# Health and Safety Monitoring of Composite Structures

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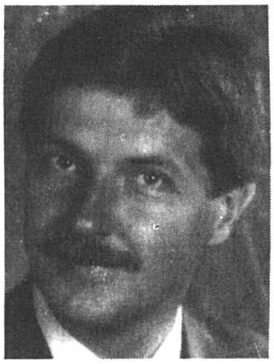

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**SUMMARY**

Structures show a typical dynamic behaviour which may be addressed as **„Vibrational Signature"**. Changes in a structure such as all kinds of damages leading to decrease of load-carrying capacity have effects on the dynamic response. This suggests the use of the dynamic response characteristic for evaluation of structural integrity. Monitoring or measurements of the dynamic response of structures makes it possible to get very fast knowledge of their actual condition.

## 1. THE BRIDGE MONITORING SYSTEM BRIMOS

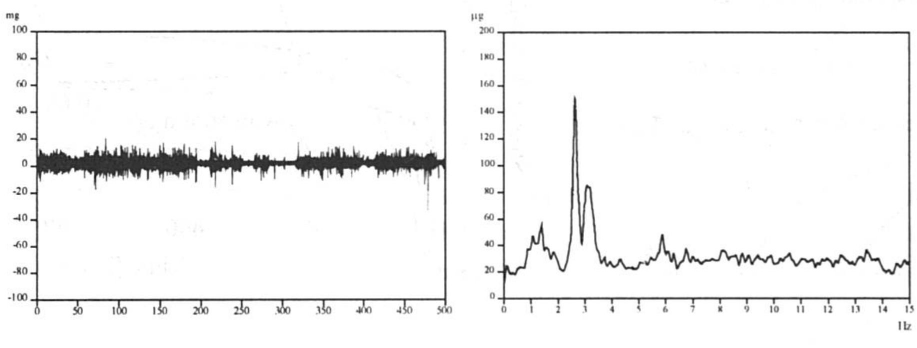


Figure 1 : Typical signal and spectrum of the Nordbrücke in Vienna (composite structure)

For the purpose of „**System Identification**“ a monitoring set-up was created, that enables quick an efficient recording as well as signal processing and report generation. The basis is the measurement of acceleration in a well determined layout of relevant locations of a structure. This provides data for the FFT analysis to generate the desired spectra. In addition data of the actual displacement of the structure is collected by infrared laser to gather information on the static behaviour and its relation to the dynamic action.

## 2. DATA PROCESSING

The collected data are processed to provide an informative report, which shall contain information on the signals itself in the desired units, the power spectrum of the readings, raw and smoothened, the drift of the readings and the relevant displacements. In a further step the readings of the various locations are combined to get an averaged spectrum and the related displacements. This is the basis for the animation of the Eigenform of the structure and the visualisation of it.

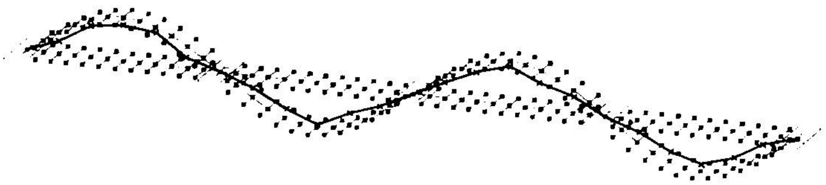
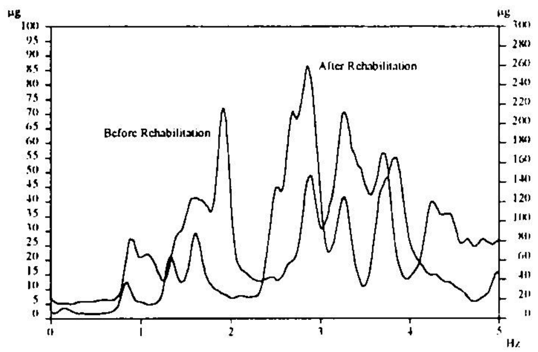


Figure 3 : First Eigenform calculated and measured of the Nordbrücke in Vienna

## 3. CONCLUSION

Due to the fact, that this bridge was monitored during 3 different stages, before, during and after the rehabilitation, valuable information was gained about the influence of the state of the structure on the response spectrum. From this basis it is tried to develop further tools to assess the quality of structures using data from dynamic monitoring.



# Time-dependent Response of Composite Structures

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**Summary**

The paper deals with the procedure of numerical computational analysis of time-dependent response and redistribution of internal forces of a composite plane beam structure due to the rheology of material. Concrete creep is taken into account according to the linear theory of creep. The influence of concrete ageing and shrinkage is, similarly to the influence of concrete creep, considered by the adequate constitutive law of material.

## 1. Introduction

In order to make an adequate computational simulation of the behaviour of composite structures within time, it is necessary to consider also the influence of the rheology of material as well as the influence of cracks on the time-dependent response of the structure. Due to the rheology of material some extreme stresses appearing in the structure during the construction can decrease significantly after a certain time, but their influence on the strains or displacements of the structure often remains to a large extent the same. For this reason also the influence of gradual construction and the building technology have to be considered in the numerical simulation of the time dependent stress-strain relationship of structures.

## 2. Response of the Structure to the External Load

The time dependent response of the composite structure is simulated on a computer by using the finite element method. The geometrical nonlinearity of the structure is considered with adequate kinematic equations of the structure [1]. Physical nonlinearity and the rheology of material are taken into account using the adequate constitutive laws of materials. The influence of cracks on the behaviour of composite structures is taken into account by way of the constitutive equations of cross-sections [2, 3]. The time-dependent behaviour of the concrete according to the linear theory of concrete creep is modelled in the accordance with the well known constitutive law of concrete in its integral form. For the reinforcing steel, prestressing steel and profile steel of composed structures, a bilinear stress-strain diagram is taken into account.

The constitutive equations of the cross-section as the relationships between the internal forces, the elongation and the curvature of the element axis are obtained by the integration of the stresses through the whole composite cross-section consisting of concrete, reinforcing steel, prestressing steel and profile steel [2]. Bernoulli-Navier hypothesis is considered.

## 3. Computational Example

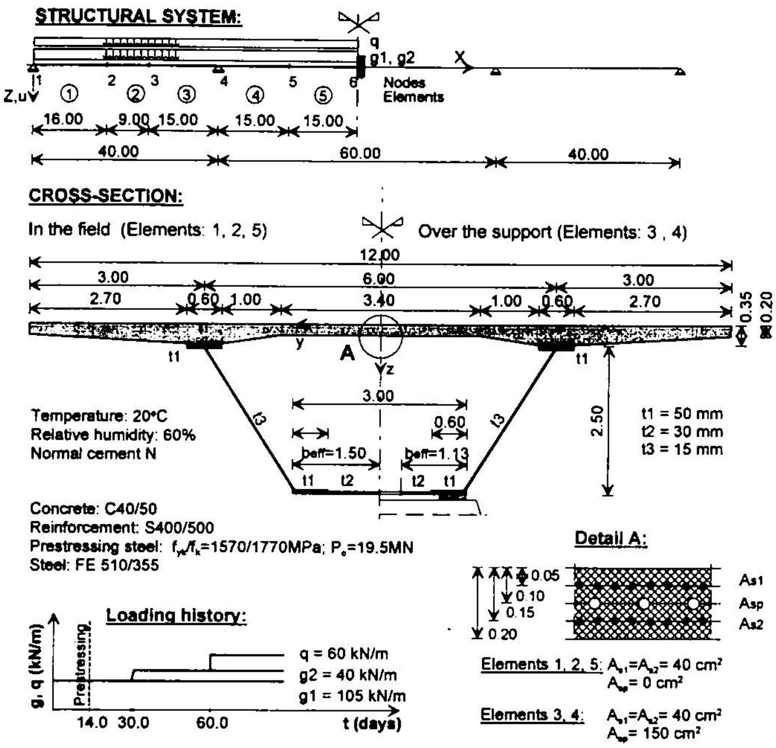


Fig. 1: Computational example - composite box girder

A composite bridge presented in Fig. 1 with spans of 40.0 + 60.0 + 40.0 m was analysed with the prepared software for the prediction of the behaviour of structures. The analysis takes into account concrete ageing, shrinkage and creep, relaxation of the prestressed steel and the influence of gradual construction. A part of the obtained results is presented on figures 2 and 3.

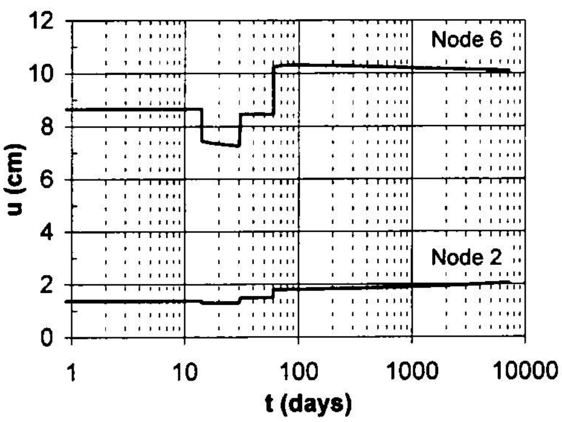


*Fig. 2: Deflections time-history*

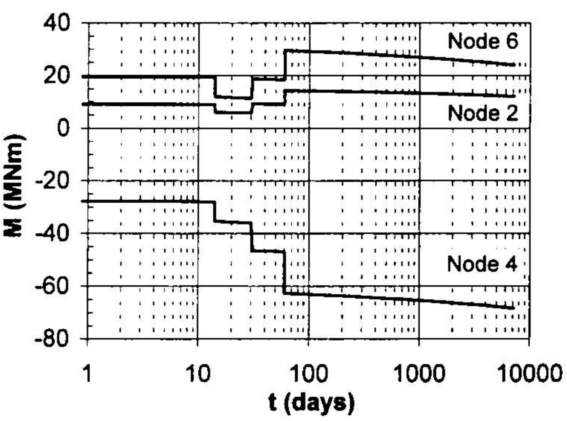


*Fig. 3: Bending moment time-history*

## 4. References

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# Study of Behaviour of Concrete Beams Strengthened by Steel Plates against Shearing Force

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## Summary

The paper presents an experimental and analytical study of concrete beams strengthened by vertically encased steel plates to increase the capacity against the shearing force. It was found that if such plates were properly arranged, the shearing capacity could be effectively increased and be applied to actual bridges, where the beam depth is severely restricted. The authors also developed a new FEM method of nonlinear two-dimensional rigid body-spring system, which proved to be useful to examine the behavior of such composite beams analytically in detail.

## 1. Introduction

The depth of beams in a concrete bridge is often severely restricted because of the clearance under the bridge or the depth of end parts of beams is curtailed by almost half to provide hinge-supports in a cantilever-type bridge. Occasionally, the shearing resistance in these parts is not large enough to prevent local damages in a long service period or under increased loading condition. The authors have developed a method for strengthening against the shearing force in which steel plates are vertically encased in the case of newly constructed bridges and they are attached on outer faces by studs and adhesive in the case of existing bridges.

The authors conducted a series of experiment, using various test specimens and also examined their elasto-plastic behavior by a new analytical method of two-dimensional rigid body-spring system, which was developed by the authors.

## 2. Experiment

The test setup and an example of the specimens (Model A-1) is shown in **Fig. 1**. Steel plates with studs are encased, which are separated at the center to prevent them from resisting the bending moment. A sufficiently thick steel plate with studs is attached also on the lower side. The amount of reinforcing bars was minimized, so as to investigate mainly the effect of steel plates. Several specimens were made for the test, in which the thickness of encased steel plates and the pattern of arrangement of the studs were varied. In addition to uni-axial strain gauges, tri-axial ones were installed to measure the distribution of shearing stresses in the plates.

In result cracks occurred in all the specimens at the central portion due to bending moment, and also large cracks occurred diagonally between the end supports and the loading points. **Fig. 2** shows an example of cracks of concrete and the distribution of maximum and minimum principal stresses in the encased web plate at the maximum load. **Fig. 3** shows an example of distribution of the shearing stress in the web plate. The stress appeared higher at the mid-depth. Comparing the results of fracture of the specimens, it seems that the studs located near the upper and lower edges act more effectively than those located in the mid-depth region.

### 3. Analytical Investigation

The concrete and steel portions are respectively divided into triangular elements. Adjoining elements and studs are connected with each other by springs in the two directions, parallel and perpendicular to each side of elements. The springs are provided with peculiar nonlinear characteristics, in order to examine the plastic behavior of the beam. **Fig. 4** indicates an example to show how fracture or yield develops in the beam.

### 4. Conclusions

As the result of experiment and analysis the following conclusions are drawn:
1) The steel plates with studs encased at the end portions of concrete beams are effective for strengthening against shearing force and the studs arranged near the edges are more effective.
2) The analytical method of FEM of two-dimensional rigid body-spring system, which was developed by the authors, are useful to accurately account for the mechanism of fracture of such composite structures, which can not be detected in details by experiments.

**Acknowledgment:** The authors would like to express their gratitude to Mr. Masaki Toba and Mr. Katsuhiko Nakai for their cooperation in the experiment and analysis.

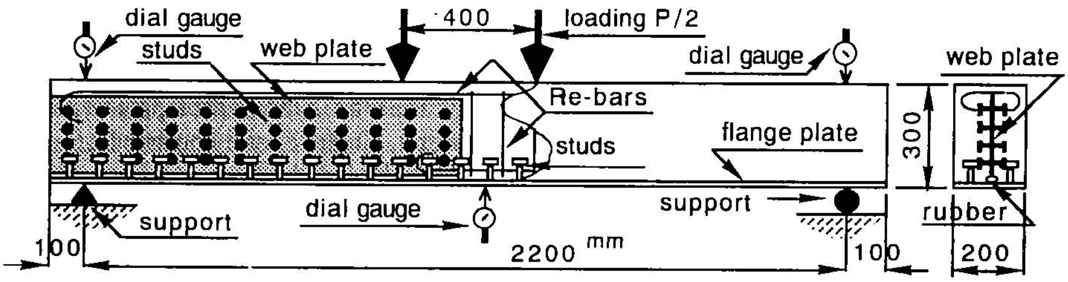


*Fig. 1 Setup for test and a specimen (Model A-1)*

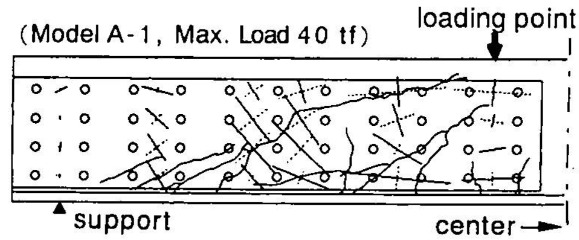


*Fig. 2 Cracks of concrete and strains of web plate*

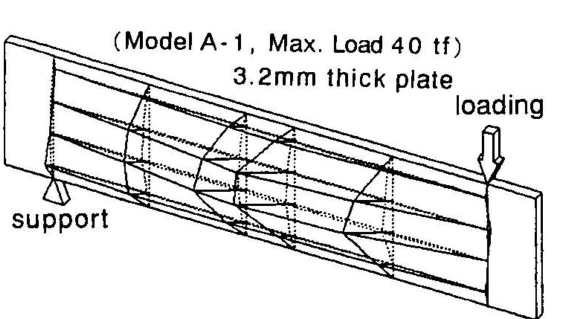


*Fig. 3 Distribution of shearing stress in web plate*

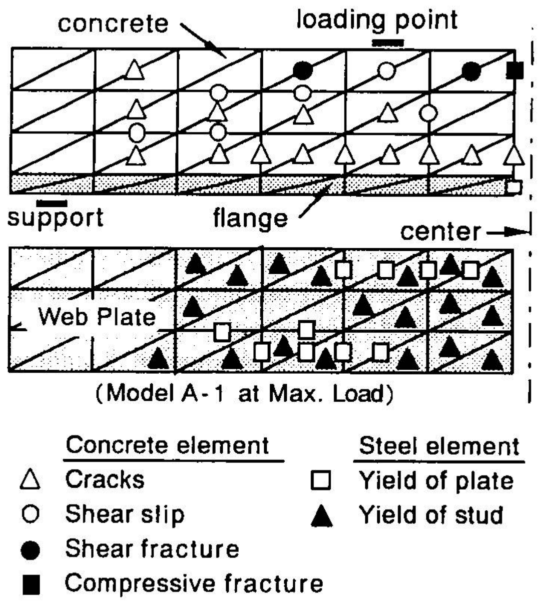


*Fig. 4 Development of fracture or yielding of concrete and web plate*

# Composite Cylinders Subjected to External Pressure

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**Douglas Goode, born 1930, spent 6 years as an engineer with Sir Robert McAlpine & Sons Ltd then 28 years at The University of Manchester, retiring in 1990.**

**Summary**

Nine tests on cylinders with a composite, steel-concrete-steel, wall subjected to external pressure are described. The results show the advantage of this form of construction.

## 1. Composite Construction

The unpublished research reported in this paper was commissioned by Tecnomare SPA of Italy as part of their study to develop vessels to resist high external pressure for hydrocarbon production in deep water. It was carried out by the University of Manchester, England, and consisted of testing nine cylinders with steel-concrete-steel walls with a hemispherical dome at one end. Such vessels are relatively insensitive to initial imperfections and fail, after the steel skins have yielded, by strength breakdown of the filler material. Previously published reports describe the theory and show it is a faithful guide to the actual structural behaviour and strength.

## 2. Tests

The objectives of the tests were:

1. To compare the experimental pre-collapse behaviour and failure pressure with theoretical predictions.
2. To examine the cylinder/hemisphere interaction.
3. To study the effect of a penetration through the cylinder wall.
4. To measure the change in radial deformation with time under sustained pressure and the effect of sustained pressure on the ultimate strength.

All nine shells tested were cylinders, with an outside diameter ($D_o$) of 495 mm and steel skin thickness ($t_o$=$t_i$) of 2.00 mm ($f_y$ = 297 N/mm$^2$; except TEC 6 where, for the inside skin, $t_i$ = 1.91 mm, $f_y$ = 260 N/mm$^2$), with a hemispherical dome as closure at one end (the steel in the skins of the dome was thinner 1.25 mm to 1.4 mm and of lower strength). The strength of the concrete filler varied, cube and cylinder strength is given for each cylinder in Table 1. The cylindrical portion was 1000 mm (approx. 2$D_o$) except for TEC 6 where it was 683 mm. TEC 5, 6, 7 & 8 had penetrations, formed of steel tubes 108 mm outside diameter, through the cylinder wall. In TEC 9 the pressure was sustained for 38 days at 6 N/mm$^2$, causing yield of the inside skin. The test failure pressure ($p_{fx}$) is compared with the simple limit theory failure pressure ($p_f$) in Table 1.

| Shell | Wall thick. $h_{av}$ mm | cube $f_{cu}$ N/mm² | cylinder $f'_c$ N/mm² | Test $p_{fx}$ N/mm² | Theory $p_f$ N/mm² | $p_{fx} / p_f$ |
|---|---|---|---|---|---|---|
| TEC 1 | 23.1 | 49.7 | 42.2 | 7.6 | 8.28 | 0.92 |
| TEC 2 | 23.2 | 49.4 | 45.4 | 8.6 | 8.29 | 1.04 |
| TEC 3 | 22.0 | 47.0 | 41.1 | 7.5 | 7.93 | 0.95 |
| TEC 4 | 20.3 | 48.0 | 43.3 | 7.2 | 7.68 | 0.94 |
| TEC 5 | 23.0 | 50.2 | 40.8 | 8.6 | 8.29 | 1.04 |
| TEC 6 | 22.8 | 52.3 | 42.5 | 9.6 | 7.90 | 1.22 |
| TEC 7 | 20.6 | 46.8 | 43.8 | 7.8 | 7.68 | 1.02 |
| TEC 8 | 23.1 | 52.0 | 44.9 | 8.6 | 8.42 | 1.02 |
| TEC 9 | 22.9 | 56.8 | 46.6 | 9.1 | 8.65 | 1.05 |
| | | | | Average (excluding TEC 6) | | 1.00 |

*Table 1. Test results compared with theory*

The failures occurred with an inward facing lobe, about 600 mm long and 220 mm wide, in the cylinder portion of the shell and were not affected by the dome or penetrations or the sustained pressure. The higher strength achieved by TEC 6 is attributed to its shorter length ($L/D_o = 1.38$), with the restraint provided by the stiffer dome and closed end enhancing its strength.

Theoretically failure is assumed to occur when the maximum principal stress in the concrete $\sigma_1$ (which will be the circumferential stress near the inside skin) reaches: $\sigma_1 = \sigma_{uniax} + 3\,\sigma_3$ where $\sigma_{uniax}$ is the uniaxial strength of the concrete (taken as 0.75 $f_{cu}$ to compare with tests, though 0.67 $f_{cu}/\gamma_m$ should be used in design) and $\sigma_3$ is the minimum principal stress (the radial stress in the concrete at the interface with the inside skin).

**The simple limit state failure pressure is given by:**

$$p_f = 2\,[\,t_o f_{yo} + t_i f_{yi} + (\,h_{av} - t_o - t_i\,)\,(\,0.75\,f_{cu} + 6\,t_i\,f_{yi} / (D_o - 2\,h_{av} + 2\,t_i\,)\,]\,/\,D_o$$

Table 1 shows that **this theory is a reasonable predictor of ultimate strength**. The detailed results show that **the elastic/plastic theory was a good predictor of pressure/deformation behaviour and of ultimate strength**. These results are particularly satisfying in view of the unexpectedly high yield stress of the steel. This had the effect of bringing the steel yield pressure close to the failure pressure thus causing stresses in the concrete filler to be approximately 70% of the cube strength when the steel skin first started to yield. **The domes**, which had a similar total wall thickness as the cylinder, were stronger than the cylinders even though the strength of the steel skins in the domes was less than half the strength of the skins in the cylinder. There were no problems at the **cylinder/dome intersection**; the failure zone was in the cylinder, except for TEC 3 where the failure lobe encroached into the dome. **Creep** of the concrete filler during the sustained pressure test on TEC 9 caused the initial deformation to increase by 25% after one day and by 70% after 38 days under a pressure that was 70% of the predicted failure pressure; the ultimate strength was not affected by the cylinder being subjected to sustained pressure. **Penetrations** through the composite shell wall, with a diameter up to 22% of the main cylinder diameter, gave no cause for concern either at the design working pressure or at failure. In no case did the penetration initiate failure or reduce the shell's strength.

A more detailed description of this work and references to other research on composite cylinders under external pressure is available at the poster presentation.

# The Safety of Composite Sub-Sea Structures

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**Summary**

This paper discusses the safety of sub-sea structures which are subjected to external water pressure comparing the use of a 'depth margin' with the standard 'load factor' approach and the advantages of composite construction for this situation.

## 1. Safety

There have been some significant, and costly, failures of offshore structures in the North Sea during tow-out or commissioning (Frigg, during tow-out Oct. 1974 and Sleipner 'A', Gandsfjord near Stavanger, 23 Aug. 1991[(1)] ). Although inadequate design and/or construction defects may have played a part in the sinking of these structures it is the author's opinion that an inappropriate loading philosophy was the major cause.

Vessels subject to water pressure due to depth are currently designed by applying a load factor to the design depth pressure. The design depth should allow for the tidal range and the expected maximum wave height over the design life. The sea-water pressure is usually considered a dead load ('permanent action' in Eurocode 4 terminology) with load factors of between 1.2 and 1.4 applied to this pressure when considering the ultimate limit state. Norwegian designers use 1.2 for tempoary loads during construction (1.3 for permanent work), China uses 1.2, Australia 1.25, Eurocode 4 uses 1.35, and in the UK 1.4 is applied to dead loads. This approach gives a low safety margin for shallow depths and excessive safety (overdesign) for deep depths.

A more appropriate method would be to add a 'depth margin' to the design depth to allow for inaccurate modelling of the actions and uncertainties in the profile of the sea-bed, for sea-bed vessels, or accidental excursion into deeper water for submersibles and then multiply this by a small load factor (to allow for uncertainties in the assessment of the effects of the actions). The choice of values for these will depend on the accuracy with which the tidal range, storm surge and wave height have been assessed; the author considers an 80 m depth margin desirable when these are not well known decreasing to say 50 m when they have been well assessed. In both approaches partial safety factors would also be applied to the materials, or in the USA and Australia 'capacity reduction factors' to the equations, to obtain the 'safe' resistance of structural members to the action effects at the ultimate limit state.

Table 1 compares these two approaches to safety philosophy for various design depths from 67 m (the depth to the probable failure point on Sleipner 'A') to 2 km (recognising that oil exploration is being carried out in 2 km water depths), when 80 m is used for the 'depth margin' with a load factor of 1.10. This shows that at 67 m design depth a structure designed using the 'depth margin' approach would be designed for a pressure twice that of the 1.2 load factor and that they would give the same ultimate design pressure at 880 m depth. Comparing with a load factor of 1.4 shows the 'depth margin' approach gives safer structures until a design depth of 293 m is reached. At 1000 m the vessel would have to decend a further 400 m before reaching the ultimate pressure when the load factor of 1.4 is used; this does seem excesively safe.

| Design depth (m) | Design pressure ($N/mm^2$) | 'depth margin' 80 m load factor of 1.1 $p_{ult}$ ($N/mm^2$) | load factor of 1.2 $p_{ult}$ ($N/mm^2$) | load factor of 1.4 $p_{ult}$ ($N/mm^2$) |
|---|---|---|---|---|
| 67 | 0.68 | 1.63 | 0.81 | 0.95 |
| 100 | 1.01 | 2.00 | 1.21 | 1.41 |
| **293** | 2.96 | **4.14** | 3.55 | **4.14** |
| 500 | 5.05 | 6.44 | 6.06 | 7.07 |
| **880** | 8.89 | **10.67** | **10.67** | 12.44 |
| 1000 | 10.10 | 12.00 | 12.12 | 14.14 |
| 2000 | 20.20 | 23.11 | 24.24 | 28.28 |

*Table 1. Depth margin and load factor approach compared at ultimate limit state pressure ($p_{ult}$)*

## 2. Composite Construction

Cylinders subjected to external pressure, such as occurs in sub-sea vessels, are sensitive to geometrical and material imperfections which can lead to instability failure before the material strength of the vessel is reached. The thinner the wall thickness the worse is this situation; and vessels designed for shallow depths will have thin walls. This is where composite construction (a steel-concrete-steel wall) has advantages over all steel construction[2,3]. The composite requires a thicker wall, which is stiffer and so not prone to instability, yet cheaper for the same strength; less steel is used as the concrete carries a proportion of the load (the proportion depending on the percentage of steel). At failure of the composite wall the steel will be at yield and the concrete, being subject to triaxial compression, exceeds its uniaxial strength. Failure of the composite cylinder invariably occurs where the wall thickness is thinnest and so it is better to base the resistance of the cylinder on an estimate of the thinnest wall thickness, calculated allowing for construction tolerances, rather than the nominal thickness shown on the drawing.

## 3. References

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