Performance of the Brent B Offshore Platform

Comportement de la Plateforme Marine Brent B

O.E. HANSTEEN Norwegian Geotechnical Institute, Oslo, Norway
E. DiBIAGIO
K.H. ANDERSEN

SYNOPSIS The Shell Brent B Condeep platform was one of the first offshore gravity platforms installed in the North Sea. This platform was chosen for a joint industry-sponsored research project to monitor the behaviour of such structures. This paper describes the instrumentation for geotechnical and dynamic measurements, and presents some of the main results from these measurements.

INTRODUCTION

The Brent B Condeep platform was built by Norwegian Contractors, and was installed in block 211/29 of the British sector of the North Sea in August, 1975. This made it one of the first gravity platforms in the North Sea, and the water depth of 140 m was the largest yet for such structures.

The concrete gravity structures represented a novel concept, resulting in a number of unfamiliar problems which had to be tackled during design. The need for comprehensive measurements of the actual behaviour of one of the prototype structures was obvious. Consequently a joint research project was established, sponsored by major oil companies, public authorities, classification societies, the platform builder, and research institutions. The execution of the project was undertaken by the PI group, a joint venture with participation from the Norwegian Central Institute for Industrial Research, Det norske Veritas and the Norwegian Geotechnical Institute.

The project covered a two-year period during which the behaviour of the platform and its foundation, wave heights and other environmental data were measured and registered. The data were analyzed and compared to theoretical computations in order to evaluate the adequacy of the design procedures that had been used. The data also provide a basis for development of improved design procedures.

The most important finding in the project was that nothing really unexpected happened to the platform and its foundation. No indication was found in the measurement or the analyses that the platform will perform less satisfactorily than anticipated at the design stage.

This paper presents the main results of the project of interest to the geotechnical engineer. A detailed presentation of the project and its results was made at a seminar in London in November, 1979 (See References).

PRINCIPAL FEATURES OF PLATFORM AND FOUNDATION

The Brent B platform is a Condeep design with a 61 m high, 19 cell caisson measuring 100 m across as shown in Fig. 1. The base area is 6 300 m^2. The deck is carried by three shafts with a circular cross section, and a diameter varying from 20 m just above the caisson to 12 m in the upper part. The deck area is 3650 m^2, and the top load during the measurements was 150 MN. The total submerged weight of the structure varied between 1750 and 2050 MN.

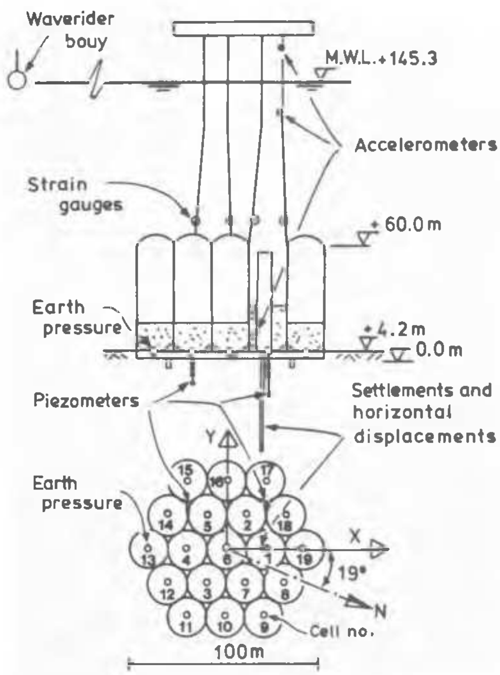
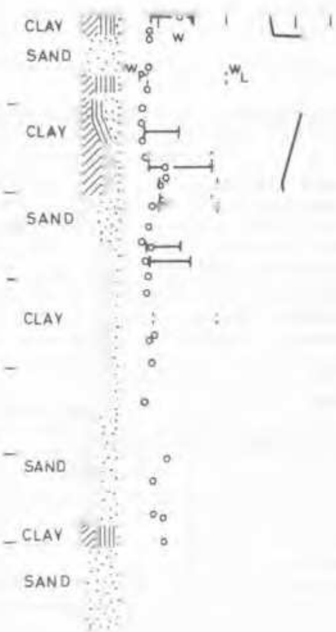


Fig. 1 Geometry of platform and location of sensors.


CLAY
SAND
W
W_P
W_L
CLAY
SAND
CLAY
SAND
CLAY
SAND

WAVE LOADS

Several severe storms occurred during the period of measurements. During both the winter seasons covered, significant wave heights of up to 10.3 m were recorded, corresponding to maximum single waves with height of about 20 m. This is 2/3 of the 100-year wave height of 30 m used in design.

For 12 selected 20-min. registration periods during storms, the theoretical wave forces on the structure were calculated. The basis for these calculations was the measured sea state, and a stochastic analysis was performed.

In design, a maximum total horizontal force on the structure of 510 MN and a maximum overturning moment of 20 000 MNm were used. The most probable maximum horizontal force occurring during the project was found to be 34% of the design value, and the most probable maximum moment was found to have been 43% of the design moment.

DYNAMIC BEHAVIOUR

The actual dynamic behaviour of the structure was studied through an extensive onshore analysis of the signals from 6 of the selected 20-min. registration periods during storms.

The displacements were found by integrating the measured accelerations. A critical comparison of alternative integration methods concluded with selecting integration in the frequency domain as the most reliable one of the methods compared. It was also found that the gravity components caused by dynamic rotation of the horizontal accelerometers had to be subtracted from the measured signals before integration. Failure to do this would lead to overestimates of from 35% (deck) to 80% (sea bed) in the actual horizontal displacements.

The highest standard deviation of horizontal base displacements during the winter 1976/77 was found to be ≈1 mm, corresponding to a maximum amplitude of about 4 mm. The highest standard deviation of horizontal deck displacements during the same winter was found to be ≈1.5 cm, corresponding to a maximum amplitude of about 6 cm.

The analysis of the accelerometer signals also comprised an investigation of the directional behaviour of the response. The results demonstrated that in addition to displacements in the principal direction of wave propagation, displacements always occurred in the transverse direction as well. These displacements had standard deviations of from about 30% to about 60% of those in the principal direction. This indicates that the wave energy was somewhat more concentrated along the principal direction of wave propagation than frequently assumed when directional wave spectra are considered.

The first two resonance periods of the platform were found to be 1.78 and 1.72 sec. The corresponding mode shapes were primarily bending modes, in the platform x and y directions respectively. The third mode was primarily torsional, with a period of 1.19 sec.

Fig. 3 shows a plot of total horizontal forces on the platform, computed from the observed wave heights, vs. horizontal displacements at sea bed computed from measured accelerations. For both quantities, the square root of the sum of variances in two orthogonal directions is used in the plot, in order to obtain independence of directional spreading. With good approximation, the points in the figure plot along a straight line, the slope of which represents the total horizontal stiffness of the foundation. This stiffness corresponds to an elastic modulus E ≈ 480 MPa in an equivalent elastic halfspace. There is no indication of nonlinear behaviour in the plot. This is also not expected for the relatively low load levels represented in the figure. It is noted, however, that brief periods with a lower ratio between force and displacement (secant modulus) could well occur without noticeable effects in the plot.

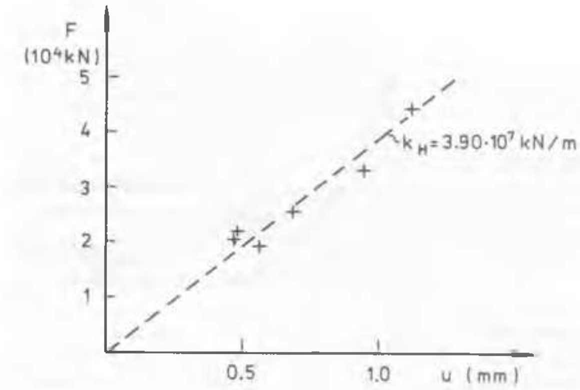


Fig. 3 Magnitude of horizontal base displacement vs. magnitude of horizontal force at sea bed.

The measured response was compared to results from theoretical, stochastic dynamic analyses. These analyses used a simple, three-dimensional model, where the foundation stiffness, the concrete modulus and the deck stiffness were varied in order to obtain agreement between the theoretical values and the measurements.

The theoretical model that gave the best agreement had a horizontal foundation stiffness corresponding to an E-modulus of 450 MPa in an equivalent halfspace, and a rotational stiffness corresponding to E = 750 MPa in an equivalent halfspace. The concrete modulus was 38 500 MPa, which is about 20% higher than assumed in design.

Using these values, there was very good agreement between the theoretical values and measurements as regards resonance periods, as shown in Table 1. Also for the corresponding mode shapes, the moments in the shafts and the horizontal displacements at sea bed excellent agreement was obtained. For the other displacement components the theoretical values were 10-18% higher than measured, while the theoretical axial forces in the shafts were about 18% lower than measured.

TABLE I

Comparison of calculated and measured resonance periods

	Mode 1	Mode 2	Mode 3
Calculated	1.76	1.71	0.97
Measured	1.78	1.72	1.19

FOUNDATION BEHAVIOUR

The settlement observations started 4 months after platform installation (Fig. 4). The time-settlement curve indicates that there is good repeatability in the measurements. Some of the irregularities that do exist, may be related to changes in platform weight with time.

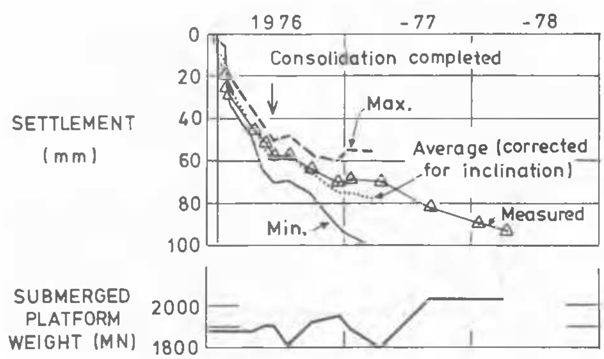


Fig. 4 Measured settlements.

The shape of the settlement curve indicates that the consolidation settlements were completed in June/July 1976, corresponding to a consolidation time of 10 months. This compares well with the 13 months assumed in design.

Figure 4 shows that the platform has settled 9 cm during the 27 months after the observations were started. However, a major part of the settlements occurred before the settlement gauge was installed. By extrapolation of observations and by theoretical calculations, the total settlements at the end of consolidation are estimated to be 35 cm. This is 30% less than assumed at the design stage.

Figure 4 shows that the settlements continue after the consolidation is completed. The rate of these secondary settlements is approximately 1.3 cm/year, and agree well with the 1.0 cm/year used in design.

The inclination of the platform was evaluated from the mean accelerometer signals. During the 14 months from January 1976 to March 1977 there has been an inclination corresponding to a differential settlement of 4-5 cm across the platform base. The lowest point is on the South side.

The permanent long-term horizontal displacement has been small. The measurements were started 6 months after platform installation, and during the following 20 months, the permanent horizontal displacements have been less than 2 cm.

The pore pressure observations indicate that the pore pressure reached equilibrium in June/July 1976, 10 months after the platform was installed. This agrees with the consolidation time deduced from the settlement observations.

During the major storm in the first winter, a pore pressure build-up in the soil due to cyclic wave loading was observed. The measured pore pressures during this storm are presented in Figure 5. The significant wave height was 10.3 m. The storm has its maximum intensity when the difference between maximum and minimum pore pressure is greatest. Figure 5 shows that the mean pore pressure then increases with about 10 kPa. This is a relatively modest pore pressure increase, but it is of the same order of magnitude as expected from laboratory test results and theoretical calculations. The pore pressure increase will lead to a temporary reduction in soil stiffness and strength. This effect of cyclic loading was taken into account in design.

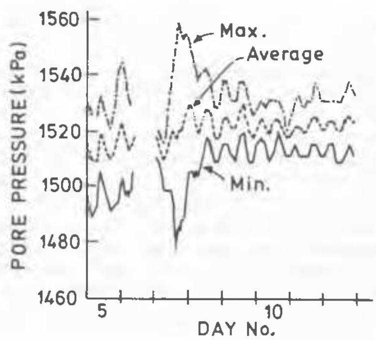


Fig. 5 Pore pressure in storm with significant wave height of 10.3 m during the first winter. Depth=4 m beneath sea floor.

The major storm which occurred the second winter had the same significant wave height, 10.3 m, as the major storm the first winter. It is interesting to observe that while a pore pressure of 10 kPa was generated the first winter, there is no tendency for a pore pressure generation the second winter. The main reason is most probably that the soil strength has increased due to the consolidation which has taken place between the two storm periods.

The soil pressure against the platform base governs the design of the base. During installation of the platform a non-uniform base contact stress variation developed (Figure 6). In the most severely loaded dome the effective base contact stress has temporarily been 1950 kPa, or almost 4 times the average. The reason for this high stress on dome 15 is believed to be a local sand layer just beneath it.

The changes in base contact stresses which have occurred during the 2 year observation period,

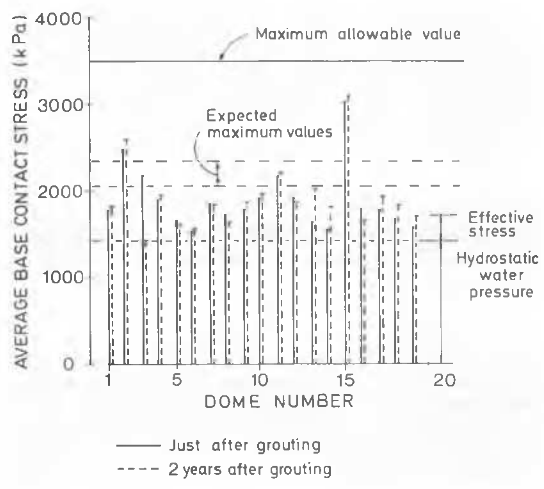


Fig. 6 Base contact stresses just after grouting and two years later.

are mainly due to special causes, like changes in submerged platform weight, grouting, installation of conductors and drainage to lower the pore pressure in the soil. The non-uniform base contact stresses which developed during installation thus seem to remain. One important observation, however, is that for none of the cells has there been tendencies for long term increases in base contact stress which may lead to overloading of the base.

As mentioned previously the measured soil stiffness is very high, with an E-modulus for an equivalent elastic half-space of 450 - 750 MPa. It is important, however, to be aware that these measurements were made the winter 1976/77, after consolidation of the soil had been completed and strengthened the soil. Further, the maximum wave forces in the observation period were less than 45% of the design wave forces (43% for moment and 34% for horizontal force), and the soil stiffness will decrease with increasing shear stresses. The backcalculated soil stiffness given above must therefore not be used uncritically for other conditions. Theoretical calculations indicate that if the 100-year design storm occurs before any consolidation has taken place, the equivalent soil stiffness may be only 10 - 15% of the measured one.

Extrapolation of measurements by means of results from cyclic laboratory tests and theoretical finite element analyses give expected future maximum cyclic displacements at sea floor during a 100-year storm of 3 cm horizontally and 1.3 cm vertically at the periphery. Extrapolation to the situation where the 100-year storm comes before any consolidation has occurred, gives cyclic displacements of 9 cm horizontally and 3 cm vertically at the periphery (single amplitude values). This is slightly less than assumed in design, when a cyclic horizontal displacement of 10 - 15 cm was assumed.

CONCLUSION

The Brent B Instrumentation Project provided comprehensive data on the actual performance of an offshore concrete gravity structure and its foundation. The results support the basic assumptions and principles used in design of the prototype structures of this kind. They also provide valuable information for the design of new structures of similar types.

REFERENCES

Andersen, K.H. & Aas, P.M. (1979). Foundation performance. Brent B Instrumentation Project, SUT Symposium, London.

DiBiagio, E., Hansen, S.B. & Wetlesen, T. (1976) Instrumentation and data aquisition. Brent B instrumentation project, SUT symposium, London.

Foss, I. (1979). A project overview. Brent B Instrumentation, SUT Symposium, London.

Hansteen, O.E. (1979). Dynamic performance. Brent B Instrumentation Project, SUT Symposium, London.

Klein, K., DiBiagio, E. & Michaelsen, M. (1976). A system for full scale measurements and continuous monitoring of gravity platform behaviour in the North Sea. Automation in Offshore Oil Field operations IFAC/IFIP Symposium, Bergen.

Larsen, L.G. & Føsker, T. (1979). Brent platform B design, and installation. Brent B Instrumentation Project, SUT Symposium, London.

Olsen, O.A. (1979). Environmental conditions and loads. Brent B Instrumentation Project, SUT Symposium, London.

Skaar, K.T. & Hansvold, C.H. (1979). Structural performance. Brent B Instrumentation Project, SUT Symposium, London.