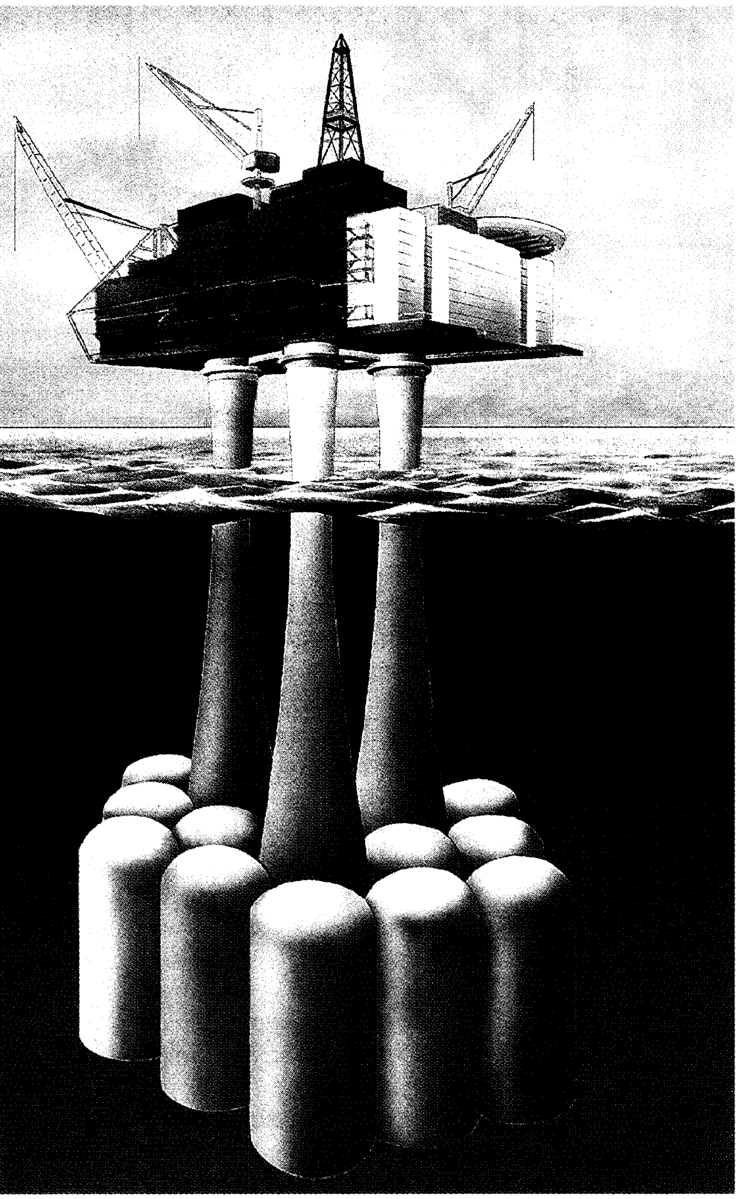
FAILURE OF AN OFFSHORE PLATFORM



Condeep offshore platform

The old pre-computer, slide-rule design techniques of 20 years ago proved their worth after a new Condeep gas platform failed in a Norwegian fjord.

By Michael P. Collins, P. Eng., Frank J. Vecchio, P. Eng., Robert G. Selby, P.Eng., and Pawan R. Gupta, P.Eng.

The challenge of extracting oil and gas from beneath the North Sea, one of the world's most hostile ocean environments, led to the development of the Condeep platforms. Standing in water of up to 300 metres, these elegant reinforced concrete structures are impressive feats of structural engineering that have advanced the art of concrete design and construction.

The construction of a typical Condeep platform starts in a large dry dock where the lower domes and part of the cylindrical walls of the cluster of buoyancy cells are cast. After the dry dock is flooded, the partially completed structure is floated out and anchored at a deep-water site in a sheltered Norwegian fjord. Slipformed construction extends the structure upwards, and as it does so solid ballast and water ballast are added to the buoyancy cells to lower the base of the structure deeper into the water. Usually three or four cells are extended upwards to form the

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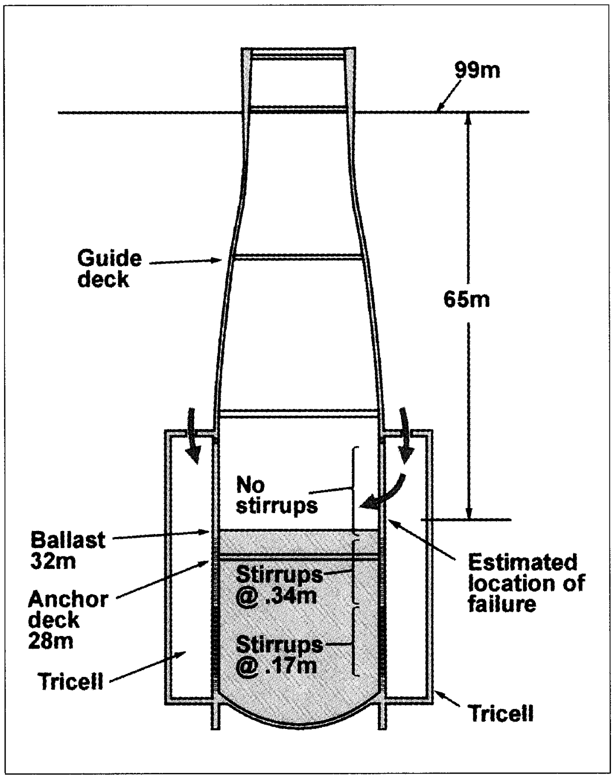
The above is a condensed version of a paper that appeared in the August 1997 of Concrete International. The authors were recently presented the 1999 Structural Engineering Award from the American Concrete Institute for the clarity of their analysis of the failure of an element in a complex structure and for their prescriptions to prevent similar failures.

shafts, which will support the deck and provide conduits for the drilling and the oil pipes. When the concrete structure is completed, additional water ballast is added until the top of the concrete structure is nearly submerged.

At this stage the top deck of the platform, which provides accommodations for about 200 people and supports the drilling and process equipment, all of which may weigh about 40,000 tonnes, is floated over the top of the concrete structure. Ballast water is then pumped out of the buoyancy cells and as the concrete structure rises it mates with and lifts the deck structure. After deck-mating, the completed structure is towed to its offshore site and lowered to its final destination on the sea floor.

A critical factor in the design of a deep-water concrete platform is the thickness of the walls. If the walls are too thin, they may fail under the very high water pressures to which they are subjected during deck-mating.

However, unlike the situation for a typical land-based structure, the designer does not have the option of greatly increasing the wall thickness to ensure a very conservative design. If the walls are too thick, the structure will not float, or will not be hydrostatically stable during the tow to the field. These severe constraints mean that for these weight-sensitive structures rather low factors of safety are employed. As a consequence, great care is required in all aspects of design and construction.



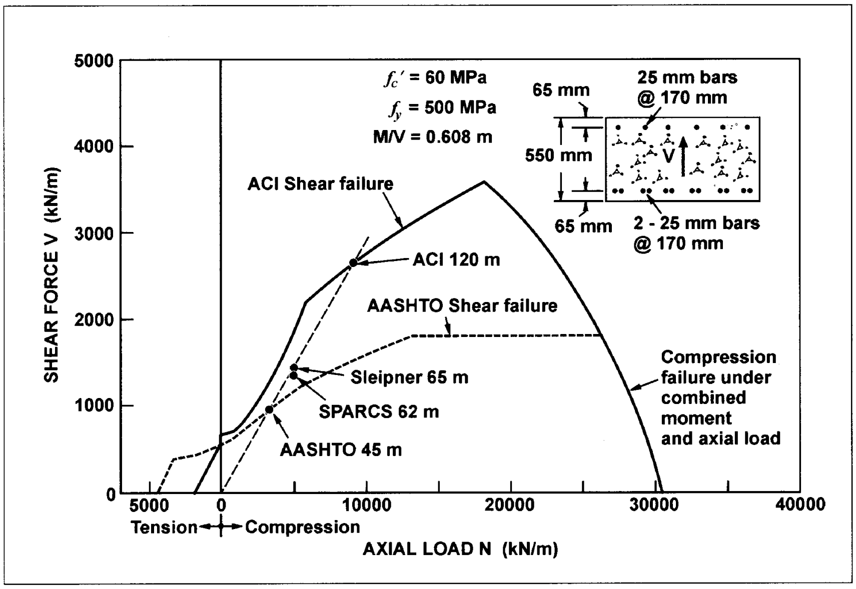
Details of drill shaft D3 at the time of failure

The loss of the Sleipner A

On August 23, 1991 the concrete base structure for the Sleipner A platform was being lowered into the Gandsfjord in preparation for deck-mating. During deck-mating a Condeep structure is about 20 metres deeper in the water than it is during operation and experiences the critical hydrostatic pressures. When the structure was about 5 metres from the planned deck-mating depth a cell wall failed, allowing water to rush into the drill shaft. The emergency deballasting pumps could not keep up with the water flow and hence, the structure sank. As it went deeper into the fjord, the buoyancy cells imploded, totally destroying the $180 million (U.S.) structure. All that remained was a pile of rubble at the bottom of the fjord.

The concrete gravity base structure of the Sleipner A platform, which was 110 metres high, consisted of a cluster of 24 cells, four of which extended upwards to form the shafts. While the exterior walls of the cells were circular, with a radius of 12 metres, the interior walls, which separated the cells, were straight. At the intersection points of these interior walls, a small triangular void called a tricell was formed. There were a total of 32 such tricells. Because these tricells had openings at the top, they filled with water once the tops of the cells were submerged. Therefore, the walls of the tricells had to resist a substantial hydrostatic pressure.

The loss of the structure was attributed to the failure of the wall



Shear force - axial force interaction diagram for the tricell wall of the Sleipner platform

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of one of these tricells. A grid of horizontal and vertical reinforcement was provided near each of the faces of the tricell wall. The bars had a diameter of 25 mm and were spaced at 170 mm, centre to centre. On the inside face of the tricell walls additional bars were placed near the ends of the walls. Thus, at these locations there were two bars every 170 mm. In addition to the grids of horizontal and vertical 25 mm bars, the tricell walls also contained 12 mm diameter stirrups. For about the bottom third of the height of the tricell walls these stirrups were spaced about 170 mm apart horizontally and 170 mm apart vertically. Near the mid-height of the walls the spacing was 170 mm apart horizontally and 340 mm apart vertically. These stirrups stopped just below the failure location. The other reinforcing detail that is important to note is the T-headed bar placed across the throat of the tricell joint. This 25 mm diameter bar was about 1 metre long and had steel plates welded on its ends to provide anchorage.

After the failure of the structure, and the results of the research, it was clear there were major problems with the previous design calculations.

What failed?

Professors Michael P. Collins and Frank J. Vecchio of the University of Toronto were retained by Dr. techn.Olav Olsen a.s., the Norwegian structural engineering firm responsible for the design, to develop a better understanding of the factors influencing the failure of the tricell wall. A series of non-linear finite element analyses were conducted using software tools developed at the University of Toronto, where considerable effort over the past 25 years has been directed toward the development of improved analysis procedures for reinforced concrete structures.

The approach taken in developing the analytical techniques was to concentrate on the formulation of simple but realistic material behaviour models for reinforced concrete. Specialized test facilities were constructed and comprehensive experiments were done to obtain the necessary data. From this, the University of Toronto researchers formulated the modified compression field theory, providing a new conceptual model for reinforced concrete.

The first computer run on the Sleipner analytical model predicted that the as-built structure would fail when the applied water pressure on the inner faces of the tricell reached a head of 62 metres, a value in excellent agreement with the estimated 65 metres head that caused Sleipner to fail. The program predicted a shear failure of the wall around the ends of the T-headed reinforcement.

The designers of the structure were interested in how the strength of the tricell wall would have changed if the stirrups, which were used just below the failure location, had continued higher up the wall. They also wanted to know how the length of the T-headed bar might have influenced the failure. To answer these questions a total of 14 different analyses were conducted.

The results showed that when the tricell walls do not contain stirrups, the T-headed bars only marginally increase the strength of the tricell until the length of these bars is long enough to penetrate three-quarters of the way into the cell wall (i.e., a length of 1.3 metres). However, if the cell walls contain stirrups, the T-headed bars significantly increase the strength of the tricell once the bars are long enough to penetrate one-quarter of the way into the cell wall (i.e., a length of about 0.8 metres). The tricell could have resisted about an additional 20 metres of water head if either the stirrups had been continued higher up the wall or if the T-headed bars in regions with no stirrups had been about half a metre longer.

The tricell wall that failed did not contain stirrups because the global finite element analysis performed as part of the design seriously underestimated the magnitude of the shear at the ends of the wall, while the sectional design procedures used seriously overestimated the beneficial effects of axial compression on the shear strength of the wall. The design procedures used to estimate the shear strength of the wall were those contained in the 1977 Norwegian concrete code, which had been influenced by the shear provisions of the 1971 American Concrete Institute (ACI) building code. These provisions, which remain unchanged in the current ACI code, predicted failure of the tricell wall when the water pressure reached a head of 120 metres, almost twice the observed failure pressure.

The Sleipner concrete gravity base structure had taken about three years to design and construct. Extensive use was made of the sophisticated computer software that had been developed for the design of previous Condeep platforms. These global analysis and sectional design software tools enabled several thousand locations on the structure to be checked for several hundred different load cases. It is indicative of the perceived precision of the design and construction that the thickness of the curved exterior walls of the buoyancy cells was specified to be 490 mm rather than 500 mm. The software identified critical locations and loadings which the engineers could check manually. Unfortunately, because the applied shear was underestimated by the global analysis and the shear strength was overestimated by the sectional analysis, the ends of the tricell walls were not identified as critical locations.

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Back to basics

After the failure of the structure, and the results of the above research, it was clear there were major problems with the previous design calculations. The tricell wall failed under a water head of about 65 metres whereas it should have been capable of safely resisting a water head of 70 metres. To give a factor of safety of 1.5 the wall should not have failed until the water head reached 105 metres.

It was recognized that finding and correcting flaws in the computer analysis and design routines was going to be a major task. Further, with the income from the lost production of the gas field being valued at perhaps $1 million (U.S.) a day, it was evident that a replacement structure needed to be designed and built in the shortest possible time.

A decision was made to proceed with the design using the pre-computer, slide-rule era techniques that had been used for the first Condeep platforms designed 20 years previously. By the time the new computer results were available, all of the structure had been designed by hand and most of the structure had been built. On April 29, 1993 the new concrete gravity base structure was successfully mated with the deck and Sleipner was ready to be towed to sea.

The failure of the Sleipner base structure, which involved a total loss of about $700 million, was probably the most expensive shear failure ever. The accident, the subsequent investigations, and the successful redesign, offer several lessons for structural engineers.

First, when designing for shear, even for shear in walls, it is prudent to be generous with the use of stirrups.

Second, no matter how complex the structure or how sophisticated the computer software, it is always possible to obtain most of the important design parameters by relatively simple hand calculations. Such calculations should always be done, both to check the computer results and to improve the engineers' understanding of the critical design issues. In this respect it is important to note that the design errors were not detected by the expensive and very formal quality assurance procedures that were employed. **CCE**

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