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Nawawi Chouw
Chunwei Zhang *Editors*

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Editors

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Preface

The series of ACMSM conferences was first held in Sydney in 1967. It is one of the longest running and reputable conferences in the field, taking place every 2–3 years either in Australia or New Zealand. The ACMSM26 has been postponed three times due to the COVID-19 pandemic. But finally, after more than 35 years, the conference returns to Auckland. Over these decades, the topics of the conference have expanded to include much broader research areas, beyond structural and material mechanics, as this year's conference demonstrates. The conference is no longer a premier forum largely for participants from Australia and New Zealand, but also an essential gathering of emerging and established researchers, as well as practicing engineers, from many countries. The conference presents an ideal platform for participants to extensively exchange knowledge and experiences as well as the development of new friendships and collaborations. Despite the reverberation of the pandemic, the ACMSM26 has attracted over 120 participants from four continents.

These ACMSM26 e-proceedings contain a selected set of 83 papers that cover a range of topics in the mechanics of structures and materials. We hope that the papers will ignite new ideas and trigger new collaborations in your research.

We would like to express our sincere gratitude to all authors, reviewers of the papers, and sponsors, i.e., the University of Auckland, Faculty of Engineering Transportation Research Centre, Asian Concrete Federation, Shenyang University of Technology, HIWAY Group, South China University of Technology, Downer, and Road Science for making this conference possible.

Auckland, New Zealand

Nawawi Chouw
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Contents

Composite Structures, Concrete, and Pavements

Spatial Correlation of Flexural Tensile Bond Strength in Unreinforced Masonry Walls

L. J. Gooch, M. J. Masia, M. G. Stewart, and C. Collard

Abstract The flexural tensile bond strength of the unit-mortar interface is an important material property in defining the load-carrying capacity of unreinforced masonry (URM) walls. This property often governs the response of URM walls subject to in-plane and out-of-plane flexure. Masonry is an inherently variability building material, with properties such as the flexural tensile bond strength having been observed to vary considerably when comparing adjacent mortar joints. This spatial variability influences the performance of URM structures and is an important consideration when performing stochastic assessments of masonry behaviour. This study describes an experimental investigation in which an URM wall was sequentially deconstructed utilising a bond wrench, and the bending stress to failure of each individual mortar joint was recorded. The bending capacity of each of these joints allows for an assessment of the spatial correlation of joint strengths within a masonry wall. Furthermore, 15 additional mortar joints, constructed in piers under identical conditions, and using the same unit type and the same mortar batch, as the examined wall, have been tested using a bond wrench. These supplemental tests are a standard form of estimating the strength of mortar joints within an URM structure. Examination of these specimens, therefore, provides insight into how accurately such tests estimate the true strength of joints within a wall.

Keywords Unreinforced masonry · Spatial variability · Correlation · Flexural tensile strength · Statistics · Materials

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1 Introduction

Recent studies into the variability and structural reliability of unreinforced masonry (URM) walls have utilised computational methods of estimating load–displacement behaviour [5, 9]. A key component of these methods is the accurate representation of material properties within a modelled masonry structure. These values are commonly sourced from experimental testing, standardised or recommended values such as those in AS 3700 [12] and NZSEE [10], or from extensive published databases [6, 8]. However, in order to produce a more accurate computational representation of an URM wall, consideration of how these properties varying throughout a structure should be made.

Previous studies by Heffler et al. [4] and Corrêa et al. [1] address the spatial variability of masonry flexural bond strength through the quantification of a correlation coefficient, ρ_k. This descriptor quantifies the degree of dependence that the flexural bond strength of any given mortar joint has from other joints within a course of a masonry wall. For example, a correlation coefficient equal to 1.0 for the bond strength of adjacent mortar joints implies that a full correlation between these distinct interfaces is present, and the determination of the strength of any one joint informs the strengths of all adjacent joints. In contrast, a coefficient equal to 0 implies that no correlation is present, and that no information is gained about adjacent elements within a wall. Finally, a value of -1.0 indicates that a full negative correlation exists. This results in alternating joint strengths of above then below average (or vice versa), with the changes in strength maintaining a constant value. This concept is presented in Fig. 1.

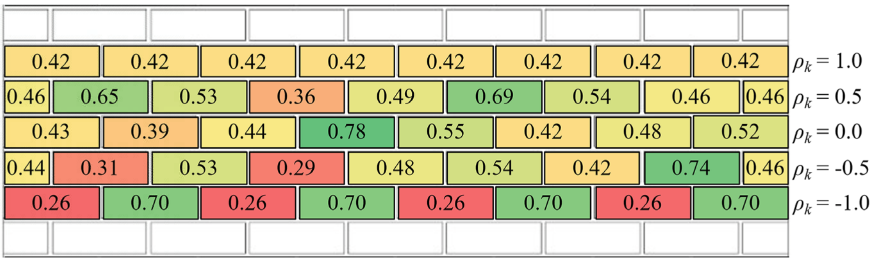

Fig. 1 Indicative flexural bond strengths for various correlation coefficients

The current study begins to expand upon the results of these previous investigations. A wall specimen constructed after those by Gooch et al. [3] was sequentially deconstructed using the standardised bond wrench test presented in AS 3700 [12]. Furthermore, as in the investigation by Corrêa et al. [1], several supplemental masonry piers were tested in the same manner. These additional tests, constructed using the same masonry units, mortar batch and bricklayer, facilitate research into the reliability of material characterisation tests.

2 Description of Experimental Testing

This initial study focuses on the results associated with a single URM wall specimen. This wall was constructed in a running bond pattern, with eight standard Australian masonry units (230 mm × 110 mm × 76 mm) per course and a total of fourteen courses. Mortar joint widths of approximately 10 mm were adopted, and a single wythe of units was used. The overall geometry of this wall specimen, as well as the supplemental prisms discussed in Sect. 3.2, are presented in Fig. 2.

Both the wall specimen and supplemental masonry prisms were constructed by the same individual bricklayer on the same day. A standard 1:1:6 (cement: lime: sand by volume) mortar mix was adopted, as specified in AS 3700 [12], and the same batch was utilised in the construction of all specimens. Finally, all specimens were aged for an extensive period of 222 days. This protracted age time is significantly longer than the 7 days required by AS 3700 [12], or the 28 days common to masonry research. However, the longer curing period effectively eliminates the potential for variability induced from inconsistent curing, both throughout the wall specimen, and between the wall and prisms.

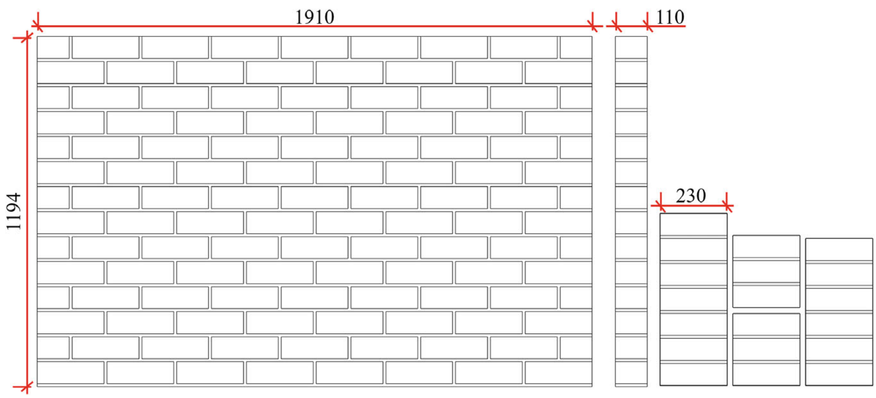

Fig. 2 Wall specimen and prism geometries (all dimensions are presented in mm)

2.1 Bond Wrench Test Method

The estimation of the flexural tensile bond strength of each mortar joint was achieved through the application of the bond wrench test. This test involves attaching a clamp to an individual masonry unit and applying a bending moment to its bed joint through the steady application of force. While the testing of masonry prisms can be readily undertaken as per the arrangement shown in Fig. 3, in order to test bed joints within the wall specimen, the vertical perpend joints were first removed. This was achieved using a sharp handsaw. A manual saw was utilised rather to minimise the vibrations of and potential damage to adjacent joints that could arise through the use of power tools.

The omission of perpend joints from this study was a result of two considerations. Firstly, flexural tensile testing of perpend joints is difficult, and their strengths are typically less critical to the overall strength of a given URM wall (see [7, 10, 12]). Secondly, it may be expected that some correlation exists between adjacent bed joints within an URM wall. These joints are typically laid in a sequence, with a bricklayer first placing a run of mortar capable of seating several masonry units. Perpend joints, however, are placed sequentially. Mortar is applied individually to one header face of a masonry unit before it is placed onto the mortar bed and tapped into position. As such, it is expected that a much weaker relationship between adjacent perpend joints exists. Hence no consideration of perpend joint strengths have been made in this study, nor that of Heffler et al. [4] and Corrêa et al. [1].

The flexural tensile bond strength f_{sp} of a given joint can be estimated from Eq. (1). In this study a bond wrench with a mass m_1 of 8.2 kg, an arm length d_2 of 1300 mm, and a distance to the centre-of-mass d_1 of 389 mm. The mass of each unit m_3 was found to be approximately 2.9 kg, or 1.5 kg in the case of the half bricks at each end of every alternate course.

$$f_{sp} = \left(\frac{M_{sp}}{Z_d}\right) - \left(\frac{F_{sp}}{A_d}\right) \quad (1)$$

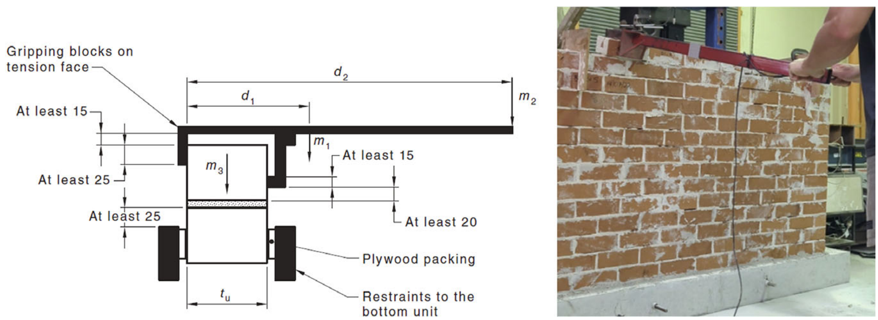

Fig. 3 Schematic and test set-up of the bond wrench test [12]

where M_{sp} is the applied bending moment about the centroid of the bedded area of the examined joint, estimated as per Eq. (2), and Z_d is the section modulus of the bedded area about the axis about which the bending moment is applied. Similarly, F_{sp} is the applied compressive force, calculated via Eq. (3), and A_d is the total bedded area.

$$M_{sp} = 9.81m_2 \cdot \left(d_2 - \frac{t_u}{2}\right) + 9.81m_1 \cdot \left(d_1 - \frac{t_u}{2}\right) \quad (2)$$

$$F_{sp} = 9.81 \cdot (m_1 + m_2 + m_3) \quad (3)$$

Load applied to the specimen m_2 is applied by the individual performing the test either through their own body mass or through the application of weights. This force must be applied at an even rate to avoid dynamic load effects until failure of the joint is achieved. A constant unit thickness t_u of 110 mm was present in all specimens tested in this study.

3 Experimental Results

The estimated flexural tensile bond strengths of each bed joint within the wall specimen are presented in Fig. 4. An average strength of 0.48 MPa with a coefficient of variation (COV) of 0.30 was estimated. These values are consistent with the expected bond strengths of URM walls, such as those presented in McNeilly et al. [8]. In addition, the average strengths of each course within the wall are presented in Fig. 5. These values, ranging from 0.33 MPa to 0.63 MPa, are important to the determination of the spatial correlation coefficient (see Eq. 4). The observed spread of 0.30 MPa between these mean values may be significant (approximately 63% of the overall mean) but is not unexpected in the highly variable masonry unit-mortar interface.

In Fig. 4 it may be observed that two values have been omitted (the final tested units in courses 4 and 5). In this case, during the testing of the final unit of course 5, both of these bed joints, as well as the final perpend joint of course 4, failed. The subsequently recorded force at failure was disregarded, as this test result was deemed invalid. In addition, it may be observed that no flexural bond strength values are presented for the bottom course of units in Fig. 4. As these units were bonded to a reinforced concrete beam, rather than to another course of units, the resultant values of bond strength are expected to be inconsistent with the others presented in this study. As such, these values were also disregarded.

0.28	0.31	0.30	0.34	0.24	0.29	0.33	0.52	
0.56	0.35	0.28	0.46	0.39	0.40	0.36	0.36	0.48
0.38	0.32	0.61	0.69	0.53	0.31	0.36	0.35	
0.41	0.43	0.36	0.40	0.38	0.34	0.20	0.32	0.46
0.50	0.51	0.36	0.72	0.26	0.42	0.55	0.41	
0.35	0.56	0.58	0.42	0.58	0.33	0.43	0.60	0.56
0.54	0.45	0.64	0.84	0.55	0.27	0.36	0.53	
0.62	0.64	0.65	0.38	0.64	0.54	0.38	0.45	0.66
0.41	0.39	0.45	0.44	0.68	0.37	0.50	N/A	
0.63	0.68	0.43	0.56	0.53	0.47	0.47	0.21	N/A
0.59	0.45	0.65	0.58	0.60	0.48	0.62	0.64	
0.95	0.52	0.62	0.53	0.55	0.56	0.53	0.45	0.97
0.72	0.28	0.39	0.45	0.55	0.48	0.56	0.73	
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Fig. 4 Flexural tensile bond strengths estimated from wall specimen bed joints (all values are in MPa)

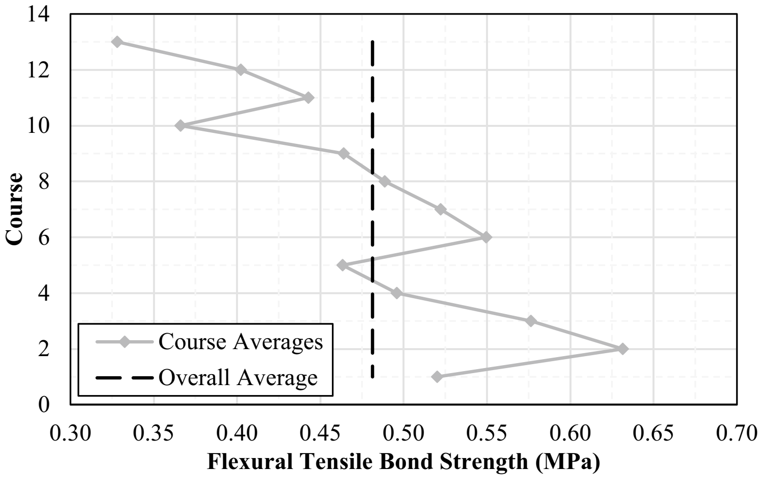

Fig. 5 Average flexural tensile bond strength of each course within wall specimen

3.1 Spatial Correlation of Adjacent Joint Strengths

The spatial correlation of flexural bond strengths was quantified using the autocorrelation function, ρ_k. This function is calculated for a "lag" k representing the space between data points (in this case the bed joints within a course). There are a number of estimates for the autocorrelation function, however, the bias estimate adopted in this study, is the most popular variance estimator and produces a smaller error than an unbiased case ([4] after, [2, 11]).

For a sample size N, the k^{th} autocorrelation function may be estimated from Eq. (4). Where z_i refers to the flexural tensile bond strength of the i^{th} joint within a course, and μ_z is the mean bond strength of that course.

$$\rho_k = \frac{\sum_{i=1}^{N-k} (z_i - \mu_z)(z_{i+k} - \mu_z)}{\sqrt{\sum_{i=1}^{N-k} (z_i - \mu_z)^2 \sum_{i=1}^{N-k} (z_{i+k} - \mu_z)^2}} \tag{4}$$

In addition to Eq. (4), Priestley [11] and Fenton [2] suggest that values of ρ_k that fall within the bounds of $\pm 2\sqrt{1/N}$ are not significantly distinct from statistically independent variables (i.e.: $\rho_k = 0$). In the case of the current study, this limit is equal to between 0.67 and 0.76 depending on the number of data points in the course under consideration. This range is quite large due to the relatively small number of units in each course, however, conclusions regarding the strength of the observed correlations may still be drawn despite this limitation.

The resultant autocorrelation function values, presented in Fig. 6, indicate that a weak correlation exists between adjacent bed joints ($k = 1$). The autocorrelation function value at this point is highly variable, with values of $\rho_{k=1}$ ranging from 0.70 to -0.61. While this result is not inconsistent with the findings of Heffler et al. [4] who observed values of $\rho_{k=1}$ between approximately 0.9 and -0.6, it reinforces the conclusion of a weak and variable degree of correlation between adjacent bed joint strengths presented in this previous investigation.

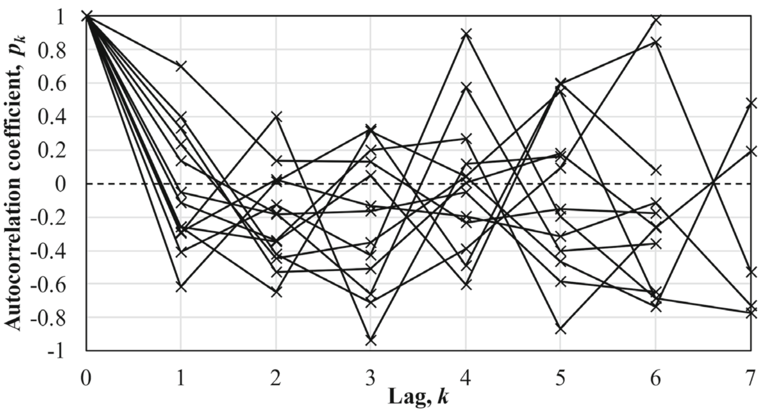

Fig. 6 Autocorrelation function values for each course in the wall specimen

As may be expected, the strength of this correlation diminishes at higher lag values. Despite this general trend, seen in the results of this study and that of Heffler et al. [4], there are significant fluctuations in the estimated values of ρ_k. Several large spikes or dips in ρ_k may be observed in Fig. 6, indicative of a very strong or positive or negative correlation. There are two likely causes of this variability. Firstly, despite the standard construction practice of placing units sequentially within a course, the first are last units of a course are often placed first. This is done to ensure that the wall remains flush at each end. As a result, it may be expected that a higher degree of correlation exists between joints strengths at the highest values of k. In addition, it was observed upon the completion of construction that the overall quality of workmanship of this wall specimen was low. This may be seen in part from the partially filled joints shown in Fig. 3. This characteristic does not explicitly relate to the spatial correlation of joint strength but would act to exacerbate the inherent variability of these relationships.

3.2 Relationship Between Wall and Prism Joint Strengths

In addition to the assessment of the spatial correlation of adjacent joint strengths, the suitability of the supplemental prism joint tests to represent the average strength of joints within the wall specimen was examined. This was achieved through the application of a t-Test, as described by Corrêa et al. [1]. This test found that, at a 5% significance level, the null hypothesis was rejected, i.e.: the tested prism joints do not accurately represent the statistical properties of the wall joint bond strengths.

This result, presented in Table 1, is consistent with the findings of Corrêa et al. [1] who found that the majority of URM prisms tested were not suitably accurate representations of their corresponding wall specimens.

In addition to the t-Test results, a comparison of the estimated mean strengths provides insight into the suitability of the prism joint strengths to reflect the properties of the wall specimen. In the case of this study, the prism joints underestimate the wall strength by approximately 14%. These values are conformant to the strength compliance requirements of AS 3700 [12] and are on the conservative side of those results presented by Corrêa et al. [1], who observed differences between wall and prisms mean strengths ranging from 3 to 270%.

Table 1 Summary of sample means, COVs and t-Test results

Wall			Prism			t-Test result (same population)
n	Mean (MPa)	COV	n	Mean (MPa)	COV	
108	0.48	0.30	15	0.41	0.25	No

4 Conclusions

The experimental results presented in this study indicate that a weak, but highly variable, correlation exists between the flexural tensile bond strengths of adjacent bed joints within an URM wall. The observed mean strength of 0.48 MPa with a COV of 0.30 are consistent with values observed in literature, and the determined values of an autocorrelation function are similar to those presented in previous, similar investigations. Further testing of larger, repeat wall specimens are planned in a future study, in order to refine the conclusions presented in this paper. The inclusion of a larger number of tested joints per course is expected to produce a more reliable estimate of the correlation coefficient presented in this study.

In addition to these findings, it was concluded through the application of a t-Test that the accompanying supplemental prism specimens constructed in concert with the tested URM wall did not accurately reflect the statistical properties of the wall. While a suitable estimation of the mean strength and COV was determined, as defined by the current Australian standard for masonry design, this observation suggests that care should be taken in the interpretation of similar characterisation tests.

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Seismic Assessment and Upgrade of Concrete Wharf Structure

F. Yu, X. Qin, and C. Wicaksana

Abstract The KiwiRail's Inter-island Resilient Connection (iReX) program aims to increase the capacity and efficiency of ferry between the north island and the south island of New Zealand. It introduces two new ferries and terminals at Kaiwharawhara in Wellington and Waitohi Picton. The terminal upgrade for the Picton port involving the construction of a new wharf and the upgrade of the existing wharfs. This paper presents the upgrade work for the largest wharf at the current Picton terminal. The upgrade involves partial demolition and replacement of the wharf deck. New pile supports will also be constructed for the new decks. A mechanical hinge bridge and automatic mooring unit will be installed on these new decks, enabling the operation of new ferries with various sizes. A seismic analysis was conducted to evaluate the seismic performance of the wharf before and after the upgrade work. It was acknowledged that the existing pile might fail in tension or compression due to the additional seismic force. Therefore, the design of the new members allowed the potential uplift movement of the existing piles such that all tension demand will be resisted by the new piles. A finite element model was constructed. Tensionless springs were used during the modelling of the existing piles. The seismic behaviour of the wharf and the evaluation of the wharf performance will be presented.

Keywords Seismic assessment · Wharf structures · Pile uplift

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1 Introduction

1.1 Background

Picton Interislander ferry terminal consist of 2 berths, Berth no 1 and Berth no 2. Currently Berth No 2 can only be used by Aratere, while the other three Interislander ferries, Kaitaki, Kaiarahi and Valentine are using Berth no 1. To facilitate the construction of the new berthing and terminal facilities, current berth No 1 needs to be demolished. To maintain ferry operations during the construction of the new berth, existing Berth No 2 required to be modified to enable all four ferries to use Berth no 2. Picton Enabling Work focused on the design of temporary structure and modifications of the existing structures to enable the existing Berth No 2 to be used for berthing for all four Interislander ferries.

There are several new structures proposed to be constructed as part of the Berth 2 upgrade, which includes a new Hinge Bridge, new foundation structures to support the new Automated Mooring Units (AMU), and a new foundation structure to support the existing short-arm end fender. Figure 1 shows a plan view of those proposed structures and their relative position to the long wharf. To allow these new structures to be constructed, some part of the existing structures required to be demolished.

The Hinge Bridge is a bascule bridge structure that enables vehicle loading and unloading of the K-class vessels (Kaiarahi and Kaitaki) and the Valentine onto the existing rail linkspan at Berth No 2. The bridge will be lifted in a stowed position when the Aratere is berthing and allow the vessel to access the existing rail linkspan within the nest, as in the original berthing configuration. The foundation of the hinge bridge is a portal frame structure with a concrete deck supported on 12 bore piles. The deck spans over two bays of the long arm wharf. The existing deck within the bays will be removed while the existing piles, kerbs and capping beams remain unchanged.

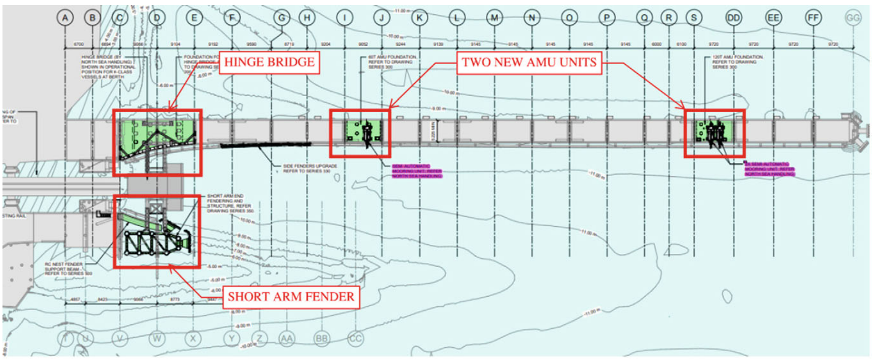

Fig. 1 Plan view of the new structures proposed (hinge bridge, AMUs and short arm fender)

Based on mooring analysis, two new AMUs are required to be installed at the end of the wharf. A new foundation structure at each location to seat the AMUs required. To facilitate the construction of the mooring unit foundations, the existing precast prestressed deck panel are to be removed and replaced with a new structure joining into the existing structure on either side.

A number of finite element analysis software, including SAP 2000, have been used to carry out static analysis of the structure based on the loadings applied and the nonlinear geotechnical springs defined. However, the modelling process requires a certain degree of knowledge in the integration of structural and geotechnical engineering field, as well as careful calibration of multiple parameters to obtain a more realistic results. This paper discussed how a more accurate estimate of the actual wharf structure behaviour and demands were obtained. Details methodology employed in the analysis were shared including the findings which may be referenced for any similar future projects.

1.2 Geotechnical Analysis

Prior to the structural analysis and design being undertaken, site-specific geotechnical investigations were carried out by external parties including borehole/CPT tests, lab analysis and geophysical investigation using Multi-channel Analysis of Surface Waves (MASW).

The majority of boreholes were drilled at the neighbouring wharves' locations. The ground conditions were inferred from those boreholes. The marine sediments were mostly comprised of soft silts and loose sands. Below the sediments were medium dense gravels and sandy gravels of alluvial outwash embedded in a clay/silt matrix. These materials generally become denser with the depth. The bedrock is anticipated at approximately −30.0 m RL at the coastline and indicatively dips down towards the northeast.

2 Methodology

2.1 Modelling

During the optioneering stage, an Oasys GSA model was established to allow an exploration of the structural geometry and adaption to the client demand. Later in the review stage, SAP2000 was used for nonlinear seismic analysis.

The structural members (such as beams, pile caps and piles) are modelled using beam elements. A grillage model was constructed for the concrete deck of the wharf. Reinforced concrete members are modelled with linear elastic material properties