

IMPACT FROM THE CONSTRUCTION OF A WORKING PLATFORM AND GIRDER LIFTING OPERATIONS ON PILE/COLUMN DISPLACEMENTS

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ABSTRACT

To facilitate the initial construction of the bridge pier piles on the bank of Nambucca River in NSW Australia, a working platform was constructed over 25 m thick soft alluvial deposits. Subsequent to the pile installation and the construction of columns and headstock, it was determined that the platform needed to be raised by 1.5 m in order to allow for adequate clearance between the boom and the headstock. The raising of the working platform generated sub-soil flow that caused the piles to displace horizontally of up to 50 mm towards the river. Further, the erection of each 160 tonne girder was carried out by a super-lift operation at the raised platform. This exerted enormous bearing pressure on the platform. Surveying indicated that during the first two girder lifts, the tops of the bridge columns had displaced a further 39 mm towards the river, then recovered by about 15 mm after the lifting operation. To mitigate potential further lateral displacement for the remaining two girder lifts, a 5 m wide berm was constructed on the riverside of the pier. Surveying indicated that the placement of this counterweight had effectively limited further movement of the columns to less than 10 mm. One of the concerns with the lateral displacement of the pier was that the bending moment induced in the piles may exceed the moment capacity. Numerical analysis was carried out to back-analyse the measured lateral displacement and to assess the likely future performance of the piles.

This paper focuses on the geotechnical design of the working platform and the predictions of the lateral displacements of the pier piles, with or without the counter weights at the riverside of the pier. The comparison of the predictions and measurements, as well as the permanent effects on the pier piles are also outlined.

1 INTRODUCTION

The Nambucca River Bridge is located along a new section of the Pacific Highway between Warrell Creek and Nambucca Heads. The bridge is approximately 850m long and comprises 20 piers. Driven steel tubular piles of up to 2 m diameter were used to support abutments and piers of the bridge structure. To create the spans between each of the piers, and the abutments at either end, four prefabricated U-shaped concrete girders between 39.5m and 41.5m in length and weighing up to 153 tonnes each were lifted into place.

To facilitate the initial construction of the pier pile foundations, working platforms were constructed over soft alluvial deposits of up to 25 m thickness along the alignment of the bridge. Subsequent to the piles installation and the in-situ casting of the columns and headstocks, a challenge arose when it was determined that a working platform at Pier 09 next to the river bank needed to be raised by 1.5m in order to allow for adequate clearance between the crane boom and the pier during the girder lift operation. Furthermore, the girders were erected by a super-lift operation at the raised platform with a lift radius of up to 33 m. This has increased significantly the bearing pressure of the 600 tonne crawler crane, which was positioned in close proximity of the pier. While the working platform was designed to have adequate factor of safety for stability, the existing structures such as the pier piles founded in soft soils could experience movements and induced bending moment and stresses when the surrounding ground was loaded by platform filling followed by crane loading. Such movements and stresses could potentially impact on the future performance of the installed pier pile structures if not recognised or detected.

This paper reports a case study where the construction of a working platform and the crane operations caused lateral displacement of the existing pier piles.

2 GROUND CONDITIONS

The soil profile in the vicinity of Pier 09 comprises approximately 22 m thick soft to firm clay/ sandy clay, followed by 4.5 m thick firm to stiff sandy clay interbedded with gravels, and underlain by Phyllite bedrock. The adopted water level is at about RL 0.0m, which is approximately 0.4 m below the existing ground surface. Figure 1a shows the adopted undrained shear strength, S_u , derived from a nearby piezocone CPTU based on the corrected cone resistance (q_c) and using a cone factor (N_{kt}) of 16. The adopted over-consolidation ratio (OCR) for the alluvial clay layer varies from 1.05 to 2.3

as shown in Figure 1b. The assessed OCR values are consistent with the adopted S_u profile through SHANSEP relationship proposed by Ladd (1991):

$$S_u = S(OCR)^m \sigma'_{v0} \quad (1)$$

where S and m are SHANSEP parameters and σ'_{v0} is the vertical effective stress. The adopted S and m values are 0.22 and 0.94, respectively, which have been back-analysed from the survey monitoring for settlement for the preload fill embankment at the bridge abutments and approaches. The back-analysed compressibility of the soils, including the compression ratio ($CR = c_c / (1 + e_0)$), the recompression ratio ($RR = c_r / (1 + e_0)$) and the creep strain rate ($c_\alpha / (1 + e_0)$) are 0.18, 0.025 and 0.006, respectively. The symbols c_c , c_r , c_α and e_0 are the compression index, recompression index, creep coefficient and initial void ratio, respectively. There are five dissipation tests undertaken at the CPTU. The assessment of soil permeability uses the evaluated dissipation time, t_{50} , derived from the dissipation test results and the relationship presented in Figure 1c.

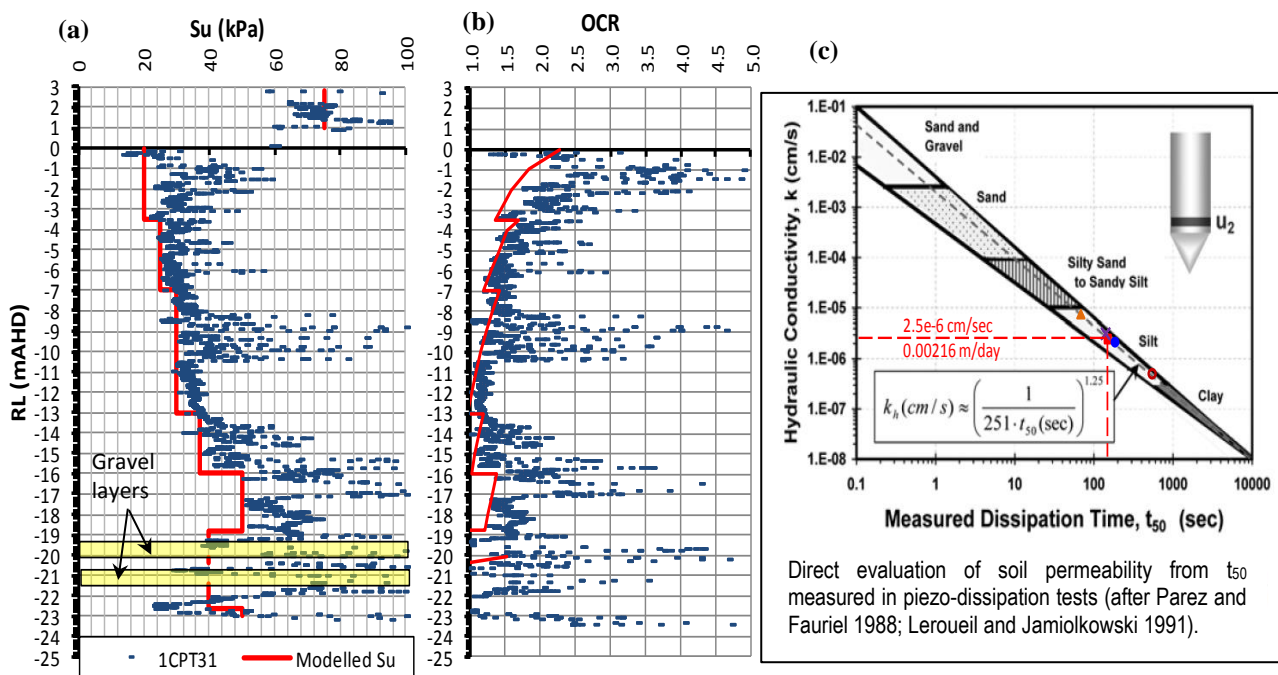


Figure 1: (a) S_u vs. RL profile inferred from 1CPT31, (b) Inferred OCR profile from S_u via SHANSEP, (c) Permeability vs. t_{50} inferred from dissipation tests conducted at 1CPT31

3 BACKGROUND AND DESIGN OPTIONS

Pier 09 of the Nambucca River Bridge is located on the northern riverbank. It consists of two columns, each supported by a 2 m diameter steel tubular pile driven into medium strength or better Phyllite bedrock. To facilitate the initial construction of pier pile foundation, a 0.8 m to 1.1 m thick rockfill platform with high strength geotextile was constructed to RL 1.9 m. The high strength geotextile has a short-term characteristic tensile strength of 800 kN/m, placed perpendicularly to the pier at approximately RL 1.1 m. As the working platform was constructed long before the installation of the piles, any consideration settlement induced by the self-weight of the platform was not considered to have impacted on the piles. A survey conducted at the time of the pile construction indicated that the level of the platform was lower, in parts, than the original level by up to about 350 mm. For increased quality of the obtained force measurement during piling driving, the mounted gauges of the Pile Driving Analyser (PDA) on the pile wall needed to be away from the impact and as high as possible above the ground. This resulted in the need to excavate the rock-fill platform around the piles, thereby damaging the high strength geotextile and exposing the existing ground surrounding the piles.

Subsequent to the piles installation and the in-situ casting of the columns and headstocks, it was determined that the working platform at Pier 9 next to the river bank needed to be raised to RL 3.0m in order to allow for adequate clearance between the boom of the crane lifting the girders and the headstock of Pier 09. Note that Pier 09 is located beneath the summit of the bridge; it has the highest bridge columns with the bearing levels setting at RL 14.65 m. The raising of the platform fill, about 1.3 m thickness, would activate renewed sub-soil flow that was likely to cause horizontal displacement of the installed piles. Further, the girders were to be erected by a 600 tonne crane positioned in close proximity to the

pier, along with its super-lift counterweight of 300 tonnes. This would exert enormous bearing pressure on the platform, further exacerbating the complexity of the design tasks for platform stability and impact assessment of the pier piles.

A number of design options were considered to address the platform stability and the issue of potential pile movements. These include: (1) Incremental launching method (ILM) for bridge span installation; (2) Soft ground treatments using stone columns, deep soil mixing etc.; (3) Installation of temporary supporting piles beneath crawler crane location; (4) Installation of sheet pile walls with tie-back reinforcements at the river bank in front Pier 09; (5) Installation of stability berm in front of Pier 09; or (6) Placement of additional high strength geotextiles within the raised platform plus the use of steel mat for load spreading.

The ILM was ruled out since it is an expensive procedure for bridge construction. It would require a considerable amount of analysis and design expertise and specialised construction equipment that was not readily available. The implementations of soft ground treatment (Option 2) or crane-supporting piles (Option 3) were not economical for the reason of high mobilisation costs and delay in construction. The installation of sheet pile wall in front of the installed Pier 09 columns (Option 4) would not be possible because of space constraint. A platform built out from the shore into the river is required for sheet piling construction. In this instance, however, sheet pile wall would not be necessary since the extended platform fill in front of Pier 09 could act as a berm for platform stability (Option 5). The berm option was also considered an effective and economical method to limit the potential pile movement. The use of the river for berm construction was not unacceptable by the RMS, provided that a detailed environmental management plan was set in place for the temporary work to minimise environmental impacts. Some of the controls required in the plan included the use of clean rock fill, and the application of floating barriers around the berm to contain and control the dispersion of silt in the river. The last option (Option 6) would involve the minimum construction effort, and is therefore the most economical method from the construction standpoint. Whilst this method might be able to achieve a satisfactory factor of safety for platform stability, it was considered less effective in controlling the potential movements of the piles. The design prediction of the lateral pile displacement and the induced forces and bending moments in the piles had become vital in determining the feasibility of this option. Unfortunately, it is well known that, due to the non-homogenous and anisotropic nature of the soils, the horizontal pile displacement induced by the lateral soil flow is difficult to predict accurately (Poulos, 1972). The variation in the analysis results meant that an observational approach to this design option should be adopted.

Upon review of the ground conditions and in consultation with the Construction Team, a combination of design Options 5 and 6 was adopted. In this approach, a reasonable design with the minimum construction cost (i.e. Option 6) was adopted (see Figures 2a, 4c and 8a). The performances of the crane settlement and horizontal displacements of the columns and headstock at Pier 09 were extensively and carefully monitored during the girder lift operation. A suitable remedial berm (i.e. Option 5) would be instituted should the survey measurements indicated the need (see Figure 8b).

4 GIRDER LIFTING PLAN

The erection of bridge girders involved a non-routine lifting operation that required detailed planning. Following the adopted design option discussed in Section 3, the rock platform was firstly raised to the required elevation at RL3.0 m (Figure 2a). The crawler crane was further raised to RL3.25 m via the placement of 250 mm thick 310UC steel mats. These crane mats were used to spread the load imparted by the crane crawler, and to add extra height to compensate for the ground settlement during the lifting operation. Further, the configuration and the movement of the crane were predetermined. Figure 2a (also shown in Figure 4c) indicates that the 1.5 m wide crawler tracks were centralised over the 4.5 m wide steel mats, which had a clear spacing of about 3.1 m from the crest of the rock fill batter over the river bank, and about 2.1m from Pier 9 column face at the closest point. As per the lifting plan outlined in Figure 2b, the 600 tonne Demag CC2800-1 crawler crane picked up the 158 tonne girder from the haul road at a lift radius of 14 m and tracked back over the steel mats to the lift position. The crane then slewed 90°, connected the 284 tonne super-lift tray with counterweights, and lowered the lattice boom to a maximum of 33 m lift radius to place girder. This process was repeated successively for the erection of four 41.4 m long girders spanning over Pier 8 (at the river) and Pier 9 (at the riverbank).

5 TRACK PRESSURE LOADS

The track bearing pressure was assessed for a range of operation and lift orientation that was carried out. To assess the performance of the raised rock platform and the impact of crane loading on the permanent pile design, an equivalent “block” pressure was obtained. This was obtained by first identifying the track bearing pressure for the critical orientation of the crane as it lowered the 158 tonne girder into position with a maximum slew radius of 33 m. The trapezoidal/triangular distribution of this critical bearing pressure was then converted to an equivalent rectangular or “block” pressure by determining the effective length and width of the crawler crane using Meyerhof (1991) method. Note that for the plane strain analyses undertaken as part of the stability and deformation assessments to be discussed later in this paper, the adoption of an equivalent “block” pressure was considered appropriate to reflect how the crane would impart load onto the rock platform.

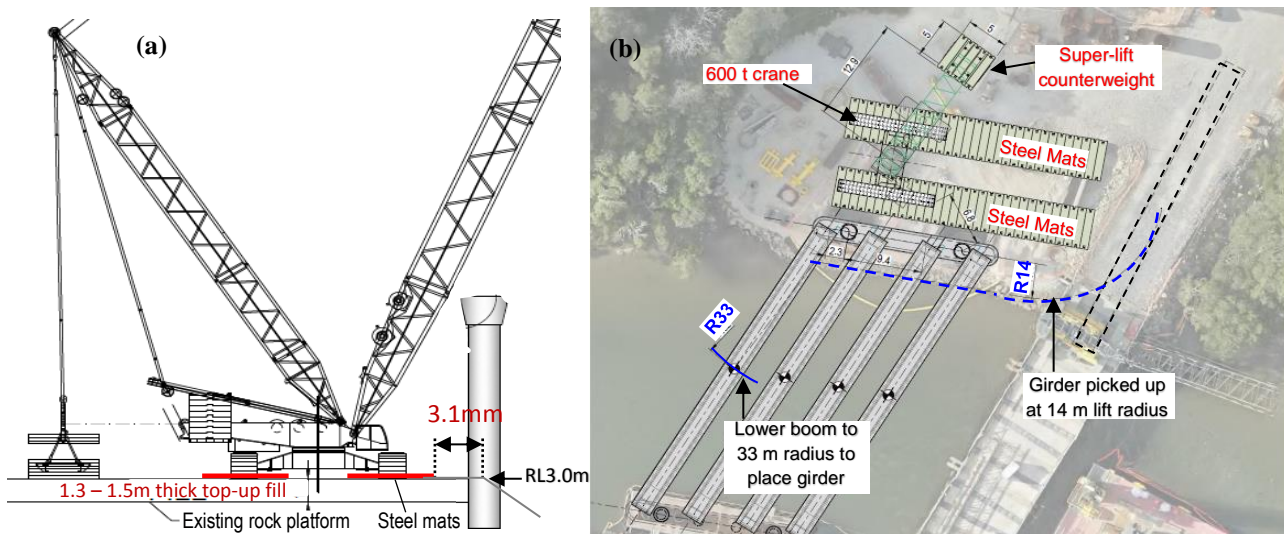


Figure 2: (a) Position of crane relative to as-installed Pier 09 column, (b) Lifting Plan

Steel mats were placed underneath the crawler tracks in order to assist with spreading load over a larger area. Each steel mat was constructed from twelve 200UC52.2 universal columns (Figure 3a), each 4.5 m in length, which had been welded together and placed perpendicular to each crawler track. While the total width of the steel mat is 4.5 m, it is less likely that the applied crawler load would be spread over the entire mat width since the steel mat experienced some bending under loading. A 2D plain strain finite element analysis (FEA) using PLAXIS 2D (Version 2016.01) was conducted to model the steel mat and the overlying imposed equivalent block pressure of 592 kPa closest to the river (Figure 3a). More details of the FEA are discussed in Section 7. Figure 3b shows the pressure distribution at the underside of the steel mats derived from the 2D FEA. The analysis result indicates that the effective spread width can be considered to be about 3.5 m. For design purposes, in particular for the stability check of the platform using limit equilibrium method (discussed in Section 6), an averaged block pressure of 254 kPa ($= 592 \text{ kPa} \times 1.5 \text{ m} / 3.5 \text{ m}$) could be adopted based on this assessed effective width at the underside of the steel plates closest to the river. For the other side of the crawler track away from the river, the assessed equivalent block pressures at the undersides of the track and steel plate were 425 kPa and 182 kPa, respectively, as depicted in Figure 4c.

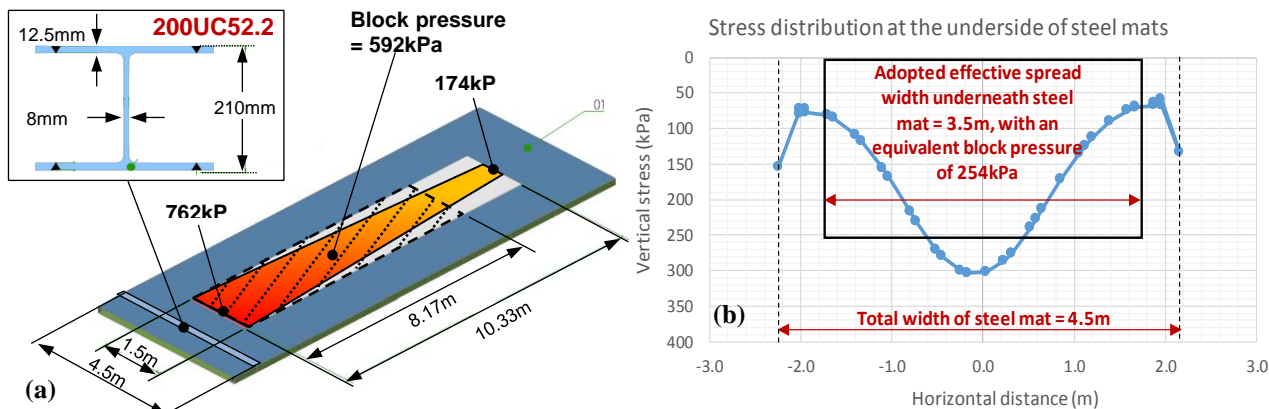


Figure 3: (a) Track pressure over steel mat, (b) Assessed pressure distribution from PLAXIS at the underside of the steel mat for an imposed equivalent block pressure of 592kPa.

6 PLATFORM STABILITY

The stability of the raised working platform was critical for the girder lifting operation. The consequence of platform failure or significant damage to the installed pier piles and columns was unacceptable. The working platform must be designed to have adequate factor of safety for stability against the following failure modes: (i) punching failure; (ii) slope failure; (iii) lateral sliding of fill above basal reinforcement and (iv) foundation extrusion. The following stability measures were proposed for the working platform depicted in Figure 4c.

- The total thickness of the rock fill working platform varied between 2.1 m and 2.6 m. This consisted of the initial 0.8-1.1 m thick piling platform and a top-up thickness of about 1.3 - 1.5 m. Survey measurements indicated that the initial piling platform had settled, in parts, by up to 300 mm after the piling operation.
- There was a high strength geotextile with a tensile strength of 800kN/m within the initial piling platform. This layer of geotextile was assumed not to be anchored on account that it had been damaged during the installation of the pier piles as discussed in Section 3.
- Two layers of high strength geotextile, minimum tensile strength of 800kN/m and 600kN/m, were installed as part of the platform raising at RL 1.5m and RL 1.8m respectively. These reinforcements were rolled out perpendicularly to the Pier 09 piles, extended over the full width of the platform, and anchored with a minimum wrap length of 5.5 m at either end (discussed in Section 6.3).
- Steel mats as described in Section 5 were positioned in relation to the pier columns and the ridge of the platform batter as outlined in Section 4. The slope of the platform batter is about 1.7H:1V.

6.1 PUNCHING FAILURE MODE

For the applied equivalent block pressure of 254 kPa over an effective area of 3.5 m × 8.2 m at the underside of the steel mat closest to the river (Figure 3b), the total bearing resistance was considered to be the sum of the shear required to punch through a vertical plane of the platform and the bearing capacity of the subgrade. This simplified punching failure mechanism is outlined in the practice guide for working platforms for tracked plant produced by Building Research Development (BRE), 2004. As the effective strip width of 3.5 m was offset at a distance of about 4.3 m parallel to the crest of the batter, it was considered sufficiently far away from and not to be impacted by the adjacent batter. Further, punching failure was not considered the most critical failure mode due to the relatively thick platform in place (2.1-2.6 m) and the presence of multi-layered high strength geotextiles.

6.2 SLOPE FAILURE MODE

The assessment of slope stability using the conventional method of slices was not able to consider load spreading through the rock fill platform. It was therefore necessary to apply a dispersed design pressure, as opposed to the bearing pressure at the underside of the steel mats, in the global stability analysis. The load spreading was assumed to follow a line with a vertical-to-horizontal slope of 2:1, all around the equivalent block pressure at the underside of the steel plate, until it intercepted the base of the working platform. The thickness of the working platform was thicker (approximately 2.6 m) beneath crawler track closest to the river than that away from the river (about 2.1 m). Therefore, the dispersed pressures and their extents at the different sides of the track are different. Note that the adopted 2V:1H load spreading was considered conservative for the compacted rock fill. It can be demonstrated using elastic solution or more sophisticated finite element methods that the angle of load spread will increase (up to 1V:1H) as the stiffness of the soil increases.

6.3 LATERAL SLIDING

The stability against lateral sliding of the fill wedge above high strength geotextile was assessed in principle following clause 8.3.2.6 of *BS8006.1 2010: Code of practice for strengthened/reinforced soils and other fills* (BS8006). The determination of the minimum reinforcement bond length to resist the horizontal outward thrust of the platform fill given by BS8006 assumes that a surcharge load is applied uniformly across the top of the embankment. The situation that occurred during girder lift however did not have a uniform surcharge applied across the platform but rather two strips of crawler track loads. The resulting stress distribution within the platform fill material is shown in Figure 4a, and the minimum reinforcement bond length, L_e , was given by the following equation slightly different to that of BS8006:

$$L_e \geq \frac{(0.5K_a f_{fs} \gamma_{fill} H^2 + f_q \int_0^H \sigma_H d\sigma) f_s f_n}{\gamma_{fill} h \frac{a' \tan \phi'_{cv}}{f_{ms}}} \quad (2)$$

where the partial factors f_{fs} , f_q , f_s , f_n and f_{ms} are defined in BS8006.1 2010. The other parameters of the equation including ϕ'_{cv} , K_a , γ_{fill} , H , h , a' and σ_H are defined in Figure 4a and the legend in Figure 4. In particular, σ_H denotes the lateral pressure distribution within the platform fill caused by crawler track loadings, and was assessed based on the theory of elasticity. For design purposes, it was determined along a vertical plane immediately adjacent to the edge of the strip load closest to the crest of the rock fill batter, i.e. the dashed red line shown in Figure 4a. The σ_H was calculated as a result of the two strip loads, each applied over an effective width of 3.5m, and where $q_1 = 254$ kPa and $q_2 = 182$ kPa (offset 8.40m from the vertical plane along which stress are to be determined), as shown in Figures 4a and 4c.

From the aforementioned method outlined, it was determined that the minimum reinforcement bond length of approximately 10 m was required to resist the horizontal outward thrust of the platform fill. This was achieved by anchoring the high strength geotextiles at the batter slope via a minimum of 5.5 m wrap length as depicted in Figure 4c.

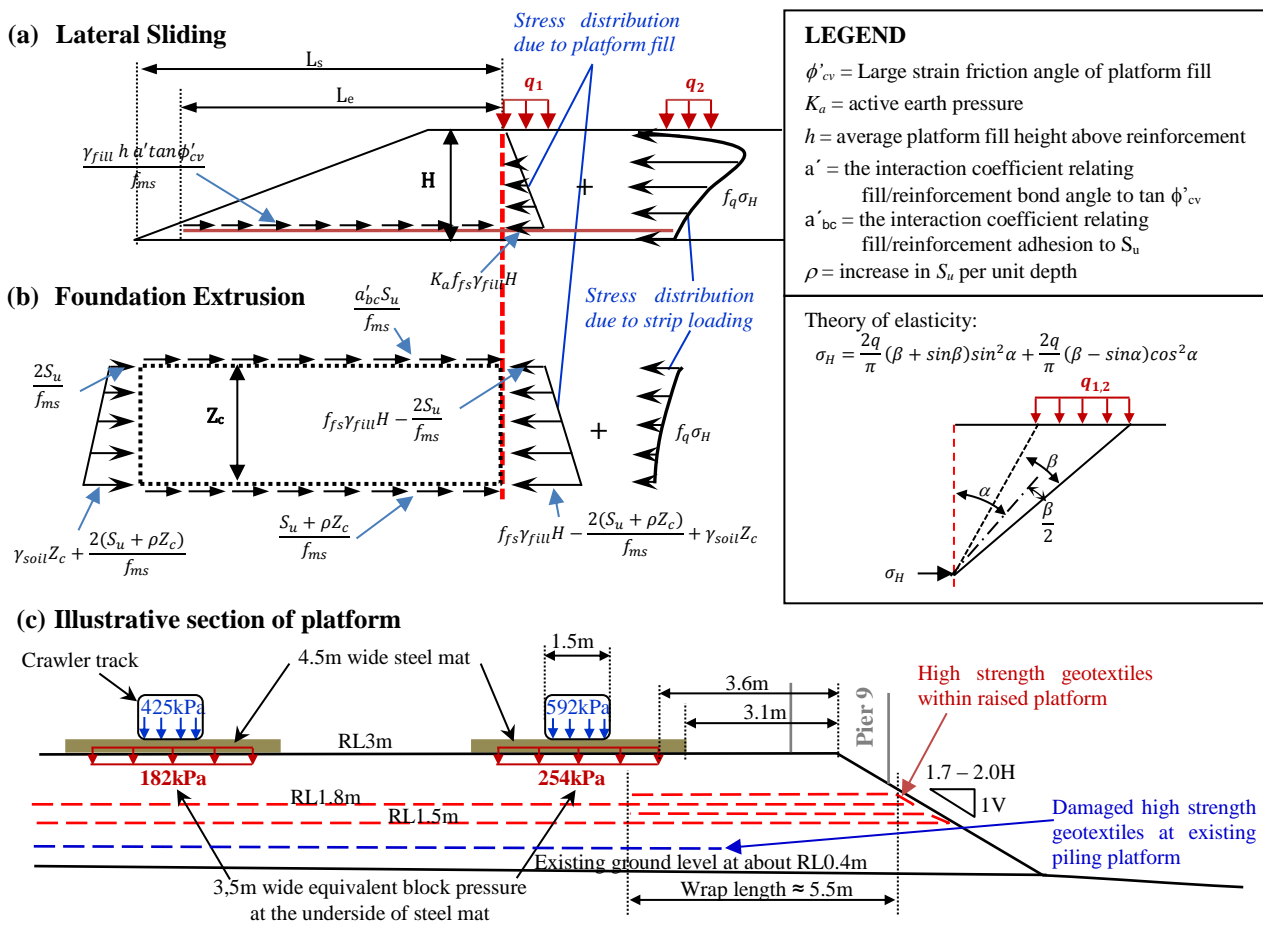


Figure 4: (a) Force components for lateral sliding, (b) Force components for foundation extrusion, (c) Illustrative section of platform

6.4 FOUNDATION EXTRUSION

An assessment of foundation extrusion, which occurs when the geometry of the embankment induces outward shear stresses within the soft foundation soil, and where the foundation soil is soft and of limited depth, was undertaken in a manner similar to that outlined in clause 8.3.2.6 of BS 8006.1:2010. Figure 4b schematically illustrates the force components in the foundation extrusion stability analysis. Two strip loads were applied, as opposed to an uniform surcharge, over the top of the working platform. The minimum side slope length, L_s , required to prevent foundation extrusion was calculated based on:

$$L_s \geq \frac{\left(\left(f_s \gamma_{fill} H - \frac{2(2S_u + \rho Z_c)}{f_{ms}} \right) Z_c + f_q \int_{-Z_c}^0 \sigma_H d\sigma \right)}{\frac{(\alpha'_{bc} + 1) S_u + \rho Z_c}{f_{ms}}} \quad (3)$$

where the engineering parameters including S_u , γ_{soil} , a'_{bc} and Z_c are defined in Figure 4b and the legend in Figure 4. The term σ_H is the lateral pressure distribution within the foundation soil. Noting the apparent change in undrained shear strength at RL -7.0m (see Figure 1a), it was considered that this would be an appropriate cut-off in respect of determining the lateral pressure within the soil that should be considered when assessing lateral extrusion. With the assessed σ_H reducing with depth, the foundation extrusion was not considered as a critical failure mode. The side slope length depicted in Figure 4c was assessed to be greater than the minimum required length L_s .

7 IMPACT OF PLATFORM RAISING AND CRANE LOADING ON PIER PILES

The impact of platform raising and crane loading on pier piles was assessed by the combination of a 2D finite element (FE) method and a boundary-element (BE) method. The 2D FE analysis was carried out first to generate the lateral soil movements due to the above construction activities without the presence of the pile. These generated free-field soil movements along the pile depth were then used in the BE analysis as the applied soil movement loads to assess the pile response. The BE analysis considered the 3D nature of the pile.

The preceding FE analysis was carried out using PLAXIS 2D. In this analysis, the soft clays were modelled using Soft Soil Model in the PLAXIS programme, which resembles the Modified Cam-Clay model with a Mohr-Coulomb hexagon yield surface in the deviatoric plane. Further, the FE analysis involved coupled consolidation to assess soil movements with time. The adopted coefficient permeability, k , was related to the void ratio, e , in accordance with $e = e_0 + c_k \log(k/k_0)$, where e_0 is the initial void ratio, c_k is the permeability index and k_0 is the initial permeability value. Consistent with the laboratory results, the e_0 for the low to medium plasticity clays is 1.2. The c_k can be related to e_0 in the form of $c_k=0.5 e_0$ as proposed by Tavenas *et al.* (1983), thus $c_k = 0.5$ was adopted. The adopted k_0 was 2.5×10^6 cm/sec, which is consistent with the results inferred from the dissipation tests shown in Figure 1c. As mentioned in Section 5, the FE analysis has also considered the placement of steel mat, 4.5 m wide, centred beneath each 1.5 m wide crawler track of the crane. To assess the soil deformations as a consequence of girder lifting operations, equivalent block pressures of 425 kPa and 592 kPa were imposed across the respective width of the crawler tracks as depicted in Figure 4c. The adopted construction sequences are summarised in Figure 5a. Figure 5b shows the assessed horizontal soil displacement contour 1 day following the application of the sustained crane lifting load. Figure 5c shows the assessed horizontal soil displacement at the inferred location of Pier 09, which is 2.1 m from the edge of the steel mat, 3.15 m from the edge of the crane crawler AC. The analysis results indicate that the predicted maximum horizontal free-field soil displacement at the pile location due to platform raising plus 1 day of sustained crane loading is about 65 mm.

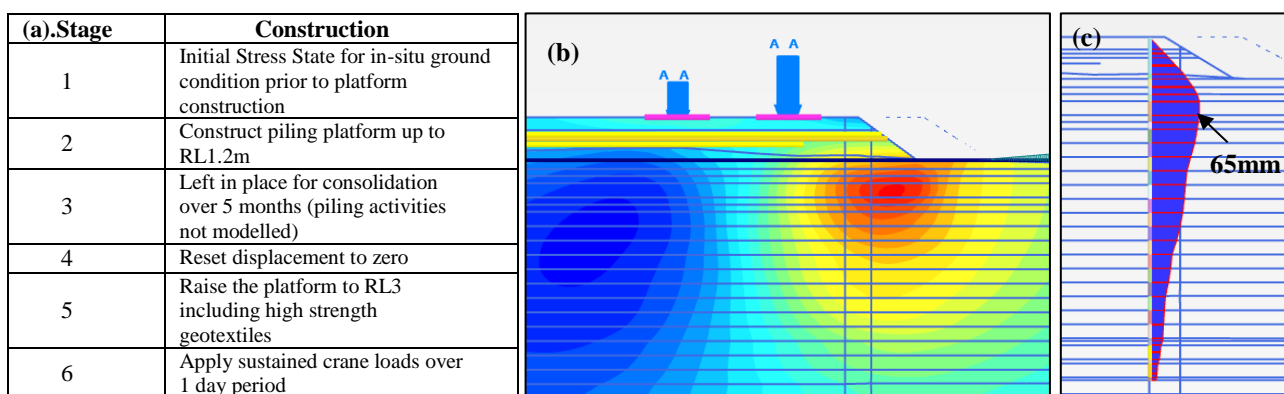


Figure 5 – (a) Construction sequence in PLAXIS analysis, (b) Contour of horizontal displacement from PLAXIS, (c) Predicted horizontal greenfield displacement from PLAXIS along the pile surface of the proposed pile location

The computed lateral soil movements from the FE analysis were then used as input into the BE program for pile response analysis. The BE program used is named PALLAS (for Piles And Lateral Loading Analysis) and has been described in Poulos *et al.* (1995). PALLAS uses a simplified form of BE analysis in which the pile is idealised as an elastic beam and the soil as an elastic continuum, but with limiting pressures, p_y , at the pile-soil interface to allow considerations of yielding of soil surrounding the pile. The program can consider a single pile or a group of non-identical piles, and take into account of the proximity of the pile to slope batter based on Poulos (1976). The input parameters for the piles consist of the bending stiffness, diameter and length of individual pile within the group. The soil model requires the specification of the Poisson’s ratio and the distributions with depth of p_y and soil Young’s modulus, E_h for horizontal loading. In the design, $p_y = 9S_u$ and $E_h = 225S_u$ were adopted. Note that E_h was taken as 75% of E_v , where E_v is the soil Young’s modulus for vertical loading and $E_v = 300 \times S_u$ was adopted. The red curve in Figure 6a shows the BE analysis input of the specified greenfield horizontal soil displacement under crane loading as obtained from PLAXIS (Figure 5c). The analysis results, as shown by the red curves in Figures 6b to 6e, include the pile horizontal displacement, rotation, shear force (normal to the pile) and bending moment and shear force with depth. The predicted lateral displacement and rotation of the pile/column at the top of the platform were about 57 mm and 0.0014 rad, respectively. The predicted horizontal displacement at the top of Pier 9 headstock, which was about 11 m above the top of the platform, was approximately 72 mm (= 57 mm + 0.0014 rad \times 11000 mm).

8 MONITORING RESULTS DURING LIFTING OPERATION

Pier 09 of the Nambucca River Bridge comprises two columns, COL 9-01 and COL 9-02. Following the construction of COL 9-01 and COL 9-02 the columns were surveyed to be deviated less than 5 mm in any direction from the design location for the column. Prior to the first two girder lifts for Span 09, the Pier 09 columns were re-surveyed. This survey indicated that the tops of the columns had displaced between 25 mm and 50 mm towards Nambucca River since the as-built survey was completed. These displacements recorded prior to the Span 09 girder lifts were inferred to have occurred as a result of the platform raising from approximately RL 1.5m to RL 3.0m, with the additional rock having increased the lateral pressure acting on the Pier 09 piles since no additional rock being placed on the riverside of the piles.

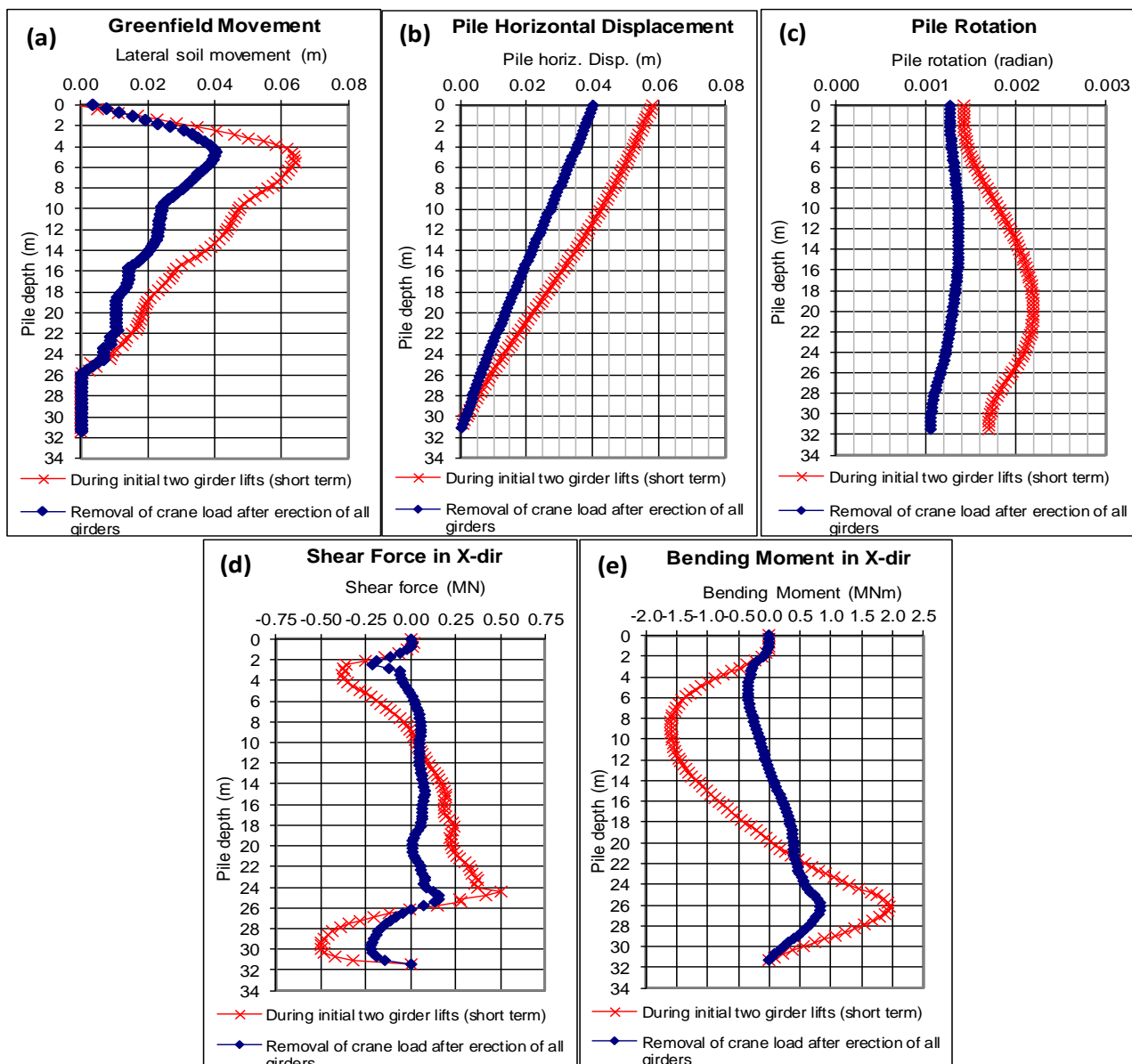


Figure 6: (a) Predicted greenfield horizontal soil displacement from PLAXIS, (b) Predicted pile horizontal displacements, (c) Predicted pile rotations, (d) Predicted shear forces in piles, (e) Predicted bending moments in piles

There were in total four girders to be lifted at Span 09. Following the completion of the second girder lift the displacement at the top of columns COL 9-01 and COL 9-02 were surveyed again, which indicated that an additional 39 mm and 28 mm of horizontal displacement took place, respectively, towards the river. The total horizontal displacements of the column tops at this stage were between 64 and 78 mm, as shown in Figure 7. The average of these displacements has reached the design prediction of 72 mm outlined in Section 7 at the end of the second girder lift, and the subsequent lifting operation was halted. There were some gradual recovery of the lateral displacements perpendicular to Pier 09 alignment, up to about 15 mm of rebound, when the columns were re-surveyed after 9 days following the second girder lift. The rock platform did not show signs of distress in the form of cracking or slumping near the platform slope. In fact, the occurrence of rebound in horizontal displacement of the columns after unloading indicated that the foundation soil was still within the elastic range, in a global sense, before developing a plastic slip plane.

As a response to the observed displacements to the Pier 09 columns as a result of the first and second girder lifts, the Construction Team has deployed the back-up plan, in which the rock platform was widened on the riverside of the Pier 09 columns by constructing a 5 m wide unreinforced berm (Figure 8b). The berm was constructed of the same rock material as the existing rock platform, had a batter slope no steeper than 1.7H:1V, and be raised to a level commensurate with the existing rock platform, i.e. RL 3.0 m. There were some minor restoring movements of the columns observed, less than 5 mm, after the installation of the rock berm (Figure 7). Further, the berm has greatly limited further

displacements of the columns in the subsequent girder lifts. Survey monitoring indicated that the additional incremental horizontal displacements as a result of the third and the fourth girder lifts are up to 8 mm towards Nambucca River.

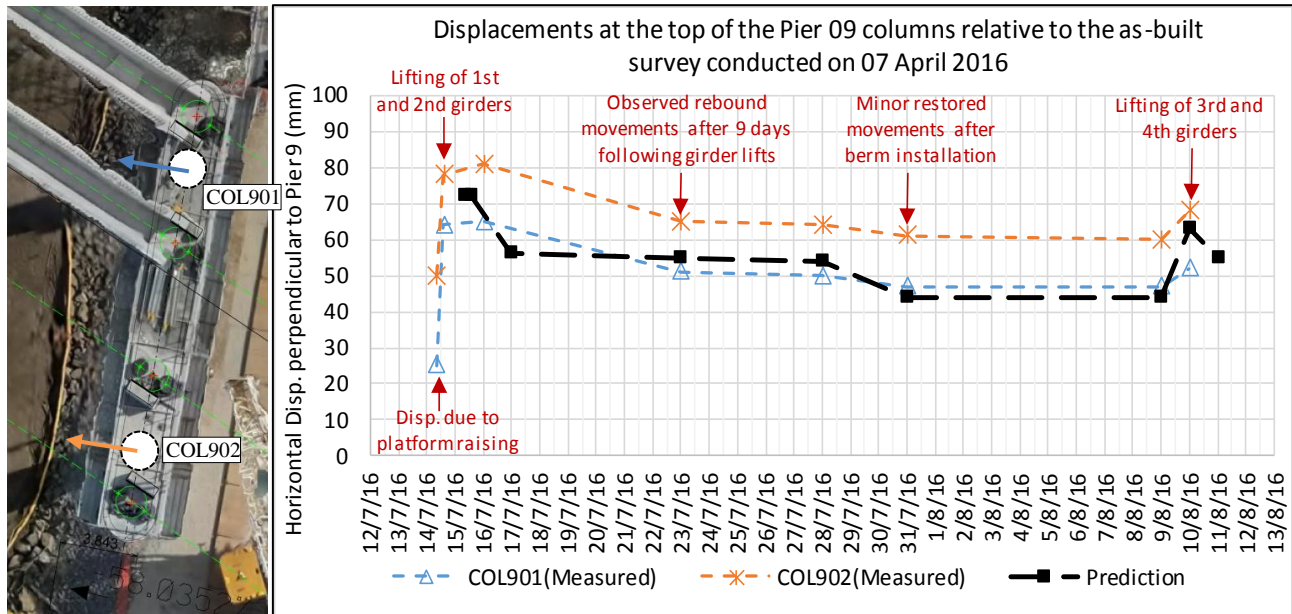


Figure 7 – Graphical representation of recorded horizontal longitudinal displacements at the top of Pier 09 columns

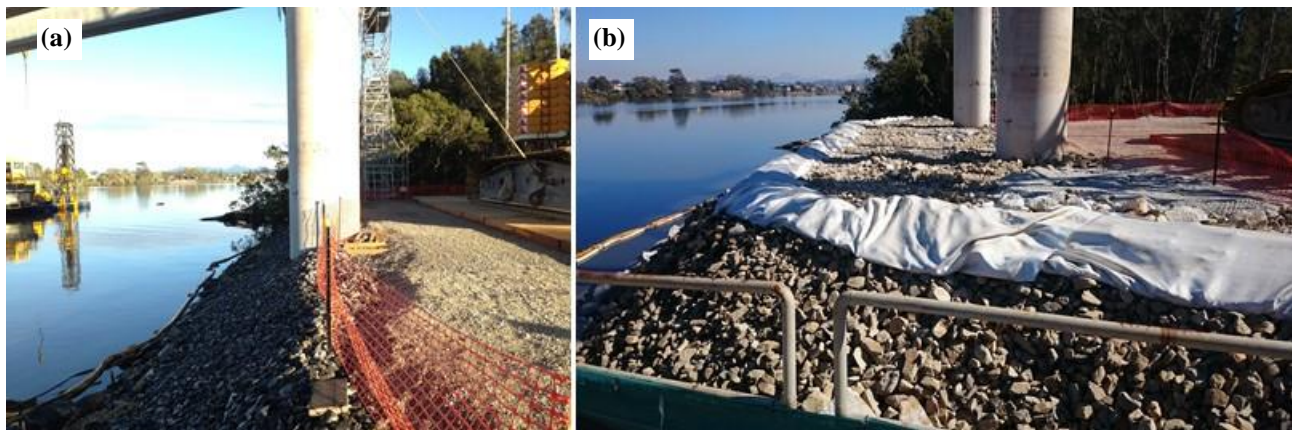


Figure 8: (a) Platform Design Option 6 (without stability berm at the riverside) - High strength geotextile reinforced working platform and the use of steel mats underneath crawler crane; (b) Platform Design Option 6 with an additional 5 m wide stability berm installed at the riverside

9 IMPACT ON PERMANENT DESIGN OF PIER PILES

The assessed bending moment of the piles during girder lifting presented by the red curve in Figure 6e was a temporary induced load caused by short term girder lift operation. After the unloading of the crane and the stripping of the working platform, there would be invariably some recovery of horizontal displacements, leading to a reduced bending moment of the piles. This is evident from the survey monitoring, which showed a rebound horizontal displacement of up to about 15 mm at the top of columns after the removal of crane load; followed by a further minor recovery of column displacements after the installation of the rock berm at the riverside of Pier 09.

To assess the impact on the permanent design of the pier piles, numerical analyses utilising the 2D FE and BE methods as outlined in Section 7 were carried out to further simulate the horizontal movements of the column top and the induced pile forces corresponding the main construction activities after the initial two girder lifts. For all of the pile analyses using BE methods, in particular, the pile heads were considered free to translate and rotate without restraints. It is acknowledged that some lateral resistance could have developed progressively against longitudinal movement at the top of Pier 09 as the girders were installed, which propped the pier top back to Pier 08. However, full restraints due to full frame action of the bridge portal frame was not considered to have established since the erection of Span 09 girders were ahead of those for the adjoining Spans 08 and 10 as shown in Figure 9.

The black dashed curve in Figure 7 shows the predicted pier top longitudinal displacements for the different construction activities in relation to girder lifts at Pier 09. The predicted displacements are in general comparable with the surveyed movements in terms of the trend. The predicted total recovery of longitudinal displacements since the initial two girder is about 25 mm, which includes the contribution of elastic rebound and the installation of rock berm. The survey measurements shows a slightly lower corresponding recovery of about 15-20 mm. There was no significant shear distortion observed at the elastomeric bearings beneath the seated girders, indicating that the recovered longitudinal displacement could have been transferred, either totally or partially, to Pier 08 at the other end of Span 09. However, we have not been able to confirm this mechanism as no survey data was available for Pier 08 movements.

The blue curves in Figures 6d and 6e shows the assessed shear force and residual bending moment of the piles under SLS after the removal of crane load following the erection of the 3rd and the 4th girders. It would be anticipated that in the finished condition after the stripping of the platform, the residual pile forces and displacement would further reduce. The assessed residual bending moment in the piles is much lower than the maximum values during the application of the crane loading (loaded and un-loaded). In accordance with an assessment conducted by the design Structural Engineer, the piles and columns would have sufficient reserve capacity to accommodate the additional bending moments due to eccentricity of vertical loads and the assessed residual bending moments in the piles. Bending due to the design eccentricity of the vertical loads was a relatively small component of the total bending moment so the additional moment from increased eccentricity was relatively small. The largest bending moments came from horizontal forces such as braking, creep and shrinkage and earthquake which were not affected by the eccentricities. Bending in the column also came from frame action of the portal frame which were similarly not affected. The residual bending moment in the pile was also relatively small and the locations of the peak values did not coincide with the locations of the peak values from other design loads and effects. Furthermore, the piles would have sufficient capacity to withstand the assessed short term maximum bending moment occurred during construction as a result of the crane loading.

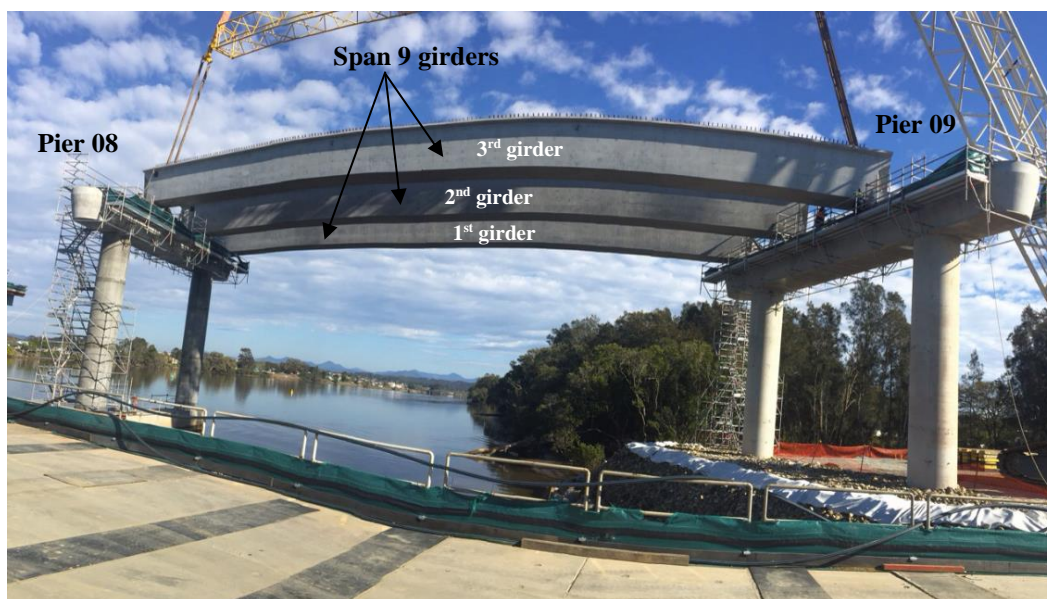


Figure 9: Photo taken during the erection of the 3rd girder

10 DISCUSSION – SIGNIFICANCE OF CONSTRUCTION PHASE DESIGN REVIEW

The erection of Span 09 girders was by far the most difficult compared to other girder lifts along Nambucca River Bridge. This was due to a combination of factors including heavy lifting of 160 tonne girders in close proximity to unconfined piles at the riverbank over deep soft soil profile. The Geotechnical Team on site has made extensive assessments prior to the lifting operation. These include:

- Assessment of ground conditions surrounding the piles including the strength, compressibility and draining characteristic of the soft soil alluvium.
- Site visit by site geotechnical representatives to assess proximity hazards such as the nearby river and pier structures.
- Proactive communication with the Construction Team on the design options in relation to girder lifting.
- Review of the girder lifting plan and the assessment of applied track pressure loads.
- Design of the working platform for stability against the failure modes that are particularly relevant to lifting next to slope crest at the riverbank. These include (i) punching failure, (ii) slope failure, (iii) lateral sliding of fill above basal reinforcement and (iv) foundation extrusion.

- Assessment of the short term (during the lift operation) pier pile displacement and bending moment induced by lateral sub-soil movement resulting from the crane and platform loading.
- Assessment of the long term residual bending moment in the pile after the removal of the crane and platform loading.
- Devising a procedure / back-up plan to deal with some reasonably foreseeable situations such as pile / platform movements exceeding the predictions.
- Devising a monitoring plan for crane and pier pile/column movements.
- The deployment of the back-up plan (i.e. the construction of the 5 m wide berm) when the survey measurements of Pier 09 longitudinal displacement reached the design prediction after the initial two out of the four girder lifts.

The proactive approach by the site Geotechnical Team provided the following benefits:

- The assessment provided information to the Construction team to mitigate and avoid possible structural damage of Pier 09 due to crane lifting operation.
- Saving delays and money to the project.
- Improvement to construction process.
- Safe and good practice to the lifting operation.

11 CONCLUDING REMARKS

Heavy lifting is a high risk operation that must be planned and executed properly, in particular when it occurs close to a permanent structure. There are currently a number of guidelines/standards available for cranes (e.g. AS2550.5:2016) and for working platform design (e.g. BR470 2004). These references aim at preventing lifting failure caused by supporting ground not being firm enough, highlighting the importance of having a proper lifting plan and the design of working platform for stability. For soft soil foundation, the experience with unconfined pile and pier movement at the riverbank emphasises the need for careful consideration and management of the risk of structure displacements resulting from the temporary crane loading in close proximity. Proactive approach shall be established before the temporary construction between the site Geotechnical and the Construction Teams to mitigate and avoid possible structural damages due to soft ground displacements associated with temporary construction loading such as platform construction and crane lifting operations.

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