Fatigue Behaviour of the Segmental Precast Concrete Deck Pre-tensioned with the GFRP Rods

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Abstract

This study investigated the cyclic behaviour of segmental precast concrete decks reinforced and pre-tensioned with GFRP rods simulating the effects of sea waves over their service lives. Large-scale decks with different levels of pre-tensioning and loading conditions were prepared and evaluated under a repetitive three-point flexural load. The results of the experimental tests including load-displacement behaviour, strain response, axial load, and failure mechanisms are presented and analysed. According to the results, the level of fatigue loading strongly impacts the behaviour of GFRP-reinforced monolithic concrete decks. It was shown that a monolithic deck with a reinforcement ratio of 1.68% could withstand one million cycles of wave loading at fatigue stress equal to $0.21M_u$ without showing any sign of cracking. However, increasing the fatigue stress to $0.3M_u$ caused failure after 20,585 cycles due to shear compression failure. By applying pre-compression stress of 1 MPa, the segmental precast concrete decks could achieve higher than 1,000,000 fatigue cycles without any failure but only a slight GFRP rod axial load relaxation. The outcome of this study enhances the understanding of the fatigue behaviour of the segmental deck with non-corrosive GFRP rods demonstrating their suitability for onshore maritime infrastructures.

Keywords: Fatigue, Pre-tensioned segmental deck, Precast concrete, GFRP, Cyclic behaviour

1. INTRODUCTION

The high costs associated with repairing and maintaining reinforced concrete structures with steel corrosion have increased the use of glass fibre-reinforced polymer (GFRP) bars. Studies have demonstrated the excellent performance of GFRP bars in harsh environments (Yang et al. 2024) makes them preferred reinforcing solutions for concrete structures located or near the marine environment (Manalo et al. 2021). Moreover, the high tensile strength-to-weight ratio of GFRP bars makes them an effective reinforcing system for concrete bridge decks (Ahmed et al. 2017) and bridge girders (Wang et al. 2024). However, the lower modulus of elasticity of GFRP than steel results in a comparatively higher deformation which can be problematic in segmental systems (Sheikh and Kharal 2018). Although using CFRP tendons with a high modulus can potentially be a suitable solution; their high cost makes it uneconomical for use in most applications. Applying pre-tension to the GFRP reinforcing systems may effectively alleviate this issue by providing additional compression on the segmental deck and increasing the stiffness.

Depending on their application, concrete structures experience fatigue from the repetitive motion of sea waves acting on them over their service lives in the maritime environment or from traffic loading on the bridge. Therefore, understanding fatigue behaviour caused by exposure to cyclic loading is essential to GFRP-reinforced concrete marine structures. A case study on the segmental bridge deck with a post-tension internal steel tendon (Ynys-y-Gwas bridge) showed that the fatigue effect in the long term may result in bridge collapses (Al-Mosawe et al. 2022). Fortunately, no fatalities were reported in this incident. In 2018 however, the Morandi's Polcevera Viaduct in Italy collapsed resulting in 43 deaths and 9 injuries (Morgese et al. 2020). Between 1980 and 2012, 1062 bridge failures were reported in the United States (Lee et al. 2013) where about 60% of the total failures were caused by fatigue or the indirect effect of fatigue. A prominent example of this is the collapse of the I-35W bridge

Peiris et al. (2006) found that fixed supports extended fatigue life by reducing mid-span bending moments, while Gao et al. (2024) introduced a conversion factor of 0.59 to translate fatigue life from three-point bending to rolling loads. Higher reinforcement ratios improved stiffness and fatigue life (Ali et al. 2024), but selecting a higher upper fatigue load significantly reduced the fatigue life. GFRP-RC decks showed superior (El-Ragaby et al. 2007) or comparable (Van Cao 2024) fatigue performance to steel decks. After 2 million cycles, the flexural strength only reduced by 1.6% for monolithic GFRP-RC beams (Januš et al. 2021), though Ali et al. (2024) highlighted the sensitivity to loading protocols. Nagy et al. (2024) found that ribbed GFRP bars provided better fatigue life than sand-coated bars and recommended stress limits for concrete due to creep rupture concerns.

Fatigue in monolithic decks due to the arching effect might not be the main concern (CSA 2019), as also demonstrated from available experimental works, which investigated the fatigue behaviour of monolithic steel and GFRP-RC decks. However, the structural performance of segmental construction systems heavily depends on the stress level of the GFRP rod and the concrete's behaviour at the joint— an area with limited understanding. This is especially critical for post-tensioned, unbonded GFRP rods, which lack composite action with the surrounding concrete. Understanding of the fatigue behaviour is important as a segmental concrete system can provide many advantages, including an accelerated construction process, facilitated transportation and handling, and the ability to repair in future incidents. Noël and Soudki (2014) highlighted that bond-slip between the reinforcement and concrete, combined with stress gradients in the bar, can cause premature GFRP failure compared to bare bars tested under unbonded conditions. Furthermore, the creep behaviour of GFRP bars has led to a negligible degradation in tensile strength after 10000 hours (Youssef and Benmokrane 2011) but a 20% increase in GFRP stress due to creep and shrinkage when exposed to long-term compression (Bujotzek et al. 2024).

This study addresses the knowledge gap on the cyclic behaviour of the monolithic/segmental GFRP-RC decks. This focuses on evaluating the fatigue performance of large-scale monolithic and segmental decks until failure. A single deck with a different fatigue load level was initially tested, and its post-fatigue failure behaviour was evaluated under static loading. This was followed by testing two decks connected by GFRP rods and exposed to cyclic loading. Different levels of post-tensioning in the GFRP rods were applied and subjected to cyclic loading until failure. Pre-compression stress of 1 MPa on the concrete has been used to evaluate the different loading frequencies and the performance of the pre-damaged segmental deck. The outcomes of this study will provide a better understanding of the fatigue behaviour of segmental concrete construction systems reinforced by GFRP bars that can be used for their effective and safe design under the repetitive actions of waves in marine environments.

EXPERIMENTAL PROGRAM

Design criteria for segmental decks

The segmental concrete decks were designed to meet the criteria set by the Queensland Department of Transport and Main Roads (DTMR) for floating walkways and pontoon infrastructure (DTMR 2015) and New South Wales guidelines (NSW Boat Ramp Facility Guidelines 2015). These guidelines were applied alongside the relevant Australian Standards, including AS 3600 Concrete Structures (AS 2018) and AS 4997 Maritime Structures (AS 2005), to ensure the segmental decks met both structural performance and durability requirements. The segmental decks were constructed following the DTMR's precast concrete member criteria (DTMR 2019) to achieve a design life of up to 50 years. Each deck was designed for a Class 5 deck load classification under (AS 2005), accounting for a uniformly distributed live load of 5 kPa and a concentrated load of 20 kN. The segmental deck is designed based on the concentrated load of 5 kN, typical for the RC boat ramp (NSW Boat Ramp Facility Guidelines (2015)). Concrete with an average compressive strength of 37 MPa and with a standard deviation of 6.2 MPa was utilised to manufacture the segmental decks that adhered to the AS3600 (2018) exposure classification C2 (near seawater).

Material characteristics

The concrete segments were reinforced with Grade III (#4) GFRP bars, having a nominal diameter of 12.7 mm. A threaded GFRP bar, called a GFRP rod in the present research, as shown in Fig. 1 (right), was used to connect the precast concrete segments.

The system for anchoring consisted of a threaded stainless-steel tube with a steel nut on each end and the incorporation of the steel plate between the tube and concrete, as illustrated in Fig. 1 (left). The thread of the GFRP rod and the internal thread of the steel tube provide an interlock for applying the tension to the segmental deck. After placing the rod into the precast concrete segments, the steel nuts were tightened with a wrench to post-tension the sample. Table 1 provides a summary of the properties of the GFRP bars, GFRP rods and concrete.

Specimens detail

The concrete segments with dimensions of 1000 × 600 × 125 mm (length × width × thickness) were assembled and tested. The width and thickness are determined following the outlined dimensions in (DTMR 2015), simulating the structural components for floating systems. The segment's length of 1000 mm was taken as the typical modular length of the pontoon. The internal GFRP reinforcements are spaced transversely (250 mm) and longitudinally (150 mm) and result in reinforcement ratios ($\rho_f = \frac{A_f}{bd}$) of 1.01% and 1.68% per segment, respectively. The balanced reinforcement ratio of the prestressed concrete section (Eq. 1), following the (ACI 2004) was in the range of 0.24% to 0.27%, where ε_{fe} is the strain in the FRP rod by the initial prestressing and the maximum compressive strain of the concrete, ε_{cu}, is taken as 0.003 following the (ACI 2015). The internal reinforcement did not contribute to the load-carrying capacity (Ebrahimzadeh et al. 2024) of the segmental decks because of the discontinuity at the joint. Considering just the presence of the GFRP rod, the segmental deck's reinforcement ratio (ρ_f) is equal to 2.3%, showing the segmental deck is over-reinforced. Following equations 2 – 6 and based on (ACI 2015), the cracking and ultimate moment of the GFRP-RC deck are 6.85 kN-m and 41.51 kN-m, respectively.

$$\rho_{fb} = 0.85\beta_1 \frac{f_c^{'}}{f_{fu}} \times \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu} - \varepsilon_{fe}} \qquad \text{Eq. 1}$$

$$\alpha_1 = 0.85 \qquad \text{Eq. 2}$$

$$\beta_1 = 0.85 - \frac{0.05(f_c^{'} - 27.6)}{6.9} \geq 0.67 \qquad \text{Eq. 3}$$

$$f_r = 0.62\lambda\sqrt{f_c^{'}} \qquad \text{Eq. 4}$$

$$M_{cr} = f_r \times \frac{I_g}{y_t} \qquad \text{Eq. 5}$$

$$f_f = \sqrt{\frac{(E_f \varepsilon_{cu})^2}{4} + \frac{0.85\beta_1 f_c^{'}}{\rho_f} E_f \varepsilon_{cu}} - 0.5 E_f \varepsilon_{cu} < f_{fu} \qquad \text{Eq. 6}$$

$$M_u = \rho_f f_f b d^2 (1 - 0.59 \frac{\rho_f f_f}{f_c^{'}}) \qquad \text{Eq. 7}$$

A total of fourteen decks were manufactured and tested. Table 2 demonstrates the tested specimens, including details such as the number of segments (1S for one segment and 2S for two segments), the type of loading (S for static, C for cyclic, and D for the displacement mode-controlled loading in a fast rate), the post-tension level (indicated by the number following PT), and any special features that describe the different segments. For instance, the designation "D250k" refers to a damaged specimen tested after 250,000 cycles. Adjustments in loading frequency or maximum upper fatigue load are specified as 0.5 or 1 Hz (2S-C-PT36-1Hz) and 30 or 40 kN (1S-C-40). It needs to be mentioned that the monolithic terminology is chosen as opposed to the segmented system.

Loading protocol

Section 4.4 of the guideline for prestressed concrete structures with FRP tendons in ACI (2004) and AASHTO (2018) indicated that fatigue in monolithic beams post-tensioned/reinforced with FRP is unlikely to be a problem in uncracked members; hence, no standard procedure is recommended. CSA (2019) similarly has no recommended testing protocol to evaluate the fatigue behaviour of FRP-reinforced concrete elements. The only recommendation available is to avoid creep failure by limiting the sustained load to less than 40% as per ACI (2004) and 25% (AASHTO 2018; CSA 2019) of the design tensile strength of GFRP reinforcement. Section 4.3.2 of ACI (2015) provides information on fatigue behaviour without a standard loading/testing protocol, while Annex J of CSA S806 (CSA 2021) offers a test method for tensile fatigue of FRP reinforcing bars and rods. This study conducted experimentation in force-control mode as suggested by CSA S806 (CSA 2021). The maximum design load of the segmental decks was 5 kN, which was adopted from the NSW Boat Ramp Facility Guidelines (2015).

The maximum fatigue load was calculated based on the cracking moment formulation provided by ACI (2015) and CSA S806 (CSA 2021) for monolithic concrete segments. The ratio of the minimum to maximum applied load of 0.1 was based on CSA (2021). Although CSA (2021) recommended 4 million cycles of testing, a maximum of 1 million cycles per test was adopted from CSA S807 (2019) because of time limitations, but this was deemed sufficient to understand the fatigue behaviour of the segmental decks' floating maritime infrastructure, as suggested by the DTMR.

Test setup and instrumentation

The segmental decks were subjected to 3-point static loading, and the load was applied through a spreader steel beam situated 150 mm from the joint (Fig. 2). The applied load and displacement were measured with the MTS hydraulic actuator and the displacement transducer, respectively. A load cell was positioned between the steel nut and steel plate to measure the initial post-tension load, and the rod's axial load increment during the test was recorded using a data acquisition system. For static tests, a digital image correlation (DIC) was utilised to capture the deflection along the length of the deck. In a single deck, 3 mm-long electrical resistance strain gauges, positioned at mid-span and in the vicinity of the support, attached to the internal reinforcement (both tension and compression), have been used. The test setup and configuration create a span-to-depth ratio of 3.8. Note that the test setup was carefully designed to capture the critical structural responses of monolithic and GFRP-RC segmental decks, as only one deck was tested per specimen type because of the time to complete each test.

BEHAVIOUR OF MONOLITHIC DECK

The effect of the cyclic load on the monolithic concrete segment in terms of load-carrying capacity, strain behaviour, and failure mechanism in cyclic and post-cyclic stages was investigated and discussed in the following section.

Effect of the levels of cyclic load

The effect of the cyclic loading on the monolithic GFRP-RC deck is evaluated by analysing the behaviour of specimens loaded under 30kN (1S-C-30) and 40kN (1S-C-40). The load-displacement and strain behaviour of these specimens are presented in Fig. 3.

Load-displacement behaviour

A 1.7% reduction in flexural stiffness in deck 1S-C-30 after being subjected to 1 million of cyclic loading is observed (Fig. 3a; Fig. 4). According to ACI (2015), the cracking moment can be predicted by ($\frac{0.62\lambda\sqrt{f_c^{'}}I_g}{y_t}$), where f'_c, I_g, and y_t are the concrete compressive strength, the gross moment of inertia, and the distance from the centroidal axis of the gross section, respectively. λ is the reduction factor for lightweight concrete, but is equal to 1.0 for normaldensity concrete. The 30 kN applied load creates a cracking moment of 6 kN-m for this section, equivalent to $0.21M_u$. However, the deck only cracked at 8 kN-m, equivalent to $0.3M_u$, after 20,584 cycles, indicating that the ACI (2015) provides a conservative estimation of the cracking moment. The higher cracking moment is because the sample has a span-to-depth ratio of 3.8, which created arch action and facilitated the load path for the uncracked concrete section to transfer the load to the support. Hence, the predominant mechanism of the shear load transfer was arch action throughout the uncracked section and the aggregate interlock. In contrast, the contribution of the GFRP rebar for its dowel action was minimal. This was previously observed by Li et al. (2021), as their FRP-RC beams shared identical cross-sectional areas. The difference in the experiment and calculated cracking moment was only related to the modulus of rupture of concrete, not the reinforcing arrangements. They also concluded that both CSA (2021) and ACI (2015) underestimate the cracking moment, as in this study. It also needs to be mentioned that, based on the result of Kim and Park (1996), when the concrete is reinforced with steel rebar, the shear force is transferred by the uncracked concrete of the compression zone (30-40%), the interlocking action of aggregates (33-50%); and the dowel action (15-25%). Accordingly, the dowel action is contributing, but its role is not as significant as that of the uncracked section and the aggregate interlock. This was confirmed by Barris et al. (2017), where the cracking load of the GFRP-RC beam (2 – 16 mm diameter) was only between 0.1 and 6% (depending on the parameter they investigated) lower than the steel-reinforced beam (2 – 12 mm diameter) with the same cross-sectional area.

The upper fatigue load of the 1S-C-40 was 33% higher than the 1S-C-30. It needs to be mentioned that the stiffness of the 1S-C-40, at first cycle, was 20% higher than 1S-C-30. The use of a rubber pad beneath the load spreader in 1S-C-30, which was removed in subsequent tests, may have contributed to this discrepancy. While this may have influenced the stiffness measurement in the first cycle, it does not affect the overall behaviour; however, it needs to be refined in future testing.

The applied load of 40 kN, resulting in 0.3 of the ultimate flexural capacity, is an acceptable threshold for the service load of a one-way GFRP-RC slab (El-Nemr et al., 2018). Nevertheless, it is suggested that the GFRP-RC deck used in maritime infrastructures should not be loaded exceeding $0.21M_u$ to ensure minimal reduction in strength and stiffness at a 1 million fatigue cycle. The initial shear crack (Fig. 5a), on both sides (Fig. 5b), was observed at 20,584 cycles (Fig. 3b), which caused the stiffness to decrease from 55.1 kN/mm to 36.62 kN/mm (Fig. 4), a 33.6 % reduction. This observation also showed that for decks with a span-to-depth ratio of 3.8, the constant of the cracking moment formula (0.62) can be increased up to 0.82. A reduction in fatigue life (in a monolithic hybrid GFRP/steel RC beam) has been observed in a similar study (Zhu et al. 2017), as the upper fatigue load increased from $0.5P_u$ to $0.6P_u$, and the fatigue life decreased from 1.354 to 0.461 million cycles. From cycle 20,585 to 500,000, the stiffness steadily decreased from 36.62 kN/mm to 29.12 kN/mm. In this cracked phase, the reinforcement's dowel action resists the crack opening and arch action through the uncracked section, providing a loading path to the support, unlike the uncracked stage in which the dowel action plays a less significant role. The cyclic loading causes the crack to propagate to the loading point (red line in Fig. 5a) gradually, which causes the shear resistance and stiffness reduction. The stiffness reduction from cycle 500,000 to cycle 750,000 was gradual and about 10% (Fig. 4), associated with a slight crack propagation. The difference between this stage and the previous one is the lower rate of stiffness reduction and crack development. After this, the stiffness was almost steady. Accordingly, stiffness reduction and the deck's fatigue behaviour can be divided into three stages (gradual or pre-cracking, sudden cracking, and gradual or post-cracking stage) in which the stiffness degrades, as described by ACI (2021). In the case of cyclic loading with a constant upper fatigue load, the fourth and fifth stages can be added to this, which indicate the lowered stiffness reduction and the steady stages, and the latter reflects the stopping in the crack development, coinciding with the opening/closing of the formed cracks.

Strain behaviour

The strain behaviour was demonstrated through the gauge attached to the bar in compression (MC, mid-span in compression zone) and tension (MT, mid-span in tension zone) in the mid-span of the concrete segment, as well as the gauge attached to the bar in the tension close support in the tension zone (ST referred to the location close to support and in the tension zone). The 1S-C-30 was uncracked during the cyclic loading; hence, the strain behaviour of 1S-C-30 showed a negligible contribution of the GFRP rebar on the load-carrying capacity (Fig. 3c), 200 $\mu\varepsilon$ or lower than 0.95 % of the ultimate tensile strength based on the reported bar engineering characteristic (Ebrahimzadeh et al., 2024). Since the deck's behaviour was linear (uncracked throughout the 1 million cycles), the strain's fluctuation (caused by the crack's opening/closing) was minimal (Fig. 3c).

The strain behaviour of 1S-C-40 was measured through the first 100,000 cycles and was similar to that of 1S-C-30 until cycle 20,584, with a minimal increase and fluctuation. As soon as the initial crack formed, the strain on the bar in tension (MT and ST) increased, showing the deck's behaviour becoming non-linear. The measurement of MT suddenly increased to 500 $\mu\varepsilon$ (2.4 % of the ultimate tensile strength) and fluctuated between 400 $\mu\varepsilon$ to 600 $\mu\varepsilon$ (Fig. 3d). This sudden increase justifies a sudden reduction in stiffness. The steady increase in the bar's strain and steady increase of the fluctuation shows the gradual stiffness degradation and the crack propagation which is associated with the diagonal shear crack reaching the loading point (Fig. 5a). A similar behaviour was observed with ST with a lower strain increase and lower fluctuation through the cyclic loading, proportional to distance (crack formed 255 mm from the support and 145 mm from the loading point). The strain behaviour of the GFRP bar shows a continuous increase in the post-crack stage, indicating that once cracking occurs, the reinforcement starts contributing to load transfer. The dowel action primarily resists transverse shear resistance across the crack, while the longitudinal reinforcement continues to resist crack widening through tensile forces. The MC strain increase was minimal (-150 με) and did not reach the range of 3000-3500 με associated with concrete crushing (ACI 2015; CSA 2021).

Failure behaviour

The span-to-depth ratio of 3.8 can make the deck fail in the diagonal shear failure (Kani 1964). The deep ($0 < a/d < 1$) and short ($1 < a/d < 2.5$) beams typically fail due to tied arch (anchorage failure or crushing of the support) and shear compression, respectively. With a span-to-depth ratio higher than 6, failure is governed by flexure rather than shear (Mak & Lees, 2023). The a/d ratio of 3.8 is in the range of shear failure ($2.5 < a/d < 6$), which can be identified as a transition range from the shear to flexural shear cracks (Muttoni & Fernández Ruiz 2008; Kani 1964). Accordingly, it can be concluded that while 1S-C-30 did not crack, 1S-C-40 suffered from the shear compression crack.

The post-cyclic behaviour

Load-displacement behaviour

After 1 million cycles of cyclic loading, deck 1S-C-40 was subjected to static loading (1S-S-D1M) (Fig. 6a). The initial stiffness of the deck 1S-S-D1M was 29.1 kN/mm (same as the last cycle of 1S-C-40) which is 79.3% lower than the static testing of the intact deck's, 1S-S (140.3 kN/mm). This was due to the presence of the initial diagonal shear crack. Hence, the GFRP-RC monolithic deck loading should not exceed $0.21M_u$ to ensure strength/stiffness retention at 1 million fatigue cycles. The initial crack caused the deck to show a non-linear behaviour from the beginning, compared to the linear behaviour of the 1S-S in the pre-crack stage. This non-linear load-displacement behaviour can be caused by the formed crack in the tension zone of the concrete deck segment. At the applied bending moment of $0.5M_u$, a second shear crack was developed closer to the support, which significantly dropped the load-carrying capacity of the deck. In this stage, the strain gauge attached to the bar in tension close to the support (ST) and in the mid-length (MT) started to measure strain as shown (Fig. 6b). The stiffness in 1S-S-D1M was lower than 1S-S, possibly because of the cracks formed throughout the depth of the deck.

Strain behaviour

The strain behaviour of the MT at the 1S-S-D1M explains the deck's non-linear behaviour under static test. It increased from the beginning of the loading because of the presence of the crack. This shows that once cracks form in the deck, the load increases non-linearly due to the widening of that crack. The strain on the ST did not increase up to 71 kN, $0.5M_u$. After this stage, a drop in the load-carrying capacity (Fig. 6a) and the load strain (Fig. 6b) is observed. In this stage, the shear crack formed, and both MT and ST's strain increased up to 3500 $\mu\varepsilon$ (16.6% of the ultimate tensile strength). Before $0.5M_u$, the strain on the MT was higher than ST, but after this load, the strain in both gauges increased equally, showing the failure mechanism is governed by the shear crack forming between the loading point and support.

Failure behaviour

The failure mechanism of both 1S-S and 1S-S-D1M included the initiation of the shear compression cracks in between the support and loading point, and ultimate failure was governed by shear failure due to the span-to-depth ratio of 3.8.

BEHAVIOUR OF THE SEGMENTAL DECK

The effect of the cyclic loading on the load-carrying capacity, the rod's axial load increment-deflection behaviour, and the failure behaviour of the segmental concrete decks was investigated. The influence of the exposure to cyclic loading and the post-cyclic behaviour, post-tension forces (testing up to failure), loading frequency, and the presence of the initial damage on the fatigue behaviour of the segmental deck are discussed.

Effect of the cyclic loading

Loading to 250,000 cycles

The effect of the cyclic loading on the segmental deck with a hand-tight level of post-tensioning was investigated by analysing the 2S-C-PT0-250k subjected to 250,000 cycles. The post-cyclic behaviour of this panel was then evaluated by conducting 3-point static testing (2S-S-PT0-D250k) and compared to the load displacement of the intact deck, 2S-S-PT0.

Load-displacement behaviour

The segmental deck under cyclic loading within a range of 0.5 to 5 kN, equivalent to 1.8 to 18% of ultimate flexural strength, behaved linearly with a lower stiffness compared to a monolithic one. The stiffness of the segmental deck decreased from 0.6 kN/mm to 0.43 kN/mm (28.3 % reduction) after 250,000 cycles and caused a permanent deflection of 6 mm (Fig. 7a). The measured stiffness reduction and the permanent deflection are caused by the concrete crushing in the joint (Fig. 7e). The cyclic loading and unloading also caused a constant interaction between the rod and concrete in the joint. It should be noted that the interaction caused by the cyclic loading and unloading induced repeated relative movement between the segments at the joint. Additionally, the top part of the GFRP rod remained in contact with the concrete segment at the joint. Ebrahimzadeh et al. (2024) showed that the stress concentration in the vicinity of the joint at the segmental system can cause interlaminar shear of the GFRP rod. Continuous bending/ unbending due to cyclic loading causes high friction between the rod and concrete at the joint (interface between the rod and concrete), resulting in the wearing of the resin-rich surface of the rods. Also, the cyclic exposure of the rod surface to the sharp edge of the PVC/concrete might cause extra wear.

Rod's axial load behaviour

The rod's axial load fluctuated between 12 kN (loading) and 4 kN (unloading) (Fig. 7c). Axial load of 12 kN in the rod is equivalent to stress of 26.6 MPa, which is 3.65% of the shearing off the strength of the end anchor strength as evaluated by (Ebrahimzadeh et al., 2024). As the loading continued, the increment on the rod axial load increased by about 2 kN (16% increase), in a way the fluctuation of the cycle 250,000 was mainly around 14 kN, a 4.26 % of its ultimate anchor strength, which was caused by the concrete crushing at the joint and imposing a higher deflection.

The post-cyclic behaviour

Load-displacement behaviour

Both 2S-S-PT0 and 2S-S-PT-D250k behaved linear-elastic (Fig. 7b); however, an exposure of the hand-tight segmental deck to the 250,000 cycles of fatigue load of 0.18 M_u reduced stiffness by 28.3 % due to initial concrete crushing at the joint. With more than 5 kN applied load, 0.18 M_u, the stiffness of the 2S-S-PT-D250k deviates from 2S-S-PT0. The concrete in both decks after 0.36 M_u is completely crushed. Hence, the reason for the lower stiffness of 2S-S-PT-D250k is caused by the degradation of the GFRP rod due to fatigue. Interestingly, the behaviour of the 2S-S-PT0-D250k became similar to that of 2S-S-PT0 after the drop in load-carrying capacity. The drop in 2S-S-PT0 was caused by the interlaminar shear of the rod and similar behaviour shows cyclic loading causing the interlaminar shear to occur in a lower load (Fig. 7f). The interlaminar shear of 2S-S-PT0 occurred in an applied load of 27.55 kN (equal to M_u), but in 2S-S-PT-D250k it initiated in 0.46M_u.

Rod's axial load behaviour

The rod's axial load for deck 2S-S-PT0-D250k increases (Fig. 7d) as the applied load increases and the deflection increases. This increases the beam's curvature, which causes an increase in the strain and consequently the increase in the GFRP rod's axial load. When the deck deflected by 65 mm, the axial load reached 72 kN or 16.6 % of the anchor strength showing cyclic load can cause GFRP rod failure without reaching its ultimate tensile capacity. The failure of a GFRP rod before reaching its ultimate nominal strength can occur for two reasons. First, in the mechanism of the rod, the fatigue loading leads to interactions between unidirectional fibres, causing delamination and degrading the rod's properties at the joint, where it is continuously bent and unbent. The second is due to an external contact, as the friction at the concrete-rod interface at the joint causes wear and generates heat. Due to the low heat conductivity of GFRP (0.39 W/m. K (Su et al. 2020) compared to 45 W/m. K in steel), this heat tends to concentrate at the joint rather than disperse, contributing to extra surface degradation. This also explains why increased loading frequency can reduce fatigue life, as a reduction in stiffness is observed in section 4.3.

Failure behaviour

Concrete crushing at the joint is observed in both decks (2S-S-PT0 and 2S-C-PT0-D250K), as shown in Fig. 7e. Deck 2S-S-PT0 also suffered from the interlaminar shear failure of the GFRP rod at the interface of the concrete and rod because of the high deflection and the radius of curvature. However, the 1S-S-PT0-D250K, due to the cyclic loading, failed at a lower level of load/deflection. Accordingly, cyclic loading on a segmental concrete deck with the hand-tight post-tensioned GFRP rod causes the rod's interlaminar shear to change to rod rupture (Fig. 7f) in a lower load-carrying capacity.

Effect of the post-tensioning of the GFRP rod

The effect of the post-tension of the GFRP rod was evaluated by comparing the behaviour of decks 2S-C-PT0, 2S-C-PT18, and 2S-C-PT36-0.5HZ. Figure 8 shows the load-deflection behaviour and the axial load developed in the rod with cyclic loading.

Load-displacement behaviour

The hand-tight segmental deck failed at 401,892 cycles (Fig. 8a), while 2S-C-PT18 failed at 498,942 cycles (Fig. 8b). This shows that even a small precompression stress of 0.48 MPa, applied to the concrete at the joint, can increase the fatigue life of the segmental deck by 24.1%. This is because the post-tension force increases the neutral axis depth and has a deeper compression depth between the segments, providing higher resistance against the applied load. This result shows that although the transverse shear resistance of the GFRP bar/rod is lower than steel, and the segmental system relies on the frictional interaction at the joint, pre-compression at the joint can mitigate stress concentrations and enhance shear resistance.

The initial pre-compression stress of 0.98 MPa is needed to reach 1 million cycles (Fig. 8c). As explained by Ebrahimzadeh et al. (2024), neutral axis depth can be calculated in the form of $\frac{2A_f(f_f + f_{fe})}{0.85 f_c^{'} b \beta_1}$, knowing ($A_f f_{fe}$) is the applied initial post-tensioning and ($A_f f_f$) is the axial load on the rod. $f^{'}_c$, b, and β_1 are 37.1 MPa, 600 mm, and 0.78, respectively. Accordingly, the neutral axis depth of 2S-C-PT0 and 2S-C-PT18 when the maximum fatigue load applies to the deck is 3.24 mm and 5.04 mm, a 35.7% increase. This shows that increasing the initial post-tension force can increase neutral axis depth leading to an increase in initial stiffness. Correspondingly, deck 2S-C-PT36-0.5Hz exhibited the highest stiffness (5 kN/mm), deflected the least, and exceeded 1 million cycles of fatigue loading without any failure. Moreover, increasing the post-tension force provides higher resistance, which causes a lower curvature. This reduces the stress concentration on the GFRP rod. The result of Zhu et al. (2017) also indicated that the fatigue life increases as the effective reinforcement ratio increases. When the reinforcement ratio increased from 1.12% to 1.75%, the fatigue life increased from 1.354 to 1.678 million cycles. El-Ragaby et al. (2007) concluded that the fatigue life of the GFRP-RC beam is three times higher than steel-RC beam. The better fatigue performance of the purely GFRP-RC compared to steel-reinforced, with almost the same reinforcement ratio, can be attributed to the lower modulus of elasticity of GFRP, which is closer to concrete and the high strain to failure. These unique properties of GFRP bars cause the GFRP-RC beam to behave more homogenously and provide better performance under fatigue loading. However, in segmental and hybrid concrete decks, stiffer beam imposes less deflection and provide higher fatigue life.

The reduction in stiffness with loading cycles showed a linear trend (Fig. 9a). This can be explained by the linear elastic behaviour of GFRP rods and the linear increment of the axial load on the rod. The stiffness in 2S-C-PT36-0.5Hz decreased linearly and gradually from 5.05 kN/mm to 4.81 kN/mm (4.3% reduction). The stiffness in 2S-C-PT18 decreased from 0.66 kN/mm to 0.48 kN/mm (cycle 450,000) gradually (27.2% reduction). The segmental deck suffered a reduction to 0.39 kN/mm (at the last cycle of 498,942), associated with a sudden increase in the permanent deflection (Fig. 9b). The stiffness in 2S-C-PT0 reduced gradually until the very last cycles (from 0.6 kN/mm to 0.37 kN/mm). From cycle 401,000 to 401,892 (892 cycles), it reduced to 0.2 kN/mm and then failed. This stiffness behaviour showed two points: firstly, the 2S-C-PT36, due to the higher neutral axis depth, did not deflect significantly (the opening of the joint was limited), and the stiffness reduction was minimal. Secondly, having an initial pre-compression stress, whether sufficient (2S-C-PT36-0.5Hz) or not (2S-C-PT18), reduces the stress on the GFRP rod due to the lower decks' curvature and improves fatigue life. It needs to be mentioned that sufficient pre-compression means that at least the 1MPa compression is applied on concrete at the joint.

Rod's axial load behaviour

The deflection is higher, and the rod's axial load in the anchor fluctuates between the higher magnitudes when the segmental deck's stiffness is lower. Rod's axial load in the hand-tightened segmental deck fluctuated between 4 kN and 12 kN (Fig. 8d). In comparison, in 2S-C-PT18 (Fig. 8e) this fluctuation reduced to 5 kN (15 kN < rod's axial load< 20 kN), due to less deflection imposed to the segmental deck causing less curvature which causing a less strain on rod and eventually smaller magnitude of fluctuation of the axial load. The rod's axial load of the deck 2S-C-PT36-0.5Hz (Fig. 8f) was measured over three intervals (100,000 cycles per interval and at the beginning, middle, and end of the testing). The higher stiffness caused the deck's behaviour to become stiffer and the rod's axial load not to fluctuate (in each cycle of loading and unloading) in all intervals (Fig. 8f). In summary the rod's when post-tension stress increases the axial load fluctuation decreases due to higher stiffness and lower imposed deflection.

Failure behaviour

The failure mechanism in a segmental deck with a post-tensioned GFRP rod is initiated at the joint with concrete crushing. Increasing the pre-compression stress imposed on concrete at the joint from 0.48 MPa (Fig. 10a) to 0.98 MPa (Fig. 10b) increased the neutral axis depth. This was observed in the static testing of the segmental deck and numerical modelling (Ebrahizmadeh et al., 2024a). The ultimate failure of the decks with the pre-compression stress of 0.05 MPa and 0.48 MPa is caused by the rupture of the GFRP rod (Fig. 10a); however, having 0.98 MPa compression stress changed this into interlaminar shear in the post-cyclic stage (2S-S-PT36-D1M) (Fig. 10b). Accordingly, the bar did not rupture even under a higher load at the static testing, as the 0.98 MPa compression stress provided enough fatigue resistance for the deck to guarantee the minimal strength degradation.

Load-displacement behaviour in the post-cyclic stage

2S-C-PT36-0.5Hz after exposure to 1 million cycles was subjected to static loading, 2S-S-PT36-D1M (Fig. 11a) and compared with static testing of an intact segmental deck with an initial post-tension force of 32.8 kN on each rod, 0.85 MPa compression stress on the joint. Both decks showed linear elastic behaviour up to an applied load of 10 kN, equating to 0.32 M_u, and an almost similar stiffness. After this point, the deck's non-linear behaviour is induced by crushing the concrete at the joint.

Rod's axial load behaviour in the post-cyclic stage

The axial load-displacement behaviour of the rod (Fig. 11b) showed that when the pre-compression stress of 0.98 MPa was provided, resistance to avoid concrete crushing at the joint, hence, the fatigue was not affecting the post-cyclic behaviour. Due to around 1 MPa concrete pre-compression at the joint, the appropriate fatigue resistance causes the rods' axial loading behaviour (load increment-displacement) between the intact deck and the deck in the post-cyclic stage to be identical.

Effect of the loading frequency

Adimi et al. (1998) investigated the effect of the loading frequency on the fatigue behaviour of the CFRP rebar embedded in concrete. In their study, a minimum-to-maximum stress ratio of 0.1 and a maximum stress of 50% of the initial strength resulted in more than 400,000 cycles of the fatigue life with a loading frequency of 0.5. Their result showed that the fatigue life decreases as the loading frequency increases in the range of 0.5 to 8 Hz. This can be attributed to the higher friction and abrasion between the concrete and rebar as the loading frequency increases, causing debonding between the concrete and internal reinforcement. In a segmental deck, this can potentially cause extra wear in the vicinity of the joint as the rod and concrete constantly come in contact with each other. In the current study, a comparison within the range of 1,000,000 cycles is conducted by increasing the loading frequency by 100% (1 Hz).

Load-displacement behaviour

Decks 2S-C-PT36-0.5Hz (Fig. 8c) and 2S-C-PT36-1Hz (Fig. 12a) have an almost identical initial bending stiffness (5 kN/mm), and both endured 1,000,000 cycles. The stiffness in 2S-C-PT36-0.5hZ decreased gradually to 4.8 kN/mm at cycle 1,000,000 (Fig. 13a). Before cycle 900,000, the stiffness reduction in both decks is comparable. This gradual and negligible reduction can be attributed to the abrasion of the segments at the joint. When the loading frequency increases, the stiffness reduces afterwards in deck 2S-C-PT36-1Hz. As a result, the permanent deflection of the segmental deck increased to 0.16 mm after 9,000,000 cycles, while in 2S-C-PT36-0.5Hz, it was steady at 0.1 mm (Fig. 13b). The permanent deflection was caused by an axial load relaxation of the rod. According to the previous sections, when the concrete crushes in the joint due to an increase in the curvature and strain on the rod, the axial load increases too; however, the axial load slightly decreased (Fig. 12b) from 37.1 to 36.5 kN, by about 1.64 % of the initial post-tensioning stress, after cycle 905,000 while the deflection increased. Moreover, the abrasion of concrete (any failure in concrete at the joint) entails an increase in the axial load of the GFRP rod, which was not observed in higher loading frequencies. This shows that increasing the frequency can cause a rod's stress relaxation.

Rod's axial load behaviour

The GFRP rod's axial load in both 2S-C-PT36-0.5Hz (Fig. 8f) and 2S-C-PT36-1Hz (Fig. 12b) was measured at different intervals during the cyclic loading. Due to the high deck's stiffness and limited deflection, concrete wasn't crushed, hence, there was no rod's axial load increment. The initial post-tension stress of 83MPa (equivalent to 0.98 MPa compression on the joint) was kept constant during the test; however, in deck 2S-C-PT36-1Hz, slight relaxation was observed, and the axial load decreased by 1.64% just after cycle 905,000.

Effect of the initial damage on post-tensioned segmental deck

The fatigue life of the pre-damaged deck (loaded up to $0.5M_u$) under the service load of 5kN, $0.16M_u$, simulating after incident scenario, in this section is discussed. The high-speed displacement-controlled load with a rate of 50 mm/sec (total loading duration of 1.4 seconds), simulating the load due to raging water, is applied to a segmental deck with an initial pre-compression stress level of 0.85 MPa, compared with the static loading. Then the pre-compression stress increased to 0.98, and the cyclic loading with a frequency of 0.5 Hz was investigated.

Load-displacement behaviour

The shear resistance of the segmental deck with post-tension GFRP rod arises from the compression between the segments in the joint and the concrete's resistance against crushing in a localised zone. The bending moment in both simply supported decks increased linearly throughout the loading procedure. The high-speed displacement-mode controlled loading caused the joint to open with uncontrolled behaviour compared to the static loading. As a result, the initial stiffness of the 2S-D-PT32 was 0.7 kN/mm, which is 86% lower than that of the 2S-S-PT32 (Fig. 14a). The progressive concrete crushing also resulted in lower stiffness at the after-opening stage, as the concrete was unexpectedly crushed. The maximum load-carrying capacity of the 2S-D-PT32 was almost equivalent to the post-drop load-carrying capacity of the slow-speed static loading, which was 50% lower than that of the 2S-S-PT32.

The GFRP rods were tightened to provide 0.98 MPa of compression stress on the joint (2S-C-PT36-D), and then the deck was subjected to cyclic loading (Fig. 14c). In the first cycle, the stiffness was 0.4 kN/mm, which is 92% lower than 2S-C-PT36-0.5Hz. This lower stiffness caused the imposed deflection to be between 0-10 mm in the first cycle, instead of 0-1mm, in 2S-C-PT36-0.5Hz. The stiffness gradually decreased, and it dropped to 0.18 kN/mm in the last cycle (55% reduction), which was accompanied by a GFRP rod rupture. Although this will be discussed in detail in the failure behaviour section, the results of this test indicate that a pre-damaged segmental deck with a pre-compression stress of 1MPa can withstand 68,532 cycles of loading with a magnitude of the designed service load of $0.16M_u$.

Rod's axial load behaviour

The rod's axial load increment in both loading scenarios (high-and-low speed static loading) did not significantly alter, as both exhibited linear responses (Fig. 14b), due to the linear behaviour of the GFRP rod. In the 2S-S-PT32, the axial load increased gradually because the concrete cracking was more controlled, and the progression of concrete crushing was slower.

In 2S-D-PT32, the axial load increased when the joint opened, as the concrete in the joint suddenly crushed. After the joint opened / concrete was crushed, the slope of the load-deflection curve in both cases was similar.

Unlike 2S-C-PT36-0.5Hz, the rod's axial load due to higher imposed deflection in 2S-C-PT36-D fluctuated in load/unloading. An uneven damaged crushing on two sides of the deck caused the rod's axial load increment reached to 40 kN, considering the post-tension load equal to 23.1 % of anchor strength, and another one with a higher joint opening (due to more damage) reached to 45 kN (24.6% of anchor strength).

Failure behaviour

The failure mechanism of the 2S-S-PT32 specimen was initiated by concrete crushing at the joint, followed by interlaminar shear of the rod at the joint. In the case of 2S-D-PT32, the initial damage caused the concrete to crush more on one side of the deck. This unsymmetrical concrete crushing led to uneven deflection under cyclic loading, with one GFRP rod bearing more deflection than the other. Consequently, the joint opening was more pronounced on one side, leading to the eventual rupture of the GFRP rod at cycle 68,532. Eventually, one rod ruptured and the whole weight of the segmental desk was carried by one GFRP rod, and the test stopped. This shows that 1 MPa compression can provide fatigue resistance for the pre-damaged segmental deck by having a compression depth throughout the aggregate at the compression zone.

CONCLUSION

This study investigated the cyclic behaviour of the monolithic deck internally reinforced with GFRP bars and the segmental decks connected with the post-tensioned GFRP rods. The large-scale decks with varying post-tensioning levels were tested, examining the hysteresis curve (under cyclic loading), load displacement (under static loading), load strain in monolithic and axial load in the segmental decks, and their failure behaviour. The key findings from this work are the following:

- The level of fatigue loading significantly affects the behaviour of the GFRP-RC monolithic deck. A concrete deck reinforced with at least 1.68% of high modulus GFRP rebar can tolerate 1 million cycles of wave loading without concrete cracking under a fatigue load equivalent to 21% of ultimate bending capacity ($0.21M_u$). Increasing the fatigue loading to $0.3M_u$ resulted in the deck's failure after only 20,585 cycles due to the initiation of the shear-compression crack in the concrete.
- The GFRP-RC monolithic deck will retain its static bending strength and stiffness even after 1 million fatigue-loading cycles at $0.21M_u$. However, the fatigue load of $0.3M_u$ will decrease the bending strength and stiffness by at least 20.7%.
- A segmental concrete deck connected with GFRP rods but without any post-tension and loaded under fatigue equivalent to $0.18M_u$ reduced its bending stiffness by 28.3% after 250,000 cycles because of the compressive crushing of the concrete at the joint.
- In the post-cyclic stage of a hand-tight segmental deck, as the applied bending moment surpassed $0.18M_u$, the deviation in the load-carrying capacity grew, ultimately reducing the deck's load-carrying capacity by 53.5% due to the GFRP rod rupture.
- The additional concrete compression stress of 0.48 MPa created by applying post-tensioning in the GFRP rods led to a 24.1% increase in the fatigue life of the segmental deck under cyclic loading of 5 kN ($0.18M_u$), compared to hand-tight GFRP rods. An additional concrete compression stress of 1 MPa created by post-tensioning is required to achieve a fatigue life of 1 million cycles for the segmental deck. This segmental deck will retain 100% of its strength and stiffness even after 1 million fatigue-loading cycles.
- Increasing the frequency of applied fatigue loading by 0.5Hz to 1.0Hz did not affect the fatigue behaviour during the first 800,000 cycles of the segmental deck with 1 MPa

concrete compressive stress created by post-tensioning in the GFRP rod. Stiffness reduction was 3.5% at 1 Hz compared to steady behaviour at 0.5 Hz. This was attributed to the higher fibre-to-fibre interactions in the longitudinal fibre of the rod and the increased wear at the interface of the concrete and GFRP rod when the loading frequency increased.

- Applying pre-compression stress of 1 MPa at the joint with the concrete already crushed can help the pre-damaged segmental deck survive fatigue loading of at least 68,500 cycles. Theoretical pre-compression stress of 1MPa provides enough stress at the joint, throughout the aggregate at the compression zone, to provide this fatigue resistance.
- The fatigue behaviour of the segmental deck can be enhanced by minimising the local compression zone at the joint and eliminating interlaminar shear failure of the GFRP rods. This can be achieved by post-tensioning, as this increases the neutral axis depth in the concrete and improves the stiffness of the segmental decks, which leads to a lesser deflection and more uniform stress distribution at the joint.

The preliminary result of the current study from testing one large-scale deck per specimen type showed the suitability of the segmental precast concrete system post-tensioned with GFRP rods in marine infrastructures. Additional tests with repeated specimens will help validate the findings and enhance the robustness of the conclusions arrived at in this study. Future studies should also include a theoretical approach and an extended test matrix to confirm the repeatability of the observed behaviour and refine design recommendations.

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Declaration of interests

The authors would like to declare and confirm that there is no conflict of interest in this manuscript.

Data availability

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

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Table 1. Material properties

Property	Test method	No.	Value	
			Mean	Standard deviation
GFRP rods				
Nominal diameter (mm)	CSA S807-19 (2019)	9	23.5	-
Cross-sectional area (mm^2)	CSA S807-19 (2019)	9	416	-
Tensile strength (MPa)	CSA S807-19 (2019)	6	1340	68
Modulus of Elasticity (GPa)	CSA S807-19 (2019)	6	65	2
Ultimate strain (%)	CSA S807-19 (2019)	6	2	0.1
Fiber content (%)	ASTM D2584 (2018)	9	81.9	1.2
End anchorage				
Maximum tensile load (kN)	ASTM D7205 (2021)	6	328.1	9.4
Tensile strength (MPa)	ASTM D7205 (2021)	6	756.8	21.6
Ultimate strain (%)	ASTM D7205 (2021)	6	0.86	0.07
Concrete				
Compressive strength (MPa)	AS1012.9 (2014)	9	37.1	6.2
Internal GFRP				
Nominal diameter (mm)	CSA S807 (2019)	9	12.7	-
Nominal area (mm^2)	CSA S807 (2019)	9	126.6	-
Tensile strength (MPa)	ASTM D7205 (2021)	6	1281	33.3
Modulus of Elasticity (GPa)	ASTM D7205(2021)	6	61.3	1.3
Ultimate strain (%)	ASTM D7205 (2021)	6	2.1	0.1

Table 2. Segmental deck specimen's detail

No.	Specimen	Loading description	Deck's description	Deck's condition	Post-tension on rod (kN)	Equivalent stress (MPa)	Equivalent stress on concrete (MPa)
1	1S-C-30	Cyclic	Monolithic	Intact	0	0	0
2	1S-C-40	Cyclic	Monolithic	Intact	0	0	0
3	1S-S-D1M	Static	Monolithic	Post-cyclic	0	0	0
4	1S-S	Static	Monolithic	Intact	0	0	0
5	2S-C-PT0-250k	Cyclic-250,000 cycles	Segmental	Intact	2	4.1	0.05
6	2S-S-PT0-D250k	Static	Segmental	Post-cyclic	2	4.1	0.05
7	2S-C-PT0	Cyclic	Segmental	Intact	2	4.1	0.05
8	2S-C-PT18	Cyclic	Segmental	Intact	18	42	0.48
9	2S-D-PT32	fast-rate static	Segmental	Intact	32	74	0.85
10	2S-S-PT32	Static	Segmental	Intact	32	74	0.85
11	2S-C-PT36-D	Cyclic	Segmental	Damaged	36	83	0.98
12	2S-C-PT36-0.5Hz	Cyclic-0.5Hz	Segmental	Intact	36	83	0.98
13	2S-C-PT36-1Hz	Cyclic-1Hz	Segmental	Intact	36	83	0.98
14	2S-S-PT36-D1M	Static	Segmental	Post-cyclic	36	83	0.98