge can be regarded as possessing five parts: the foundations, the piers, the stiffened girder, the tower and the cables (see in Figure 1.a) and be created in Doctor Bridge 4.0 by the users. Then the finite element model could be built by the program instead of manual work (see in Figure 1.b).



a. Structural component model created by the users



b. The corresponding finite element model generated by the program automatically

Figure 1. The Process of the Component-oriented Modeling Technique

Many new techniques relating to the information input for component axes, sections, rebars and tendons have been developed and applied in Doctor Bridge 4.0 to improve the efficiency of data entry tasks significantly. Chapters, sections, figures, tables and references should be written in this way.

3 Finite Element Analysis

Doctor Bridge 4.0 is capable of performing nonlinear static and dynamic analysis and supports various element types, such as the Euler-Bernoulli beam element, the Timoshenko beam element, the truss element and the cable element. The seventh degree of freedom of beam elements could be considered to calculate the warping stress caused by restrained torsion.

4 Bridge-specific Mechanins

Doctor Bridge 4.0 provides all-around solutions to different bridge-specific mechanical problems. It supports the Hambly grillage method and provides techniques for the analysis of folding surface grid, space grid, bridges constructed with incremental launching method, and interlaminar shear forces in composite beams.

5 Design Specifications

Doctor Bridge 4.0 provides powerful component cross-section checking features which are in accordance with Chinese design specifications for highway bridges, railway bridges and urban bridges. The strength, stress, stiffness of concrete section and component pre-camber would be checked with the design specifications automatically.

6 Conclusions

As a software specialized in bridge structure analysis, Doctor Bridge 4.0, with independent intelligent property right, is a technical crystallization of the construction experience of Chinese bridges in the past two decades. Its innovative ideas and technologies would bring new insights and experience to the design of bridge structures. Doctor Bridge 4.0 takes a leading position in the graphic platform, the modeling technique, the computational core, analysis of bridge-specific mechanical problems and provides powerful support to current design specifications.

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Fracture mechanics based fatigue life prediction method for RC slabs in a punching shear failure mode

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Abstract

A fracture mechanics-based analytical method is proposed for the fatigue behaviors of RC bridge deck slabs which fail in an unexpected but widely observed punching shear failure mode under moving wheel loads. A concrete bridging degradation and an r/c interface bond slip degradation are accounted for as the source of the propagation of the critical punching shear cracks in the proposed method. With the obtained growths of the punching shear cracks, the fatigue life is predicted following a certain punching shear failure criterion combined with experimental observations. Method application is then conducted on an RC slab tested by the Civil Engineering Research Institute (CERI) for Cold Region, where a good coincidence between predictions from the proposed method and results from experiments as well as some empirical equations from statistically fitting experimental results confirms the reliability and applicability of this method.

Keywords: RC slab; Moving wheel load; Punching shear; Fracture mechanics; Fatigue.

1 Introduction

In last few decades, due to an overlook of a fatigue shear capacity of RC bridge deck slabs in previous design codes, a brittle and catastrophic punching shear failure has been usually reported. So far, extensive experimental, empirical, and numerical works have been conducted to uncover this failure mode with a moving wheel loading condition [1, 2]. However, the existing empirical life prediction equations were from statistically fitting experimental data which means the inner degradation mechanisms cannot be reflected. And the numerical approaches are very time-consuming. The computing time can even reach 1 day/cycle before failure.

Therefore, a fatigue analysis method for RC slabs subjected to moving wheel loads is proposed based on fracture mechanics and degradation mechanisms. With the same computing devices, the proposed method reduces the computing time to about 20 mins./cycle. Therefore, this proposed method provides a much more mechanism-based and efficient approach for investigating some key degradation behaviors for RC bridge deck slabs under moving wheel loads.

2 Problem simplification

According to a slab-to-beam cracking process and the punching shear failure mode observed in tests of RC slabs under moving wheel loads, empirical life prediction equations have been derived through fitting different sets of experimental data following an idea originally reported in [1]. In these equations, the punching shear capacity of a critical RC beam is the only parameter used, which means the fatigue life of the RC slab depends on the fatigue life of the critical RC beam. Therefore, in this study, the life prediction of an RC bridge slab under a moving wheel load is simplified into the life prediction of a critical RC beam focusing on the punching shear cracks.

3 Problem formulation

For the critical RC beam subjected to general loads, sectional stresses acting on a cross-section along the punching shear cracks are shown in *Figure 1*. Theoretically, after every loading cycle, if the effects from applied loads, concrete and rebar stresses are available, the cracking states can be calculated and the fatigue life can be predicted.

In this study, firstly, all material strains and the crack opening profile are expressed with *α*, *β* and *δ* (Crack Mouth Opening Displacement, CMOD) following three assumptions as follows: (1) Plane cross-section; (2) Cracked cross-section rotates around the neutral axis; (3) Linear crack opening profile. And then, stresses of all materials are expressed with *α*, *β*, and *δ* following appropriate material stress-strain and concrete bridging models. The concrete bridging and bond-slip degradations are included in the material models. To obtain *α*, *β* and *δ* after every loading cycle, one force equilibrium equation along the *x*-axis, one moment equilibrium equation, and one CMOD decomposition equations [3, 4] are established. Through solving the 3 equations, one can obtain *α*, *β*, and *δ*, which are then exploited in predicting the failure moment following a punching shear failure criterion with experimental observations.

4 Method application

In this study, the proposed method is applied to analyze an RC slab (C0) tested by the CERI for Cold Region [5]. *Figure 2* shows fatigue life predictions from the proposed method, some empirical equations, and the experimental fatigue life. It is found that the analytical S-N relation agrees well with the experimental fatigue life and is almost the average results of the four reported equations from statistical operations. The good agreements verify its applicability and reliability.

5 Conclusions

This paper proposed a fracture mechanics based analytical fatigue life prediction method for RC slabs which fail in a punching shear mode. The fatigue crack growth of the critical punching shear cracks was predicted and then exploited in predicting the fatigue life according to punching shear criteria with experimental observations. Moreover, method application was conducted on an RC slab tested in CERI for Cold Region, where a good agreement between the method predictions and experimental results and existing life prediction equations confirmed the reliability of the method. In addition, a time-saving characteristic makes this method extremely suitable for parametric studies and further purposeful design code improvement.

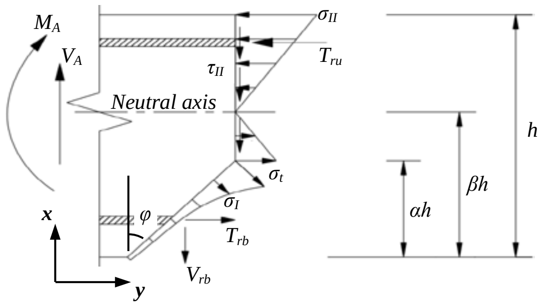

Figure 1. Sectional stresses and forces

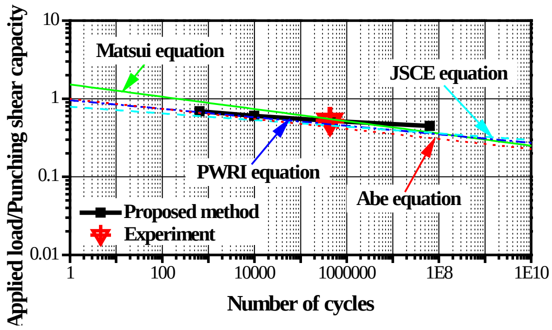

Figure 2. Fatigue life comparison

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