REPAIRING OF TUBULAR JOINTS DAMAGED UNDER CYCLIC LOADINGS: AN INNOVATIVE TECHNIQUE

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SUMMARY

Offshore platforms suffer damages during their service life because of various reasons, as a result of which the integrity of the structure is endangered. Therefore they have to be necessarily repaired. In the past many existing offshore platforms, damaged during their operation, have been repaired using a broad variety of techniques. All of them have certain limitations in terms of their applicability. Therefore an innovative technique was developed by the authors to rehabilitate tubular joints damaged under cyclic loading. The proposed repair technique has made use of ferrocement jackets and High Performance Grout (HPG). Three corrosion fatigue damaged tubular joints of nominal chord and brace diameter 324 mm and 219 mm and thickness 12 mm and 8 mm respectively were repaired using the proposed technique. They were tested under axial brace compression loading to check the efficacy of the proposed repair technique and also study the behaviour of the repaired joint. Composite action was observed during testing of the repaired joints due to the excellent bond of the HPG with the tubular joints. It was also observed that the original stiffness of the damaged joint was restored after repair. The bond strength of the HPG was almost three times greater than the conventional cement grout. Analytical model to predict the strength of the repaired joints was also developed considering the strain compatibility and force equilibrium of the repaired section. The paper presents a detailed description of the innovative repair technique, testing of repaired joints, observations made during testing, development of the analytical model, and conclusions drawn therefrom.

INTRODUCTION

Steel tubular framed structures are installed on the sea bed for production of oil from sea bottom. They support drilling and production activities above the elevation of waves. They serve as artificial bases and hence are called offshore platforms. Among a variety of offshore platforms used for exploration and production of oil, the most popular structures for shallow water depth of up to 200 m are the jacket platforms that are constructed using steel cylindrical tubular sections. In the tubular framed structures, the intersection between the various tubular members is welded and forms tubular joints. The main member is called a chord and the secondary member a brace. Joints without reinforcements of any sort are called unstiffened joints. Those, which are reinforced with annular rings welded inside the chord, are called internally ring stiffened joints. Offshore platforms have to serve in hostile environments and hence are likely to be damaged due to cyclic loading derived from wave action which excites a vibration at wave frequency of 1/6 Hz (Booth, 1983) and environmental corrosion. Ocean wave loading causes fluctuation in stress at the welded intersection of the joint. The magnitude of the stress at the welded intersection is higher by many factors due to the weldments and change in geometry. With the accumulation of such a large stress fluctuations, a crack will initiate at the welded intersection and will grow further with the number of cycles. The sea besides providing the major source of cyclic loading from wave action, also provides the environmental means to accelerate the rates of crack growth. The damage results in loss of stiffness of the joint.

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The damage to the structure endangers its integrity and hence the structure has to be repaired so that it continues to serve its intended function during its design life. A review of the available literature (Thandavamoorthy and Santhakumar, 1996 a; Thandavamoorthy et al., 1996 b; Dier and Lalani, 1997) has disclosed that in the past, many existing platforms have suffered structural damages of varying degrees. They have subsequently been rehabilitated using a broad variety of techniques, viz., welding, bolting, and grouting with or without clamps. All these techniques have limitations in terms of their applicability. Therefore an innovative technique (Fig. 1), that has overcome many of the problems associated with the conventional technique, has been proposed (Thandavamoorthy, 1998). This scheme has made use of ferrocement jackets and HPG for repairing of fatigue damaged internally ring stiffened T joints. Ferrocement consists of closely spaced multiple layers of mesh or fine rods completely impregnated with cement mortar. It can be formed into thin panels or sections, mostly less than 25 mm thick. According to this technique, ferrocement jackets were cast into two halves and deployed over the damaged tubular joints in such a way that an annular space is created between the tubular members and the ferrocement jackets. The annular space was grouted with HPG, developed specially for this purpose. The repaired joints were tested under axial brace compression loading. The paper presents a description of the innovative repair technique, experimental investigation carried out to study the behaviour of the repaired joints and the development of the analytical model to predict the strength of the repaired joints.

INNOVATIVE REPAIR TECHNIQUE

A masonry mould, to suit to one half of the T joint, was constructed. It was finished smooth with wax to facilitate smooth casting of ferrocement jacket. Reinforcement grill consisting of 3mm wire welded mesh wrapped with chicken mesh was fabricated to suit to the shape of the mould. It was placed in the mould with a cover of 10 mm. Cement mortar 1:2 was prepared and placed in the mould in layers, and compacted well using needle vibrator. The thickness of the ferrocement jacket was controlled using a wooden template. After initial hardening it was water cured for seven days. Later on the jackets were deployed on the joint in such a way that an annular space of 5 mm was left between the inside of the jacket and the outside of the tubular member. This annular space was then grouted with HPG which was developed specially for this purpose. The HPG consisted of epoxy, fly ash and sand. First fly ash and sand were measured in required proportion and mixed well in dry condition. Secondly resin and hardner were mixed thoroughly in required proportion. Then epoxy was mixed with fly ash-sand mixture and mixed thoroughly to get a flowable mixture devoid of air bubbles (Thandavamoorthy et al., 1998 a, b). Fatigue damaged joint was erected vertically. The HPG was poured manually from top in the annular space with a lot of care. The specimen was cured for seven days in air. The properties of the HPG are given in Table 1.

Table 1

Properties	Values
Specific gravity	1.78
Compressive strength (MPa)	70 - 100
Flexural Tensile Strength (MPa)	45 - 60
Bond strength (MPa)	17
Elastic modulus (MPa)	1.286×10^4

Three numbers of fatigue damaged internally ring stiffened tubular T joints (Fig. 2) were selected for rehabilitation from the lot available in the Fatigue Testing Laboratory of the Structural Engineering Research Centre, Madras. These joints have been stiffened internally by three annular rings of width 75 mm and thickness 12 mm. Dimensions of the selected joints are given in Table 2. These joints had earlier been tested under axial brace tensile fatigue loading under corrosive environments of sea water, synthesised according to the ASTM Standards D 1141 (ASTM, 1980). Due to the synergistic action of fatigue and corrosion, these joints had developed through-thickness crack at the weld toe. The crack was formed on the top side of the chord member along its circumference. The length of the circumferential crack for each joint was measured and is given in Table 2. The extent of the damage varied from 22 to 45 percent. Because of the aggressive environment, corrosion patches have been formed on the immersed portion of the welded intersection of brace and chord member. However, there was no significant reduction in the wall thickness.

Table 2

S. No.	Specimen No.	Dimensions of the joint, (mm)				Length of the circumferential crack, (mm)	
		Chord		Brace		Quadrants	
		Diameter	Thickness	Diameter	Thickness	I	II
1	RTS1	321.36	12.00	232.37	8.00	177	130
2	RTS2	325.31	12.00	220.27	8.00	130	100
3	RTS3	320.54	12.00	218.68	8.00	242	213

Experimental Investigation

The repaired joints were white washed before testing. Grid lines at every 50 mm spacing were drawn on the joint to mark the crack pattern that may develop during loading. The repaired joint was tested under monotonic loading of axial brace compression. The joints were fixed to the steel pedestals by bolting. The pedestals, in turn, were fixed to the strong concrete floor by means of mild steel bolts of 60 mm size. The entire assembly was placed under a reaction frame. On the flange of the brace member a built-up steel joist assembly and two 2000 kN hydraulic jacks were placed. They were inserted between the horizontal cross beam of the reaction frame and the flange. Another 1000 kN hydraulic jack and a 1000 kN Proceq load cell were placed in a self-straining frame that was kept by the side of the main test set-up. The jacks were connected to the electrically operated pumping unit by means of distributors and high pressure rubber hoses. Axial brace compression loading was applied on the joints by means of the hydraulic jacks. Dial gauges were mounted beneath the joint to measure the deflection of the joint under loading. Load was applied in increments and for each increment deflection at mid-span was measured. Typical load vs. midspan deflection curve of a repaired joint is shown in Fig. 3. As the load was increased gradually the first visible fine crack was formed in the ferrocement jacket along its circumference near midspan. The load corresponding to this first crack for all tested joints are stated in Table 3. On further loading the crack increased in length and some more cracks were formed along the span. Cracks were all essentially flexural in nature and at uniform spacing. The measured ultimate load of each repaired joint is given in Table 3.

Evaluation of Bond Strength

Bond strength of the High Performance grout was evaluated by connecting two steel pipes using the same grout and testing the specimen under static load. One of the pipes was larger than the other. Larger pipe was of diameter of 70 mm and length 155 mm. Smaller pipe was of size 50 mm in diameter and 230 mm in length. They were arranged concentrically with smaller pipe inside the larger one so that an annular space between them was created. The inner pipe was protruding out 20 mm on one side of the outer pipe and 55 mm on the other side. The annular space was grouted with the same material that was used for grouting of the tubular joints. The specimen was cured in air.

The specimen was tested under monotonic loading in the MTS dynamic testing system. In this loading arrangement the inner pipe with 55 mm projection was pushed out while holding the outer pipe against the upper platten of the machine. Load on the specimen was applied gradually. The loaded end of the inner pipe was bulged out at a load of 370 kN. The specimen could sustain 416 kN. At the ultimate load of 416 kN the inner pipe was pushed out fracturing the outer pipe. For the purposes of comparison, similar specimen with cement mortar was grouted and tested in exactly the same way. It could sustain an ultimate load of only 147 kN and the inner pipe was pushed out at that load. A bond stress, which is defined as the failure load divided by the bond area, was calculated at ultimate failure. The calculated bond stress of the cement mortar grouted specimen was 6 MPa. This is in conformity with the observation made by previous investigators that the characteristic bond strength was directly proportional to the square root of the characteristic grout compressive strength.. The specimen grouted with HPG was having a bond stress of 17 MPa. This is almost three times greater than that of the cement grouted specimen.

Table 3

S. No.	Specimen No.	First Crack Load, (kN)	Ultimate Load, (kN)	
			Experimental	Theoretical
1	RTS1	1071.50	1714.40	1592.20
2	RTS2	535.75	1999.50	1724.30
3	RTS3	642.90	1999.50	1804.77

DEVELOPMENT OF AN ANALYTICAL MODEL

It was observed during the experimental programme that the joint was bending like a prismatic member. There was composite action with excellent bond of the HPG with the tubular members. Considering the composite action of the damaged joint, ferrocement jackets and the high performance grout, an analytical model was developed to assess the strength of the repaired tubular joints. The principle of reinforced concrete theory was used in the development of the model. Necessary equations for the forces and moments were derived considering the compatibility of strain and equilibrium of forces of the different components of the repaired joints. Figure 4 shows a typical cross section of a joint repaired using the technique described above. This consists of a cracked steel tubular section (Fig. 5), HPG (Fig. 6) and the ferrocement jackets (Fig. 7). Different stress distributions for each of the components were assumed as shown in Figs. 5, 6, and 7. For these assumed stress distributions, equations required for various forces and moments were derived. While a detailed descriptions about the derivation of equations for the computation of forces and moments for the cracked section, grout section, and ferrocement jackets have been published elsewhere (Thandavamoorthy, 1998), important moment equations are presented in this section.

Moments in Cracked Section

$$M_{c11} = \frac{R^2 t\sigma_{11}}{(\cos\phi_1 + \frac{e_d}{R})}\{\frac{1}{4}(2\psi_{11} - \sin 2\psi_{11}) + 2(\frac{e_d}{R})(1-\cos\psi_{11}) + \psi_{11}(\frac{e_d}{R})^2\} \qquad (1)$$

$$M_{c12a} = R^2 t\sigma_u \{\sin(\phi_2 + \psi_{12a}) - \sin\phi_2 + \psi_{12a} (\frac{e_d}{R})\} \qquad (2)$$

$$M_{12b} = \frac{R^2 t\sigma_u}{(\sin\psi_{12b} + \frac{e_d}{R})}\{\frac{1}{4}(2\psi_{12b} - \sin 2\psi_{12b}) + 2(\frac{e_d}{R})(1-\cos\psi_{12b}) + \psi_{12b}(\frac{e_d}{R})^2\} \qquad (3)$$

$$M_{c2} = \frac{2R^2 t\sigma_{12b}}{(\frac{e_d}{R})}\{\frac{1}{4}(2\psi_2 - \sin 2\psi_2) - 2(\frac{e_d}{R})(1-\cos\psi_2) + \psi_2(\frac{e_d}{R})^2\} \qquad (4)$$

$$M_t = \frac{2R^2 t\sigma_t}{(1 - \frac{e_d}{R})}\{\frac{1}{4}(2\delta + \sin 2\delta) - 2(\frac{e_d}{R})\sin\delta + \delta(\frac{e_d}{R})^2\} \quad (5)$$

Moments in HPG Section

$$M_{cg11} = 2R_g^2 t_g \sigma_{gy}\{\sin\theta_{g1} + \theta_{g1}(\frac{e_d}{R_g}) \quad (6)$$

$$M_{cg12} = 2R_g^2 t_g\{\sigma_{g1}[1 - \cos\psi_{g1} + \psi_{g1}(\frac{e_d}{R_g})] + \frac{(\sigma_{gy} - \sigma_{g1})}{\cos\theta_{g1}}[\frac{1}{4}(2\psi_{g1} - \sin 2\psi_{g1}) + (1 - \cos\psi_{g1})(\frac{e_d}{R_g})]\} \quad (7)$$

$$M_{cg2} = \frac{2R_g^2 t_g \sigma_{g1}}{(\frac{e_d}{R_g})}\{\frac{1}{4}(2\psi_{g2} - \sin 2\psi_{g2}) - 2(\frac{e_d}{R_g})(1 - \cos\psi_{g2}) + \psi_{g2}(\frac{e_d}{R_g})^2\} \quad (8)$$

$$M_{gt} = \frac{2R_g^2 t_g \sigma_{gt}}{(1 - \frac{e_d}{R_g})}\{\frac{1}{4}(2\delta_g + \sin 2\delta_g) - 2(\frac{e_d}{R_g})\sin\delta_g + \delta_g(\frac{e_d}{R_g})^2\} \quad (9)$$

Moments in Ferrocement Jackets

$$M_{cj11} = 2R_j^2 t_j \sigma_{fcu}\{\sin\theta_{j1} + \theta_{j1}(\frac{e_d}{R_j})\} \quad (10)$$

$$M_{cj12} = 2R_j^2 t_j\{\sigma_{fcu2}[1 - \cos\psi_{j1} + \psi_{j1}(\frac{e_d}{R_j})] + \frac{(\sigma_{fcu} - \sigma_{fcu2})}{\sin\psi_{j1}}[\frac{1}{4}(2\psi_{j1} - \sin 2\psi_{j1}) + (1 - \cos\psi_{j1})(\frac{e_d}{R_j})]\} \quad (11)$$

$$M_{cj2} = \frac{2R_j^2 t_j \sigma_{fcu2}}{(\frac{e_d}{R_j})}\{\frac{1}{4}(2\psi_{j2} - \sin 2\psi_{j2}) - 2(\frac{e_d}{R_j})(1 - \cos\psi_{j2}) + \psi_{j2}(\frac{e_d}{R_j})^2\} \quad (12)$$

$$M_{wmc} = 2A\sigma_{wy}\{4(\frac{e_d}{R_j}) + \cos 9.58 + \cos 41.74 + \cos 73.9 - \sin 16.1\} \quad (13)$$

$$M_{wmt} = 2A\sigma_{wy}\{\cos 9.58 + \cos 41.74 - 2(\frac{e_d}{R_j})\} \quad (14)$$

The total moment capacity of the repaired joint consisting of the cracked section, HPG and ferrocement jackets is obtained by adding Eq. (1) through Eq. (14). The computed ultimate loads of the repaired joints are given in Table 3.

RESULTS AND DISCUSSION

Three internally ring stiffened tubular T joints damaged under cyclic loading have been repaired using an innovative technique by making use of ferrocement jackets and HPG. The repaired joints were tested under axial brace compression loading. The predominant mode of the repaired joint was found to be bending. It was observed in this experimental investigation that there was excellent bond between the HPG and the tubular members. The bond strength of the HPG was almost thrice that of the cement grout. The tensile strength of the HPG was ten times greater than that of the cement slurry. It was observed in this experimental investigation that the original stiffness of the joint was restored after repair. In two of the repaired joints RTS1 and RTS2, it was observed that the protruding brace member out of the ferrocement jackets were bulged out near the ultimate load. With high load and bond slip not occurring the energy was absorbed by the deformation of the cross section of the brace member. This goes to prove that the bond between the ferrocement jackets and the tubular members were excellent. This observation was similar to the behaviour noticed in the case of pipe-to-pipe connection formed using HPG. The analytical model was based on the composite action of the cracked section, HPG and the ferrocement jackets. They were derived by considering the strain compatibility and force equilibrium and were based on Euler bending theory. Good agreement between the measured and predicted values of the ultimate loads was observed.

CONCLUSIONS

An innovative technique to rehabilitate fatigue damaged internally ring stiffened tubular joints was developed in this experimental investigation. A HPG consisting of the epoxy, fly ash, and sand was developed in this investigation and used for the repairing fatigue damaged stiffened joints. A better corrosion resistant ferrocement jackets were used in this investigation in lieu of the metallic sleeves used in the conventional technique. The metallic sleeve is highly vulnerable to the attack of marine corrosion. Three ring stiffened fatigue damaged joints were repaired using the innovative technique and their strengths were measured. The nominal chord and brace diameter of the joints were 324 mm and 219 mm and thickness 12 mm and 8 mm respectively. Composite action of the joint was observed in this experimental investigation. An analytical model based on strain compatibility and force equilibrium following reinforced concrete theory was developed to predict the strength of repaired joints and the same validated with the experimental values. The proposed model can be incorporated in the existing design codes.

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