



## Review

# Experimental testing of grouted connections for offshore substructures: A critical review



Paul Dallyn <sup>a,\*</sup>, Ashraf El-Hamalawi <sup>b</sup>, Alessandro Palmeri <sup>b</sup>, Robert Knight <sup>c</sup>

<sup>a</sup> CICE, Centre for Innovative and Collaborative Construction Engineering, Loughborough University, Loughborough, UK, LE11 3TU

<sup>b</sup> The School of Civil and Building Engineering, Loughborough University, Loughborough, UK, LE11 3TU

<sup>c</sup> Civil Engineering, E.ON New Build and Technology, Nottingham, UK, NG11 0EE

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## ABSTRACT

Grouted connections have been extensively used in the oil and gas industry for decades, and more recently their application has been extended to the offshore wind industry. Unfortunately plain-pipe grouted connections for large-diameter monopile foundations have recently exhibited clear signs of insufficient axial capacity, resulting in slippage between the transition piece and monopile. Motivated by the emergence of such problems, this paper presents a critical review of the technical literature related to the experimental testing for grouted connections for offshore substructures, covering all the key material and design parameters that influence their capacity, including the confinement provided by pile and sleeve, surface finish, simultaneous bending action, connection length, dynamic loading, early-age cycling during grout curing, grout shrinkage, radial pre-stress and temperature. The review also focuses on the relevance of such parameters for offshore wind applications and addresses what needs to be considered to ensure that their design achieves the desired capacity, behaviour and efficiency.

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## 1. Introduction

Grouted connections have had extensive applications for the foundation of oil and gas platforms, where they have been used for main, skirt and cluster piles, as shown within Fig. 1. A grouted joint is a structural connection formed by use of cementitious grout cast in an annulus formed between two concentric circular tubes with different diameters. The principal methods of load transfer are through shear

friction mobilised by the normal stress induced through interlocking of surface imperfections and compression of the grout.

With the aim of optimising the design of platform foundations and reducing material quantities, extensive work was carried out in the late 1970s and early 1980s to quantify the performance of both plain-pipe and shear-key grouted connections through experimental testing, particularly for offshore applications. This stream of work was predominantly focused on the influence of grout strength, shear-key height and spacing, ratio of diameter to thickness of the piles, outer sleeves and grout annulus on the ultimate capacity of the connection.

Since 2002, grouted connections have also been used extensively in the offshore wind industry, where large-diameter-sleeved grouted connections comprise around 60% of installations in Europe [2]. Some

\* Corresponding author.

E-mail addresses: [P.A.Dallyn@lboro.ac.uk](mailto:P.A.Dallyn@lboro.ac.uk) (P. Dallyn), [A.El-Hamalawi@lboro.ac.uk](mailto:A.El-Hamalawi@lboro.ac.uk) (A. El-Hamalawi), [A.Palmeri@lboro.ac.uk](mailto:A.Palmeri@lboro.ac.uk), [Dynamics.Structures@gmail.com](mailto:Dynamics.Structures@gmail.com) (A. Palmeri), [Bob.Knight@eon.com](mailto:Bob.Knight@eon.com) (R. Knight).

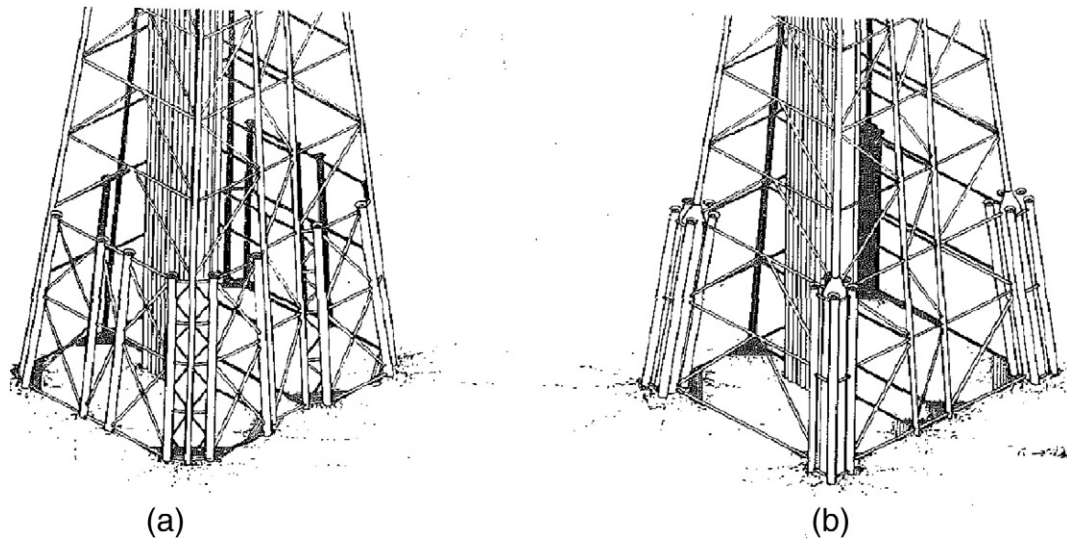


Fig. 1. Typical foundation pile arrangements where grouted pile – sleeve connections are used; a) Main and skirt Piles, b) Cluster piles [1].

typical details for an energy converter 80 m tall, capable of 80GWh/year are shown in Fig. 2. Unfortunately it has been reported that the axial capacity of these plain-pipe grouted connections may be insufficient over the design life of the plant, with significant unexpected early-stage settlements resulting from this insufficient design [3]. This has led to very expensive ongoing remediation works being required to existing

foundations affected by these failures. In addition to the use of grouted connections for monopiles, they are starting to be more widely used between the pin piles and jackets for offshore WTG (wind turbine generator) in sites with deeper mean water level (MWL), with the market share of jackets and tri-pile substructures increasing from around 10% to 20% from 2010 to 2013 respectively [2]. The reason for the continued

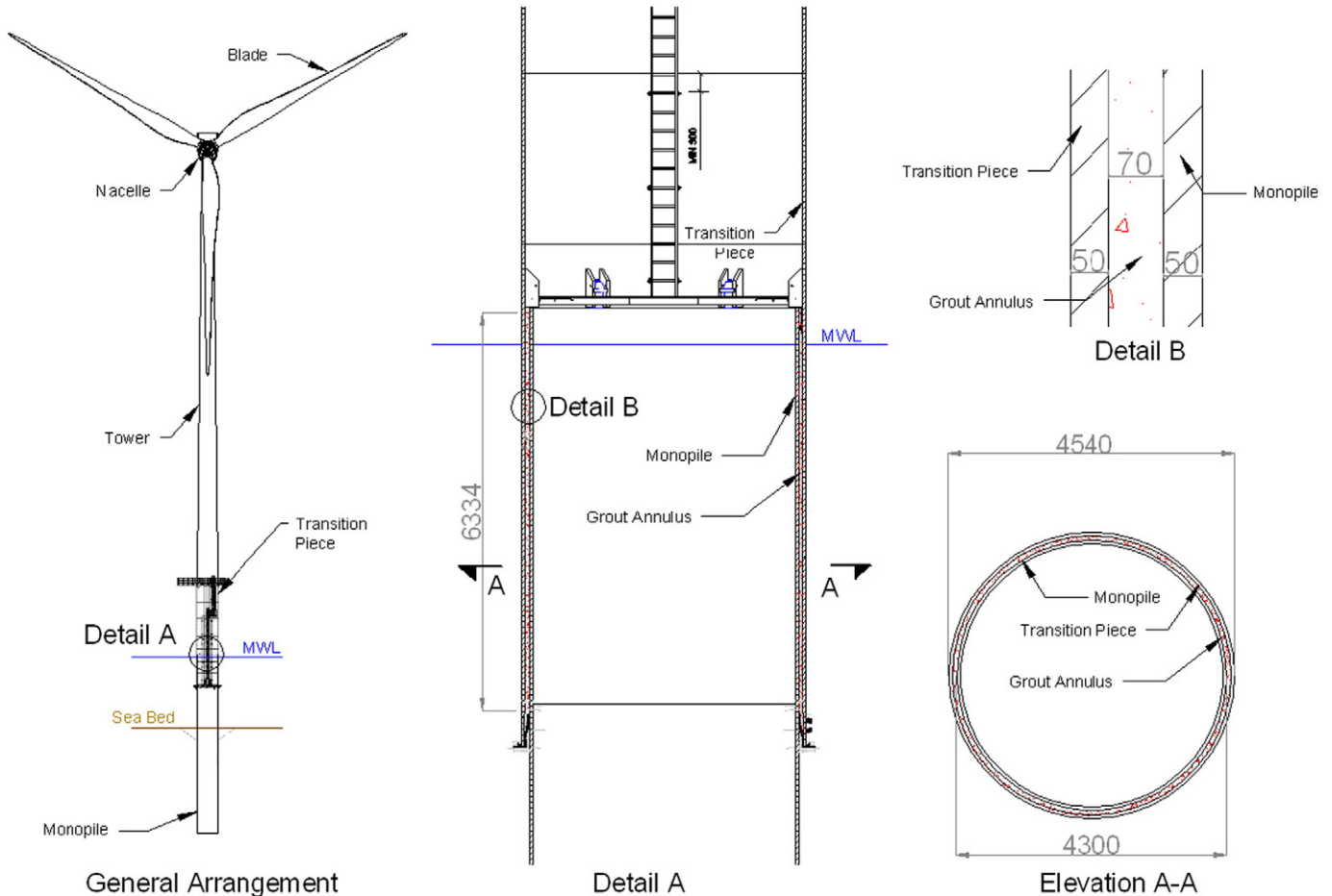


Fig. 2. Typical general arrangement of a plain-pipe grouted connection for an offshore WTG foundation.

use of the grouted connection is that with the inclusion of shear-keys it offers a cost-effective way to provide an efficient structural connection, while being able to accommodate piling installation tolerances.

The unsatisfactory performance of grouted connections reported in recent offshore wind installations, such as Horns Rev 1, Kentish Flats and Belwind, as well as the predicted growth of this highly strategic energy sector in future years, motivate this review paper, which is mainly focussed on grouted pile sleeve connections, with the aim to understand whether: i) the cost of these failures could have been avoided; ii) occurrence of similar issues can be reduced in the future. References pre-dating 2007 have been reviewed to provide a detailed assessment of the level of knowledge and understanding within the offshore wind industry at the time when grouted foundations with insufficient capacity were designed. As detailed in the following sections, the majority of these references consist of research related to the behaviour of axially loaded connections, as used in the oil and gas industry for jacket structures, while the effects of bending actions were marginally investigated before 2007, predominantly in relation to the loading conditions experienced by wind turbines. An account of research carried out in more recent years is also given, to identify current trends and challenges.

For the sake of clarity, the literature review has been broken down into sections based on the different areas investigated by the papers and the relevance of the testing methodology to offshore wind applications, with papers presented chronologically to demonstrate how industry knowledge has developed over the years. A summary of the key testing information from each paper is presented in Table 2 (which includes also a comparison with typical offshore WTG foundation grouted connections, for both monopile and jacket pile connections), which accompanies and supports the key conclusions of previous work and recommendation for future research.

## 2. Static axial testing

Billington and Lewis [4] have carried out pioneering work on the experimental assessment of the axial capacity of grouted connections in the 1970s. They have used the results of over 400 tests to determine the effects of surface condition of steel, radial stiffness, grout properties and length-to-diameter ratio. Key grout properties included Young's modulus, compressive strength and expansion/shrinkage during curing. In particular, the influence of shrinkage on full-scale specimens was investigated, showing that even though compressive strength of grout increased by 61%, effects of shrinkage were to reduce the bond strength by 42% for plain-pipe connections; while for shear-key connections there were no noticeable effects of grout shrinkage or expansion on bond strength. Bond strength is defined as the ultimate axial capacity of the connection divided by the surface area of the grout annulus/pile interface. It was also shown that variations in surface roughness have a dramatic effect on bond strength of plain-pipe connections. Piles tested with typical surface rusting, as used in practice, showed greater variance in results and up to 25% less bond strength than shot-blasted surface test samples (Fig. 3). Having used over 30 tests to derive each curve, these results are statistically significant and so there is confidence in the results presented.

Tests on plain-pipes indicate that the reduction in radial stiffness, which accompanies an increase in pile diameter ( $D_p$ ), for a given wall thickness ( $t$ ), can lead to unacceptable reductions in ultimate axial bond strength. Axial capacity testing was undertaken on eight samples of varying stiffness factors  $K$  (Eq. (1)) and showed limited scatter.

$$K = \left( \frac{E_g}{E_s} \right) (t/d)_g + \left[ (d/t)_s + (d/t)_p \right]^{-1}, \quad (1)$$

where subscripts g, p and s denotes grout, pile and sleeve respectively;  $t$  is the thickness;  $d$  is the diameter; and  $E$  is the Young's Modulus.

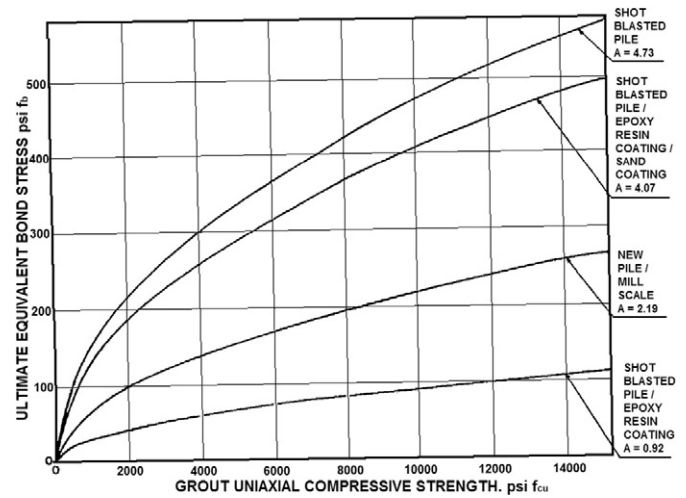


Fig. 3. The effect of surface roughness on the relationship between bond strength and compressive strength for plain-pipes; where  $A$  represents the plain-pipe factor, which depends upon surface roughness and pile/sleeve geometry [4].

Based on this, it was shown that the relationship between bond strength ( $f_b$  or  $f_{bu}$ ) and grout cube strength ( $f_{cu}$ ) could be represented by the empirical expression:

$$f_b = B \cdot K \cdot f_{cu}^{1/2}, \quad (2)$$

where  $B$  is a factor dependent on length-to-diameter ratio and surface roughness. The authors compared these findings to the bond strength predicted by the American Petroleum Institute (API) and the Department of Energy (DOE) and found that the safety factor reduced to 1.5 for low stiffness connections, well below the recommended value of 6.0. For shear-key connections the results are only based on five

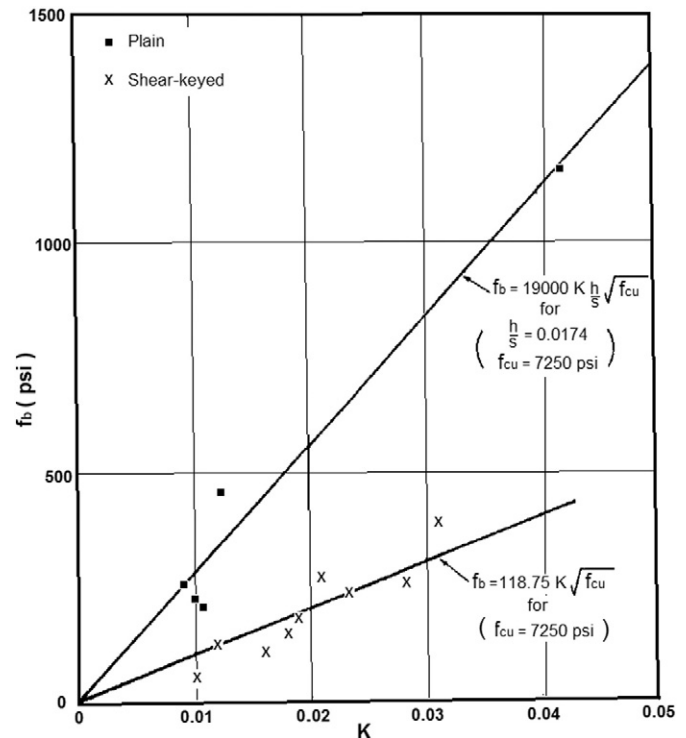


Fig. 4. The effect of stiffness factor on bond strength for both plain-pipes and pipes with mechanical shear-keys, adapted from [4].

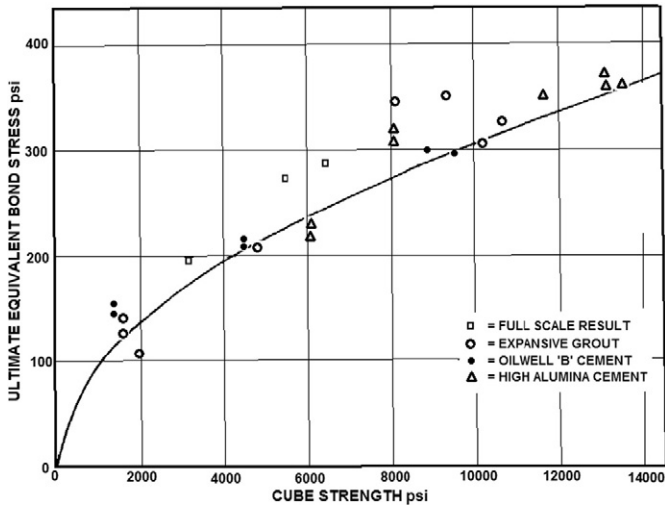


Fig. 5. Results of 1:4 scale bond tests using three different grouts compared with full-scale results [4].

samples and the distribution of stiffness factors is very limited, and so the significance of the derived relationship is more limited. For shear-key connections a relationship between bond strength and stiffness factor was derived in the form:

$$f_b = C \cdot K \cdot (h/s) \cdot f_{cu}^{1/2}, \quad (3)$$

where  $C$  is a factor dependent on length-to-diameter ratio; and  $(h/s)$  is the shear-key height divided by spacing. The devised relationships between bond strength and stiffness for both plain and shear-key connections are shown in Fig. 4, demonstrating the enhanced performance of the second type of connection, whose mode of failure consists of crushing of the grout ahead of the shear-key and diagonal cracking originating from the head of the shear-key.

Comparison of full-scale and small-scale (1:4) tests on shear-key connections were reported to give directly comparable results in terms of bond strength (Fig. 5). However, no testing was carried out on plain-pipe connections and so the influence of scale effects on this connection type was not assessed. In addition, the maximum full-scale grout strength was 45 MPa, while agreement cannot be confirmed beyond those values.

The test results were stated as being conservative due to the pile being in tension because of the loading arrangement and so the corresponding effects of the Poisson's ratio tend to separate the steel from the grout. Moreover, in this pioneering study there is no mention of the influence of the increased effective radial stiffness of the pile and sleeve provided by the loading plates due to their close proximity to the grouted connection.

The same data set as in Ref. [4] was further analysed by Billington [5], pointing out that large scatter in the experimental results, particularly for plain connections ( $h/s = 0$ ) needs careful consideration (Fig. 6).

It is confirmed that bond strength is proportional to the square root of grout compressive strength,  $f_{bu} \propto f_{cu}^{0.5}$ . Research into the benefit of composite structures was also presented, which showed significant stress reductions in principal stresses of around 60% for the grouted chords and braces compared to similar non-grouted configurations. The results were reasonably significant from a statistical point of view, as they were based on five samples for each type. Some results were also presented for the influence of bending actions on the axial capacity of annular connections, which are reviewed and discussed in Section 5.

Billington and Tebbett [6] continued the work presented in Refs. [4] and [5] to derive DOE formulae for the ultimate capacity of plain and shear-key grouted connections, filling some gaps and further increasing the significance of the results previously derived. They also presented results of subsequent phases of testing, looking specifically at the effects of cyclic loading. A more detailed investigation into the partial safety factors applied for assessing the ultimate bond strength, with reference to their laboratory work experience, indicated that an overall safety factor of 4.5 could be used, rather than the larger value of 6.0 commonly adopted in the offshore industry at that time. Although the paper states that for plain connections the ultimate bond strength ( $f_{bu}$ ) is proportional to the square root of grout cube strength ( $f_{cu}$ ), the results

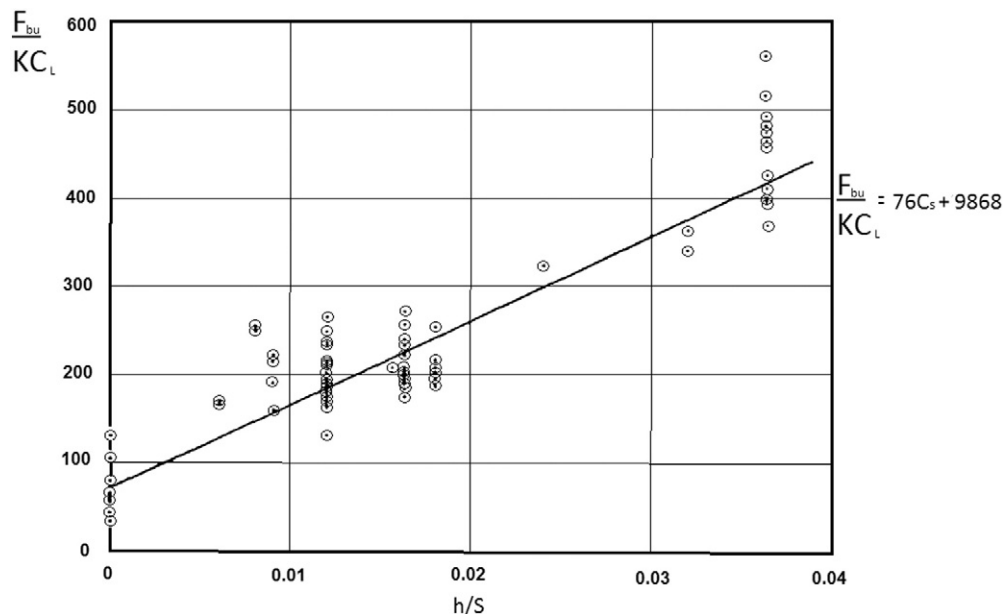


Fig. 6. Relationship between  $F_{bu}/K C_L$  and  $h/s$  [5].

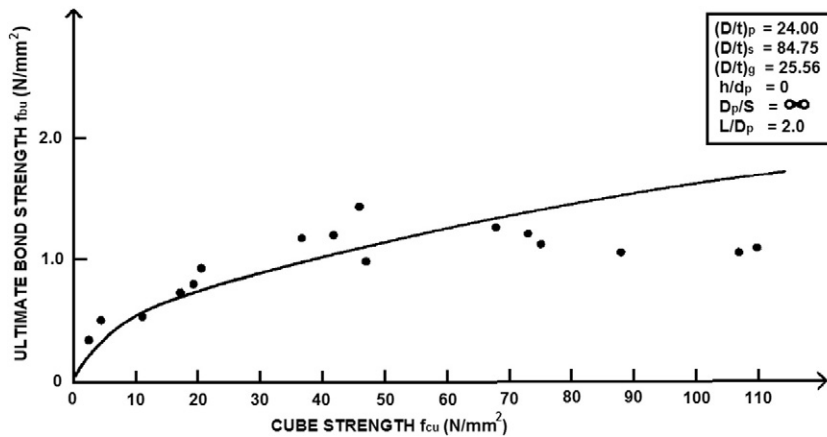


Fig. 7. Relationship between bond strength and compressive strength for plain-pipes [6].

presented in Fig. 7 indicate that for values of  $f_{cu}$  larger than 45 MPa,  $f_{bu}$  does not increase further, at least for the analysed conditions. The paper however fails to identify this aspect, which is quite important in practical applications, as it appears that there is little advantage in using grout with higher compressive strength if the failure is dictated by the steel-bonding strength in plain-pipe grouted connections.

A linear relationship for bond strength against shear connector spacing is presented, but the relationship is only based on three ratios of bond strength to spacing and for this reason it has little significance. A relationship for relative axial displacement between the pile and grout was also presented based on small-scale (1:4) samples. Results indicated that at upper bond strength (normal stress  $\sigma = f_{bu}$ ), the axial displacement is  $\delta_{um} = D_p/40$ , meaning that for typical offshore wind monopile foundations this would be equivalent to more than 100 mm axial displacement. For lower loads, the following relationships were proposed:  $\delta_{um}/10 = D_p/400$  for  $\sigma = f_{bu}/2$ ;  $\delta_{um}/50$  for  $\sigma = f_{bu}/6$ . Preliminary results of fatigue tests were also reported, with five shear-key samples tested under zero-mean stress and equal tension/compression cycles, which did not show signs of fatigue damage up to  $10^7$  cycles for normal stress up to 40% of the ultimate strength.

In parallel with the work conducted by Billington and his associates, another large experimental campaign was led by Karsan and Krah [7], aimed at deriving the design equations for the new API code of recommended practice for grouted connections and justifying them through a reliability analysis. This was based on 201 tests, reduced to 147 to ensure grout strength greater than 17 MPa; of which 62 were plain-pipe connections. They also compared this to the DOE's testing, which had 117 tests, consisting of 44 plain and 73 shear-key samples. The results of the comparison therefore have high statistical significance.

Similarly to Billington [4], they observed a considerably greater variance in the factor of safety for the plain-pipe connection tests than for shear-key connections (Fig. 8), again demonstrating that the codes at the time did not correctly account for factors which influenced the capacity of plain-pipe grouted connections.

Interestingly, the authors stated that the effects of loading conditions other than axial, such as bending, transverse shear or torque, may be important and such loads, if significant, should be considered in design by appropriate analytical or testing procedures due to lack of published data, but no work was carried out by Krah and Karsan to quantify their significance.

Further results by Krah and Karsan are presented in Ref. [1]. As in the previous papers, there is a considerable scatter of data for plain-pipe connections, and it can be seen in Fig. 9 that for values of the ultimate grout strength  $f_{cu} > 50$  MPa the ultimate bond strength  $f_{bu}$  does not increase further. This is in contradiction to Billington [4,5], who had proposed  $f_{bu} \propto f_{cu}^{0.5}$ , with higher limits of equation validity on grout strength than 50 MPa. As stated in both papers by Krah and Karsan [1,7] it highlights the importance of not using design equations beyond the limits of the experimentation that they were derived from, as the data trends may not apply beyond these limits.

Like Billington [4], the authors discuss the failure modes of the grout being a combination of grout crushing and slippage at the pile-grout interface, with diagonal cracks between shear-keys, but they developed it further analytically by considering the equilibrium of a free body diagram for a piece of grout between two consecutive cracks, as shown within Fig. 10. They also proposed other possible failure modes if shearkey spacing and heights are sufficient, including pure shear of the grout. However, none of the experimental testing undertaken was

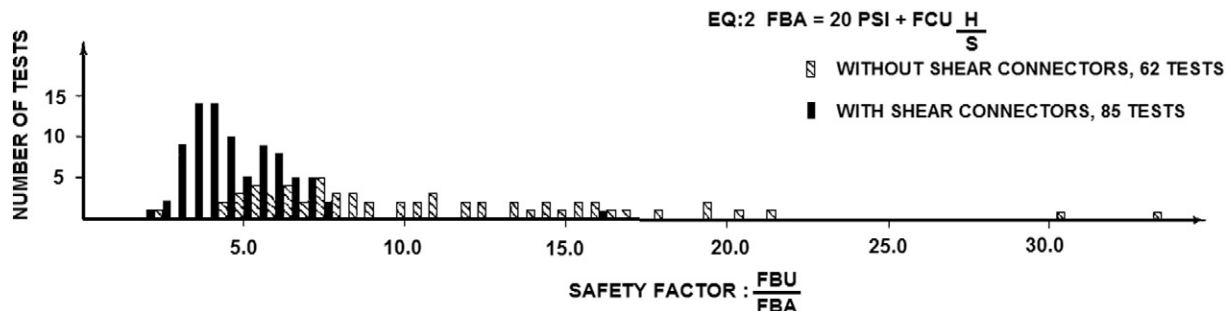


Fig. 8. Histogram of safety factors for EQ:2 compared with test data [7].

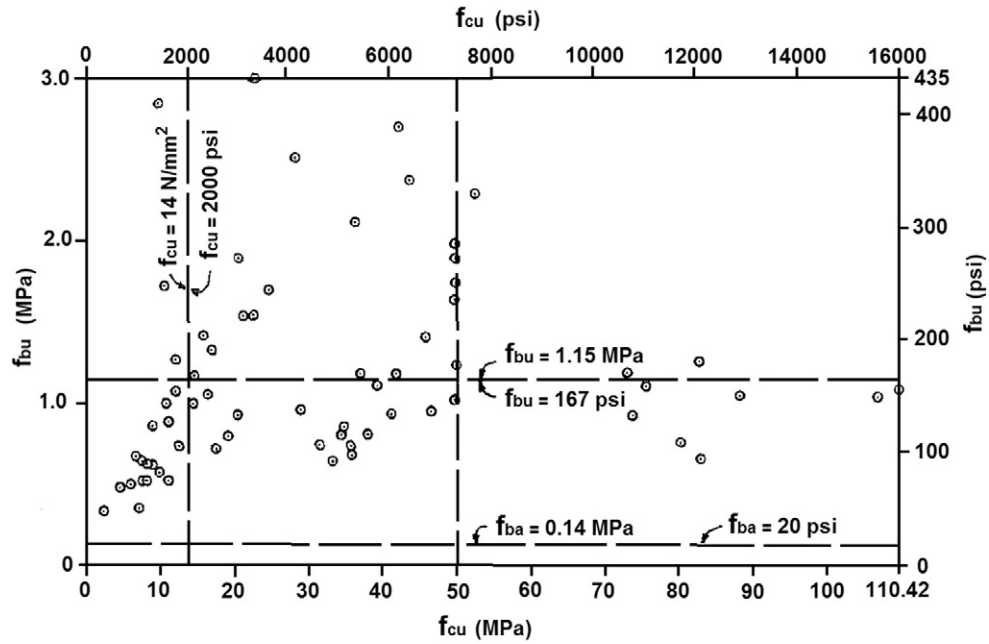


Fig. 9. Measured bond strength versus grout compressive strength for tests of grouted tubular connections without shear-keys [1].

in the region that these failure modes would be expected and so the limits of occurrence were not validated.

Tebbett and Billington [8] and Tebbett [9] reviewed some previous work undertaken at Wimpey laboratories and carried out additional testing to extend the range of validity of the DOE equations. It was noted that when high sleeve stiffness ( $(D/t)_s < 40$ ) is combined with

high pile stiffness ( $(D/t)_p < 32$ ), then the DOE design code overestimates the connection strength. Early-age cyclic loading during curing was also found to lead to larger displacements at ultimate load, roughly 200% greater than samples cured under static load (see Table 1), but the ultimate load capacity remains unchanged. This data provided substance to ideas earlier touched upon by Billington [4,5], with

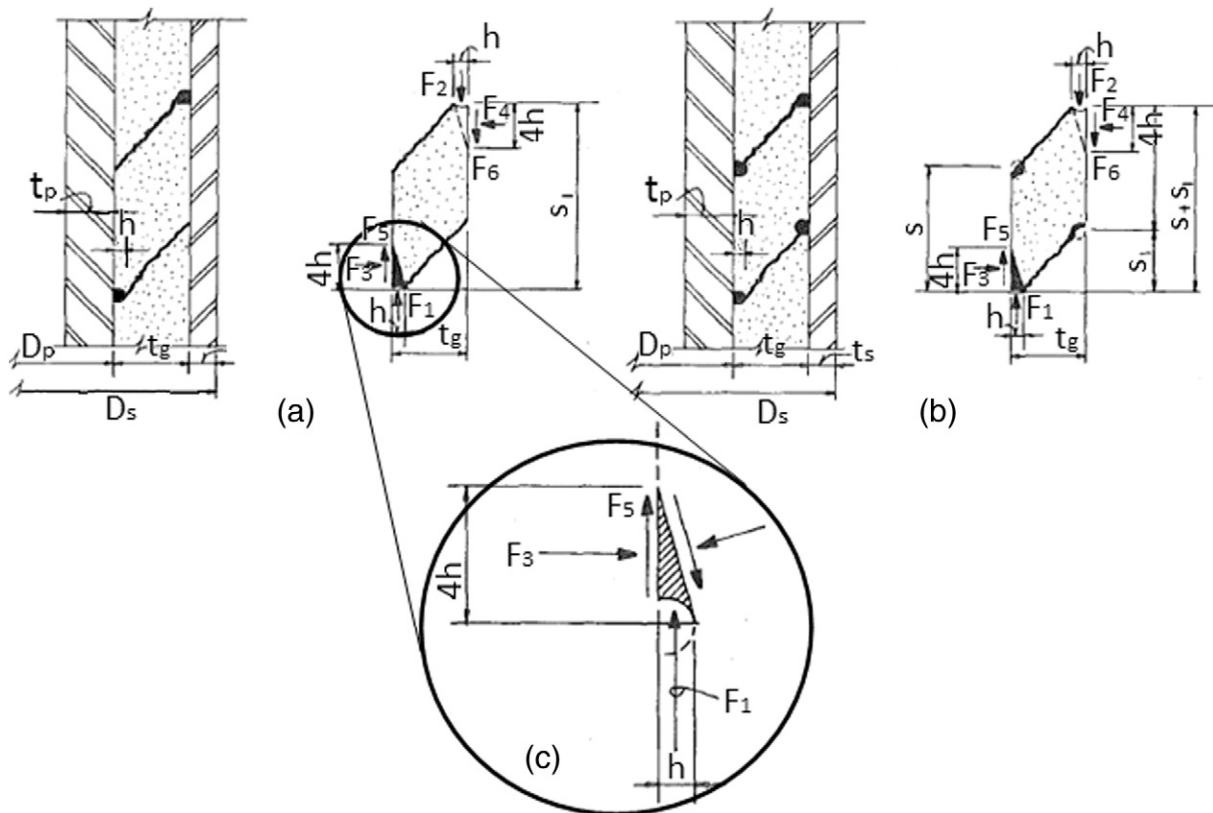


Fig. 10. Free body diagram of cracked annular grout (full ring with cross section shown) a) larger  $s/t_s$ , b) smaller  $s/t_s$ , c) crushing forces in grout at shear-key [1].

**Table 1**  
Comparison of typical displacements at ultimate load.

Specimen Type	Displacement at Ultimate load	
	Static Curing (mm)	Cyclic curing (mm)
OB with shear connectors	8	18
OB plain-pipe	18	45
HAC with shear connectors	8	15
HAC plain-pipe	18	30

reasonable significance as these conclusions were based on eight samples.

Unlike Billington [4], Tebbett [9] stated that specimen size may influence test results and so recommended full-scale tests should be performed.

For fatigue loading in which the applied stress was less than 40% of the ultimate bond strength, no failures occurred at less than  $10^7$  cycles for shear-key connections, which is in agreement with the previous findings of Billington [4,5] at the same test laboratories, but this is now substantiated by a greater number of experiments (ten samples). For higher loads, failure is due to degradation of the grout, through fatigue and void formation around the shear-keys. The cyclic movement induced significant reduction in stiffness at both working and ultimate load, but only very close to the end of life. For plain-pipe connections subjected to cyclic axial testing, it was found that they are less susceptible to fatigue than connections with shear-keys, but only one sample was tested and so no S-N relationship was presented and the significance of the results is limited.

It was also demonstrated that API's constant strength approach for plain-pipe connections overestimates the actual strength by an average of 12%, and the extrapolation of relationships from limited data sets was considered the main reason for such inaccuracies, again highlighting the importance of design limits.

Lamport, Jirsa and Yura [10] looked at determining the effects of relative shearkey location, grout strength, pile to sleeve eccentricity and combined axial and proportionally applied moment loading on the resultant axial capacity of the connections. Testing was reasonably statistically significant, with 18 samples tested, but typically only two values of each variable were investigated, and so no trend could be determined. Overall, they found no noticeable effects of grout thickness or relative position of shear-keys on the overall capacity of the connection. This is in disagreement with the findings of Forsyth and Tebbett [11], which appear to demonstrate that grout thickness is an important contributory factor, as in a thicker grout annulus, grout compression struts are orientated closer to the radial direction and therefore shearing of the grout is more likely to happen. They also considered that the optimum value for the spacing of the shear-keys will depend on their geometry, as well as on the radial stiffness of pile, sleeve and grout. Previous tests for a constant height-to-spacing ratio (h/s) showed large scatter, suggesting that small geometric variations can have an effect. Their results were reasonably significant with four h/s ratios, repeated at least three times using the same connection specifications. This provided an indication of the optimum height to spacing ratio of approximately 0.075, which being outside the limits of previous work [1,6,8,9] (h/s = 0.04), therefore demonstrating the benefit of increasing such ratio in order to improve the efficiency of the connection.

Smith and Tebbett [12] presented findings of testing related to remediation works for what was the largest gas production platform in the world, North Rankin A, off the Northwest coast of Australia. Testing was required as the pile geometries were outside limits of existing design equations. These works included the use of grout plugs to improve the pile end bearing capacity and pile sleeve connections to transfer load from the piles to under-reamed pile bells via a tubular insert. As part of this research, they investigated scale effects with 0.25, 0.3 and 1.0 scale samples for grout plugs and the applicability of

using conventional grouted pile-sleeve connections design codes. This validated the previous hypothesis of Tebbett [8,9] that noticeable scale effects affect the experimental results, with a reduction factor of 0.80 between full and quarter scale. The investigation of varying the h/s ratio revealed the different failure modes, previously hypothesised by Krahel and Karsan [1], with shear failure of the grout across the tips of the shear-keys, resulting in 35% of the capacity predicted by design guidelines available at that time, which assumed shear from pile to sleeve shear-key tips. This is in agreement with the suggestion of Forsyth and Tebbett [11] that there is an optimum value for the ratio h/s. Results from the testing were reasonably significant, with four different shear-key height-to-spacing ratios being investigated, but with insufficient repetition to confirm any trend. As in most previous reported works, it was demonstrated that extrapolation beyond design code limits can lead to a reduction in terms of safety factors.

Sele and Kjeøy [13] presented the background to design equations for the draft rules of fixed offshore structures developed by DNV (Det Norske Veritas). Friction tests were carried out on grout, based on oil well cement, under varying normal loads, which exhibited a friction coefficient  $\mu = 0.7$  and cohesion strength of 0.1 MPa.

Key findings of axial capacity tests were that a small gap had formed between the grout and steel due to shrinkage, even with so called non-shrink grouts. For 30–50 mm of grout, shrinkage was in the order of 0.01 mm. These are significantly smaller than those reported by Billington [4] for normal grouts. Failure mode for shear-key connections were described and in agreement with previous papers [1,4,11,12].

Testing showed pronounced slip then stick action, Fig. 11, i.e. large displacement under a constant or reducing load followed by small displacement for an even higher load. This suggests that significant displacement must occur in order to mobilise the capacity of the connection, which agrees with the findings of Billington and Tebbett [6,9]. At ultimate axial load, shear is essentially mobilised from interlocking due to surface imperfections which induces normal stresses and therefore friction, with little or no effect from cohesion or adhesion.

This work also demonstrated the importance of surface irregularities' magnitude, with machined surfaces having a significantly reduced ultimate capacity and not showing any radical change in the coefficient of friction (dynamic to static), which was further elaborated in a later

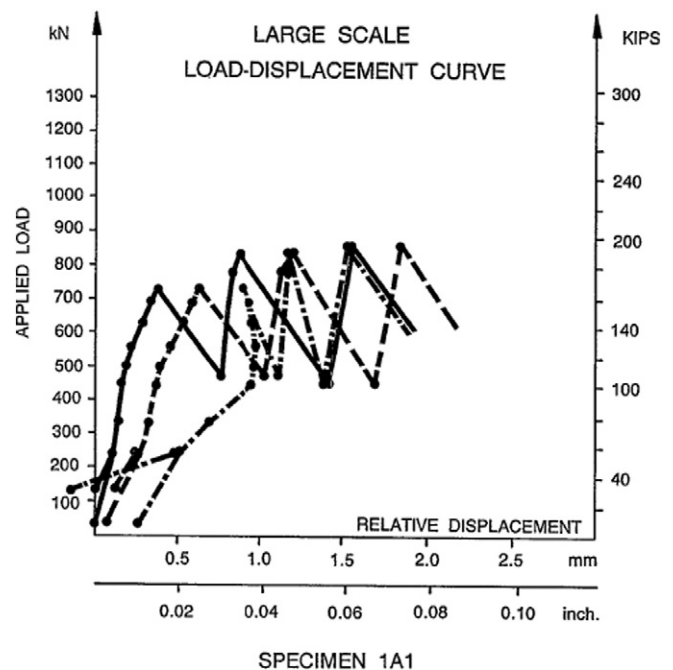


Fig. 11. Load deformation recording from a pile sleeve test [13].

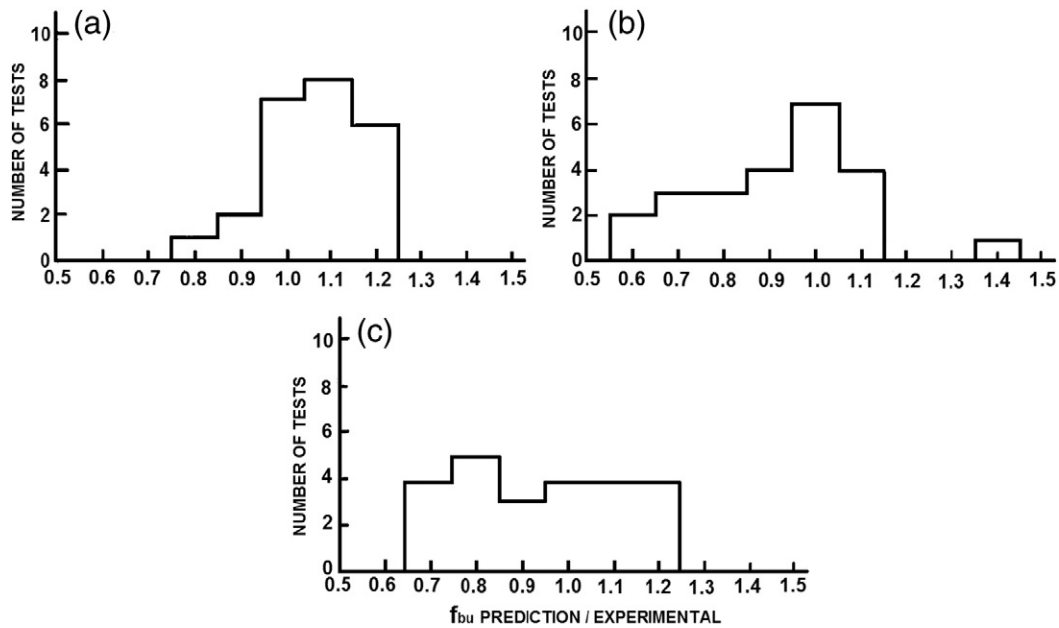


Fig. 12. Bond strength prediction a) proposed numerical methods b) DOE Formula c) API Formula [16].

work by the same authors [14]. However, because this test comprised only one sample, it had limited statistical significance.

Aritenang et al. [15] focussed their research into load transfer and failure mechanisms involved within shear-key connections, with the aim of deriving a numerical model that was to be calibrated against the results of a limited test programme; the derivation and validation of which are presented in [16]. A key point raised in this testing methodology was to ensure the boundary effects of the loading rig are minimised, with the load applied at one diameter from the connection end, as any additional confinement in close proximity to the connection will fictitiously increase its strength (a well-known phenomenon, for instance, when cubic and cylindrical concrete samples are tested). This was not mentioned in any of the other previous works where axial tests were undertaken, but are reported by the author as being a feature of the DOE testing. Only six samples were tested, investigating two weld bead heights and three levels of confinement of the pile, so limited

significance can be drawn from these results. Detailed structural monitoring was also undertaken for the first time on an axially tested grouted connection, through installed strain gauges close to the weld beads in both the axial and hoop direction. This investigation, along with the post failure inspection of the sample, indicates that confinement is a key parameter to the ultimate strength, and the observed failure mechanism with 45° cracks between shear-keys showed good agreement with the findings of other researchers [1,4,11–13]. However, for the first time they proposed a mechanism in which these cracks initiate at the centre of the grout annulus and then propagate towards the shear-key upon loading beyond ultimate failure, but shows good agreement with the later strut and tie model presented by Löhning and Muurholm [17]. The results of the influence of increased shear-key height on connection strength were also less than the DOE predictions, for which shear-key height was directly proportional to connection strength. This was hypothesised as being a result of increased local

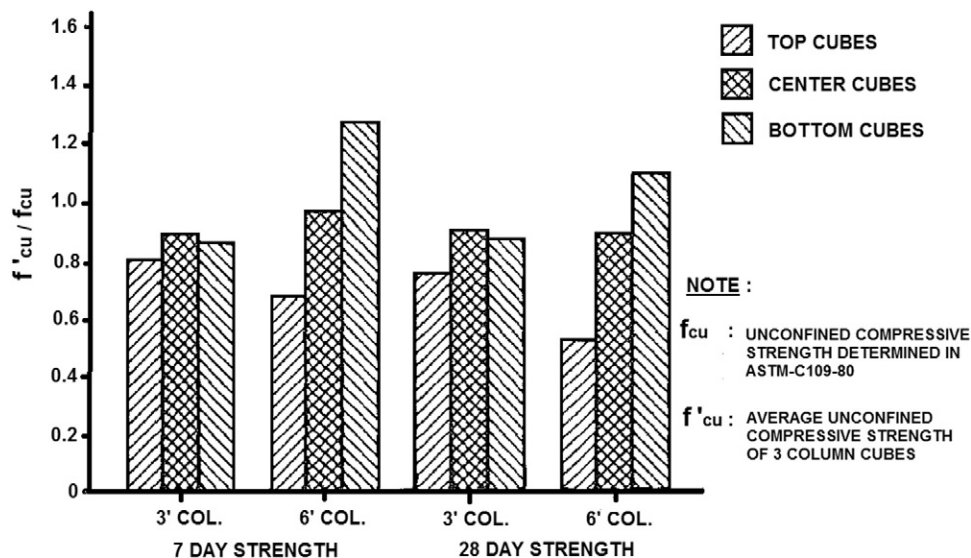


Fig. 13. Histogram of injected grout column cubes versus ASTM C190 Cubes for 27–41 MPa grout [18].

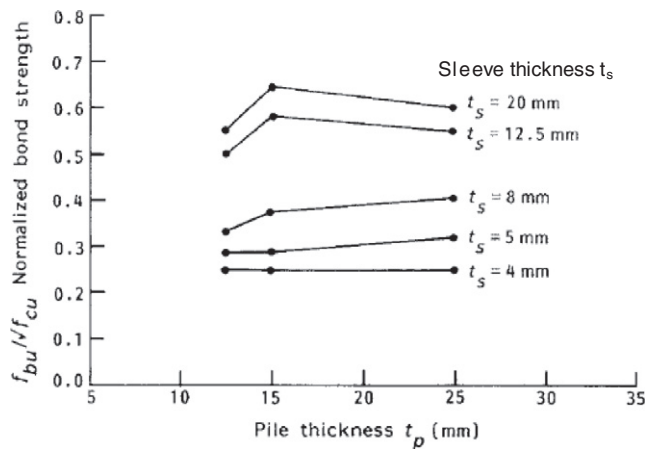


Fig. 14. Effect of pile and sleeve thickness, adapted from [20].

plate bending due to a larger shear-key height, reducing the effective stiffness of the pile.

Elnashai and Aritenang [16] used previous experimental testing [15] for validating a new numerical model. Comparison with the results of six samples, with three different pile thicknesses and two shear-key heights, showed a good agreement, with the greatest discrepancy being 18%. As limited samples for the number of investigated parameters were tested, more validation with experimental testing would be required before results from the model can be considered significant. The comparison of predicted results with the experimentation undertaken in Aritenang et al. [15] highlights the accuracy of the new numerical model over the API and DOE formulae through a less dispersed distribution of predictions, as shown within Fig. 12. In addition, Elnashai and Aritenang [16] concluded that the exclusion of a radial stiffness parameter from the API code could not be justified given the inconsistent results from the API formula.

Lamport, Jirsa and Yura [18] looked at reducing the safety factors of future design in comparison to the high safety factors used by the API, DNV and DOE design codes. They extended the work presented in [10], considering the influence of factors that had not been previously tested in depth, such as the effects of moment loading, relative pile and sleeve shear-key position and pile sleeve eccentricity. Testing was

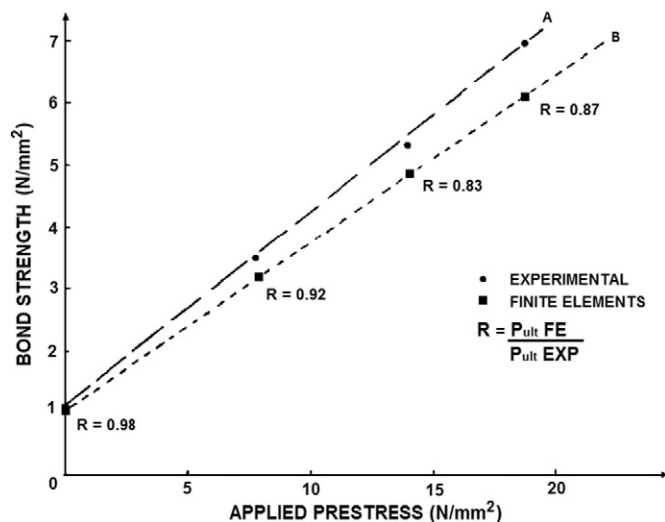
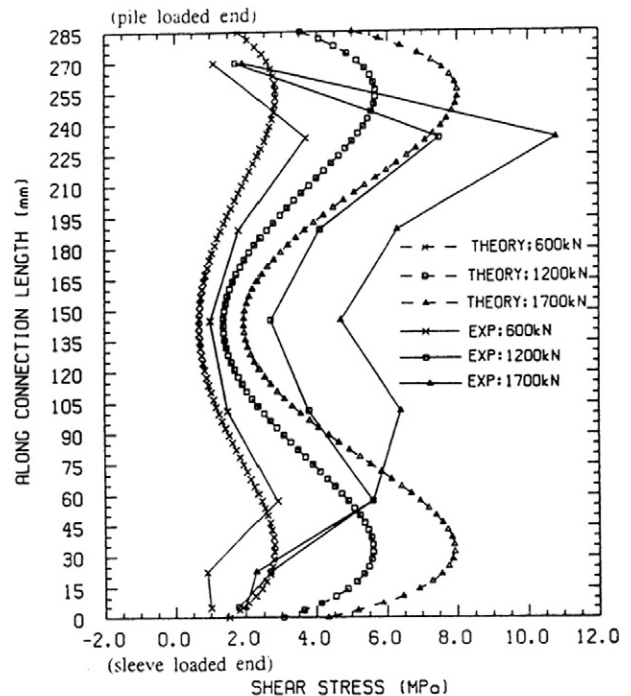


Fig. 15. Comparison of the effect of applied pressure on bond strength; experimental and FEM [22].

also undertaken to validate a proposed analytical model and to investigate the effect of these parameters, but had limited statistical significance, as there were only three variations for each parameter investigated. Unlike other testing methodologies reported in the reviewed literature to date, the tested samples were manufactured using the same procedure as in the actual offshore application, with the grout injected from the bottom of the connection, displacing water as it filled the annulus until a change in colour of the grout is noted in the overflow. Cube strengths were taken at the top, centre and bottom of both a 0.9 m (3 ft) and 1.8 m (6 ft) column, with only the lower part of the 1.8 m column having a similar strength to that of the unconfined cube strength of samples prepared to ASTM C109 [19]. For the tall column, the top sample showed only 50% of the reference cube strength; more generally, the strength was seen to decrease with the height, as shown in Fig. 13. With three samples taken at each height, the results have minimum statistical significance. This finding is particularly relevant to the offshore wind energy industry, as typically grout used for such applications has to travel large heights within the connection and pumping is stopped as soon as overtopping is seen. It follows that a strong possibility exists of significant variation of grout properties over the height of the connection, particularly in the top two metres. In the oil and gas industry much attention has been paid to ensuring the required quality of grout completely fills the connection, using density gauges and significant over-pumping.

The other key finding in Ref. [18] was that, based on the limited experiments, eccentricity of the pile sleeve connection or relative shear-key position had no noticeable effect on axial strength. Like Refs. [1,5,12–15] post-failure investigation revealed grout failure cracks between 20° and 60°. The use of the grout compression strut was then used to derive an analytical model, similar to Ref. [1]. Comparison with predicted results from DNV, API and DOE also highlighted that the DOE showed the lowest variation with respect to the measured values of strength.

Finally, similarly to the work of Forsyth and Tebbett [11], the analytical model by Lamport, Jirsa and Yura [18] suggested an optimum h/s



LM4:  $L/D_p = 1.0$  (shot blasted surface)

Fig. 16. Interface shear – analysis versus test [25].

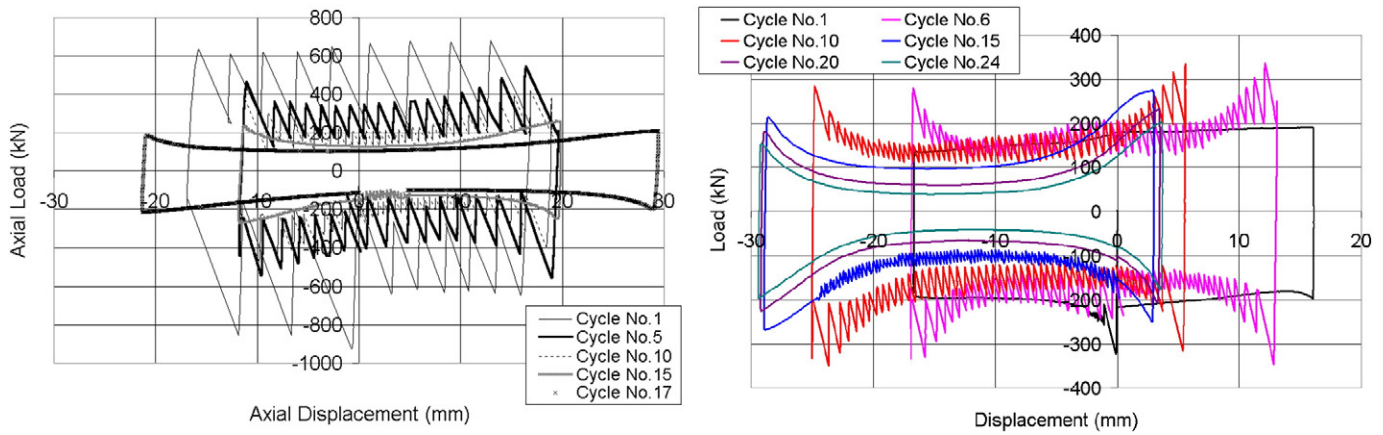


Fig. 17. Load-displacement of low pre-stress (left) and high pre-stress (right) [29].

value of about 0.075. Although not validated experimentally, this was explained by the change in the failure mode of the grout from compression struts to pure cylindrical shear failure plane, as hypothesised by Krahl and Karsan [1] as well as by Forsyth and Tebbett [11].

Aritenang, Elnashai and Dowling [20] built on their previous work by investigating a larger range of parameters beyond those previously covered and extending it to a finite element (FE) model for welded shear-keys. The FE model outputs showed good agreement with the derived analytical model. When compared to previous experimental testing, there was reasonable agreement for a variety of parameters. Like Krahl and Karsan [1], they suggested a grout strength limit above which bond strength does not increase; however this was seen to be 35 MPa rather than 50 MPa (i.e. 30% less). The experiments reported in Ref. [19] also investigated the effects of sleeve thickness on the bond strength in grouted connections. As shown in Fig. 14, the tests conducted on five different sleeve thicknesses (from 4 to 20 mm) and three values of pile thickness (from 12.5 to 25 mm) revealed that sleeve thickness has a noticeable effect.

Sele and Skjoldde [14] used data from 750 tests to assess the predictiveness of available design equations and identify new trends. They concluded that the DNV equation provided more robust predictions than the DOE and API equations when compared to the test data set, which contradicts the finding of Ref. [18], where the DOE were shown to be more accurate, having the lowest variation. However the statistical significance of Ref. [18] is lower as only 16 samples were used, while 258 formed the basis in Sele and Skjoldde's comparison.

It was found that a “wedging” action caused by the uneven surface of rolled steel generates hoop stresses in the pile and sleeve and is the main source of axial strength for plain-pipe connections. This was demonstrated by tests performed by DNV using pipes, which have been turned down in a lathe to produce a uniform surface, showing a radical reduction in axial load carrying capacity. Strength of plain pile connections was therefore concluded to be dependent on the magnitude of the wall surface unevenness, as well as on the hoop stresses of the pile and sleeve.

Failure modes for shear-key connections were reported as either being grout compression struts with 45° failure planes between shear-keys or cylindrical shear failure of the grout at the tip of shear-keys, depending on the shear-key h/s ratio and grout strength, which is in agreement with [1,4,12,13,15,18].

A detailed investigation was also made into the nominal dimensions quoted for tube thickness and this showed variations of 5–15%, especially for smaller samples. Interestingly, this was rarely measured in previous testing, and could then account for some of the scatter in the data, as the confinement is one of the key parameters for the connection axial strength.

Harwood et al. [21] provided the background to the formation of the ISO standard. As this consisted of a review of 30 testing programmes with 626 individual tests screened to 193 results for axial capacity of grouted connections based on well-defined criteria, there is a good significance to their findings. A new interface transfer strength term was proposed to replace the traditionally adopted bond strength in order

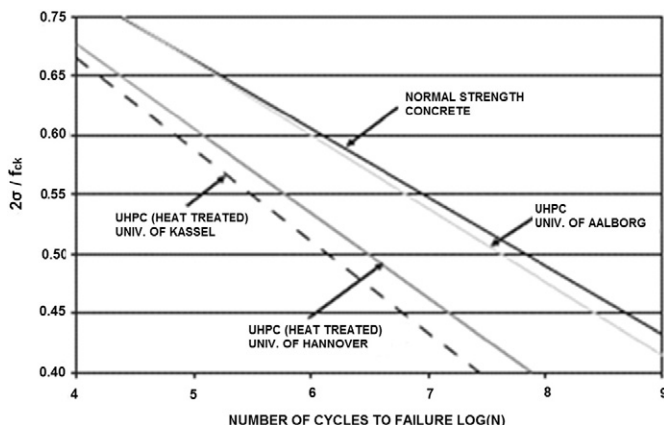


Fig. 18. Comparison of different fatigue tests on UHPC mixes [30].

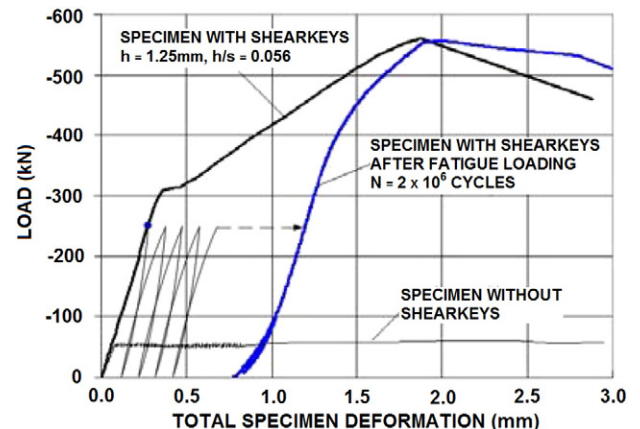


Fig. 19. Load-displacement curves for different small scale specimen [32].

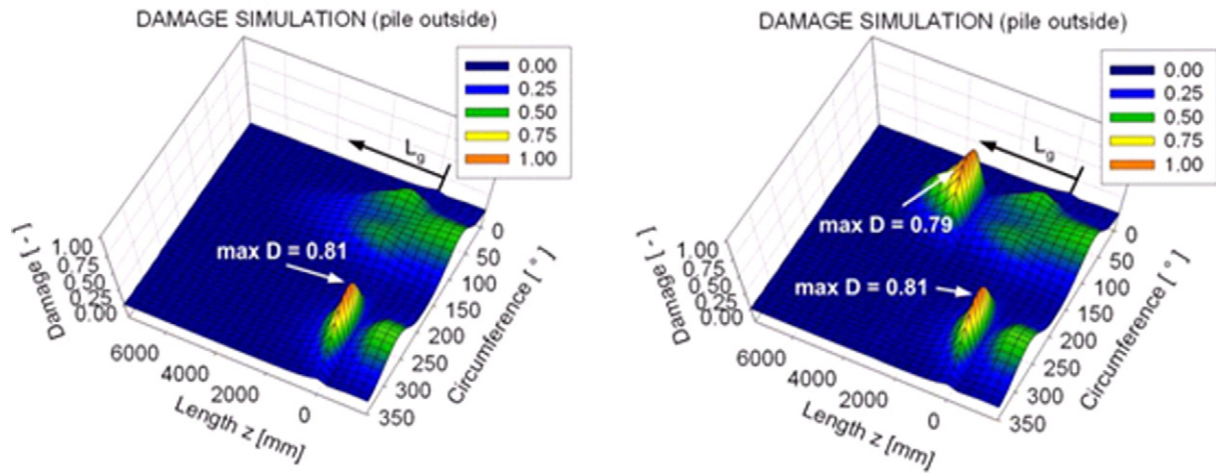


Fig. 20. Fatigue damage comparison between shear-keys concentrated to the middle third of the connection (left) and evenly distributed along the whole length (right) [32].

to highlight that little adhesion (bond) is experienced in practice. A statistical review of five design formulae highlighted that the predicted strength from a modified HSE design approach showed the best agreement with the experimental measurements. This therefore formed the basis for the ISO formulation, with exclusion of some parameters that showed little significance, such as load and length parameters, and modification of others such as radial stiffness, shear-key density and grout strength. The design formulae covered the two failure modes shown by Smith and Tebbet [12] of grout compression struts or cylindrical shearing of the grout matrix occurring at higher  $h/s$  values. The validity ranges based on the limits of the screened data and over which the formulae were shown to be accurate are clearly stated. Like Lampert et al. [18], Harwood et al. [21] note the variability in grout strength over the length of the connection and propose the use of an effective length to take account this. An  $h/s$  of 0.05–0.07 is presented for optimum connection capacity, which is lower than the 0.075 factor suggested by the authors of Refs. [11] and [18]. A detailed look at the influence of early age cyclic movement during curing on axial capacity showed agreement with the authors of Refs. [8] and [9] in terms of marginal influence on static strength at  $h/s = 0.012$ . However, for  $h/s > 0.05$  adverse effects on static strength were observed and so a factor to account for this is

included in the design equation, with clear guidance on the radial displacement magnitudes used in its derivation ( $\pm 0.35\%$  of  $D_p$ ).

### 3. Pre-stress

Dowling, Elnashai and Carroll [22] provided a good review of the previous research undertaken on grouted pile-sleeve connections, which highlighted that there had been no successful attempt to analytically model the connection. The experimentation showed the importance of confinement of the grout: that, by simply applying a radial pre-stress, the bond strength for a plain connection was increased by about six times over the unstressed connection, as the pre-stress overcomes the loss of radial confinement due to curing shrinkage (Fig. 15). The proposed analytical model showed a reasonable agreement with experiments, but it was only applicable to a single value of material strength for grout and steel. Although four different levels of pre-stress were used to define the relationship, no attempts were made to demonstrate repeatability. Similarly to the authors of Refs. [1,4–9, 11–16,20], the experimentation does not consider the confinement provided by the test rig, as the loading is directly applied at the end of the connection for the push-out test performed.

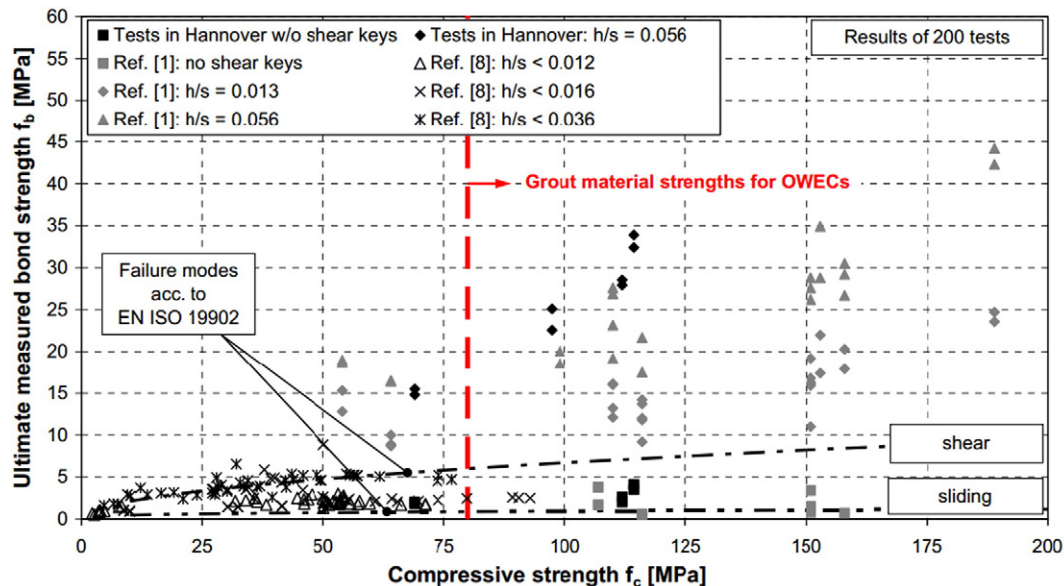


Fig. 21. Test results from static axial force capacity tests, curves for shear and sliding failure acc. to EN ISO 19902 (2007) ( $C_p = 1$ ,  $K = 0.015$ ) [34].

The report also touches upon tensile loads resulting in a reduction of bond strength compared to compressive loads, which is in agreement with Billington [4].

Elnashai, Carroll and Dowling [23] further investigated the effects of mechanical pre-stress on the capacity of grouted connections. They reported a minimum of six times improvement by using pre-stress in the ultimate capacity of plain connections for the same dimensions, which was based on seven tests and so with good statistical significance. They were the first to report reduction in capacity upon reload of around 23% for expansive grout tests. They also reported decreasing bond strength with increasing length-to-diameter  $L/D$  ratio of connection for both plain and pre-stressed connections, as the entire slip length  $L$  is not contributory to strength. This is in agreement with non-pre-stressed connection findings presented in Refs. [4–6,20].

Elnashai et al. [24] further studied the use of pre-stress, both mechanically and chemically (expansive agent in grout), investigating the effects of confinement and length of connection through two different  $D/t$  (diameter-to-thickness) and  $L/D$  (length-to-diameter) ratios. Similar strength improvements were reported for both techniques, but are limited by the radial stiffness of the connection. The difference in axial strength of the two samples of different radial stiffness was explained by Elnashai et al. [24] as being due to the areas near the end of the connection showing less separation between grout and steel, due to the Poisson dilation effect being less for higher radial stiffness samples, resulting in lower loss of pre-stress. They also developed a FE model that showed a good agreement with the experimental testing, their prediction always being within 8% of the measured average bond strength for the six samples tested. However, only two samples were tested with the expansive grout, and so there is limited significance for this type of pre-stress.

Gundy and Kiu [25] also investigated the enhanced axial capacity that can be achieved through pre-stress. This was done both experimentally and analytically, based on shell bending theory and FE modelling. Their findings were in agreement with Elnashai et al. [20,22] for plain connections, but also showed a 50% improvement of bond strength by pre-stress on shear-key connections. The results of this research were reasonably significant with limited scatter for the eight tests undertaken. They also studied the mechanics of connection strength, which indicated that capacity is mobilised over a finite length at the end of the connection until first slip occurs, with peak shear occurring at  $0.1D_p$  (35 mm) from the end of the connection and negligible by  $0.56D_p$  (200 mm) (Fig. 16). This was used to explain the effective reduction in bond strength with increasing  $L/D$  ratios. The analytical approach based on classical shell theory was very closely matched to the FE results, but as can be seen from Fig. 16, it only produced similar trends to the experimental findings of shear distribution, not magnitudes.

#### 4. Dynamic axial loading

Boswell and D'Mello [26] investigated clamp fatigue performance based on experimental testing of ten samples with shear-keys, using 0.1 and 0.5 Hz as loading frequencies, which are typical of wave action in the North Sea. The key findings were that higher strength grout showed a relatively poorer fatigue performance, with a fatigue limit at around 20.7% of ultimate strength for the higher strength grout, compared to 32.5% for the lower strength. This was around half the critical value predicted by the work reported in Refs. [4,6,8,9], but this could be down to variation in connection type, shear-key configuration, etcetera, as the influence of these additional parameters was not investigated. The results themselves have reasonable significance with nine samples tested.

Ingebrigtsen, Løset and Neilsen [27] investigated static and fatigue design of grouted pile sleeve connections. It consisted of over 150 tests, of both previous (Ref. [13]) and new test data, for 1:3 and 1:5 scale, with 68 plain-pipe samples and 96 shear-key. They highlighted the different definition of the ultimate load capacity within DNV (as

the load at first slip) and DOE (as peak load, independently of the corresponding displacement).

It was found that plain-pipe connections are highly resistant to dynamic loading, while connections with shear-keys appeared to be less resistant compared to their respective higher static strength for tensile-compressive loading. However, only one plain-pipe connection was tested with reversed stress cyclic loading and so very little significance can be drawn from this. For axial compression-compression cyclic loading the same results were concluded and these were significant given at least nine samples for each connection type were tested. It was stated by the authors that fatigue is not an issue if tensile stress is less than 20% of the static stress for shear-key connections, which is in agreement with the findings of Boswell [26]. The fatigue design equations presented were based on 30 samples tested as part of this and previous research and so are statistically significant.

Harwood et al. [21] reported initial onset of fatigue damage being evident by small relative movements between the pile and grout which increased with increasing number of cycles. They reported a large reduction in fatigue performance of connections that had been subjected to early age cyclic movements and that there was evidence that fatigue performance reduced with increasing  $h/s$ . Similar to authors of Refs. [27] and [28], a mean fatigue endurance limit of 20% was indicated by the screened data for shear-key connections. It was also shown that the magnitudes of the reverse cycle loading previously used for axial fatigue testing were onerous compared to the actual service loading.

Etterdal et al. [28] investigated the use of high-strength grout for strengthening of offshore steel components using the commercially available Densit Ducorit® grout S5 and D4, as part of the research required for the strengthening of 75 braces/chords on Norwegian Ekofisk oil and gas platform jackets due to sea bed subsidence. Although not directly applicable to previous grouted connection testing, this experimental work highlighted the effect of the load history, with grout not very effective on first load-up, but marked improvement in capacity on repeat load cycles in the same direction. However, if the load is reversed, efficiency is lost until the loading is repeated. This was in agreement with the findings of Sele and Kjeøy [13].

In terms of the relative displacement between grout and steel, Zhao, Grundy and Lee [29] investigated grouted sleeve connections under large-deformation cyclic axial loading, for applications in earthquake engineering. As a result, displacements were in the order of ten times those typically seen in monopile connections for offshore wind energy installations. They reported an increased peak load capacity with increasing cycle numbers for lower pre-stress levels, which was explained by thermal expansion of the inner pile increasing the pre-stress. However, this was counteracted by decreased capacity in the coefficient of friction due to powdering of the grout at the grout steel interface, along with degradation of the grout through cracking and spalling with increasing cycles. This was the dominant factor for high pre-stress samples where a reduction of capacity was seen from the outset. Fig. 17 shows both the low and high pre-stress results. These rates of capacity reduction were found to be dependent on the magnitude of the axial displacement, with greater displacements, i.e. distance walked, showing quicker reductions in capacity. No investigation on the implication of surface finish was made. Only eight samples covering three different variables were used to derive the influence of each factor and so limited significance from the results can be drawn. It does however provide some useful insight and highlight some areas worth of further investigations, which have not been previously mentioned by other authors.

Although the WTG grouted connections do not utilise pre-stress and the magnitudes of the relative displacements are considerably smaller, the normal compressive stress between the grout and the steel required for wear and grout powdering is provided by the large overturning moment experienced by the connection. The high number of such load cycles means that the findings of Zhao et al. [31] that the axial

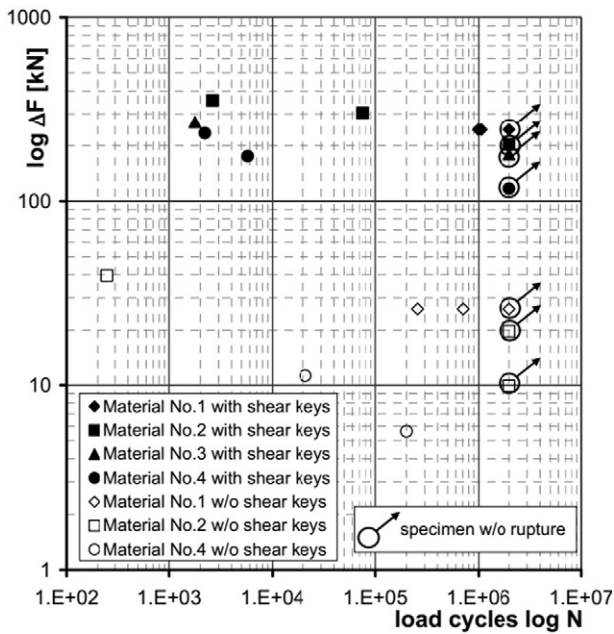


Fig. 22. S-N curves for small scaled specimens [34].

capacity tends to reduce with the number of cycles and the powdering of grout occurs are applicable to the overall lifetime of offshore WTG with plain-pipe grouted connections. This wear could not only result in exacerbation of the insufficient axial capacity already experienced, as surface irregularities become worn and the degree of interlocking between the grout and steel interface reduces, but material loss can also produce significant dynamic loads.

Lohaus and Anders [30] presented a comparison of high cycle fatigue response of different ultra-high performance concretes (135–225 MPa), with static and fatigue testing performed on scaled shear-key connections. It was reported that the fatigue strength seemed to be marginally lower for ultra-high performance concrete (UHPC) (Fig. 18), which is in agreement with the findings of Boswell and D'Mello [25].

They also hypothesised that a relative axial displacement of 2% of the connection length under ultimate axial load for a shear-key connection might have a detrimental effect on fatigue strength, which is similar to the findings reported within Refs. [8,9], where it is hypothesised that under large loads and displacements the formation of voids due to grout crushing would be detrimental to fatigue behaviour.

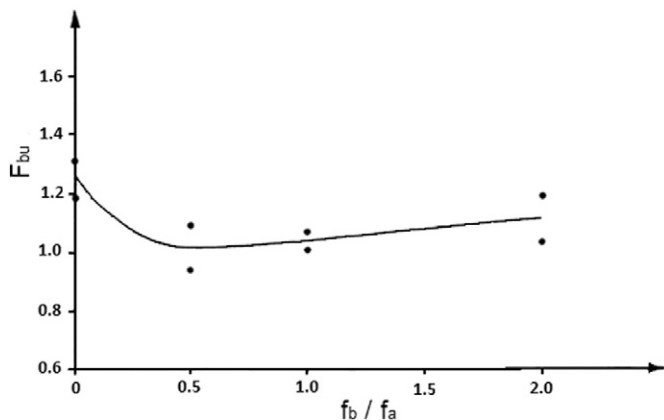


Fig. 23. Interaction curve for combined bending and tension specimens [5].

The effect of temperature on grouted connections was investigated by Zhao et al. [31], with the focus being on the fire engineering aspects of composite steel-concrete tubular connections. The paper demonstrated that rising sleeve temperature, and therefore increasing the temperature difference between sleeve and pile, causes a decrease in the axial strength, which validates part of the hypothesis of Zhao et al. [29]. The testing was fairly significant with a minimum of three samples tested per variable. The developed model predicts the response of the tests with reasonable accuracy, but the properties of the grout are not considered, as no attempt was made to record properties such as elastic modulus, Poisson's ratio or strength.

Schaumann and Wilke [32] presented findings of numerical and experimental modelling of grouted connections with and without shear-keys. Axial testing based on Densit Ducorit® S5 high-strength grout showed that the overall strength of the connection is considerably increased by the presence of shear-keys (Fig. 19), which is in agreement with all previous axial testing. The authors therefore recommend applying additional mechanical interlock even for monopiles with relatively low axial forces, given the significant gain in capacity shown by the addition of shear-keys.

Similar to previous work reported in Refs. [1,4,12–15,18], grout failure modes were investigated and showed that transverse cracking of the bottom compression strut occurs at 50% ultimate load, indicated by the kink in load response within Fig. 19 [32], with magnitude of around 1.5% the length of the grouted connection. The observed degradation in the capacity of the connection validates the theories of Tebbett and Billington [8], Tebbett [9] and Lohaus and Anders [30].

Schaumann and Wilke [39] also stated that capacities of tested samples are reduced if bending moments due to loading plates are excluded from numerical analysis, which is in agreement with the findings of Aritenang et al. [15]. Numerical analysis was shown to give good agreement with experimental results, but this was only for one sample. After two million cycles, specimens almost reach the load of static tests, as shown in Fig. 19, showing limited damage occurring at that load level for the number of significant load cycles expected in offshore environments; a result that is in agreement with early findings reported within Refs. [8,9]. Additionally, it should be noted that this study only considered compression-compression cycles, which explains why the performance is better than what was reported by Boswell and D'Mello [26]. It was also found that local deterioration of the grout around shear-keys, represented by a dimensionless damage parameter  $D$ , does not reduce the global capacity. They also demonstrated that the detrimental effect of shear-keys, in terms of pile steel fatigue for predominantly bending moment loaded connections, can be avoided if they are placed in the middle third of the grouted sleeve, illustrated by comparison of graphs in Fig. 20.

Anders and Lohaus [33] also investigated axial loading of grouted connections and the same aspect of the influence of the increase in compressive strength of grout and found similar failure planes to Refs. [1,4,12–15,18,32]. As well as this, the use of reinforcing fibres in the grout were considered and results showed an improvement in axial strength by about 25%. Details of the testing procedures were given in another paper, but a qualitative summary of fatigue results showed that there was significant scatter for the number of cycles survived for different grout strengths, but no significant difference in S-N curves for compression-compression loading, which is in agreement with Refs. [8,9,32]. The effects of confinement are mentioned as being of importance, but no specific investigation has been undertaken.

In 2010, after the unexpected settlement of large-diameter grouted connections for offshore WTGs had been reported, Schaumann, Bechtel and Lochte-Holtgreven [34] covered axially loaded testing in more detail, with a look at the results of the ULS (ultimate limit state) axial tests that were used to derive the International Organisation for Standardisation (ISO) design codes in comparison to their own tests and recent work with the same conditions, but higher compressive strengths of grout (Fig. 21).

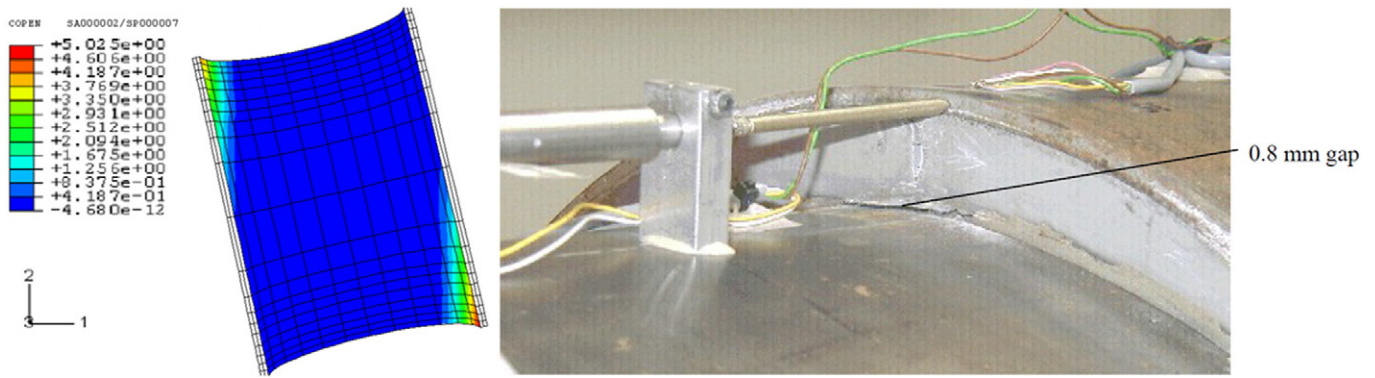


Fig. 24. Gap between pile and grout at load equivalent to extreme Load [38].

Limitations in the design codes are stated as mainly being the use of grout strengths greater than those for which the codes were validated for. With reference to previous work by the authors [36] showing that the use of linear damage accumulation for fatigue life was not appropriate and present the possible use of an energy approach, referenced to seminal works. There were only three samples for each grout strength, Fig. 22, and so the conclusions drawn by the authors of stronger grout providing better fatigue performance are of limited significance without further work. The findings were in contradiction to those of Refs. [8,9,26,30], who reported reduced or negligible change to fatigue performance for high-strength grouts.

Schaumann et al. [37] presented an overview of the unexpected settlement of the grouted connection and investigated possible solutions, such as the use of reinforcement fibres and remedial solutions. The experimental investigation on the use of shear-keys indicated an increase in axial strength of approximately six times that of a plain connection, which is similar to the findings of previous axial testing. The effect of the compressive strength of grout was also investigated and found to increase the axial strength of the connection. In both these tests there were only two samples, and so limited significance can be taken from these tests, although they agree with all previous historic test data. The test results on four samples did show an increase in strength with the fibre content, but the reduced slump was stated as making it impractical for offshore pumping, as void formation and blockages would be likely to occur and so their use was not recommended.

## 5. Bending/gapping

Billington [5] presented results of tests carried out by BP (British Petroleum), investigating the strength improvements of composite

steel grout tubes at annular joint connections, subject to both axial compressive-tensile and combined axial-bending loading conditions. The results indicate that if a bending load is applied to the brace, the axial capacity of the connection is reduced by up to 18% if the bending stress is less than the axial stress loading (Fig. 23). As only two samples were tested for each load condition and for only four different conditions, the overall significance is limited, but testing work was still being undertaken at the time of publishing.

These findings are in disagreement with those reported by Lamport [18], who showed no detrimental effects of combined axial and bending actions, but reported instead an increase of 14% in the capacity. However, details of the load combinations were not presented, making it difficult to assess the validity of such conclusions. Rotation was also reported as being pronounced, but there was no mention if this led to gapping between the grout and pile or sleeve faces.

Sele and Kjeøy [13] showed that the static capacity of a grouted clamp after a significant moment has been applied shows no change in axial capacity. However, this is not comparable with other experiments, as the loads were not simultaneously applied.

Andersen and Petersen [38] presented the findings of experimentation undertaken in early 2000's to document the performance of grouted connections used at Horns Rev, the first large-scale offshore wind farm (160 MW), 15 km off the coast of Denmark, along with development of a FE model to reduce the need for expensive laboratory experimentation. Testing consisted of a 1:8 scale connection, tested in both ULS and FLS (Fatigue Limit State). The loading regime was not however explicitly stated in this paper. Experimental gapping was reported under ULS conditions, equivalent to 6.4 mm at full-scale. The FE model showed reasonable agreement with the experimental testing, predicting a gap of 5 mm at ULS, as shown in Fig. 24, but with only one experimental setup and dimensions modelled, further work would be required for a more robust validation. The testing however

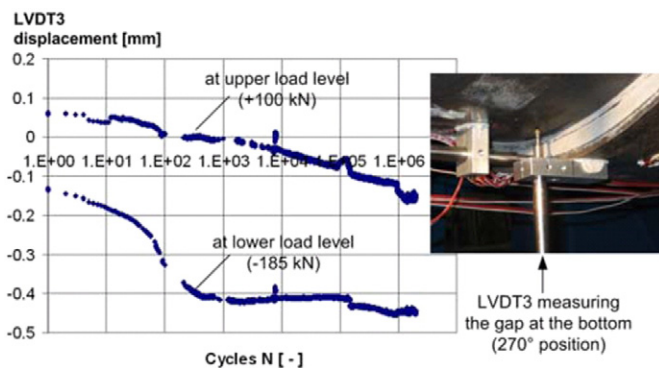


Fig. 25. Measured relative displacements between outside surfaces of pile and sleeve (negative values denote gap) at the 270° position [39].

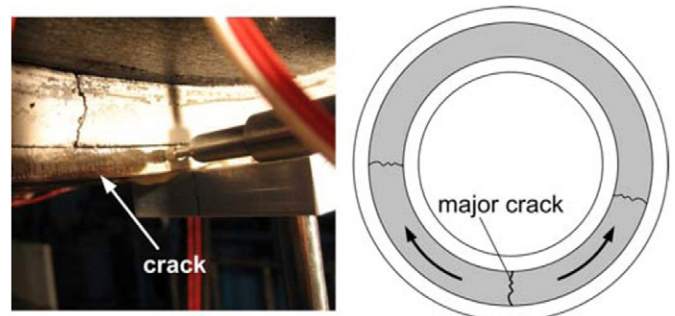


Fig. 26. Observed cracking [39].

only investigated the influence of pure bending, not combined axial and bending actions, as would be typically experienced in an offshore WTG foundation. Additionally, no account was made for the influence of surface finish, and the environmental conditions of the tested sample were significantly different from those typically expected in the operational life of the foundation (Fig. 24, right).

Schaumann and Wilke [39] presented findings of a 1:6.25 scale four-point bending test. As in Ref. [38], gapping was reported, but in this case it was noted that gapping also occurred under FLS, which increased in size with the number of cycles before eventually stabilising (Fig. 25).

Unlike previous tests, fracture of the grout under FLS was also reported, with radial cracks due to tensile hoop stress in the grout (Fig. 26). This resulted in a reduction of the bending stiffness, indicating the importance of grout properties, which is in agreement with previous findings, e.g. Refs. [3,9,13–15,32]. This could also lead to a reduction of the axial capacity of the connection. The presence of hairline radial cracks after curing is also reported, indicating that shrinkage has occurred, which in turn reduces the confinement and then the axial capacity (see Refs. [4,6,13,14,21–25]). However, the influence of the cracks on the overall structural bending behaviour is stated to be small (about 5%), with justification by FE analysis, as long as the grout remains able to transfer the lateral stress. As with other bending tests undertaken, apart from Lampert et al. [19] and Billington [5], there is no mention of the effect on combined axial and bending behaviour.

Interestingly, Fig. 27 shows the pre-2007 connection parameters for offshore WTG (grey bars) and the extent to which they are outside the limit range of validity for the NORSOK N-004, 2004, Rev 2 (purple) and Det Norske Veritas-DNV-OS-J101, 2004 (blue) codes for both compressive strength of the grout and slenderness of the pile.

The authors also noted that results of a number of research projects will have to be incorporated in the future design guide for grouted joints, especially test results in the tension-compression regime and the influence of test frequencies on the fatigue strength in order to better understand fatigue response of high-strength grouts.

There is a mention of the disadvantage of shear keys because of stress concentrations due to the joint geometry and the fatigue strength of the weld being reduced in comparison to that of the base metal of the pile and transition piece/sleeve walls, and the effect that surface irregularities have in transmitting the shear forces between the grout and steel, as mentioned in previous axial testing (e.g. Refs. [4,6,13,14]).

Schaumann, Wilke and Lochte-Holtgreven [36] investigated the influence of shear-keys on bending stiffness through the same experimental test set-up used by Schaumann and Wilke [39]. It was shown that global bending stiffness of the connection was increased up to 20% and the gapping between the steel and the grout reduced by over 50% by the inclusion of shear-keys. A FE model was also developed, which showed good agreement with only 7% error, but this was only validated for one geometrical configuration. A parametric model was developed to model the global dynamic behaviour of an Offshore WTG based on various grouted connection parameters including steel

diameters and thicknesses, coefficient of friction and connection length. This highlighted that the key parameter for the connections' dynamic behaviour is connection length, but no validation was reported.

The paper by Klose et al. [35] takes the form of a review of current research by developing FE models calibrated to large scale testing, so to be representative of the actual slenderness found in currently used transition piece dimensions. Tests were carried out for both ULS and FLS, but only considered single loading conditions, i.e. axial or bending, not the combination of both as would be found in operation. The loading cases undertaken do however represent a good example of current and future technology at the time, with consideration of application to 2.5 MW and 6 MW wind energy converters. Important points noted in the paper were the uncertainty associated with the high-strength grout properties, and therefore the higher material safety factors when compared to steel design. Current design formulae for grouted connections having only been validated for grout with compressive strengths up to 80 MPa, whereas current grouts in use can have strengths up to 210 MPa. The fatigue assessment for the grout being based on concepts originally developed for concrete structures, as there was no explicit S-N curve for high-strength grouts, which was particularly worrying as a slight change in the gradient of this curve can lead to fatigue calculations that vary by a factor of 100, or even more due to its logarithmic formulation [17]. Finally the fatigue formulae used for design are based on load cycle numbers which are only a fraction of the number seen in the 25-year design life span of the turbine foundations.

Lotsberg et al. [40] provided a general summary of the work that has been undertaken by DNV to investigate the capacity of grouted connections following the unexpected settlements reported in 2009. They are in agreement with previous research of Billington [5,6] and Lampert et al. [18], on the mechanism behind the capacity of plain connections referring to surface tolerance and roughness and radial stiffness, but state that a minimum surface tolerance should be included in fabrication standards to ensure these are mobilised. As minimum surface tolerances would be impractical, it is recommended that these are not considered in design, but kept in mind when assessing test results. The authors are also in agreement with Refs. [5,6,15,18] for the capacity of shear-key connections, stating that radial stiffness of the steel is important and load transfer between pile and transition piece is via formation of compression struts within the grout between the shear-keys. Upon failure of cylindrical grouted samples, similar failure planes to Refs. [1,4,12–15,18,32] were reported. Like Refs. [38–40], gapping was evident and the resulting ovalisation due to the reduced confinement of the higher D/t ratio leads to relative sliding between the grout and steel. Consideration has been given to scale effects, which was stated to overestimate the capacity of full-scale equivalents, showing agreement with the findings of Smith and Tebbett [12]. The scaling effect was minimised through using equivalent stiffness box tests for the scaled tests, which represent a segment of the connection at full scale and was validated through comparison of a small-scale 800 mm diameter connection and the equivalent stiffness box section. Good agreement in terms of relative displacement was reported for the cylindrical test and the proposed analytical equation. However, this was only for one sample geometry and so little significance can be drawn from the results. The small-scale 800 mm connection with shear-keys was tested under a constant axial load with reversed bending moment and so was representative of the loading conditions experienced by offshore WTG grouted connections. The implication of representing environmental conditions, such as the presence of water was not considered in terms of the influence on friction coefficient, but was considered for the influence on S-N curve for grouted connections, where the in-air curve should be reduced by a factor of 0.8 [41].

Lotsberg [42] offers more detail on the derivation of the analytical equations for capacities of grouted connections under combined axial and bending moment loading conditions with and without shear-keys based on the principles described in Ref. [40]. The work improved on

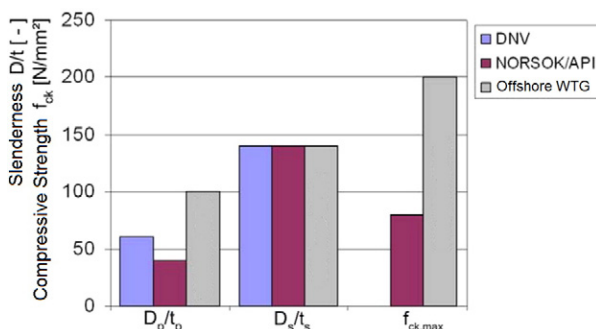


Fig. 27. Limits for the application of design rules selected offshore standards [39].

**Table 2**  
Comparison of grout papers.

Author(s)	Year	Title	Connection Type			Axial Loading		Bending	Combined Loading	Limits of Validity				
			Plain	Shear Key	Pre-stress	Static	Fatigue			(D/t) <sub>p</sub>	(D/t) <sub>s</sub>	L/D	(D/t) <sub>g</sub>	f <sub>cu</sub> (MPa)
Billington and Lewis	1978	The Strength of Large Diameter Grouted Connections	✓	✓		✓				9.6 < x < 45	18 < x < 120	1 < x < 12	7.6 < x < 26	2 < x < 110
Billington	1980	Research into Composite Tubular Construction for Offshore Jacket Structures	✓	✓		✓		✓	✓	<40	<80	>2	7 < x < 45	17 < x < 110
Billington and Tebbet	1980	The Basis for new Design Formulae for Grouted Jacket to Pile Connections	✓	✓		✓	✓	✓	✓	<40	<80	>2	7 < x < 45	17 < x < 110
Karsan and Krahl	1984	New API Equation for Grouted Pile to Structure Connections	✓	✓		✓				24 < x < 40	50 < x < 140	1 < x < 12	10 < x < 45	2 < x < 110
Krahl and Karsan	1985	Axial Strength of grouted Pile-to-Sleeve Connections	✓	✓		✓				24 < x < 40	50 < x < 140	1 < x < 12	10 < x < 45	2 < x < 110
Tebbett and Billington	1985	Recent Developments in the Design of Grouted Connections	✓	✓		✓	✓			24 < x < 40	50 < x < 140	<2	10 < x < 45	2 < x < 110
Tebbett	1987	Recent Developments in the Design of Grouted Connections	✓	✓		✓	✓			24 < x < 40	50 < x < 140	<3	10 < x < 45	2 < x < 110
Lamport, Jirsa and Yura	1987	Grouted Pile-to-Sleeve Connection Tests		✓		✓		✓	✓	28.8	42	1 < x < 2	20	27 < x < 73
Forsyth and Tebbett	1988	New Test data on the Strength of Grouted Connections With Closely Spaced Weld Beads		✓		✓				34.1	88.5	0.9	15.7	9 < x < 61
Smith and Tebbett	1989	New Data on Grouted Connections with Large Grout Dimensions		✓		✓				18.66	36	2	6.5	12 < x < 45
Sele and Kjeøy	1989	Background for the New Design Equations for grouted Connections in the DNV Draft Rules for Fixed Offshore Structure	✓	✓		✓		✓		10 < x < 60	18 < x < 140	1 < x < 12	—	4 < x < 90
Aritenang, Elnashai, Dowling and Carroll	1990	Failure Mechanisms of Weld-Beaded Grouted Pile/ Sleeve Connections		✓		✓				8 < x < 15	72	1.1	14	50 < x < 64
Elnashai and Aritenang	1991	Nonlinear Modelling of Weld-beaded Composite Tubular Connections		✓		✓				8 < x < 15	72	1.1	14	50 < x < 64
Lamport, Jirsa and Yura	1991	Strength and Behaviour of Grouted Pile-to-Sleeve Connections		✓		✓				28.8	42	1 < x < 2	20	27 < x < 73
Aritenang, Elnashai and Dowling	1992	Analysis-based Design Equations for Composite Tubular Connections		✓		✓				8 < x < 15	72	1.1	14	50 < x < 64
Sele and Skjolde	1993	Design Provisions for Offshore Grouted Construction	✓	✓		✓				<40	<80	>2	7 < x < 45	17 < x < 110
Harwood, Billington, Buitrago, Sele and Sharp	1996	Grouted Pile to Sleeves Connections: Design Provisions for the New ISO Standard for Offshore Structures	✓	✓		✓	✓			20 < x < 40	30 < x < 140	1 < x < 10	10 < x < 50	10 < x < 80
Dowling, Elnashai and Carroll	1983	A New Pressurised Grouted Connection for Steel Tubulars			✓					17.7	34.4	2.2	7.8	70
Elnashai, Carroll and	1985	A Prestressed, High-Strength Grouted Connection for	✓		✓	✓				17 < x < 32.6	< x < 2	< x < 7.6	< x < —	

(continued on next page)

Table 2 (continued)

Author(s)	Year	Title	Connection Type			Axial Loading		Bending	Combined Loading	Limits of Validity				
			Plain	Shear Key	Pre-stress	Static	Fatigue			(D/t) <sub>p</sub>	(D/t) <sub>s</sub>	L/D	(D/t) <sub>g</sub>	f <sub>cu</sub> (MPa)
Dowling		Offshore Construction								50	36	20	17	
Elnashai, Carroll, Dowling and Billington	1986	Full Scale Testing and Analysis of Prestressed grouted Pile/Platform Connections	✓		✓	✓				25 < x < 50	30.3 < x < 36	1 < x < 2	17	–
Grundy and Kiu	1991	Prestress Enhancement of Grouted Pile/Sleeve Connections	✓	✓	✓	✓				30 < x < 37	43 < x < 100	1 < x < 4	13 < x < 27	–
Boswell and D'Mello	1986	The Fatigue Strength of Grouted Repaired Tubular Members		✓			✓			32.4	73.2	1.0	22.25	72 < x < 93
Ingebrigtsen, Løset and Neilsen	1990	Fatigue Design and Overall Safety of Grouted Pile Sleeve Connections	✓	✓		✓	✓			24.5 < x < 28.8	29.5 < x < 65.5	2.0	28.2 < x < 33.8	27 < x < 49
Etterdal, Askheim, Grigorian and Gladsø	2001	Strengthening of Offshore Steel Components using High-Strength Grout Component Testing Analytical Methods	✓			✓	✓	✓	✓	N / A	25.7	1.0	N / A	140, 213
Zhao, Grundy and Lee	2002	Grout Sleeve Connections under Large Deformation Cyclic Loading	✓		✓		✓			26.0, 33.0	19.0, 27.0	1.3 < x < 4.3	7.5, 20.1	–
Andersen and Petersen	2004	Structural Design of Grouted Connections in Offshore Steel Monopile Foundations	✓			✓	✓	✓		72.0, 84.4	–, 80.0	1.5	–, 68.0	140, 114
Lohaus and Anders	2006	High-cycle Fatigue of Ultra-high Performance Concrete – Fatigue Strength and Damage Development		✓			✓			5.5	14.3	1.5	5.2	135 < x < 225
Zhao, Ghojel, Grundy and Han	2006	Behaviour of Grouted Sleeve Connections at Elevated Temperatures	✓		✓		✓			21.2	33.0	1.4 < x < 3.1	7.6	–
Schaumann and Wilke	2006	Fatigue of Grouted Joint Connections	✓	✓		✓	✓			5.5, 100	14.3, 107.0	1.3, 1.5	4.6, 42.0	133
<i>Design of plain sided monopile foundations for offshore WTG</i>														
Schaumann and Wilke	2007	Design of Large Diameter Hybrid Connections Grouted with High Performance concrete	✓			✓	✓	✓		100.0	107.0	1.3	42.0	130
Anders and Lohaus	2008	Optimized High Performance Concrete in Grouted Connections		✓		✓	✓			5.5	14.3	1.5	5.2	60 < x < 170
Schaumann, Wilke and Lochte-Holtgreven	2008	Grout-Verbindungen von Monopile-Gründungsstrukturen - Trag- und Ermüdungsverhalten	✓	✓				✓		100.0	107.0	1.3	42.0	130
Klose, Faber, Schaumann and Lochte-Holtgreven	2008	Grouted Connections for Offshore Wind Turbines	✓	✓				✓		100.0	107.0	1.3	42.0	130
<i>Unexpected settlements occurred</i>														
Schaumann, Bechtel and Lochte-Holtgreven	2010	Fatigue Design for Prevailing Axially Loaded Grouted Connections of Offshore Wind Turbine Support Structures in Deeper Waters	✓	✓		✓	✓			4.8	14.3	1.5	5.2	70 < x < 115
Schaumann, Lochte-Holtgreven, Lohaus and Lindschulte	2010	Durchrutschende Grout-Verbindungen in OWEA – Tragverhalten, Instandsetzung und Optimierung	✓	✓		✓				5.5	14.3	1.5	5.2	60 < x < 170
Lotsberg, Serednicki, Oerlemans, Bertnes and Lervik	2013	Capacity of Cylindrical Shaped Grouted Connections with Shear Keys in Offshore Structures		✓		✓	✓	✓	✓	–	–	–	–	–
Lotsberg	2013	Structural Mechanics for Design of Grouted Connections in Monopile Wind Turbine Structures		✓		✓	✓	✓	✓	–	–	–	–	–
Typical Offshore WTG Grouted Connection *			Specification											
Monopile			✓			✓	✓	✓	✓	90	86	1.5	63	130
Jacket Pin Pile			✓	✓		✓	✓	✓	✓	48	57	2	8.5	130

\* Relative values for a typical offshore WTG grouted connection have been provided to assist in appreciation of the scale and potential validity, of research and development.

the significance of the results of the author's previous work [40] by comparing the analytical predictions with the experimental results from six additional cylindrical bending tests with varying grout strength, connection lengths and number of shear-keys undertaken at the University of Leibniz, as reported in Refs. [36,39]. This saw very good agreement in all but one of the tests, which was explained by the variation in experimental testing. However, the gaps in the validation when representing environmental conditions in operation still remain. As part of the conclusions of this paper, it stated that non shear-key connections could not be recommended due to the low long term axial capacity, in agreement with Ref. [39], and that contact pressure should be limited to minimise the potential of cracking of the grout and abrasive wear.

## 6. Summary and conclusions

Table 2 summarises the main areas investigated by each of the papers reviewed in the previous sections, including their limits of validity, which play a crucial role on the applicability of any lesson learnt and conclusion drawn in each of them.

The review of the publicly available technical papers indicates that there has been significant development within the area of grouted connections over the years, which is often driven by the need for optimisation, but lately to improve the understanding of the reported insufficient axial capacities in the offshore wind industry. A large amount of work was initially carried out on the axial capacity of grouted connections. Once the increased capacity of shear-key connections was proved, the research focus shifted onto other aspects, including, effects such as pre-stress and damage accumulation, with an increasing number of challenging applications in the oil, gas and offshore wind industry.

From the papers published before mid 2009, when early-age unexpected settlements started to be reported in large-diameter grouted connections for offshore WTG foundations, this review paper demonstrates that:

1. A reason for this unsatisfactory performance could be an inadequate understanding of the limits of existing design codes, particularly because of the complex composite interaction under a high number of multi-axial stress cycles.
2. Limited testing was undertaken that was representative of the actual loading conditions experienced by offshore WTG structures, and even less representative of the confinement similar to current monopile structures, and no testing under representative environmental conditions.
3. In some circumstances it may not be conservative to assume that bending and axial loads do not interact, as was assumed in the offshore WTG grouted connection designs that have experienced settlements.
4. Previous testing had indicated that combined axial and bending may lead to reduction of the axial capacity and that gapping and relative displacements could then occur well below the ultimate load capacity.
5. Offshore filling procedures and curing under changing environmental conditions could influence the grout properties and therefore the resulting connection strength, but these factors have not yet been considered when evaluating more recent experimental testing and WTG installation procedures.
6. The combination of grout powder formation under cyclic deformation and the presence of water due to the submerged nature of the connections, potentially reducing the steel-grout coefficient of friction, it could explain how the design capacities of the connection may not have been conservative.

For these reasons, the occurrence of such conditions and the insufficient axial capacity in the offshore WTG should not have been completely unexpected in non-shear-key connections.

Current research has indicated that some of these previous assumptions and understanding of the behaviour may have been incorrect for such plain-pipe connections, including scaling effects for certain parameters and operational conditions, but it is evident that further work is still required to fully understand all the influencing factors. It is also worth emphasising the key observation that, as far as the applicability of design guidelines is concerned, this can only be guaranteed up to the original limits of derivation, e.g. the experimentation carried out, which have been far exceeded by today's dimensions of offshore WTG structures.

It is therefore recommended that experimental campaigns undertaken in the future should be truly representative of the actual environmental conditions expected on site, such as the presence of water and environmentally degraded steel, to investigate the influence of these factors on the capacity and durability of the connections over their design life. There also appears to be a gap in knowledge on material behaviour such as the effect of multi-axial stress states on the fatigue life of different strength grouts, and therefore this aspect should also be investigated. Structural monitoring equipment installed on plain-pipe connections exhibiting signs of insufficient axial capacity has detected continual relative movement. Although in itself, very small relative movements are not a sign of insufficient axial capacity, the influence of this movement on key parameters such as coefficient of friction and grout/steel integrity should also be investigated under representative conditions, testing for which is underway. In this respect, a possible experimental setup has been recently proposed in Ref. [43], and results of this testing campaign are currently being collected.

Overall, this review shows that:

1. Further testing should have been undertaken to understand the behaviour beyond those limits, as also recommended by many authors in the reviewed material.
2. Inter-industry review of previous experience plays an important role when existing technological solutions are used to solve new and different problems or to upscale the size of the construction. Specifically, as historic oil and gas testing provided evidence that there was a risk of overestimating axial capacity and so would have justified the cost of undertaking testing applicable to the conditions of offshore WTG grout connections.
3. A lack of information flow between researchers, design standard organisations, designers and operators can severely affect the deployment of technological solutions outside their original development. Importantly, this flow of information should not just be one way, but provide feedback throughout the knowledge loop to ensure testing, standards and design are relevant to operation performance of the structure and design conditions. Not only does this bring the benefit of validation of design assumptions potentially from structural condition monitoring, but ensures the cost of such engineering challenges are minimised. This virtuous circle would also offer potential for optimising the design of future installations, therefore reducing the cost of offshore wind energy.

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