03 Jun 1988, 10:00 am - 5:30 pm

GBS Platform Evaluation Using Field Instrumentation

Joseph M. Keaveny
Norwegian Geotechnical Institute, Oslo, Norway

Per Magne Aas
Norwegian Geotechnical Institute, Oslo, Norway

Farrokh Nadim
Norwegian Geotechnical Institute, Oslo, Norway

Proceedings: Second International Conference on Case Histories in Geotechnical Engineering, June 1-5, 1988, St. Louis, Mo., Paper No. 6.29

GBS Platform Evaluation Using Field Instrumentation

Joseph M. Keaveny
Senior Engineer, Norwegian Geotechnical Institute, Oslo, Norway

Farrokh Nadim
Senior Engineer, Norwegian Geotechnical Institute, Oslo, Norway

Per Magne Aas
Senior Engineer, Norwegian Geotechnical Institute, Oslo, Norway

SYNOPSIS: A case history of the foundation behaviour of an offshore gravity base structure (GBS) is presented. The platform rests on an overconsolidated fissured clay, bounded, top and bottom, by pervious sand layers. Sixteen piezometers have been placed within this 30 m layer. Based on one-dimensional consolidation theory, independent analyses using both settlement and pore pressure measurements indicated a high degree of consolidation had occurred much sooner than was estimated in the initial design phase. These analyses indicated that laboratory oedometer tests underpredicted the coefficient of consolidation by one to two orders of magnitude. Updated settlements and stability analyses yielded 50% of the initially anticipated settlement and a 20% increase in the available safety factor. In addition, the certainty that the theory relating pore pressure to settlements was appropriate, led to confidence in the piezometer performance, and in turn the procedure used to install them.

INTRODUCTION

Following the installation of Gullfaks A platform in May 1986, the Norwegian State Oil Company (Statoil) asked the Norwegian Geotechnical Institute (NGI) to perform a verification of the initial design based on accrued measurements after installation. The design verification was performed in cooperation with Norwegian Contractors (NC). This paper presents partial results of that work.

Analysis of measured pore pressures and settlements at the Gullfaks A platform were performed in order to verify the initial design settlement and stability analyses. The platform is a concrete gravity base structure (GBS) located in the North Sea. A schematic diagram of the platform is shown on Fig. 1, and key data concerning it are given in Table 1. The soil conditions and piezometer locations are summarized on Fig. 2.

Table 1. Key figures concerning platform

Foundation area, m^2	11 000
Embedment, m	1
Depth of water at site, m	134
Submerged weight, MN	3 600
Horizontal force at mudline, MN	865
Overturning moment at mudline, MNm	35 930
Load coefficient to be applied	1.3

The upper 3 to 4 m consists of a layer of very dense gravel and sand with clay overlying very stiff fissured clay to a depth of approximately 13 m. Below 13 m exists a very stiff clay. Of special note is the soil at a depth of approximately 34 m which is clay exhibiting pockets and seams of fine sand. This layer was found at all the boring locations at the site. The clay layer bracketed by the two drainage layers at 3 and 34 m is the layer in which the piezometers are embedded. Because of the large width of the platform relative to the thickness of this layer, one-dimensional consolidation theory (Terzaghi, 1943) and extensions of it (Taylor, 1948; Scott, 1963) were used in evaluating the measured settlements and piezometer pore pressure response due to platform loading.

EQUIPMENT DESCRIPTION AND INSTALLATION PROCEDURE

Two piezometer strings are installed in the soil beneath the Gullfaks A platform (Fig. 2). One is a plain piezometer string installation (PP1) with two vibrating wire transducers implanted at five different levels. The other installation is a combination of a piezometer assembly, similar to the first one (PP2), and a long term settlement measuring equipment for measurement of platform settlement relative to a fixed point 75 m below seabed.

Both installations have been made in predrilled and stabilized boreholes. The boreholes were sealed after installation with a cement-tixoton grout.

EVALUATION OF DEGREE OF CONSOLIDATION

The method used to evaluate the average degree of consolidation in the clay layers beneath the platform can be summarized as follows:

1. Determine the consolidation settlement as a function of time for the load time history, by plotting measurements and subtracting out the initial (immediate) deformations.

2. Construct an equivalent consolidation settlement as a function of time curve for instantaneous loading using a method first described by Taylor (1948).

3. Estimate the coefficient of consolidation, C_v, and the average degree of consolidation at any time using above constructed curve and one-dimensional consolidation theory.

4. Estimate the average degree of consolidation at any time using measured pore pressures and one-dimensional consolidation theory.

5. Compare values determined in Items 3. and 4. The average degree of consolidation in both cases should be similar to each other.

Consolidation settlement-time relationship

The load-time curve is shown on Fig. 3. Full ballasting occurred within the period 25-30 May 1986. Total settlement as a function of time up to 7 Sept. 1986 is shown on Fig. 4. During periods of loading relatively large settlements occurred that can be directly attributed to initial settlement. Using the settlement data from Fig. 4 and subtracting the initial settlement (about 100 mm) an actual consolidation settlement versus log-time curve was constructed and is shown on Fig. 5. This consolidation-time relationship is a function of an approximately linearly increasing load with time during the period from 11 May to 29 May 1986. By a graphical technique first proposed by Taylor (1948), an equivalent instantaneous loading consolidation settlement-time curve can for 11 May be constructed (Fig. 6).

An empirical value of the coefficient of consolidation, C_v, was determined using the log time and square root of time methods (Lambe, 1951) on the consolidation settlement-time data derived for instantaneous loading.

From this value a value of the time factor, T, was estimated using the one-dimensional consolidation theory, where

$$T = \frac{C_v\, t}{H^2} \qquad (1)$$

and t = time after instantaneous loading (taken from 11 May 1986)
H = drainage height ≈ 15.5 m
C_v = coefficient of consolidation.

Pore pressure-time relationship

A second way of estimating the time factor, T, was to use the measured pore pressures to determine the consolidation ratio $U_z = 1-\Delta u/\Delta u_i$, where Δu is the excess pore pressure at a given depth and time and Δu_i is the initial excess pore pressure due to the platform weight (Fig. 7, for 8 June 1986). From this plot, the time factor, T, was estimated directly.

Results and discussion concerning degree of consolidation

Once T is known, an estimate of the average degree of consolidation, U, can be made using Fig. 8. A summary of the estimated C_v, T and U values using the various methods is given on Table 2.

Table 2. Coefficient of consolidation, C_v, time factor, T, and average degree of consolidaiton, U, determined from settlement and pore pressure readings.

Method	Coefficient of consolidation (m^2/year)	Time factor T	Average degree of consolidation U, %	Comment
Settlement-log time	Unable to determine	-	-	Instantaneous load settlement curve.
Settlement-square root of time	4236	1.36	>90	Instantaneous load-settlement curve. T and U taken at 8 June 1986.
Measured pore pressures	1184	0.38	68	- From 11 May to 8 June 1986 - Taking lower bound U_z values
Measured pore pressures	150	0.35	65	- From 11 May to 2 December - Taking lower bound U_z values
Measured pore pressures	1846	0.59	81	- From 11 May to 8 June 1986 - Taking average U_z values
Measured pore pressures	2512	0.81	90	- From 11 May to 8 June 1986 - Taking upper bound U_z values.

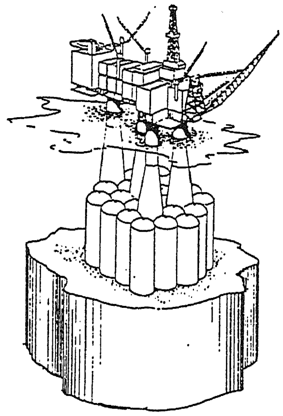


Fig. 1 General View of the Gullfaks A Structure.

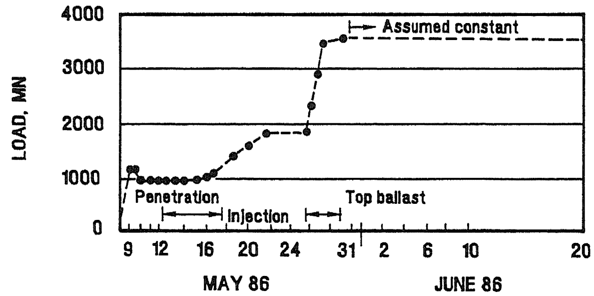
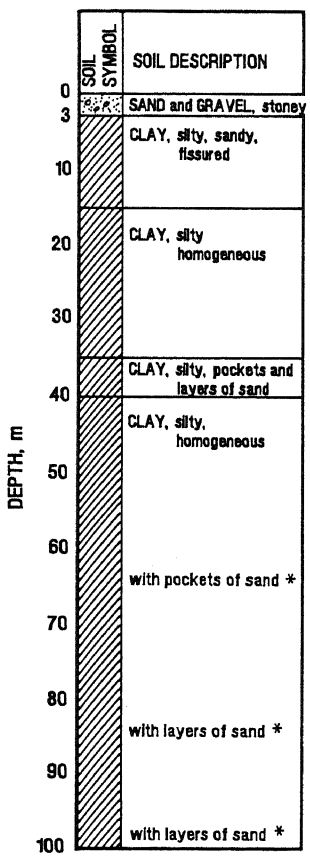
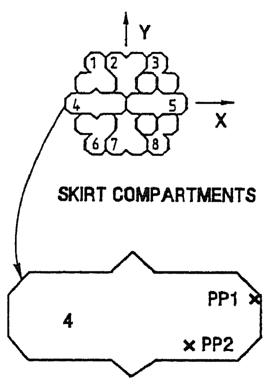


Fig. 3 Load-time Relationship





DEPTH, m	PIEZOMETER PP1 HYD.	PP1 ELEC.	STRING PP2 HYD.	PP2 ELEC.
3	H11	PP11A	H21	PP21A
6	H12	PP12A	H22	PP22A
9	H13	PP13A	H23	PP23A
12	H14	PP14A	H24	PP24A
16	H15	PP15A	H25	PP25A

* Based on one test boring

Fig. 2 Simplified Soil Profile and Piezometer Locations.

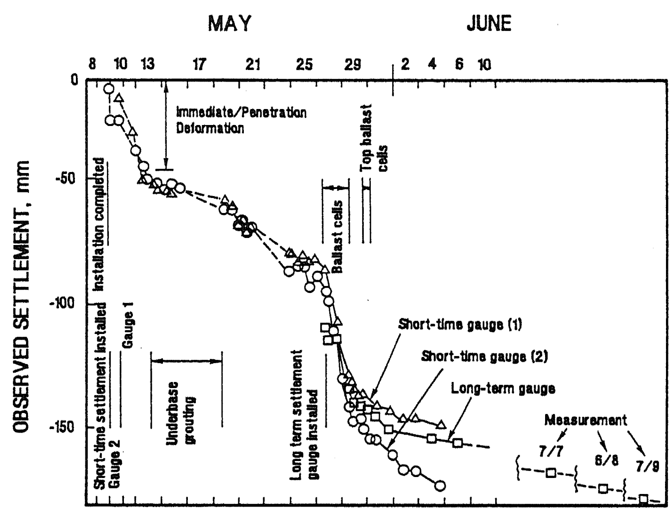


Fig. 4 Measured Settlement versus Time.

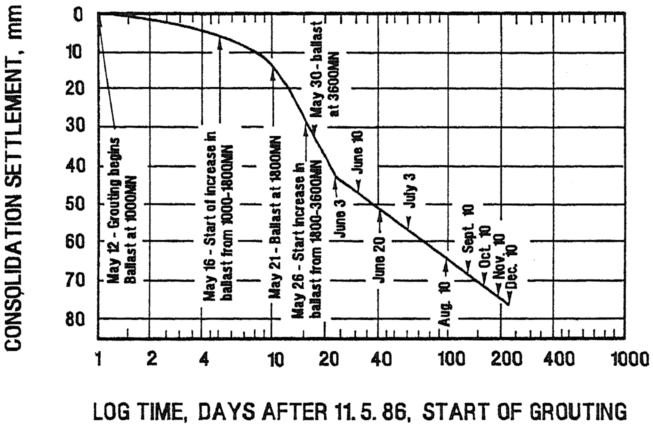
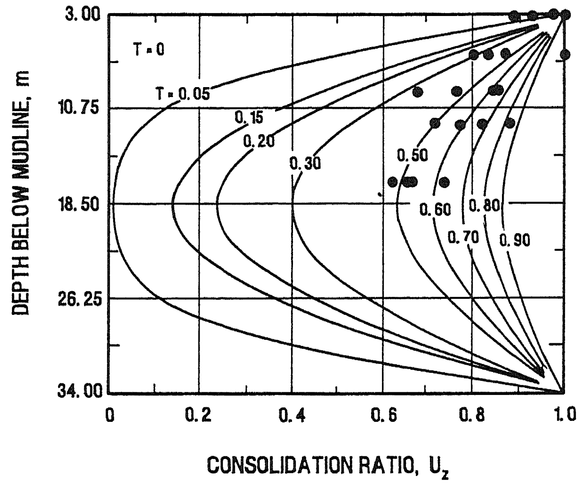


Fig. 5 Consolidation Settlement versus Log-time.



● Based on measured pore pressure

Fig. 7 Consolidation Ratio, U_z, Based on Measured Pore Pressure Values on June 8, 1986.

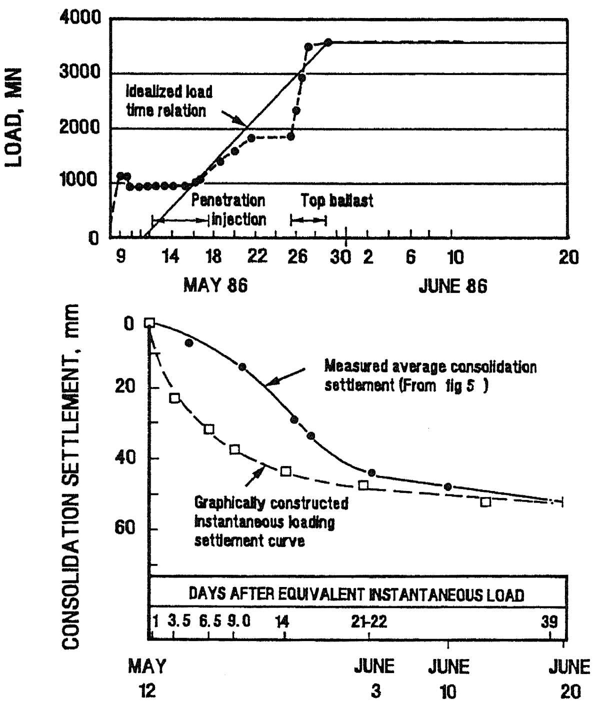
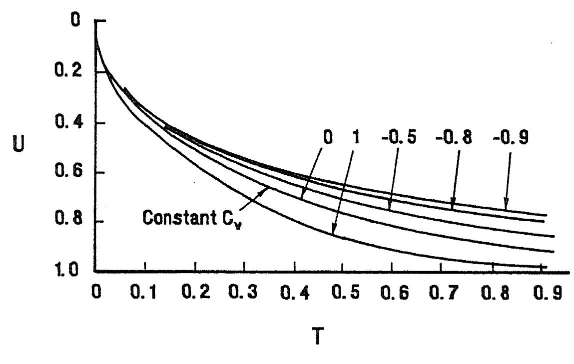


Fig. 6 Instantaneous Loading consolidation Settlement versus Time using Taylor (1948) Method.



NOTE: C_v varies linearly with effective stress

$C_v = C_{vo}(1 + aU)$ values of 'a' shown on curves.

Fig. 8 Average Consolidation, U, with Varying Coefficient of Consolidation Obtained by Numerical Analysis (Scott, 1963)

In general the values of C_v appear to range between one and two orders of magnitude greater than the value of 30 m²/year used for the initial design of the platform. Reasons for this discrepancy can be explained as follows. The initial design value was based on lower bound laboratory oedometer results that ranged up to 150 m²/year. In a fissured clay the in situ permeability can easily be one to two orders of magnitude higher since the soil sample tested in the laboratory will be relatively intact compared to the overall fabric beneath the platform. Since

$$C_v = \frac{k\ M}{\gamma_w} \qquad (2)$$

where

k = permeability
M = constrained modulus
γ_w = unit weight of water

then, a two order increase in k would directly influence C_v. In addition, based on settlement measurements, the actual constrained modulus in the field appears to be a factor of approximately 4 greater than the laboratory determined value. This again would have the effect of increasing the actual value of C_v.

It could be argued that an overestimate of the drainage height, H, has led to excessively high field estimates of C_v, using Eq. (1) and measured pore pressure determined T values. The drainage height would have to be a factor of 10 less in order to have field and laboratory C_v values coincide, and this is not indicated by the field test borings.

One important note is that both the settlement readings and the pore pressure readings indicate a high degree of consolidation had taken place by as early as 8 June 1986. Further testament to the fact that a high degree of consolidation has taken place is that the slope of the vertical strain-log time plot, called the coefficient of secondary compression, C_α, compares favourably with measured values for similar soils. This is shown on Table 3.

Table 3. Coefficients of secondary compression.

Method	C_α (%)
• Backcalculated from settlement measurements assuming layer thickness = 100 m	0.1
• Predicted from empirical relationships	
- water content as basis (Navdocks, DM-7, 1961)	0.2–0.4
- void ratio as basis (Kapp et al., 1966)	0.2–0.5
- typical values recommended by Lambe and Whitman, 1969, for clays with OCR > 2	<0.1

This relatively slow rate of compression implies that either primary consolidation has completed or that the rate of excess pore pressure dissipation in the later stages of consolidation is very slow.

EVALUATION OF PIEZOMETER PERFORMANCE

Since pore pressure measurements were used in one of the methods for determining the degree of consolidation, an analysis of piezometer performance at Gullfaks A was performed by comparing predicted pore pressure response with those actually measured by piezometers below the platform. One-dimensional consolidation theory assuming a layer from depth 3 m - 34 m with double drainage was used as the prediction model. The following conclusions can be made:

1. The high degree of consolidation as determined by piezometer-measured pore pressures and one-dimensional consolidation theory agrees well with the degree of consolidation as determined by settlement readings alone, or in conjunction with one-dimensional consolidation theory. A high degree of consolidation is also indicated by the current slope of the measured vertical strain versus log time plot.

2. One half year after the end of loading (December 1986) the pore pressures indicate that the layer from 3 m - 34 m has reached approximately 80% consolidation. The pore pressures have not dissipated appreciably since June 1986. This can be explained in the following way. As the layer consolidates the coefficient of consolidation decreases. This is indicated by comparing the values of C_v in laboratory oedometer tests at in situ stresses and at stresses associated with full consolidation. By computing the parameter a

$$a = \frac{\left(\frac{C_v}{C_{vo}} - 1\right)}{U} \qquad (3)$$

where C_v = coefficient of consolidation at effective stress associated with an average consolidation ratio, U.
C_{vo} = coefficient of consolidation at in situ vertical effective stress prior to platform loading (Note: Values associated with the reload cycle were used).
U = 75%.

The average value for the clay layer was $a = -0.6$. This decreasing C_v with increasing vertical effective stress causes the U-T curve to shift upward (Fig. 8). Thus the more consolidated the layer becomes the longer it takes to dissipate the remaining excess pore pressures. As can be seen, theoretically for values of T of approximately 0.9 (i.e. about the value predicted after June) the average consolidation ratio approaches a value of about 80%. The pore pressures measured during the last half of 1986 indicate a value of $U \approx 80\%$.

3. Using the extended one-dimensional consolidation theory (Scott, 1963) for varying C_v, and an initial coefficient of consolidation as determined by the settlement - square root of time method using the instantaneous load-settlement curve (i.e., C_v = 4236 m²/year) yields predicted pore pressure as a function of time values similar to those measured (Fig. 9) and in particular after 1 June 1986. There is, however, a general overprediction in peak response during the brief period of topside loading. The ratio of measured to predicted excess pore pressures during this time ranges from 0.3 at 3 m depth to 0.7 at a depth of 16 m. This indicates a possible decrease in permeability of the grout seal with depth, and a partial hydraulic connection between, at least, the upper piezometer and top sand layer. In addition, just before the measured pore pressure response starts increasing (point A', Fig. 9), the pore pressures are decreasing. Point A' corresponds in time to the end of the grouting operation. This indicates that the grout might temporarily be causing the initial low response.

4. The time it takes to reach the peak measured response (horizontal distance between Points A' and B') corresponds to the time it took to load the platform from 1800 to 3600 MN (Fig. 6). Theoretically, substantial pore pressure dissipation can occur in that amount of time (vertical distance between Points B and D on Fig. 9). In light of this, comparing the vertical distance between A and D on the predicted curves and A' and B' on the measured curves yields a favourable result.

On 5 August 1986 the water pressure in skirt compartment 4 was reduced by 30 kPa. It was assumed that the boundary conditions change only at the top of the clay layer (3.0 m depth). This is to say that the piezometric level in the sand layer above the clay is lowered by 30 kPa, while the piezometric level at depth 34 m remains unchanged. This condition sets up a triangular negative excess pore pressure. Again using one-dimensional consolidation theory, Fig. 10 shows the theoretical relationship between consolidation ratio U_z versus depth relationship as a function of time factor, T. The magnitude of the decrease in pore pressure at any depth or time is estimated by multiplying ΔU_z, at an appropriate depth and time factor, by 30 kPa. For a $C_v \approx 2000$ m²/year (since by 5 August the C_v has decreased due to increased consolidation) and a time t of 0.01 years (5 August to 9 August) the computed time factor T is 0.085. Table 4 compares predicted versus measured responses. The measured response is in general greater than predicted at greater depths indicating a higher coefficient of consolidation than assumed.

Table 4. Comparison of predicted versus measured pore pressure response due to skirt compartment suction pressure

Depth (m)	Average measured decrease in pore pressure (kPa)	Predicted decrease in pore pressure (kPa)
3	28	30
6	23	19
9	25	9
12	17	5
16	12	2

In conclusion it appears that the piezometers at Gullfaks A are functioning properly, but that some may have partial hydraulic connection with the upper sand layer.

EVALUATION OF PLATFORM STABILITY

Based on the analysis presented, it was conservatively assumed that at 2 December 1986 the clay layer between the depths of 3 m and 34 m below seabed exhibited an average degree of consolidation, U = 65%. This value of U corresponds to an approximate lower bound value based on empirical methods of determining the coefficient of consolidation and subsequent value of U outlined above. Assuming a low value of U is conservative in that the undrained strength of the clay layer increases with increased U. Assuming that the platform load occurred instantaneously around 11 May 1986, then an average coefficient of consolidation, C_v, of about 150 m²/year is indicated.

This value of C_v, which is approximately equal to the upper bound of laboratory measured values (but still an order of magnitude less than what is in the field), was used to calculate the degree of consolidation at various times after installation.

Strength increase due to consolidation

In situ undrained shear strength as would be measured in a laboratory triaxial compression test, CU_A, as a function of time and depth is shown on Fig. 11. Similar relationships for triaxial extension and direct simple shear undrained shear strengths were also developed in that the bearing capacity method used (Lauritzsen and Schjetne, 1976; Kvitrud, To and Lauritzsen, 1985) utilize a weighted average of these strengths depending on the depth and shape of the assumed failure surface. In situ stresses beneath the platform, as a function of depth and time, increase due to the consolidation process. In situ strength, as a function of depth and time, can be established by relating it to the laboratory strength determined under the same effective confining stresses.

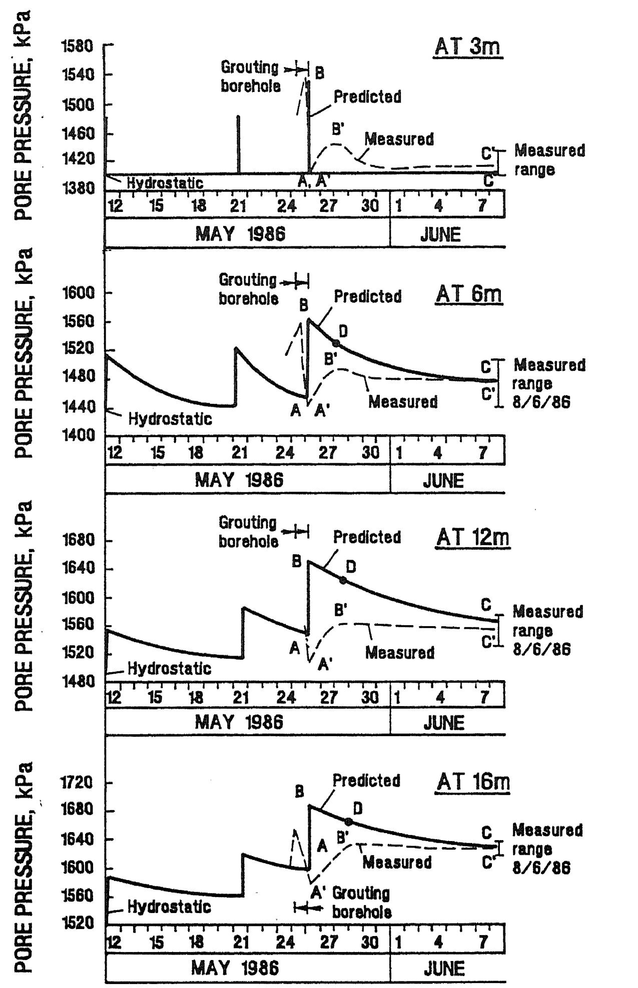


Fig. 9 Predicted Versus Measured Pore Pressure Response - Three Load Pulse Assumption. C_v Varies from 4236 m^2/Year (May 11) to 2329 m^2/Year (June 8) According to Increase in Consolidation.

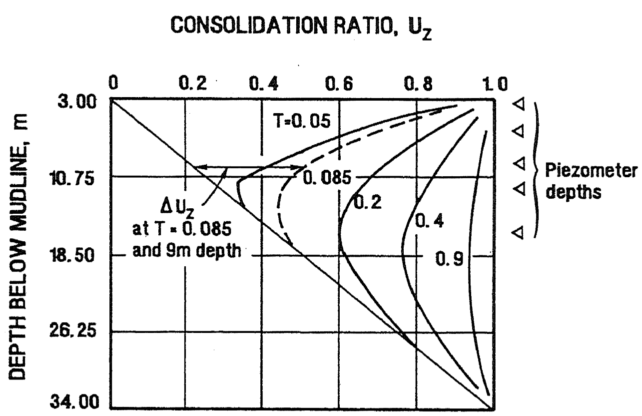


Fig. 10 Consolidation Ratio, U_z, as a Function of Depth and Time Factor, T, due to Application of Skirt Water Pressure at Depth 3 m.

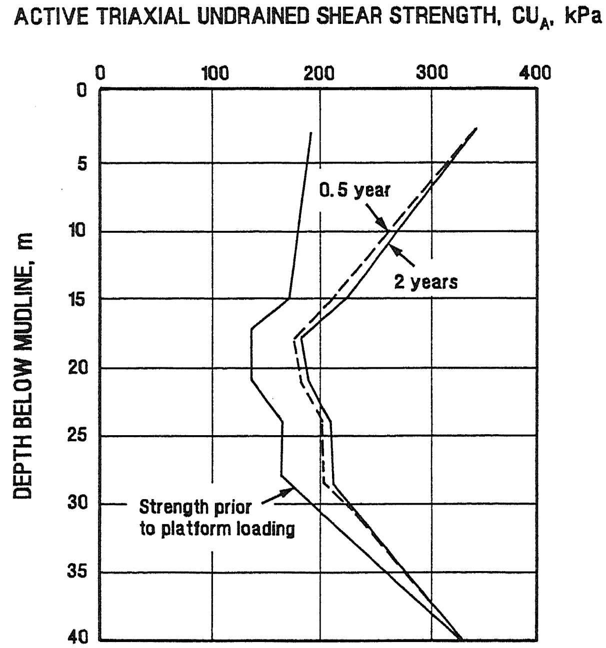


Fig. 11 Active Triaxial Undrained Shear Strength Profile as a Function of Time after Installation.

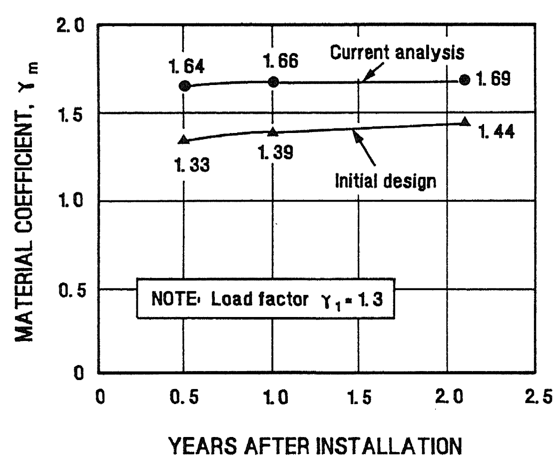


Fig. 12 Increase in Material Coefficient as a Function of Time after Installation.

Results of stability analysis

The strength profiles at various times were used in a bearing capacity analysis using the slip surface method developed at NGI (Lauritzsen and Schjetne, 1976; Kvitrud, To and Lauritzsen, 1985). Environmental loads are multiplied by a load coefficient in accordance with the regulations given by the Norwegian Petroleum Directorate (1985). Horizontal force equilibrium is evaluated with the ratio of the overall horizontal component of soil resistance to the overall horizontal component of the factored loads being the material coefficient, γ_m. The material coefficient as a function of years after installation is shown on Fig. 12. As can be seen, for a load coefficient, $\gamma_L = 1.3$, the material coefficient ranges from 1.64 to 1.69 within a two year period after installation. This compares favourably to the range of 1.35 to 1.44 estimated in the design phase.

CONCLUSIONS

The main conclusions of the study are:

1. Both settlement and pore pressure measurements indicate that the layer down to a depth of 34 m beneath the platform has been subjected to a high degree of consolidation. This implies that the coefficient of consolidation, C_v, is approximately two orders of magnitude higher than assumed in the design phase.

2. There is strong indication that the piezometers are giving reliable readings.

3. Based on the new consolidation parameters determined from measurements made in the field, an updated settlement and stability analysis was performed yielding approximately 50% of the initially anticipated settlements and a 20% increase in the available material coefficient at any given time after platform installation.

ACKNOWLEDGEMENT

The authors would like to thank Mr. Graham Bach of Statoil for his enthusiastic support. Permission of Statoil for publication of the data presented in this paper is gratefully acknowledged.

REFERENCES

Kapp, M.S., D.L. York, A. Aronowitz and H. Sitomer (1966), "Construction on marshland deposits: Treatments and results", Highway research board, Record No. 113, pp. 1-22.

Kvitrud, A., P. To and R. Lauritzsen (1985), "User manual for the computer program CAP". Internal Report 52406-47, October.

Lambe, T.W. (1951), "Soil testing for engineers" John Wiley and Sons, New York.

Lambe, T.W. and R.V. Whitman (1969), "Soil mechanics", John Wiley and Sons, New York.

Lauritzsen, R.A. and K. Schjetne (1976), "Stability calculations for offshore gravity structures", Offshore Technology Conference 8, Houston, Texas, Proceedings, Vol. 1, pp. 75-61.

Navdocks, DM-7 (1961), "Soil mechancis, foundations and earth structures", Design manual. Department of the Navy, bureau of Yards and Docks, Washington.

Norwegian Petroleum Directorate (1985), "Regulations for structural design of load bearing structures intended for exploration of petroleum resources"

Scott, R.F. (1963), "Principles of soil meachanics", Addison-Wesley Publishing Co., Reading, MA.

Taylor, D.W. (1948), "Fundamentals of soil mechanics", John Wiley and Sons, New York.

Terzaghi, K. (1943), Theoretical soil mechanics, John Wiley and Sons, New York.