AUSTRALIAN NETWORK OF STRUCTURAL HEALTH MONITORING

Newsletter

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President Message

Tommy Chan

Professor in Civil Engineering, Queensland University of Technology

Dear All,
As many of you have noticed in early May, Humen Bridge, a bridge in southern China started shaking like waves due to strong winds on 5 May.

Figure 1 – A snapshot from a video reported by South China Morning Post dated 6 May 2020 to see how the bridge shakes like waves. (https://www.youtube.com/watch?v=VGjrZDNk_EQ)

According to *the Paper* (https://www.thepaper.cn/newsDetail_forward_7283720) dated 7 May 2020, Humen Bridge was closed for traffic starting from around 14:00 on 5 May and the vibration stopped later at around 16:00 but started to vibrate again at around 20:00 when there was no traffic on the bridge.
Because of the lessons learnt from the collapse of Tacoma Bridge in 1940 (https://www.youtube.com/watch?v=XggxeuFDaDU), such vibration in Humen Bridge draws a lot of attention in the social media and a lot of people considers that Humen Bridge will collapse just like the Tacoma Bridge.

Figure 2 Collapse of the Tacoma Narrows Bridge, Washington state, 1940. (https://www.britannica.com/technology/bridge-engineering/Tacoma-Narrows)

After the collapse of Tacoma bridge, engineers are more concerned about the aerodynamic instability of cable supported bridges under wind. Basically, there are four types of wind-induced vibration and aerodynamic instability problems and all these need to be considered in the design of a long cable supported bridge, which are: Vortex shedding excitations and Buffeting excitations, Galloping instabilities and Flutter instabilities. It is important to consider all these vibrations for the design of cable supported bridges and it will be even better if wind tunnel tests could be carried out to investigate and confirm the wind-induced behaviours of the structures. I also got involved in the wind induced vibration analysis for the design of the Stonecutter Bridge in Hong Kong. For Humen Bridge,

I do not have all the details about the bridge. However just looking at the videos via the internet. I suspected that the vibration was caused by vortex shedding excitation, which was due to the water filled barriers setup on the two side of the bridge which in turn largely affected the streamline of the bridge section. The management company of the bridge gave a similar explanation that the bridge's aerodynamic properties changed after the water filled barriers were placed along both sides of the bridge as they were replacing hangers of the bridge (https://www.globaltimes.cn/content/1187605.shtml). After the barriers were removed and together with a thorough study of the data acquired by the Structural Health Monitoring system installed on the bridge, the bridge was considered safe and the bridge was re-opened on 15 May 2020. Once again it could be seen how important it is for a SHM system which can help provide all the important data for the investigation of this abnormal vibration for its mitigation and the structural safety assessment of the bridge. It was noticed that during the serious vortex vibration caused by changing the aerodynamic properties of the bridge, the SHM system detected that the magnitude of the vibration is under 20 cm. It may look alarming and yet the bridge is designed from a deflection ratio of 1/400. The span length of Humen Bridge is 888m which allows a deflection up to 2 m. Hence the SHM system shows the vibration is within limits.
An effective SHM system installed on a structure could provide data to check various health indexes of the structure as well as providing data to ensure it is performed as designed. The example of Humen Bridge incident demonstrates well how the SHM installed could provide the necessary data for the bridge management team to make a timely decision to take any traffic control measure and take any action to prevent any more serious bridge incident happens. Also, it helps the authorities to determine what actions need to be taken and when the bridge can be re-opened.
Vortex induced Vibration (VIV) is not uncommon for bridge structures. The following table shows three examples demonstrating Vortex Induced Vibrations on bridges.

Name	Where	Type	Incidents	Video/Reference
Rio-Niterói Bridge	Brazil	A twin box girder bridge with central 848-m-long steel structure (74+200+300 +200+74)	Opened to traffic in 1974 and a major VIV happened in August 1980 during a storm.	https://doi.org/10.1016/S0167-6105(99)00108-7
Volgograd Bridge	Russia	Concrete Girder Bridge	Opened to traffic on 10 Oct 2009 and closed on 20 May 2010 due to abnormal vibration and reopened on 25 May 2010. Twelve semi-active tuned mass dampers were installed in Autumn 2011 to suppress the vibration.	https://www.youtube.com/watch?v=DMbTVVs1zQA

Trans-Tokyo Bay Crossing Bridge	Japan	A straight ten-span continuous steel box-girder bridge with three cells in the cross section and the two longest spans are 240m each.	Opened in 1997 and the maximum vertical amplitude of vibration could be larger than 50 cm for the longest spans (240m). After installing Tuned Mass Dampers, the amplitude could be reduced to 5-6 cm.	DOI:10.1061/(ASCE) 0733-9445(2002)128 :8(1012)

Ten days before the incident of Humen Bridge, another long span suspension bridge in China, the Wuhan Parrot Island Yangtze River Bridge also exhibited wave-like vibration on 26 April 2020. Like Humen Bridge, there is also a SHM system installed on the Wuhan Parrot Island Yangtze River Bridge. There are more than 200 sensors installed on the bridge to acquire the wind speeds, directions, deflections and stresses. The system could effectively alarm the management team about the abnormal vibration of the structure on 26 April. According to real time health indexes provided by the SHM system, the management team could quickly decide the bridge is safe for operation.
It can be seen how an effective SHM system could help the authorities to make a timely decision as well as collecting useful data for further studies of unusual structural behaviours to improve future design. It also can be seen that the two incidents were happened during the time of COVID-19. Without such a SHM system, the management team could find it difficult to investigate the causes and control such vibrations and make a right decision. These two incidents well demonstrate how SHM could help even during the time that we are so concerned about social distancing. I still remember I can collect a lot of valuable data to study wind-structure interaction for cable supported bridges and published several good quality journal papers on this kind of studies[1]. Just imagine without a SHM system, how could a research team collect the field data of a structure when the Typhoon Signal 10 (the highest) is hoisted?

Below are the updates of the month.

ATCSI Proposal

[1] For example, Chan, T.H.T., Li, Z.X., and Ko, J.M. (2004), “Evaluation of Typhoon Induced Fatigue Damage using Health Monitoring Data for the Tsing Ma Bridge”, Structural Engineering and Mechanics, Vol.17, No.5., pp. 655 – 670; and Li, Z.X., Chan, T.H.T. and Ko, J.M. (2002) "Evaluation of typhoon induced fatigue damage for Tsing Ma Bridge" *Engineering Structures* Vol. 24, pp. 1035-1047.

We are still awaiting the announcement of the outcome of our proposed ARC Training Centre for Smart Data Driven Next Generation Infrastructure (ATCSI). As mentioned in the last updates, according to the ARC page, its anticipated announcement is stated as 2nd quarter of 2020, so we may be able to know the result by end of the month. I really look forward to having this training centre established to enhance our national safety and efficiency of infrastructure, and ability to drive growth, productivity and competitiveness, as well as to train more professionals to meet the increase demand of the SHM technologies.

Research Collaboration - Building 4.0 CRC & SmartCrete CRC

In the last monthly update, I mentioned about the research collaboration opportunities under Building 4.0 CRC. In our last EC meeting, we had some discussion on how we could engage individually as well as collaboratively in this CRC. In the discussion, we consider Programs 2 (digital transformation) and 3 (building transformation) are more relevant to ANSHM activities e.g. digital platform, digital twin, IoT, data-driven simulation, new sensors for construction safety; data analytics for improving building energy, especially building defects. Tuan will keep ANSHM informed and explored opportunities.

Beside Building 4.0 CRC, we should also take note of SmartCrete CRC, which was also announced recently by the Hon Karen Andrews MP, Minister for Industry, Science and Technology receiving $21M in Australian Government grant funds (https://www.smartcretecrc.com.au/). It also has 3 Research Programs as follows:

I. Program 1 – Engineering Solutions
II. Program 2 – Asset Management
III. Program 3 – Sustainability, Environmental & Disposal

Hence it could be seen that there are a lot of collaborative opportunities for us in the area of SHM to get involved in these two CRCs. Having said that we should also understand that CRC is led by industry so it depends on whether the research idea can fit their interest. Also, I appreciate so much that the significance of SHM has been realised more and more by the industry as indicated by these two recently awarded CRCs. However, in these two CRCs, SHM is only a part of their programs and not their main focus. Our proposed ATCSI is wholly on developing and implementing SHM technologies for smoother operations and lower maintenance and rehabilitation costs; improved design and construction; and enhanced safety and performance. Besides, it will train a new generation of professionals to meet the increasing demands of applying SHM technologies. Therefore, these three centres will be complementary with one another. As mentioned in the last update, I really hope that our proposed ATCSI will also be successful so that these three centres could generate a synergistic effect.

ANSHM Page (www.ANSHM.org.au)

Migration of the server for hosting ANSHM webpage to the Squiz Matrix platform has been successfully completed. The platform is being looked after by QUT team and yet we have checked that we should be able to edit the contents and add new pages. All the required functions of the page on the new platform have also been tested. Many thanks again for Hong Guan and her team for making this migration so successful.

ANSHM Special Issue in IJSSD

We are approaching the completion of the review process for this special issue. Only a few papers are at its final stage of their review processes and all the other papers have completed their review processes. It is expected that we should be able to inform Prof CM Wang, the chief editor of IJSSD about the completion by mid-June and the proposed date of publication will be informed in due course.

Publication generated from the 11th ANSHM Workshop

We have a very good discussion in our last EC meeting about this publication. We consider it's about time to resume the preparation for this publication. We consider making this publication as a monograph to celebrate our 10th Anniversary and showcase our achievements in these 10 years. We plan to use this publication to help engineers and young researchers to understand some basics topics in SHM as well as introducing some latest advancements. Hong proposed six potential topics to be included and nominated a lead for each topic. She is seeking feedback from the EC members regarding her proposal. Then she will work with Jianchun and myself to finalise the topic and formulate a writing plan for the publication.

Mini-Symposium (MS26) in SHMII-10

Since the Organising Committee of SHMII-10 would like to go ahead with SHMII-10, which will be held in Porto, Portugal, 30 June to 2 July 2021, www.fe.up.pt/shmii10, we decided to organise the Mini-Symposium (MS26, a forum for ANSHM members; SHM researchers and practitioners in Oceania) as originally planned. A reminder was sent by Andy to ANSHM and ISHMII ECR-Oceania members about the deadline for submitting an abstract to SHMII-10. Abstract submission by the deadline of 30 June to MS26 should be made directly in the conference webpage. Please visit https://web.fe.up.pt/~shmii10/submission/abstract-submission/ and select the theme no. 26.

12th ANSHM Workshop

It is quite challenging for us to organise our annual workshop this year, as there are a lot of uncertainties caused by COVID-19. As we have decided to have our important annual event to be held in Sydney this time, Jianchun will explore on having the booking of venue first and then we could decide later, say in July when the situation is clearer. He will also explore the best time for holding this workshop.

Engagement with Industry

We are so pleased to have John Vazey to attend our last EC meeting as an Advisory Board Member. John is our Internal Affairs and Industry Coordinator. He is so keen to get more ANSHM members from the industry to get involved in ANSHM activities, e.g. coordinate with other members from the industry to contribute articles to Newsletter, identify industry-oriented topics for webforum and ANSHM technical workshops. He has made a lot of good suggestions like proposing secondment plans for postgraduate students working in the industry and we will investigate all that to see how these could be consolidated.

In the next sections, we will have two articles from our members. The first article provides a technical note on railway bridge transition defects and maintenance on Australian heavy haul operated tracks from the researchers from Western Sydney University and the second article introduced the N79 Living Laboratory Project at Griffith University Enabling Structural Health Monitoring.

With kind regards,

Tommy Chan
President, ANSHM
www.ANSHM.org.au

Professor Tommy H.T. Chan PhD, ThM, MDiv, BE (Hons I), FHKIE, MIE Aust, CP Eng, NPER, MICE, C Eng, RPE, MCSCE
President ANSHM (www.ANSHM.org.au)
School of Civil & Environmental Engineering, Queensland University of Technology (QUT)
GPO Box 2434, Brisbane, QLD 4001, AUSTRALIA.
Ph. +61 7 3138 6732; Fax. +61 7 3138 1170; email: tommy.chan@qut.edu.au;
Research profile | **Research publications** | **Google Scholar citations**

A Technical Note on Railway Bridge Transition Defects and Maintenance on Australian Heavy Haul Operated Tracks

Ralph Zhang, Helen Wu and Richard (Chunhui) Yang
School of Engineering, Western Sydney University
E-mail: 17602059@student.westernsydney.edu.au; Helen.Wu@westernsydney.edu.au; r.yang@westernsydney.edu.au

Abstract

Poor track conditions at the bridge/track transition area, especially for the non-ballast steel bridges has been identified by the railway companies and authorities for decades. The defects include significant loss of top and long twist at bridge transitions and across the ballast top under-bridge, track slack presented at the transition from embankment formation to ballast top formation at the bridge abutments and highly fouled ballast. These defects are generated from bridge transition and soft soil track formation. In this article, classic bridge transition defects are reviewed, and long term and short team remedial solutions are discussed accordingly.

KEYWORDS: railway bridge, transition, defects, maintenance, heavy haul

Introduction

Bridge/track transition condition problems, especially for the non-ballast steel bridges, have been acknowledged by the heavy haul revenue railway companies and authorities for decades and some specified remedial solutions were developed to solve them [1]. In recent years, track deterioration and high frequent track maintenance of ballasted concrete bridges were frequently reported at their track-bridge transition area. Poor track conditions at the bridge/track transition areas lead to the defects including significant loss of top and long twist at the bridge transitions and across the ballast top under bridge, track slack presented at the transition from the embankment formation to the ballast top formation at the bridge abutments, and highly fouled ballast. This article focuses on the reviewing the defects and their causes and solutions are discussed with using information collected from in-field studies.

Track Defects at Transition Areas

Rail track transition points occur in substructures between embankments, bridges and tunnels. From the track maintenance point of view, the transition areas in the railway networks are the locations where have significant changes in the vertical rail support, i.e. its stiffness variation. The significant stiffness variations at these transition areas as illustrated in Fig. 1 will generate high dynamic loads, and these dynamic loads will increase the deterioration of the track components.

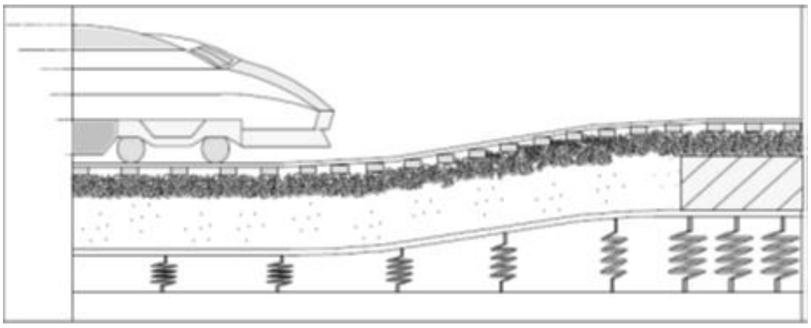

Fig. 1 Principle of train responses at a transition

Track Geometry Issues

Significant track geometry defects, especially the track geometry defects of top, alignment and long twist, are widely existed at the transition area between the steel bridges and their adjacent tracks, as shown in Figure 2.

Fig. 2 Transition between embankment and steel girder bridge

Figure 3 shows a defective track transition from a high embankment onto a bridge [2]. At this location, the problems are arisen from the rapid change of the track support stiffness. In addition, the "pumping action (vertical dynamic movement)" can be observed, when trains pass through this transition area. Moreover, the repeatedly-wet track beds and voids below sleepers were developed as the embankment settles. The continuing differential settlement required high frequently track maintenance work (track resurfacing - lifting and packing) to enable safe operation and temporary or permanent speed restrictions were set up at these locations.

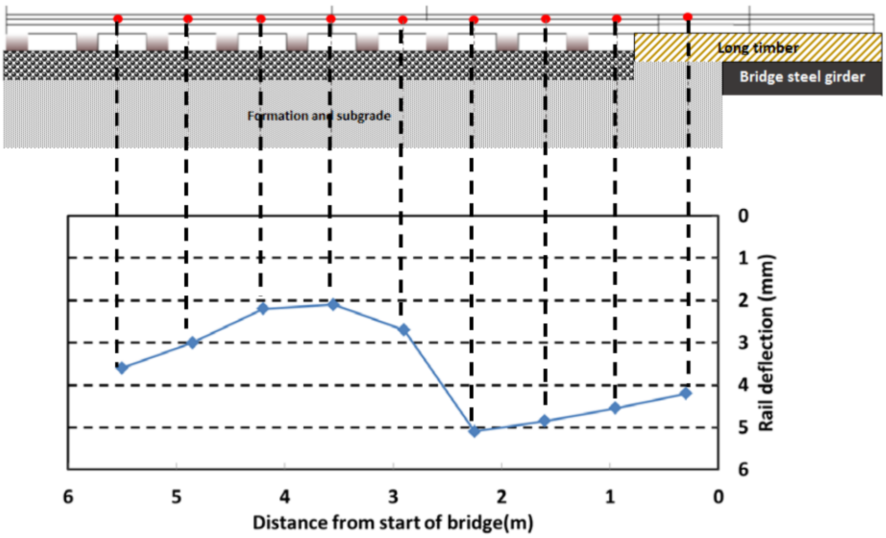

Fig. 3 General arrangement and maximum dynamic displacement at the approach to an overbridge

<u>Track Settlement and Subgrade Issues</u>

Similarly, to the transition issues happened at the bridge approach zones, the track settlement and subgrade issues also exist at the bridge approach section. However, for the practices of track maintenance, this type of issues is usually used to describe those happened between the plane track and a concrete ballast top bridge, which has much less track stiffness difference compared with the plane track and steel bridge. Regarding to the plane track to concrete ballast top bridge transition zone issues, there is a pronounced track slack present at the transition from the embankment formation to the ballast top formation at the bridge abutments. The track slack at the bridge transitions is likely to be due to settlement of the poor-quality fill under train loading and the presence of a significant and abrupt change in track modulus from a low modulus track formation to a ballast top structure.

Regarding to track geometry and maintenance at this location, the track geometry inspection car's records indicate: on both Up and Down lines - significant loss of top on Up and Down rails at the city and country bridge transitions (within 5 m of bridge) and across the ballast top underbridge. Significant (Maximum exceedance 4) long twist is present at the approach to the underbridge and occurs in the centre of the underbridge. Visual assessment of track condition along the up and down lines confirmed the inspection car results with a pronounced track slack present at the transition from the embankment formation to the ballast top formation at the bridge abutments. Survey measurements along the up line up rail at the city transition noted a drop-in rail level of 37 mm from the transition over a distance of 3m to the city (1 in 80 slack) compared to the 1 in 220 grade across the bridge. Selected site photographs showing site conditions are presented below.

<u>Short Spot Track Dynamic Response (Pumping Actions from Impact Load)</u>

The short spot track dynamic response issues are used to describe the significant dynamic vertical impact action between the wheel and track that happened in a very short distance (usually no longer than 2m along the railway track). On Australian heavy haul operated tracks, these short spot track dynamic response issues are happened at the concrete support logs of the non-ballast steel girder bridge ends. For many of the non-ballast steel bridges, the distance between the last concrete sleeper on the plain track and the first timber transom on the steel bridge is significantly longer than the standard sleeper spacing. When the trains pass these locations, the poor support from the larger sleeper-transom spacing is to be combined with the significant differential track stiffness and results high vertical dynamic impact action. As a result, track settlement, bog-hole, track formation failure and severe ballast deterioration are generated at this area.

Deterioration of Ballast & Drainage

The dynamic impact load and differential track stiffness generated high ballast pressure underneath the concrete sleeper and pumping action, increasing the deterioration of the ballast material and the failure of track formation. At the plain track and bridge transition areas, the geotechnical investigation through the excavation of test pits (to depths ranging between 0.65m to 1.8m below top of rail level). Dynamic Cone Penetrometer testing (DCP) is utilised to assess soil strength at subgrade level. Perched water was observed at 1.35m to 1.45m below rail level in the ash and in the structural fill layer in test pits. This groundwater level is well above the embankment toe level and suggests the presence of perched water most likely trapped in a dished ash and structural fill profile beneath the coal lines.

Practical Remedial Methods at Transition Areas

Dynamic interactions between train and track are influenced significantly by track stiffness variations and can be improved by adjustments in the track stiffness. To optimise the track stiffness, the first step is to obtain the required stiffness of the track. It is generally predominated by a requirement of the maximum deflection of the rail. For the heavy haul freight predominated railway tracks, the value in North America is no more than 5mm. It is varied under different track revenue conditions:

- Higher axle loads and/or speed needs a stiffer track, due to higher dynamic forces;
- A stiff track structure leads to a higher load at each fastening point and on the rail;
- A soft track distributes the load over more fastening points but is less stable.

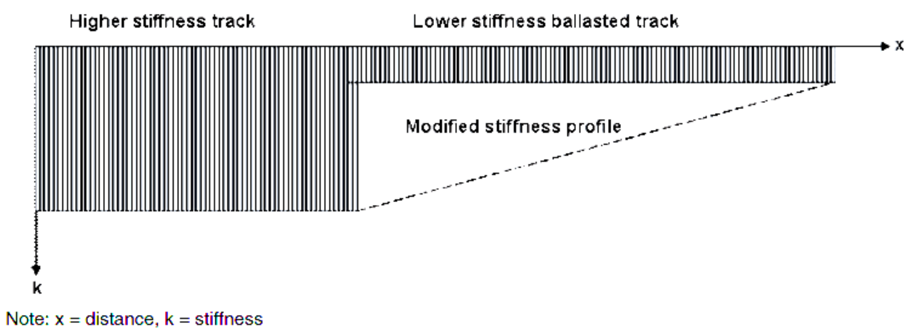

Fig. 4 Transition remedy in which the stiffness step change is modified with a gradual increase in stiffness

Theoretically, the solution is sought to reduce the stiffness on the stiffer track structure and foremost to build a smooth transition zone to the track with lower stiffness. The most widely used technique to deal with this problem is to smoothen the stiffness/modulus step change at the interface by gradually increasing stiffness on the lower stiffness side of the transition [2], see Fig. 4.

Full Depth Track Reconditioning

Track reconditioning work is required as the remedial action for the track formation failure at the transition zone, especially at ends of a steel girder bridge. Usually the reconstruction work needs to be conducted during a 72-hour possession. For the heavy haul operated railway tracks in Australia, the site-specific scope of the bridge end track reconditioning work involves:

- Removal of ballast, capping and subgrade materials to achieve design excavation levels;
- The installation of a strengthened transition zone to both the city and country ends of the steel underbridge.
- Geogrids to be Tensar SS20 (Terragrid TG3030) or equivalent, Geofabric to be Bidim A34 or equivalent. The agricultural drain shall comprise 225mm class four pipe (225mm slotted ADS N-12 ST IB Pipe), wrapped with geofabric and graded at 1V:30H to a suitable of the shelf headwall which freely discharges;
- The formation outside the bridge approach area is installed at a standard depth of 1.4m BTOR, unless directed otherwise by railway authorities' Geotechnical Representative;
- Capping quality material shall be used for all bridge approach formation;
- Refer to earthwork guidelines near structures on railway network;
- The track resurfacing work is must to be occurred after the new transoms on the steel girder bridge have been installed and secured down, and the resurfacing work on the plain track must to suit the transom design alignment over the steel girder bridge and is to be started away from the bridge abutments on the two ends of the bridge.

Additional Fastening System on the Concrete Log

The short spot track dynamic response which is caused by the large distance between the last concrete sleeper on the plain track and the first timber transom on the steel bridge can be mitigated by installing the Delkor pad lift on the concrete support logs of the non-ballast steel girder bridge ends.

The ALT.1 Delkor plates on the abutment / ballast log is required to better allow rail support under train loading. This is to be done using an appropriate HDPE packer, to be placed between the ballast log and the ALT.1 plate. A gap of 4 ± 1 mm between the plate and the foot of rail is to be achieved.

Undercutting/Cleaning Ballast Profile + New Geo-Synthetic

The remedial actions include undercutting/cleaning the ballast profile and strengthen the track transition area by using a special type newly developed geo-synthetic - Tracktex. It has been created to address the track and ballast deterioration problem caused by "Erosion Pumping Failure". This method is designed to address the transition problems where site is located on a broad flat low-lying area associated with the floodplain of the river, where are the sensitive wetland swampy areas [5]. To deal with the poor ballast condition on the concrete bridge, bridge transition, and adjacent track the full cross-section undercutting (300 mm below the underneath of sleeper) is to be applied instead of the excavation that can reduce the risk of encountering groundwater in excavation to design subgrade level (Fig. 5). Tracktex is featured as a composite material consisting of a unique micro-porous filter, sandwiched between two high strength protection geotextiles, as shown in Figs. 5 and 6.

When installed at the ballast formation interface, Tracktex prevents rain water penetrating through to the underlying deposits whilst under load, allowing a controlled upward movement of water through capillary action and filtering and retaining any fine soil particles.

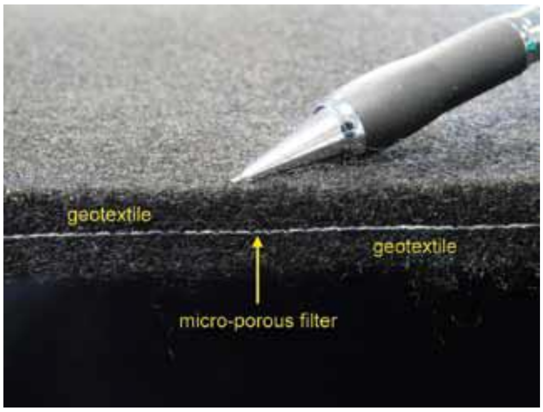

Fig. 5: The concept of Tracktex Geosynthetics

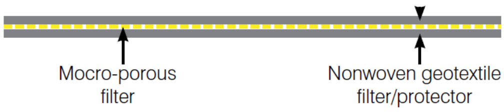

Fig. 6: The details of the cross-section of Tracktex Geosynthetics

Furthermore, to prevent the ballast damage the Tracktex, the Tensar SS40 geogrid is to be installed above Tracktex. In addition, track damping is an important consideration for ride quality and track component life. To decrease the damping action to bridge transition area, Tracktex together with Tensar SS40 geogrid that installed at the undercut bridge transition is used to provide acceptably smooth bridge approaches. This ramp can provide a smoother transition from bridge to open track.

Ballast Glue

Ballast glue/bond at the bridge/track transition area has shown its capacity to improve the performance and effectiveness at the bridge end area. End of the bridge was instrumented with accelerometers to measure vibration responses. The bridge is a transom-top on the steel girders. The track configuration consists of 60kg rails, standard guard rails, Pandrol fastening system (fast-clip type), low profile medium duty concrete sleepers, ballast bed and formation. Both ends of the steel girder bridge were resurfaced by tamping machine. During a 40-hour possession, the bridge ends were lifted and adjusted to the alignment by the tamping machine. Then, the ballast glue/bond material was applied to both bridge ends. The MC-ballastbond 70 was used. The glue is varied about 250-300 mm deep from the top surface of ballast, resulting in about 70 - 100mm underneath the sleeper soffit (250-180 = 70mm). In addition, from the technical specification it is stated that the ballast-bond material has a structural design life of 50 years.

Post the inspection the vibration measurements were performed to examine the track condition after ballast gluing. It was observed that the glue did not seepage much through the voids of ballast aggregates, even though the ballast had recently been lifted and tamped. Vibration measurements were carried out at two stages, initial condition (pre-ballast glue), and after condition (post ballast glue). The tests will allow benchmarking the dynamic performance of the bridge ends and evaluation of the effectiveness of the ballast glue method. The comparison of averaging peak vibration magnitudes at the bridge end shown that the ballast glue changes the dynamic track characteristics [6]. The maximum amplitude of vibration of rail at the bridge end tends to significantly increase due to ballast adhesion, whilst the vibration amplitude of sleeper at the bridge end surprisingly remains at the same level. It is noted that the rail vibration levels at the bridge end are in a similar range with those over the bridge. This shows that the track stiffness at the bridge end increases significantly, which consequently increases the wheel-rail dynamic interaction.

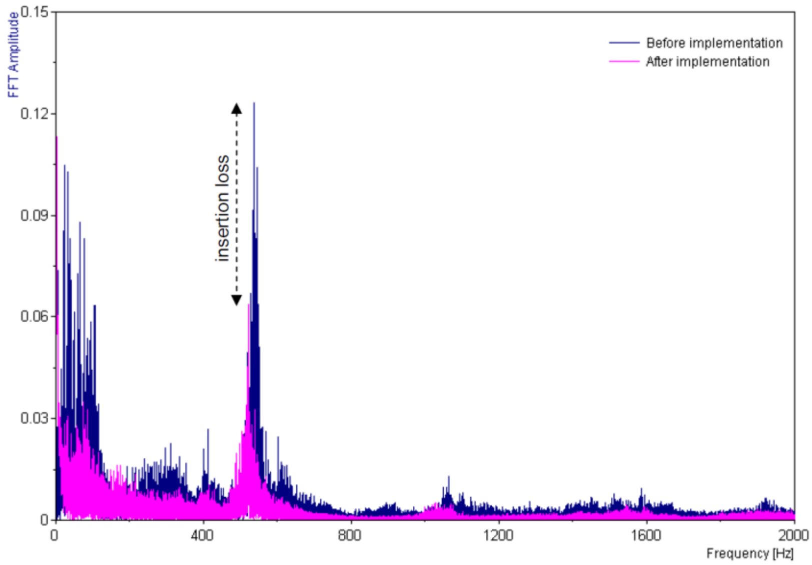

Fig. 7 Comparative frequency responses of bridge end - Sleeper's peak vibration at bridge end

It is noticeable that the sleeper is damped by the ballast cohesion. The vibration suppression level is very high as the dynamic effect of larger lumped mass attached to sleeper is pronounced. Also, the duration of sleeper's vibration spectra is shortened due to ballast glue (Fig. 7). Based on the vibration test measurements, it is evident that the ballast glue can help suppress the vibration of sleepers and ballast at the bridge approaches in a short term. Track inspection vehicle data shows that the dynamic rail deflection deviation is improved by about 30% in a short term. The linear regression prediction of track settlements shows that the ballast glue will strengthen the bridge ends and reduce the level of track settlement overtime.

Concluding Remarks

The transition from ballasted track forms to non-ballasted track forms often causes a high rate of track deterioration due to the dynamic impact excitation by the abrupt change of track stiffness. This results in accelerated rates of track geometry and component degradation, high maintenance need, poor ride comfort, and high fatigue stress threshold for passing rolling stocks.

These issues represented as the bridge transition defects are including: poor track geometry, significant track settlement and formation/subgrade failure, deterioration of ballast and drainage, and short spot track dynamic response (pumping actions from impact load), etc.

From the point of view of structural mechanics, the significant dynamic and impact loads are caused by the high differential of track stiffness at these track/bridge transition areas. A criterion for the stiffness transition is to smooth the stiffness interface between the dissimilar track types.

Feasible options are to design the transition to:

- Option 1: Equalise the stiffness and rail deflection of the ballasted and non-ballasted tracks, by controlling the resilience of the rail on the non-ballasted track;
- Option 2: Provide a gradual increase in the stiffness of the ballasted track to match that of the non-ballasted track.

Regarding to the option 2, within Australian some successful and effective methodologies and maintenance practices have been developed and introduced. These including: specially designed full depth track reconditioning, install the Delkor pad on the concrete support logs of the non-ballast steel girder bridge ends, undercutting/cleaning ballast profile plus the installation of new geo-synthetic, and ballast glue at bridge end. All of these methods have shown its effectiveness and/or portantial to mitigate the problem.

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N79 Living Laboratory Project at Griffith University Enabling Structural Health Monitoring

Sanam Aghdamy[a], Hong Guan[a], Dominic Ong[a], Cheryl Desha[a], Andy Nguyen[b]

School of Engineering and Built Environment, Griffith University[a], School of Civil Engineering and Surveying, University of Southern Queensland[b]

Emails: s.aghdamy@griffith.edu.au; h.guan@griffith.edu.au

In 2016 the Griffith University Council agreed to add a new building to the University's Nathan campus, to meet an increasing demand for engineering and built environment facilities across programs in engineering, architecture, design and aviation. The "ETA" (*Engineering, Technology and Aviation*) building, Building N79, was completed in October 2019 and now houses innovative research, learning and teaching facilities for the University's School of Built Environment and Engineering.

N79 Building is a six-story reinforced and prestressed concrete structure. Fig. 1 shows the building elevation view. The whole building is designed to act as a 'Living Laboratory' where more than 30 sensors relay information about the building's energy, water, and structural performance in real-time.

Fig.1. N79 Building

The long-term structural health monitoring (SHM) system has been installed at sub-surface and surface levels. The electronic sensors, which were placed at sub-surface level, can capture environmental effects including rainfall, temperature and groundwater fluctuations within the design life of the sensors. The surface level sensors, which are in the process of being installed on the building, will capture how the building responds to short-term and long-term loading effects, wind, shrinkage, creep, laboratory machineries induced vibrations and so forth.

Sub-surface Sensors

Two vibrating wire piezometers, *VWP*, with capacity limit up to 350kPa and one observation well, *OW*, were installed at a depth of about 6m below the surface of the slope on an easily accessible green-field location immediately across the road (Western Creek Road) from the N79 building (see Fig. 2). These locations were selected so that the piezometers could measure the actual ground water table fluctuation over different environment/weather conditions. The installation process of the piezometers and observation well is illustrated in Fig. 3.

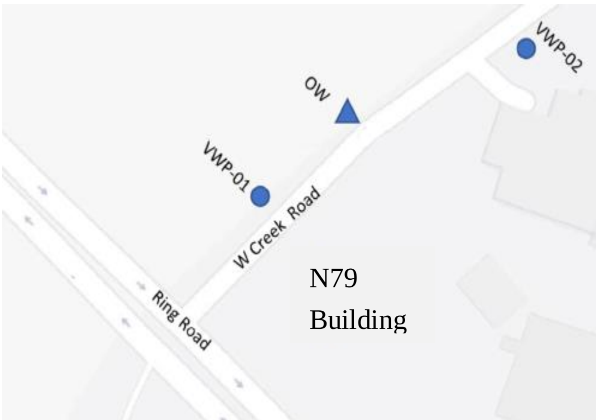

Fig. 2. Locations of vibrating wire piezometers and observation well

Fig. 3. Installation process of vibrating wire piezometers and observation well

Piezometers were embedded in the ground in pre-drilled holes. The piezometer tips were encapsulated within a section of porous sand. The top and bottom sections of the encapsulation were sealed in situ by cement-bentonite mix placed around the annulus of the oversized drilled hole to ensure that the sensitive tip only measures the water pressure generated by the groundwater table and not from other seepage sources.

Seven vibrating wire earth pressure cells (with capacity limit up to 1,000kPa) were installed underneath five footings (see Fig. 4) of the N79 building to measure the bearing pressure of the founding material (soil or rock), the carrying capacity of which can be affected by the ground water table fluctuations. Earth pressure cell, measuring about 30cm in diameter with a flat, thin face, was typically embedded in specially prepared soil bedding in between the in situ founding material (soil or rock) and the building's reinforced concrete footing (see Fig. 5). It must be placed face-down on the founding material, preferably using the axial-mounted type to maximise contact pressure.

A wireless tipping bucket rain gauge was installed on the rooftop of N79 to collect rainfall data, which would be used to develop understanding on the behaviour of the groundwater fluctuation readings measured by the piezometers. The location on the rooftop was chosen to ensure that the rain gauge will be unobstructed and free from tree covers.

All the above-mentioned sensors are connected respectively to vibrating wire interfaces (with radio modules) thus enabling wireless communication with the data logger (model CR1000X-AN-ST-SW). This is to ensure that the building and environmental data is reliably and efficiently collated and stored in the University central data collection centre managed by the Facilities department.

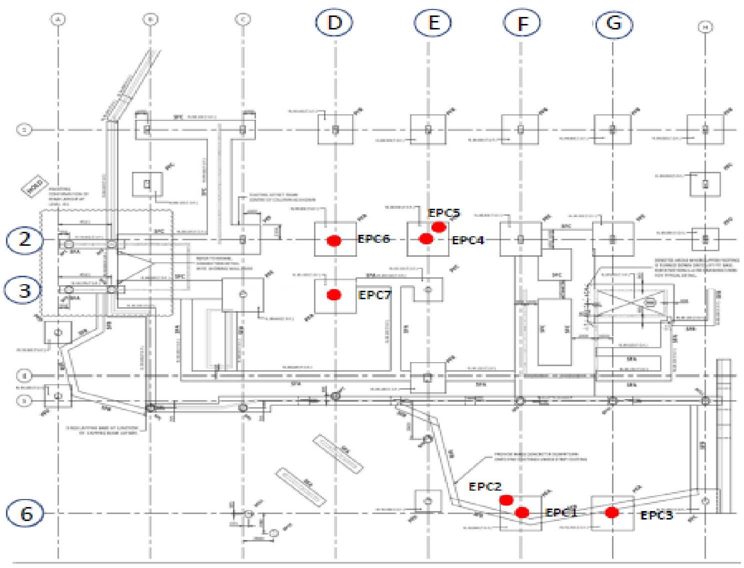

Fig. 4. Locations of the vibrating wire earth pressure cells

Fig. 5. Installation process of vibrating wire earth pressure cells underneath building footings

Surface Sensors

Various types of sensors, including accelerometers, strain gauges, outdoor temperature and humidity sensors, wind speed and direction sensors are being installed on N79 Building at/above the surface level to continuously acquire its structural vibration responses and environmental conditions.

Fig. 6. A single-axis accelerometer installed on one of the reinforced concrete columns

The single-axis accelerometers (Bestech Wilcoxson (Model ID. 731-207); Nominal sensitivity of 10V/g) are being installed on the building to monitor the global vibrations of the whole structure. Fig. 6 shows a photo of the installed single-axis accelerometer on one of the reinforced concrete columns. Additionally, Fig. 7 shows the location of the twelve accelerometers. As N79 building is obviously not a slender building (see Fig. 7), such high-sensitivity (*i.e.*, 10V/g) piezoelectric accelerometers are deemed necessary to pick up small vibration responses generated under ambient excitation conditions. Given the semi-rectangular shape of the building, the accelerometers will be positioned on its three corners (North-West, North-East, and South-East) (Fig. 7). Whilst there will be one single-axis accelerometer on the North-West and South-East corners of the building on the first, third and fifth levels, there will be two single-axis accelerometers on the North-East corner of the building on the first, third and fifth levels (*i.e.*, 12 accelerometers in total).

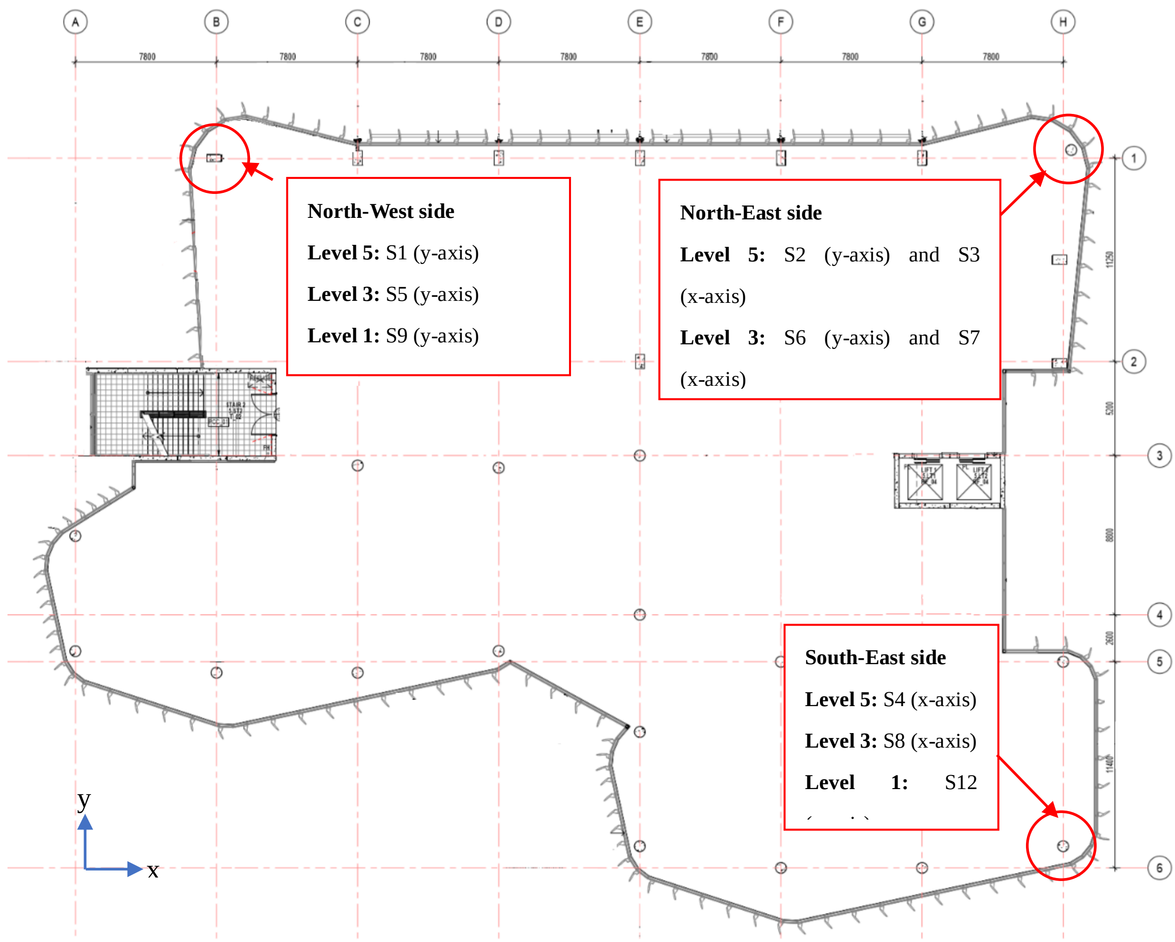

Fig. 7. Location of accelerometers on N79 building

A total of 15 strain gauges (PCB Piezotronics (Model ID: 740B02)) are being installed on the building to measure: (1) concrete creep and shrinkage, (2) structural vibration due to people movement, (3) structural vibration induced by machineries, (4) gantry beam deflection due to crane roll over, and (5) deflection of the building's facade due to wind gusts. The use of piezoelectric strain gauges is to ensure that instantaneous strains under dynamic loads can be captured to enable live monitoring observations. These strain gauges are being installed on Level one. Two of these strain gauges are allocated to measure concrete creep and shrinkage. Each of these will be installed on a column close to the underside (bottom face) of the slab (i.e., ceiling of level one). Given the higher level of axial load on Level one compared to the upper levels, positioning these sensors on this level would maximize the opportunity to obtain more reliable and meaningful results. Two strain gauges are allocated to measure the local vibration induced by people movement. Each of these are being mounted to the underside of slab of Level two (*i.e.*, ceiling of Level one). These sensors are being installed on longer span slabs exhibiting larger deflection to maximize the opportunity to obtain pronounced and meaningful results. There are two strain gauges to measure local structural vibration induced by machinery. These sensors will be mounted underside of slab of Level two (i.e., ceiling of level one), where the material testing laboratory is located. The crane gantry beam is located in the high-bay area of flexi-laboratory. To measure the beam's deflection due to the gantry crane roll over, two strain gauges are being installed underside of the beam. Whilst one of them is being installed at the mid-span the other one is being installed at one third of the beam span from one of the supports. Two strain gauges are allocated to measure the deflection of the facades at the high-bay area of flexi-laboratory (Level one) due to the wind gusts. These strain gauges will be mounted on the backside of the aluminium header. Finally, four strain gauges will be mounted on the upper side of the walls on Level one to measure the deflection due to the wind loads. Since the other strain gauges will be located on Level one, these four strain gauges have been chosen to be on the same level for convenience, thereby reducing the installation time and cost.

A Weather Station (Model ID: ATMOS 41) is also being installed on the roof of N79 building for continuous monitoring of environmental variables, including air temperature, relative humidity, wind speed, wind direction, maximum wind gust, barometric pressure and vapor pressure.

National Instruments Chassis (Model ID: cDAQ-9189 CompactDAQ Chassis) along with National Instruments Sound and Vibration Input Module (Model ID: 9234 C Series, 4-Ch, 51.2 kS/s, IEPE and AC/DC) are being used as the data acquisition system.

End-user Interface and Dashboard

A key part of the living laboratory is the interface between the data itself and the researchers/staff/students accessing the data. Such end-user interface is being developed in collaboration with the Urban Institute (UI), which develops UrbanInsight as an innovative data analytics, visualisation and storage platform for complex urban applications. Fig. 8 shows the end-user interface that has been developed so far to access, visualise and extract the installed accelerometers' data. The sampling rate of vibrating wire earth pressure cells, vibrating wire piezometers, and wireless tipping bucket rain gauge is set to 0.1Hz. Additionally, the sampling rate of accelerometers and strain gauges is set to 200Hz and 20Hz, respectively. The environmental condition data will be recorded every minute.

Fig. 8. End-user to access, visualise and extract the installed accelerometers' data

Conclusions

The N79 living laboratory project has developed a platform, which enables various damage diagnosis and prognosis structural health monitoring research studies to be conducted by providing a large database on sub-surface and surface responses of the structure subjected to various environmental and loading conditions.

Conference News

- The 10th Australasian Congress on Applied Mechanics (ACAM10), to be held at The University of Adelaide, Adelaide, Australia, from 25-27 November 2020. Chair: Assoc. Prof. Alex Ng. Webpage: https://acamconference.com.au/
  Abstract submission due: 27 April 2020

  Full paper due: 20 July 2020

- Mini Symposium "Latest advances on SHM and smart structures in Australia/Oceania" in the Tenth International Conference on Structural Health Monitoring of Intelligent Infrastructure, Porto, Portugal, from 30 June to 2 July 2021. Organised by Dr Andy Nguyen, Assoc. Prof. Alex Ng, Prof. Tommy Chan.

  Webpage: https://web.fe.up.pt/~shmii10/conference/mini-symposia/

  Abstract submission due: 30 June 2020

  Full paper due: 10 Jan 2021

- Mini Symposium "Innovative data-driven techniques for Structural Health Monitoring" in the Tenth International Conference on Structural Health Monitoring of Intelligent Infrastructure, Porto, Portugal, from 30 June to 2 July 2021. Organised by Assoc. Prof. Jun Li, Prof. Ting-Hua Yi.

  Webpage: https://web.fe.up.pt/~shmii10/conference/mini-symposia/

  Abstract submission due: 30 June 2020

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If you have any comments and suggestions, please contact

Newsletter Editors:

Dr. Mehrisadat Makki Alamdari, University of New South Wales.

Email: m.makkialamdari@unsw.edu.au

Dr. Jun Li, Curtin University.

Email: junli@curtin.edu.au

Dr Andy Nguyen, University of Southern Queensland

Email: Andy.Nguyen@usq.edu.au

Prof. Richard Yang, Western Sydney University.

Email: R.Yang@westernsydney.edu.au