The Failure of an Offshore Platform

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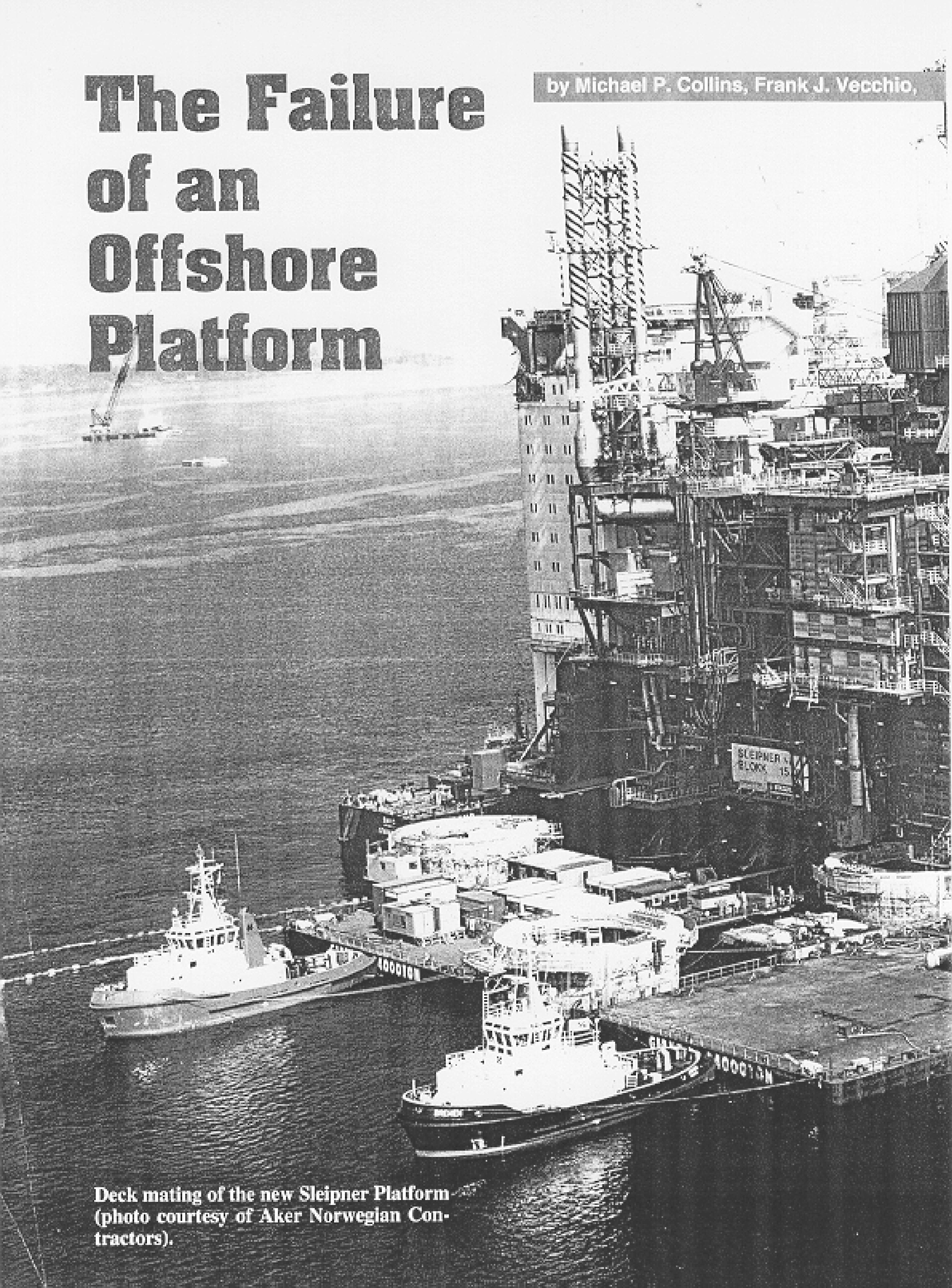

Deck mating of the new Sleipner Platform (photo courtesy of Aker Norwegian Contractors).

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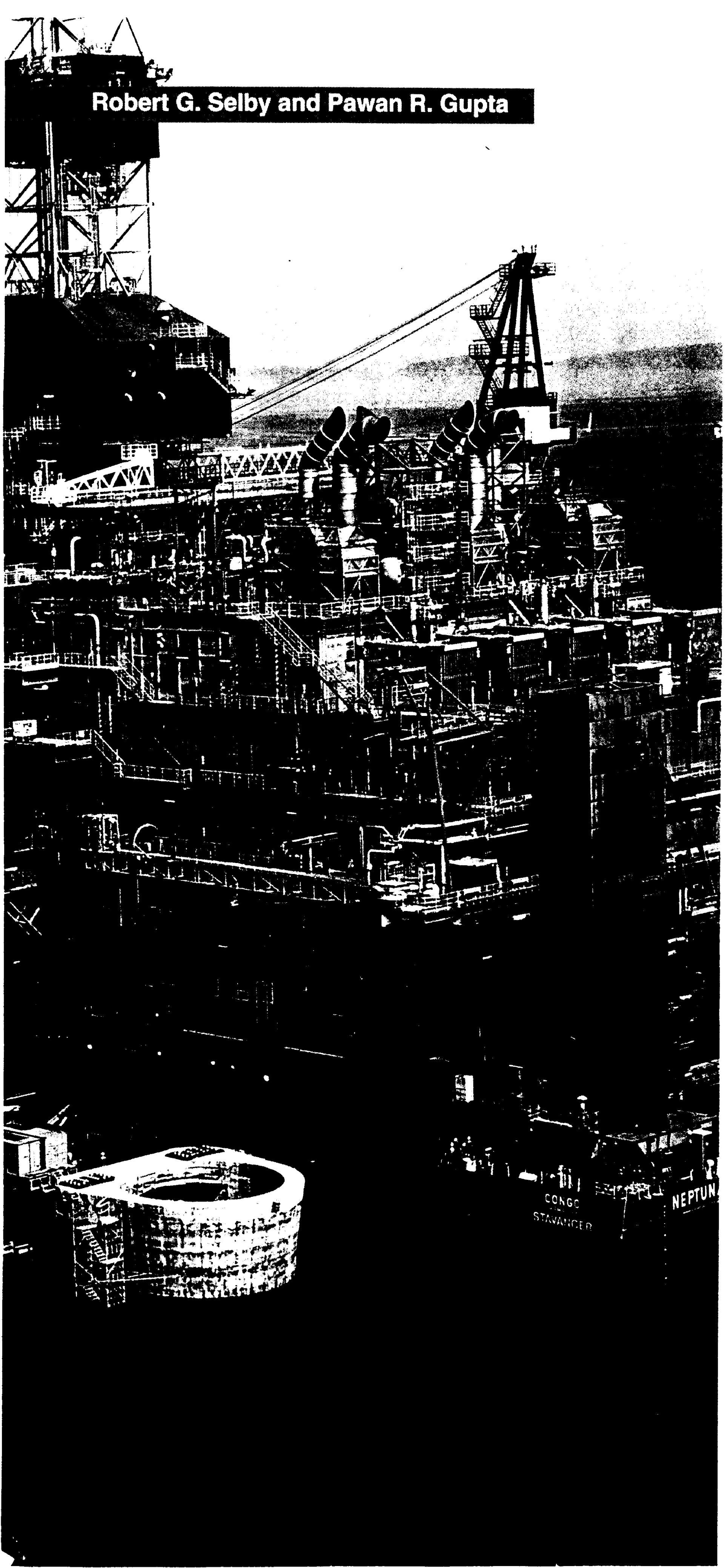

The challenge of extracting oil and gas from beneath the North Sea, one of the world's most hostile ocean environments, led to the development of Condeep platforms. Standing in water depths of up to 300 m (980 ft), these elegant reinforced concrete structures are impressive feats of structural engineering that have advanced the art of concrete design and construction. As such, they are worthy of comparison with the Roman Pantheon, a much earlier state-of-the-art concrete structure, which also had as its key components a concrete dome on top of a concrete cylinder (Fig. 1).

The construction of a typical Condeep platform starts in a large drydock where the lower domes and part of the cylindrical walls of the cluster of buoyancy cells are cast. After flooding of the drydock, the partially completed structure is floated out and anchored at a deep-water site in a sheltered Norwegian fjord. As the slipformed construction extends the structure upwards, 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 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 equipment and process equipment, all of which may weigh about 40,000 tonnes (44,000 tons), 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.

On August 23, 1991 the concrete base structure for the Sleipner A platform was being lowered into Gandsfjord in preparation for deck mating. In comparison to the 11 previous Condeeps, Sleipner A was a relatively small structure planned for a water depth of 82.5 m (271 ft) (Fig. 2). During deck mating a Condeep structure is about 20 m (66 ft) deeper in the water than it is during operation. Thus, for a structure planned for a 145 m (476 ft) deep site, deck mating would increase the pressure at the base by about 14 percent and would increase the pressure at the top of the buoyancy cell by about 24 percent.

However, for Sleipner A the increase during deck mating was to be about 26 percent at the base and about 75 percent at the top of the buoyancy cells. When the structure was about 5 m (16 ft) 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 deep into the fjord, the buoyancy cells imploded, totally destroying the $180 million structure. All that remained was a pile of rubble at the bottom of the fjord.

In the weeks following the Sleipner accident a number of investigations were launched with the aim of identifying the cause of the failure so that a replacement structure could be designed and built. This paper will describe one such investigation, which studied the shear strength of the wall at the failure location and how this strength was influenced by reinforcement detailing. The results of the study indicated that the current ACI code provisions for the shear strength of members subjected to high axial compression can be seriously unconservative.

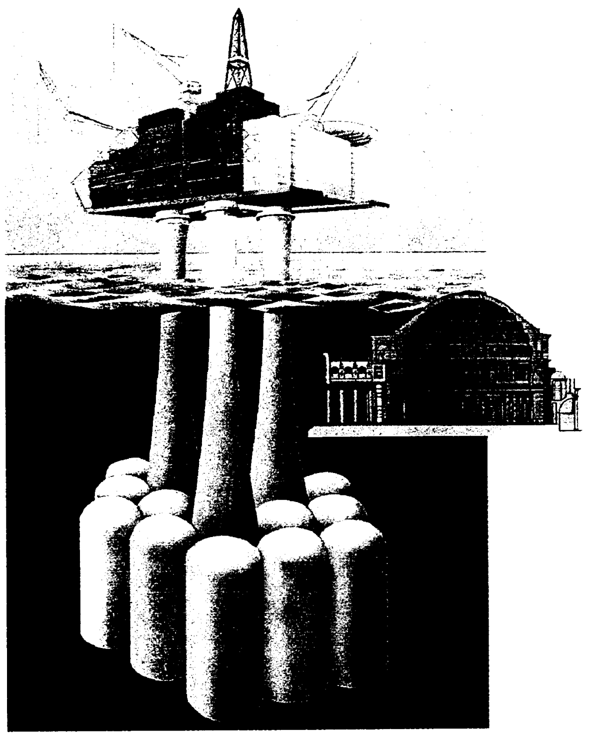

Fig.1 — A Condeep offshore oil platform and the Roman Pantheon: two state-of-the-art concrete structures.

Details of the structure and of the collapse

The concrete gravity base structure of the Sleipner A platform, which was 110 m (361 ft) high, consisted of a cluster of 24 cells, four of which extended upwards to form the shafts (Fig. 3 and 4). While the exterior walls of the cells were circular, with a radius of 12 m (39 ft), the interior walls, which separate 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. Because of this, the walls of the tricells had to resist a substantial hydrostatic pressure.

On August 23, 1991, ballast water was being pumped into the buoyancy cells to cause the structure to descend at a rate of about 1 m (3 ft) every 20 minutes. It was intended to lower the structure until its base was 104 m (341 ft) below the surface. However, when a depth of 99 m (325 ft) was reached a loud rumbling noise was heard from one of the two drill shafts. Water could be heard pouring into the drill shaft from a location that was estimated to be about 2 m (6.6 ft) above the surface of the ballast water (Fig. 5). After a few minutes, the structure was sinking at a rate of about one meter every minute and therefore had to be abandoned. A few minutes after it disappeared below the surface of the fjord a series of implosions were felt as the buoyancy cells collapsed. A local seismograph station recorded the event as a magnitude 3.0 earthquake. A later survey of the bottom of the 220 m (722 ft) deep fjord revealed that no debris larger than 10 m (33 ft) remained.

The loss of the structure was attributed to the failure of the wall of Tricell 23 adjacent to drill shaft D3 (Fig. 3). At failure this 550 mm (22 in.) thick wall was resisting a 65 m (213 ft) head of sea water, resulting in a pressure of about 655 kN/m^2 (13.7 kips/ft^2) as shown in Fig. 6. As the clear span of the wall was 4.378 m (14.36 ft) the shear at each end of the wall must have been $pL/2 = 655 \times 4.378/2 = 1434$ kN/m (98.3 kips/ft). If the flexural stiffness of the tricell wall was uniform along its span the fixed-end moment would be $pL^2/12$. Flexural cracking at the ends of the wall would have caused some redistribution of these moments. For this member the ACI Code[1] would suggest that this redistribution will reduce the "negative moment" by about 17 percent, resulting in an end moment of about 870 kNm/m (196 kip-ft/ft).

If an individual buoyancy cell was separated from the rest of the structure and was sub-

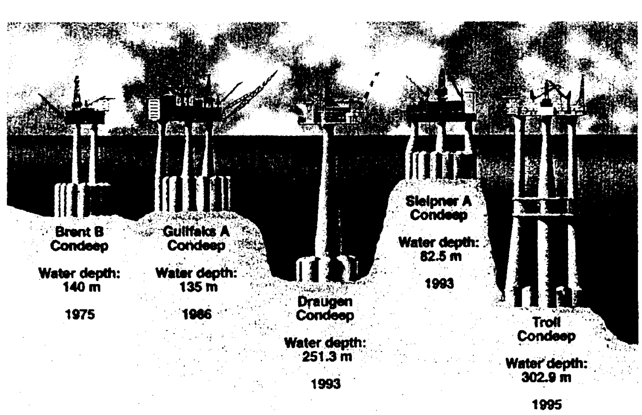

Fig. 2 — A comparison of some Condeep platform (photo courtesy of Aker Norwegian Contractors).

jected to an external pressure of p, the circumferential compression in the walls of the cell, away from the end domes, would be pR, where R is the radius to the outside face. Thus, for a pressure of 655 kN/m^2 (13.7 kips/ft^2)and a radius of 12.5 m (41 ft) the axial compression would be 8189 kN/m (561 kips/ft). When the 24 cells are joined together, the determination of the axial compressions in the different walls is a more complex problem. The circumferential compression reduces the diameters of the cells. However, at the top and bottom of the cells, the horizontal stiffness of the domes will prevent the overall dimensions of the cluster from being substantially reduced. As a result, the vertical centerlines of the exterior cells will bend inwards towards the center of the cluster. This so-called "caisson effect" will reduce the axial compression in the walls of the cells, with the reduction becoming greater towards the center of the cluster. Based on a finite element analysis of the total structure it is estimated that the axial compression in the walls of Tricell 23 at the location of failure was 5000 kN/m (343 kips/ft).

The reinforcement details in the tricell walls near the failure location are described in Fig. 7. A grid of horizontal and vertical bars was provided near each face of each wall. The bars had a diameter of 25 mm (1 in.) and were spaced at 170 mm (6.7 in.), center to center. On the inside face of the tricell walls additional horizontal 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 diameter bars, the tricell walls also contained 12 mm (0.5 in.) diameter stirrups. For about the bottom third of the height of the tricell walls these stirrups were spaced 170 mm apart horizontally and 170 mm apart vertically. Near mid-height of the walls the spacing was 170 mm apart horizontally and 340 mm (13.4 in.) apart vertically. These stir-

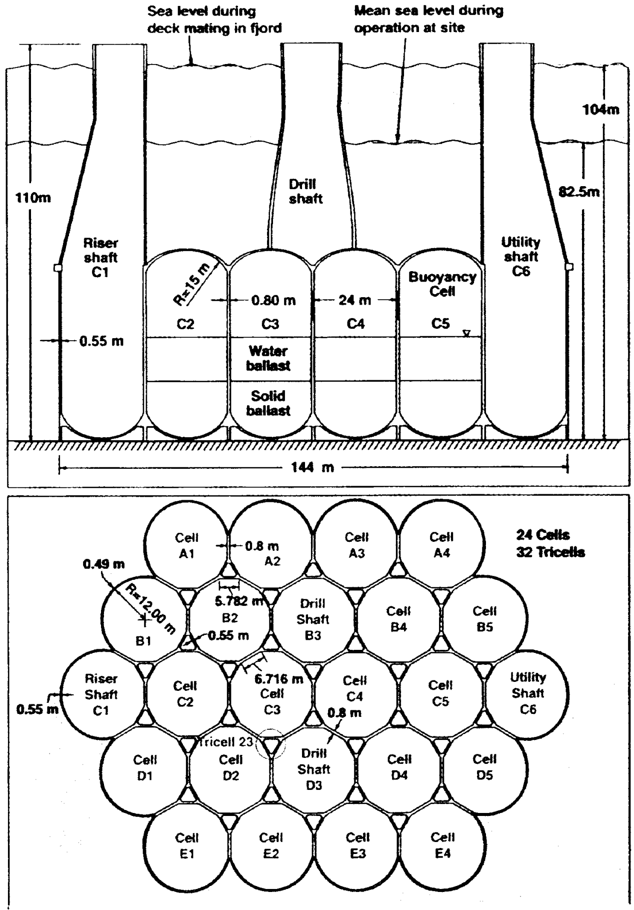

Fig. 3 — Geometric details of the Sleipner concrete base structure.

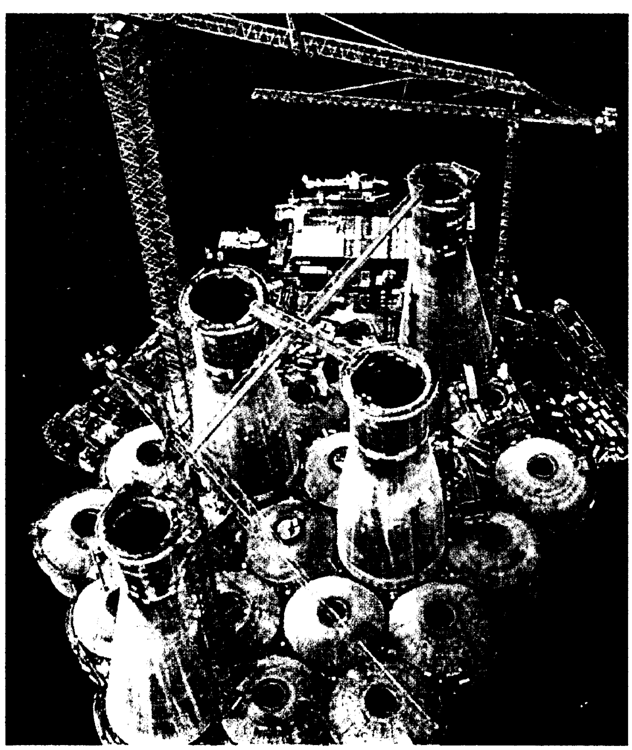

Fig. 4 — The nearly completed concrete base structure floating in Gandsfjord (photo courtesy of Aker Norwegian Contractors).

rups stopped just below the failure location (Fig. 5). The other reinforcing detail that is important to note in Fig. 7 is the T- headed bar placed across the throat of the tricell joint. This 25 mm diameter bar was about 1 m (3 ft) long and had steel plates welded on its ends to provide anchorage.

Fig. 8 is a photograph of the reinforcing details near the tricell joint taken in June 1990, when the construction had reached the top of the tricell walls. As can be seen, at this location there were stirrups in the tricell walls and three T-headed bars across the throat of the joint.

The specified concrete quality for Sleipner A was C65, which implies that the characteristic concrete cube strength is at least 65 MPa (9425 psi) at an age of 28 days. It is estimated that at the time of the failure the concrete cylinder strength was at least 60 MPa (8700 psi). The specified reinforcement grade was K500TS, which implies a "minimum" yield strength (lowest 5 percent) of 500 MPa (73 ksi) and means that the average yield strength would have been about 550 MPa (80 ksi).

Nonlinear finite element analyses of the tricell

To develop a better understanding of the factors influencing the failure of the tricell wall a series of nonlinear finite element analyses were conducted using the SPARCS program.[2,3] This program, which was developed at the University of Toronto, is formulated to model the three-dimensional response of reinforced concrete structures. It uses a secant stiffness based solution scheme that involves substituting successively better estimates of material stiffness into a linear elastic finite element algorithm. The brick, wedge, and truss elements are based on linear displacement functions. These low powered elements, when used in sufficient quantity, can

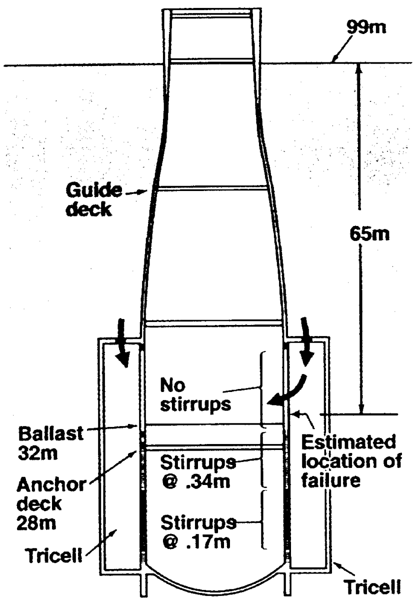

Fig. 5 — Details of drill shaft D3 at the time of failure.

predict accurately the load-deformation response of reinforced concrete structures provided that the stress-strain relationships for the cracked concrete are modeled accurately. In SPARCS, these constitutive relationships are derived from the modified compression field theory.[4]

The finite element model of the tricell is described in Fig. 9. Because of symmetry, only one-sixth of the tricell needed to be modeled. The one element thick mesh contained 342 brick elements, 338 wedge elements and 1172 nodes. Roller supports were used to model the symmetry conditions at midspan of the tricell wall and along the centerline of the 800 mm (31 in.) thick cell wall. Roller supports were also used to restrain the vertical movement of the bottom layer of nodes.

In the analyses the horizontal axial force in the 550 mm (22 in.) thick tricell wall was held constant at 5000 kN/m (343 kips/ft) while the hydrostatic pressure on the inner face of the wall was increased until failure was predicted to occur. In addition, a vertical compressive stress of 7 MPa (1015 psi) was applied to both walls and was held constant as the hydrostatic pressure increased.

The first run of SPARCS program, predicted that the as-built structure would fail when the applied water pressure on the inner faces of the tricell reached 625 kN/m^2 (13 kips/ft^2). This corresponds to a head of sea water 62 m (203 ft) high, a value in excellent agreement with the estimated 65 m (213 ft) head that caused Sleipner to fail. The predicted pattern of deflections at failure is shown in Fig. 10. Note that failure is associated with a diagonal band of extremely distorted elements near each end of the tricell's walls. Also note that the thickness of the walls is substantially increasing near their ends. This "bulging" of the section combined with the diagonal pattern of damage, indicates a shear failure of the wall.

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 been continued higher up the wall. They also wanted to know how the length of the T-headed bar influences the failure. To answer these questions a total of 14 different nonlinear finite element analyses were conducted. For one set of 7 analyses the walls of the tricell were assumed to contain 12 mm (0.5 in.) diameter stirrups spaced at 170 mm (6.7 in.) horizontally and 340 mm (13.4 in.) vertically, while for

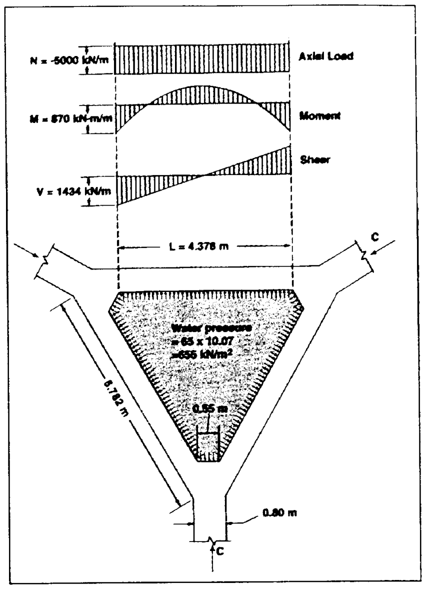

Fig. 6 — Details of the geometry and loading for tricell 23.

the other set the walls were assumed to contain no stirrups. Within each set the length of the T-headed bar was varied from zero (i.e., no bar) to the maximum possible length, which was 1.5 m (5 ft) (Fig. 7).

The results of the additional finite element analyses are summarized in Fig. 11, which shows both the predicted failure pressures and the predicted deflected shapes at failure. For the two cases with no T-headed bars, failure is predicted to occur at a pressure of about 0.6 MPa (87 psi) — a 60 m (197 ft) head of water — by yielding of the reinforcing bars crossing the throat of the tricell. For the cases where there are no stirrups, adding 0.8 m (2.6 ft) long T-headed bars to the throat does not increase the failure pressure but changes the predicted mode of failure from a flexural failure in the throat region to a shear failure near the ends of the tricell

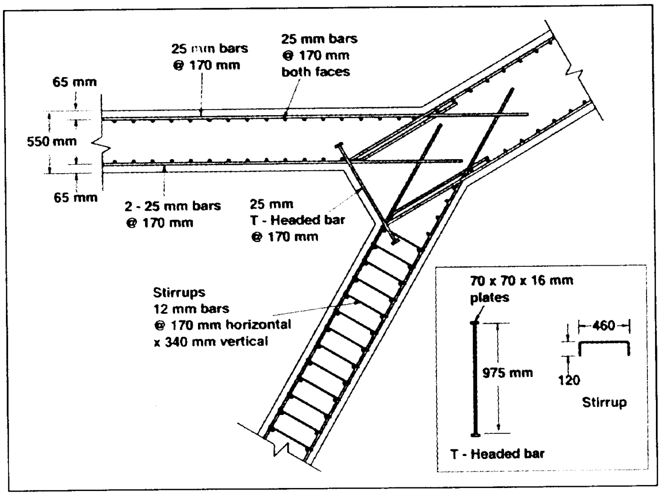

Fig. 7 — Reinforcement details for tricell 23.

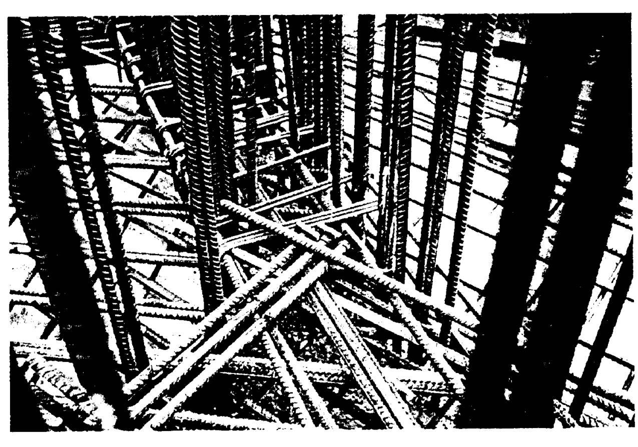

Fig. 8 — Placement of T-headed bars across the throat of the tricell.

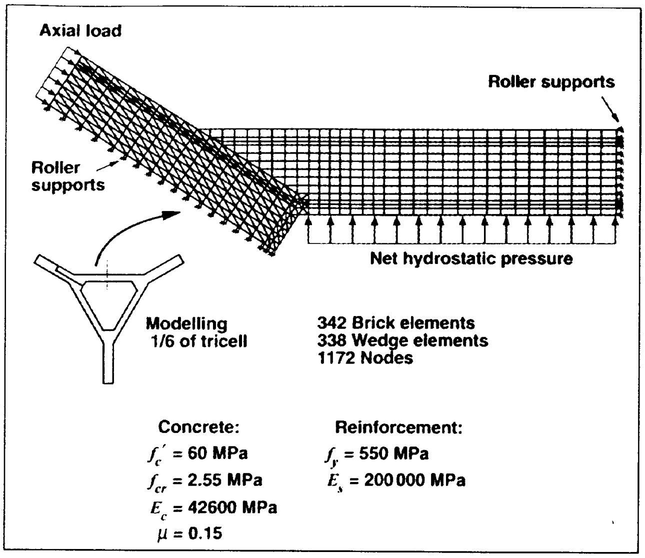

Fig. 9 — Finite element model of the tricell.

wall. As the length of the T-headed bar is increased beyond 0.8 m, the predicted failure pressure increases and the zone of shear failure moves outwards. When the T-headed bar is 1.5 m long, the predicted failure pressure is about 0.85 MPa (123 psi) — an 85 m (279 ft) head of water — and the failure mode changes back to a flexural failure of the throat region.

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 about three-quarters of the way into the cell wall (i.e., a length of 1.3 m [4.3 ft]). 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 m [2.6 ft]). For this case, extending the T-bar length from 0.8 m to 1.5 m (5 ft) only marginally increases the strength as the failure becomes governed by the flexural capacity of the throat region (Fig. 11). The results of these studies indicate that the tricell could have resisted about an additional 20 m (66 ft) of water head if either the stirrups had been continued up the wall or if the T-headed bars in regions with no stirrups had been about half a meter longer.

Shear strength calculations using ACI and AASHTO

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 procedure used seriously overestimated the beneficial effects of the 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,[5] which had been influenced by the shear provisions of the 1971 ACI building code.[6] These ACI shear provisions were formulated in the period following the Air Force warehouse collapses,[7] failures which were believed to be due to the detrimental effect of axial tension on shear strength. Thus, it is not surprising that these provisions, which remain unchanged in the current ACI code,[1] suggest that axial tension substantially reduces shear capacity while axial compression substantially increases shear capacity.

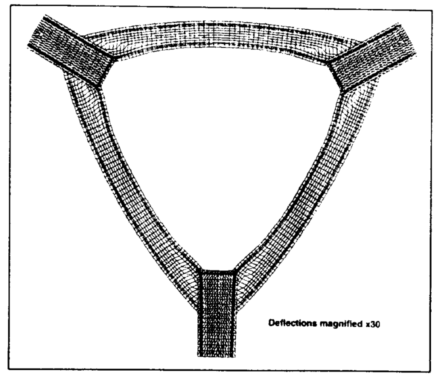

Fig. 10 — Deflected shape of the tricell at failure as predicted by the finite element model.

The failure shear-axial load interaction diagram for the Sleipner tricell wall section calculated using Equations (11-5) to (11-8) of ACI 318-95[1] is shown in Fig. 12. Also shown in this figure is the failure shear-axial load interaction diagram calculated using Section 5.8.3 of the 1994 AASHTO LRFD Specifications.[8] These AASHTO shear design provisions are based on the modified compression field theory[9,10] and reflect advances in understanding of shear behavior that have occurred in the last 25 years. It can be seen that while, for this section, the two sets of shear provisions lead to similar estimates of shear strength when the axial load is zero, the predicted changes in shear strength with change of axial load are very different. For the Sleipner tricell wall the axial compression on the wall increased as the shear force increased. In this situation the predicted

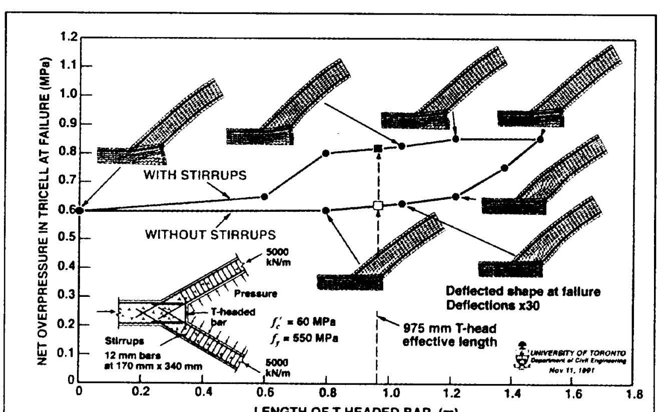

Fig. 11 — The influence of stirrups and length of the T-headed bars on the failure pressure and failure mode of the tricell.

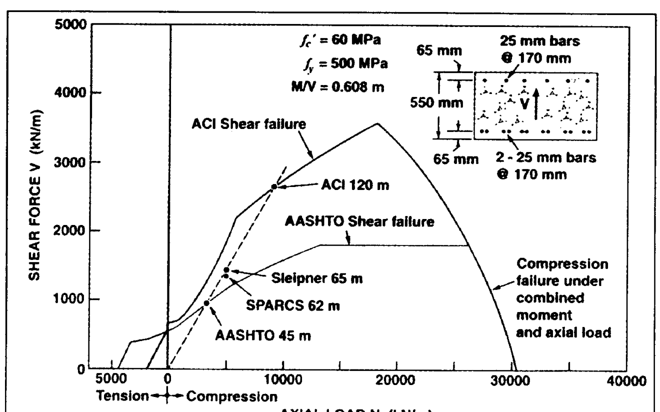

Fig. 12 — Shear force-axial load interaction diagram for the tricell wall of the Sleipner platform.

failure load is very sensitive to the assumed interaction between shear strength and axial load. As can be seen from Fig. 12, for the Sleipner loading ratios the ACI interaction diagram predicts a failure load corresponding to a 120 m (394 ft) water head, while the AASHTO interaction diagram predicts a 45 m (148 ft) water head at failure.

Fig. 13 — Engineers from the Sleipner project team test a specimen representing a portion of the rebuilt platform.

Wall elements subjected to combined axial compression and shear

In the six years since the Sleipner failure a large number of experiments[11] have been conducted in the University of Toronto's shell element tester[12] to study the response of reinforced concrete wall elements subjected to combined axial compression and shear. Some of these experiments were conducted to assist the engineers involved in the design and construction of the replacement Sleipner platform (Fig. 13). Most of the tests were aimed at identifying appropriate design techniques for reinforced concrete members subjected to compression and shear.

The results of one series of tests in which three wall elements, PC19, PC20 and PC21, were loaded at different ratios of axial compression to shear are illustrated in Fig. 14. Also shown in this diagram are the shear strengths for this wall section predicted by the ACI and the AASHTO shear design provisions. It can be seen that the AASHTO predictions are considerably more accurate. Further, while all three of the specimens failed in shear, the ACI provisions predict that specimens PC20 and PC19 would fail in combined flexure and axial load prior to reaching their shear capacities.

Specimen PC21 was subjected to loading ratios that were reasonably comparable to those experienced by the tricell wall of the Sleipner platform. The observed load-deformation response and crack development for this specimen are recorded in Fig. 15. Flexural cracking was predicted to begin near the ends of the specimen when the applied shear reached about 100 kN (22.5 kips). As the cracking developed there was a substantial reduction in the stiffness of the element resulting in a nonlinear load-deformation response. By the time the shear reached 600 kN (135 kips) — load stage 4 — a number of small diagonal cracks had formed and small expansions of the wall thickness (i.e., bulging of the wall) were recorded. By load stage 6, at a shear of 770 kN (173 kips), the diagonal cracks had reached a width of about 1.5 mm (59 mils) and the wall had increased in thickness by about 0.75 mm (30 mils).

While the readings were being taken at load stage 6, the deformation of the specimen was held constant and as a result, the load decreased. When the specimen was reloaded after load stage 6, a loud thud was heard as a sudden shear failure of the wall occurred. A wide diagonal crack formed at about 25° to the longitudinal axis of the member (Fig. 16). It should be appreciated that the formation of such cracks at the ends of the tricell walls in the Sleipner platform would result in the sinking of the structure.

Conclusion

The Sleipner concrete gravity base structure, which was destroyed on August 23, 1991, had taken about three years to design and construct. During this period 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 (19.3 in.) rather than 500 mm (19.7 in.) as shown in Fig. 3. 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

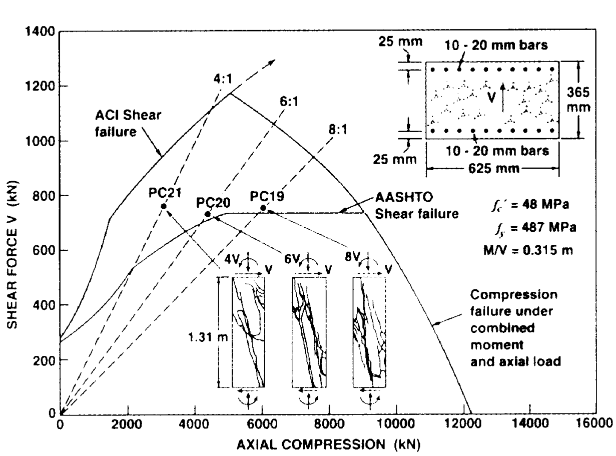

Fig. 14 — Shear force-axial compression interaction diagram for reinforced concrete wall.

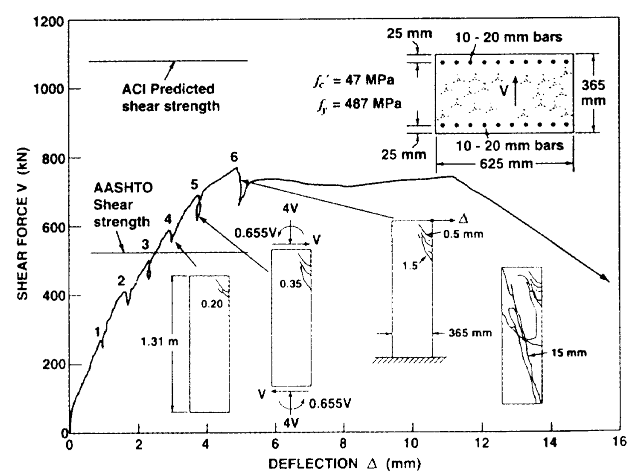

Fig. 15 — Load-deformation response of reinforced concrete wall.

analysis, the ends of the tricell walls were not identified as critical locations.

After the failure of the structure it was clear that there were major problems with the previous design calculations. The tricell wall failed under a water head of about 65 m (213 ft) whereas it should have been capable of safely resisting a water head of 70 m (230 ft). To give a factor of safety of 1.5 the wall should not have failed until the water head reached 105 m (345 ft). It was recognized that finding and correcting the 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 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 (See photo on title page).

The failure of the Sleipner base structure, which involved a total economic 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. 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 in Sleipner were not detected by the extensive and very formal quality assurance procedures that were employed. When designing for shear, even for shear in walls, it is prudent to be generous with the use of stirrups. A portion of the tricell walls, about 15 m (49 ft) in height, did not contain stirrups. It would have taken about an additional 70 tonnes (77 tons) of stirrups to reinforce this height. The analyses described in this paper indicate that if these stirrups had been provided the platform would not have failed. If an engineer is faced with designing reinforced concrete elements subjected to high compression and shear it would be unwise to use the shear provisions of the current ACI building code as these provisions can be dangerously unconservative. In this situation the shear provisions of the AASHTO-LRFD specifications will result in a more conservative and more accurate estimate of shear capacity.

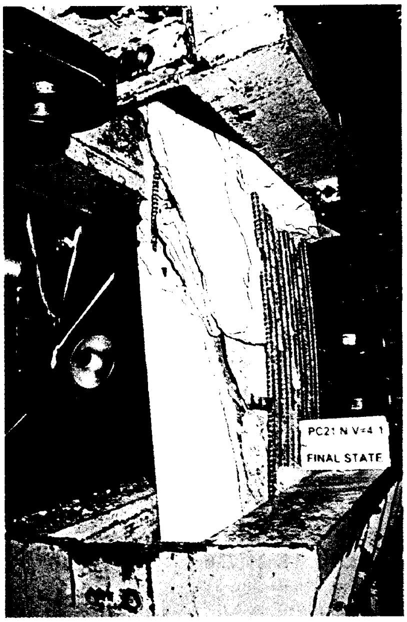

Fig. 16 — Reinforced concrete wall after failure.

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