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FACTORS OF SAFETY FOR GROUTED CONNECTIONS

PHASE II

FATIGUE STRENGTH OF GROUTED CONNECTIONS

1. INTRODUCTION

This PHASE II extension follows on from work done to estimate factors of safety for use in ultimate strength design which will give similar levels of accuracy to that achieved for other structural components. The PHASE I work indicated that the factors of safety entrenched in the API RP 2A LRFD code and DnV's rules for classification were adequate. In the case of the requirements entrenched in D. En.'s guidance for offshore construction it was found that there was room for a significant reduction of the specified factors of safety.

The stated aim of this project is to utilise the available test information on grouted connections to establish optimised design recommendations. There has been a strongly felt need in the industry to establish uniform safety measures for the design of grouted connections. Having calculated the factors of safety necessary to obtain safety provisions for static strength of grouted connections which are uniform and of a similar magnitude to that of other structural components it is necessary to establish that the provisions for other failure modes will still be adequate. Typically the provisions of the D. En. guidance has also consciously been aimed at providing a sufficient measure of safety against fatigue failure of grouted connections. If the reduced factors of safety for ultimate strength are to be implemented in the instance of the D. En. provisions it is therefor necessary to verify that fatigue is adequately provided for.

Work done about ten years ago established that for grouted connections which were plain or had a modest shear key intensity as was common at that time the effect of fatigue was modest and could be neglected when large factors of safety were used in ultimate strength design. Since then designers have moved towards larger shear key intensities (H/S). If safety margins on ultimate strength requirements are to be reduced it is necessary review the provisions which need to be made for cyclic loading.

The Department of Energy along with a number of offshore operators have recently performed early age cycling and fatigue on a series of specimens with very dense shear key spacing. These tests have shown greater influences from cyclic loading than previously experienced. Work performed by VERITEC has shown that the very dense shear key spacing are not relevant for design as no further ultimate strength benefit is gained when the shear key spacing is reduced below a certain limiting value. As the previous tests which were performed on specimens with a greater shear key spacing (and lower shear key intensity) showed less impact from cyclic loads, it is of great importance to establish whether there is a significant impact on specimens with a high shear key intensity (H/S) and with a greater shear key spacing. As cyclic loading is seen to cause degradation of the grout immediately surrounding the shear key discontinuity it may indeed be speculated that the impact on specimens with greater shear key spacing will be reduced.

2. BACKGROUND

Fatigue does not have significant impact on the performance of plain grouted connections. Available knowledge about grouted connections with shear keys is summarised in figs. 2.1 to 2.3. The following trends appear to be observable:

* The peak load rather than the load range appears to determine load capacity.

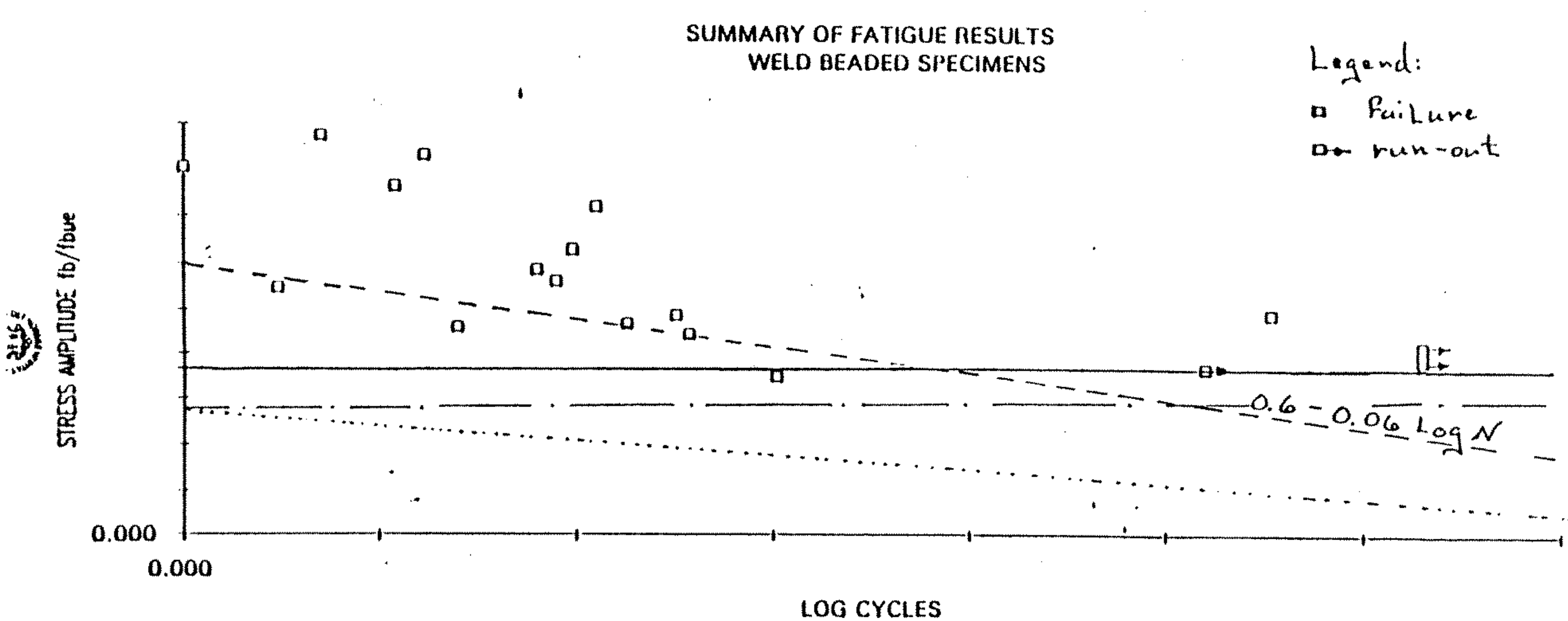
* There appears to be a fatigue limit at about 45% of the static capacity. (It should be recognised that this is of a similar magnitude to the capacity of the plain connection.)

It should further be recognised that the test results quoted all apply to low shear key intensities (H/S of about 0.01).

The solid horizontal line in fig. 2.1 corresponds to the maximum static capacity determined by the DnV rules using the partial factors of safety determined in phase I of this project for dead load. The chain line corresponds to environmental loading. Applying these partial factors of safety fatigue would not have been critical for any of the specimens tested. The dotted line indicates the order of magnitude of the long term distribution associated with the allowable ultimate strength design.

In order to make these results generally valid it is necessary to establish how they are influenced by higher shear key densities (higher H/S ratioes). Such project work has been undertaken in Britain and is presently being concluded. Early indications have been that early age cycling has a significant influence on the fatigue performance for specimens with higher H/S ratioes. A special feature of these tests is that they have all been performed on specimens with very short shear key spacing. Work performed by DnV /1/, /2/ has shown that for shear key spacings less than the square root of the product of the pile diameter and thickness ($S < \sqrt{D_p * T_p}$) no further load capacity increase is obtained. Tests performed by Wimpey have confirmed this /3/, /4/. See also Appendix A. The tests performed in Britain have all been performed on specimens with small and densely spaced shear keys in the range where benefit is not gained from the closer spacing. These test results are therefor not in a particularly design relevant parametric range.

As it is observable that the effect of cyclic loading is to cause deterioration of the grout material immediately surrounding the shear keys it is still judged to be necessary to establish whether cyclic loading will have a significant impact on more widely spaced shear keys as would be used in practical design applications.



SUMMARY OF FATIGUE RESULTS
WELD BEADED SPECIMENS
Legend:
Failure
run-out
STRESS AMPLITUDE fb/fbue
0.000
0.000
LOG CYCLES
0.6 - 0.06 Log N

FIG. 2.1

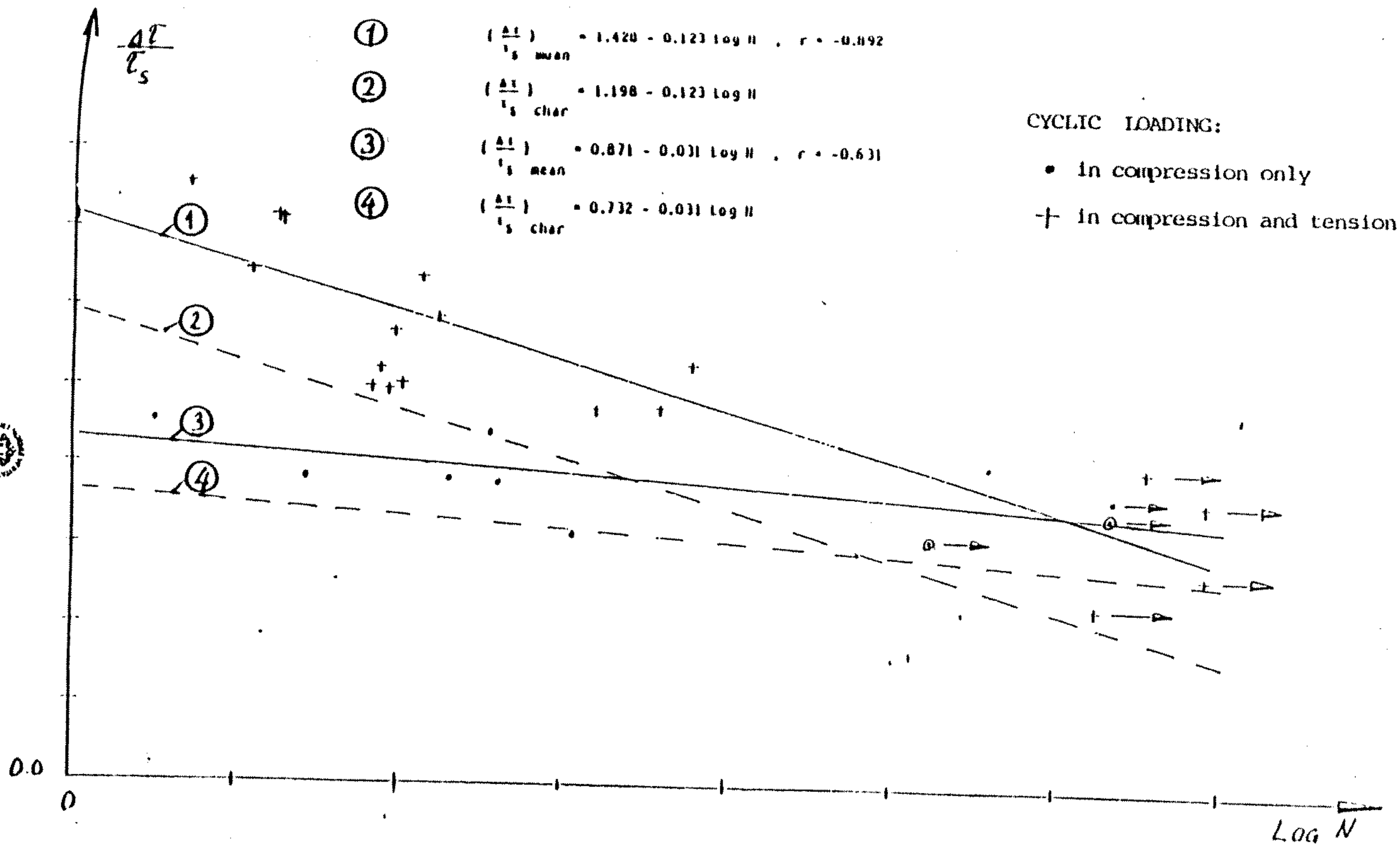


FIG. 2.2 LINEAR REGRESSION AND CHARACTERISTIC LINES OF COMPRESIOSN-COMPRESSION AND COMPRESSION-TENSION SPECIMENS, VERITAS AND WIMPEY LAB, $\Delta\tau/\tau s$ V.S LOG N

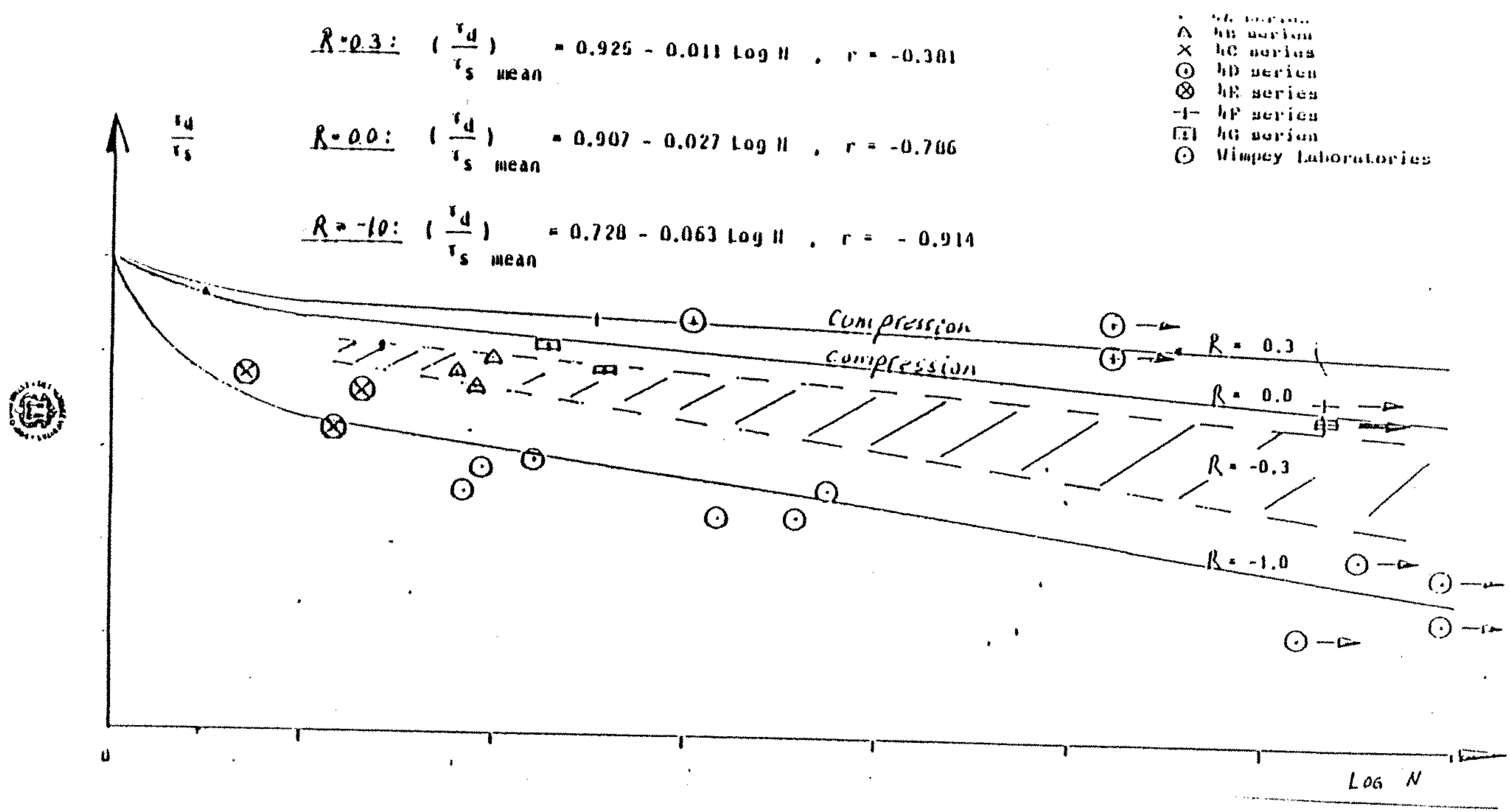


FIG. 2.3 LINEAR REGRESSION LINES FOR R = 0.3, 0.0 AND 1.0. $\tau/\tau s$ V.S. LOG N

3. PROJECT DESCRIPTION

The aim of this project is to test specimens with high H/S ratios and widely spaced shear keys exposed to fatigue as well as early age cycling. In order to clarify these aspects it is proposed to test closely similar specimens where only shear key pitch and the inclusion of early age cycling is carried out. It is proposed to test ten specimens. The following specimen configuration will be used:

$H/S = 0.05$
f_{cu} = approximately 50 MPa
$L/D = 2$
$D_p/T_p = 40$
$D_s/T_s = 60$

The test load level will be kept at about one third of the ultimate static capacity and the cyclic stress ratio (R) will be kept at -1.

S/√RT	Fatigue load	Fatigue and early age cycling
1	No 3	No 4
3	No 5	No 6
6	No 1	No 2

Specimens No's 7,8,9 and 10 will be allocated to making repeat tests to verify trends by supplying more statistically representative samples. (As an example specimens 7 and 8 could be allocated to repeating the test for specimen No 1 and specimens 9 and 10 to repeating the test on specimen No 3.) The specimen sequence has been fixed such that if there is a no trend indication at an early stage subsequent specimens can be relocated.

The tests will be executed at VERITEC's facilities in Høvik, Norway. A dynamic actuator with a capacity of +/- 400 tonnes and a maximum cyclic rate of 5 Hz will be used. In order to ensure a uniform test condition during fatiguing the actual accelerated fatigue testing at higher frequencies will only be initiated after 28 curing. The specimens that are subjected to early age cycling will during the curing period initially be exposed to a specified displacement at a rate of 0.1 Hz. When specific load threshold is reached the load application will be shifted to force control and the load cycling continued at the specified level and a frequency of 0.1 Hz until the curing is complete.

4. ACTIVITY DESCRIPTION

COST-TIME-RESOURCE SHEET

CTR TITLE: FABRICATION OF TEST FIXTURES

CTR NO. : 1
REVISION : 0

OBJECTIVE:
To provide fixtures for a ± 400 tons test rig for fatigue testing of grouted connections.

SCOPE:
An approximately 800 mm dia. steel tube will be provided with flanges either end to anchor a specimen as well as for anchorage to the test rig. Two sets of specimen end flanges will be provided.

INPUT DATA:
Not required.

DELIVERABLES:
- One tubular test fixture with end flanges and apertures.
- Two sets of specimen end flanges.

START DATE: **DURATION:** 6 weeks

COST:

Design	NOK 15,000
Test fixture fabrication	NOK 40,000
2 sets of end flanges	NOK 15,000
	NOK 70,000

COST-TIME-RESOURCE SHEET

CTR TITLE: SPECIMEN FABRICATION AND TESTING

CTR NO. : 2
REVISION : 0

OBJECTIVE:
To fabricate specimens of grouted connections and to test them in fatigue with preconditioning by early age cycling as specified. The fatigue testing will be continued until failure defined as a maximum relative displacement or run-out defined as $2x10^6$ cycles.

SCOPE:
The specimens will be fabricated with shear keys made from round reinforcing bars and end flanges will be welded on. Grouting will be performed with a plain oil well cement and water slurry and the specimens will be cured for 28 days before fatigue testing is commenced. Early age load cycling will be applied as a prescribed maximum force applied at a rate of 0.1 hz.

INPUT DATA:
Test matrix to be approved by the Steering Committee.

DELIVERABLES:
- Fatigue test duration.
- Cube strength values for grout.

START DATE: **DURATION:** 1+4+2 weeks

COST:

Rig hire fatigue	NOK	20,000
Fabrication	NOK	30,000
Cube testing	NOK	5,000
Fatigue testing	NOK	40,000
Installation	NOK	20,000
	NOK	115,000
Early age cycling	NOK	20,000
Rig hire	NOK	20,000
	NOK	40,000

COST-TIME-RESOURCE SHEET

CTR TITLE: REPORTING OF 10 FATIGUE TESTS

CTR NO. : 3
REVISION : 0

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OBJECTIVE:
To provide a report on all relevant aspects of the tests executed.

SCOPE:
The report will contain the following chapters:

0. Executive summary
1. Introduction
2. Rig description
3. Specimen description
4. Test procedure
5. Results
6. Conclusions

The draft will be circulated for comments. Comments received within 4 weeks of draft issue will be considered for inclusion in the final issue.

INPUT DATA:
Test results.

DELIVERABLES:
- Draft test report for comment, 1 copy.
- Final test report, 1 copy.

START DATE: **DURATION:** 4 weeks

COST:

NOK 55,000

5. COST AND CONTRIBUTION

The second phase of the project will be initiated when a minimum of five specimens can be tested. The minimum budget is thus:

Test fixture fabrication	NOK 70 000
Fatigue testing of five specimens	NOK 575 000
Cyclic testing of two specimens	NOK 80 000
Reporting	NOK 60 000
Cost	NOK 785 000

6.724 NOK = $1.00 US

A minimum of four participants is required each contributing NOK 195 000 (approximately USD 29 000). Participation in excess of four will contribute to increasing the scope of work to the full 10 specimen programme.

New participants will also be invited. The new participants will be offered to obtain the results of Phase I for an additional contribution of 50% (NOK 98 000). $14,575

Public bodies are required to contribute at half the above rate which applies to industrial participants.

6. REFERENCES

1. Sele, Arne and Seow Phoi Fah:"Recent Developments in Design and Use of Grouted Repair and Strengthening of Structures and Pipelines." Paper presented to the 7th Offshore South East Asia Conference, Singapore, 1988.

2. "Factors of safety for grouted construction; report on screening of data base on test results." VERITEC report no. 90-3864.

3. U. K. Department of Energy:"The Strength of Grouted Pile-Sleeve Connections.", OTH 86 210, 1986.

4. Forsyth, P. and Tebbet, I. E.:"New Test Data on the Strength of Grouted Connections with Closely Spaced Weld Beads.", OTC paper no 5833, 1988.

APPENDIX A

THE EFFECT OF CLOSELY SPACED SHEAR KEYS:

1. NOTATION:

A:	Bolt cross sectional area
C:	Cohesion
d:	pile diameter
D:	Sleeve diameter
E_g:	Young's modulus for grout
E_s:	Young's modulus for steel
f_g:	Compressive strength of grout
h:	Grout thickness
H:	Shear key upstand
k:	coefficient
K:	Stiffness coefficient
m:	Modular ratio; E/E
r:	Pile radius
R:	Sleeve radius
s:	Bolt spacing
S:	Shear key spacing
t:	Pile thickness
τ:	Sleeve thickness
δ:	Absolute value of roughness
E:	Strain
κ:	Wave length constant
λ:	Wave length constant
μ:	Friction coefficient
σ:	Axial stress
T:	Shear stress (interface bond)

2. PLAIN PILE SLEEVE CONNECTION

Tests performed by VERITAS with the aim to establish further details on the actual physical performance of grouted joints and connections have revealed a number of facts which strongly indicate that adhesive bond is not mobilised in these grouted components:

1. Even grout with a shrinkage compensating agent (expansive grout) has a net shrinkage when cured under confined and sealed conditions. In grouted connections the net shrinkage causes debonding and leaves a small gap at the grout to steel interface.

2. Adhesion of grout to steel between parallel plates has been tested and found to be negligible.

3. Shear tests performed on grout between parallel plates under varying levels of normal stress reveal that shear transfer is essentially frictional.

4. Measurements on grouted clamps show that there is significant end displacement prior to slip. This displacement is accompanied with significant redistribution of stresses which necessarily must be accompanied with relative displacements between grout and steel.

5. Tests on grouted clamps exhibit pronounced slip-stick performance in the post-ultimate range. (See Figure A). This is a typical feature of a friction connection reflecting the difference between the static and the cynamic coefficients of friction and the elastic energy of the test frame.

From these findings, it must be concluded that the concept of adhesive bond cannot be applicable to the actual physical mode of action of grouted sleeves. As it has been found that there is a significant amount of debonding between grout and steel at the interface, it seems that the bond stresses which are being mobilized must be due to friction.

Mast (1) was the first to formulate the shear friction hypothesis for concrete. Based on measurements, he found the following values of the coefficient of friction:

Concrete to cracked concrete	:	$\mu = 1.4$
Concrete to steel	:	$\mu = 1.0$
Concrete to smooth concrete	:	$\mu = 0.7$

At a later date, Cowen /4/ has produced test results showing that cracked concrete can more accurately be modelled as a Coulomb material with:

Concrete:

Cohesion	:	$C = 0.15\ f_c$
Coefficient of friction	:	$\mu = 0.75$

From tests executed at VERITAS Structural Laboratory at Singapore Science Park, it has been found that in the case of grout to steel interfaces the cohesive term is negligible and that the coefficient of friction is $\mu = 0.7$.

Grout to steel:

Cohesion	:	$C = 0$
Coefficient of Friction	:	$\mu = 0.7$

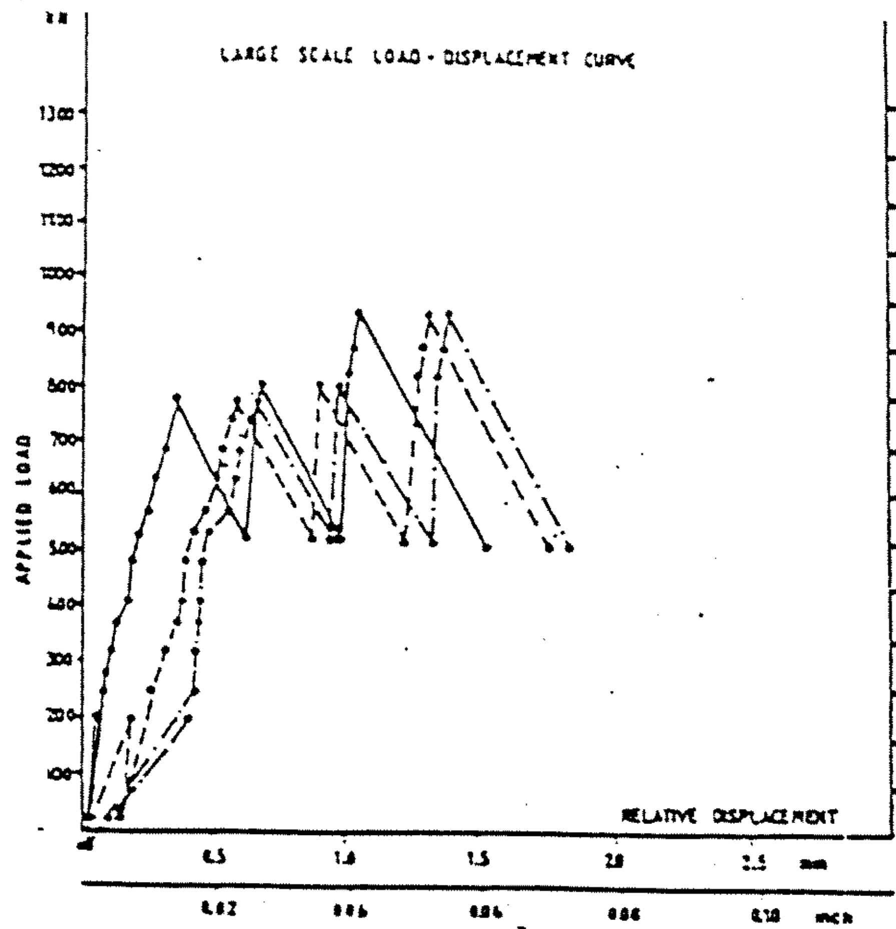


FIG. A LOAD DEFORMATION RECORDING FROM PILE SLEEVE TEST

The basis of the shear friction approach to interpretation of the strength of grouted clamps is shown in Figure B. The plane of rupture is on the interface between the grout and the inner steel pipe. The surface of this rupture plane is not completely smooth but has small undulations and irregularities of height δ given rise to friction. In order to slide along this plane the inner tube must compress (δ_t) and the outer grout and steel tube must expand slightly (δ_t) as well as the grout compressing slightly (δ_n), the combined deformation equalling.

$$\delta = \delta_t + \delta_g + \delta_n \qquad [1]$$

The inner tube contraction is:

$$\delta_t = r\varepsilon_t \qquad [2a]$$

The outer grout annulus and tube expands by:

$$\delta_T = RE_T \qquad [2b]$$

The grout layer compresses over its thickness by:

$$\delta_g = hE_g \qquad [2c]$$

In further, it is taken that R>>h:

σ_g = radial stress in grout layer

Which gives rise to the following hoop stresses:

The inner tube: $\sigma_t = r/t \ \sigma_g$
The outer tube: $\sigma_T = R/\tau \ \sigma_g$ [3]

The equivalent thickness of the combined outer tube and grout is:

$$\tau_{eq} = \tau + \frac{E_g}{E_s} h$$

Combining the above, we obtain:

$$\delta = \frac{r^2}{t} \frac{\sigma_g}{E_s} + \frac{h}{E_g} \sigma_y + \frac{Rr}{T + \frac{E_g}{E_s} h} \frac{\sigma_g}{E_s}$$

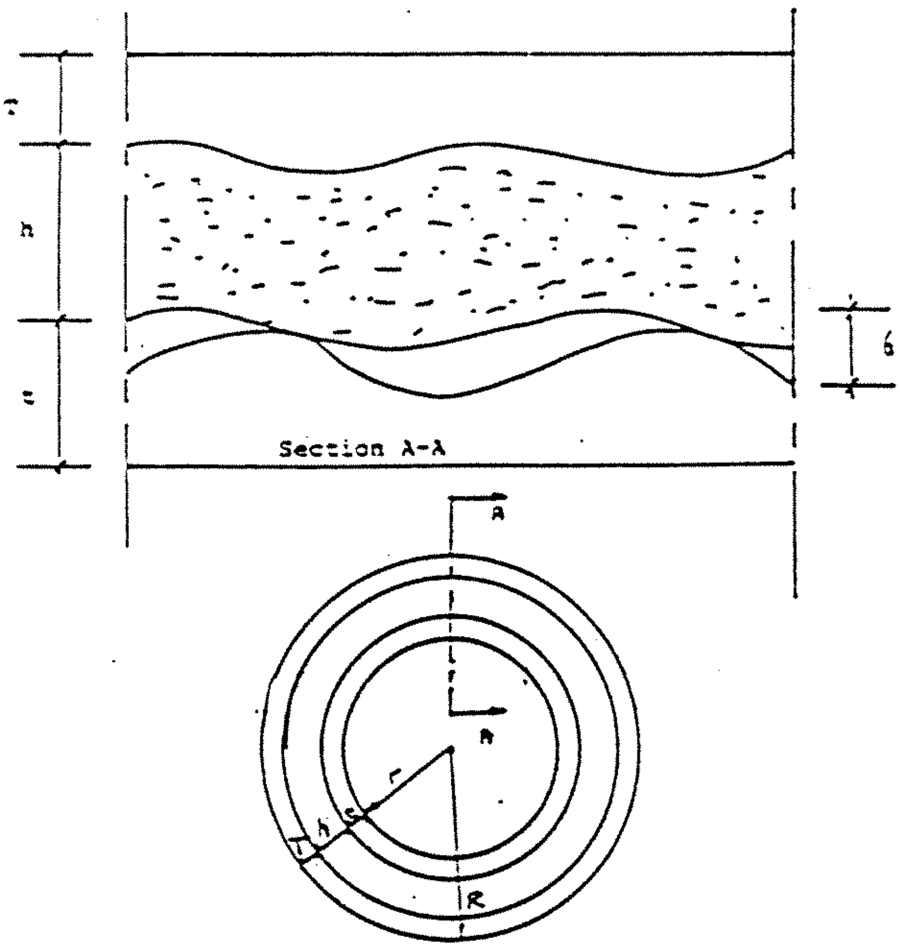


FIG. B **DIAGRAMATIC ILLUSTRATION OF WEDGING EFFECT ARISING FROM SURFACE IRREGULARITIES**

From which we obtain by rearrangement:

$$\sigma_g = \frac{\frac{\sigma}{r} E_s}{\frac{r}{t} + \frac{E_s}{E_g}\frac{h}{r} + \frac{R}{T + \frac{E_g}{E_s} h}}$$

By the shear friction analogy, the interface shear is:

$$\tau = \mu \sigma_g = \mu \frac{\delta}{r} E_s \left(\frac{r}{t} + \frac{E_s}{E_g}\frac{h}{r} + \frac{R}{T + \frac{E_g}{E_s} h} \right)^{-1}$$

E = 210 000 MPa
μ = 0.7

E_g, the modulus of elasticity of grout varies considerably depending on grout composition. In tests performed Eg has in general not been measured. It is known that Eg is proportional to the compressive strength of grout cubes f_g. It is also known that tests on grouted clamps have in general been performed on a pure cement water grout using oil well cement. For this type of grout, it is estimated that:

$$E_g = 150 \, f_g \qquad [6]$$

From test data it has further been calculated that:

$$\frac{\delta}{r} = 0.25 x 10^{-3}$$

4.0 BOLTED CLAMPS

Tests performed on bolt assembled grouted clamps have given lower results than predicted by the D En formulae. So far this has been explained by grout segregation occurring as a consequence of casting in a horizontal position. This undoubtedly will contribute.

From the shear-friction analogy is does, however, follow that the stiffness of the bolts must also be accounted for as this will contribute to the force deformation relationship. Tests performed at DnV's Laboratories in Singapore Science Park show that in cases the D En formulae would give a massive several fold over-prediction of the capacities of some bolted clamps if applied to these.

By an analogous development to that in the preceding section a formula incorporating the bolt stiffness is found:

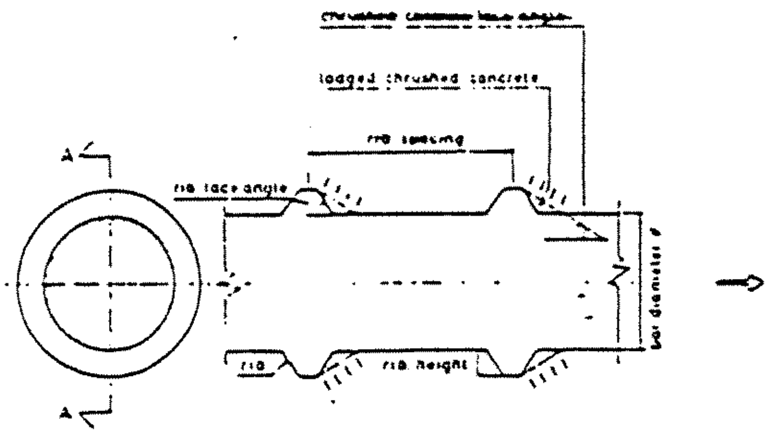
$$\tau = \mu \frac{\delta}{r} E_s \left(\frac{r}{t} + \frac{E_s}{E_g}\frac{h}{r} + \frac{R}{T + \frac{E_g}{E_s} h} + \frac{ls}{2A} \right)^{-1}$$

Where:

l = bolt length
S = bolt spacing
A = cross sectional area of bolts

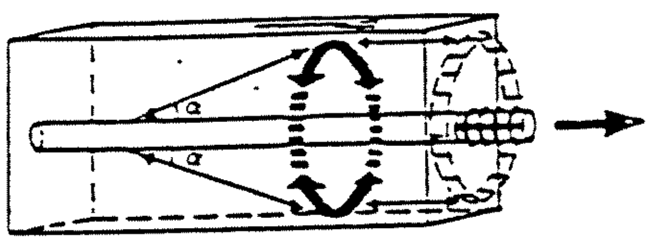
5.0 CLAMPS WITH SHEAR KEYS

A plane pile sleeve type connection is a connection of modest strength. A very significant increase in strength can, however, be achieved by incorporating shear keys in the form of evenly spaced weld beads. These weld beads closely resemble the ribs of high tensile deformed bars for concrete reinforcement. It can quite reasonably be assumed that the mode of action is the same. Ralejs Tepfers (2) has shown (Figure C) that for deformed reinforcing bars crushing of concrete in front of the ribs causes the formation of small wedges of concrete (or grout) with an angle of approximately 45 degrees in front of each rib or shear key. This again causes radial forces in the same way as for the plain connection, but in the form of a concentrated force at the location of the shear key see fig. D. In the case of a pile sleeve connection this concentrated force will cause a localized deflection at the location of the shear key. Reference (3) gives the relationship between force and deformation for a uniform circular line load:



CUT A - A

The geometry of a deformed reinforcing bar and the mechanical interaction between the bar and the concrete.



Schematic representation of how the radial components of the bond forces are balanced against tensile stress rings in the concrete in an anchorage zone.

FIG. C

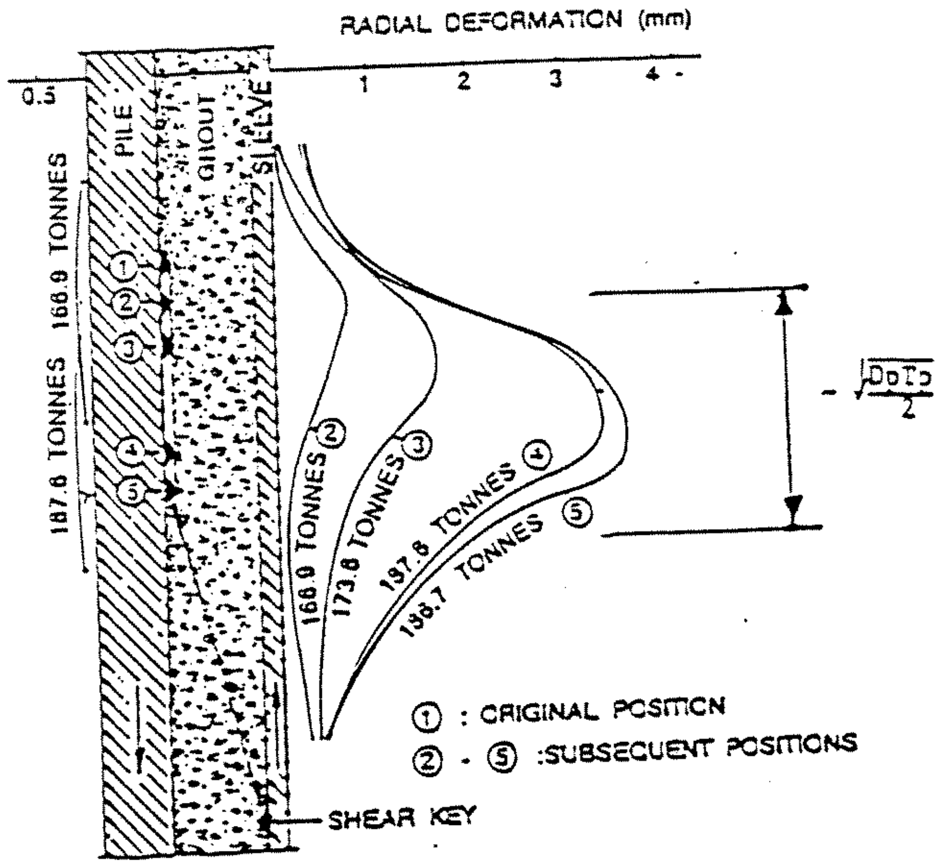


FIG. D RADIAL DEFORMATION OF PILE AND SLEEVE IN THE VICINITY OF A SHEAR KEY RECORDED IN A PILE SLEEVE TEST.

$$P=\delta\frac{8K\kappa^3}{r^3}\frac{Sinh2\lambda+Sinh2\lambda}{Cosh^2\lambda+COS^2\lambda}$$

$$\kappa^4=3(1-\upsilon^2)\frac{r^2}{t^2}$$

$$\lambda^4+4\kappa^4=0$$

$$K=\frac{E_s t^3}{12(1-\upsilon^2)}$$

From this, it can be derived that the "effective width" associated with one weld bead is proportional to $\sqrt{rt}$. That is:

$$b_{eff}=k\sqrt{rt}$$

In the case of shear keys, the term δ/r thus becomes H/r where H is the shear key upstand. The force created by the shear keys would however only act in the locality of the shear keys. If this force is to be expressed as an average contact stress, we must modify this by a factor $k\sqrt{rt}/S$ where "S" is the shear key spacing. This yields:

$$\tau=\mu E_s\left(\frac{r}{t}+\frac{E_s}{E_y}\frac{h}{r}+\frac{R}{T+\frac{E_g}{E_s}h}\right)^{-1}\left(\frac{\delta}{r}+k\frac{H}{r}\frac{\sqrt{rt}}{s}\right)$$

$$\tau=\mu E_s\left(\frac{r}{t}+\frac{E_s}{E_g}\frac{h}{r}+\frac{R}{T+\frac{E_g}{E_s}h}\right)^{-1}\left(\frac{\delta}{r}+k\frac{H}{S}\sqrt{\frac{t}{r}}\right)$$

The above derivation has only taken account of the shear force transmitted due to the shear friction mobilized by the induced radial stress. Due to the inclination of the wedge, there will also be a component of stress normal to the wedge surface in the tangential surface. This component is a function of the wedge angle. Reference (2) has for the case of deformed reinforcing bars found that this angle is a function of the concrete strength. By fitting to test data, it has been found that the coefficient k is a function of the grout strength:

$$k=\frac{1}{16}F_g^{0.3}$$

which leads to:

$$\tau=\mu E_s\left(\frac{r}{t}+\frac{E_g}{E_s}\frac{h}{r}+\frac{R}{T+\frac{E_g}{E_s}h}+\frac{l_s}{2A}\right)^{-1}\left(\frac{\delta}{r}+\frac{H}{16}S\sqrt{\frac{t}{r}}f_g^{0.3}\right)$$

It follows from the above that when the shear key spacing (s) approaches the effective width, no further significant increase in strength will be obtained by decreasing the shear key spacing. It follows that in calculating the strength, we must also introduce the following approximate limitation:

$S > 1.4\sqrt{rt}$

Test observations performed by Veritec have in fact confirmed the wedging effect occurring at shear keys. Fig. D shows how the sleeve deformers at a shear key location as relative longitudinal movement is induced.

References:

1. Mast, R.F.: "Auxiliary Reinforcement In Concrete Connections". Proceedings of the ASCE Vol 94, No ST6, June 1968, pp 1485 - 1504.

2. Tepfers,R.: "A Theory Of Bond Applied To Overlapped Tensile Reinforcement Splices For Deformed Bars". Publication 73:2 Division of Concrete Structures, Chalmers University of Technology, Gothenburg.

3. Flugge. W.: "Stresses in Shells" Second Edition Springen Verlag, Berlin, 1973 pp 287.

4. Coen, J. and Cruden, A.: "Second thoughts om shear friction". CONCRETE, Aug 1975, VOL 9, No 8.