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1.0 GENERAL

The ability of offshore oil and gas pipelines to respond to subsea soil movements is an important consideration in pipeline design and route selection. These soil movements may be due to submarine landslides or mudslides, seismic activity such as faulting or lateral spreading, or a variety of other causes. The load transfer behaviour between a pipe and the surrounding earth is not well understood.

Offshore pipelines subjected to lateral soil movement will interact with the surrounding soil with relative movement and experience bending, as depicted in Figure 1. (The bending occurs as part of the pipe is embedded in moving soil and part is anchored in intact soil). These two components have been the subject of an ongoing research program at C-CORE funded by MMS, the Geological Survey of Canada (GSC) and others. The first component of lateral pipe soil interaction has previously included full-scale tests in dry dense sand but not submerged sand conditions. C-CORE conducted a research project for the GSC (Hurley et al., 1998a&b) on the second component. That project measured the large strain flexural response of a pipe subjected to bending when buried in dense sand.

The purpose of this current research project for the MMS, as per C-CORE Proposal PCF-97-014 was:

1. To investigate the effect of submerged testbed conditions on the force-displacement response of a pipe buried in dense sand.

2. To conduct a lateral pipe bending test to higher pipe ovalization levels than those previously accomplished.

The results of this project are directly applicable to problems of offshore pipeline/soil interaction resulting from offshore mudslides or as the result of offshore seismic activity (i.e. faulting or lateral spreading). Further, the results are applicable to the previous GSC tests as the soil, pipe, and loading conditions were similar.

These tests thus provide the two-dimensional force-displacement (p-y) response of the pipe. These data were used to back-analyse the behaviour of the GSC pipe using pipeline stress analysis software and methods, that is discrete finite element analyses (soil springs). The MMS and GSC have full access to each other's data; release to third parties will require consent of both the MMS and GSC. Combined, the scope of the current phase of the work consists of a review of offshore and onshore pipeline/soil interaction analyses, a summary of all the tests and results, a comparison of the buried pipeline response to that reported in the literature for a unburied pipeline, and preliminary pipeline stress analyses using conventional finite element methods.

The data reports for this project contain high-quality experimental results of lateral pipeline/soil interaction; pipeline strains, curvature, and ovalisation; soil conditions during testing; internal soil deformations; load-displacement response of the pipelines; and displacement dependance of ultimate loads based on three tests. This final report contains a review of accepted methods of offshore pipeline/soil interaction analysis and a comparison of these accepted methods with experimental results. The results and comparison of the final element analyses were presented in Hurley et al. (1999). The results of this project will lead to more cost-effective designs and increased confidence in the operating integrity of offshore pipelines.

The results obtained from the study are of direct use to industry and regulatory bodies, being quantitative in nature and directly applicable to full-scale events. The results can be used to develop design criteria for future pipeline engineering and for back-analysis of the risk to pipes from submarine soil movements. Industry is interested in this research as the predicted response of pipelines subjected to potential soil movements can then be based upon pipeline-specific data. This will allow cost savings to be made by definition of areas where heavy wall pipe might be needed and thus reduce the probability of failure, increase time between remediation activity, and increase confidence in the operating integrity of the pipeline.

1.1 Program Summary

Table 1 summarizes the four tests conducted as part of the Full Scale Pipe-Soil Interaction project for the Minerals Management Service. Test MMS 01 was conducted in dry sand while MMS 02(02R) was conducted with submerged testbed conditions. Test MMS 02(02R) more closely simulated offshore conditions and a comparison of these two tests should improve understanding of the effects of submerged testbed conditions on the force-displacement response of a buried offshore pipeline.

Each of these two-dimensional tests involved laterally displacing a 0.2m diameter, instrumented pipeline section a distance of 0.1m. The pipeline was initially be placed on a 0.2m- thick bedding layer and with enough cover to correspond to the conditions in the GSC bending tests described above. The pipeline was displaced at a rate of 10mm/hour. The testbed material was a concrete sand available locally. These soils were be prepared in the test tank under conditions similar to those during the GSC tests. The two tests were conducted in moist sand and submerged sand so that saturation effects on the interaction could be assessed. The test facility, actuators, pipelines, instrumentation, data acquisition system, and testing procedures were as described by Paulin et al. (1998).

Test MMS 03 was a lateral pipe bending test and was similar to the two GSC tests, described by Konuk et al (1999). The purpose of MMS 03 was to produce a plastic hinge near the pipe centreline while monitoring pipe ovalization during end displacement. Maximum measured ovalizations during the GSC test program were on the order of 1.5%. It was thought that the experience and knowledge gained through the GSC program could be incorporated into MMS 03 to obtain higher ovalization levels.

Testing began in March 1999 and ended in June 1999. A data report was provided for each test after completion of each test. The data reports include a brief description of the equipment, experimental technique, and plots in engineering units of the data including force-displacement records. The relevant raw data files are available on CD Rom as described in section 4 of this report. This final report summarizes the data reports and is sufficiently thorough to allow full and original use of the data by the MMS. This report also contains a review of accepted methods of offshore pipeline/soil interaction analysis and a comparison of these accepted methods with experimental results.

Table 1 - Details of MMS Experiments

Test	Soil Conditions	Sand Density (kg/m^3)	Avg. Disp. Rate (mm/Hr)	Peak Load (kN)	Disp. To Peak Load (mm)	Date[1]	C-CORE Report
MMS 01	Dry	1984	10.2	97.8	11.3	17/03/99	99-C12
MMS 02	Submerged	1941	9.6	35.4	4.6	12/04/99	99-C18
MMS 02R	Submerged	1864	10.0	37.0	5.5	10/06/99	99-C22
MMS 03	Dry	1861	9.9	-	65.0	19/05/99	99-C19

Notes: 1 - Day/Month/Year

Readers are referred to the reports cited in table 1 for testing details and experimental results. Test MMS 02 was repeated due to problems encountered with frictional loads at the pipe ends. All testbeds were constructed using locally obtained sand from Concrete Products Ltd.. Some goetechnical properties of the testbed sand are given in Table 2.

Table 2 - Geotechnical Properties of Concrete Products Sand

Property Shipment:	#1	#2	#3	Average
Friction Angle (Degrees)	53.1	53.3	52.0	52.7
Maximum Dry Unit Weight, $\gamma_{d\ max}$ (kN/m^3)	19.9	19.8	19.3	19.7
Minimum Dry Unit Weight, $\gamma_{d\ min}$ (kN/m^3)	16.3	16.2	15.5	16.0
Maximum Void Ratio, e_{max}	N/A	N/A	N/A	0.62
Minimum Void Ratio, e_{min}	N/A	N/A	N/A	0.32
Effective Grain Size, d_{10} (mm)	0.15	0.13	0.17	0.15
Mean Grain Size, d_{50} (mm)	0.60	0.65	0.65	.63
Constrained Grain Size, d_{60} (mm)	0.84	0.94	0.89	0.89
Uniformity Coefficient, C_u	5.6	7.2	5.2	5.9
Fines Content (%)	2.5	3.1	1.3	2.3

2.0 SUMMARY OF EXPERIMENTAL RESULTS

Four full-scale pipe - soil interaction tests were undertaken in this project as summarised in

Table 1. The individual test details have been submitted to MMS as cited in the table, that is Hurley and Phillips (1999a, b&c) and Hurley et al (1999). Data from MMS 02 is not presented in this summary because the test was only partially successful and was repeated as MMS 02R. The remaining 3 tests investigated the effect of submerged conditions on purely lateral pipe soil interaction on buried rigid pipe (tests MMS-01 and 02R) and flexure on a flexible buried pipe (test MMS 03).

The load-displacement results from MMS 01 and MMS 02R are presented in Figures 2 and 3 respectively. Test MMS 02R was conducted with a submerged testbed with all other aspects of both tests remaining identical (i.e. same pipe, same burial depth, similar displacement rates). Total peak loads for MMS 01 and MMS 02R were 97.5 kN and 37.0 kN respectively. Displacements to peak load were 11.3 mm for MMS 01 and 5.5 mm for MMS 02R. The submerged conditions reduced the peak load by about 62% and resulted in a 50% reduction in displacement to peak load. These results are further discussed in section 3.1.2.

Figure 4 presents load-displacement results from test MMS 03 which was a lateral pipe bending test. Results from this test have also been submitted to the MMS in the form of a data report (Hurley et al, 1999). The load-displacement plot in Figure 4 is not presented in terms of total load as the objective of the test was to “buckle” the pipe and not to displace the pipe through the sand. The test objective was to generate a plastic hinge near the pipe centreline, monitor the pipe bending profile and ovalization during displacement and to produce ovalization measurements which were larger than those measured during the two GSC tests. These objectives were achieved as discussed later in sub-section 3.1.2.

3.0 COMPARISON OF RESULTS

The results from the full-scale MMS pipe/soil interaction tests (MMS 01 and MMS 02R) were compared to suggested methods of analysis. A similar method was used by Paulin et al (1998) and Paulin et al (1996) to compare full-scale pipe/soil interaction test results conducted in dense sand to predicted results.

3.1 Comparison of MMS 01 and MMS 02R With Methods in the Literature

3.1.1 Review of Suggested Methods of Analysis

Committee on Gas and Liquid Fuel Lifelines (1984)

In the 1980's, the ASCE Technical Council on Lifeline Earthquake Engineering formed the Committee on Gas and Liquid Fuel Lifelines to study the effect of seismic activity on lifelines. The resulting documents (Committee on Gas and Liquid Fuel Lifelines, 1984) was intended to provide guidance for the seismic design of most major components of pipeline systems. The transverse horizontal (lateral) soil loading theory presented in the document is inferred from footing and vertical anchor plate pull out capacity theory and laboratory tests on model pipelines simulating horizontal pipeline movements, particularly Audibert and Nyman (1977) and Trautman and O'Rourke (1983).

The guideline presents, for sand, a hyperbolic p-y curve of the form

$$p = \frac{y}{A' + B'y}$$

$$A' = 0.15 y_u / p_u$$

where

$$B' = 0.85 / p_u$$

and

It is suggested the ultimate lateral soil load on the pipeline, p_u, and the displacement to ultimate load, y_u, be calculated from

$$p_u = \overline{\gamma} \; H N_{qh} \; D$$

$$y_u = 0.07 \quad 0.10(H + D/2)$$

and

$$y_u = 0.03 \quad 0.05(H + D/2)$$

for loose sand;

$$y_u = 0.02 \quad 0.03(H + D/2)$$

for medium sand; and

for dense sand where - is the effective unit weight of the soil, H is the depth to the pipe springline, N_{qh} is the horizontal bearing capacity factor and D is the external pipeline diameter. The guidelines suggest two models can be used to obtain the horizontal bearing capacity factor. The first is based on the work of Audibert and Nyman (1977), which is discussed below, who adapted Hansens (1961) model for vertical piles subjected to lateral loading and found good agreement with experimental results. The adapted curves to determine N_{qh} are presented in Figure 5. The second is based on the work of Trautman and O'Rourke (1983), which is also discussed below, who found good agreement between theory for vertical plate anchors subjected to horizontal loading (Ovesen and Stromann, 1972) and experimental results. Design curves were developed for the horizontal bearing capacity factor as presented in Figure 6. The guideline (Committee on Gas and Liquid Fuel Lifelines, 1984) cautions that horizontal bearing capacity factors based on the model of Hansen (1961) are 50-100% larger than those of the Ovesen and Stromann (1972) based model for similar burial geometry and soil properties.

Audibert and Nyman (1977)

The soil resistance against horizontal movement of a buried pipeline or conduit is generally called the coefficient of horizontal subgrade reaction, k_h (Audibert and Nyman, 1977). This coefficient is not a unique soil parameter but rather varies with contact pressure, the geometry of the interface, the type of soil, and the density of the soil. Audibert and Nyman (1977) conducted a literature review and found that there was limited information available on analytical methods to determine values of k_h for buried conduit or culverts and that methods for obtaining the coefficient are poorly understood and inaccurate. The authors point out that k_h will vary continuously along any nonlinear force-displacement curve and thus recommend that appropriate p-y curves be developed for the pipeline under consideration. The authors also state that numerical formulations proposed in the literature are inadequate for the safe design of conduits subjected to lateral soil movement.

An experimental program was conducted by the authors to determine p-y curves for a buried pipeline in sand, to reveal failure mechanisms, and to investigate the influence of such parameters as pipeline diameter, embedment ratio and soil density. The tests were conducted in a testing box which was filled with an air-dried medium sand, Carver sand. This sand has a uniformity coefficient of 2.7 and the grain size varies primarily between 3 and 0.2 mm. The sand was spread in 25 mm layers using a slumping technique to achieve a loose condition and dynamic compaction to achieve a dense sample. Three model pipeline test sections were used which had diameters of 25 mm, 60 mm, and 111 mm. Cover ratios (depth of cover/pipe diameter) investigated were: 1, 3, 6, 12, and 24; 1, 3, and 6; 1 and 2 respectively for each of the model pipelines. The ends of the pipelines were connected to a hydraulic ram outside the test box via cables which passed through the walls of the test box. Force and displacement readings were continuously monitored.

Failure mechanisms were observed through a plexiglass wall on one of the sides of the testing box. Typical failure mechanisms for shallow to intermediate and deep burial are presented in Figure 7. The authors note for shallow to intermediate burial that a front passive wedge bounded by a logarithmic spiral failure surface formed in front of the model pipeline. For deep burial conditions, a confined zone of soil flow was observed to extend 1 diameter in front of the pipeline for dense sand and 2 to 3 pipeline diameters for loose sand. The authors normalized their data with respect to the maximum

$$\overline{p} = \frac{p}{p_u}$$

soil pressure, p_u, and a displacement to maximum soil pressure, y_u, as

$$\overline{y} = \frac{y}{y_u}$$

for $p \leq p_u$ and

$$\overline{p} = \frac{\overline{y}}{0.145 + 0.855\,\overline{y}}$$

for $y \leq y_u$ and found that the trend of the normalized curves could be described by
A rearrangement of the above equations by the authors led to their recommended formulae for the analytical prediction of p-y curves

$$p = \frac{y}{A' + B'y}$$

with

$$A' = \frac{0.145\ y_u}{q_u}$$

$$B' = \frac{0.855}{q_u}$$

and

and where q_u is the predicted ultimate lateral pressure acting on the pipeline.

The authors compared the experimental ultimate lateral pressure (p_u) results with those predicted by Hansens (1961) method, found good agreement, and recommended that this method be used to estimate the ultimate lateral pressure in the above equation (Figure 5). A value for y_u equivalent to 1.5-2% of the depth of embedment was also recommended. The authors also present suggestions for the bilinear representation of the hyperbolic p-y curve.

Audibert and Nyman (1977) also conducted a single field test to assess their proposed method of developing p-y curves. The test was conducted in a natural deposit of Carver sand with a unit weight of 16.9 kN/m^3 and an estimated angle of friction of 35°. The pipeline test section was 2.36 m in length, had a diameter of 229 mm, and the springline was located 80 cm below the soil surface. The calculated ultimate load was found to be only 6% less than that measured while the calculated displacement to ultimate load approximately 10% lower than the experimental value. The p-y curve developed using the authors proposed method gave a reasonable approximation to the field curve.

Trautman and O'Rourke (1983, 1985)

Trautman and O'Rourke (1983, 1985) conducted an experimental program to investigate the lateral force-displacement response of buried pipe. A review of the literature by the authors indicated that formulations utilizing the coefficient of horizontal subgrade reaction were inadequate for the design of buried pipelines. The authors also found that various analytical models used to estimate the maximum lateral resistance of buried pipelines led to predictions differing by as much as 240%.

The goals of the experimental program were to characterize force-displacement behaviour in terms of a simple model suitable for design practice and to compare the results with other published results. The tests were varied to ascertain the effects of burial depth, soil density, pipeline diameter, and pipeline surface roughness. The tests were conducted in a test compartment measuring 1.2 m by 2.3 m in plan by 1.2 m deep. Two 1.2 m long pipeline test sections were used which had diameters of

102 mm and 324 mm and wall thicknesses of 6.4 and 9.5 mm. The pipelines were connected to a hydraulic ram outside the test box via stiff rods which passed through the walls of the test box and which were strain gauged to measure loads. The sand was placed in 25 mm layers using a spreader to achieve densities of 14.8, 16.4, and 17.7 kN/m^3. Corresponding direct shear friction angles were 31°, 36°, and 44° respectively. Thirty tests were conducted in the different sands at embedment ratios (H/D) of 1.5, 3.5, 5.5, 8, and 11. To determine the effects of pipeline roughness, a pipeline was covered with sandpaper for two of the tests and covered with plastic film coated with oil for two other tests.

$$N_h = \frac{F_m}{\gamma HDL}$$

The test results are presented in Figure 8 as nondimensional force, N_h, which is defined as
versus embedment ratio. In the above equation, F_m is the maximum measured force, γ is the soil unit weight, H is the depth to the pipe springline, D is the pipe diameter, and L is the length of the pipeline segment. The displacement at F_m is termed the dimensionless displacement, Y_f. For dense sand the peak force could be easily identified but for loose sand Y_f was taken as the point where the force-displacement curve became linear. The displacements to maximum force were found to vary with soil density and were 0.13 H, 0.08 H, and 0.03 H respectively for the loose, medium, and dense sand. The pipeline surface roughness was found to have little effect on soil response; N_h for the rough pipeline was only 10% greater than from the smooth pipeline. Soil density was found to have significant impact as shown in Figure 8. The N_h values for the larger diameter pipe were found on average to be only 8% greater than the smaller diameter pipe in loose sand and only 1% higher in the dense sand. The authors conclude from this that the data can be extrapolated to pipeline diameters greater than 324 mm.

A comparison of the test results was made with the analytical models of Hansen (1961), Ovesen (1964), Neely *et al.* (1973), and Rowe and Davis (1982b). The comparison indicated the best agreement was with the models of Ovesen (1964) and Rowe and Davis (1982b) while the models of Hansen (1961) and Neely *et al.* (1973) overpredicted the measured forces by 150-200%. Because of the close correspondence of results with Ovesen (1964), the authors developed the plot of Figure 6 based on the work of Ovesen (1964) and proposed that this chart could be used in pipeline design. The authors further analysed their experimental data and found that there was a hyperbolic relationship between normalized force (F") and

$$F'' = \frac{Y''}{0.17 + 0.83\ Y''}$$

normalized displacement (Y") which could be expressed as

$$F'' = (F/\gamma HDL)/N_h$$

in which
and

$$Y'' = (Y/D)/(Y_f/D)$$

and where F is the force measured at each increment of displacement, Y. If a bilinear relationship is needed for the force displacement curve, the authors suggest that Figure 6 can be used to determine the maximum lateral force and the elastic horizontal soil stiffness should be taken as a secant slope defined at 70% of the maximum force. This is given by the following expression:

$$K_{h70} = C_k \; N_h \; \gamma \, D L$$

where K_{h70} is the secant slope of the bilinear relationship defined at 70% of the maximum force.

Rowe and Davis (1982)

Rowe and Davis (1982) conducted a theoretical investigation into the behaviour of horizontally and vertically orientated anchor plates in sand. The study was conducted using an elasto-plastic finite element analysis and assumed the anchor was thin and perfectly rigid. The anchor was considered to be of height B, and buried to a depth h to its base. The anchor was also considered to be an infinite strip to simulate plane-strain conditions. The soil was assumed to have a Mohr-Coulomb failure criterion and either an associated or non-associated flow rule. As part of the analysis, the effects of anchor embedment depth, soil angle of friction, soil dilatancy, initial stress state of the soil and anchor roughness were examined.

Rowe and Davis (1982) suggest the average applied pressure, q_u, required to fail a vertically orientated anchor in sand can be

$$q_u = \gamma \, h \, F_\gamma'$$

expressed by

where γ is the unit weight of the soil above the anchor, h is the depth of embedment, (the depth to the bottom of the anchor) and F_γ' is an anchor capacity factor which is a function of embedment ratio, anchor roughness, soil angle of friction, soil

$$F_\gamma' = F_\gamma \; R_\varphi \; R_R \; R_K$$

dilatancy, and initial stress state. F_γ' is expressed approximately as

where F_γ is the anchor capacity factor for the basic case of a smooth anchor resting in a soil with a coefficient of earth pressure (K_o) at rest equal to 1 and which deforms plastically at constant volume (Ψ = 0); R_Ψ, R_R, and R_K are correction factors for the effect of soil dilatancy, anchor roughness and initial stress state respectively. The variation in F_γ with soil friction angle, φ, is presented in Figure 9. Details on the calculation of the correction factors presented in Equation [19] are given by Rowe and

Davis (1982).

The authors conclude that the theoretical results show encouraging comparison with results from their model tests as well as other model and field test data. They also conclude that soil dilatancy and anchor roughness significantly affect certain cases of anchor capacity but the effect of the coefficient of earth pressure at rest is typically small.

3.1.2 Comparison of Results

Values published in the literature for the three estimation methods discussed above generally are only available for a friction angle up to 45°; data have been extrapolated to 53° for comparison of force-displacement curves with the dense sand results. Predicted distances to ultimate load (as defined above) and predicted ultimate loads are given in Table 3.

Table 3 Predicted Peak Loads and Displacements to Peak Load

Measurement / Prediction	**Test**	**Displacement to Peak Load (mm)**	**Peak Load (kN)**
Measurement	MMS 01	11.3	97.8
Audibert & Nyman (1977)		12.2	281
Trautman & O'Roarke (1983)		15.3	76
Rowe and Davis (1982)		15.3	79
Measurement	MMS 02R	5.5	37.0
Audibert & Nyman (1977)		12.2	122
Trautman &O'Roarke (1983)		15.3	33
Rowe and Davis (1982)		15.3	34

The displacements predicted to peak load are the same for both MMS 01 and MMS 02R as they depend only on the embedment depth and the pipe diameter (i.e. geometry). These geometric parameters were practically the same for the two tests.

The measured load-displacement curves are compared in Figure 10. These curves are individually compared in Figure 11 with the relationships predicted by the three different analyses. The distance to peak load for the Rowe and Davis construction has been taken as the Committee on Gas and Liquid Fuel Lifelines (1984) recommendation. The Trautman and

O'Rourke (1983) and Rowe and Davis (1982) suggested analysis methods under predicts the peak experimental loads. The theoretical force-displacement curve based on the Rowe and Davis (1982) method of analysis provides the closest values to the peaks of the experimental data. The Audibert and Nyman (1977) construction over predicts the peak load from MMS 01 by more than 288% and 330% for MMS 02R. It should be pointed out that data have been extrapolated off of published data charts to obtain these predictions. The good agreement between the experimental dense sand results and the Rowe and Davis (1982) predicted results is also consistent with the findings of Paulin et al (1998). All comparisons between the MMS experimental data and suggested methods of analysis are consistent with Paulin et al (1996).

The experimental load-displacement plots are characterised by significant differences in peak loads and displacement to peak load and by the post-peak response. The difference in peak load between tests is mainly due to the submerged testbed conditions in test MMS 02R as explained below. The different values of pipe displacement to peak load is due to the different effective stress levels in the two tests. This is illustrated in Figure 12 which presents results from a series of direct shear box tests on the Concrete Products sand. It can be seen that as the vertical effective stress increases, the displacement required to reach maximum shear stress also increases. This dependancy is not included in the analyses considered.

The difference in post peak response is attributed to the different test bed densities. The MMS 01 testbed can be classified as dense while the MMS 02R testbed, with a somewhat lower density, can possibly be classified as medium dense.

The measured loads have been normalised using equation 4 to obtain a mobilised horizontal bearing capacity factor, N_{qh}. These factors were calculated for both tests and are compared in Figure 13. There is reasonably good agreement between both tests with the maximum value of N_{qh} ranging between 15 and 16. The difference in peak load is then attributed to the different effective stress levels at which the tests were conducted. The N values are approximately equal to the values of N_{qh} determined from Figures 6 and 8.

3.2 Comparison of MMS 03 With Previous Pipe Bending Tests

A comparison was made among test MMS 03 and the two pipe bending tests conducted for the Geological Survey of Canada (GSC 01 and GSC 02). A complete description of the GSC tests was provided in Hurley et al (1998a,b) and Konuk et al (1999).

The objective of the first GSC test was to develop a plastic hinge at the pipe centreline and to monitor the pipe bending profile and ovalization during displacement through a dense sand testbed. A plastic hinge was developed during GSC 01 but hinge formation occurred at about 1.3 m from each end of the pipe and away from most of the instrumentation. Even though the test was only partially successful, a great deal of information was learned and this information was used, along with an FE analysis, to determine optimum pipe burial depth for test GSC 02. Test GSC 02 resulted in pipe deformation at the centreline but a plastic hinge did not completely develop. This was due to the fact that the pipe translated through the soil after about 50 mm displacement. Pipe ovalizations were limited to about 0.3% in GSC 01 and about 1.3% in GSC 02.

Test MMS 03 was a lateral pipe bending test similar to the two previous tests. Prior to constructing the testbed, an FE analysis was performed to determine the optimum pipe burial depth. The actual pipe yield strength, determined from coupon tests of the pipe material, was used as input to the FE model. The pipe had to be buried at a sufficient depth to prevent translation during end displacement but to allow formation of a plastic hinge near the pipe centreline.

Figure 14 presents a comparison between pipe ovalizations and bend sensor bending strains from MMS 03 and both GSC tests. The BS 8010 prediction of pipe ovalization due to lateral bending is also included on the figure for comparison. The ovalization level of 3% recorded during MMS 03 exceeded previous GSC measurements. The ovalisation response is consistent with those previously measured. These measured responses are consistently different to that predicted from lateral bending, such as BS8010.

The observed response was attributed by Konuk et al (1999) to the effect of differential soil pressures acting on the pipe. They proposed the use of the modified Iowa formula to predict the response. This finding was confirmed in a recent finite element study for GSC by Popescu (1999).

Some pipeline design codes, such as CSA Z662-96 & DNV '81, limit pipe ovalisation to about 3%, if effects such as reduction in moment capacity, geometrical restrictions and cyclic stresses caused by ovalisation have not been considered. In the MMS 03 test, the measured ovalisation level of 3% reached this recommended design level. Ovalisation of buried pipe due to differential soil pressures may therefore need to be considered in some applications as a design load condition.

4.0 ELECTRONIC DATA

Electronic data from the Full-Scale Pipe/Soil Interaction project have been provided on CD-ROM with this report. The CD contains a directory for each of the four tests as well as a directory containing the final report. Each test directory contains the text of the progress report (in WordPerfect format), experimental data in Excel format (eng. units) and report tables in either WordPerfect or Excel format. Photos from each of the four progress reports are included on the CD as well. Software such as Corel Photopaint or Photoshop can be used to view the photos. Table 4 lists the file names and a short description of each file on the CD.

Table 4 CD Contents

Test/Directory	Filename	Description
MMS 01 / docs	mms01.wpd	MMS 01 Data Report
MMS 01 /exp_dat	mms01_1.xls, mms01_2.xls	MMS 01 Electronic Data
MMS 01 / tables	table1.wpd	Geot. Properties of Testbed Sand
MMS 01 / tables	table2.xls	Density and Moisture Content Measurements
MMS 01 / tables	table3.wpd	Testbed Survey Data
MMS 01 / tables	table4.wpd	Pre-Test VDT Locations
MMS 01 / tables	table5.wpd	Pre-Test IDM Locations
MMS 01 / tables	table6.wpd	Post-Test VDT Locations
MMS 01 / tables	table7.wpd	Post-Test IDM Locations
MMS 01 / photos	fig3.tif	Figure 3 from Progress Report
MMS 01 / photos	fig4.tif	Figure 4 from Progress Report
MMS 01 / photos	fig5.tif	Figure 5 from Progress Report
MMS 01 / photos	fig10.tif	Figure 10 from Progress Report

MMS 01 / photos	fig21.tif	Figure 21 from Progress Report
MMS 02 / docs	mms02.wpd	MMS 02 Data Report
MMS 02 / exp_dat	mms02_1.xls, mms02_2.xls	MMS 02 Electronic Data
MMS 02 / tables	table1.wpd	Geot. Properties of Testbed Sand
MMS 02 / tables	table2.xls	Density and Moisture Content Measurements
MMS 02 / tables	table3.wpd	Testbed Survey Data
MMS 02 / tables	table4.wpd	Pre-Test VDT Locations
MMS 02 / tables	table5.wpd	Pre-Test IDM Locations
MMS 02 / tables	table6.wpd	Post-Test VDT Locations
MMS 02 / tables	table7.wpd	Post-Test IDM Locations
MMS 02 / photos	fig4.tif	Figure 4 from Progress Report
MMS 02 / photos	fig9.tif	Figure 9 from Progress Report
MMS 02 / photos	fig18.tif	Figure 18 from Progress Report
MMS 02 / photos	fig21.tif	Figure 21 from Progress Report
MMS 02r / docs	mms02r.wpd	MMS 02r Data Report
MMS02r / tables	table1.wpd	Geot. Properties of Testbed Sand
MMS02r / tables	table2.xls	Density and Moisture Content Measurements
MMS02r / tables	table3.wpd	Testbed Survey Data
MMS02r / tables	table4.wpd	Pre-Test VDT Locations
MMS02r / tables	table5.wpd	Pre-Test IDM Locations
MMS02r / tables	table6.wpd	Post-Test VDT Locations
MMS02r / tables	table7.wpd	Post-Test IDM Locations
MMS 02r / photos	fig3.tif	Figure 3 from Progress Report
MMS 02r / photos	fig8.tif	Figure 8 from Progress Report
MMS 02r / photos	fig17.tif	Figure 17 from Progress Report
MMS 02r / photos	fig20.tif	Figure 20 from Progress Report
MMS 03 / docs	mms03.wpd	MMS 03 Data Report

MMS 03 / exp_dat	mms03.xls	MMS 03 Electronic Data
MMS 03 / tables	table1.wpd	Geot. Properties of Testbed Sand
MMS 03 / tables	table2.xls	Density and Moisture Content Measurements
MMS 03 / tables	table3.wpd	Testbed Survey Data
MMS 03 / tables	table4.wpd	Pre-Test VDT Locations
MMS 03 / tables	table5.wpd	Post-Test VDT Locations
MMS 03 / tables	table6.wpd	Post-Test Pipe Ovalizations
MMS 03 / photos	fig4.tif	Figure 4 from Progress Report
MMS 03 / photos	fig8.tif	Figure 8 from Progress Report
MMS 03 / photos	fig24.tif	Figure 24 from Progress Report
MMS 03 / photos	fig26.tif	Figure 26 from Progress Report
final	MMS_final.wpd	MMS Final Report

5.0 SUMMARY

A series of four full-scale pipe/soil interaction tests have been conducted for the Minerals Management Service of the U.S. Department of the Interior. The first test was conducted to obtain the force-displacement response of a pipe buried in a testbed of moist (wc = 3-4%) dense sand. This test was completely successful and resulted in a maximum average peak end load of 48.9 kN. Displacement to peak load was about 11 mm.

The second test in the experimental program was conducted to ascertain the effect of a submerged testbed on the force displacement response of the buried pipe. The second test was only partially successful due to excessive frictional forces at the pipe ends. This test was eventually repeated and it was found that the submerged conditions reduced the maximum average peak end loads by about 62%. Displacement to peak load was reduced by about 50%. The lower peak force can be attributed to the submerged testbed conditions while the lower displacement to peak force may be due to lower effective stress levels in the submerged test.

This was the first submerged full-scale pipe/soil interaction test conducted in C-CORE's full-scale pipeline/soil interaction test facility. This type of test closely simulates offshore conditions and there is a need for more research in this area, possibly with varying pipe diameters, cover depths and displacement rates, so that the complete effect of the submerged testbed conditions on the force-displacement response of a buried pipe is fully understood.

The primary objective of the final test was to produce a plastic hinge near the pipe centreline while monitoring pipe ovalization, end loads, end displacement and the pipe bending profile. An FE analysis was used to determine the optimum pipe burial depth. Measured pipe ovalizations were then compared to previous pipe bending tests. A plastic hinge was produced in the pipe about 200 mm from the pipe centreline. Maximum pipe ovalizations were on the order of 3%. This amount of pipe ovalization is an order of magnitude higher than the GSC 01 ovalizations, more than twice the measured ovalizations from GSC 02 and closer to ovalization limits used for pipeline design.

5.1 Recommendations for Further Work

The significant effect of submerged testbed conditions on the load-displacement response of a buried pipe was illustrated in a comparison of tests MMS 01 and MMS 02R. To improve the understanding of the effects of submerged conditions, it is recommended that the MMS support further pipe/soil interaction numerical and physical model studies. Parameters to be investigated should include the effect of different soil types and cover to pipe diameter ratios.

Significant pipe ovalisation may be caused in buried pipe systems by differential soil pressures. The effect of internal pipe pressure in mitigating this ovalisation should be investigated.

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Appendix A

Figures

List of Figures