THE INSTITUTE OF MARINE ENGINEERS
76 Mark Lane, London EC3R 7JN Telephone: 01-481 8493 Telex: 886841

TRANSACTIONS (TM)

STRUCTURAL REPAIRS TO OFFSHORE INSTALLATIONS

W. J. Winkworth, D. Harris and R. Boon

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Structural Repairs to Offshore Installations

W. J. Winkworth, CEng, MIMechE, D. Harris, CEng, MIMechE, MRAeS and R. Boon, CEng, MIStructE
Lloyd's Register of Shipping

SYNOPSIS

Structural damage to offshore installations has been due to a number of causes including fatigue, boat impact, dropped objects and installation problems. For fixed platforms the repair of the structure in situ, often underwater, can present considerable difficulties. This paper reviews structural repairs from the viewpoint of the Certification/ Classification Authority. Typical examples of damage and the repair solutions adopted will be described.

INTRODUCTION

In 1972 Lloyd's Register approved its first fixed steel platform for certification, although prior to this the Society had been extensively involved in the approval for class of mobile offshore drilling units. Many of the early fixed steel platforms approved for certification were existing platforms that had been installed as early as 1966 in the North Sea and 1971 in Indonesia.

The Society's involvement offshore has grown over the years to the extent that it has now been involved in the approval of over 500 offshore structures worldwide. With such a large number of structures, including some of the earliest designs, it is not surprising that a number of them have suffered structural damage and required repairs.

This paper reviews the current state of the art with regard to structural repair of offshore installations and concentrates particularly on fixed platforms because of the difficulty that in situ repair often presents.

CERTIFICATION REQUIREMENTS

The Mineral Workings (Offshore Installations) Act 1971 and the Offshore Installations (Construction and Survey) Regulations 1974, applicable to the UK Sector of the Continental shelf, require that the structural integrity of offshore installations is maintained. These regulations provide for the issue and termination of Certificates of Fitness and the appointment of Certifying Authorities. Similar schemes have been introduced by a number of countries with offshore reserves of oil and gas.

To ensure that the structures are maintained in a safe condition the regulations require that periodic surveys of the installation are carried out. The primary aim of the surveys is to locate defects which impair the safety of the installation, and an agreed schedule of inspection is drawn up by the owners and Certifying Authority.

Additionally, the Certifying Authority has to be notified immediately if the installation is damaged or is suspected of being damaged in a manner likely to affect the validity of the Certificate of Fitness. Any necessary repair work to the structure has to be approved by the Certifying Authority.

CAUSES OF DAMAGE

A comprehensive list of the causes of damage is beyond the scope of this paper. However, some of the more common causes of damage for both steel and concrete structures are discussed below.

Walter Winkworth is a Principal Surveyor and Deputy Head of the Ocean Engineering Department in the Offshore Services Group of Lloyd's Register of Shipping. Prior to joining Lloyd's Register of Shipping he first gained experience in the petrochemical industry working on the structural design and analysis of plant pipework for Kellogg International Corporation. Following this he moved to the British Aircraft Corporation, Weybridge where he was engaged as a stress engineer on such projects as Concorde and the Tornado. In 1971 he joined the Ocean Engineering Department of Lloyd's Register of Shipping as a Structural Engineer. During this time he performed design appraisal analyses on many different types of offshore structures, including fixed steel platforms, semi-submersibles and jack-ups. He has also been involved in the development of the computer programs and analysis techniques used by the Society in this work. In 1978 he was appointed Deputy Head of the Ocean Engineering Department.

David Harris was apprenticed to Short Brothers Ltd and trained in their works and design offices. This was followed by 25 years in structure departments of major firms in the aircraft industry, finally in the capacity of Assistant Designer. In 1974 he joined the Ocean Engineering Department of Lloyd's Register of Shipping and as Project Leader has been involved in the monitoring and assessment of data from offshore structures, the post-damage strength of fixed steel platforms and the preparation of draft guidance notes for the repair of offshore steel and concrete structures.

Robert Boon became a chartered structural engineer in 1972. After working for a series of consulting engineers he joined Lloyd's Register of Shipping in 1974, where he is now employed as a Senior Structural Engineer.

Fatigue

In the late 1960s and early 1970s fatigue was not considered in the design of offshore structures. Even when fatigue began to be specifically considered, a comparatively simplified analysis method was used. This neglected the vertical forces due to waves and used generalised stress concentration factors which did not account for out-of-plane bending. In addition, the conductor bracing area was represented by idealised members which were sized on stiffness considerations.

Re-calculation of the fatigue lives of members in a typical existing platform using a more sophisticated fatigue analysis, the latest parametric stress concentration factors and a fully idealised conductor bracing area gives the results indicated in Fig. 1. This analysis shows that a simplified approach can underestimate the fatigue damage in the conductor bracing and horizontal levels generally, particularly at those levels close to the waterline. It is at this location that much of the fatigue damage on fixed steel platforms has occurred.

Not all fatigue damage can be attributed to deficiencies in the design process. Some fatigue damage has been due in part to poor fabrication and in other cases to excessive build up of marine growth.

Boat Damage

Another fairly common cause of damage in offshore installations is supply boat impact. The need to load and off-load supplies necessitates supply boats manoeuvring in close proximity to the platform. Despite the precautions taken, collisions between the substructure and supply boat do occur and in some instances cause serious damage.

Dropped Objects

Most of this damage has been caused by tubular sections (piles, followers, risers etc.). When dropped end on there is little water resistance to impede their fall and considerable damage is caused when they strike a part of the substructure.

Installation damage

Load out, tow, launch and installation are high-risk phases for an offshore installation. Damage from a variety of causes has occurred to some structures during these phases. In some cases the circumstances of this damage has necessitated repair in situ.

'Overstressed' existing platforms

In some cases reinforcement has been undertaken on undamaged structures which have been shown by design calculations to be overstressed in extreme storm conditions. This situation may have arisen because of increased deck loading, upward revision of environmental criteria and/or changes in the strength code requirements.

In recent years, however, the trend has been to avoid reinforcement where possible by carefully assessing the existing structure to determine whether, under realistic loading and modelling of the actual load-deflection behaviour of the substructure and foundation, the structure can be shown to be satisfactory. In some cases this work has included full-scale testing of representative joints to evaluate fully the existing structural strength.

Foundation problems

A variety of foundation problems have been experienced on offshore structures requiring either repairs or modifications. In some cases this has been caused by actual settlement and in others it has come about because of new information and/or calculations showing the foundation is under strength even though no significant settlement or tilt has occurred.

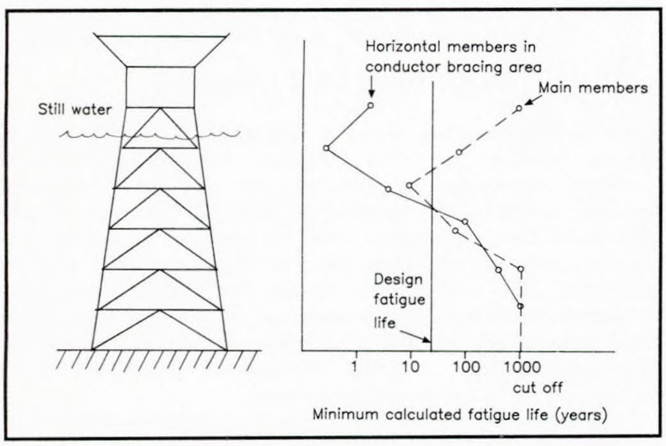


FIG. 1: Plot of fatigue life against platform elevation

Concrete structures

The above causes of damage apply generally to both steel and concrete platforms, although concrete structures have not shown themselves to be susceptible to fatigue damage. There are some potential causes of damage that are more specific to concrete structures such as loss of drawdown or corrosion of reinforcement, prestressing etc.

ASSESSMENT OF THE DAMAGED STRUCTURE

Inspection requirements

When damage has occurred it is usually necessary to undertake further inspection as soon as possible in order to determine with confidence the extent of damage that has taken place. This is required for two reasons: first to assess the structural integrity of the 'as damaged' structure and secondly to evaluate the extent of repairs that will be necessary.

The programme of inspection will depend on the scale of damage and its cause. Where the damage is due to a boat strike on one member then the inspection required will be as follows:

1. NDT (non-destructive testing) at the point of impact on the member.
2. Straightness checks on the member.
3. NDT at the nodes at the end of the damaged member.

If damage is suspected due to a dropped object incident then visual inspection of the whole jacket may be required. This need not involve divers but could probably be accomplished using a remotely operated (inspection) vehicle (ROV). In some cases where a tubular section has been dropped, it has struck the jacket at a number of levels. In others it has passed completely through the jacket without damage. By identifying the location of the fallen object on the sea bed, its probable path through the jacket can be calculated and the most likely areas of impact determined.

In the case of fatigue cracks found during annual survey, then further inspection of similar low-life joints will be necessary. Close visual inspection is unlikely to find any but the most serious cracks and therefore some form of NDT will be required at these joints. If the cracking is unexpected (ie the predicted fatigue life of the joint in question is high) then further structural analysis would be essential in order to discover the cause and, if the problem is fatigue, identify other low-life joints.

If welded or clamp type repairs are required a more detailed inspection and a dimensional survey will be necessary at a later stage to facilitate the design and installation of the repair.

The inspection requirements may well represent a significant part of the total cost of an offshore repair.

Continued operation

If the damage is serious a decision must be made as to whether the platform can continue to operate. This will depend on the degree to which the strength of the structure has been impaired, the weather conditions prevalent at that season and other factors. A structural analysis will probably be required to provide the answer to these questions.

Some form of demanning or other limitation may have to be imposed immediately and then revised in the light of detailed calculations when these become available.

In many cases, even though the strength of the platform has been impaired, the platform may continue to operate during the summer season because of the reduced wave height.

The criteria for continued operation from the Society's viewpoint is that:

1. The platform completely meets the code requirements for a 50 year seasonal storm.

2. The fatigue life of the platform is acceptable during the time taken for the repairs to be completed.

3. Loss of redundancy is considered acceptable.

If the platform is found to be safe for the summer period only but unable to meet the 50 year winter storm requirement then a condition is placed on the certificate requiring repairs to be completed before the onset of the winter season.

The structural analysis of the damaged structure should also determine whether a repair is required. If it can be shown that the platform completely meets the code requirements for strength and fatigue for both the total structure and the damaged area in particular, then it may well be found acceptable for certification purposes. However, even in these cases a repair is often undertaken by the operator in order to maintain the reserve strength of the platform.

REQUIREMENTS FOR REPAIR

Choice of repair

The type and method of repair selected is the operator's responsibility and will be based on the type of structure, the timescale allowed for the repair and economic considerations.

For a repair to meet certification requirements the structure does not necessarily have to be returned to its original design strength or configuration but it must meet all the relevant strength, fatigue and corrosion standards.

While every repair situation is unique, the options available for repair of an offshore structure fall into several general catergories, some of which are discussed later.

Analysis of the repaired structure

Once the choice of repair has been made it may be necessary to re-analyse the structure in the repaired condition to ensure that the strength and fatigue requirements are met fully.

Where complete failure occurs in a member under permanent loading, the load is redistributed to the surrounding structure. Unless special measures are taken during the repair this loading will not be recovered by the replacement member. This new load condition must be checked and if found too onerous an attempt must be made either to preload the replacement member or to reinforce the jacket in the overstressed areas.

This aspect is illustrated by the example shown in Fig. 2. A failure occured in the member indicated at level 3. The deck load applied at the centre leg is reacted in the diagonal braces between level 2 and level 3, which in turn puts a tension load in the horizontal member for the still water case. When the member failed this load was redistributed to the other members on the jacket, most of the horizontal reaction being supplied by additional bending in the main leg. The extreme storm loads in the jacket members are shown for two cases, first with the structure as originally designed before the failure and secondly after the failure and assuming the member was simply repaired with no attempt to replace the original tension load.

It can be seen that after the repair the tension load in this member is reduced while the bending moment in the main leg is increased by almost 60%. Therefore in this case it will be necessary to preload the repaired member or reinforce the overstressed area of the main leg.

In some cases of dented or damaged braces, which still have sufficient strength to resist permanent loads, it has been found preferable not to remove the damaged brace but simply to weld a replacement brace alongside the damaged one. This has the advantage of reducing load redistribution and avoiding a period when the structure is even weaker with the damaged brace removed but the repair brace not yet fitted. An example of such a repair is shown in Fig. 3.

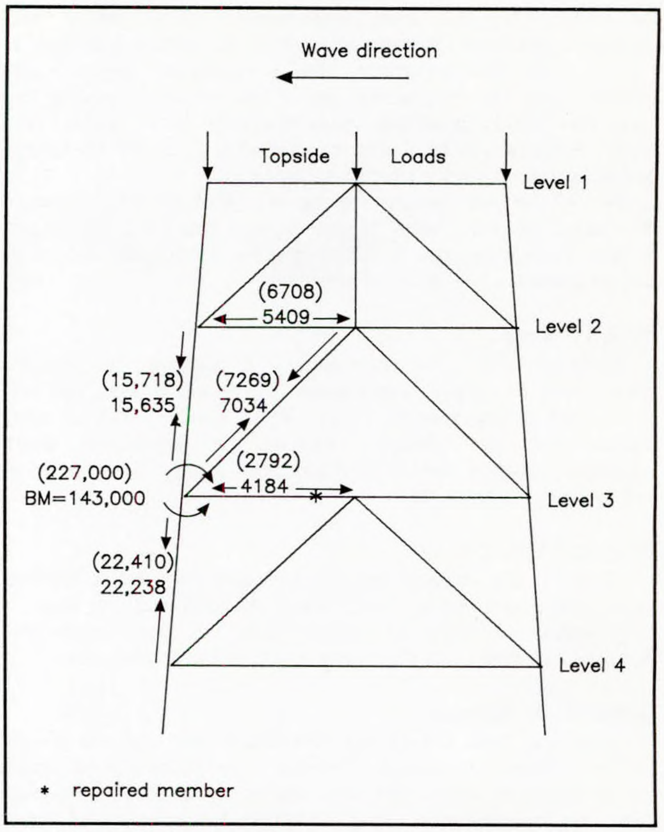


FIG. 2: Comparison of loads in jacket before and after repair (loads after repair shown in brackets)

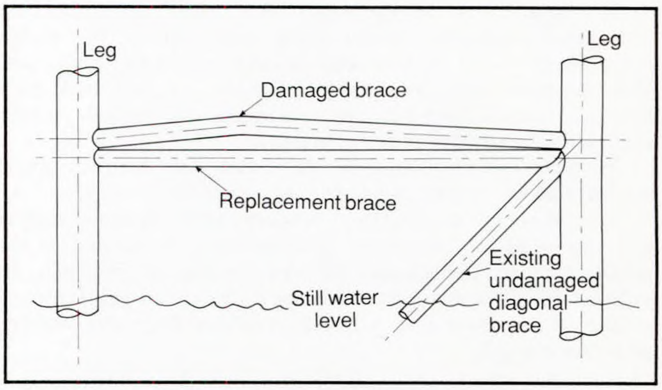


FIG. 3: Proposed repair scheme for horizontal brace between legs

GRINDING OUT DEFECTS

Where cracking has occurred, regardless of cause, in a steel tubular member then it is frequently possible to repair the member effectively by grinding out the crack. Because grinding involves the removal of material from an already critical location the process must be very carefully controlled so that more material than required is not removed and the situation made worse than before. In addition, the stress concentration factor is very dependent on the geometry of the final groove and detailed information on this is required by the design engineer in order to determine the final strength and fatigue life at the ground-out location.

The following general procedure is normally specified. The grinding is carried out utilizing a spherical burr mounted on a

FIG. 4: Tungsten spherical burr mounted on hand-held grinder

FIG. 5: Profile gauge

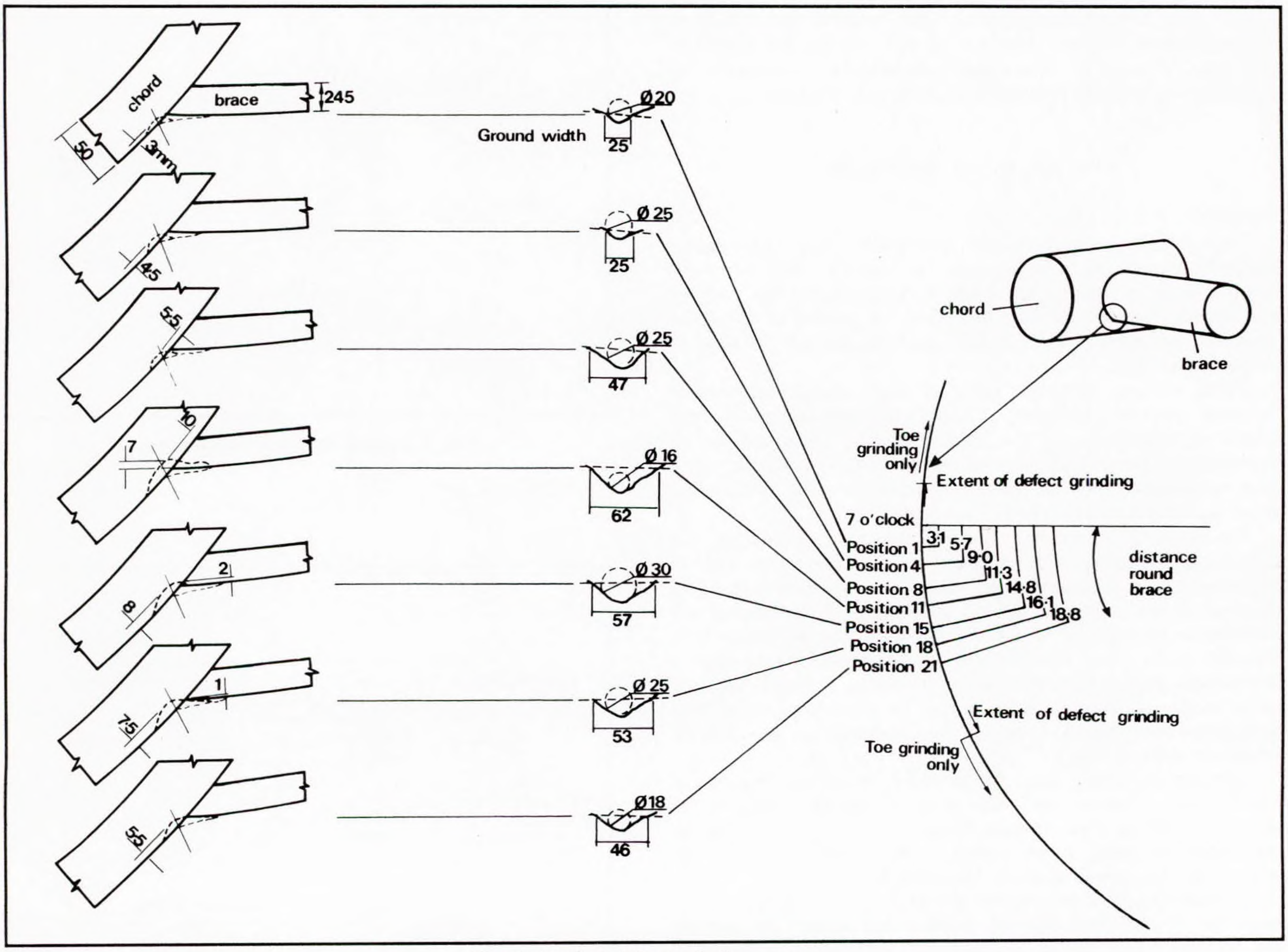


FIG. 6: Typical defect grinding report

hand-held grinder (see Fig. 4). The spherical burr is chosen to give a pre-determined radius to the groove. A typical value is 12 to 25 mm diameter. The area around the crack where grinding is to be performed will have been cleaned using a wire brush prior to inspection. Where the crack is not visible, magnetic particle inspection (MPI) is carried out and the location of the crack marked using a punch every 2 cm. A guide groove is then ground out along the crack after which the alignment is checked. The depth of the groove is then increased in 2 mm steps, after which MPI is carried out to confirm the crack existence and location.

When MPI has confirmed that the crack has been completely removed the geometry of the groove must be carefully recorded. The location round the member is marked out and the shape of the groove measured every 5 cm using a profile gauge. Such a gauge is shown in Fig. 5, where it is being used for measurement of corrosion pitting on a weld. The geometry of the groove can then be reported. Typical reports are shown in Figs 6 and 7. A permanent record can also be obtained by taking an epoxy resin mould of the groove. This will also indicate the surface roughness obtained in the groove.

Once the length, depth and geometry of the groove are known, calculations are required to determine that the static strength and fatigue life are satisfactory. The current approach used by the Society is to re-calculate the fatigue life increasing the stress concentration factor (SCF) to account for the ground notch. An increase in fatigue life of 2, because of the grinding, is allowed and if the grinding is light there will be no increase in the SCF. However, where deep grinding is required, the increase in the SCF will, in very general terms, reduce the original fatigue life by a factor of 4.

There is some evidence that the above approach may be conservative since much of the testing presently being done indicates that grinding gives a significant increase in the time to crack initiation and that, depending on the stress level, the factor of 2 on life underestimates this effect. Most of the current work being carried out in this area is unfortunately confidential and cannot be discussed in any detail. However, in some cases it may be possible that severe storms could initiate minute cracks and much of the benefit of grinding would be lost. In view of this the Society considers that any repair of this type should be monitored by inspection until further experience in service is obtained.

MECHANICAL REPAIRS

General

This type of repair relies principally upon mechanical forces between its components to satisfy the structural requirements of repair, as distinct from relying on welded attachments to provide this capability. It generally takes the form of two steel sleeves bolted together through flanges, as shown in Fig. 8.

There are two principal types of such mechanical repairs, the pure mechanical (metal to metal) and the composite type where the capability of the basic repair is enhanced by an intermediate annulus of cement grout. (Other materials have been considered, such as neoprene rubber or resin, but cement grout has been the most widely used material.)

The grouted repair has the advantage of improving the friction grip provided by a metal to metal contact and in addition a close fit between the clamp sleeve and the damaged member is not required. Where the damage has distorted the member to be repaired it may be the only type of clamp it is possible to fit. The disadvantage of a grouted connection is the complications associated with providing a sound infill of grout in the offshore situation, often in very deep water. The most common type of clamp is the T clamp, an example of which is shown in Fig. 9.

Transfer of forces may be provided by a combination of friction grip, bearing and the inherent bending stiffness of the metal sleeve. The overall strength of the sleeve may be enhanced by using a so called 'strong-back' member. An example of this type of clamp is shown in Fig. 10.

In order to give an overall picture of the type of clamps used, the bolt design and the reasons for repair, the Society has reviewed the data for 21 platforms with which it has been involved. A total of 303 clamped repairs have been fitted to these platforms. Table I indicates the type of clamp, whether grouted or steel to steel, the bolt design, whether long or short, and the bolt material as originally fitted. In some cases new bolts have been fitted and the bolt material changed. Table I lists only the original bolt material. It should be understood that because this table represents clamps fitted over a period of years it does not accurately indicate the trend in the design of clamped repairs. The present trend is towards grouted clamps with long bolts and continuous top plates.

Table II gives a breakdown of the causes of damage for which clamped repairs have been adopted. It can be seen that

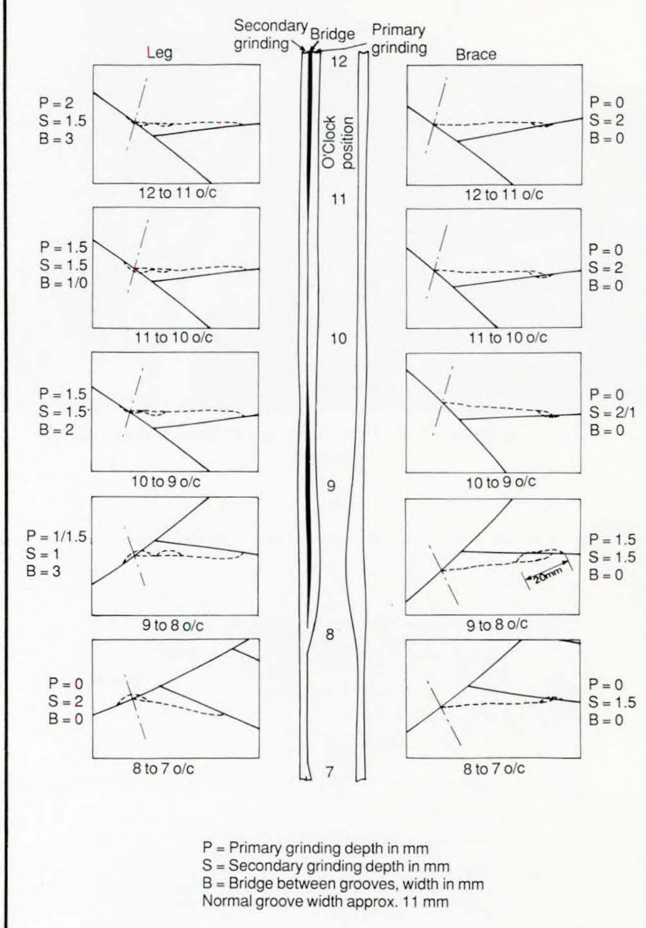


FIG. 7: Typical defect grinding report

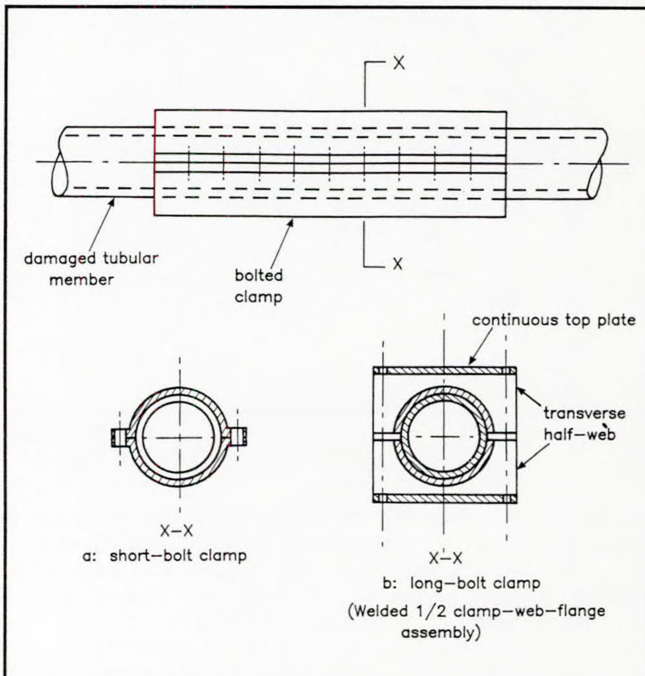


FIG. 8: Basic mechanical repair

Table I: Types of clamp repair

Number of clamp types	*Number of bolt types*	*Number of bolt material types (as originally fitted)*
126 Stressed grouted clamps	123 Long bolts	17 Monel 75 Macalloy 29 L7M 2 HSFG †
	3 Short bolts	2 Monel 1 Ferralium
177 Steel to steel clamps	115 * Long bolts	2 Monel 103 Macalloy 10 L7M
	64 * Short bolts	2 Monel 3 Macalloy 59 HSFG †

* Two friction clamps used a combination of both long and short bolts, hence 177 total clamps, but 179 bolt types

† HSFG bolts — material for bolts unknown other than being high strength steel

Table II: Clamp repairs — cause of damage

Cause of damage necessitating repair	*Number of clamp repairs*
Transportation	3
Installation	9
Dropped objects	2
Platform strengthening	25
Boat impact	3
Fatigue	261
Total	303

fatigue is the principal cause accounting for over 85% of the total number of clamp repairs. It is interesting to note that a clamp repair solution has not often been adopted in cases of boat damage. This is presumably because this type of damage occurs at the splash zone where a welded repair provides a better alternative.

Design of mechanical repairs

The coefficient of friction to be assumed is of primary importance. Tests conducted with bolted steel clamps applied to steel tubulars suggest that a maximum value of 0.25 should apply to the coefficient of static friction for clean mating surfaces. Some designers assume that the clamp acts as a membrane and increase the friction force by a factor of $\pi/2$. This is incorrect as the friction coefficient should be applied directly to the bolt clamping force. Other tests with flat plates have given higher values but these are not considered representative of tubular clamps because the effect of tolerances will be less significant.

Where grouted clamped connections are employed the values of friction assumed may be very significantly increased. This is due partly to higher static friction and partly to the grout bond. However, test data must be provided to justify the higher value of friction.

The maximum ultimate coefficient of friction recommended by the Society for metal to metal connections and for grouted connections where test data are not provided is 0.25. A safety factor is used with this value and those presently recommended by the Society are consistent with the general safety factors given in the API code.[1] These are:

1.25 for maximum environmental plus gravity loads

1.67 for dead and live loads.

Where a characteristic value for the friction coefficient has been established by tests, a higher safety factor may be appropriate. The safety factor used on the grout bond part of the sliding resistance is 4.5.

The calculation of the applied slip force is also uncertain. Most of the test data are based on purely axial load tests. However, real clamps have to transmit all six components of loading. Axial load and torsion can only be carried by friction on the contact surface. Shear loads and bending moments in both planes do not necessarily require a friction

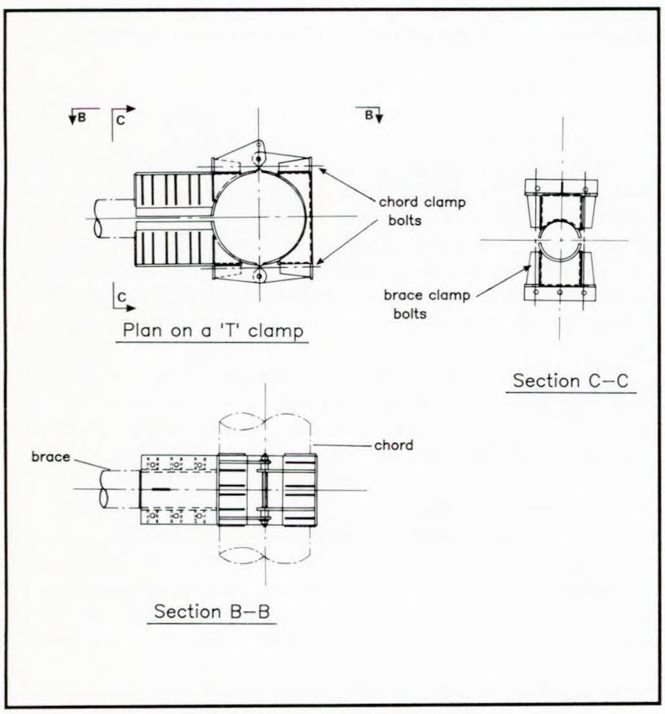


FIG. 9: Example of a T clamp

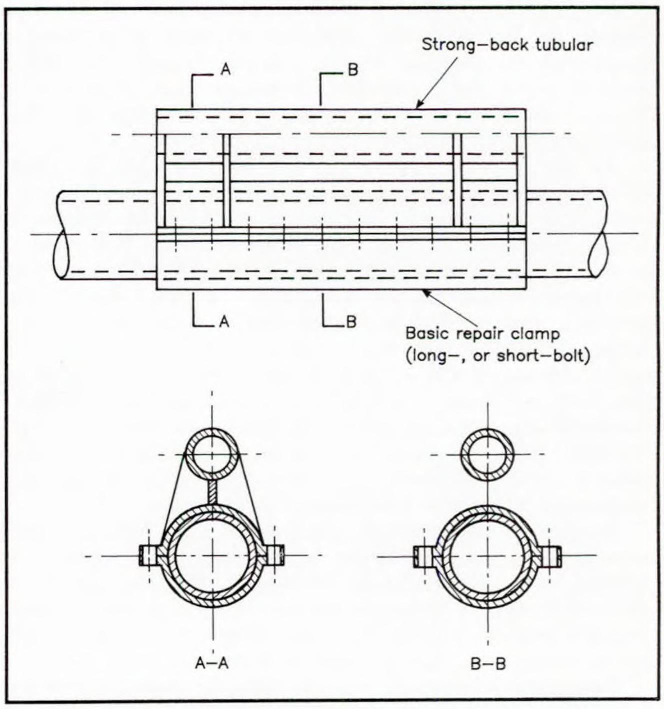


FIG. 10: Schematic diagram of a strong-back repair

connection and some designers make the assumption that 100% of these loads are carried in bearing.

There is a need for more detailed design guidance in this area. In general the Society believes that at least some of the shear and bending moment should be considered as increasing the slip force to be resisted at the joints. In some cases the reduction in the clamp force because of the applied loads must be considered when calculating the allowable slip force available. Care should be taken to ensure that the bolt clamp force is sufficient to maintain a positive contact pressure under maximum loads over the whole of the clamp connection.

Where the required member has suffered a partial fatigue crack it is normally assumed for the clamp design that the crack has propagated completely around the member, ie the residual strength of the member is ignored. The reason for this conservative assumption is the difficulty of accurately predicting crack propagation and the impossibility of further inspection. Some exceptions to this rule have been agreed but very thorough analysis is required together with careful MPI and grinding out of the fatigue crack.

Bolt design

The clamping force is provided by the bolts. Long bolt designs are generally preferred to short bolt designs (see Fig. 8) because the indications are that these provide a more uniform contact between the clamp and the damaged member and greater resistance to nut loosening caused by relaxation or creep of the bolt and clamp elements.

The clamp repairs must be designed for the maximum bolt load applied together with other loads such as hydrostatic pressure.

In determining the friction grip caused by clamping pressure allowance must be made for long- and short-term relaxation of the bolts and the repair element details. To date the Society has applied a reduction factor of 0.8 to the bolt load to account for the above effects (ie bolt design loads increased by 20%). However, experience is indicating that for bolts that are highly loaded, ie up to 0.85 of yield stress, the relaxation could be considerably higher than this. Therefore it is recommended that the initial pre-tension stress of the bolt is limited to a lower value, say to 0.6 of yield stress.

The selection of bolt materials requires careful consideration. The Society has been involved with clamped repairs from the beginning. During this time there have been changes in the materials used for the bolts as a result of experience in service. In the earlier repairs the bolting material used was generally Macalloy bars, followed by Monel bolts until the present time when L7 bolts (BS 4882: 1% chromium-molybdenum steel) have taken over.

As can be seen from Table I, almost 60% of the clamps surveyed have employed Macalloy bolts. During inspection some bolts were found to have failed and were replaced by Monel bolts (later failures were replaced by L7 bolts). As far as the Society is aware the reason for the failure has not been conclusively determined, and many of the clamps with Macalloy bolts have performed in a satisfactory manner. Initial failures were on bolts with threads cut only up to the end of the nut. It was assumed that the thread at the end of the nut produced a stress concentration and the bolt suffered a brittle failure. Bars were then threaded over the full length, although some of these also failed. The reason for these failures was believed to be hydrogen embrittlement encouraged by the high tensile forces on the bolts.

Because of the problems encountered with Macalloy bolts, clamps were installed using Monel bolts and again some failures were experienced in service. In this case the failures were attributed to hydrogen assisted stress corrosion cracking, possibly brought on by the close proximity of the cathodic protection systems which produced hydrogen.

The present trend is to use 1% chromium-molybdenum bolts. Two main types are in use, B7 and L7. Both are similar, the main difference being that the B7 is manufactured for elevated temperature service and the L7 materials have been specially approved for sub-zero temperatures. When using these bolts it would be advisable to use material with a low hardness value as this automatically reduces the tendency towards hydrogen embrittlement.

WELDED REPAIRS

General

The main advantage of a welded repair is that it may enable the structure to be restored to its original condition, depending upon the technique adopted and the local conditions. Welded repairs can be considered:

1. To re-connect or replace damaged jacket members. Such repairs require high standards of strength, toughness and ductility.
2. For temporary attachments, temporary repairs to primary structure, or permanent repairs to secondary structure. Lower standards of strength, toughness and ductility may be acceptable for these applications as long as primary structure is not permanently impaired.
3. For applications such as attaching replacement anodes. Whilst strength may not be critical for this type of repair, toughness, hardness and ductility must be controlled as a permanent weld is required.

Where the damage is above the waterline onshore quality welds may be achieved if special attention is given to protection of welding consumables and to welding procedures to avoid moisture pick-up and the consequent risk of hydrogen-assisted cold cracking. If the original item was stress relieved after fabrication, the repair welds may also require to be stress relieved. When this is considered impracticable procedures may be developed to achieve acceptable defect tolerance in the 'as welded' repair weld. This could involve a combination of enhanced toughness (eg grinding, buttering, control of bead size etc. to increase retempering of the weld metal and parent plate HAZ), reduction of stresses (eg reduction of SCFs, increased weld sizes etc.) and stricter NDE to ensure smaller initial defect size.

Where the damage is below the waterline, some form of underwater welding may be considered, subject to access and quality limitations. Underwater welding can be divided into four basic types with different local environment conditions:

1. Dry habitat welding.
2. One atmosphere dry habitat welding.
3. Local dry spot or limited protection welding.
4. Wet welding.

With the exception of the one atmosphere dry habitat welding, all underwater welding is subject to hyperbaric pressure. Close to the surface the increase is slight, becoming greater with increasing depth. This results in deterioration of the welding arc stability, weld appearance and weld properties during both wet and dry welding. Wet welding is additionally subject to adverse effects of faster cooling and hydrogen pick-up.

The main potential quality problems relate to toughness and the risk of hydrogen-assisted cold cracking, especially where wet welding is concerned. Porosity, slag entrainment and poor weld shape can also be potential problems.

Dry habitat welding

Dry habitat welding is regularly used for the repair of primary structure, since to achieve welded repairs with metallurgical properties comparable to those achieved in similar structures onshore it is normally necessary to exclude water from the welding environment. This may be done by constructing a chamber or habitat around the area to be repaired. The habitat is sufficiently sealed and filled with air or inert gas at a pressure just above ambient. Habitats may be

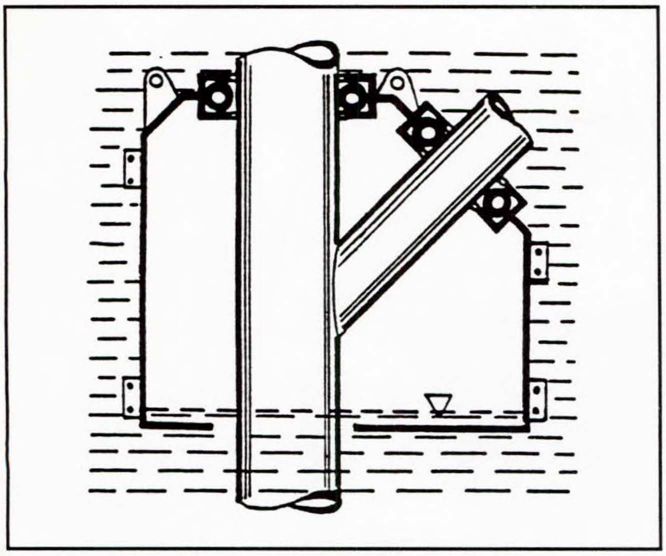


FIG. 11: Specially designed habitat fitted to the diagonal of a steel structure

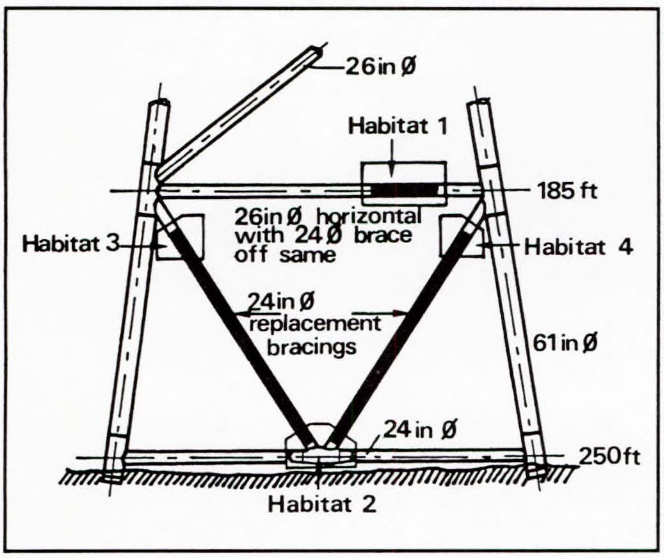


FIG. 12: Elevation showing replacement members and welding habitats used in repair

fully or partly enclosed. A typical cross-section through a habitat is shown in Fig. 11 and typical locations are shown in Fig. 12.

Air may be used down to 50 metres. Argon or helium must be used at greater depths because of the risk of fire or explosion. Below 400 metres either remote control welding or a one atmosphere habitat may be required.

It is generally necessary to modify welding consumables and procedures to compensate for the effects of hyperbaric pressure and high humidity. If necessary, welds may be stress relieved or procedures may be developed to obviate the need for stress relief as outlined above.

One atmosphere dry welding

One atmosphere dry welding could be used for the repair of pipelines below the saturation diving limit and for riser tie-ins. However, this technique is not generally feasible for the repair of jacket structures because of the difficulty of sealing the structure. An exception is the regular use of cofferdams in or near the splash zone.

Local dry spot or limited protection welding

Local dry spot systems can produce weld quality comparable to a full habitat and can be used when a habitat is not practical. However, they may be impractical for positional welding, especially overhead, or for use in confined spaces.

Wet welding

Wet welding has been used for many years mostly in relatively warm and shallow waters where toughness and fatigue are less critical. Because of the generally low quality of weld achieved, wet welding has normally been restricted to temporary repairs, repairs to secondary structures or to such uses as stud welding for the attachment of anodes to secondary structure. It is not considered suitable for any welds to primary structures in the North Sea or similar sites.

CONCRETE PLATFORMS

General

This paper has been primarily concerned with steel structures, which are the most common type of primary structure for offshore platforms. There are, however, a number of concrete platforms, mainly in the British and Norwegian sectors of the North Sea. Some repair work has been necessary on these structures. It is the intention of this part of the paper to review briefly the possible approaches and repair methods that have been developed.

Potential causes of damage to concrete structures have already been mentioned. The main difference between steel and concrete structures from a potential damage point of view is fatigue. Concrete offshore structures have not yet proved susceptible to fatigue and the various research programmes in the last decade do not suggest that there is a problem with existing structures. However, steel decks on concrete structures and any external supporting steel work for conductors etc. may be vulnerable to fatigue.

Approach to repair problems

Regardless of whether the damage is due to corrosion over a number of years identified during periodic surveys or a more immediate incident such as a boat strike, the Certifying Authority needs to assess the situation with regard to the current Certificate. Major incidents may require an analysis of the complete structure in the damage condition as indicated previously.

Where immediate action is required a temporary repair may be necessary to protect the structure while inspection is carried out and a permanent solution is developed.

Design considerations for concrete repairs

Important design considerations relating to the whole structure and the repair area include:

1. Redistribution of loads.
2. Loss of prestressing because of damage or relaxation following a fire.
3. Bond between existing and new materials. (This is normally more important than the absolute strength of repair materials.)
4. Forming a watertight seal.
5. Shrinkage and creep.
6. Local corrosion cells set up between the repair area and existing structure.
7. Durability of the repair.

Materials

Selection of materials has to be based partly on testing and partly on experience. To date research has focused on cementitious materials, polymer-modified cements, and epoxy resins.

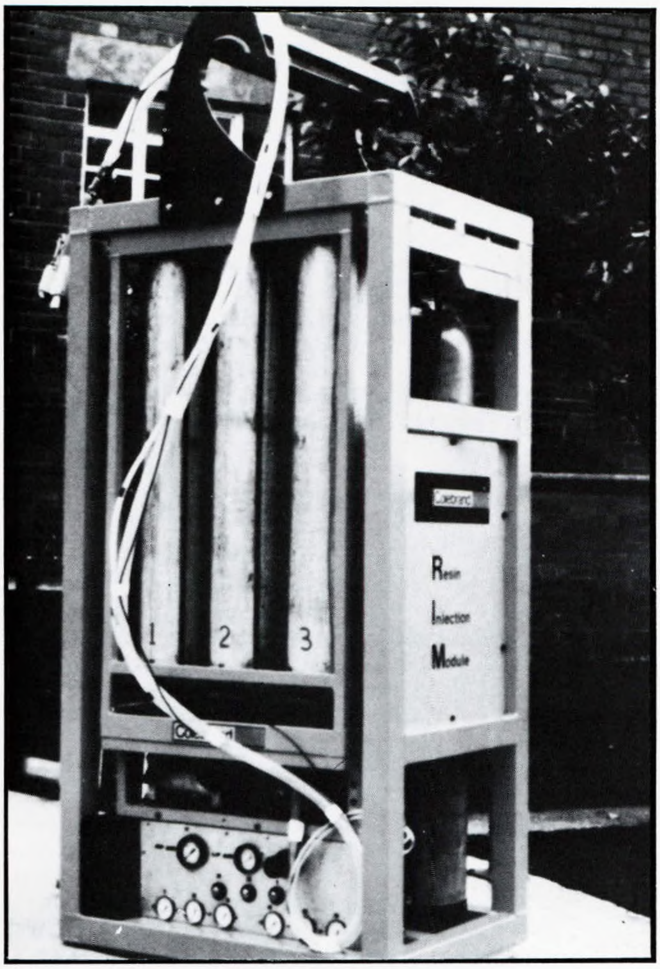


FIG. 13: Epoxy injection

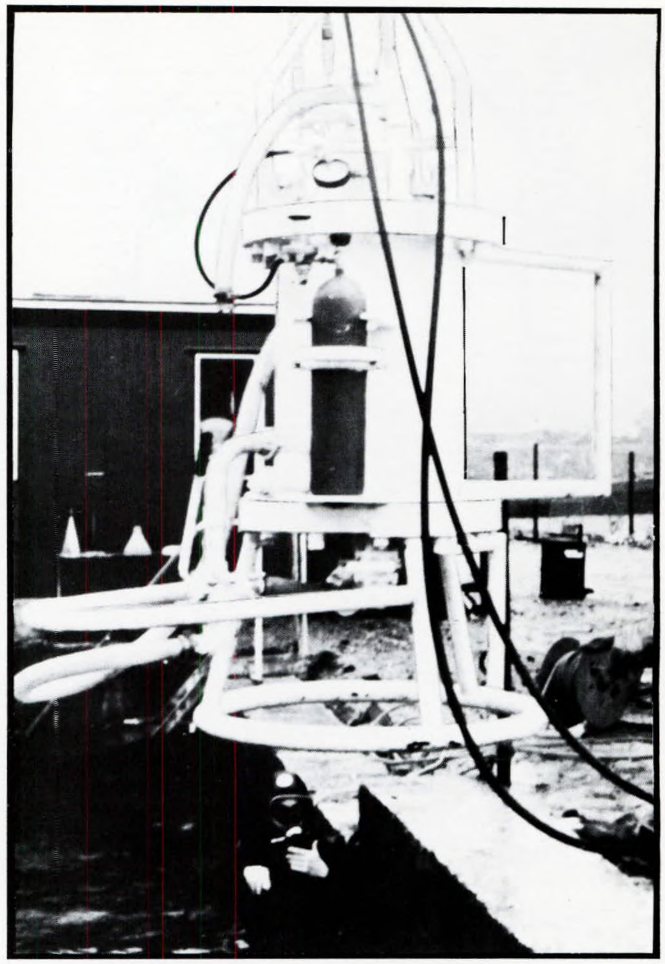


FIG. 14: Grout injection module developed for underwater repair of concrete structures

In selecting materials consideration must be given to placing, handling problems and access to the damage site for cleaning, as well as design aspects. Underwater systems for handling two part epoxies and for injecting grout have already been developed, examples of which are shown in Figs 13 and 14.

Testing procedures for bond, permeability, strength, elasticity etc. have been established and one company's approach is shown in Table III.

Review of repairs and repair materials

The type of damage requiring repair can range from a punching shear type failure caused by boat impact or dropped objects to relatively minor gouges and cracks. With such a variety of problems an appropriate range of solutions and techniques is required. Repair methods have already been developed by companies active in this field and in some cases their techniques have been used in practice.

Large pours underwater require the use of an enclosed shutter. This may mean pre-placing the aggregate and injecting mortar/grout. The final concrete strength may be taken as 80% of the mortar/grout strength. Details of a major repair are given in Ref. 2. Ideally the concrete mix should be close to that specified for the existing concrete. This usually means the cement is OPC or an OPC/Pozzolan blend.

Polymer-modified concretes have been developed for placement underwater without the use of an enclosed shutter. Some care is required when considering their application as problems have been encountered during test work with the bond to existing concrete. A large repair although principally formed with concrete will also have provision for injection of epoxy resin around the perimeter in order to ensure a seal.

For large repairs attachment to the existing structure will primarily be obtained by mechanical means, for example exposing existing reinforcement, anchorage bolts or reinstating pre-stress in the repair area. Connection of reinforcement can be either by lapping bars, couplers or welding.

The ability of a material to bond to the existing structure is more important than absolute strength, in fact for many situations forming a watertight seal is a priority requirement. For this reason considerable research has gone into epoxy resins. Of the synthetic resins available this has proven to be the best choice for bonding to concrete, particularly in wet conditions at low temperatures (8 °C). The other important factor is that epoxy resins can be produced with a very low viscosity, consequently they can be injected into pre-stressing ducts and cracks. They also have low shrinkage during curing.

The disadvantages of epoxy resins are that they have a different Youngs Modulus and coefficient of thermal expansion to concrete; also they have lower resistance to fire than concrete. Because of heat given out during curing, only thin layers can be used and great care is required with the mix proportions. The ability of epoxy resin to bond with concrete over a long period is still the subject of investigation but to date, along with cementitious materials, it forms the basis of current offshore repair work.

A particular problem with trying to achieve a bond to existing underwater members is the speed at which micro-organisms reform on the surface after it has been cleaned. This means that although major cleaning may be done in advance, a final cleaning should be made 12—24 hours before the materials are placed for cementitious repairs and 1 hour for epoxy repairs. Cleaning may be done with high-pressure water jets, pneumatic-powered hand tools and hydraulic-powered tools. Pneumatic tools may only be used at a limited water depth because of exhaust problems.

Where the damage is at the waterline or just below it is possible to construct a cofferdam and repair in the dry.

Cracks in concrete can be caused by overstressing during construction or in service, shrinkage, corrosion or damage caused by impact. Reinforced concrete is designed to crack and it will not normally be necessary to repair crack widths below 0.6 mm in the submerged zone and 0.3 mm in the splash zone. Determining the cause of structural cracking is important as remedial measures other than filling the cracks may be required. To fill the cracks the basic procedure is to seal the crack at the surface and then inject the repair material, usually through small bore pipes inserted into drilled holes. Major crack repairs or cracks over a large area may be sealed at the surface with a shutter prior to injection. An example of this is given in Ref. 3.

Sealing against water pressure has proved to be difficult and even if successful may result in another area of weakness being exposed. One solution is to provide pressure relief valves in the repair, which at least helps stop the problem spreading. The objective with many of the repairs at cracks and construction joints etc. is to reduce the flow of seawater and improve the corrosion situation.

Techniques have also been developed for dealing with gouges and local loss of cover. When the repair is shallow, or the amount of material required is small, an epoxy resin will probably be selected.

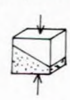
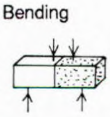
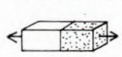
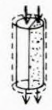
The usual method of application is to erect a transparent shutter (to aid inspection) and inject the resin behind it working from the bottom of the shutter. The surface should be properly prepared prior to erecting the shutter and water should be flushed through for a final clean about 1 hour before resin injection. Attention should be given to the method of sealing the shutter.

For a larger hole or gouge, probably with reinforcement exposed (and for which a cementitious repair may generally be specified), the problem is similar. The engineer will have to be sure that good grout is in the repair. The top of the shutter will normally be open and the engineer may be able to observe using an ROV and/or a return may be obtained using an overflow pipe and a special sample bottle. It is good practice to have the top of the shutter positioned a reasonable distance above the repair.

Considerable research work has been directed at the problem of corrosion.[4] The results are interesting and in some cases contrary to the popular views with regard to corrosion protection. The overall conclusion, however, is that for the current generation of North Sea concrete structures the only area with potential corrosion problems is the splash zone.

The effect of different materials adjacent to each other following a repair has not been fully researched, but where

Table III: Special strength tests for concrete repair

Purpose	*Samples*	*Method of testing*	*Evaluation*
Bond shear strength	Cylinders 6"dia x 12"long	Axial unconfined compression	1. For bonding two concrete surfaces together but could be used for injection of cracks 2. Consistant contact surfaces artificially produced by casting against place, then prepared as in practice or artificially roughened
Shear strength (bond)	Standard 100 mm cubes of concrete cut in half and made up to cube with repair material or two halves bonded together	Axial unconfined compression 	1. Samples of standard size 2. Consistent contact surface artificially produced by cutting which can be prepared as in practice or artificially roughened 3. Similar sample can be cut from large-scale trials and tested 4. Cut contact surface gives severe test of techniques
Flexural strength (bond)	Beams cast 100 x 100 x 500 and cut in half then made up with repair material or two halves bonded together	Bending 	1. Samples of standard size 2. Has been used for testing concrete to concrete bonding for many years 3. Cut contact gives severe test of technique 4. Consistent contact surface artificially produced by cutting which can be prepared as in practice or artificially roughened
Direct tensile strength (bond)	Briquettes or cylinders or prisms cast or cut from cubes or trial repairs	Axial tension 	1. Standard test for resins 2. Easy and quick for laboratory comparisons 3. Disadvantages of smaller sizes of sample can be overcome by using larger sizes and stronger test rigs 4. Care needed to ensure axial loading
Permeability	Cylinders cast or cut 50—100 dia to include repair interface, which must cross top and bottom faces	Water flow under high pressure 	1. Accepted method for measurement of permeability of rock, ie materials with low permeability 2. Will be used initially for single material to determine permeability of concrete and repair materials 3. Water can be dyed then sections cut after pressure test will show visually path of water seepage 4. Very severe test
Permeability	Cubes or cylinder cast or cut with repair interface across section	External pressure applied to cube 	1. Pressure gradient from 1000 lb/in^2 to atmospheric can be used 2. Water flow can be easily measured 3. Quick and easy for laboratory comparisons 4. Samples from test 3 above can be used
Absorbtion	Any sample	Immersion in dyed water for set period under pressure equivalent to depth of repair below sea level	1. Simple test on any shape or size of sample which will give indication of permeability 2. Quantitative results cannot be obtained

chloride-affected concrete is present there will be a different electrochemical potential between the repair and the existing concrete. It is preferred that reinforcement that has been partially exposed by damage is further exposed so that the repair material can get round the full circumference. It has also been suggested[5] that exposed reinforcement should be primed with low viscosity grout. This is to prevent an electrochemical reaction between the reinforcement in the existing chloride-affected concrete and reinforcement in the new concrete.

TOTAL PLATFORM STRENGTHENING AND FOUNDATION STRENGTHENING

In some cases it has been necessary to strengthen a complete jacket structure. There is obviously no standard method of completing this type of reinforcement and each case must be considered on its merits. One such repair for which the Society acted as Certification Authority involved one of the earliest platforms to be installed in the southern North Sea. Other methods of repair had not been successful and the operator decided to install a tripod strengthening tower on either side of the jacket, with the three structures connected above the waterline.

The two towers and their foundations were designed for both strength and fatigue to take the full extreme storm loads with the original jacket bracing assumed redundant but still attracting wave loads. The gravity load, which was comparatively small, was distributed according to structural stiffness. The finite element model used by the Society for analysis and approval of the combined structures is shown in Fig. 15.

Repair to platform foundations may be required for a number of reasons. Service experience such as settlement or tilt may show the foundation to be inadequate or it may need strengthening because of increased deck or environment loads. In some cases more detailed foundation investigations, at a later stage, have shown the foundation to be weaker than assumed in the original design.

Foundation repairs fall into two general categories. The first is the installation of additional piles. The main difficulty with this type of repair is attachment of the piles to the jacket structure. The second method is improvement of the existing piles. A number of methods are available such as driving or drilling and grouting insert piles to a greater depth, belling, providing grout plugs on the soil column, injecting the soil with chemicals etc. The method chosen is very dependent on the soil conditions at the site in question. Work on one of the largest foundation strengthening projects to date is underway on the Woodside N. Rankin A platform, off the coast of Australia. The details of this repair have recently been made public by Woodside.[6]

CONCLUSIONS

As the present generation of offshore platforms get older it is likely that the requirement for repairs will increase. This paper has reviewed some of the repair methods presently available for fixed platforms. However, much of the design data available from research projects dealing with repair methods are confidential and it is therefore impossible to give specific guidance. Nevertheless, the problem areas have been discussed and acceptable methods of approach outlined.

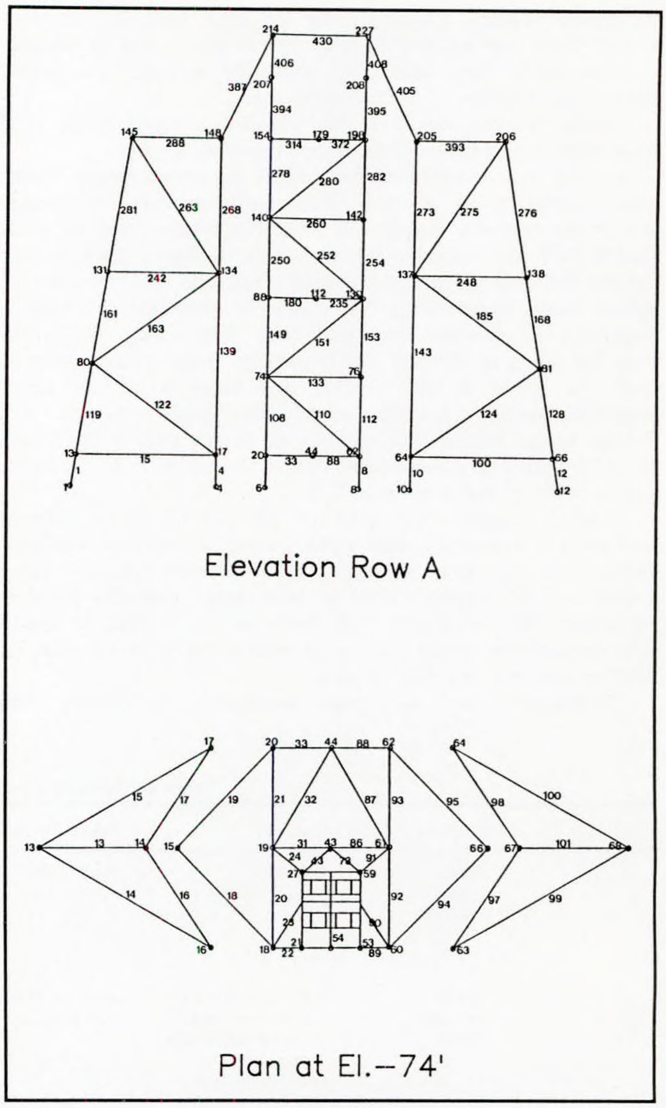


FIG. 15: Finite element model

ACKNOWLEDGEMENTS

The authors are grateful to the Society for permission to publish this paper and thank McAlpine Offshore Ltd for the provision of photographs and Table III.

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Discussion

A. R. MCINTOSH (Department of Energy): First of all, I should like to congratulate the authors for presenting us with such an interesting review of offshore structural repairs.

I note their conclusion that the requirement for repairs is likely to increase as platforms get older. Could the authors give an indication of the rate of repairs necessary in recent years and whether there is any trend to be observed. For instance, a large number of repairs were made necessary because of inadequate initial design. As these have been rectified, and the newer installations designed to more appropiate standards, this population of repairs could be expected to decline. Perhaps the authors could be tempted to speculate on future repair requirements.

One of the interesting aspects in the paper is the increasing difficulty and cost of repairs in deep water. Is there a practical limit to the water depth in which repairs can be made?

It would be interesting if the authors could comment on their attempts to identify the causes of defects and how they have used this information to improve their methods of analysis and acceptance criteria.

The authors have commented on boat damage. The Department of Energy has commissioned a number of recent studies on this topic. Collisions or impacts have been considered in two distinct categories:

1. Collisions involving passing vessels.
2. Impacts involving attending (or service) vessels.

The possibility of designing structures to withstand collisions involving passing vessels has been discounted for practical purposes. Attention has therefore been focussed on estimating the risk of such an event, which varies enormously from location to location depending upon proximity to shipping routes and shipping density. However, the second category is very different. Impact damage is all too common. There were 145 cases reported to the Department of Energy in the 10 years to 1985. Clearly, structures must be able to withstand events of this frequency. We are having guidance prepared on this topic and expect to issue a consultative document towards the end of March. Part of the study involved a detailed investigation by Lloyd's Register of Shipping of the impact energy absorbed by the structure in the worst 11 cases of impact damage. Perhaps the authors would comment briefly on this work.

On the subject of dropped objects, could the authors estimate what proportion of these incidents occurred during the construction phase.

As regards bolting materials, it may be of interest to know that a joint industry research project on the behaviour of high-strength bolts in seawater is in progess at Harwell.

It may be worth noting that Lloyd's Register of Shipping has recently reported work on re-analysis of early North Sea structures and fatigue correlation studies. Perhaps the authors would care to comment.

Finally, the Department of Energy is preparing guidance on repairs. This is currently the subject of consultation with industry and the certifying authorities.

P. BUSBY (Atkins Oil and Gas Engineering Ltd): I should like to thank the authors for a most interesting paper. Repairs to offshore structures are not to be undertaken lightly as they can involve substantial risk and are very expensive both in terms of initial cost and subsequent requirements for inspection.

The authors refer to the need for analysis both prior to committing to repair and, if repair is deemed necessary, to the post-repair situation. I would agree with this but would like to emphasise the importance of the 'quality' of such analyses. Simplistic, linear, 'design type' analyses are totally unsuited for such assessments and can result in unnecessary and expensive repair decisions which, in some cases, can even worsen the situation. Most real structures behave in a non-linear way to some degree and this must be recognised when carrying out assessments of damage or reported overstress.

To illustrate this I would like to present a simple example of the non-linear behaviour of a X-braced jacket structure. Figure D1(a) shows a transverse frame from a fairly typical southern North Sea jacket. For the purpose of this example, the axial loads shown are thoses obtained from a conventional 'rigid joint' design analysis. The lower X-joint is at punching failure under 50 year storm axial load of 225 kips plus combined in-plane and out-of-plane bending stress.

If the non-linear load/deflection and moment/rotation characteristics of the joint are taken into consideration, a very different picture emerges. Typical P curves as obtained from test and associated M against ϕ curves are shown in Figs D2 and D3.

Performing a non-linear analysis using these characteristics, whereby moment shedding and axial load shedding can occur as the joint goes plastic, the load factors shown in Fig. D1(b) and (c) are obtained. At a load factor of 1.8 times the 50 year storm load the capacity of the joint is reduced to zero and the axial compression in the compression brace reaches its ultimate axial load.

As can be seen from Fig. D1(b) there has also been some axial load shedding from the compression to the tension brace. With further increases in applied load the axial load in the tension member of the X-brace pair increases, with no further increase in the compression brace, until at a load factor of 2.2 the combined axial tension and bending stresses in the through brace reach yield and the frame is more or less at failure. This is a realistic example and the X-joint in question was a candidate for a clamp repair based on

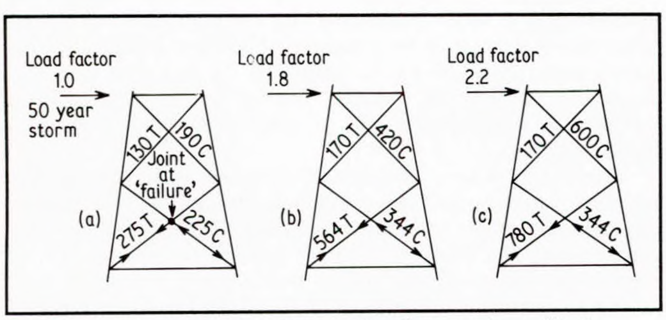

FIG. D1: Transverse frame of typical southern North Sea jacket

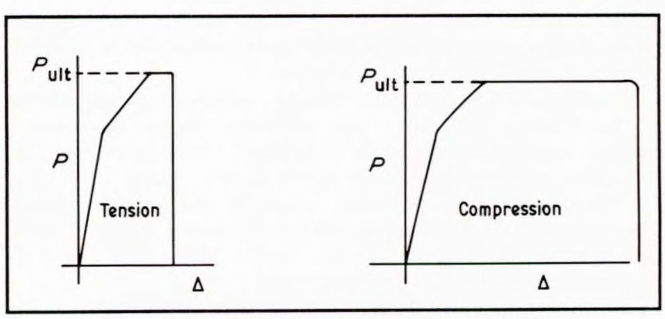

FIG. D2: Typical P curves obtained from test for the jacket structure of Fig. D1

conventional analysis prior to the non-linear assessment being performed.

Turning to the post-repair scenario, the above transverse frame is part of an eight-legged jacket and is parallel with three similar frames. An analysis was performed simulating one lower X-joint repair as a rigid joint and the remaining joints as non-linear elasto-plastic as described above. The effect was that the rigid joint attracted 25% more axial load than would be indicated by a conventional analysis with all the joints modelled as rigid. This result from the stiffening effect of repairs is fairly typical.

To summarise, do not commit to repair unless a rigorous analysis proves it to be necessary, and if repair is still deemed to be required ensure that the analytical model of the post-repair condition is realistic. Engineering is cheap compared with the cost and risk of putting steel underwater.

D. BROWN (British Gas plc): As the structures in the North Sea approach maturity, the whole area of damage assessment and repair is bound to assume increasing importance. British Gas is certainly aware of the need to develop technology in the fields of inspection, defect assesment and repair and this has been reflected in in-house R & D projects and in our support of relevant industry group sponsored projects.

One type of repair mentioned in the paper, in which we have been particularly active, is the use of redemial grinding as a means of stabilising fatigue cracks in nodal welds of steel structures.

This can be an extremely effective technique and it does have the added attraction of being far less costly than more radical solutions such as underwater welding or mechanical clamps.

I would like to add one or two additional suggestions to the points made by the authors.

First, before commencing remedial grinding it is essential to establish beyond reasonable doubt that the crack in question is a fatigue crack and that the profile of the crack is known to be within prescribed limits. Otherwise remedial grinding may not be appropriate and there may be a danger of extensive chasing of sub-surface cracks, due to other causes.

Within British Gas we would normally carry out a detailed local stress analysis in order to confirm the existence of high cyclic stress. This would be followed by detailed local inspection, with particular emphasis on the measurement of crack depth, in order to establish the profile of the crack.

Having decided that grinding is the appropriate form of remedial treatment, it is essential to lay down a procedure which includes frequent checks in order to ensure that the diver does not remove more metal than is absolutely necessary. The one thing I would add to the authors' proposals in this respect concerns the procedure for measuring grind depth. Our experience has shown that simple mechanical profile or depth gauges can give a misleading measurement of grind depth. This is due to the complex topogaphy at tubular joint intersections. Our approach is to measure the thickness of the remaining ligament within the excavation and hence infer the amount of metal removed.

It is clear that the benfit of remedial grinding derives from the significant increase in life to initiation. Recent R & D within British Gas has shown that within the initiation phase, fatigue damage has a much greater dependency of stress range than within the propagation phase.

During crack propagation 'fatigue damage' is proportional to the third or fourth power of stress range. In contrast, during the initiation phase 'damage' is proportional to something approaching the tenth power of stress range.

This means that, for the initiation phase, the extreme waves, ie those with a return period of several years or more, are the critical ones in the sense that they have a predominant effect on life to initiation.

In view of this, would the authors comment on the principle of gearing periodic inspection of remedial grinding sites to the recorded incidence of extreme waves.

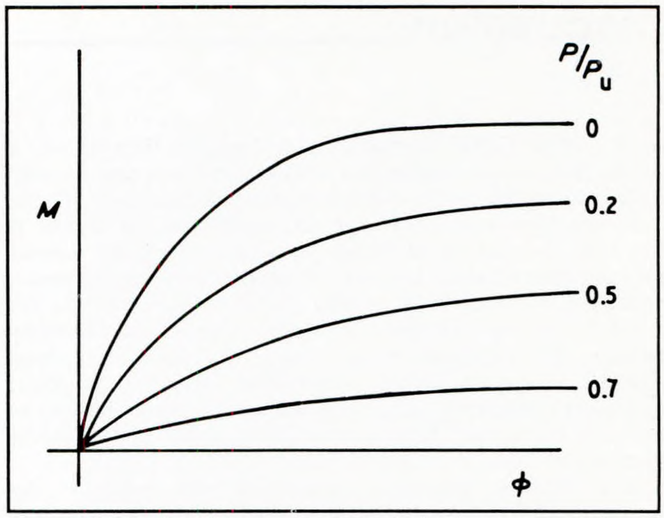

FIG. D3: M against ϕ curves for the jacket structure of Fig. D1

P. E. G. O'CONNOR (Amoco UK Exploration Co.): This paper provides an excellent overview on the subject of structural repairs as normally adopted in the offshore industry.

Amoco have 27 structures in the southern North Sea and two in the northern North Sea, all steel. We have experienced most types of repairs described in the paper, but note that there is no mention of 'grout-filled' members as a repair option. I would therefore like to add a few words on the subject.

Grout-filled tubular members are used to enhance strength and improve lives of tubular joints and members, to stabilise existing damaged joints and/or members by preventing further deformation, and to facilitate installation of stressed clamps. The advantages are that this method is very cost effective in the right situation, is simple and quick to carry out offshore, and leaves joints and members clear for future inspections and monitoring.

However, careful analysis is required to represent relative stiffnesses of the repaired joint and other joints in the structure. Care is also required when filling horizontal water-filled members, because of the presence of a water bubble which moves arbitrarily in a simlar way to a bubble in a spirit level. In addition, verification of work is difficult because of the lack of effective NDT equipment for the detection of voids

D. GARNETT (Wimpey Offshore Engineers and Constructors Ltd): Since the cost penalties for extending offshore and (especially subsea) operations is high, an important feature of grouted clamps and connections is their inherent ability to accommodate lack of fit and provide the installation tolerance required to ensure successful first-time application.

The continuous top plate type of clamp is preferred not only because of the design flexibility the arrangment provides (for example connecting the top and saddle plates by 'side plates' creates a box section of similar stiffness to the 'strong-back' type of clamp illustrated in Fig. 10) but also because it supports the exisiting tubular member against hoop collapse. A further advantage of long studbolts is their high fatigue resistance.

Unstressed grouted connections between concentric tubular members can be made by short bolt flanges on split sleeves and extends the established practise of grouted pile/leg connections. The capacity of this type of connection can be signifantly enhanced by using weld beads or stud shear connectors. Design guidance is included in the Department of Energy Offshore Design Guidance Notes.[1]

Test data are available to demonstrate a friction coefficient between cementitious grout ($f_{cu} \geq 40$ N/mm^2) and blast cleaned steel of 0.33. Results of a comprhensive research programme, the Joint Industry Repairs Research Project (JIRRP), which was sponsored by the Department of Energy and nine offshore operators, are soon to be published. This project extended over two years and involved full-scale static and fatigue tests on repaired tubular members in order to produce detailed design guidance for grouted, mechanical and stressed grouted connections and clamps.

Macalloy bolts are currently not generally adopted because of ealier stress corrosion and corrosion fatigue related failures exacerbated by high working stresses.

Alloy K500 ('Monel') studbolts have more recently suffered failure by hydrogen enbrittlement. The failures occured in high-stress applications where cathodic protection systems were active, and were primarily due to high local hardness resulting from the thread forming operations of manufacture.[2] Failures of this nature can be precluded by specifying heat treatment after instead of before thread formation, removing sulphurised cutting compounds before heat treatment, limiting material hardness to 35 on the Rockwell C scale and limiting the maximum bolt stress to 60% of the 0.2% material proof stress.

It should be noted that the 'L7M' as written in Table I refers to modified L7 material which is not the same as metric series L7 bolt material 'L7'. It is safer therefore when specifying this material to use the phase 'metric L7'.

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Dr P. A. FRIEZE (London Centre for Marine Technology, University College London): The authors recommend that normal API-RP2A safety factors (1.25 and 1.67) should be used in the design of metal to metal connections together with a coefficient of friction of 0.25. Although the API factors are not rational,[1,2] they are considered acceptable by the engineering fraternity because of the many years of experience reflected in their use.

However, the same cannot be said about these connections. They have a relatively short history so, if for no other reason, larger safety factors should be used until experience demonstrates they may be reduced.

The authors then suggest that 'where a characteristic value for the friction coefficient has been established by tests a higher safety factor may be appropiate'. Is this rational? Usually where tests are available they have the benefit of lessening our ignorance thereby providing the opportunity to reduce safety factors, unless they were too small in the first place. I would be interested to see how the authors justify their statement.

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Dr W. VISSER (Shell UK Exploration and Production): I am sure the authors agree that damage repair should not be rushed into, and indeed not all damage situations will require a repair to be carried out. Therefore carefully planned damage inspection and detailed analysis should be undertaken before a decision is made.

An interesting point of discussion is the criteria for allowing continued operation of a damaged platform (50 year seasonal storm, fatigue life until repair, acceptable loss of redundancy). I would suggest that there is scope for discussing the acceptance of less onerous critera for unmanned platforms.

Authors' reply

Mr McIntosh raises a number of interesting questions and we will deal with them in order. Concerning requirements for future repairs, our comments in the paper refer to the older existing platforms where we believe there may be an increased requirement for repairs in the future. On new platforms however, with proper application of the latest design methods and sound fabrication and inspection procedures there should not be the number of problems that have occurred on some of the existing structures.

The maximum practical depth at which repairs can be made must be limited in general to the maximum depth at which a diver can operate. However, with specially designed remote control equipment it must be possible to make repairs at a greater depth.

We entirely agree with the general comments made regarding passing vessels. Two passing vessel cases were examined in the study and in one of these the total energy available was 44 MJ although in this particular case, because of the angle at which the boat struck the platform, only 3.0 MJ of energy was absorbed by the structure. It would be very expensive to design platforms for the total level of energy involved in the above collision.

With regard to the other nine cases the maximum impact energy was 1.5 MJ. This is the level of energy absorbed in deformation of the local members only. Some additional energy would also have been absorbed in overall deflection of the structure but this was not calculated in the study.

We have not attempted to collect detailed statistics on dropped objects but from our own experience we would say that nearly all occur either during installation or during later repair work.

Regarding the correlation of analysis results with in-service experience, the Society has carried out a great deal of this type of work particularly with regard to fatigue. Some of this work has been funded by the Department of Energy and published by Her Majesty's Stationery Office.[1-3] A considerable amount of unpublished work has also been done. Both those platforms with in-service fatigue failures and those with low predicted fatigue lives that subsequent inspection has shown to be uncracked have been studied. Using the latest criteria and a sophisticated deterministic fatigue analysis we have generally obtained very good correlation between the calculated fatigue lives and the service experience.

We would endorse the points made by Mr Busby. The only comment we would add is that fatigue must also be considered because non-linear redistribution will not normally give any benefit where fatigue loading is concerned.

In reply to Mr Brown, we believe that consideration can be given to gearing periodic inspection of remedial grinding sites to the recorded incidence of extreme waves. However, each case would be treated on its merits and some degree of periodic inspection would be required regardless of the weather.

We agree with Mr O'Connor's remarks and have nothing to add.

The comments made by Mr Garnett are noted and again we have no further comments to make.

In response to Dr Frieze, for design a coeffeicient of friction 0.25 is recommend, which is based on a very limited number of tests carried out as early as 1980. The value of 0.25 established by these tests was considered a lower bound value and at the time a safety factor of 1.25 was applied. Since then a comprehensive series of tests have been carried out as part of the Joint Industry Repairs Research Project (JIRRP). The project indicates much higher values for the coefficient of friction based on a characterisitic value and also recommends higher safety factors.

While the authors are aware of this work, the results are not yet in the public domain and therefore cannot be used in a general paper. The reference in the paper to test data and higher factors is an acknowledgement of this work.

Dr Visser has raised an interesting point. Where there is a national certification scheme in operation (such as the UK sector of the North Sea) then the National Authority has the final say in such matters.

In our opinion there may be some justification for reducing the criteria for acceptance of damaged unmanned installations and this has in fact been agreed in some cases. However, measures must be taken to ensure that there will be no significant pollution in the event of failure and, in the case of a structure which may remain floating after failure, that it does not become a hazard to shipping. Finally the insurers and other interested parties must be informed of any increase in the risk.

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