Design of Offshore Concrete Structures

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Edited by Ivar Holand, Ove T.Gudmestad
and Erik Jersin

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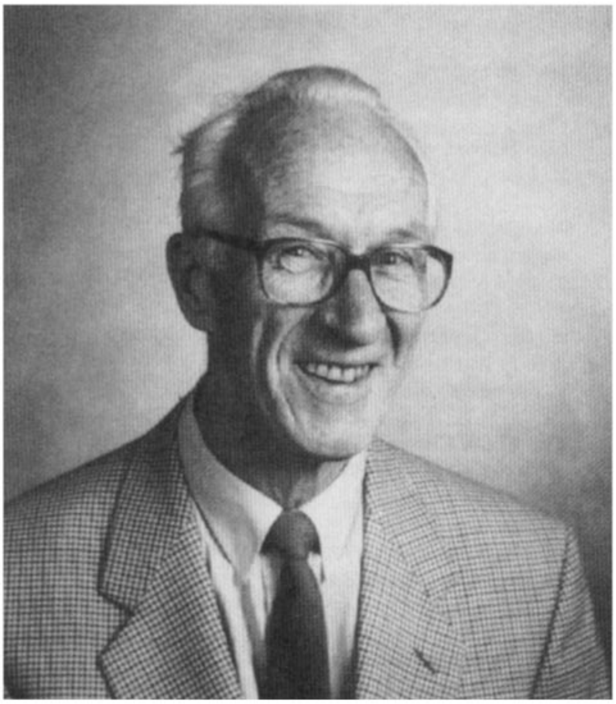
In memoriam Ivar Holand



Close to finalization of the manuscript of this book, we were informed about professor Ivar Holand's illness. We then received the message of his sudden death with much sadness. At the age of 75 he was still participing in a number of activities at SINTEF in Trondheim as well as on an international level, and we had only just started to realize that he was approaching the age of actual retirement.

Ivar Holand graduated as a civil engineer from The Norwegian Institute of Technology (NTH) in 1948. In 1958 he received his *Dr.techn* degree from NTH for his work *Design of Circular Cylindrical Shells.* From 1958 to 1991 he held positions at NTH and at SINTEF. He was Professor of Structural Mechanics at NTH from 1963 to 1991 and head of the Cement and Concrete Research Instiute at SINTEF from 1981 to 1991.

Professor Ivar Holand is probably best known as one of the pioneers of computer-based finite element methods for structural analyses. This research proved to be of major importance when the offshore industry required large offshore concrete structures.

Professor Holand also made significant contributions to the development of concrete technology and developed SINTEF's Cement and Concrete Research Institute as a leading centre for development of high strength concrete. Over a period of more than three decades he was actively engaged in the development of standards for design of safe and reliable structures, with principal emphasis on standards for concrete structural design.

Ivar Holand was on several occasions appointed by the Norwegian government and by Statoil to investigate accidents related to the offshore industry.

Ivar Holand was a member of The Royal Norwegian Society of Sciences and Letters (from 1969) and an honorary member of several associations. In 1982 he was appointed an honorary doctor of Chalmers Institute of Technology in Gothenburg, and in 1995 he was knighted with the order of St. Olav, the highest honour in Norway.

Professor Holand was the teacher and inspiration of a whole generation of structural engineers in Norway. Those of us who met him and worked with him will remember him with the greatest respect and gratitude for his professional achievements as well as his outstanding personal qualities.

Ove Tobias Gudmestad   Erik Jersin

Preface

Construction of offshore concrete platforms has to a large extent been a North Sea speciality with the majority of the activity being undertaken in Norway and UK. When this activity started around 1970, there were no standards available, specifically written for offshore concrete structures. This fact motivated development work in this field. Moreover, there were large potential economic gains from improving the concrete material and the design and production methods. The development of high strength/high performance concrete and improvement of analysis and design methods were very much stimulated by this situation.

The situation also required the development of regulations (on a government level), standards and company specifications to certify that the safety and reliability of the structures was adequate. Norwegian standards were therefore revised with the particular objective of becoming suitable for offshore concrete structures.

The activities in Norway have provided the general background for the present book. Thus, Norwegian regulations, standards and specifications have frequently been referred to in the book. When the book is used in other countries, the rules and regulations in the country in question must be followed. In such cases, references to Norwegian rules and standards may be used as illustrative examples of how various issues may be handled.

Other countries that have had similar interests and activities (but hardly on the same scale as Norway) are the UK and Canada. Furthermore, there has been interest in offshore concrete structures in France (design), the Netherlands and Australia, and rules and standards from these countries are referred to when relevant. ISO standards in this field are in drafting stage. Other relevant international documents are so far available only to a limited extent.

For the topic Quality Assurance (Chapter 6 and parts of Chapter 7) the situation is completely different. ISO standards in this field are in a mature stage and have been referred to. Chapter 6 explains how the well-known QA methodology is applied to offshore concrete structures.

The present book is based on the technical contents in “Manual for design of offshore concrete structures” prepared by the Norwegian oil company Statoil, Stavanger, Norway, in cooperation with The Foundation for Scientific and Industrial Research (SINTEF) at the University of Trondheim, Norway. A first edition of this manual was issued in Norwegian in 1993. Urged by and with the economic support from the Japan Society of Civil Engineers, the manual was translated into English, and an English version was issued by Statoil in June 1996.The editors wish to thank Statoil for the permission to use this English version of the manual as the basis for the present book.

Although the manual has been used as a basis, the content has been completely reworked. Previous chapters and appendices have been combined, parts of the previous appendices have been omitted and a new introductory chapter added. Emphasis is given to presenting an overview of important problem areas in design, and to present specific recommendations to ensure the fulfilment of a satisfactory product.

However, due among other things to restrictions on the extent of the volume of the book, the following topics are only briefly dealt with:

- Concrete platform removal, see Section 1.10, which necessitate that removal procedures are designed into the structure from the start
- Durability of concrete structures in sea water, see Section 1.8, where good experience has been acquired in the North Sea
- Construction procedures, see Section 1.5
- Geotechnical stability, which is essential for the operation of a safe and reliable structure
- Deficiencies related to corrosion of piping systems, and the need for repair procedures for this types of piping.

Different authors have been responsible for the various chapters of this book as indicated in the chapter headings. The contributors are:

Professor, Dr. Ove T.Gudmestad, Statoil, Stavanger, Norway: Chapter 2
Professor, Dr. Ivar Holand, SINTEF: Chapters 1 and 4
Senior Scientist, M.Sc. Erik Jersin, SINTEF, Trondheim, Norway: Chapter 6
Professor, Dr. Tore H.Søreide, Reinertsen Engineering and the Norwegian University of Science and Technology, Trondheim, Norway: Chapters 3 and 7
Professor Erik Thorenfeldt, SINTEF: Chapter 5

The editors thank their co-authors and the publisher for their excellent co-operation during the entire process of preparing the manuscript.

Trondheim/Stavanger, Spring 2000

Ivar Holand Ove T.Gudmestad Erik Jersin

Note:
After the sad loss of Ivar Holand, his son Per Holand volunteered to read the final proofs of the manuscript. Without his enthusiastic effort in the final phase, the publishing of this book would have been considerably delayed. Many thanks, Per!

Ove T.Gudmestad Erik Jersin

1 State of the art

Ivar Holand, SINTEF

1.1 Historical overview

The beginning of the story of the remarkable offshore concrete structures is only 30 years behind us. When the petroleum industry established activities in the North Sea in the late sixties, an immediate reaction from the Norwegian construction industry was that concrete should be able to compete with steel, that had been the traditional structural material in this industry (Fjeld and Morley, 1983), (Moksnes, 1990), (Gudmestad, Warland, and Stead, 1993). This assumption proved to be true regarding the cost of the structure as well as the maintenance costs.

One after the other of spectacular structures, 22 in total, have been placed on the sea bed in the North Sea reaching up to 30 m above sea level and down to 303 m at the deepest location, making this structure one of the tallest concrete structures in the world (Holand and Lenschow, 1996). (A general description of an offshore concrete structure is also found in Chapter 2.)

The most innovative period was around 1970, when the Ekofisk concrete platform was towed to its location (Fig. 1.1), and the first of the many Condeep platforms was on the drawing board.

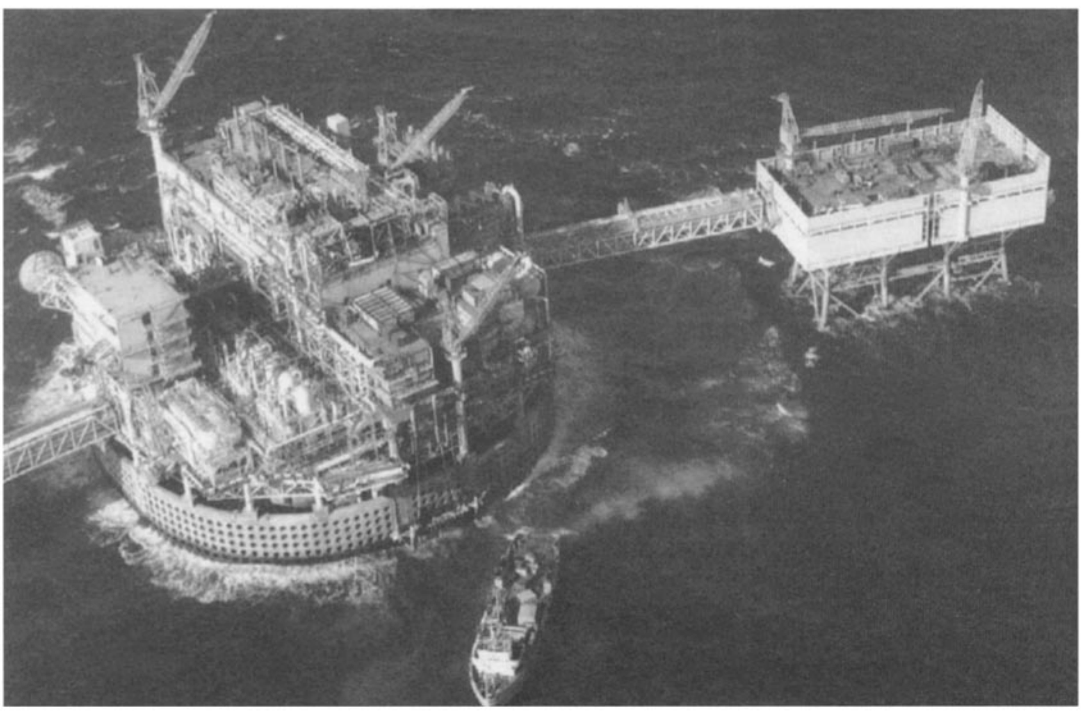


Fig. 1.1 The Ekofisk tank, completed 1973 (by courtesy of Aker Maritime)

Offshore concrete structures have proved to represent a competitive alternative for substructures in the North Sea and in other places where large offshore structures for production of oil and/or gas are required. The deep Norwegian fjords have represented a particular advantage during the construction phase, as the substructures here can be lowered deep into the sea, enabling the production plant to be floated on barges over the platform for transfer to the substructure. Hence, the production plant can be completed at quay side where the productivity is best. Hereby, costly offshore heavy lifting and hook-up activities are avoided.

Furthermore, offshore concrete structures have proved to be highly durable and to have good resistance against corrosion (Fjeld and Morely, 1983), provided that the concrete is dense, have a minimum of cracks and sufficient cover over the rebars. The Norwegian Standard NS 3473 requires 40 mm for permanently submerged parts and 60 mm in the splash zone. In the North Sea even larger rebar covers have normally been used.

Recent concrete projects are:

- in the Netherlands: F3, concrete gravity base 1992
- in the North Sea, Norwegian sector: Troll gas fixed platform (Fig. 1.2), Heidrun tension leg platform (Fig. 1.3) and Troll oil catenary anchored floating oil platform (Fig. 1.4), all completed in 1995
- in the North Sea, British sector: The BP Harding Gravity Base Tank completed in 1995
- in Congo: N'Kossa, concrete barge 1995
- in Australia: Wandoo B, Bream B, West Tuna, concrete substructures completed 1996
- on the Canadian continental shelf outside Newfoundland: Hibernia 1997
- in the North Sea, Danish sector: South Arne, to be completed in 1999.

Although the recent development has not favoured concrete platforms, there are several concept studies ongoing in the design offices. As promising floater concepts, new generations of tension leg platforms and a concrete Spar shall be mentioned. (Chabot, 1997), (Brown and Nygaard, 1997).

At present work is ongoing to develop more cost-efficient concrete structures for development of smaller hydrocarbon fields. The F3 field in the Dutch offshore sector, mentioned above, is an example; a concrete structure installed at Ravenspurne North in the British sector is another.

1.2 Design concepts

1.2.1 Cylindrical tanks

The first concrete platform was the Ekofisk platform (Fig. 1.1), that was built according to a French-Canadian concept and completed in 1973.

The decision to launch the Ekofisk platform made way for the development, not only of offshore structures but also for a development of the concrete material, design methods, construction methods, load predictions, quality management and safety evaluations.

Three additional designs in the North Sea followed mainly the Ekofisk concept (Frigg CDP-1 1975, Frigg MP-2 1976 and Ninian Centre 1978) (FIP, 1996). The huge platform built by

Mobil at the Hibernia field in Canadian waters and completed in 1997 is also mainly of the same type.

1.2.2 Condeeps and similar gravity based structures

The next concept, the Condeep, which became the winning concept for a period of time, was based on a cellular base with circular cells and one to four hollow columns (shafts), and thus had the advantage of a slim shape through the wave zone. Beryl Alpha, the first Condeep platform, was placed on the UK continental shelf in 1975. Up to 1995 a total of 14 Condeeps have been installed in the North Sea (Ågnes, 1997). Fig. 1.2 shows the largest of these structures.

Other designs were based on the same principles, except that the cells in the raft were rectangular (four platforms in the North Sea completed 1976–78, and also BP Harding in UK waters, 1995, and South Arne on Danish Continental shelf, 1999).

1.2.3 Tension leg floaters

As the exploitation of hydrocarbons moved to deeper waters, structures carried by buoyancy became more competitive than gravity based structures. For the first concrete tension leg platform, the Heidrun platform (Fig. 1.3) installed in 1995 in 345 m of water, the complete hull, including the main beams carrying a steel deck, is made of high performance lightweight aggregate concrete. The structure received the FIP (Fédération Internationale de la Précontrainte) award for outstanding structures 1998 (FIP 1998).

1.2.4 Catenary anchored floaters

Depending on several factors (depth, wave conditions, etc.) a catenary anchoring may be preferred. The first concrete platform of this type is shown in Fig. 1.4.

1.2.5 New concepts

Future concrete structures will most probably be based on a variety of new concepts (Ågnes, 1997), (Olsen, 1999), e.g.:

- Jack-up foundations (ex. BP Harding in the UK sector of the North Sea (O'Flynn, 1997))
- Anchorage Foundations for Tension Leg Platforms
- Spar buoys
- Lifting vessels for removal

A cost comparison of concrete and steel spar buoys (Chabot, 1997) shows an overall saving of 10% in the favour of the concrete option.

Fig. 1.2 Troll Gas, the largest platform of the CONDEEP type (by courtesy of Aker Maritime)

- completed 1995
- water depth 303 m
- height of concrete structure 369.4 m
- concrete volume 234 000 m^3

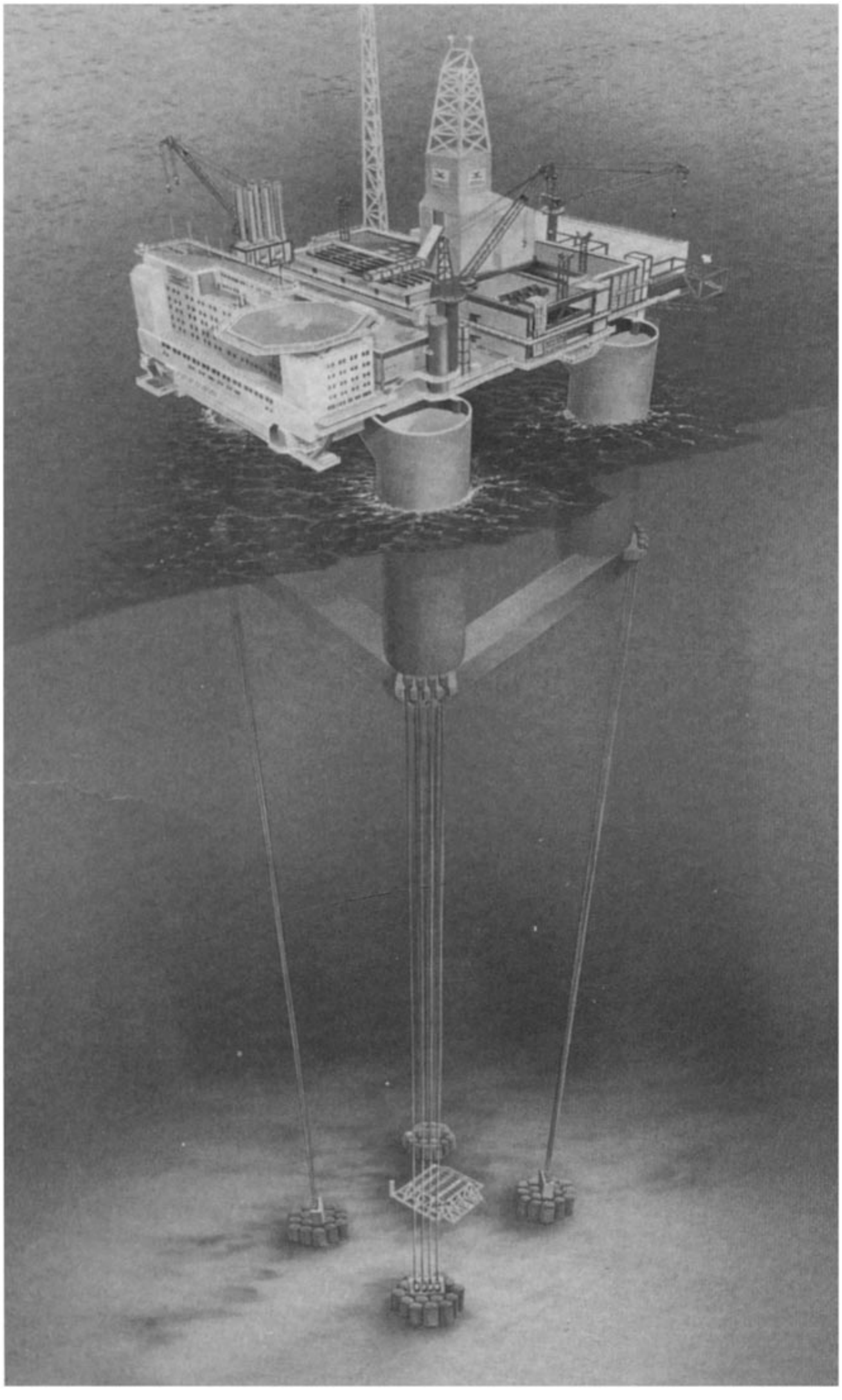


Fig. 1.3 Heidrun, the first tension leg floater with a concrete hull (by courtesy of Aker Maritime)

- completed 1995
- hull draft at field 77 m
- concrete volume 66 000 m^3, LC 60, density 1950 kg/m^3
- water depth 345 m

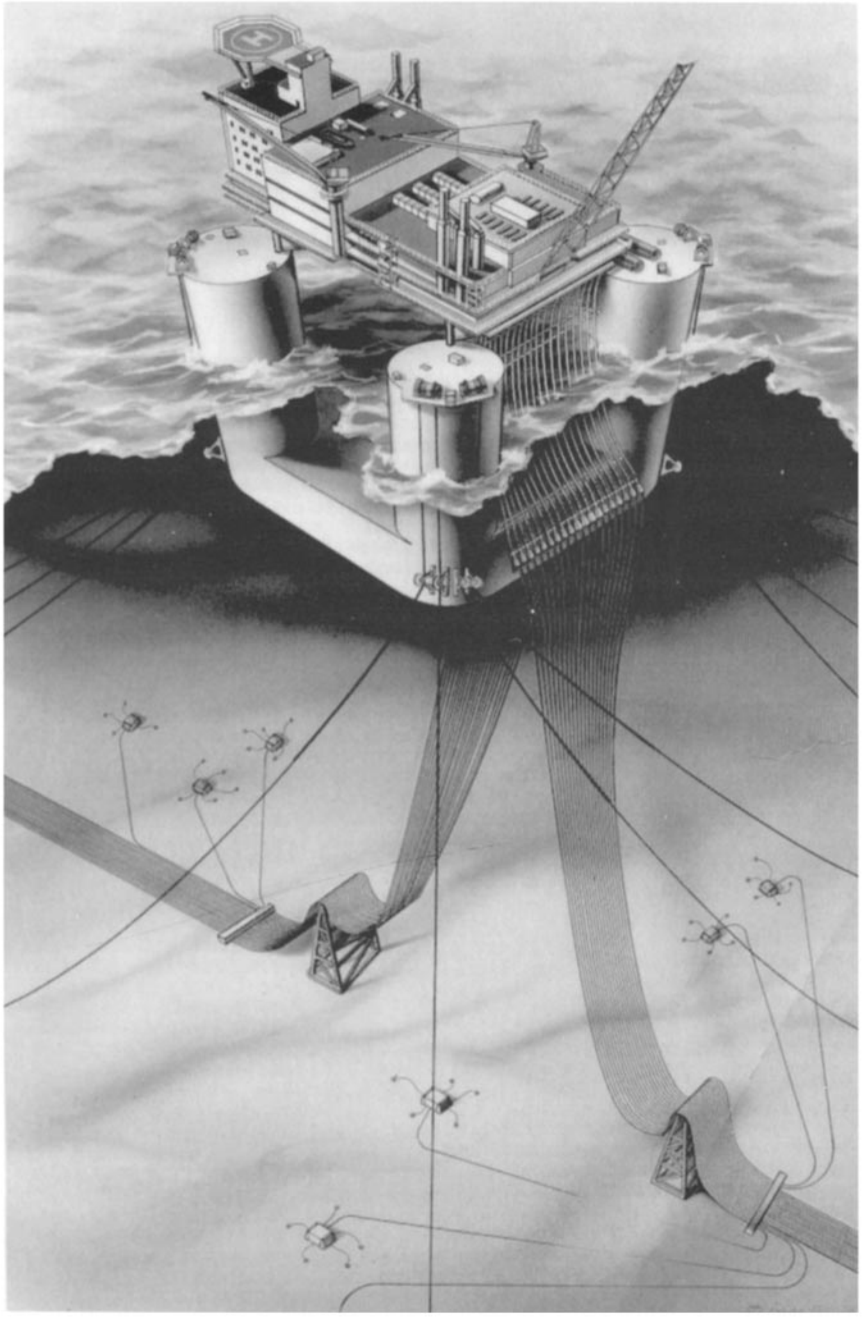


Fig 1.4 Troll Oil, the first catenary anchored floater with a concrete hull (by courtesy of Kvaerner Concrete Construction)

- completed 1995
- hull draft at field 40 m
- concrete volume 43 000 m^3
- water depth 325 m

1.3 Development of the concrete material

When the Ekofisk tank (completed 1973) was designed, the highest strength class allowed according to Norwegian Standard was used, namely B 450 with a cube strength (in present units) of 45 MPa, now denoted C 45. Economy favoured a continuous increase of concrete strength grades, in particular because cylindrical and spherical shapes were preferred. These needs contributed strongly to the development of high strength/high performance concretes. The strength grades in recent structures are, for comparison, about C 80–85. The increase has been made possible by a steadily increasing level of knowledge accumulated through experience and research. (Moksnes and Sandvik, 1996), (Neville and Aïtcin, 1998), (Moksnes and Sandvik, 1998).

Important factors contributing to the improvements of concrete qualities are:

- development of a high strength cement
- well controlled aggregate grading
- admixtures, in particular superplasticisers and retarders
- strict quality assurance procedures

The mechanical properties of high-strength concrete differ in many ways from those of traditional concrete. Thus, traditional design procedures for reinforced concrete cannot be extrapolated to new strength classes without a thorough study and relevant modifications. To avoid unnecessary restrictions to the application of high-strength concrete, the extended knowledge must be implemented as rules for high-strength concrete in standards and codes of practice (Section 1.6).

1.4 Design

1.4.1 Preliminary design

Offshore concrete platforms are constructed inshore, floated to a deep-water site for deck-mating and towed to their operation positions offshore. This construction procedure implies that the structures must be hydrodynamically stable under many different conditions. Moreover, dynamic response is important in temporary stages as well as at the operating stage. Such requirements necessitate that geometrical external shapes as well as weights and rigidities (and hence thicknesses) are reasonably well approximated in the preliminary design, and that the detailed analyses mainly serve to specify ordinary reinforcement and prestressing steel. In the preliminary design, basic understanding of structural mechanics and traditional shell theory, and experience from similar structures play an important role, but computer analyses may be also used in this phase.

1.4.2 Global analysis

The first designs of the Condeep structures were based on simple, classical shell calculations as described under preliminary analyses above. However, the intersections between the different shell elements introduce irregularities, and the wave loads and other loads introduce various forces in addition to the hydrostatic ones. Such facts call for more advanced methods of analysis.

The structural analyses have mainly been based on a linear theory of elasticity, and since the mid-seventies on the use of large finite element programs. The largest finite element calculations may involve more than one million degrees of displacement freedoms and require the use of supercomputers (such as CRAY YMP/464 that has been used for the largest analyses) (Brekke, Åldstedt and Grosch, 1994) (Galbraith, Hodgson and Darby, 1993).

1.4.3 Postprocessing. Dimensioning

The offshore platforms are subjected to a large number of loading conditions during the construction, tow-out, installation, operation and removal phases. Large hydrostatic pressures dominate during deck-mating, while wave, current and wind loads dominate during the operation phase. To permit the handling of all relevant load cases, a number of basic load cases are selected, from which the actual load cases with load factors for the relevant limit state, possible amplification factors, etc; may be obtained by linear scaling and superposition. To utilize the huge amount of data from the finite element analysis in an efficient dimensioning of the reinforced concrete sections of the structure, a post-processor that is specially developed for the purpose is needed (Brekke, Åldstedt and Grosch, 1994).

The strength of the reinforced concrete is checked point-wise by comparing the stress resultants with the strength in the same point. The strength evaluation relies on semi-empirical design formulae, mainly based on reduced scale experiments on beams and column elements, and is taking into account cracking and other non-linear effects. The design formulae are specified in codes and standards, but have also been supplemented by special procedures in the post-processors (Brekke, Åldstedt and Grosch, 1994). Refinement of the methods is still going on (Gérin and Adebar, 1998).

1.5 Construction methods

Offshore concrete platforms are constructed inshore, and vertical walls have mainly been constructed by slipforming. Slipforming has also been extended to be used for non vertical walls, variable thicknesses and variation of diameters and cross section shapes as usually needed in the shafts. The slipforming method requires a careful control of the concrete consistency in order to avoid flaws in the concrete surfaces, thus requiring an intimate interaction between material technology and construction procedure.

When the concrete structure is completed, it is floated to a deep-water site for deck-mating and towed to the operation position offshore. The production hence also includes challenging marine operations in narrow fjords.

1.6 Rules and regulations

1.6.1 Government regulations

Design and construction of offshore structures must, like structures onshore, follow rules that basically are laid down by the government that has the sovereignty of the area in question, e.g. in:

USA:	United States Department of the Interior
UK:	Department of Energy: Statutory Instruments SI 289 1974 The offshore installations
Norway:	Norwegian Petroleum Directorate Norwegian Petroleum Law with Regulations and Guidelines (NPD, latest version applies).

For the design work in Norwegian waters the following regulations are of particular relevance:

- Regulations relating to safety, etc. to Act No. 11 of March 22nd 1985, relating to the petroleum activities
- Regulations relating to loadbearing structures in the petroleum activities including:
  - * Guidelines to regulations
  - * Guidelines concerning loads and load effects
  - * Guidelines relating to concrete structures
- Regulations relating to the licensee's internal control in the petroleum activities on the Norwegian continental shelf
- Regulations relating to implementation and use of risk analyses in the petroleum activities, with Guidelines.

As for structural concrete, Norwegian Petroleum Directorate's "Regulations relating to load bearing structures with Guidelines" are mainly based on Norwegian standards; see also Section 1.6.2 and Chapter 5.

1.6.2 Standards

In many countries, government regulations use the "reference to standards" principle, implying that requirements to safety of structures is considered to be satisfied if specified standards are followed. Thus, standards play an important role for offshore structures. Relevant standards are, for instance:

- Canadian standard CSA S474–94 Concrete Structures. Part IV of the Code for the Design, Construction, and Installation of Fixed Offshore Structures. ISSN 0317-5669. June 1994.
- ISO standard 13819 Part 3 (to appear, will cover the entire engineering process for offshore concrete structures). For design, NS 3473 is referred to as a standard that covers relevant conditions (Leivestad, 1999).
- Norwegian Standard NS 3473 Concrete Structures. Design Rules. 4th edition 1992 (in English), 5th edition 1998 (English edition in print).

- Norwegian Council for Building Standardisation (1999), *Specification texts for building and construction,* NS 3420, Oslo, Norway, 2nd edition 1986, 3rd edition 1999.

Other documents may play a similar role, e.g. ACI 318–95, saying in the introduction: "The code has been written in such a form that it may be adopted by reference in a general building code..."

The European prestandard (Eurocode 2, 1991) covers concrete structures in general, but says explicitly that it does not cover offshore platforms.

Standards are in general not mandatory documents. Similarly, they may also be used outside the country or region where they were issued. As an example, the Norwegian standard for concrete structures was used for the concrete platform on the Hibernia field, Newfoundland, Canada. The reason why the Norwegian standard was preferred was mainly that the operator (Mobil) was well acquainted with this standard from previous projects in the North Sea.

1.6.3 Certification. Classification companies

Control and approval of offshore installations is regulated by national government authorities. The third party role of classification societies in this activity differs (Andersen and Collett, 1989).

The most active classification societies in offshore activities are Lloyd's Register and DNV, which may be described briefly as follows:

- Lloyd's Register is the world's premier ship classification society and a leading independent technical inspection and advisory organisation, operating from more than 260 exclusively staffed offices worldwide and served by 3,900 technical and administrative staff.
- Det Norske Veritas (DNV), Oslo is an independent, autonomous foundation established in 1864 with the objective of safeguarding life, property and the environment. DNV has 4,400 employees and 300 offices in 100 countries. DNV establishes rules for the construction of ships and mobile offshore platforms and carries out in-service inspection of ships and mobile offshore units.

1.6.4 Company specifications

Codes and standards are often not sufficient as technical contract documents. Thus, oil companies often choose to issue their own, more detailed, company specifications. Such specifications may also prescribe safety requirements in addition to those given in rules and regulations. An example of such a specification is NSD 001, issued by Statoil, a Norwegian oil company.

1.6.5 Development of codes and standards

Codes and standards are subject to a continuous scrutinizing and updating to be abreast of the technical development. Many actual decisions are, however, taken in a pre-standardization phase, where the new knowledge is digested in discussions in an international environment. Important organizations in this role are:

- *fib:* International Federation for Structural Concrete (established 1998 by merging FIP and CEB)
- ACI: American Concrete Institute
- RILEM: International Union of Testing and Research Laboratories for Materials and Structures.

1.7 Quality assurance

The highly automated analyses by using finite element methods and dimensioning by post-processors have their pit-falls. Thus, comprehensive schemes for quality assurance are implemented to avoid errors in analysis and design, including simplified checks of results of the global analysis, mainly equilibrium checks. A manual issued by the Norwegian oil company Statoil recommends that the simplified preliminary analyses discussed above are systemized in such a way as to also serve the purpose of a rough check of the results of the detailed analyses (Gudmestad, Holand, and Jersin, 1996).

The need for quality assurance procedures is well illustrated by the Sleipner accident. The gravity base structure of the Sleipner A platform is a traditional Condeep platform placed at a moderate depth of 82 m in the North Sea. The first concrete hull built for this purpose sprang a leak and sank under a controlled ballasting operation in Gandsfjorden outside Stavanger, Norway, on 23 August 1991 (Jakobsen, 1992). It was rebuilt and placed in position in 1993.

1.8 Durability

The first concrete platform was placed in the North Sea in 1973. Since then the behaviour of these structures has been investigated thoroughly by means of inspection and instrumentation programmes. In addition, data from maintenance and repair reports are available. Based on such data, the durability of offshore concrete structures has been studied by a working group appointed by FIP (FIP 1996). The conclusions of this group are, directly quoted:

- the concrete offshore platforms provide full operational safety
- they show a very high durability level
- they do not require costly maintenance and repair operations
- their effective lifespan has been underestimated and their 20 years initial design life can be greatly protracted

The document has been based on an inquiry answered by:

- The Norwegian Petroleum Directorate
- Oil companies
- Certifying authorities
- Contractors and consultants

Similar conclusions are found in (Ågnes, 1997), (Moksnes and Sandvik, 1998), (Bech and Carlsen, 1999) and (Helland and Bjerkeli, 1999).

The FIP report also contains recommendations for design, construction and inspection practice.

1.9 Competitiveness

In spite of good experience with concrete structures, they will not be competitive for all offshore projects. A few essential arguments for the choice of a concrete structure, because of cost efficiency, are listed below (Ågnes, 1997):

- <u>Topside weight.</u> Heavy topsides can be accomodated on a concrete substructure.
- <u>Storage.</u> Oil and stable condensate can be stored in concrete cells.
- <u>Durability and maintenance.</u> Concrete is favourable when long life-time is desired.
- <u>Seabed conditions.</u> On firm soils the concrete platform rests perfectly by its own weight. On soft soils long skirts provide an efficient solution.
- <u>Collision strength.</u> Concrete is robust to local damage.
- <u>Motion characteristics of floaters.</u> Concrete platforms offer better characteristics because of larger displacement.
- <u>Ice infested areas.</u> Concrete may be designed to resist ice forces.
- <u>Local content.</u> Large parts of the plain construction work can be carried out by unskilled labour under competent guidance.

Cost competitiveness is also discussed in (Collier, 1997) and (Michel, 1997). Marine concrete structures for the future are discussed by showing several options by Olsen (Olsen, 1998) and by Iorns (Iorns, 1999).

1.10 Removal. Demolition. Recycling

It is assumed that all future offshore concrete platforms shall be removed from site after decommisioning, except, perhaps, in rare cases. The decommisioning will usually start by refloating of the platform. All concrete platforms need a ballasting system, for ballasting to a proper draught, during production and tow-out and final positioning on site. In recent cases (for Condeep platforms since Statfjord B 1978) the ballasting system has also been designed to be used for refloating. Even Platform Removal Manuals have been produced during the design phase in some cases. In spite of this, the refloating is no straightforward operation and will require extensive studies of safety precautions during the operation, including possible strengthenings. Problems encountered are, for example, related to penetrations by conductors.

A re-use on another site is generally unrealistic, and the next step will therefore be demolition and preferably reuse of reinforcement steel and crushed concrete (Olsen and Høyland, 1998) (Høyland and Maslia, 1999). For the monotower platform Draugen and the floating unit at Heidrun, removal and demolition studies have been performed. Concrete Platforms for re-use have, however, also been discussed (Stead and Gudmestad, 1993).

1.11 Spin-off effects

The technology developed for the offshore concrete structures has had a number of spin-off effects for onshore or near-shore construction technology. The following know-how and analysis tools for advanced technologies are mentioned, with examples of use for other types of structures:

Know-how on:

- high performance concrete (sub-merged tunnels, any concrete structures designed for long-term durability)
- high-strength normal-weight concrete (long-span bridges)
- high-strength lightweight aggregate concrete (long-span bridges, floating bridges)
- complex slip-forming with change of thickness and change of cross-section shape (towers, silos)
- marine operations in open sea (complex marine transfer and tow operations)
- marine operations in coastal waters (floating bridges, submerged tubular bridges)
- underwater soil mechanics (submerged tunnels)
- evaluation of accidental actions (industrial plants).

Software for:

- finite element analyses (irregular box-shaped bridges)
- dynamic analyses of structures (towers, bridges built by cantilevering techniques)
- static and dynamic wave force analyses (floating bridges, submerged tubular bridges)
- pre-processors and post-processors for structural design (bridges, other complex structures).

The examples illustrate that the offshore concrete platforms have brought the total concrete design and construction technology a substantial step forward, a fact that can be utilized also in related applications of the technology. (Olsen, 1999), (Andrews and Bone, 1998).

References

Ågnes, R. (1997) Concrete for Marine Applications. *CONCRETE a feasible option for offshore construction. Two-day International Conference,* IBC Technical Services, Aberdeen, May 1997.

Andersen, H.W. and Collett, J.P. (1989) *Anchor and Balance.* Det norske Veritas 1964–1989. J.W.Cappelens Forlag A.S.Oslo.

Andrews. J. and Bone, D. (1998) The specification of concrete for coastal defence and marine works. *Concrete,* April 1998. pp. 24–26.

Bech, S. and Carlsen, J.E. (1999) Durability of high-strength offshore concrete structures. *5th International Symposium of High Strength/High Performance Concrete Structures.* Eds. Holand, I. and Sellevold, E.J.Sandefjord, Norway, 1999. pp. 1387–1394.

Brekke, D.-E., Åldstedt, E. and Grosch, H. (1994) Design of Offshore Concrete Structure Based on Postprocessing of Results from Finite Element Analysis (FEA), *Proceedings of the Fourth International Offshore and Polar Engineering Conference,* Osaka, Japan.

Brown, P. and Nygaard, C. (1997) New Generation TLP. *CONCRETE a feasible option for offshore construction. Two-day International Conference,* IBC Technical Services, Aberdeen May 1997.

Chabot, L. (1997) Spar structures—Steel versus concrete. *CONCRETE a feasible option for offshore construction. Two-day International Conference,* IBC Technical Services, Aberdeen May 1997.

Collier, D. (1997) Cost competitive concrete platforms—Innovative solutions for today's market. *CONCRETE a feasible option for offshore construction. Two-day International Conference,* IBC Technical Services, Aberdeen May 1997.

Eurocode 2 European Prestandard ENV 1992–1–1. (1991): Design of concrete structures. CEN 1991 (under revision 1999 for transformation to EN, European Standard).

FIP (1996). *State of the Art Report—Durability of concrete structures in the North Sea.* SETO, London.

FIP (1998) Awards for Outstanding Structures. XIII FIP Congress 1998, Amsterdam.

Fjeld, S. and Morley, C.T. (1983) Offshore concrete structures in *Handbook of Structural Concrete.* Eds. Kong, F.K., Evans, R.H., Cohen, E. and Roll, F., Pitman, London.

Galbraith, D.N., Hodgson, T. and Darby, K. (1993) Beryl Alpha—Condeep GBS Analysis. SPE 26689. *Offshore Europe Conference,* Aberdeen September 1993.

Gérin, M. and Adebar, P. (1998) Filtering analysis output improves the design of concrete structures. *Concrete International.* December 1998. pp. 21–26.

Gudmestad, O.T., Holand, I. and Jersin, E. (1996) *Manual for Design of Offshore Concrete Structures.* Statoil, Stavanger, Norway.

Gudmestad, O.T., Warland, T. Aa. and Stead, B.L. (1993) Concrete Structures for development of offshore fields. *Journal of Petroleum Technology,* August 1993. pp. 762–770.

Helland, S. and Bjerkeli L. (1999) Service life of concrete offshore structures. *Offshore West Africa* '99 *Conference and Exhibition,* Abidjan, Ivory Coast.

Holand, I. and Lenschow, R. (1996) Research Behind the Success of the Concrete Platforms in the North Sea. *Mete A. Sozen Symposium. ACI **SP**-**162.*** Farmington Hills, Michigan, pp. 235–272.

Høyland, K. and Maslia, J. (1999) Removal and recycling of high strength offshore concrete structures. *5th International Symposium on Utilization of High Strength/High Performance Concrete.* Sandefjord, Norway.

Irons, M.E. (1999) Low-Cost Ocean Platform Construction—A Point of view. *Concrete International.* December 1999.

Jakobsen, B. (1992) The Loss of the Sleipner A Platform. Proceedings of the Second (1992) *International Offshore and Polar Engineering Conference.* San Francisco 1992.

Leivestad, S. (1999) ISO Standard for fixed concrete structures. *5th International Symposium of High Strength/High Performance Concrete Structures.* Edited by Holand, I and Sellevold, E.J., Sandefjord, Norway, 1999. pp. 421–426.

Michel, D. (1997) The advantages of floating concrete construction. *CONCRETE a feasible option for offshore construction. Two-day International Conference,* IBC Technical Services, Aberdeen May 1997.

Moksnes, J. (1990): Oil and Gas Concrete Platforms in the North Sea—Reflections on two Decades of Experience. *Durability of Concrete in Marine Environment, An International Symposium Honoring Professor Ben C.Gerwick, Jr.,* University of California.

Moksnes, J. and Sandvik, M. (1996) Offshore concrete structure in the North Sea. A review of 25 years continuous development and practice in concrete technology. *Odd E.Gjørv Symposium on concrete for marine structures.* New Brunswick, Canada.

Moksnes, J. and Sandvik, M. (1998). Offshore concrete in the North Sea—Development and practice in Concrete Technology. *Concrete under severe conditions* 2. E & FN Spon, London, pp. 2017–2027.

Neville, A. and Aïtcin, P.-C. (1998) High performance concrete—An overview. *Materials and Structures,* Vol. 31, pp.111–117.

Norwegian Council for Building Standardisation, NBR (1998), *Concrete Structures, Design rules.* NS 3473, 4th edition, Oslo, Norway, 1992 (in English), 5th edition 1998 (English Edition in print).

Norwegian Council for Building Standardisation, NBR (1999), *Specification texts for building and construction",* NS 3420, Oslo, Norway, 2nd edition 1986, 3rd edition 1999.

Nygaard, C. (1997) Concrete—A potentially schedule competitive option. *CONCRETE a* *feasible option for offshore construction. Two-day International Conference,* IBC Technical Services, May 1997.

O'Flynn, M. (1997) Gravity base structures and jack-up platforms. *CONCRETE a feasible option for offshore construction. Two-day International Conference* IBC Technical Services, May 1997.

Olsen, T.O. and Høyland, K. (1998) Disposal of concrete offshore platforms—Is recycling of materials an acceptable option? *Sustainable Construction: Use of Recycled Concrete Aggregate.* Thomas Telford, London.

Olsen, T.O. (1998) Marine concrete structures. *Concrete under severe conditions 2*. E & FN Spon, London, pp. 1596–1605.

Olsen, T.O. (1999) New generation marine concrete structures. *5th International Symposium of High Strength/High Performance Concrete Structures.* Edited by Holand, I. and Sellevold, E.J.Sandefjord, Norway, 1999. pp 91–98.

Stead, B.L. and Gudmestad, O.T. (1993) A concrete platform for re-use in variable water depths, with varying topside functions and weights. *1993 OMAE—Vol. 1, Offshore Technology.* ASME.

2 Concept definition and project organization

Ove T.Gudmestad, Statoil

2.1 Objectives

The objectives of Chapter 2 are to contribute to:

- give an overview of the requirements for design of offshore concrete structures.
- convey the experiences from prior projects, to those having special interest in offshore concrete structures.
- promote and enhance the confidence in offshore concrete structures.
- give an overview of how to design a concrete platform, an overview which can also be suitable reading for students.

2.2 General description of an offshore concrete structure

Prior to any further discussion regarding design of an offshore concrete structure, references are made to Figures 2.1 and 2.2, which show typical fixed and floating concrete structures, respectively. It is of special importance, for further insight, to recognise the names of the various elements of the structures.

For several typical offshore concrete concepts, floating stability is not achieved if one (or more) of the compartments are damaged and flooded with water. This is representing a line of thinking in design which is *not* common in connection with ship-design. It also means that structural design must be done with particular care. For fixed bottom founded concrete structures the importance of floating stability applies during the floating phases only, as the structures cannot sink after being installed offshore.

Floating concrete structures have to be designed with sufficient safety against sinking, in case compartments facing open sea would be filled with water during operations at the field.

For design of concrete structures the requirements of Section 18 of the Norwegian Petroleum Directorate's "Regulations relating to load bearing structures in the petroleum activities" should be given special attention:

The structural system, details and components shall be such that the structures:

a) show optimum ductile properties and little sensitivity to local damage
b) are simple to make
c) provide simple stress paths with small stress concentrations
d) are resistant to corrosion and other determinations
e) are suitable for simple condition monitoring, maintenance and repair
f) are removable.

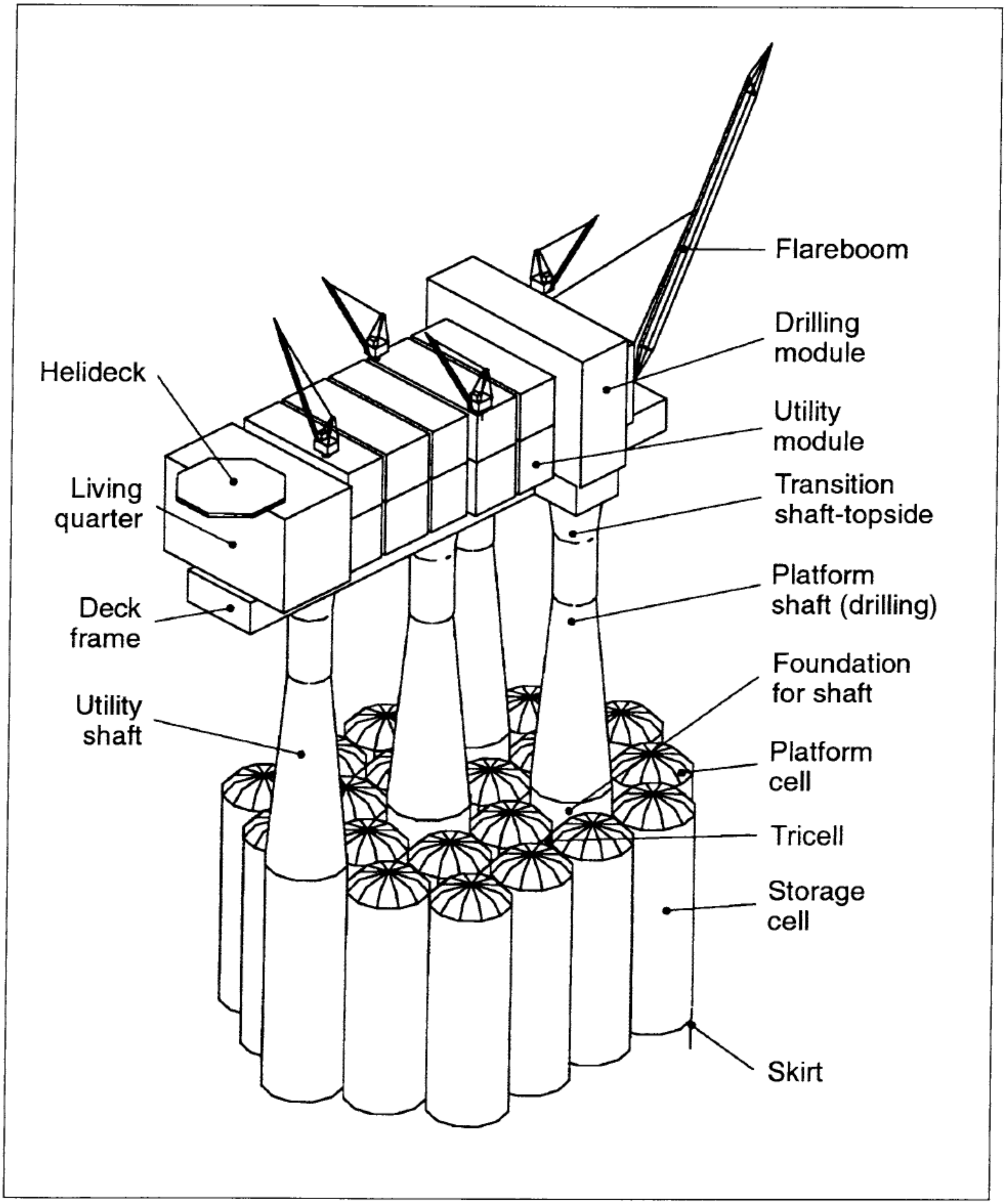


Fig. 2.1 Gravity Base Structure (Gullfaks C platform in North Sea)

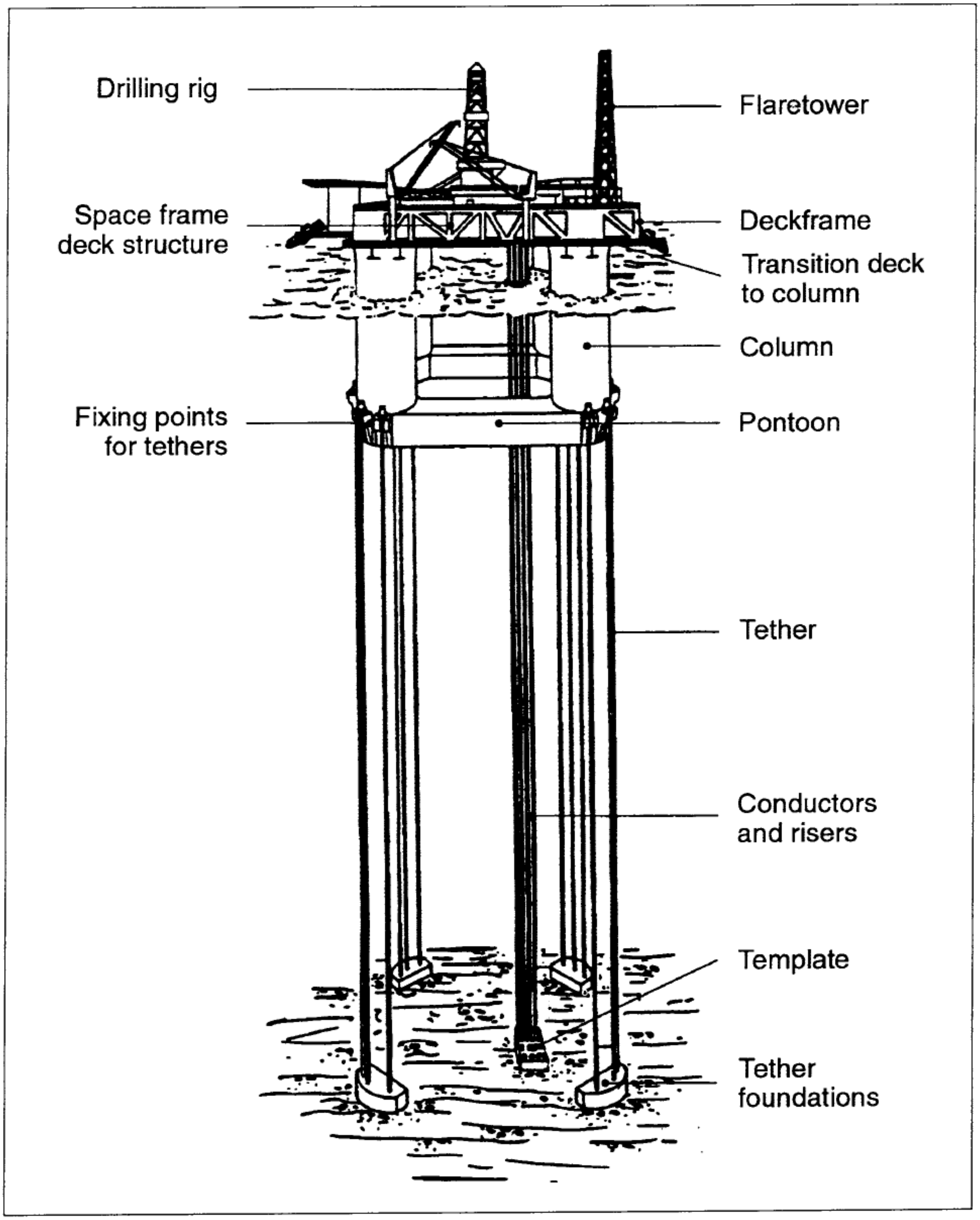


Fig. 2.2 Tension leg platform

2.3 Project phases

During design of an offshore structure it is worthwhile noticing that the work is performed in several project phases with an increasing degree of detail (Fig. 2.3). During the first phase, for example, the advantages of various structures is assessed, and comparisons are made for field developments using various types of structures. As part of the work during the detail design phase, which forms part of the construction phase (not shown in Fig. 2.3), the detailed calculations are made. For concrete structures this includes geometry drawings, rebar drawings, rebar bending schedules, etc. More detailed description of the work in the various phases are given in the following sections; see also Fig. 2.3 and Appendix A.

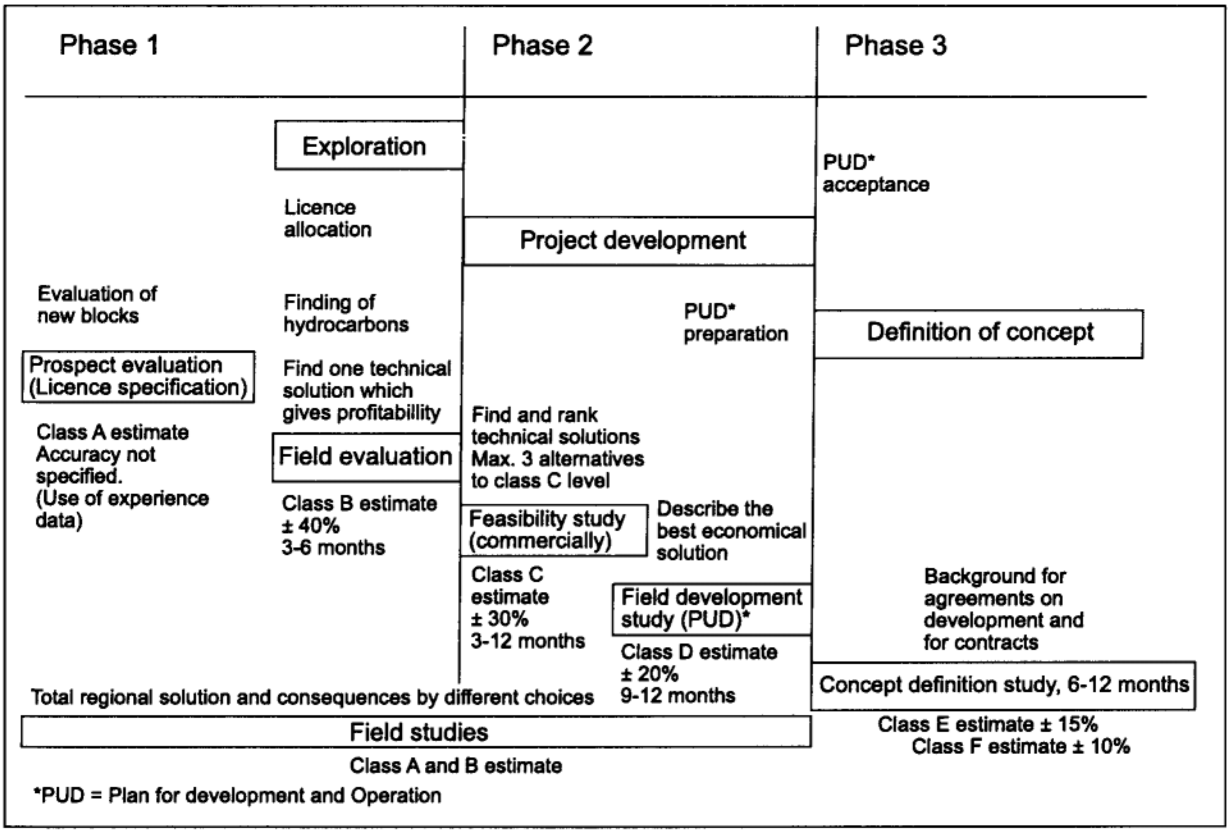


Fig. 2.3 Project Phases for Design of Marine Structures

2.4 Rules and regulations

Offshore concrete structures are to be designed according to national rules and regulations (see Section 1.6 and also (NPD, 1992), (NBR, 1998) and (NBR, 1999).

2.5 Project management

2.5.1 Project planning

(a) The objective of project planning

Design of an offshore structure should be regarded as a project, i.e. a set of tasks to be accomplished within a specified period of time, and with limited resources. Also, there must be a project organisation with responsibility for execution of the project task.

A project is a link in a chain, where the effectiveness and quality, among other things, depend on the interaction between the various links; project employer, project and supplier, Fig. 2.4.

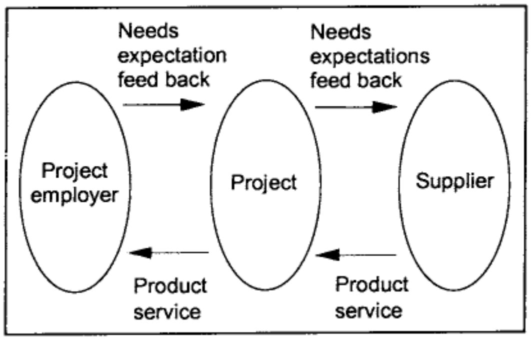


Fig. 2.4 Description of a project as a link in a chain

The purpose of the project planning is thus to:

- distribute responsibility, authority and tasks
- achieve high quality of the project results
- manage resources, time and cost and control the use of them
- reduce the use of double work and unproductive/unnecessary project tasks.

(b) Control activities

To achieve the objectives of the project planning, it is important to establish necessary control activities to ascertain the fulfilment of the objectives (Fig. 2.5).

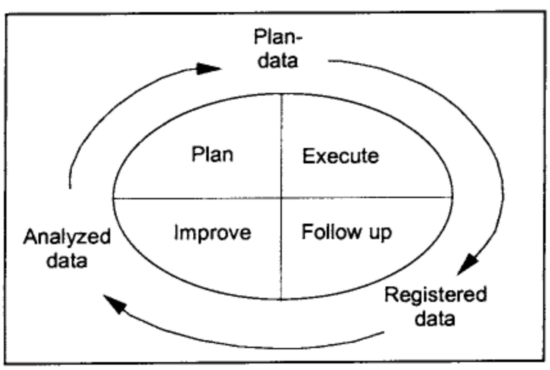


Fig. 2.5 Control activities

The control activities are:

- to establish goals
- establish an activity plan to reach the goals
- control the execution of the project in accordance with the plan
- follow up the execution
- identify and analyse plan deviation
- plan and perform improvements and, if necessary, take care of corrective activities.

Design of offshore structures will be a sub-project within a major investment project. An investment project can be characterised by a high exposure of cost, combined with high uncertainties. The uncertainties are partly linked to the investment cost for facilities and partly to future incomes.

The development of an investment project will last for years, with several decision points (milestones). The project is therefore sub-divided into project phases as discussed in the previous sections of this chapter.

2.5.2 The project control basis

(a) Introduction

The project control basis, Fig. 2.6, can, as a minimum, be defined as:

- work scope
- activity plan (network) with planned progress
- cost estimate (time distributed costs)
- authorisation.

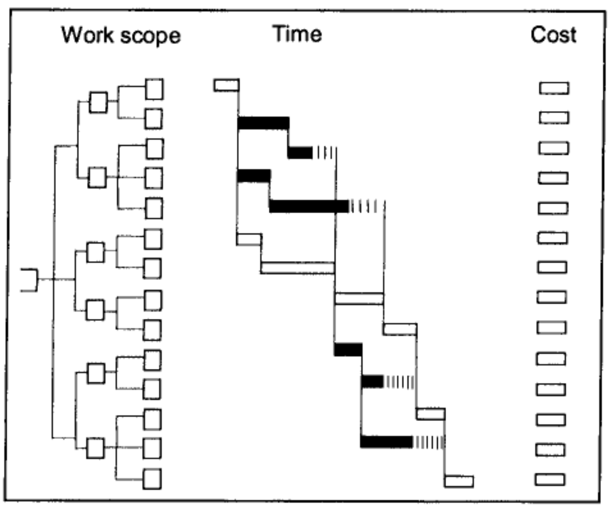


Fig. 2.6 Project control basis

The control basis should be compiled before the start of each of the phases in the investment project.

In addition, the project control basis should define the control parameters influencing the project objective.

The control parameters should be consistent through all the project phases, and should be updated when new information gives grounds for changing the parameters.

The result of the planning process: milestones, resource planning and cost phasing establish an execution plan as control basis for the next project phase.

(b) Project breakdown structure

The project control basis should be broken down according to a standard cost coding system, enabling easier planning and control of the project, such that deviations can be detected and corrective actions implemented.

The cost coding system should make allowances for various requirements, depending on the project phase, i.e. if it is in an early planning phase or in a later project phase (execution).

The cost coding system is designed such that planning data for various project alternatives can be compared and analysed in all the project phases.

The cost coding system will be the foundation for systematically feeding back of experience data and for compilation of time schedules and estimates.

The cost coding (Fig. 2.7) accommodates the following three hierarchy structures:

- Physical Breakdown Structure
- Standard Activity Breakdown
- Code of Resource.

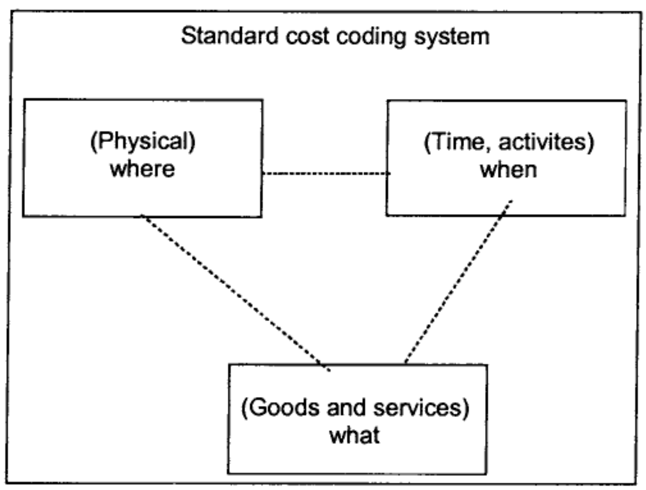


Fig. 2.7 Standard cost coding system

The combination of physical extent, standard activity and resource type gives the foundation for a standard preparation of plans, cost estimates and experience data.

The work scope is broken down into work packages (Fig. 2.8) in the *execution/development phase.*

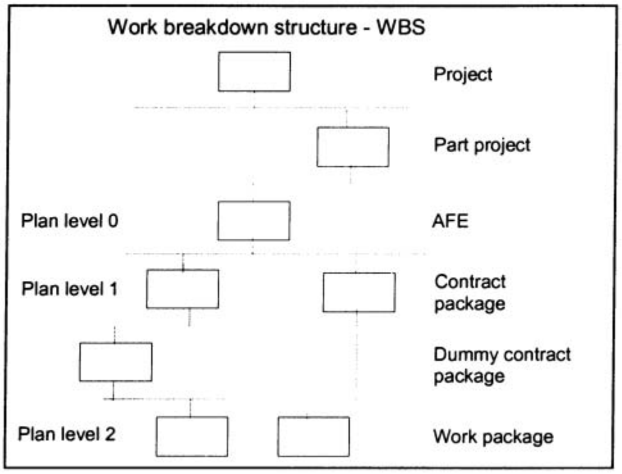


Fig. 2.8 Work breakdown structure (AFE= Authorization for Expenditure)

The breakdown into work packages should take the following into consideration:

- organisation and ownership
- contract philosophy
- supplier marked availability
- work complexity
- interface internally and externally in the project
- method of assessment and control of workmanship.

In the concept definition phase the cost coding of the control basis, in accordance with the Standard Cost Coding System, should be carried on from the project development phase.

During the execution phase, the project control basic is structured in work packages. The control basis is broken down into a level below work packages (planning level 3).

The project defines requirements to suppliers' systems. The requirements should be related to the interface between the project and additional vendors, enabling the individual vendors to use their own systems. The control basis should be possible to aggregate on all levels.

(c) Execution plan

Execution plans for the project shall be prepared with relations and limitations to:

- time
- resource
- cost.

Detailing of the execution plans is dependent on the level of ambitions and on requirement with respect to uncertainties.

The execution plan is part of the project agreement between client and project, and relates to the project's main plan, which forms the basis for project development decision. At all times must progress, milestone achievements and other activities during execution of the project be related to the project's main plan. The execution plans are thus important references for control of the project.

The execution plans should include:

- scope of work (including technical specifications)
- progress plans (including externally given milestones)
- resource plans
- cost estimates (including budgets).

The relation between scope of work, time, resources and cost are linked to the lowest level (planning level 0) in the project's Work Breakdown Structure (WBS) see Fig. 2.8.

(d) Scope of work

The client is responsible for a proper definition of the project's goals, and to ensure that the goals are understood by all parties involved.

The main goal of a project is always to strive for cost/benefit-effect (i.e. to maximise the profit on the invested capital).

The correlation between the various sub-goals for the project and the main goals can be difficult to understand. The project control parameters must therefore be clearly defined, to assure that all involved have got a mutually agreed understanding of common goals, project tasks, assumptions/frame conditions in the entire chain from client to project and to contractor/supplier.

Project agreement. The project goal and the overall control parameters shall be documented in a project agreement. The project agreement shall describe goals and tasks, assumptions and frame conditions, plans and estimates, responsibilities and authorities. The document is prepared by the client.

Contracts. The need for mutual goals and understanding of project scope, assumptions and frame conditions also applies to the supplier for those parts of the project for which he is being given responsibility.

During contract formulation (see also Section 2.8), and following-up of the contract, it must be assured that the project's requirements to management and control systems is met so that project goals can be reached.

By setting contract requirements for quality management and control to contractors/suppliers, the possibility of preventing negative deviations are increased.

(e) Schedule

The overall progress plan forming the basis for execution of the project, is called "Master Control Schedule".

During project execution, deviations will occur, hereby creating the need for schedule revisions, called Current Control Schedule.

The work packages in the project containing volume, time and cost shall be split into work orders, CTRs (cost, time, resource estimates), by the contractor/supplier.

Schedules are normally presented in two ways: A network (Fig. 2.9) containing necessary information about work sequences and logic for the aim of analyses, as well as a Gantt-diagram for presentation purposes of the project, (Fig. 2.12).

Network. The activities' dependence on each other should be modelled in a project network (Fig. 2.9). The level of detailing and complexity in the network model will be determined by the project's complexity, magnitude and requirements for quality and follow up. The network definition comprises of:

- activity dependence with type of bonding
- early start/finish
- late start/finish
- delays/overlaps.

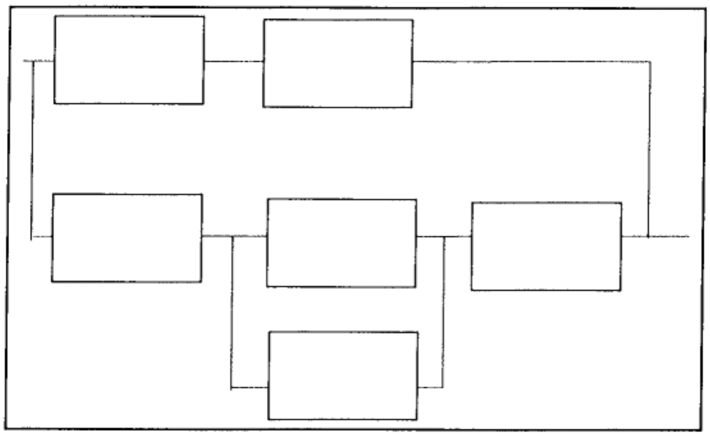


Fig. 2.9 Project network

Analysis and presentation (Gantt-diagram). The final schedule, with built in slack and overlapping activities, should be drawn up and determined from what will overall give the best project economy. The likelihood of meeting the ending date or in-between milestones should also be determined (Fig. 2.10).

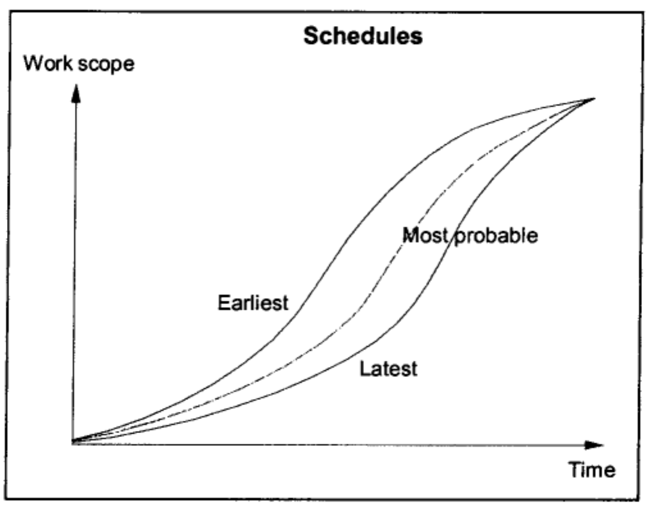


Fig. 2.10 Analysis of progress schedule

The time schedule should be presented in a Gantt chart (Fig. 2.12) with duration, start, finish and slack for each activity. When the work scope of each activity is not clearly stated in the plan, then it should be indicated separately.

The activities' mutual dependence on each other, the network structure and the anticipated use of resources in each activity, should all be well documented.

(f) Resource planning

Resource planning (Fig. 2.11) and follow up should be formalised in a system. A Code of Resource is to be used for each system where it is considered necessary, with respect to cost estimation, duration analysis and physical progress planning.

Guidelines for allocation of resources should be worked out and used for planning, registering and follow up of the physical progress.

Each activity's minimum duration should be defined, together with the required resources and the resulting costs at the same time as constraints from such factors as safety and environmental concerns are satisfied. The use of resources and funds as a function of duration should generally be determined.

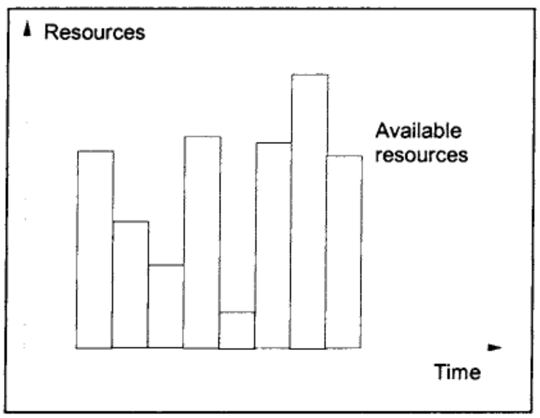


Fig. 2.11 Resource planning

(g) Cost estimate requirements

Cost estimates. The cost estimate is an approximation of the final project costs, based on facts and reasoning. The estimate should be worked out in accordance with the relevant cost coding system for the project phase. Presumptions for a cost estimate, such as:

- scope of work/technical solution
- inflation, exchange rates
- uncertainty
- specific planning competence

should be documented.

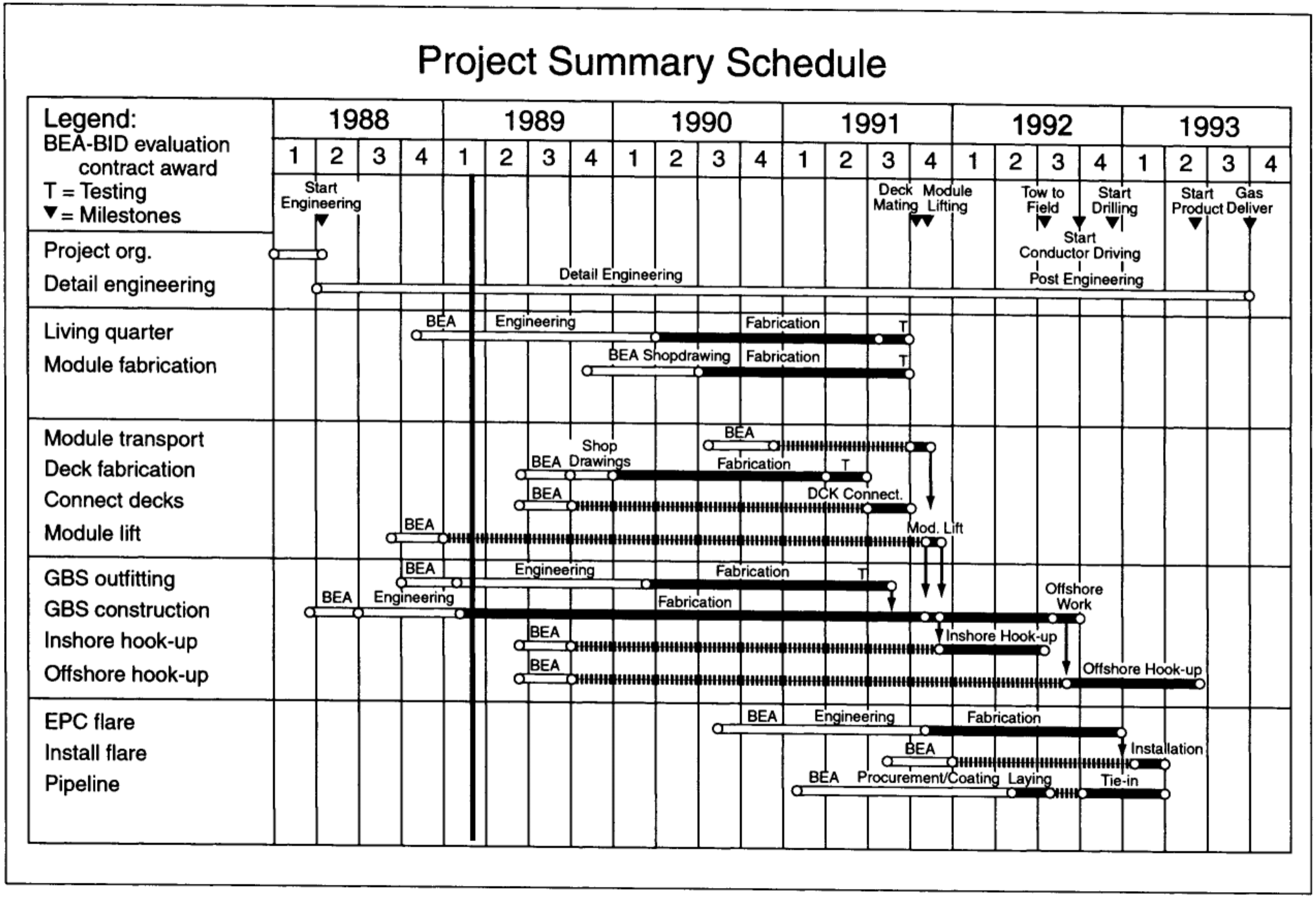


Fig. 2.12 Project summary schedule

Specific planning competence is the project foundation, as defined in the project baseline. The estimation method will depend on how many of the four variables:

- scope
- complexity
- productivity
- price

are declared.

Estimation methods (Fig. 2.13) The estimation method will be selected based on the project phase reached, level of technical definition and access to experience data.

During the early project phases when the extent and complexity of technical definition is limited, the *synthetic method* will be used, i.e. estimation by relations and factors from experience data, main parameters and technical description. The analytic method, i.e. estimation of the all contributing elements directly, where the technical concept is well defined and the scope of work and complexity can be determined, is used in later phases of the project, where the contributing factors can be specified and estimated in detail.

When new concept solutions are proposed, the analytic method will also apply for early phases.

The analytic method shall always be used for project development, concept definition and project execution phases.

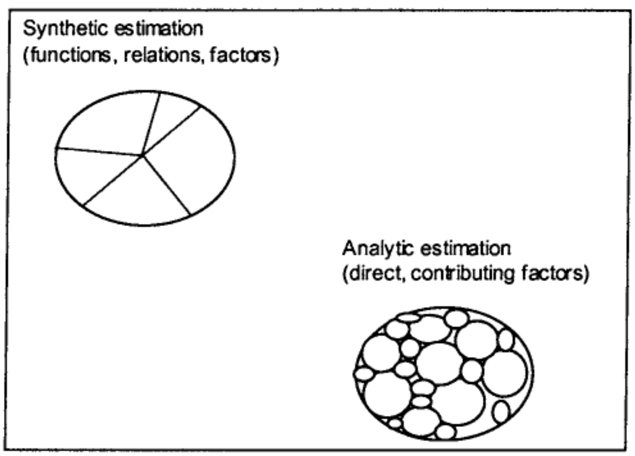


Fig. 2.13 Estimating methods

Uncertainties. Estimates shall present how much resources are needed to perform the project or how much is the cost of the project.

A cost estimate is an expression of what we believe the project will cost. We assume that the basic parameters do not change in the course of the project. The estimate is as such an uncertain value of cost and, if calculated, based on the most possible objective criteria, stating applied norms of estimation, and on professional judgement. The project is estimated reflecting the established work breakdown structure (WBS) and the chosen execution plan.

Assumptions for cost estimation method and the unit cost (productivity figures, unit rates etc.) shall be documented.

The estimation norms are so set that under given circumstances there should be equal probability for result over as under the individual unit rate (50/50 estimate).

An estimate is presented with an expectation value (50/50 estimate, i.e. the value giving the same probability for over/under-run), min/max values and confidence level.

All four variables; scope, complexity, productivity and price are related to uncertainties and they should, dependant on method used when estimating, interpretation of available data, etc., be described by a probability distribution (Fig. 2.14). Simplified, this can be a 50/50 value in addition to the low/high values.

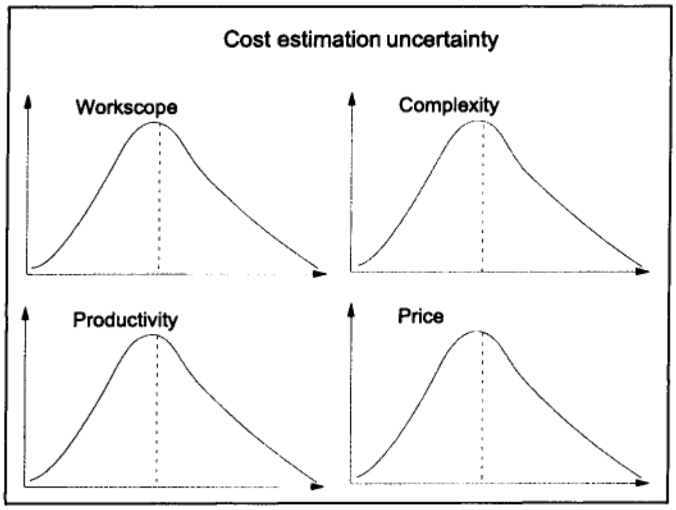


Fig. 2.14 Cost estimation uncertainty

Requirements for cost estimation and schedule classification. Requirements for cost estimation and time scheduling classification is a classification system with defined requirements to:

- basic information, work scope
- estimation method
- level of detailing
- time scheduling
- uncertainties analysis, etc.
- presentation and documentation formats.

The classification requirements shall describe the method for cost estimation and time scheduling, give requirements to technical information needed to perform the planning and the need for the accuracy of the estimate (Fig. 2.15).

Estimation class	Purpose	Accuracy(typical)
Order of magnitude	Prospect evaluations	-
Conceptual	Assess economical potentials (Decision to prepare firm development plans)	+/- 40%
Preliminary	Evaluate/rank the alternatives (Decision to continue the work and Plan for Development and Operation)	+/- 30%
Definitive	Plan for Development and Operation	+/- 20%
Control	Basis for Project Control	+/- 10%

Cost estimates are refined during the course of the project to reflect the additional detail available. A progression of five types of construction cost are normally used; order of magnitude, conceptual, preliminary, definitive and control.

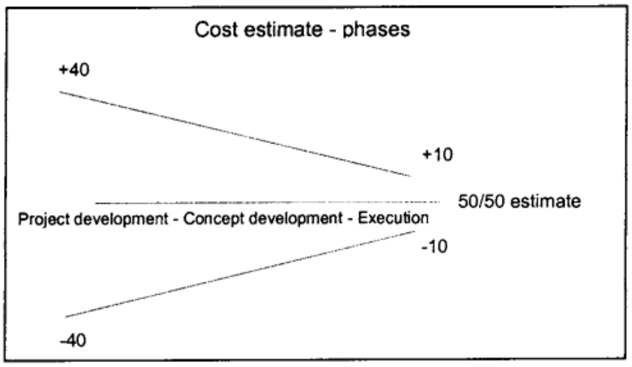


Fig. 2.15 Cost estimate in different phases

(h) Risk assessment

Project risks shall be identified, analysed and responded in order to maximise the results of positive events and to minimise the consequences of adverse events. Risk identification consists of determining which risks are likely to affect the project and documenting the characteristics of each. In project context, risk identification is concerned both with opportunities (positive outcome) as well as threats (negative outcomes). The identified risk items shall be quantified to assess the range of possible project outcomes. Risk response development involves defining enhancement steps for opportunities and responses to threats.

(i) Budget

Budget Estimate. A budget is established administratively through a management decision. The basis for the decision can be a cost estimate, but the budget itself is not linked with uncertainties. The budget is a known, deterministic figure. The budget may have different meanings:

- a cost frame for the project to be kept within
- an expression of the project's expected total cost
- a target figure for the project organisation to reach at.

The different understandings of the budget reflect the extent of authority the project has got in spending money.

Project budgeting. The budget (Master Control Estimate) is normally equal to the project's expected cost (50/50-estimate) at the start of the project (Fig. 2.16). The budget is not changed during the execution of the project unless agreed changes between client and project to scope of work or conditions are implemented. Budget changes are always made based on chosen standards of estimation from the original budget.

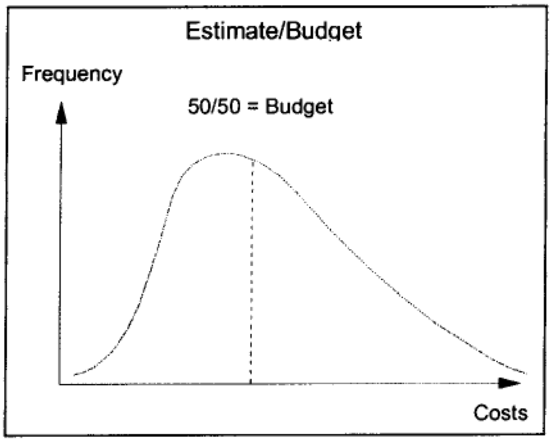


Fig. 2.16 Project budget

2.6 Work during early phases of a project

2.6.1 Early phase activities

In the early phases (i.e. Phases 1 and 2 as shown in Fig. 2.3), the basis is prepared for a good and sound field development solution. Work during the early phases are to ensure:

- that there is an economical potential to continue the project into later phases
- that the technical solutions are robust enough to ensure that minor changes in the design basis, or minor inaccuracies in structural design do not lead to large increases in volumes, dimensions or costs
- that the technical solutions actually dealt with, are feasible within the given budget. Note that the uncertainties in the cost estimates are meant to be gradually reduced during the consecutive stages of the early phase work.

The work in the early phases covers the following phases of a project, (Fig. 2.3):

Phase 1 Exploration

Prospect evaluation
Objective: Develop basis for Application for Concession

Field evaluation
Objective: Identify a combined technical and commercial solution

Phase 2 Project development

Feasibility study

Objective: Reach a decision whether the actual field is commercial and prepare a Report of Commerciality

Field development study

Objective: Describe the best economical solution for field development, and prepare a Plan for Development and Operation (PDO) of the field. In Norway This PDO-document is to be submitted to the authorities for evaluation and for final approval in the Parliament.

During the early phases the possibilities of influencing the development solution and the economical results of the final product is high. Work done in later phases tends more to focus on details, and the extent of documentation increases.

It is important to put forward requirements of what efforts are needed to ensure that enough work is performed, such that:

- the results satisfy the requirements of the phase in question
- the results are sufficient to start the work in the next phase.

As a result of this, a Discipline Activity Model, which describes the required extent of work in the various phases of the study, will be useful. A typical Discipline Activity Model for an offshore concrete structure is included in Appendix A. Note that the Appendix defines the need for analysis in the various phases of the project. Furthermore, it defines how the work shall be quality assured. The Appendix also specifies which reports are to be issued during the various phases.

In all phases, a technical basis shall be developed to serve as a basis for estimates of costs and plans. The technical basis has to build on realistic information about field parameters. Of particular importance for production are parameters like:

- production volume
- number of wells, risers and J-tubes for pull-in of production pipes from subsea wells and from other fields
- weight and layout of the production plant (topsides)
- required storage volume.

Furthermore, field specific environmental parameters are important for assessment of the field development solution. Of special importance is information regarding:

- water depth
- wave height and sea current
- earthquake conditions (in areas with potential earthquakes)
- ice conditions (when relevant)
- geotechnical conditions.

PROSPECT COST EVALUATION PROGRAM-101		14. May -93 INPUT
PROJECT	**Quantity**	**Unit**
TOPSIDES (see also below)		
No. of beds		
Oil production capacity		Sm^3/day
Gas production capacity		MM Sm^3/day
Gas reinjection capacity		MM Sm^3/day
Water injection capacity		m^3/day
Water treatment capacity		m^3/day
Drilling factor (see comment **)		(0.0-1.0)
PLATFORM WELLS & RISERS		
No. of platform wells		
No. of predrilled platform wells		
No. of rigid risers (exp/imp)		
Reservoir depth		metres
Maximum well length		metres
SUBSEA PRODUCTION SYSTEM		
No. of templates		
No. of production wells		
No. of water injection wells		
No. of gas injection wells		
Average distance to wells		km
EXPORT/ IMPORT SYSTEM		
Oil storage volume		m^3
Loading buoy type		
Tie-ins (to exist.installations)		no.of
Length of oil pipeline		km
Length of gas pipeline		km
Length of umbilical		km
SUBSTRUCTURE		
Region (N.Sea=1.Mid.Norway=2etc.)		(1-n)
Water depth		metres
Type (TLP/C=1.TLP/S=2.DCT=3)		(1-9)
CAF/C=4.CAF/S=5.BUOY=6)		
SHIP=7. GBS=8) JACKET=9)		
Oil storage volume (integrated)		
No. of shafts/legs for GBS/Jacket		
Tow-out weight for GBS (relative)		(0.0-1.1)
Jacket inst. met (lifted-launched)		(0.7-1.0)
INVESTMENT PREMISES		
Length of investment period		years
Currency exchange rate		NOK/Curr.
ADDITIONAL PROCESS INFORMATION		
Test separation		0=no 1=yes
*1. stage separation		0=no 1=yes
*2. stage separation		0=no 1=yes
*3. stage separation		0=no 1=yes
Gas recompression		0=no 1=yes
Gas lift		0=no 1=yes
Dehydration		0=no 1=yes
Condensate removal		0=no 1=yes
Gas export compression		0=no 1=yes
Flare tower		0=no 1=yes
Fiscal metering - oil		0=no 1=yes
Fiscal metering - gas		0=no 1=yes
CO_2 removal and disposal		0=no 1=yes

Fig. 2.17 Input to cost estimating programme for estimate in the prospect evaluation phase of the development

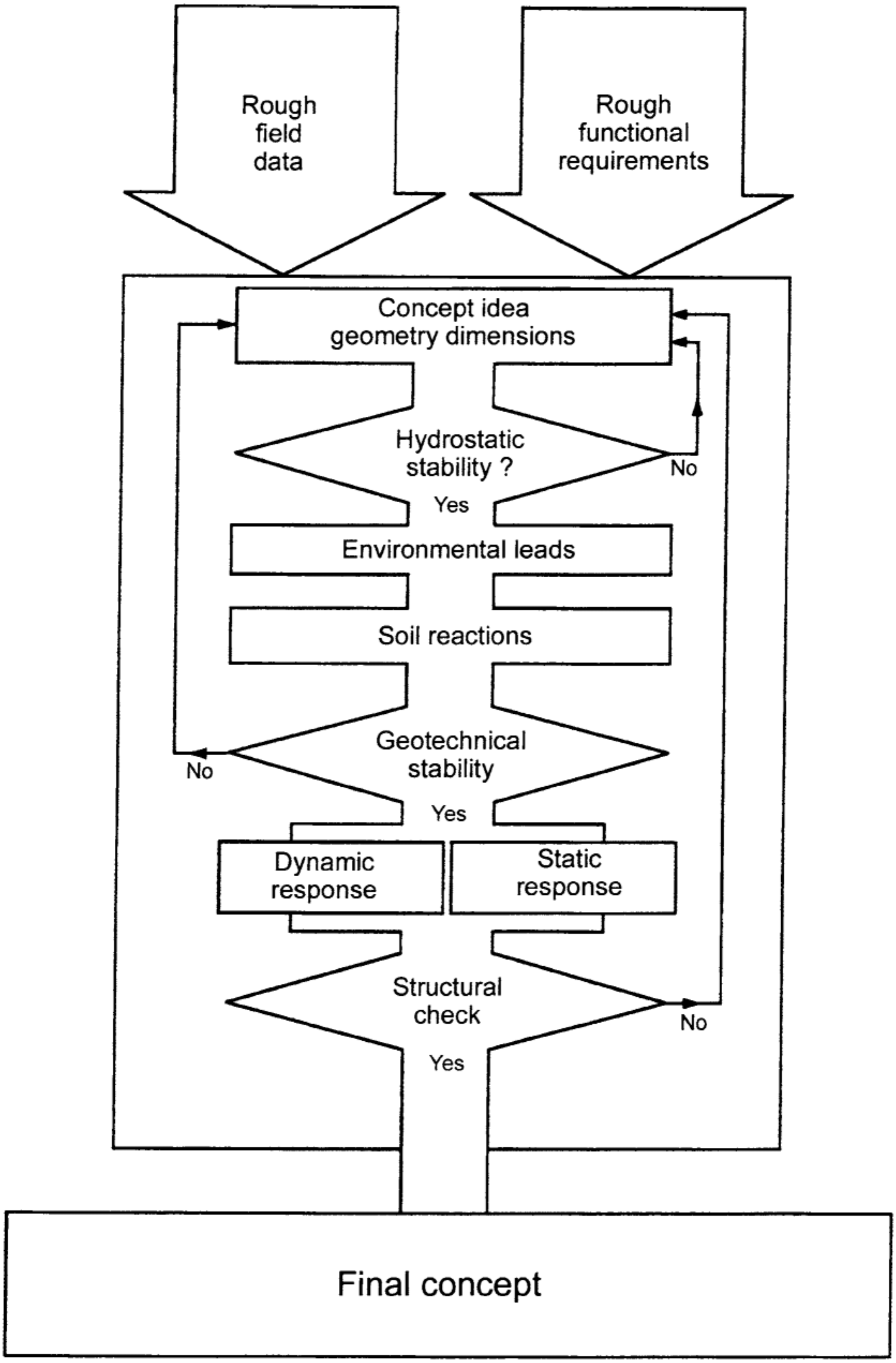


Fig. 2.18 The design process is an iterative process

2.6.2 Selection of concrete structures

Concrete structures are relevant for quite a number of various field developments. It should be noted that:

- fixed concrete structures with short skirts were developed especially for the hard seabed conditions and the large production plants needed for the North Sea during the 1970's to 1980's
- fixed concrete structures with long skirts were developed especially for the soft soil conditions into and in the Norwegian Trench (Tjelta and others, 1990)
- floating concrete structures have been developed over a long period in the 1980's and 1990's. A concrete tension leg platform with concrete foundations have been installed at the Heidrun field. For oil production at the Troll field a concrete floater with chain moorings have been installed.

2.6.3 Design of concrete structures in the early phases, examples

In the *Prospect Evaluation Phase,* data gained by experience are used to evaluate the potentials of new blocks. The work can be simplified by the use of PC-based tools. An illustration of input data needed is shown in Fig. 2.17, see also Appendix A (Discipline Activity Model for Design of Offshore Concrete Structures).

In the *Field Evaluation Phase* there is a search for one development solution which shows profitability. Even if detailed technical studies are *not* performed, it is of importance that the persons performing the job have enough experience such that:

- the actual solution *is prepared thoroughly to assure that the cost estimate* is realistic
- the actual solution has not got too many extra reserves built in such that the economical potential is lost and further work is stopped.

In the *Feasibility Study Phase* the technical solutions shall be ranked. Uncertainties in the cost estimates are to be reduced to ± 30%. In this phase concrete structures are compared with alternative solutions. This represents a special challenge to the concrete designers and forces them to establish innovative solutions. Both oil companies, relevant engineering groups and contractors are to be involved.

Notice that the development work to obtain a competitive concept is interactive, as shown in Fig. 2.18:

- constructive solutions are chosen
- load calculations are made and structural analyses are performed
- structural design is carried out

This is done for the various activities of a development project, including:

- construction
- transportation
- installation
- operation
- removing.

Since the objective of the Feasibility Study Phase is to rank the actual technical solutions, the technical work has to be performed thoroughly. In particular the need for Quality Assurance is emphasised. This is done by engaging highly experienced persons to participate in the studies and to evaluate the technical reports.

In the Feasibility Study Phase it is also relevant to consider new concepts with large potential, which so far only have been developed to a lower degree of detail. Special programs for technology development can be initiated to *qualify* development solutions with much potential.

In some countries where the technology development not yet has reached Western European levels, there is particularly large interest in offshore concrete structures, due to the fact that concrete structures can be built by means of:

- local resources (sand, cement, rebars etc.)
- extensive use of local work force.

Furthermore, the construction of concrete structures may lead to:

- development of local infrastructure (production of cement and reinforcement)
- development of engineering companies and technical know-how.

In a Field Development Study the work in the Feasibility Study Phase is carried to a higher detailed level. A more comprehensive technical study is required, to ensure that the cost estimates (≈20% accuracy) submitted to the authorities, and which also forms the basis for the authorities' decisions, are valid also for the successive phases of the development.

Appendix A (Discipline Activity Model for Design of Offshore Concrete Structures) indicates the level of detail for technical studies relating to an offshore concrete structure. Special attention shall be made to the fact that an acceptable Plan for Development and Operation (PDO) which is submitted to the authorities after the finalisation of the Field Development Study Phase, requires good insight in:

- the environmental loads and other loads like ship collisions, etc.
- the geotechnical conditions
- the interface between the substructure and the production plant (topsides), including the decision of whether transfer of the topside to the platform shall take place inshore or be lifted on at the offshore site.

To ensure that the correct concept is chosen also implies that:

- choice of concrete quality must be made (to assure a robust structure it is suggested that normal density concrete of quality C70 represents the upper limit for this stage)
- choice of routing of flowlines from wells, risers and J-tubes. Large risers for gas are generally routed inside a dry shaft or outside the structure until a location below waterline
- development of criteria for potential use of jack-up drilling platforms must be made.

This decides the layout of the foundation of the concrete structure.

Furthermore it shall be noted that tank-testing can be relevant at this stage, to assure the quality and the feasibility of the structure, see also NPD's "Regulations relating to loadbearing structures", paragraph 30.

In case the PDO-report does not open up for choices of concrete structures, it will normally not be relevant to suggest that concrete structures should be chosen in the later phases of the field development.

2.7 The concept definition phase

2.7.1 Concept report

The goal of the concept definition phase (the concept evaluation phase) is to work out a *Concept Report,* which in turn forms the basis for the detail design and the fabrication contract.

The concept report shall describe the concept in sufficient details to avoid major changes in the successive detail design phase. Furthermore, the concept shall be robust enough to accommodate minor technical changes in the design basis. The concept shall also have built in a certain "forgiveness" to allow for minor inaccuracies. It is, furthermore, needed to emphasise that weight control shall be an important parameter for those working with the topsides, and major changes in the topsides weight must be avoided. For concrete structures it is also of importance that uncertainties in weight, due to the amount of reinforcement and to water absorption in concrete, are included in the concept definition phase. This is to assure that the platform has acceptable floating stability during all temporary phases, and for floating concrete structures also during the operation phase.

The concept report shall also describe important principles for the detail design, and these principles are assumed to be unchanged throughout the construction phase. For instance, the extent of vertical post tensioning of the concrete shafts represents a principle which is defined during the concept definition phase. Other principles of importance which are suggested to be included in the concept definition phase are:

- that the geometry of the concrete is correct. Changes to the geometry (e.g. wall thickness) could lead to large consequences in the detail design phase
- that the concrete quality C70 (for normal density concrete) or LC60 for LWA-concrete (light weight aggregate concrete) are the maximum values applied during the concept definition phase. This will give the project some reserves in highly stressed areas. It is, for example, particularly unfavourable if the amount of reinforcement is significantly increased during the later phases due to the fact that shell walls are too thin in the early phases. It is also very costly

to build a structure where there is barely room enough for placing the required amount of rebars as rebar placement and compaction of the concrete then would become very demanding
- that all openings towards the sea have double barriers in case of damage
- that the lower dome of the drillshafts shall not be penetrated if a conductor-pipe is lost during the construction phase. This applies to the cases when the structure is floating with no damage stability and the lower dome is exposed to dropped objects
- that (during the concept definition phase) it should be aimed at avoiding membrane tension in the concrete walls throughout the section when the structure is to be exposed to the design wave (ultimate limit state with load coefficient of 1,3). This will contribute to a robust structure with only minor cracks and an acceptable fatigue life.

2.7.2 Technical work

The technical work during this phase will be comprehensive, as indicated in Appendix A (Discipline Activity Model for Design of Offshore Concrete Structures); see also (Fjeld and Morely, 1983) and (Tjelta, Aas, Hermstad and Andenæs, 1990). Some special problems which shall be evaluated, are as follows:

- load analysis with assessment of whether special phenomena may be present due to the geometry of the structure
- the need for model testing
- geotechnical conditions and geotechnical stability during installation and operation
- analyses of all phases during construction and installation, specially emphasising floating stability and requirement for stability in case of damage
- designing of all critical sections, with calculation of section forces and moments, including amount of rebars and detailing of the need for pre stressing (see Chapter 3 and Chapter 5). For structures of the Condeep type, the geometry in the tri-cell areas and other joints are to be designed in detail.
- element analyses of critical sections like top of shaft (see Chapter 4)
- the safety of the platform's mechanical systems e.g. ballast systems and the oil company's mechanical systems (flowlines from wells, risers, oil storage system etc.)
- choice of bulkhead sectioning and analyses of floating stability during operation. For floating platforms, specially emphasising how the platform can be designed with acceptable spare buoyancy to allow for some topside load increases in the later phases.

For fixed and floating structures, respectively, the following calculations shall be discussed in depth:

Fixed concrete structures:

- Floating stability, including level of ballast water in all essential stages of fabrication
- Global wave forces, including the requirements for calculation of forces in the shaft base (Morison wave load analysis with Stokes 5th order wave theory)

- Dynamic calculations, including demonstration of the natural response period if "springing" and "ringing" (transient dynamic response) can give a substantial contribution to the global wave forces
- Geotechnical evaluation, both of geotechnical stability as well as of soil structure interaction in conjunction with dynamic calculations, including analysis of soil pressure distribution under the foundation during various load applications Furthermore the possibility of soil erosion shall be assessed.

Floating concrete structures:

- Floating stability, including estimate of the level of ballast for all essential stages of fabrication and operation
- Detailed hydrodynamic calculations of wave forces as well as global responses. In addition, it should be noted that certain stages in connection with fabrication can lead to special design forces in parts of the structure
- Dynamic design, including identification of the various natural response periods where "springing/ringing" can contribute to a substantial part of the global wave forces
- Geotechnical calculations in connection with the mooring systems
- Analyses of the water tightness of the structure to assure that the structure can be safely operated throughout its entire design life.

The oil company normally gets most of this work performed by experienced consultants/ contractors. Concerning development of offshore concrete structures, it is essential that the contractors tendering for construction are allowed to develop competitive structures adopted to their own construction facilities. Thus, the oil company can obtain competitive bids from several competent tenderers.

2.7.3 Quality assurance and verification

It is assumed that the work performed during the concept development phase has been subject to the proper quality assurance, see Appendix A (Discipline Activity Model). The principles described here are used both by the oil company and the external contractors involved.

In addition it is relevant to use external consultants to perform verification of parts of the documentation. For this phase it is recommended to channel all communication related to verification through the oil company (see Chapter 7).

There is also a need for a rough *risk analyses* during the concept definition phase to identify whether the concept has built in major risks during the phases of construction or operation related to:

- loss of human life
- damage to the environment
- loss of values.

Thus, risk reducing efforts can be incorporated. Special emphasis should be made to focus on the following risk elements during the construction phase:

- risk of falling objects
- risk of uncontrolled ingress of water due to pipe failure or malfunctions of the ballast system
- risk of collisions
- risks during towing and installation.

In addition, the accidental design loads applied in the operation phase are to be determined.

2.7.4 Schedules and budgets

An important part of the concept definition phase is to establish detailed *schedules and budgets* for the development project.

2.7.5 Competence requirements

The concept report represents the final basis for a successful performance of a development project. Consequently, it is of the highest importance that all parties involved use their most experienced engineers and make them responsible for the project's concept evaluation phase.

2.8 Project organization phase

2.8.1 Introduction

The project Organisation Phase represents an intermediate phase inbetween the phases of concept definition and construction. Note that detail design is part of the construction phase.

During the project organisation phase the construction work (fabrication) is prepared. Several fundamental principles of technical nature have therefore to be clarified during this phase.

2.8.2 Contract

A contract for the construction phase has to be established in this phase. Included herein is choice of type of contract. The following definitions can be used for important tasks during the construction phase:

E	Engineering work (detail design)
P	Procurement
C	Construction
I	Installation

The contract for construction can include one or more of these tasks. For construction of offshore concrete structures the normal contract form has been to select one contractor for the whole work, i.e. an EPCI-contract. In some cases, though, the oil company has separated the

Main Mechanical Outfitting work (MMO-work) from this contract and placed that part of the scope with a separate mechanical contractor. This split of contract implies that the oil company has to co-ordinate the concrete and mechanical contractors.

In other cases the detail design is also assigned to a separate design contractor. The advantage is that more contractors can tender for construction after the completion of the detail design work. The drawback is that this leaves the oil company as responsible for assuring that the detail engineering is performed properly and that the design is constructable, and issued in due course. Any changes may lead to claims for costly change orders.

Most of the arguments lead to the preference of one EPCI-contract for detailed design and construction of an offshore concrete structure. Such a contract puts the responsibility on *one* contractor. The oil company has in advance qualified the tendering contractors. The judgement regarding qualifications are based on technical competence, experience and financial strength. Of special importance is to have documented that the project can be performed according to schedule.

With such a comprehensive contract format as described above, it is of paramount importance that the contract clarifies essential relations between the oil company and the contractor. This applies to *inter alia:*

- list of specifications according to which the work shall be performed
- required detailing of contractor's documentation and work, including detailed specifications of the basis for the work and requirements for co-operation with the oil company. Especially, it is regarded important that the oil company has the right to take direct control over the amount of reinforcement (payment by unit rate)
- requirements for efficient organisation of contractor's work, to ensure that safety, environment, cost and progress are secured. Included in this is clear definitions of "who to do what" in contractor's organisation, and of which subcontractors be accepted and how they are managed
- requirements to contractor's technical know-how, including requirements to the involved persons. Special requirements must be set to contractor's technical management and their co-ordination of the technical work
- requirements to the contractor's internal control
- requirements to co-operation between the contractor, the oil company and the consultant performing the third party verification, including agreement with the contractor as to how comments from the third part verification shall be duly incorporated
- procedures for reporting of results, progress, problems and deviations
- agreement of milestones for revisions, and agreement of the oil company's right to claim revisions as and when it finds it necessary. This also applies to revisions of subcontractors
- agreement on commercial relations.

Special emphasis is to be made such that the contractor shall follow the Concept Report issued by the oil company during the concept definition phase. It is assumed that the oil company has established a thorough concept report which includes:

- robustness to withstand minor changes in the design basis, for instance increased weight of the topsides within ±5%
- "forgiveness" to avoid major changes to the concept, if minor omissions are made in the concept definition phase which lead to changes in dimensions and weight.

This does, however, not mean that safety margins shall be applied beyond the requirements of the national rules and standards. Arrangements must be laid down in the contract to ensure that all changes to the concept report shall be agreed by the oil company and treated according to specially agreed procedures.

During contract preparation and negotiations it is important that the oil company's project group has high technical expertise and long administrative experience.

2.8.3 Quality assurance plan

During the Project Organisation Phase the oil company must prepare a plan for how to perform quality assurance of the work during detail engineering and construction. As to procedures and some available tools for quality assurance, see Chapter 6.

2.8.4 Verification plan

A plan for verification has to be developed. The plan shall include:

- description of the extent of work for the verification, emphasising special critical areas of the structure
- requirements to the oil company's own verification of contractor's work, with description of the follow-up work during the detail design and construction activities
- requirements to the contractors' internal quality control and surveillance
- working methods and work description valid for the consultant performing the third party verification
- requirements for prompt implementation of the results from the verification.

The choice of strategy for external verification is of special importance. The oil company must have direct contact with the third party verification consultant. Payment should be made according to spent hours within agreed limitations. Chapter 7 details two models for external verification:

1. the oil company submits the results from the verifying consultant to the contractor
2. company is managing the verification, but the verification consultant communicates mainly directly with the contractor.

Regardless of which model is followed, the verifying consultant is obliged to follow a tight schedule not to delay the progress of the contractor. The oil company must take all formal decisions in case disagreements arise between the contractor and the third party verificator. The oil company should also be the single point contact with the authorities and the partners.

2.8.5 Preparation for the organization of the construction phase

During the process of preparing for a development project, project organisations are built up within the oil company and the contractor. The oil company is to ensure that both his own personnel and those of the contractor have the acceptable competence. This is partly done by specifying competence requirements for every leading position in the project. Every leader must have thorough knowledge of the technical content of the activity he is meant to lead. It is presupposed that the project is organised to assure openness with respect to technical questions, and that the requirements for quality in performance is characterising the dialogues and feed backs. No project philosophy which suppresses technical problems should be allowed

In offshore field development projects, key persons who have worked during earlier phases in the same project should be brought in, so that continuity in the technical work is maintained. All staff in a project must agree to the project's goals and the procedures applied to reach these goals.

References

Fjeld, S. and Morely, C.T. (1983) Offshore Concrete Structures, in *Handbook of Structural Concrete.* Eds Kong, F.K., Evans, R.H., Cohen, E. and Roll, R, McGraw Hill.

Norwegian Council for Building Standardisation, NBR (1999) *Specification texts for building and construction,* NS 3420, Oslo, Norway, 2nd edition 1986, 3rd edition 1999.

Norwegian Council for Building Standardisation, NBR (1998), *Concrete Structures, Design rules.* NS 3473, 4th edition, Oslo, Norway, 1992 (in English), 5th edition 1998 (English edition in print).

Norwegian Petroleum Directorate, NPD, *Rules and Regulations for Petroleum Activities,* New edition issued every year by the Norwegian Petroleum Directorate, Stavanger, Norway.

Norwegian Petroleum Directorate, NPD (1992) *Regulations relating to loadbearing structures in the petroleum activities,* stipulated by the Norwegian Petroleum Directorate, Stavanger, Norway.

Tjelta, T.I., Aas, P.M., Hermstad, J. and Andenæs, E. (1990) The skirt piled Gullfaks C platform installation. Paper OTC 6473. *Proceedings Offshore Technology Conference,* pp. 453–462, Houston, Texas.

3 Simplified analyses

Tore H.Søreide, Reinertsen Engineering

3.1 Introduction

This Chapter deals with simplified calculation schemes for use in the engineering of marine structures, as an alternative to complex computerized techniques for response analysis. The main objective of the development of analytical techniques is to come up with a tool for early estimates of dimensions, prior to the start of the process of detail engineering. Furthermore, simplified methods, either by hand or based on spreadsheets, are also useful for the control of the complex scheme of global response analysis.

The complete analysis set-up is shown below in Section 3.2, which demonstrates that for simplified response analysis there is a need both for global and local models for capacity control.

As a basis for the evaluation of calculation methods, the loads are to be classified in accordance with the characteristics of their impact on the structural system. Section 3.3 gives a brief introduction into basic dynamics, which is also relevant when deciding the type of analysis in the global response model.

Section 3.4 presents simplified analytical techniques to be used for fixed gravity based platforms. The global response is calculated by modal techniques which keep the number of parameters to a minimum.

The analysis schemes for floating marine structures are given in Section 3.5, where catenary anchored as well as tension leg platforms are dealt with. Formulas are depicted for the analysis of first order wave effects, ringing effects as well as hydrostatic stability. Section 3.6 considers ship impact and presents methods for global response analysis.

Section 3.7 handles second order geometric effects in design, including shafts and planar walls as well as cylindrical cell walls. The geometric effects from finite rigid body rotations of floating structures are also illustrated. The problem areas dealt with for floating marine structures are also relevant for fixed structures during fabrication, tow and installation, especially the hydrostatic stability calculations, built-in forces and skew ballast.

3.2 Analysis activities

3.2.1 Analysis for detail design

This section describes the process of analysis during detail design. The purpose of this presentation is to show at which stages in engineering simplified methods are relevant as a supplement to complicated finite element response models.

Fig. 3.1 illustrates the complete scheme for analysis and capacity control. The term analysis here means the global response analysis for the specific design situations. The load effects are stresses, stress resultants and displacements. The analysis models cover all stages of fabrication, mating, tow, installation and operation. The finite element model of the raft is to be coupled to the topside model for response analysis of the completed structure.

The design activity includes the combination of results for the individual basic load cases as well as load combinations for capacity control. The input both for analysis as well as for control is taken from the Design Basis document (see Chapter 5).

Fig. 3.2 gives the analysis procedure normally applied for the load effect from first order waves. A panel model is generated from the wet part of the raft (below mean water). Regular waves with unit amplitudes and different directions and periods form the basis for the stochastic extreme value estimation of stress resultants. The response from the stochastic analysis is now represented by a regular wave with the same response value, often termed the design wave. This procedure enables the phase between section forces to be simulated, though in the form of one single regular wave.

The process of design wave evaluation ends up strictly with one design wave per response parameter. In practical design, however, emphasis is normally given to group design waves for several responses, ending up with a limited number of basic wave load cases. The panel pressures and rigid body accelerations from the hydrodynamic model go into the global stress analysis model, by which a quasi-static analysis scheme is followed.

3.2.2 Simplified analysis scheme

As a supplement to the rather complex global analysis of load effects, the simpler analytical formulas can be applied in order to produce early estimates of section forces and corresponding dimension controls. Basic structure mechanics knowledge is essential for the creation of the analytical models, where rigid body dynamic effects should also be included. A limited number of governing load situations are implemented, and insight into the response characteristics of the structural system is vital for safe selection.

For the global analysis, hand calculations can be used, or a frame model is made of the raft. The stiffness and mass characteristics of the topside are to be implemented, as well as possible foundation characteristics. For gravity base marine structures of moderate height, the dynamic motions are normally of minor importance, so that a pure static response analysis is sufficient.

The above calculations provide the static and dynamic stress resultants created by the global load carrying behaviour. Local effects from water pressure such as membrane and bending stresses in shaft and cell walls, are calculated either by analytical shell theory, or alternatively by a limited finite element model. In the case of such local finite element models being used, the boundary of the element mesh with load or displacement restrictions is to be put in some distance from the critical sections under consideration.

Fig. 3.3 illustrates the flow of calculations by means of a simplified procedure. A frame model is established and the topside connection is included. At the same time the process of local finite element analyses is set, with input from the global calculations of the boundary forces.

Load combinations, as well as the estimates of design section forces come in subsequently, in accordance with the Design Basis document and possible specifications from the operator.

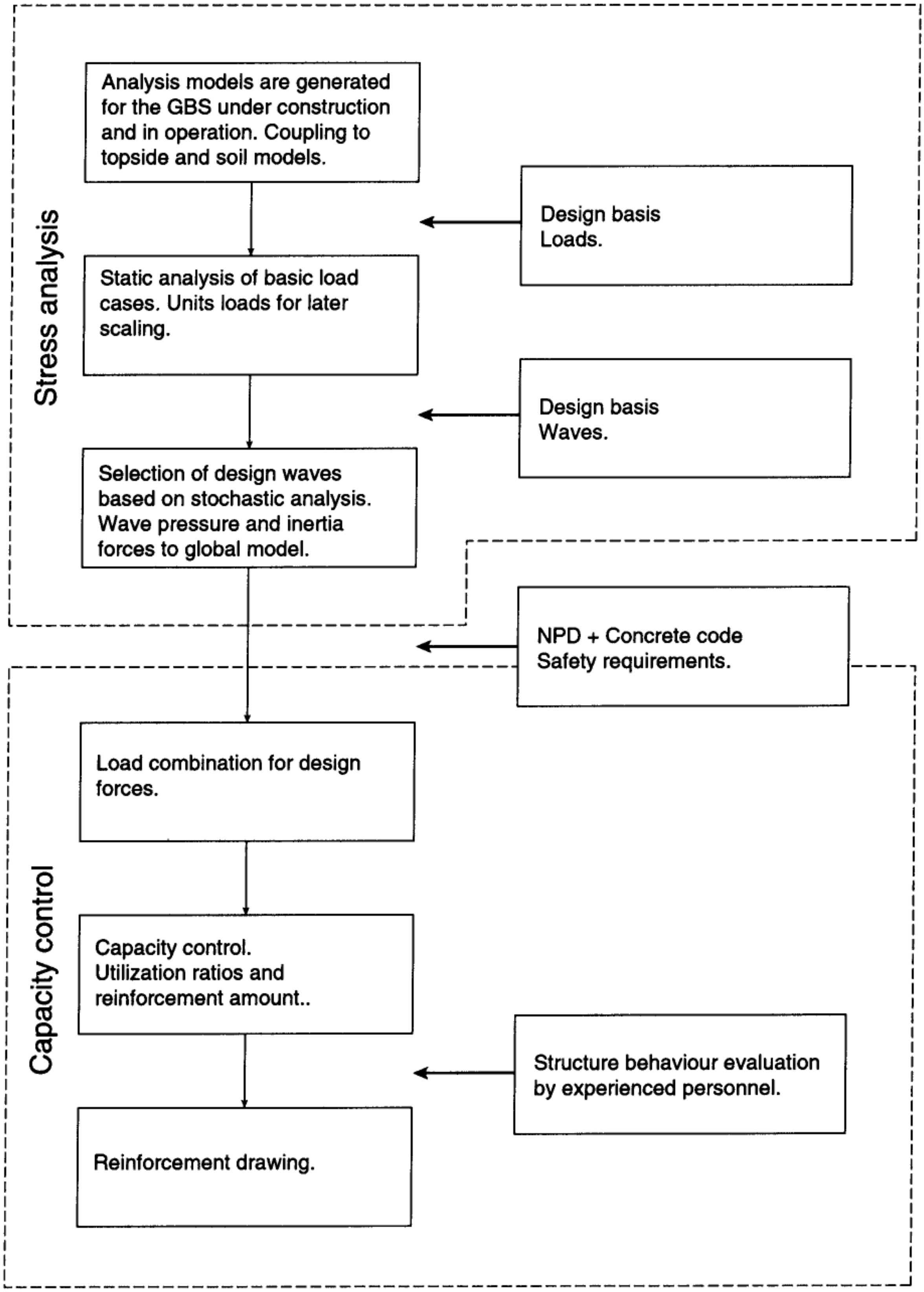


Fig. 3.1 Analysis and capacity control by computer program

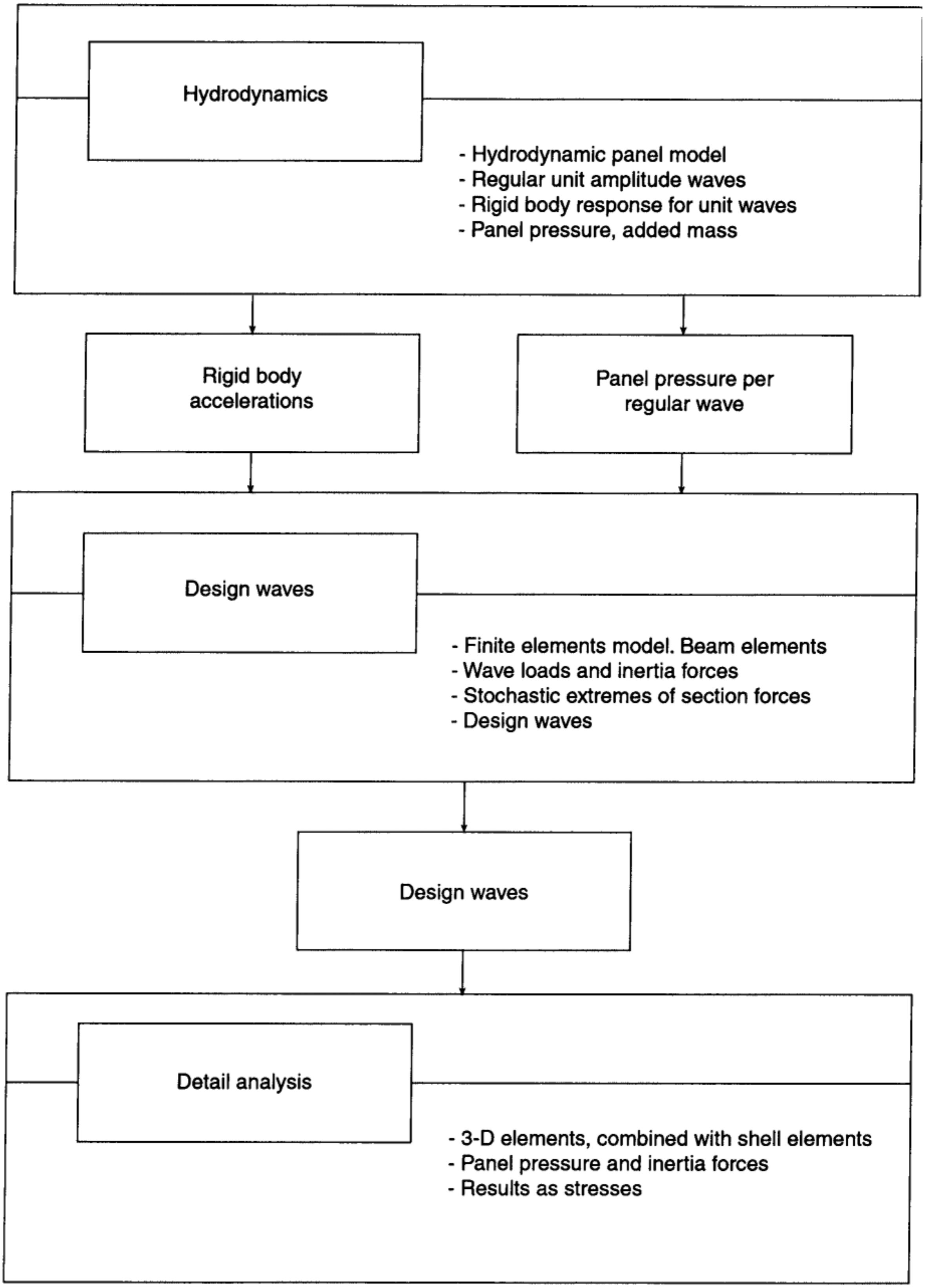


Fig. 3.2 Scheme for first order wave analysis

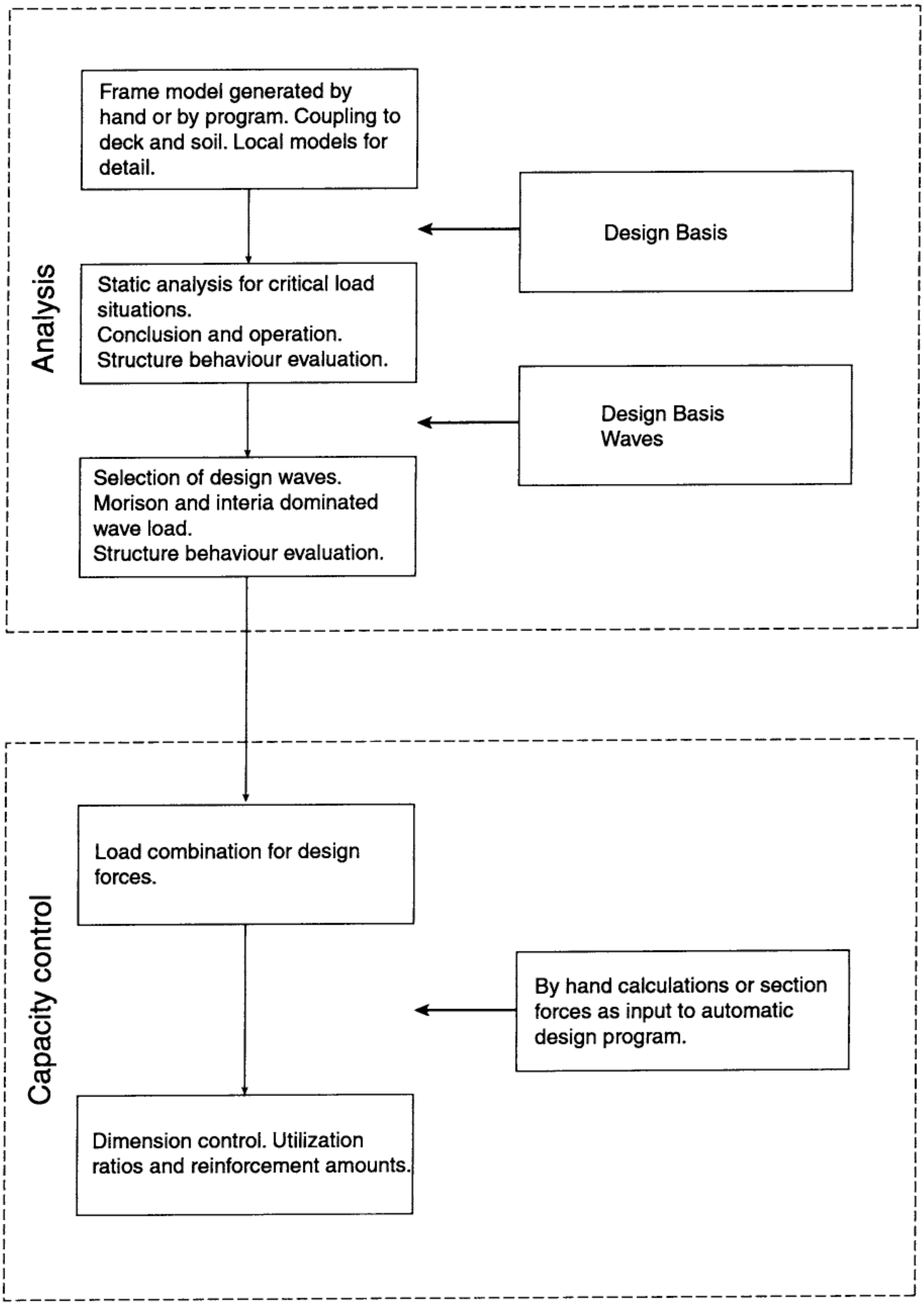


Fig. 3.3 Simplified response analysis and design

3.3 Classification of load effects

3.3.1 System analysis

Prior to the activity on final analysis models, a system analysis is to be made as a basis for the subsequent selection of analysis models. The main objective of this evaluation of response characteristics is to sort load effects into respectively static and dynamic classes of response. The characteristics of structural changes during fabrication and installation and the evaluation of load effects must produce relevant response models for all stages. It is convenient to separate the displacements of the system into rigid body motion and deformation modes, respectively.

The system analysis is to be documented. Experience from larger projects proves that all personnel involved in the engineering team benefit from a presentation of the system analysis as a basis for their considerations concerning design load situations. A description of the structural load carrying behaviour also makes the control of the global analysis results easier for the engineering personnel.

3.3.2 Load effects

Once the structural system is determined, the different loads are to be categorized in accordance with their influence on the structure. This will reveal if static response can be applied, or a dynamic model is needed. As a basis for this evaluation of response characteristics, the natural frequencies of the system should be made available, either by an element analysis, or alternatively, from simple analytical estimates.

As an example of categorization of loads, reference is made to the Norwegian Petroleum Directorate (NPD) regulations, see Section 1.6.1. The following load types normally imply static response analysis:

- Dead load (permanent load)
- Ballast (variable load)
- Prestress
- Hydrostatic pressure (permanent load)
- Tide (variable load)
- Current (variable load)
- Mean 10-min. wind (variable load)
- Built-in forces (permanent load)
- Imposed deformations (deformation load, temperature, shrinkage).

For the cases of wave load response and for impacts, dynamic effects are to be included. Dynamic wave analysis also implies the consideration of the fabrication stages, for which the deformation modes may be flexible with low natural frequencies close to the highest wave frequencies (0.5–0.2 Hz).

The upper and lower outer limits for the dynamic response characteristics are as follows:

Stiffness dominated

The frequency of the load is low when compared to the modal frequencies of the structure:

$$KX = R(t) \tag{3.1}$$

Inertia effects are neglected, as for a fixed gravity base platform in a permanent situation.

Inertia dominated

The load frequency is high when compared to the natural frequencies, resulting in dominance from inertia forces in the dynamic equilibrium:

$$M\ddot{X} = R(t) \tag{3.2}$$

This is often the situation with impact loads. For a catenary anchored floater, all six rigid body motions may be inertia dominated, while for a tension leg platform, the surge and sway directions of motion are inertia dominated.

Between the two above outer limits, stiffness, mass and damping govern the load effects. For local stress control, static analysis can normally be used.

3.4 Gravity base structures

3.4.1 Model for global response analysis

This section deals with a simplified model for global response analysis of gravity base concrete structures, which combines the contribution of the rigid element displacement of the structure and the beam effect of the shafts. The actual load effects are displacement and acceleration of the deck, and alternatively, beam moment and shear at the base of the shaft.

Fig. 3.4 points to several factors that need to be evaluated prior to running the global analysis model. These factors include: stiffness characteristics, mass motion and soil damping, as well as the deformation characteristics of the caisson cell walls that influence the degree of clamping at the base of shaft. The deck connection to the top of shafts affects also the moment and the shear force in the shafts. A total understanding of load distribution in the deck and the structure is necessary. Interaction with the surrounding water shall be accounted for. Fig. 3.5 gives an illustration of load, mass and damping that are included in the dynamic model.

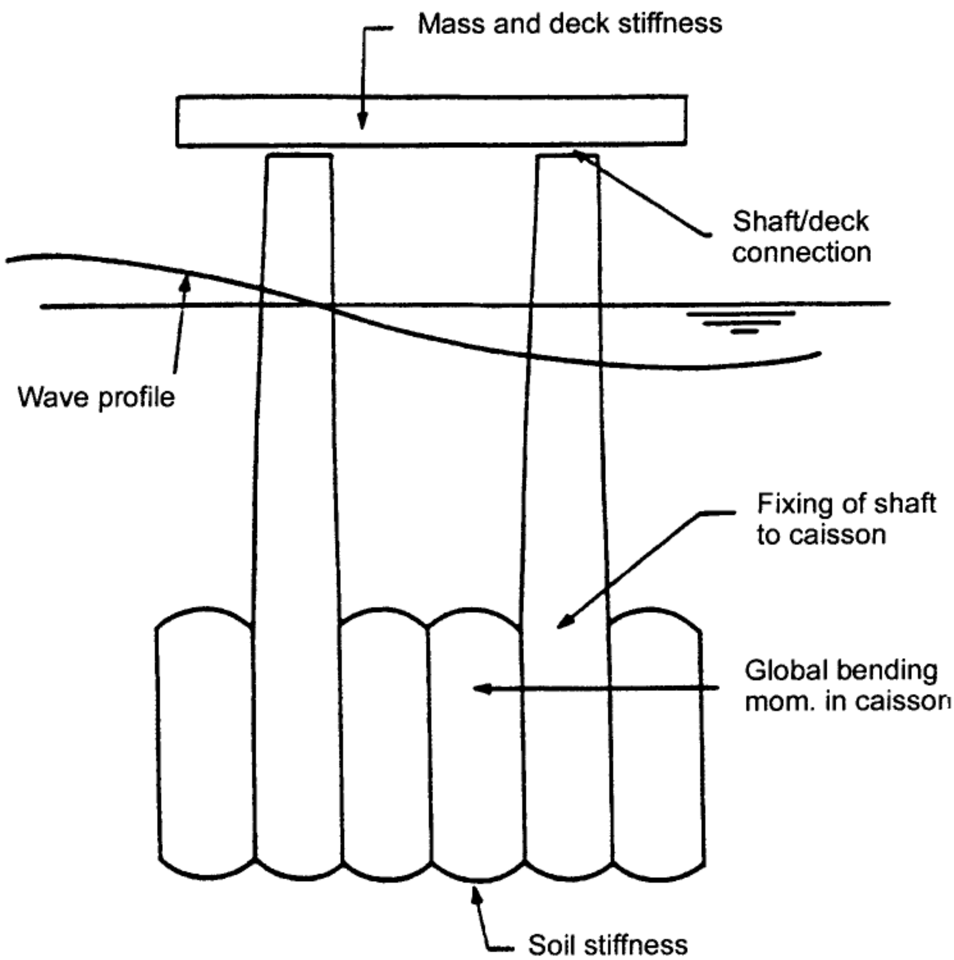


Fig. 3.4 Simplified model

There are now two ways to perform a simplified global analysis: either by a simplified element model (beam elements for shafts and shell elements in the caisson), or by hand calculations.

The following procedure gives a rough indication of these approaches:

A. The analyses should include all critical phases such as construction, towing, installation and operation.

B. For each phase an eigenvalue analysis is performed with due consideration of water mass.

C. If the natural period of the structure is substantially below the load period interval, a quasi-static calculation is performed by neglecting the mass contribution.

D. When masses are believed to influence the behaviour of a slender structure they are taken into account.

Further illustrations are based on simple hand calculations.

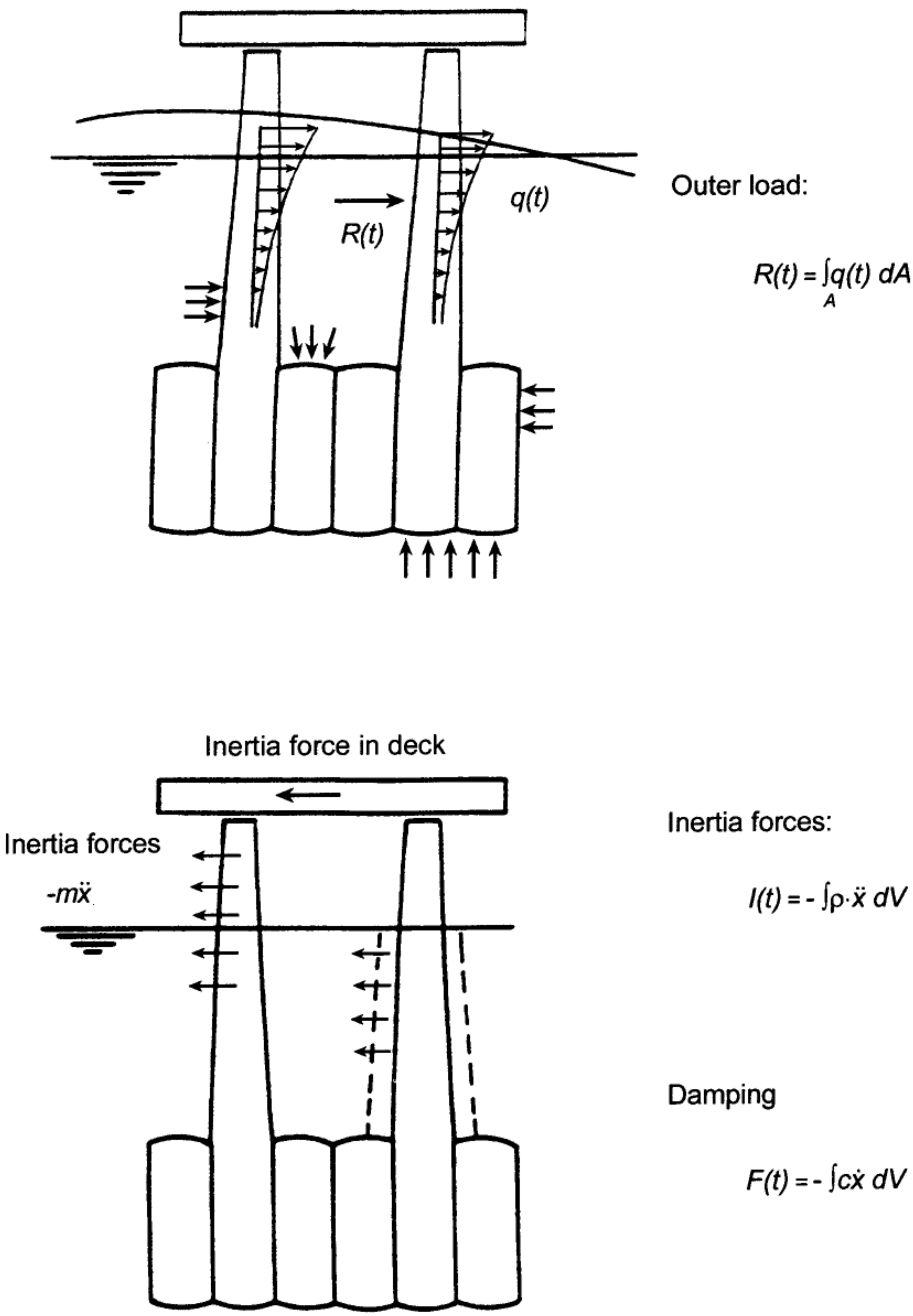
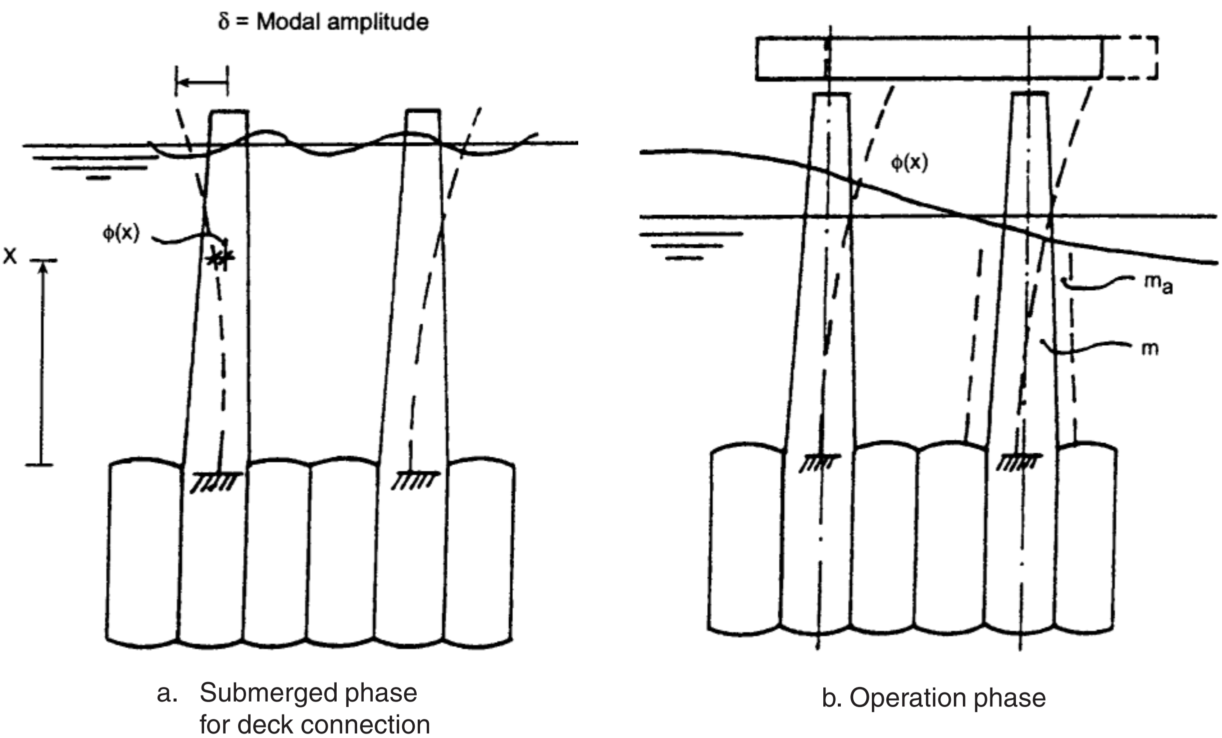


Fig. 3.5 Global dynamic effects



Simplified equations for one degree of freedom:

Modal stiffness

$$K = \int_{\text{Struct.}} EI \cdot \phi_{,xx}^{2}(x)\,dx \tag{3.3}$$

Modal mass:

$$M = \int_{\text{Struct.}} m_s \cdot \phi^2(x)\,dx + \int_{\text{Water}} m_a \cdot \phi^2(x)\,dx \tag{3.4}$$

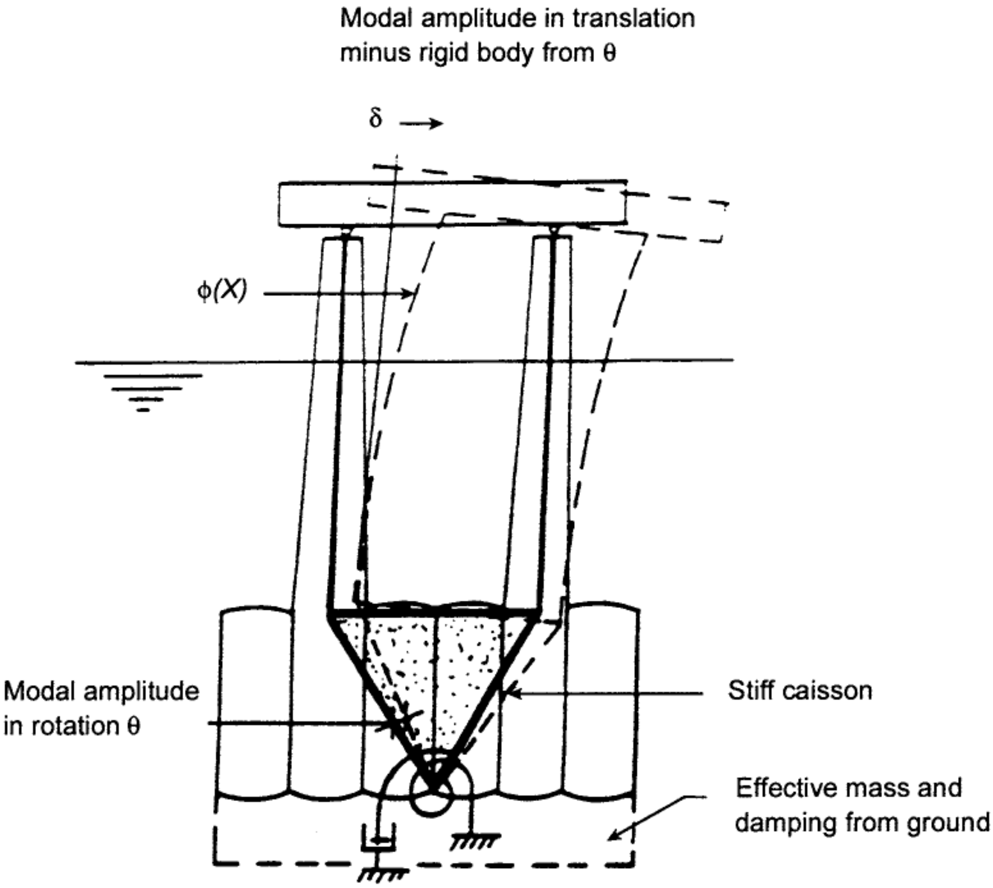
Eigenfrequency:

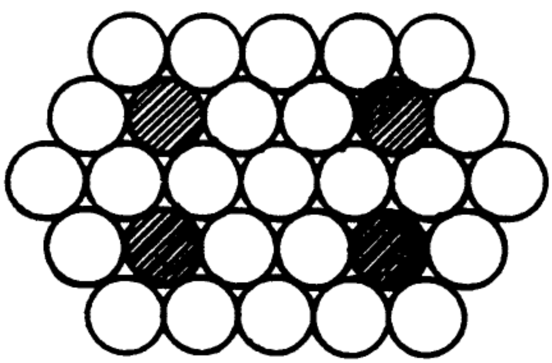
$$\omega_0 = \sqrt{\frac{K}{M}} \quad (\,rad\,/s) \tag{3.5}$$

where

$\varphi(x)$	=	assumed deformation
$\varphi,_{xx}$	=	curvature
m_s	=	mass of structure including ballast water
m_a	=	additional mass from surrounding water.

Fig. 3.6 Simplified eigenvalue calculations





The two degrees of freedom in the calculations are the displacement δ of the shafts and rigid body rotation θ of the caisson.

Fig. 3.7 Global model including ground interaction

Fig. 3.6 shows a model of the submerged phase just prior to coupling the deck to the structure and also a model in the operation phase. In both models the shafts are assumed to be clamped at the top of the caisson. For the submerged phase, it is necessary to compare the natural frequency with the wave period at the actual location. In protected water, such as in a fjord, the values are somewhere between 2 and 6 seconds. Clement weather is needed for coupling operations.

In the operation phase the composite action of the deck and the structure is considered. In Fig. 3.6b a simply supported connection is indicated between the deck and the shaft (situation just after coupling). A fixed connection is then established by stressing cables and grouting the space between the deck panels and the shaft. An indication of the deck stiffness compared to the shaft stiffness can be obtained by calculating the modal stiffness in the same mode for the deck and the shaft.

In Fig. 3.7 an alternative global model is shown, where the caisson is assumed to be stiff, but the stiffness, the mass and the damping are included for the ground.

The general equations for the elements in modal stiffness, mass and damping are:

$$K_{ij} = K_{Soil} + \int_{4 \cdot Shaft} EI_s \cdot \phi_{i,xx} \cdot \phi_{j,xx} \cdot dx \qquad (3.6)$$

$$M_{ij} = M_{Soil} + \int_{Struct.} (m_s + m_a) \cdot \phi_i \cdot \phi_j \cdot dx \qquad (3.7)$$

$$C_{ij} = C_{Soil} + \int_{Struct.} (c_s + c_a) \cdot \phi_i \cdot \phi_j \cdot dx \qquad (3.8)$$

where:

m_s	=	mass of structure including ballast water
m_a	=	additional mass of surrounding water
C_k	=	damping in structure
c_a	=	damping from surrounding water.

In relation to equations in Fig. 3.6, Fig. 3.7 includes rigid body displacement of shafts and caisson. It is appropriate to use a two-degrees of freedom system, where the modal amplitudes are, for example, the rotation of the caisson and the horizontal displacement of the top of shafts. The displacement pattern with two degrees of freedom may be described by the global modes (see Fig. 3.8):

$$u(x) = \theta \cdot y \text{ for the caisson}$$

$$u(x) = \theta \cdot (x + B) + \delta \cdot \left(\frac{x}{H}\right)^2 \text{ for the shafts}$$

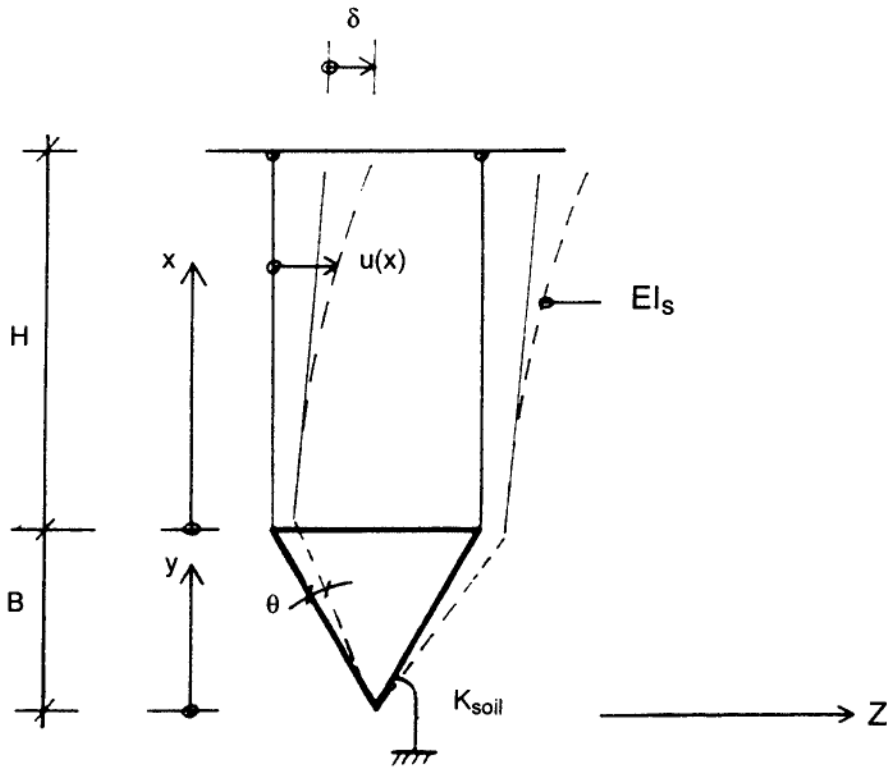


Fig. 3.8 Displacement pattern

The stiffness and mass calculations are then

Stiffness:

$$K = \begin{bmatrix} K_{\theta\theta} & 0 \\ 0 & K_{\delta\delta} \end{bmatrix} \tag{3.9}$$

$$K_{\theta\theta} = K_{Soil} \tag{3.10}$$

$$K_{\delta\delta} = \int_{4\cdot Shaft} EI_s \cdot (\frac{2}{H^2})^2 \, dx \tag{3.11}$$

Mass:

$$M = \begin{bmatrix} M_{\theta\theta} & M_{\theta\delta} \\ M_{\delta\theta} & M_{\delta\delta} \end{bmatrix} \quad (2 \times 2) \tag{3.12}$$

$$M_{\theta\theta} = M_{Soil} + \int_{Caisson} [(m_c + m_a) \cdot y^2 + m_c z^2]) dV$$

$$+ \int_{4 \bullet Shaft} [(m_s + m_a) \cdot (x+B)^2 + m_s z^2] dV \tag{3.13}$$

$$+ \int_{Deck} m_D \cdot [(B+H)^2 + z^2] dV$$

$$M_{\delta\delta} = \int_{4 \bullet Shaft} (m_s + m_a) \cdot (\frac{x}{H})^4 dV + M_{Deck} \tag{3.14}$$

$$M_{\theta\delta} = M_{\delta\theta} = \int_{4 \cdot Shaft} (m_s + m_a) \cdot (x + B) \cdot (\frac{x}{H})^2 \cdot dV + (B + H) \cdot M_{Deck} \tag{3.15}$$

Horizontal displacement:

$$u(x) = \theta \cdot \phi_\theta(x) + \delta \cdot \phi_\delta(x) \tag{3.16}$$

where $\phi_\theta(x)$ and $\phi_\delta(x)$ are the modal functions.

From this comes a 2 x 2 system of stiffness, mass and damping.

Two eigenvalues are obtained from the condition

$$\det (K - \omega^2 M) = 0 \tag{3.17}$$

and the equivalent eigenvector from

$$(K - \omega_i^2 M) X_i = 0 \ (i=1,2) \tag{3.18}$$

Simplified calculations of eigenvalues may be done by hand. An alternative method is to use a beam program that is merged to a corresponding element model of the deck. Fig. 3.9 shows such an analysis of the Condeep platform Sleipner A.

Eigenvalues can be used to study the sensitivity of the structure with regard to assumptions made in the analysis. In Fig. 3.10 a sensitivity study has been performed of eigenvalues with regard to variation in deck stiffness, ground stiffness and structure stiffness.

Eigenvectors from the calculations above are the modal displacement pattern. A global load analysis with a chosen governing wave can be calculated by hand by "placing" the wave load distribution in the modes above. The equations can be solved for each mode. The load effect is then increased by multiplying it by a dynamic factor.

If the inertia forces are substantial, the acceleration is computed for each mode. If several eigenforms are used, the total acceleration is calculated as the contribution from each individual period where the phase angles are considered. In most cases, the eigenperiods are so short with respect to the wave period that all the modes can be assumed to be in phase with the loading. The water mass is considered in the inertia forces.

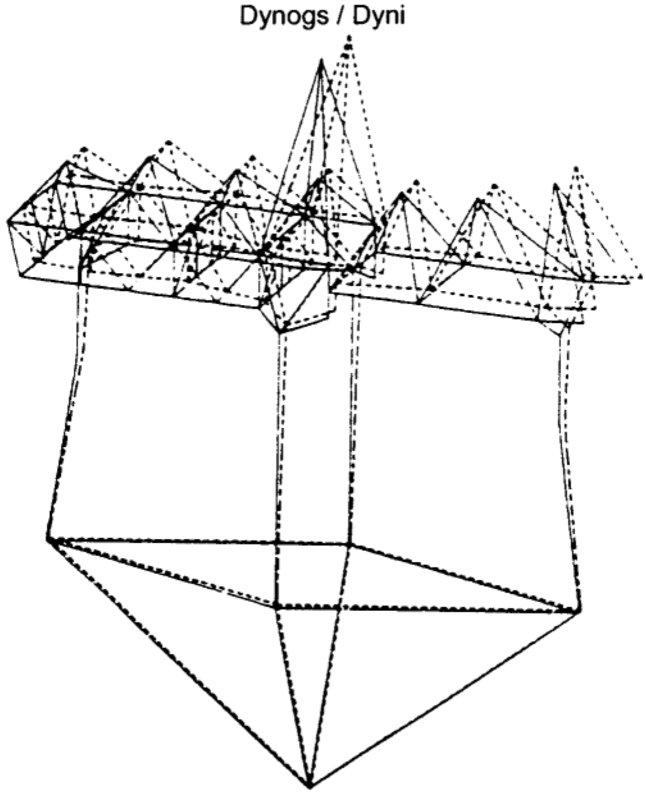


Fig. 3.9 Eigenvalue calculations with a beam program

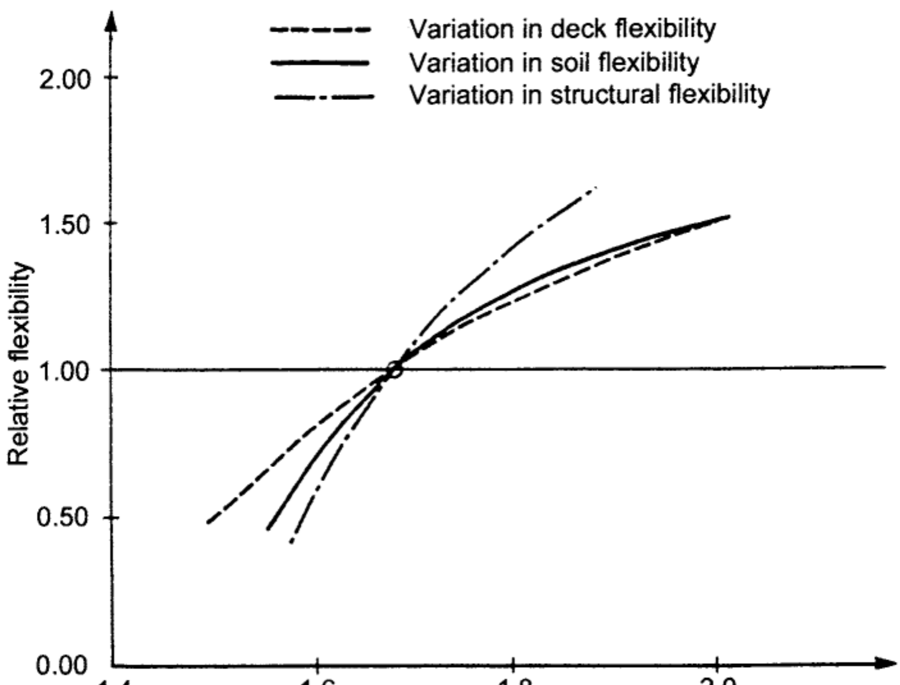


Fig. 3.10 Eigenperiod as a function of flexibility

3.4.2 Models for local load effects

For a Condeep gravity base structure simplified calculations can be performed for most of the structural parts using shell theories. With reference to Fig. 3.11 this applies to the shaft walls, upper domes, cell walls and lower domes. Common to all the parts is that the dominant load consists of compressive normal forces.

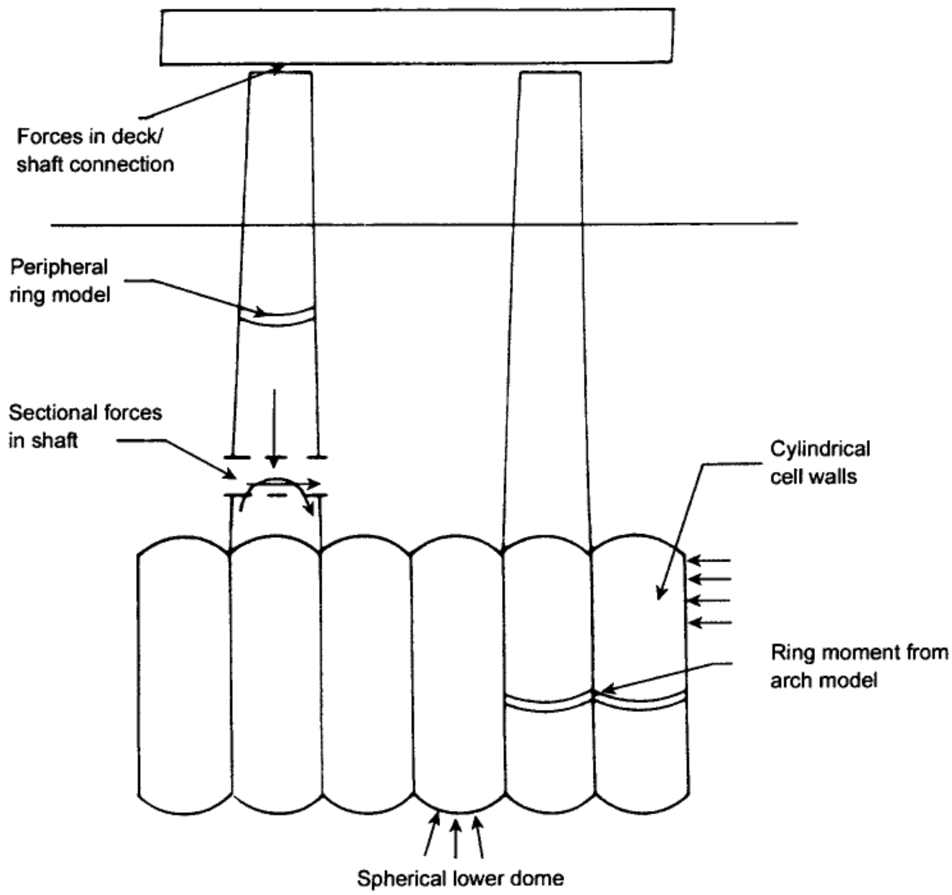


Fig. 3.11 Load effects for a simplified analysis

(a) Outer cells

For the outer parts of the lower and upper domes and outer cylinder wall of an outer cell, an axisymmetric assumption gives a good indication of the sectional forces. Such analyses can be made by hand, by satisfying the compatibility and equilibrium conditions of the domes, ring beam and cylinder wall on rotation/moment and displacement/shear force.

Another method is to use an element program modelling the cell and the domes axisymmetrically. With regard to the choice of element size and type see Chapter 4 that deals specifically with cylindrical shells.

In the unique case of no rotation or displacement at the ring beam, bending of the cylinder wall is characterized by:

$$\textit{Elastic length}: l_e = \frac{(rt)^{0.5}}{(3(1-v^2))^{0.25}} \tag{3.19}$$

$$\textit{Edge moment}: M_0 = \frac{p \cdot l_e^2}{2} \tag{3.20}$$

$$\textit{Shear force}: V_0 = p \cdot l_e \tag{3.21}$$

where

r	=	middle radius of cylinder wall
t	=	wall thickness
v	=	Poisson's ratio
p	=	outer hydrostatic pressure (uniform)
l_e	=	elastic length.

Within a distance of l_e from each edge, the compression force will be determined from a membrane solution

$$N_\varphi = p \cdot r \tag{3.22}$$

An indication of the local bending effect close to the ring beam can be calculated using Equation (3.19) with a wall thickness of t=0.60 m and a middle radius of 10 m with a corresponding elastic length of 1.90 m. For thick cylinder walls the above equations must be adjusted to account for the difference between the middle radius and the radius of surface where the load is applied.

(b) Inner cells

For the inner cells, the geometry and the boundary conditions are more complex. Apart from the circular walls, some of the caissons also have straight walls. A horizontal beam analogy is more appropriate for those walls when subjected to one-sided hydrostatic pressure.

For the upper and lower domes on the other hand, an axisymmetric solution gives appropriate sectional forces when the dome is subjected to hydrostatic pressure. Because of the stiffening effect of the surrounding cells it is a good approximation to assume clamping at the ring beam, in other words no rotation or translation.

(c) Shafts

For the shaft the axisymmetric theory can be used to estimate the bending moment and shear at the junction caisson/shaft. Furthermore, the compressive normal force along the shaft can be calculated from (3.22) with correction for thick shell.

At the boundary condition caisson/shaft, a special analysis is needed to take into account the effect of the upper domes and the surrounding cells. A local element model by shell or solid elements would be necessary. Forces from earlier global analysis are applied in the model.

During coupling of deck to the shaft, large concentrated forces develop. A finite element mesh would be required at the interface to evaluate the forces. In these areas a strut and tie method of calculation can be useful for design.

(d) Castings

It is not unusual that during detailed design, changes and adjustments occur to the geometry which then does not correspond to the global finite element model. An example is concrete castings around mechanical outfittings. These castings affect the structure, but they are not taken into account in the global analysis.

For such situations a local element model is advisable. Displacements from the global analysis are applied to the local model.

In addition to castings inside the structure, similar castings are used outside the platform such as around J-tubes and crane footings.

3.5 Floating structures

3.5.1 General

This section reviews the major load effects acting on a floating concrete structure. The description is based on simple models for hand calculations or desk computers.

The action between the raft and the deck is especially important for a floating structure. This section will nevertheless deal mainly with the concrete raft and takes into account the effect of deck stiffness at the intersection points. With regard to the mass of the deck it is also important that, for the hand calculations, the actual weight distribution and weight placement are substituted.

A system analysis with emphasis on global response in the construction and operation phases is considered. The hydrostatic stability calculations are reviewed in addition to the static loading. The static and dynamic characteristics are analysed together with critical deformation patterns of the raft. This gives an overall indication of inertia forces in the dynamic models including the water mass.

Catenary anchored platforms as well as tension leg platforms are considered.

3.5.2 System description

Design of floating platforms requires the analysis of all construction phases, mating (coupling) of the deck, installation and operational phases. Residual forces from the construction phases can be important for design.

As shown in Fig. 3.12, before placement of the deck, the raft has little global bending stiffness. In the submerged phase, such as during mating, the additional mass from the sea is substantial. This gives a high natural period (approximately 3 seconds) that could give a resonance effect for short fjord waves. For maximum depth of submergence, the line of action of the wave is highest. The effects mentioned above can govern the design of the entire raft, not only due to the high hydrostatic pressure but also due to the global effect.

Fig. 3.13 shows the total picture of the structure system with the deck included. The deck stiffens the raft at the top and cancels the deformations shown in Fig. 3.12. Load distribution between the raft and the deck is important in the design of the deck as well as the raft.

For floating structures where large motions are common, it is more appropriate to split the load effect from displacement and deformations in two, namely one set of rigid body modes superimposed with a set of deformation modes. This is a well established method for the analysis of structures with large motions. Rigid body modes are dominant in the determination of the inertia forces, while the deformation modes determine the sectional forces in the structure.

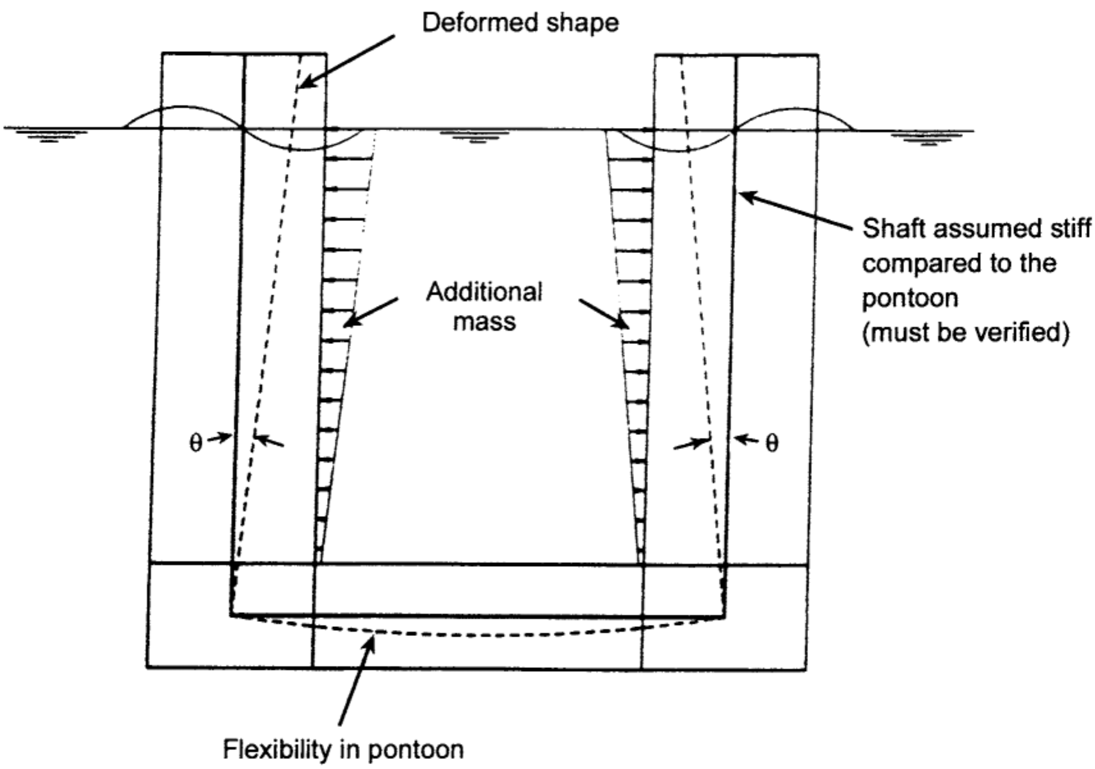


Fig. 3.12 Raft before deck mating

For static loads, the split mentioned above is easy since the calculation of load effects follows the same principle for all modes with regard to rigid body and deformation types. For both mode types a modal stiffness is calculated which is then compared with the modal load.

For dynamic loads, the dynamic increase of each individual mode must be included. It is necessary here to have a model where the contribution of the mass from the structure and the surrounding water is also included. The inertia forces will have a substantial effect on both the rigid body motion and the deformation of the raft.

The splitting into rigid body modes and deformation modes is illustrated in Fig. 3.14 for a tension leg platform (TLP).

The global stiffnesses which are associated with the rigid body modes are determined from prestressing and material stiffness in the anchorage system as well as from the raft surface water area. For the vertical modes (heave, roll and pitch) the axial stiffness of the tethers of a TLP will, for example, be more dominant than the effect of the surface water area. For a catenary anchored structure the opposite is true.

This has an influence on the natural period for the heave and roll where the two types of platform differ.

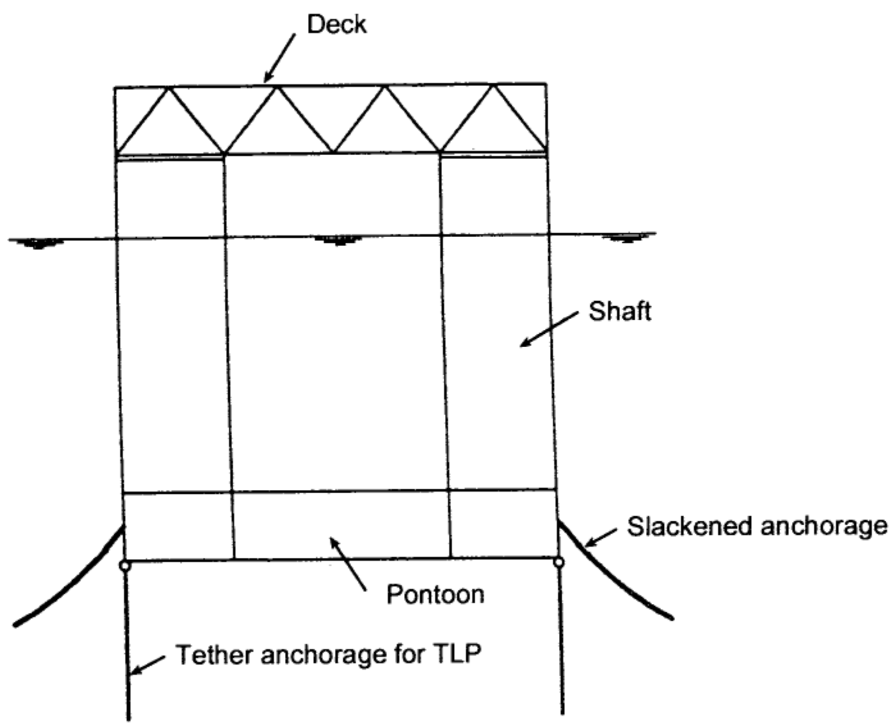


Fig. 3.13 Mated structural system

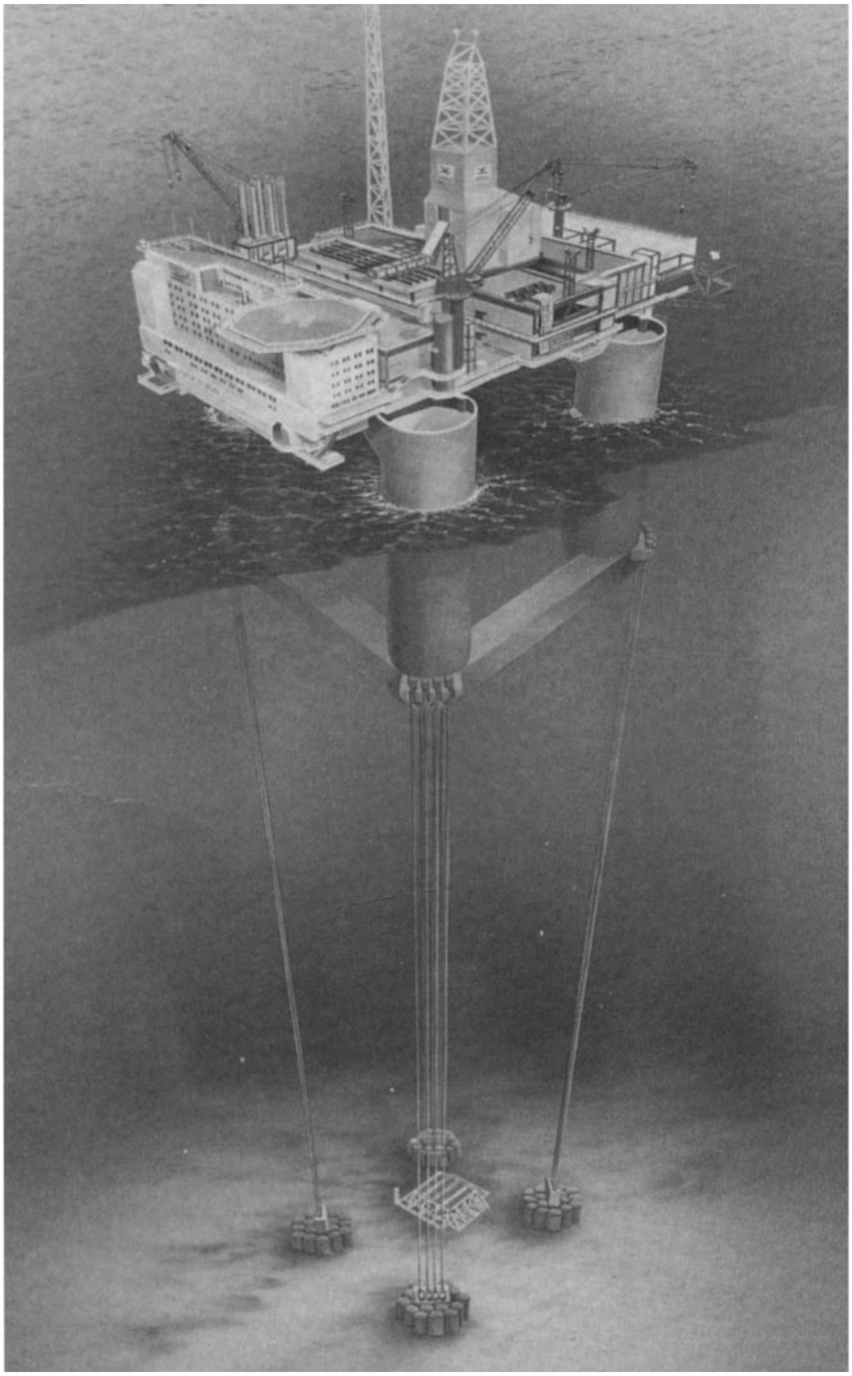


Fig. 3.14 Rigid body mode and deformation mode

3.5.3 Global static analysis

(a) Hydrostatic stability

In Fig. 3.15, the terms needed for hydrostatic stability control are shown, where:

T_R	=	riser tension
T	=	anchorage tension
G	=	weight of platform including water ballast
I_w	=	waterline moment of inertia.

For a unit rotation (1 rad), the centre of buoyancy moves a distance eB from the centre line.

$$e_B = \frac{\rho g \cdot I_W}{\nabla} \tag{3.23}$$

where

ρ	=	specific weight of water (1.027 t/m^3)
g	=	gravity acceleration (9.81 m/s^2)
∇	=	displacement (kN).

Moment equilibrium for a unit rotation relative to the anchorage level of the raft (notation see also Fig. 3.15)

$$M = \nabla \cdot (z_B + e_B - z_o) - G \cdot (z_G - z_o) + \Sigma\Delta T \cdot a - T_R \cdot (z_R - Z_o) \tag{3.24}$$

In (3.24) ΔT is the change in tension of the anchorage lines for a unit rotation, expressed for a TLP as

$$\Delta T = \frac{EA}{L} \cdot a \tag{3.25}$$

where the axial stiffness term EA/L per tension line is introduced.

For the case of a free floating structure, as during construction, towing or installation, the riser and the anchorage contribution can be eliminated from (3.24). Then ∇ = G and the stabilizing moment is

$$M = G \cdot (z_B + e_B - z_G) \tag{3.26}$$

and

$$z_0 = \frac{M}{G} = \frac{\rho g \cdot I_W}{G} - (z_G - z_B) \tag{3.27}$$

The Z_0 requirement is the stability criterion for a free-floating structure.

For a TLP in operation the tethers are the dominant items for hydrostatic stability.

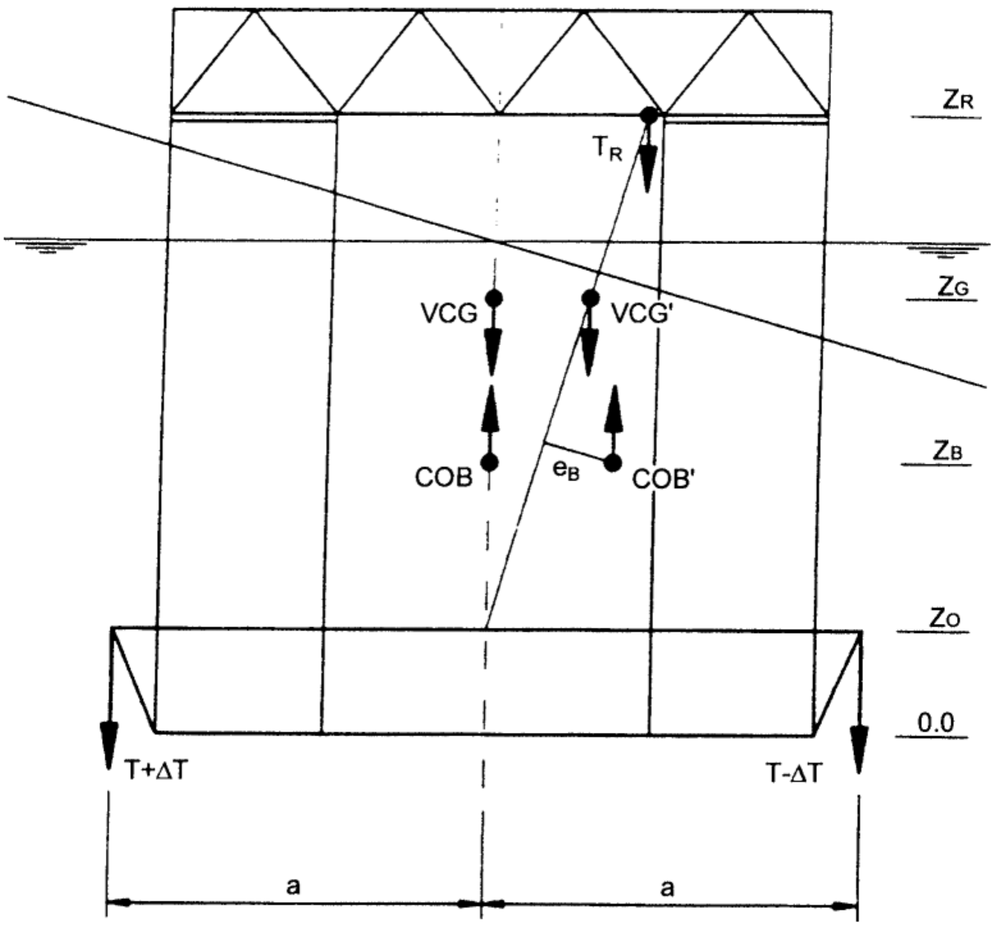


Fig. 3.15 Hydrostatic stability

(b) Static rigid body motion

The raft is assumed now as non-deformable and that the stiffness of the anchorage system determines the motion for given loads. As shown in Fig. 3.16 it is convenient to establish the equilibrium equations in the Cartesian co-ordinate system with the origin located in the centre of the anchorage system of the platform. A 6 x 6 stiffness relation is then established:

$$\begin{bmatrix} FX \\ FY \\ FZ \\ MX \\ MY \\ MZ \end{bmatrix} = \begin{bmatrix} KXX & & & & & \\ & KYY & & & & \\ & & KZZ & & & \\ & & & KRX & & \\ & & & & KRY & \\ & & & & & KRZ \end{bmatrix} \begin{bmatrix} U \\ V \\ W \\ RX \\ RY \\ RZ \end{bmatrix} \tag{3.28}$$

For a TLP the stiffness in the horizontal modes can be defined from the geometric stiffness

$$KXX = KYY = \Sigma \frac{T_i}{L_i} \tag{3.29}$$

of the tethers:

where T_i is the pretension and L_i length of tether number "i".

The vertical stiffness in heave and roll will depend on the axial stiffness of the tethers, whereas the contribution of the water surface area is secondary. We get:

$$KZZ = \Sigma \frac{EA_i}{L_i} + \rho g \cdot A_W \tag{3.30}$$

where EA/L is the axial stiffness of a tether and A_w the water surface area of the platform.

The expression in (3.28) is linear and does not take into account that the tensile forces in the anchorages change as a consequence of the platform motion. For a TLP where the stiffness of the tethers is dominant, the raft will follow an approximately circular path in the vertical plane for sideways motion. For a given horizontal displacement U, the increase in depth will be:

$$W = L[1 - \cos(asin\frac{U}{L})] \tag{3.31}$$

where L is the length of the tether.

This increase in depth can be in the order of a few metres. This gives a buoyant force that, once more, alters the force in the tether.

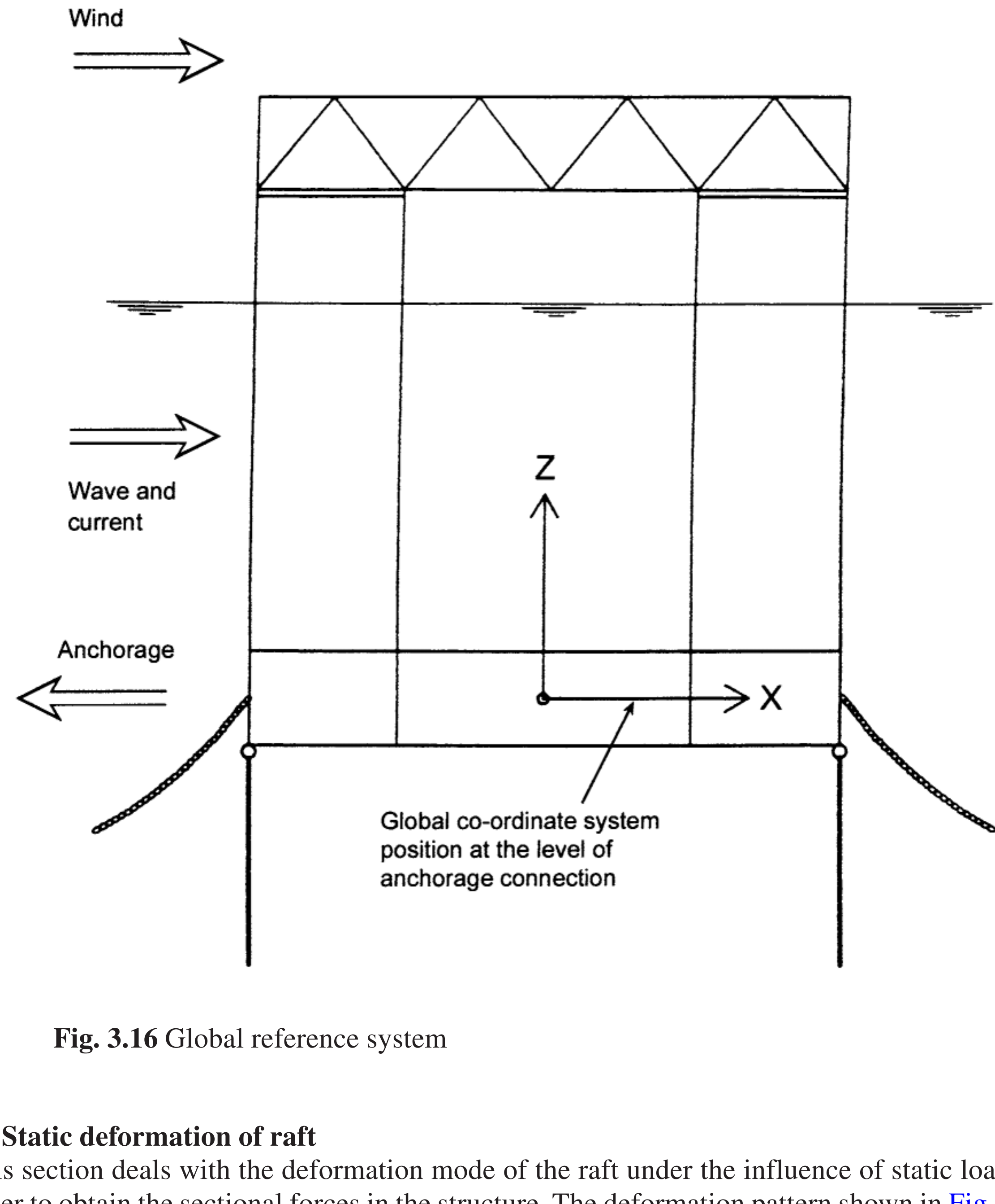


Fig. 3.16 Global reference system

(c) Static deformation of raft

This section deals with the deformation mode of the raft under the influence of static loads in order to obtain the sectional forces in the structure. The deformation pattern shown in Fig. 3.17 is a typical example for loads from waves and wind.

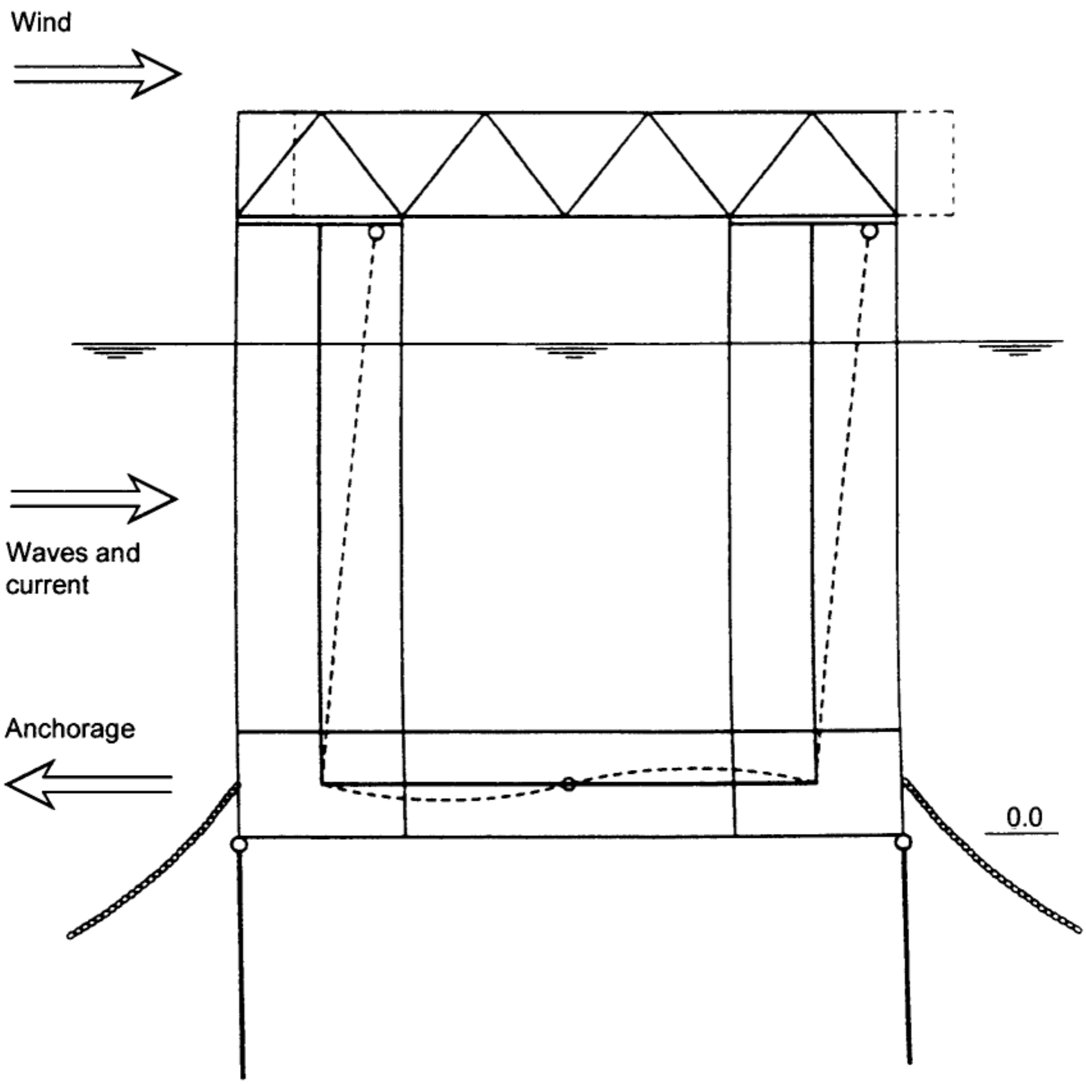


Fig. 3.17 Bending deformation of raft

To obtain the deformations and the sectional forces one needs the stiffness of the pontoons, the shafts and the deck. It is also important to get the correct stiffness model of the boundaries between the pontoon and the shafts and between the shafts and the deck.

A method using hand calculations to obtain the sectional forces is shown below. An alternative method is to use a beam model in an element program, particularly when different load situations are to be analysed.

With reference to Fig. 3.18, the virtual deformation figure is chosen with the intention of calculating the moments at the boundaries of the pontoon. It is assumed here that we still have a simply supported connection between the top of the shaft and the deck, this assumption needs to be re-evaluated for each load situation. Given the pontoon moment as M_{PON}, and $\delta\beta$ as the virtual angle at a section with M_{PON}, the internal virtual work, including two pontoons, is:

$$\delta W_i = 4 \cdot M_{PON} \cdot \delta\beta \tag{3.32}$$

For loads parallel to the pontoon (0° or 90°):

The static load resultant from wind and/or current is assumed to act with a magnitude P at a height Z_p relative to the platform co-ordinate system. It is also assumed that due to P, the change of the anchorage force will be ΔT for each shaft. For a TLP where the axial stiffness is more dominant, then:

$$\Delta T = \frac{P \cdot z_P}{4 \cdot a} \tag{3.33}$$

The outer virtual work is given as:

$$\delta W_e = P \cdot z_p \cdot (1 - \frac{d}{a}) \cdot \delta\alpha \tag{3.34}$$

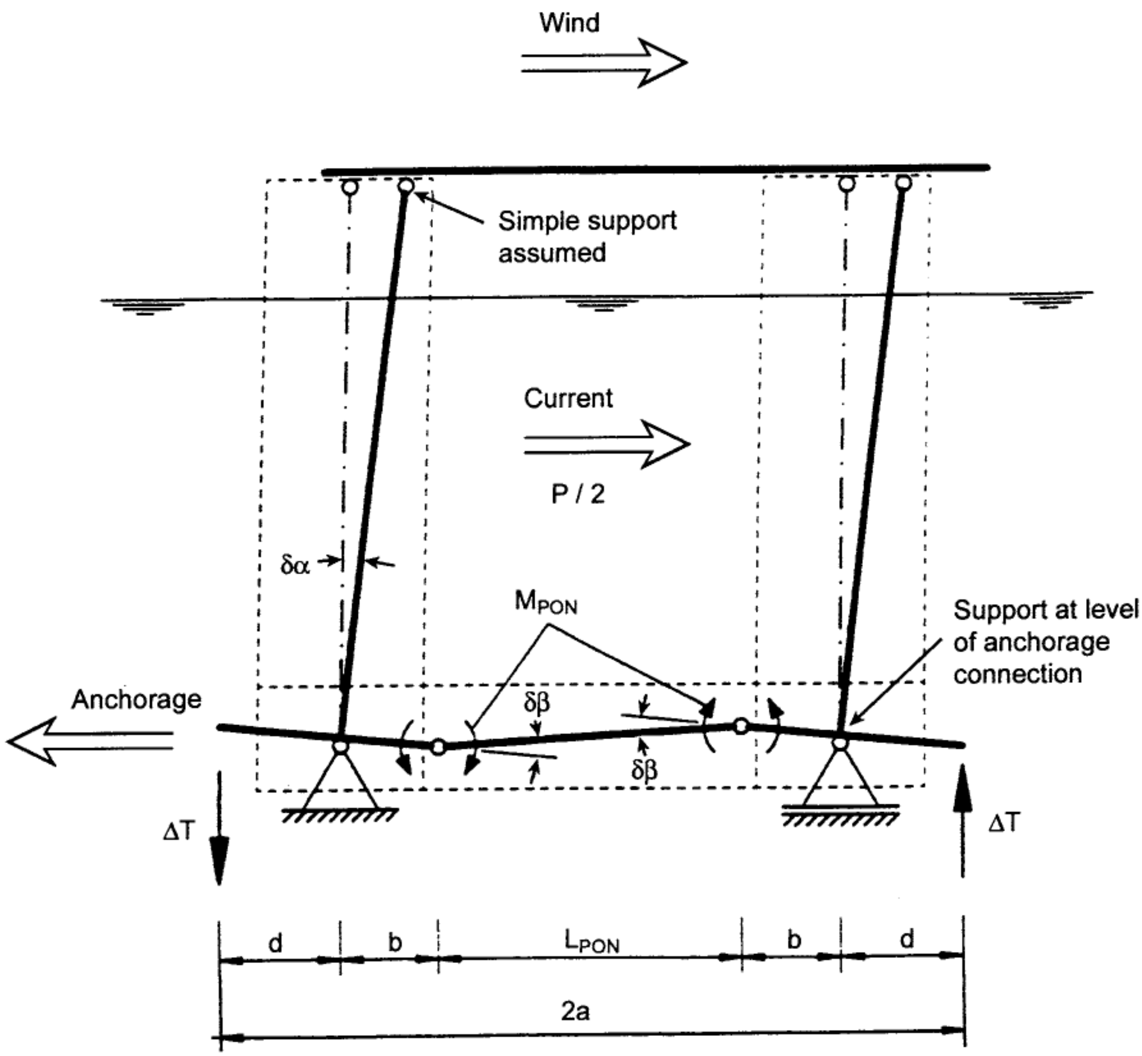


Fig. 3.18 Virtual displacement figure

See also Fig. 3.18 for notations.

The relation between δα and δβ is given from the geometry of Fig. 3.18:

$$\delta\beta = \delta\alpha \cdot (1 + \frac{2b}{L_{PON}}) \tag{3.35}$$

By comparing (5.10, 5.12 and 5.13), the bending moment at the boundary of the pontoon is:

$$M_{PON} = \frac{P \cdot z_P}{4} \cdot \frac{1 - \frac{d}{a}}{1 + \frac{2b}{L_{PON}}} \tag{3.36}$$

The relation above is evolved for a static load parallel to the actual pontoon. For a different load direction it is advantageous to use a simple frame program.

The moment diagram over the pontoon for the case shown in Fig. 3.18 would be linear with tension on the opposite side of the two pontoons and nil bending moment at the centre of the pontoon. The shear force from wind/current is constant along the pontoon.

By using the virtual work as mentioned above, it is vital to include all the loads contributing to the global equilibrium model, refer to contribution of ΔT.

(d) Residual sectional forces

Eccentric positioning of the deck load during the mating operation generally leads to the need for jacking operations to reduce the deformations and the sectional forces in the flexible raft; see Fig. 3.12. This and the simultaneous deballasting of the raft after mating leads to permanent stresses in the raft which must be accounted for in design.

The residual sectional forces can be calculated by hand or alternatively by using a beam model in a computer. In this case, it is necessary to model the shaft/deck connection with a finite element model more accurately than is possible with hand calculations. A local shell element model combined with a beam model of the lower part of the raft can be used.

(e) Uneven ballast

Fig. 3.19 illustrates the situation with uneven ballasting for a doubly symmetrical raft. Two diagonally opposite shafts have more ballast water whereas the two others have correspondingly less. The total ballast and the draft are therefore correct, but the internal redistribution introduces stresses in the raft.

By having the modal amplitude as the difference in the deflection between the two sets of shafts, the stiffness of the model would be:

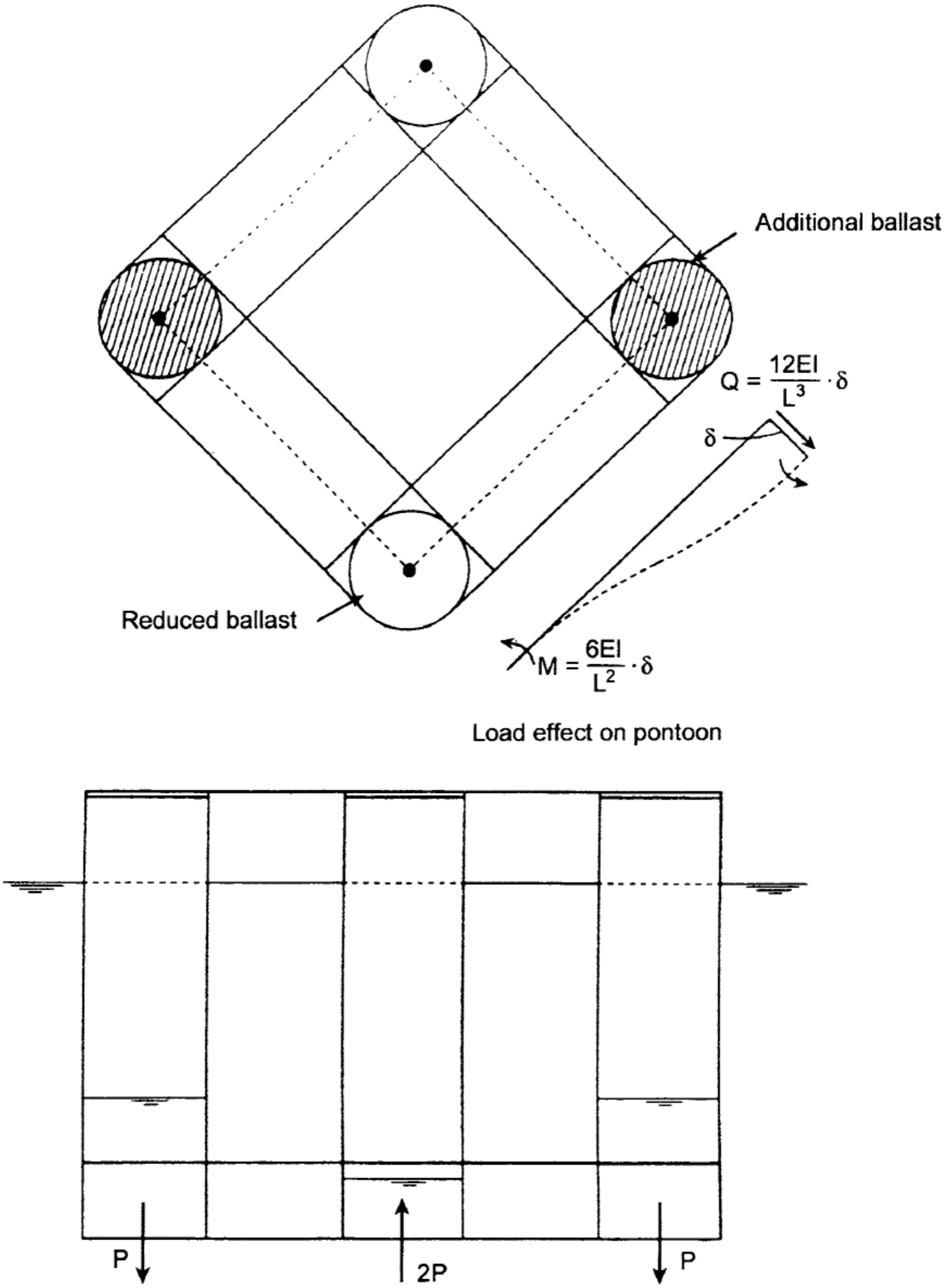


Fig. 3.19 Uneven ballast in a floating structure

$$K_{RAFT} = \frac{48 \cdot EI_{PON}}{L_{PON}^3} \tag{3.37}$$

where

EI_{PON} = sectional stiffness about the horizontal axis of the pontoon
L_{PON} = effective length of the pontoon, distance between the boundaries.

For a catenary anchored structure, the raft will carry the entire uneven load. For a TLP on the other hand, a portion of the load will be carried by the tether axial stiffness. To find the relative distribution between the tethers and the raft, a comparison is made between the raft modal stiffness in (3.37) and the modal stiffness of the tethers.

$$K_{TETH} = \frac{2 \cdot EA_{TETH}}{L_{TETH}} \tag{3.38}$$

where

EA_{TETH} = sectional stiffness of tethers for each shaft

For a TLP, the raft will carry most of the uneven load since the modal stiffness in (3.37) is 10 to 20 times larger than the contribution of the tethers (3.38).

The bending moment at the end of the pontoon is

$$M_{PON} = \frac{P \cdot L_{PON}}{4} \tag{3.39}$$

where

P = the additional ballast per shaft.

From (3.39) the bending stresses in the pontoons can be calculated.

3.5.4 Global natural period

This section deals with the stiffnesses and masses for the deformation modes during construction phases as well as the rigid body modes in the operation phases. The intention is to develop a simple design method suitable for hand calculations to estimate the natural period.

(a) Raft prior to deck mating

As mentioned in Section 3.5.2, the submerged phase just prior to deck mating is the most crucial with regard to the global response of the raft and also with regard to the large hydrostatic pressures exerted on the walls. By choosing the rotation of the shafts as modal amplitude, the modal stiffness contribution from two pontoons is obtained.

$$K_{\theta\theta} = \frac{4 \cdot EI_{PON}}{L_{PON}} \tag{3.40}$$

For distribution of masses the following notations are used:

m_{COL}	=	construction mass per unit height of column (shaft), ballast included
m_{aCOL}	=	additional mass from surrounding water per unit height of column (shaft) = C_m x ρ g A
m_{PON}	=	construction mass per unit length of a pontoon, ballast included
m_{aPON}	=	additional mass from surrounding water per unit length of pontoon = C_m x ρ g A.

With regard to the deformation mode in Fig. 3.12, the modal mass can be written

$$M_{\theta\theta} = \frac{4}{3}\left(m_{COL} \cdot L_{COL}^3 + m_{a,COL} \cdot L_{WET}^3\right) + \frac{2}{30}\left(m_{PON} + m_{a,PON}\right) \cdot L_{PON}^3 \tag{3.41}$$

In (3.41) the lengths L_{COL} , L_{WET} and L_{PON} are related to the centre axes.

An estimate of the natural period for the chosen mode is:

$$T = 2\pi \left(\frac{M_{\theta\theta}}{K_{\theta\theta}}\right)^{0,5} \tag{3.42}$$

Among the effects mentioned above, the additional mass in the shafts will contribute most to the modal mass.

In the submerged phase, Equation (3.42) can give a natural period in excess of 3 seconds. To evaluate the load effect in this phase, the natural period must be compared with the sea state at the actual location in protected water. In the Norwegian fjords waves giving natural periods from 2 to 6 seconds can be expected. Damping is also low in the 3 second area and one can therefore expect a substantial dynamic increase in the submerged phase prior to mating.

(b) Platform in operation

A catenary anchored structure is different from a tension leg platform particularly concerning heave and roll. For these motions, the TLP is much stiffer and therefore has lower natural periods. Typical natural periods for a TLP are:

Horizontally	:	60 to 120 seconds
Vertically	:	2.0 to 4.0 seconds

The horizontal natural periods are determined from the lateral stiffness of the anchorage lines, while the water surface area and the axial stiffness of the lines contribute to the vertical stiffness. For a TLP the axial contribution is more dominant; see Section 3.5.3.

For rotational modes the illustration in Fig. 3.20 is used. Since the lateral anchorage forces are small, the centre of rotation in the vertical direction will be dependent on the inertia forces. The actual masses are added in the figure. A centre of rotation is chosen with a vertical ordinate Z_o. The position of the centre of rotation Z_o is chosen such that the horizontal resultant of the mass forces is zero. After that, the mass inertia moment, or the modal mass, is determined.

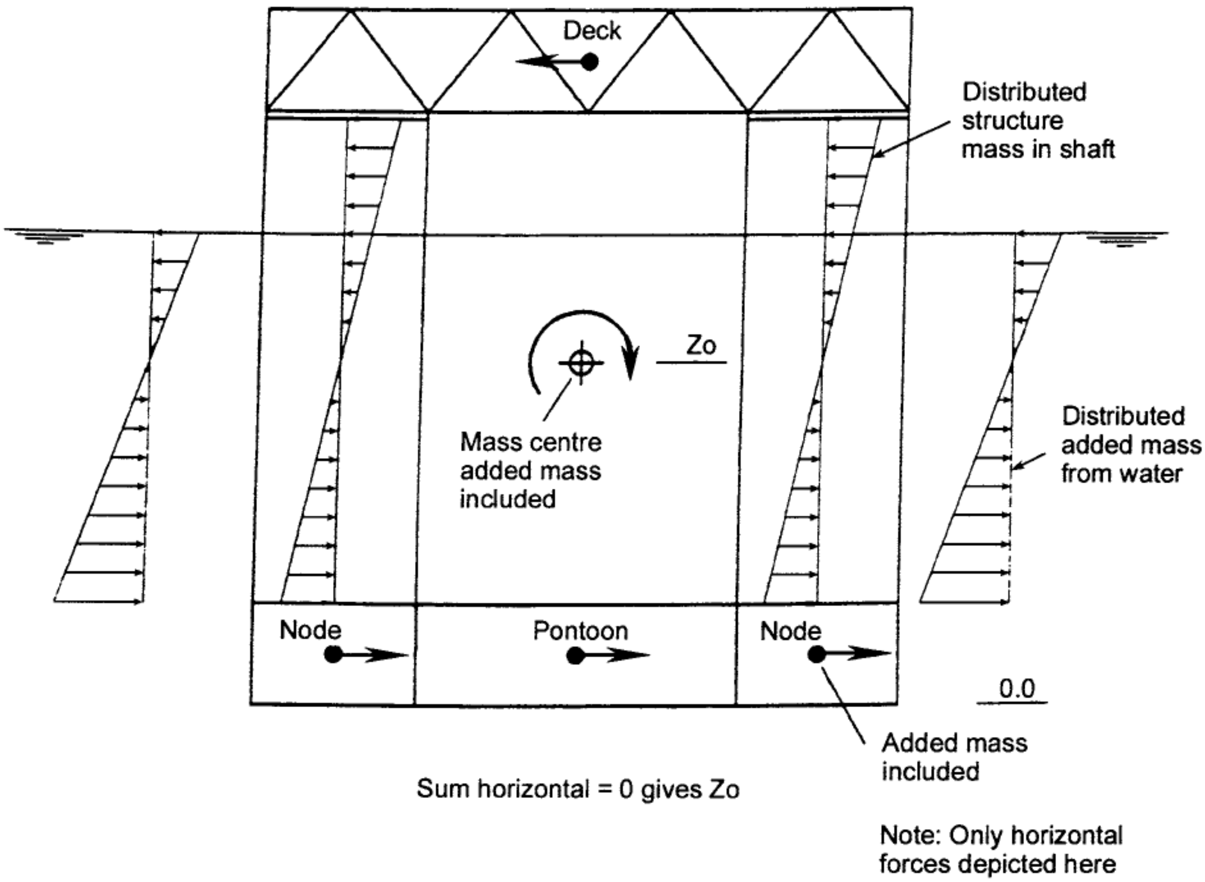


Fig. 3.20 Inertia forces due to roll

3.5.5 First order wave

This section shows the use of deterministic wave theory to give an indication of the sectional forces in the raft from the first order wave.

(a) Loads

The relation between maximum wave height and period for the actual field location is assumed to be supplied for ULS (100 year return period) and for SLS (1 year). Fig. 3.21 illustrates such a relation.

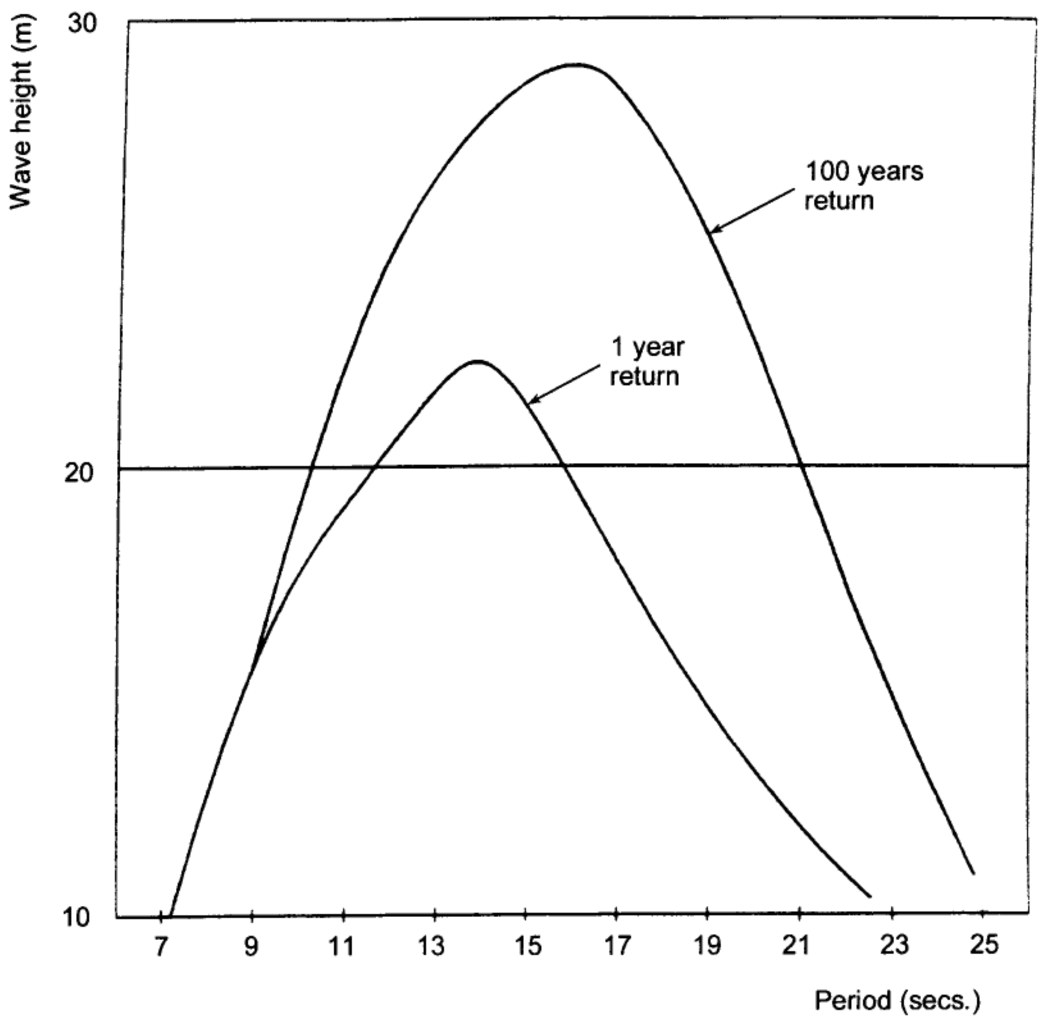


Fig. 3.21 Wave height versus period

Morison's formula gives two contributions to the load on the raft. The viscous part given as:

$$F_D = \frac{1}{2}\ \rho \cdot C_D \cdot v_r^2 \cdot A \tag{3.43}$$

where

ρ	=	specific gravity of seawater (1.027 t/m^3)
C_D	=	viscosity coefficient
v_r	=	relative velocity between wave particles and raft
A	=	area across the velocity, per linear metre.

The other contribution in Morison's formula is the mass component

$$F_M = (1 + C_m) \cdot \rho \cdot V \cdot w \tag{3.44}$$

where

C_m	=	coefficient for added mass
V	=	submerged volume per linear metre
w	=	wave particle acceleration.

As is shown below, the mass component (3.44) is generally the most dominant. From (3.43) and (3.44) we notice that the contribution is 90° out of phase.

Distributed load at the waterline per metre length of a single shaft from the mass component in Morison's formula is then:

$$F_{MO} = (1 + C_m)\ \rho \cdot \frac{\Pi}{4} D^2 \cdot (\frac{2\Pi}{T})^2 \cdot \frac{H}{2} \tag{3.45}$$

where

D	=	outer shaft diameter
T	=	wave period
H	=	maximum wave height from Fig. 3.21.

The variation of load intensity as a function of water depth z from the waterline is

$$F_M\,(Z) = F_{MO} \cdot e^{-kz} \tag{3.46}$$

where

k	=	wave number
g	=	gravity acceleration

The total wave force on a shaft over a depth d would then be:

$$P = F_{MO} \cdot \int_{o}^{d} e^{-kz}\, dz \tag{3.47}$$

$$P = F_{MO} \cdot \frac{1}{k} \cdot (1 - e^{-kd}) \tag{3.48}$$

Resultant at depth d_o below sea water level is

$$d_o = \frac{1 - e^{-kd} - kd \cdot e^{-kd}}{k(1 - e^{-kd})} \tag{3.49}$$

For short waves, approximate values are used

$$P = \frac{F_{MO}}{k} \tag{3.50}$$

$$d_o = \frac{1}{k} \tag{3.51}$$

Equations (3.48) to (3.51) are used later in the global equilibrium consideration.

For the evaluation of critical wave periods, it is interesting to look at the load resultant P as a function of the wave period. The relation H/T in Fig. 3.21 is applied in (3.45) together with the actual outer diameter, specific density of water, as well as a factor for added mass. Since the mass contribution is the most dominant and is 90° out of phase with the viscous term, only (3.45) and (3.50) are applied.

(b) Waves in 0 and 90 degree directions

Waves in 0 and 90 degree directions act along the main axes of the platform; that is; parallel to or normal to the pontoons. For the bending moment in the pontoon close to the column about the horizontal axis, the case illustrated in Fig. 3.22 often governs the design, with a wave length between the shafts. The situation in Fig. 3.22 is also often critical for the shear at the pontoon ends. All four shafts now have their maximum horizontal wave load in one direction at the same instant, implying horizontal acceleration of the platform

Fig. 3.18 is used together with the formulas in (3.3.2–3.3.6) in order to calculate the bending moment in the pontoon. The difference from the static expressions is now that the horizontal inertia force resultant is included, as indicated in Fig. 3.22. As horizontal motion is inertia dominated, for a catenary anchored platform as well as for a TLP, Equation (3.36) for static load effect can be used directly, modifying the vertical arm z to be the distance between wave resultant load and the inertia force, added mass included. P is the resulting wave load on to the raft.

Letting L (m) be the centre spacing of the shafts, the wave period T(s) representing the above situation comes out as:

$$T=0.80 \cdot L^{0.5} \tag{3.52}$$

In most cases the wave (3.52) is the shortest design wave for the structure.

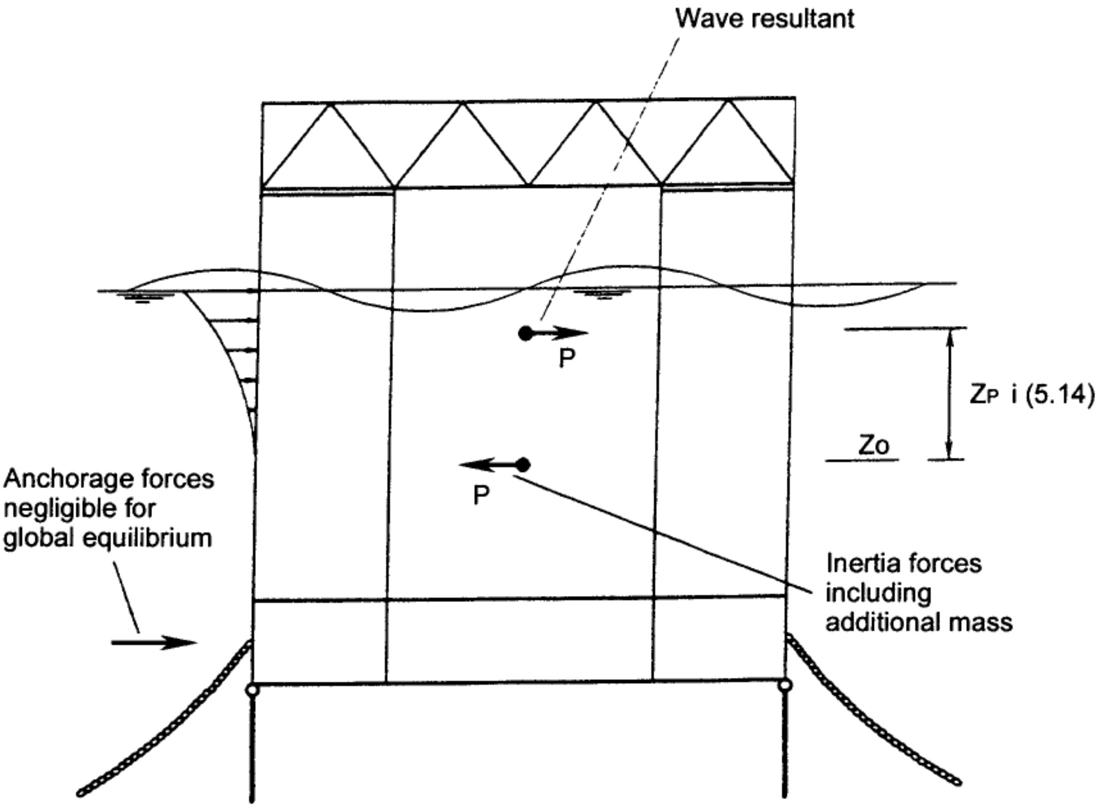
(c) Diagonal wave

For a bending moment in a pontoon about a vertical axis, a diagonal wave is often dominent. In the case of a quadratic floater with L (m) being the centre shaft spacing, the critical wavelength comes out as twice the diagonal spacing, with wave period:

$$T=1.35 \cdot L^{0.5} \tag{3.53}$$

Fig. 3.23 illustrates the mode of deformation resulting in a bending of the pontoons about the horizontal and vertical axes, as well as torsion in the pontoons. Given the modal deformation pattern in Fig. 3.23 the modal contributions to stiffness from bending and torsion can be calculated. In most cases bending of the pontoons about the vertical axis gives by far the dominant terms.

Based on the deformation pattern in Fig. 3.23 the bending and torsion moments in the pontoons are calculated after the deformation amplitude has been found from the stiffness expressions. The load situation in Fig. 3.23 for the wave phases of 0 and 180 degrees creates no resulting horizontal load on the structure, and inertia forces do not enter into the expressions. Other wave phases are also to be considered, where platform rigid body motion is to be included.



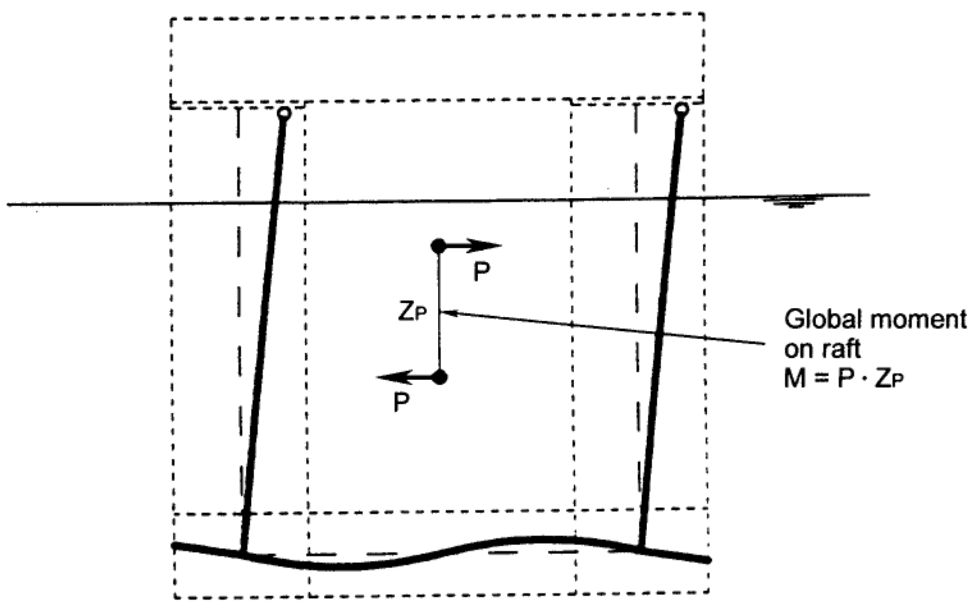


Fig. 3.22 Wave length between the shafts

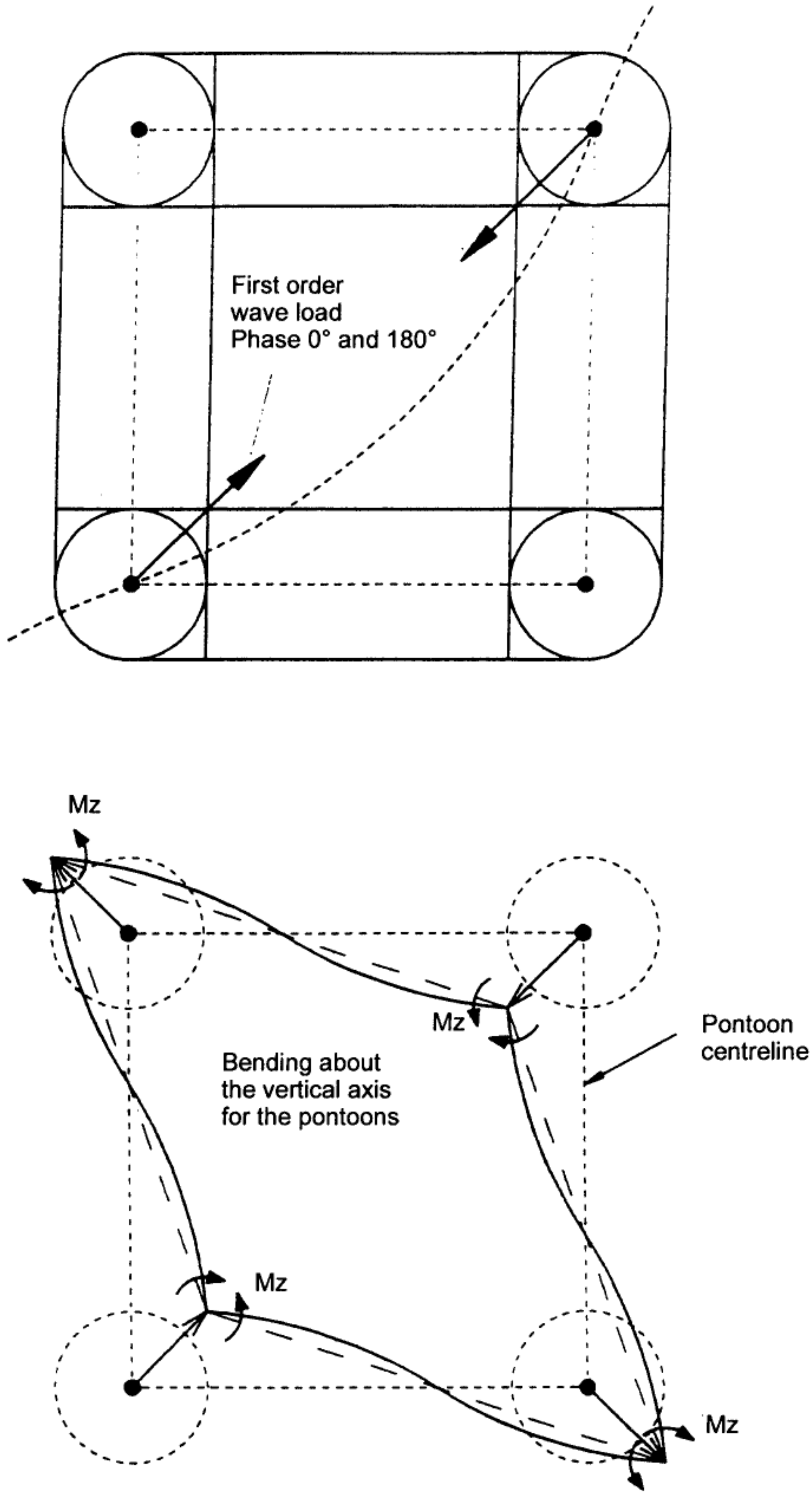


Fig. 3.23 Diagonal wave situation

3.6 Ship impact

3.6.1 General

The present section outlines simplified calculation schemes for the response due to ship impact. Major effort has been placed on techniques for calculating global responses, dynamic effects included.

The objective of presenting the simplified impact considerations below, is among other things enabling estimates of section forces from impact to be made, and by comparing these to other load effects, evaluate whether ship impact is the governing force. Impact analysis is not part of the automised design scheme and thus requires special analyses.

As a supplement to the global response, a complete analysis of impact also includes punching control. Here, the distributed load intensity from impact is the governing parameter, rather than the resultant load. Thus, hard impacts from smaller ships are often critical for punching.

Methods for impact analysis of floating and gravity base structures are given below. The illustrations are made for floating structures, so as to see the variation in dynamic effects for different modes of motion, Gravity base structures follow the techniques in Section 3.6.5 for rotation, where the eigenperiod is in the range of the duration of impact.

3.6.2 Impact load

As shown in Fig. 3.24 a central impact is assumed, so that the impact force is directed through the vertical line in the platform mass centre, added mass from water included. The velocity of the ship prior to impact is denoted v_o, mass of ship with added mass included is M_s, and platform mass with surrounding water is M_p.

Further, it is assumed that the impact force is constant during the duration of the impact, and given by the plastic capacity of the ship. This is an approximation, since for plastic deformation of the ship hull the contact force varies with the indentation. In subsequent expressions the impact force is denoted P (kN) and the duration of impact t (s).

3.6.3 Impact mechanics

In Fig. 3.25 a diagonal impact is suggested towards one of the platform shafts. The load resultant P acts in the waterline area. The global reference system lies at the elevation of the anchorages, with the X-axis directed along the impact force. The impact force has elevation h above the origin of co-ordinates.

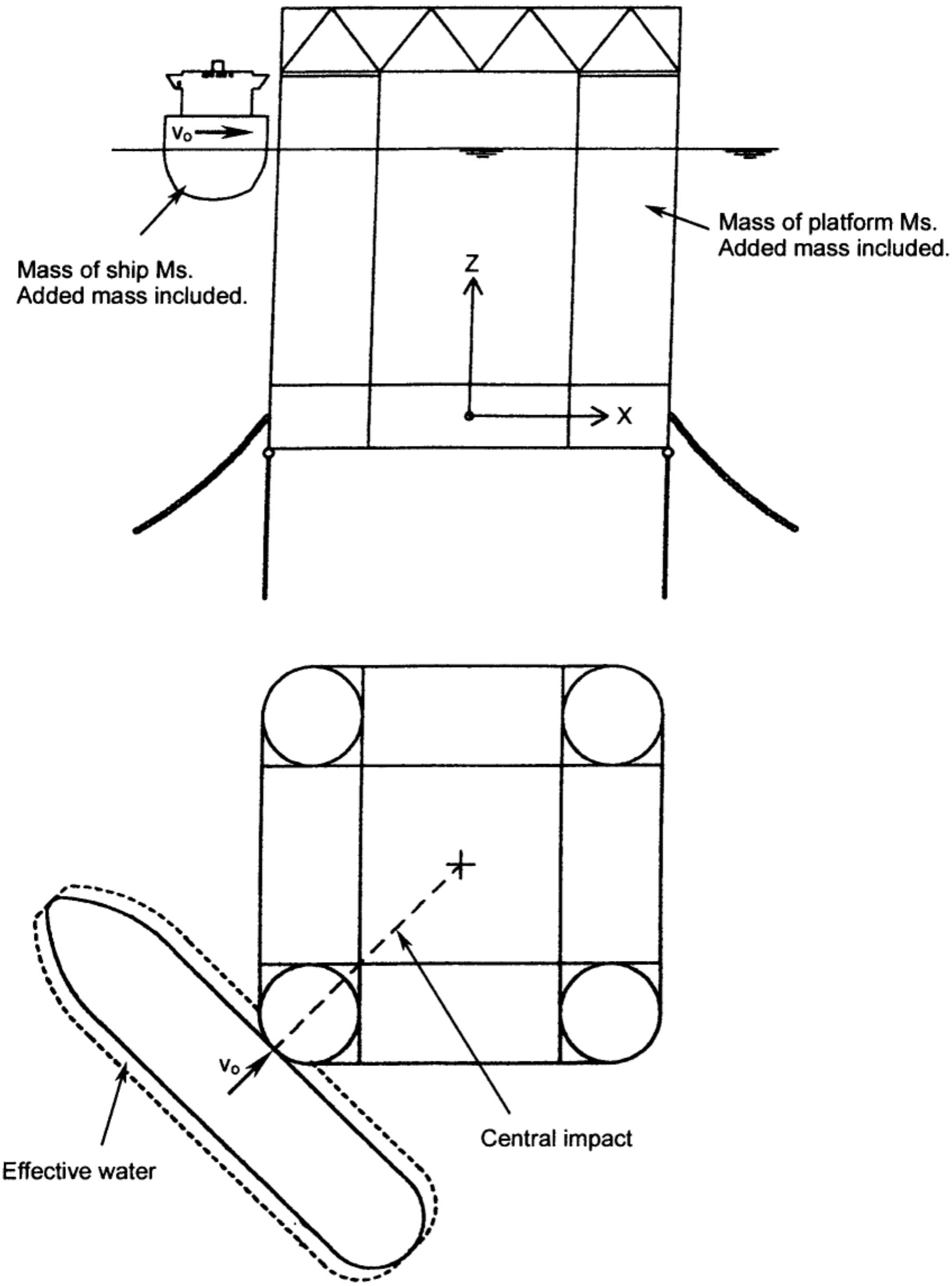


Fig. 3.24 Central impact from tanker

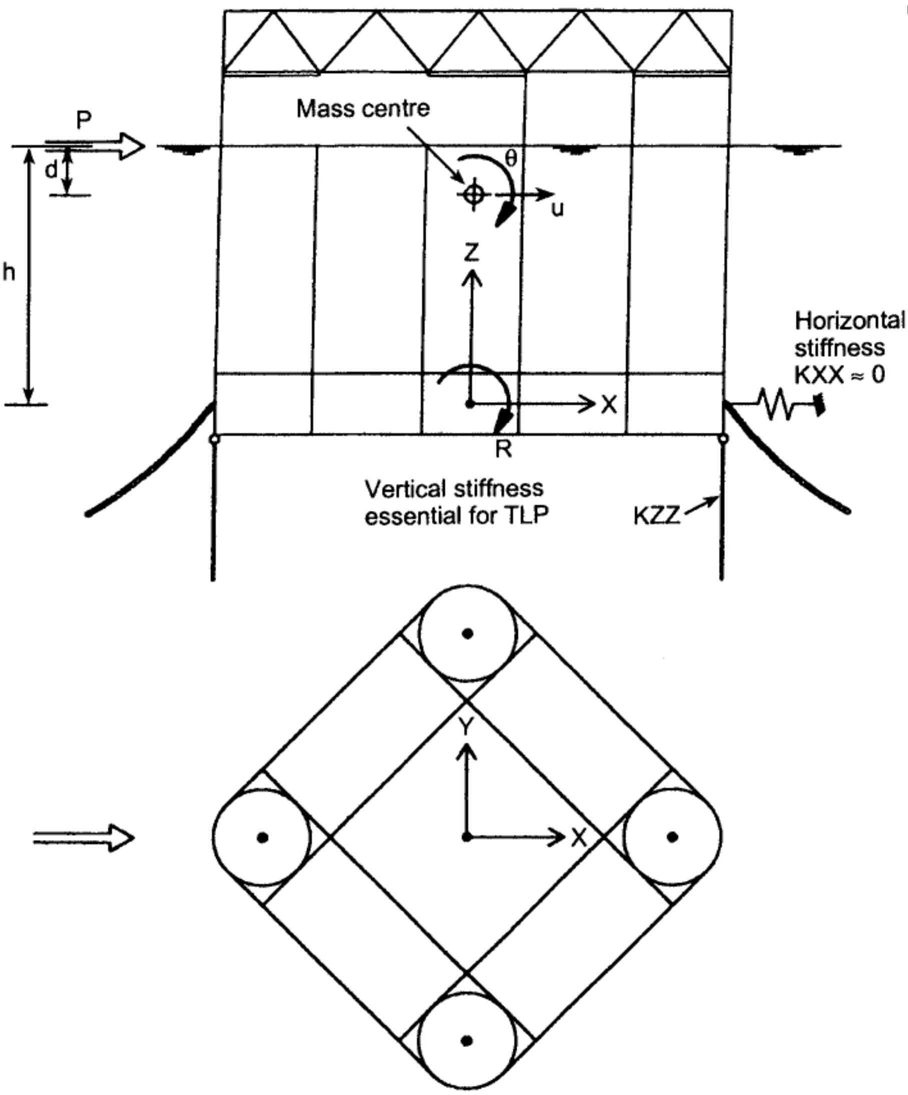


Fig. 3.25 Diagonal central impact

The two-parameter system for analysis now has X as horizontal translation and R for rotation about the global Y-axis. The following notation is used for stiffness and mass elements:

KXX	=	anchoring stiffness in translation
KRR	=	rotation stiffness about origo due to anchoring and waterline area
MXX	=	translation mass from platform and added mass
MRR	=	rotation mass about origo of platform and added mass
MXR	=	mass coupling since the mass centre lies outside origo.

With the origo in the elevation where the anchorage system enters the structure, no coupling terms enter the stiffness, and the following 2x2 relation comes out of the impact mechanics (damping neglected).

Platform:

$$\begin{bmatrix} KXX & 0 \\ 0 & KRR \end{bmatrix} \begin{bmatrix} X \\ R \end{bmatrix} + \begin{bmatrix} MXX & MXR \\ MRX & MRR \end{bmatrix} \begin{bmatrix} \ddot{X} \\ \ddot{R} \end{bmatrix} = \begin{bmatrix} P \\ P \cdot h \end{bmatrix} \tag{3.54}$$

Ship:

$$M_S \cdot \ddot{X} = -P \tag{3.55}$$

Below, Equations (3.54) and (3.55) are implemented in modes of response with different dynamic characteristics. The objective of the calculations is now to determine accelerations and inertia forces due to impact, so that sectional forces can be found.

3.6.4 Catenary anchored floater

For a floater, the motions in the horizontal plane in the form of X-translation (surge) and rotation about vertical axis Z (yaw) are inertia dominated, so that according to (3.2) the stiffness terms are neglected. It is now convenient to refer the displacements to the centre of mass for the system, added mass included. Fig. 3.25 depicts the symbols for translation and rotation directions of motion. For a catenary anchored floater the motion in roll or pitch is also inertia dominated. Equation (3.55) for the platform motion now becomes uncoupled also in terms of mass:

$$\textit{Translation: } M_P \cdot \ddot{u} = P \,, \; (0 \le t \le t_1) \tag{3.56}$$

$$\textit{Rotation: } I_{MP} \cdot \ddot{\theta} = P \cdot d \,, \; (0 \le t \le t_1) \tag{3.57}$$

where

M_P	=	translation mass of the platform, added mass included
I_{MP}	=	inertia moment of the platform, added mass included
d	=	distance from mass centre to impact force resultant
t	=	time
t_1	=	impact duration.

The ship and platform common velocity during impact is thereby:

$$\textit{Ship: } v_s = v_o - \frac{P}{M_s} \cdot t \;, \; (0 \le t \le t_1) \tag{3.58}$$

$$Platform:\ \dot{u} = \frac{P}{M_P} \cdot t\ ,\ (0 \leq\ t \leq\ t_1) \tag{3.59}$$

$$\dot{\theta} = \frac{P \cdot d}{I_{MP}} \cdot t\ ,\ (0 \leq\ t \leq\ t_1) \tag{3.60}$$

From (3.58–3.60) the duration of impact can be calculated as the time when ship and platform have the common velocity

$$v_s = \dot{u} + d \cdot \dot{\theta}\ ,\ (t = t_1) \tag{3.61}$$

Impact duration:

$$t_1 = \frac{v_o}{P \cdot \left(\frac{1}{M_s} + \frac{1}{M_P} + \frac{d^2}{I_{MP}} \right)} \tag{3.62}$$

The duration of impact t1 will normally be in the range 0.5–2.0 seconds.

An estimate of the platform indentation in the ship hull is now obtained from the displacements:

$$\Delta = \int_0^{t_1} v_s(t)dt - \int_0^{t_1} (\dot{u} + d \cdot \dot{\theta})dt \tag{3.63}$$

$$\Delta = \frac{v_o^2}{2P \cdot \left(\frac{1}{M_S} + \frac{1}{M_P} + \frac{d^2}{I_{MP}} \right)} \tag{3.64}$$

The assumption of constant impact force with time, causes (3.64) to underestimate the indentation if maximum force P is being used.

Fig 3.26 shows typical time functions for displacement u before and after impact. The maximum displacement u_{max} is obtained at time T/4 from contact, where T is the eigenperiod. The calculations for $t > t_1$ apply the homogeneous equation corresponding to (3.54), including the initial conditions at $t = t_1$. Ship and platform are to be considered as one mass.

Regarding sectional forces in the raft, the situation during impact is governed by $0 \leq t \leq t_1$. The global structure behaviour is in the form of frame response effects in the raft. This is easily calculated by hand as in Section 3.5.3, where impact force and inertia forces are included. Alternatively, a frame program is used.

3.6.5 Tension leg platform

For a TLP the eigenperiod in rotation Θ about the horizontal axis is in the range 2.0–4.0 s, and stiffness and mass determine response in this mode. Dynamic amplification of the response, as related to static stiffness dominated reactions, can take place. The horizontal displacement in surge u is inertia dominated, also for a TLP.

From the above, the tension by platform produces different dynamic characteristics for impact in the two modes of motion. As for the catenary anchored floater the stiffness KXX in (6.1) is neglected, and only inertia terms apply.

Referring to the mass centre, the parameters u and θ in coupling terms in stiffness are neglected. Referred to the mass centre, added mass included, the two dynamic equations read

$$Translation: M_P \cdot \ddot{u} = P \tag{3.65}$$

$$Rotation: K_{\Theta\Theta} \cdot \Theta + I_{MP} \cdot \ddot{\Theta} = P \cdot d \tag{3.66}$$

Equations (3.65) and (3.66) now substitute (3.56) and (3.57) for a catenary anchored floater. The equation of motion (3.55) for the ship is still valid. Implementing the initial conditions:

$$\Theta = 0 \text{ for } t = 0 \tag{3.67}$$

$$\ddot{\Theta} = 0 \text{ for } t = 0 \tag{3.68}$$

Equation (3.66) is solved

$$\Theta = \frac{P \cdot d}{K_{\Theta\Theta}} (1 - \cos \omega t), \ (0 \leq t \leq t_1) \tag{3.69}$$

with $K_{\Theta\Theta}$ being the rotation stiffness, dominated by the tethers.

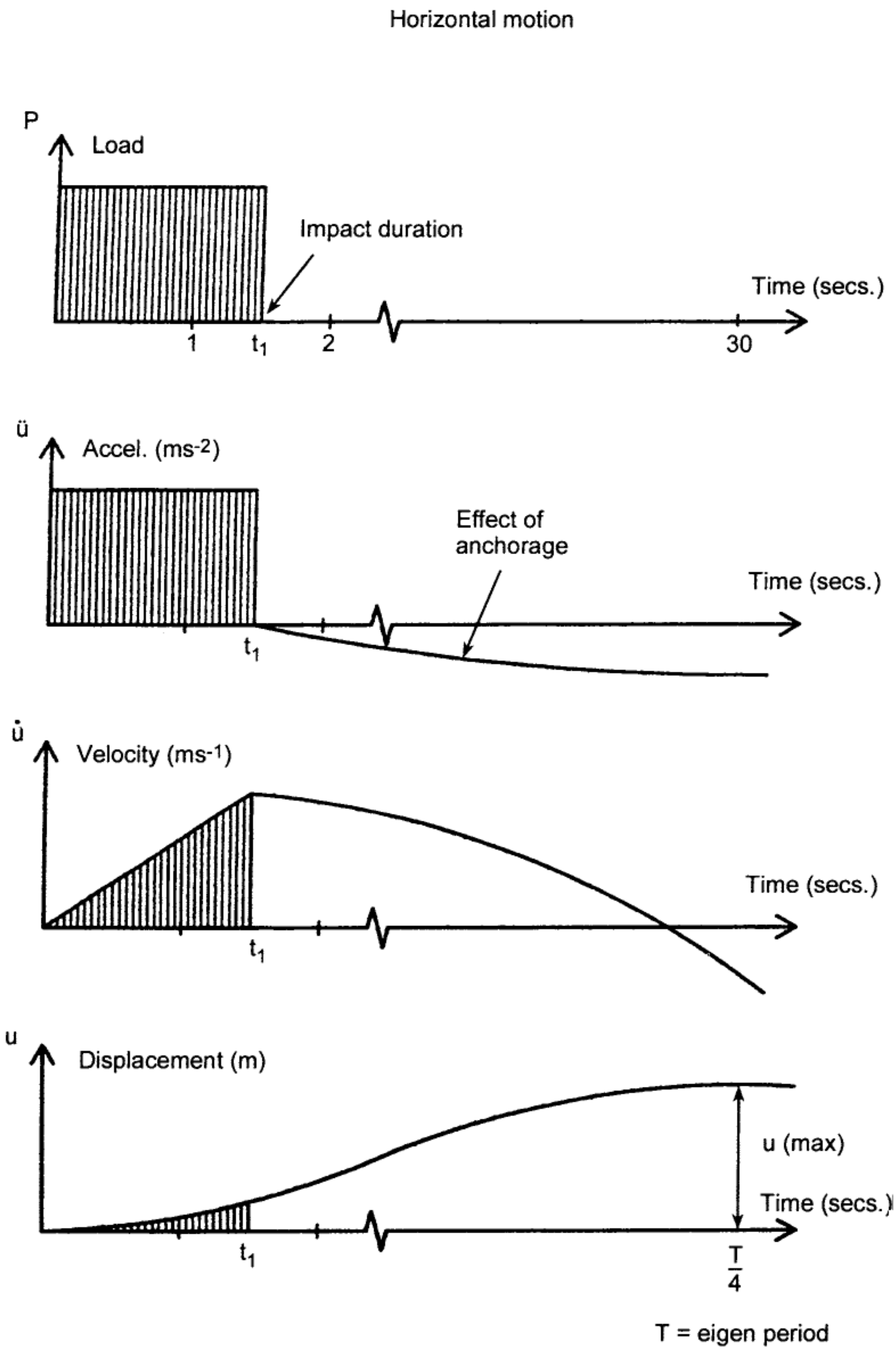


Fig. 3.26 Time history for inertia dominated displacement

Referring to Fig. 3.15, the rotation stiffness reads:

$$K_{1\Theta\Theta} = \sum \frac{E\,A_i}{L_i} \cdot a^2 \tag{3.70}$$

where EA_i / L_i is axial stiffness of a single tether. The second contribution to $K_{\Theta\Theta}$ comes from the waterline area:

$$K_{2\Theta\Theta} = \sum_4 \rho\, g\, A_j \cdot c^2 \tag{3.71}$$

In (3.71) four shafts are assumed with distance c from the platform centre. Again, (3.71) can be neglected in practical design. Equation (3.69) includes the eigenfrequency ω (rad per sec.) for rotation. As related to static solution, (3.69) comes out with a dynamic amplification:

$$DAF = (1 - \cos \omega t) \tag{3.72}$$

The extreme value of (3.72) is

$$DAF = 2{,}0 \;\; for \;\; t = \frac{T}{2} \tag{3.73}$$

where T is the eigenperiod. A dynamic amplification of 2.0 for the rotation mode seems reasonable.

The estimation of impact duration t_1 follows the procedure (3.58)—(3.62). For simplification, rotation terms may be neglected for the TLP.

Fig. 3.27 illustrates a typical time history for rotation Θ in the case of impact duration t_1 equal to half the eigenperiod T.

For global response in the raft, the instant t = 0, $t = t_1^-$ and $t = t_1^+$ are to be controlled, where t_1^- is the time just before the end of impact, and t_1^+ is correspondingly just after the impact. The reaction forces in the tethers give major contributions to retardation as soon as the impact force disappears.

The time history in Fig. 3.27 is also typical for the response in the shafts of a gravity base platform.

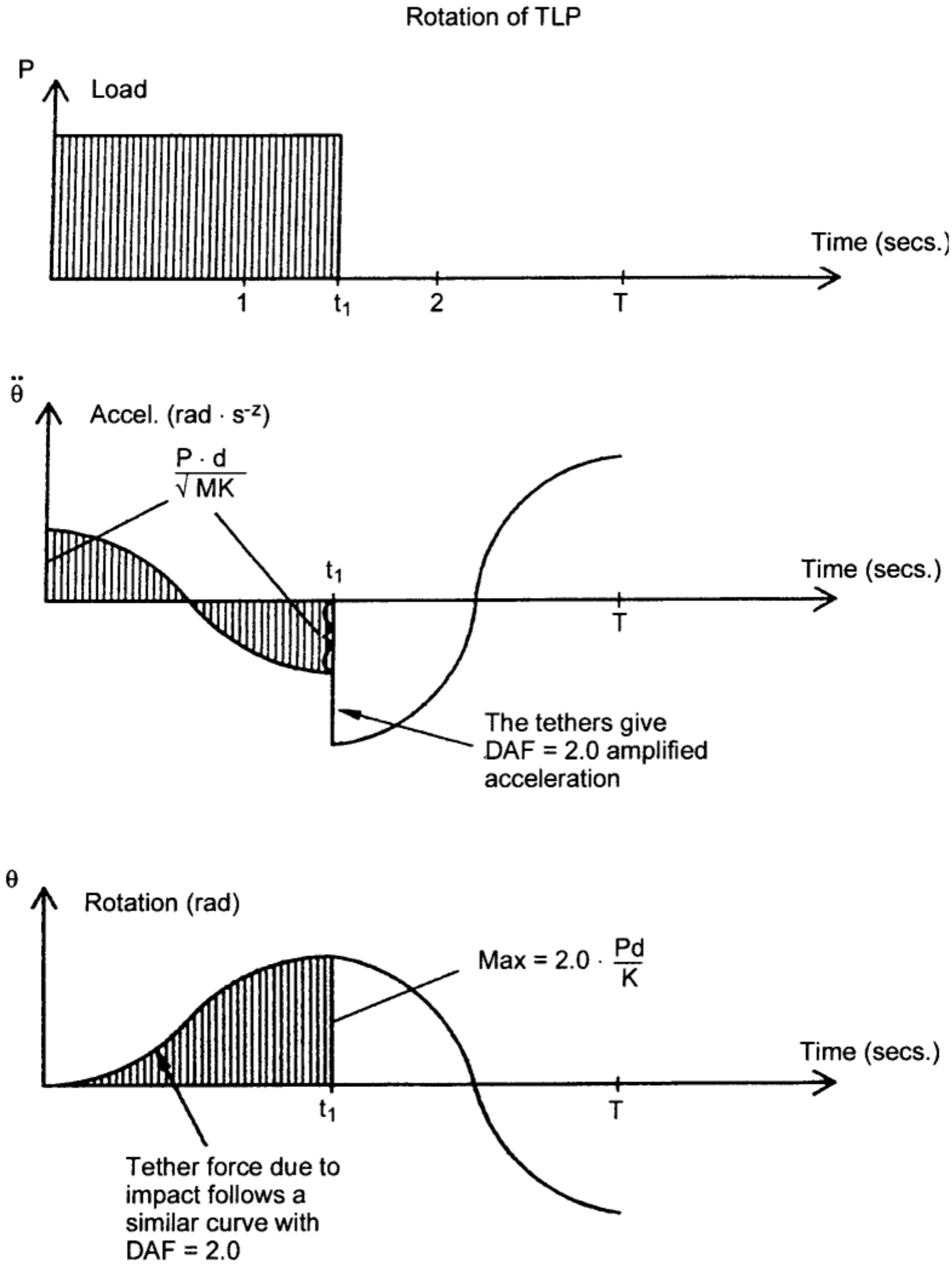


Fig. 3.27 Time history for roll of TLP

3.7 Non-linear geometric effects

3.7.1 General

This section deals with the incremental forces as a consequence of second order geometric effects. In Section 3.7.2 the basis for the second order bending moment on beams is described. The objective is to illustrate simplified rules for calculating additional moments in rules and standards (in e.g. (Eurocode 2, 1991), Section 4.3.5; (ACI 318–95, 1995), Sections 10.11–10.13; (NS 3473, 1998), Section A.12.2).

The formulas in Section 3.7.2 deal with the reduction of global stiffness of the shafts of a gravity base structure and the increase of the sectional forces due to increased deformations. Section 3.7.3 similarly shows the effect on vertical panels, and how the equivalent one-dimensional elastic buckling length can be calculated for such a two-dimensional structural element. In Section 3.7.4, circular cylindrical shells are reviewed. Finally, Section 3.7.5 shows additional forces on the raft due to large rigid body rotations.

3.7.2 Beam stiffness of shaft

The axial compression on the shaft reduces the bending stiffness and consequently the lateral deformation increases. This is illustrated by an example below.

With reference to Fig. 3.6, a free rotation is assumed at the coupling between the shaft and the deck. The deformation function along the shaft can be assumed:

$$w(x) = \delta \cdot \phi(x) \tag{3.74}$$

where δ is the displacement at the top of the shaft and φ(x) is the deformation function.

The modal material stiffness which includes the sectional elastic stiffness EI, is given as

$$K_M = \int_0^L EI\,(x) \cdot \phi_{,xx}^2 \cdot dx \tag{3.75}$$

φ_{fxx} is the curvature of the shape function.

The stiffness reduction from the axial compression N in the shaft is given by the geometric modal stiffness

$$K_G = N \cdot \int_0^L \phi_{,x}^2 \cdot dx = N \cdot K_\phi \tag{3.76}$$

where $\varphi,_x$ is the slope of the shape function.

The criterion for linear buckling is now for the one-parameter system

$$K_M - K_G = 0 \tag{3.77}$$

i.e. the ideal elastic buckling load

$$N = N_E = K_M / K_\varphi \tag{3.78}$$

For calculation of buckling load NE, the shape function φ_x is assumed and (3.75) and (3.76) integrated. Alternatively one can use the equations for variable sections from literature. A beam program with linear buckling calculations can also be used.

The modal stiffness for a given N will then be from (3.75) and (3.76):

$$\begin{aligned} K &= K_M - K_\varphi \\ K &= K_M (1 - N/N_E) \end{aligned} \tag{3.79}$$

With reference to Section 3.4 it is obvious that the natural period for lateral oscillation of the shafts increases by a factor

$$(1 - N / N_E)^{-0,5} \approx 1 + \frac{1}{2}(N / N_E) \quad \text{(moderate N)} \tag{3.80}$$

The effect on lateral displacement for a given load will be increased by the factor

$$(1 - N / N_E)^{-1} \approx 1 + N / N_E \text{ (moderate N)} \tag{3.81}$$

According to the elastic theory, Equation (3.81) also gives an incremental factor on the curvature and bending moments along the shaft

$$M_0 (x) = EI(x) \cdot k_0 (x) \tag{3.82}$$

where k_o is the curvature according to the linear solution.

The additional moment from second order geometry would then be

$$M_2(x) = M_0(x) \cdot \frac{N \cdot l_k^2}{\pi^2 \cdot EI_{eff}} \tag{3.83}$$

$$M_2(x) \approx 0.1 \cdot \frac{EI(x)}{EI_{eff}} \cdot N \cdot l_k^2 \cdot \kappa_0 \tag{3.84}$$

where l_k is the buckling length (2L for our cantilever) and EI_{eff} is the effective sectional stiffness for the shaft, adjusted for variation in EI and shape function.

The considerations above are made for one bending mode of the shafts over the caisson. Here, we are assuming full clamping at ground level and no flexibility of the caisson. The effect on the shaft from rotation of the caisson due to ground deformation would then be a rigid body translation and rotation. We can see from Equation (3.75) that the material stiffness K_M for the shafts is not affected since rigid body motion does not influence the curvature. On the other hand, (3.76) shows that the geometric stiffness is influenced by the soil stiffness. The procedures for shaft design given above are therefore not conservative for soft soils. To get the effect of ground deformation, a two-parameter system can be used that is analogous to Fig. 3.8.

3.7.3 Planar structural elements

The plane stresses σ_x, σ_y, τ_{xy} are transformed to principal stresses σ_1 and σ_2. Here σ_2 is the largest compressive stress. The stress σ_1 will be considered below only if it is compressive.

From the geometry and boundary conditions of a planar structural element, the critical buckling stress $\sigma_{2\,cr}$ is calculated with regard to the combined stress state σ_1/σ_2. Tables for ideal plate buckling from (Timoshenko and Gere, 1961) or (Column Research Committee of Japan, 1971) can be used. Another alternative procedure is:

a. Deal with each principal stress separately. Calculate the buckling stresses

$$\sigma_{2E} = K_2 \cdot \frac{\pi^2 D}{b^2 \cdot t}\ ,\ (\sigma_1 = 0) \tag{3.85}$$

$$\sigma_{1E} = K_1 \cdot \frac{\pi^2 D}{b^2 \cdot t}\ ,\ (\sigma_2 = 0) \tag{3.86}$$

where b is the effective plate width which can be different for σ_1 and σ_2, t the plate thickness, and D the sectional stiffness.

b. Find the relation α between the acting principal stresses, such that

$$\sigma_1 = \alpha \cdot \sigma_2 (\alpha > 0) \tag{3.87}$$

c. Critical principal stress σ_{2cr} from the combined effect can be derived from the interaction formula

$$\frac{1}{\sigma_{2,cr}} = \frac{1}{\sigma_{2E}} + \frac{\alpha}{\sigma_{1E}} \tag{3.88}$$

d. When the critical stress is found, the equivalent one-dimensional buckling length can be derived from:

$$\sigma_{2,cr} = \frac{\pi^2 \cdot Et^2}{12 \cdot l_{k2}^2} \tag{3.89}$$

i.e.

$$l_{k2} = 0{,}907\, t \cdot \left(\frac{E}{\sigma_{2,cr}} \right)^{0{,}5} \tag{3.90}$$

Buckling length l_{k2} from (3.90) is used in conjunction with the acting stress σ_2 for slenderness control according to Section 12.2.4 in the Norwegian Standard and to calculate the incremental moment according to (3.74) above. The two principal directions are to be considered for the incremental moment.

3.7.4 Circular cylindrical shell

The same procedure mentioned above in Section 3.7.3 is used to find the second order sectional forces. The only difference is the elastic buckling stresses.

Provided the membrane compressive stress σ_1 is in the hoop direction and σ_2 in the generatrix direction and that the membrane shear is negligible for second order effects, then the ideal buckling stress for σ_1 acting alone can be written (Timoshenko and Gere, 1961), or (Column Research Committee of Japan, 1971).

$$\sigma_{1E} = 0{,}807 \cdot \frac{Et}{l} \cdot \left[\left(\frac{1}{(1-\nu^2)} \right)^3 \cdot \frac{t^2}{r^2} \right]^{0{,}25} \tag{3.91}$$

where

E	=	modulus of elasticity of concrete
v	=	Poisson’s ratio
t	=	wall thickness
r	=	middle radius
l	=	effective length.

With σ_2 acting alone, the elastic buckling stress can be written (Timoshenko and Gere, 1961) or (Column Research Committee of Japan, 1971).

$$\sigma_{2E} = \frac{E}{(3\,(1-\nu^2\,))^{0,5}} \cdot \frac{t}{r} \tag{3.92}$$

provided that

$$l » 1.72 \cdot (rt)^{0.5} \tag{3.93}$$

With the help of interaction formulas as in (3.87) and (3.88) above, the critical stress can be calculated. It is convenient to use the hoop stress σ_1 as a reference rather then σ_2 as in (3.87) and (3.88), after which l_{k1} is calculated from (3.89) and (3.90). By setting l_{k1} in relation to the cylinder circumference, an estimate of the number of half waves in the buckling mode around the periphery is obtained. This is an important check particularly for cases where the circular cylinders do not form a full circle (similar to the cell walls of gravity base platforms). The validity of (3.91) for a partly complete cylinder can then be confirmed.

For slenderness control and for calculations of the second order moment, the procedure in Section 3.7.2 is followed. As a reminder the second order moments are to be calculated for both directions. Often the governing case is during the mating of the deck to the shaft.

For the lower part of a shaft of a floating platform or the cell walls of a gravity base structure, the stresses are derived from the water pressure p:

$$Hoop\ direction: \ \sigma_1 = \frac{p \cdot r}{t} \tag{3.94}$$

$$Vertical\ direction: \ \sigma_2 = \frac{p \cdot r}{2 \cdot t} \tag{3.95}$$

In other words, the vertical stress is half the hoop stress.

Practical values can be:

Wall thickness	t = 0.60m
Middle radius	r = 12.0m
Cylinder height	1 = 30.0m
Modulus of elasticity	E = 30 000 MPa
Poisson's ratio	v = 0.20

From (3.91) the critical buckling stress for the hoop force alone

$$\sigma_{1E} = 112MPa\ ,(\sigma_2 = 0) \tag{3.96}$$

and for vertical stress alone from (3.92)

$$\sigma_{2E} = 884\ MPa\ ,\ (\sigma_1 = 0) \tag{3.97}$$

From the stress relationship in (3.94) and (3.95) and the buckling stresses in (3.96) and (3.97) it is clear that, generally speaking, the hoop stress is more dominant with regard to second order effects.

3.7.5 Rigid body rotation of a floating structure

The computational models for the global effects on the raft, shown in Section 3.5 and Fig. 3.17, are geometrically linear as the rigid body modes are not included, and the deformation modes are assumed to give infinitesimal displacements. Note that displacements include both translation and rotation.

This last assumption about the minor displacements from the deformation modes is valid for most of the floating platforms. At the same time, the rigid body rotation in a catenary anchorage situation can be so large that additional forces are exerted on the raft. An uncontrolled accidental water ballasting situation can result in a rigid body rotation (or tilt) of several degrees.

As shown in Fig. 3.28 such a rotation will lead to a lateral load on the raft due to the weight and also an increase in the hydrostatic pressure due to the deeper draft. The lateral force from the weight corresponds to the inertia force from acceleration in translation such as:

$$\ddot{X} = - g \cdot \sin \Theta \tag{3.98}$$

The change in water pressure is

$$\Delta p = p\ g \cdot X \cdot \sin \Theta \tag{3.99}$$

For purposes of comparison, a wave acceleration for a 100 year storm of about 1.0 to 1.5 m/s^2, leads to the same lateral force on the raft as a rotation of the raft of θ=6.0^0. The tilted position, shown in Fig.3.28, can also be critical for the connection between the raft and the deck.

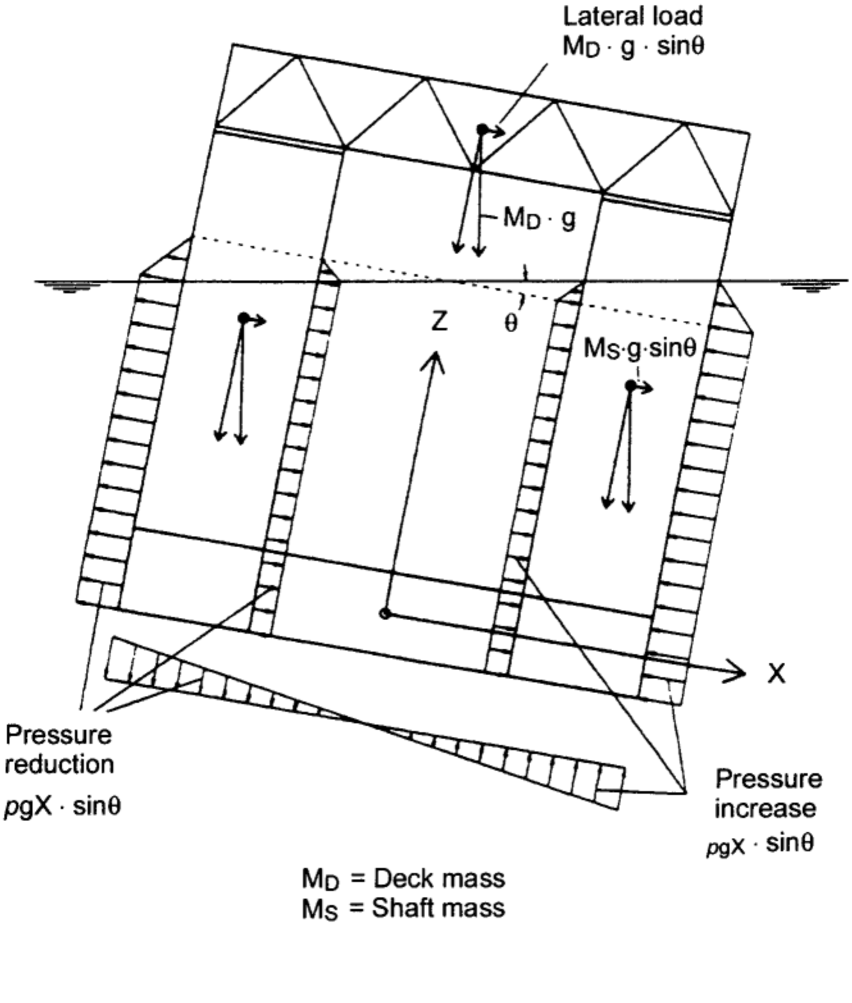


Fig. 3.28 Lateral forces from rigid body rotation

Further reading

The following references are recommended as a supplement to the present Chapter 3:

R.W.Clough and J.Penzien (1975): "Dynamics of Structures", McGraw-Hill.

S.P.Timoshenko and W.Krieger (1959): "Theory of Plates and Shells", McGraw-Hill.

S.P.Timoshenko and J.M.Gere (1961): "Theory of Elastic Stability", McGraw-Hill.

Column Research Committee of Japan (1971): "Handbook of Structural Stability", Corona Publishing Company; Tokyo.

T.Sarpkaya and M.Isaacson (1981): "Mechanics of Wave Forces on Offshore Structures", van Nostrand Reinhold Company.

T.H.Søreide (1981): "Ultimate Load Analysis of Marine Structures", Tapir, Norway.

References

American Concrete Institute (1995) Building Code Requirements for Structural Concrete (ACI 318–95) and Commentary—ACI 318R 95 (metric versions ACI 318M-95 and ACI 318RM 95)

Eurocode 2 European Prestandard ENV 1992–1–1. (1991): Design of concrete structures. CEN 1991 (under revision 1999 for transformation to EN, European Standard).

Norwegian Council for Building Standardisation, NBR (1998), *Concrete Structures, Design rules*. NS 3473, 4th edition, Oslo, Norway, 1992 (in English), 5th edition 1998 (English edition in print).

4 Global analyses

Ivar Holand, SINTEF

4.1 Objective

The analyses of offshore structures may be split in several steps:

- load analyses
- modelling, preprocessing
- global analysis
- study of selected areas by non-linear analyses
- postprocessing, dimensioning, including analysis of reinforcement needed.

In this chapter only the global analysis will be discussed. The other steps are, to some extent, discussed in Chapters 3 and 5.

The objective of the global analysis is to provide an accurate and detailed knowledge of the design load effects as they are distributed all over the structure. The load cases to be included in the linear global analyses must be chosen in such a manner as to allow all relevant load combinations to be achieved by linear combinations.

Global analyses of offshore structures are carried out exclusively by linear analyses. There are several reasons for using linear elastic analyses in accordance with the theory of elasticity, among which should be mentioned:

- The structures, in particular the caissons made up of circular cells, are complex, and the main stress distribution is not obvious.
- The offshore platforms are subjected to a large number of loading conditions during the construction, tow-out, installation, operation and removal phases. Large hydrostatic pressures dominate during deck-mating, while wave, current and wind loads dominate during the operation phase. To permit the handling of all relevant load cases, a number of basic load cases are selected. The actual load cases, with load factors for the relevant limit state, possible amplification factors etc, may in a linear analysis be obtained by linear scaling and superposition, allowing an automatic postprocessing.
- The solution found satisfies equilibrium and provides a reasonable distribution of statically indeterminate forces.
- The linear solution gives an adequate basis for design for serviceability.
- The simultaneous occurrence and the relative magnitude of several section forces may be important, e.g. the sign and magnitude of a normal force acting together with a shear force.

It is assumed that non-linear analyses will gain importance, since they provide reasonable correspondence between assumed material behaviour in an ultimate limit state design, and the distribution of internal section forces in the structure. They also enable the influence of displacements on load effects to be considered. However, when the large number of load cases and the resulting need to superimpose load effects are considered, the linear analysis is likely to remain the primary method in a foreseeable future.

Here, a global linear analysis will primarily be discussed, but non-linear analyses will also be touched upon. Dimensioning is discussed in Chapter 5 (Design). In non-linear analyses dimensioning cannot, however, always be separated from analysis.

4.2 Linear finite element methods

4.2.1 Description of methods

The structural analyses have, since the mid-seventies, mainly been based on the use of large finite element programs. Description of finite element methods are found in a number of textbooks, e.g. (Bathe, 1982), (Cook *et al*., 1989), (Zienkiewicz and Taylor, 1989), (NAFEMS, 1991), (Crisfield, 1991), (Saabye Ottoson and Petersson, 1992), (Hinton, 1992).

The finite elements may be categorized as:

- bar elements
- beam elements
- plane stress elements
- plate bending elements
- shell elements
- solid elements
- several kinds of special elements.

Whereas bar and beam elements describe the loadbearing at the same level of accuracy (or with greater accuracy) as when using the traditional strength of materials, the more sophisticated elements may be developed in several ways and with different levels of accuracy. Various versions of these elements are described in the textbooks mentioned above.

The choice of elements and models govern the accuracy. Hence, the person using a program must evaluate what accuracy can be achieved with the element type used. Accuracy here means accuracy compared with the linear theory of elasticity. Exact solutions according to the theory of elasticity may be obtained only in certain idealized cases. The power of the finite element method is that it provides a systematic approach and, correctly applied, a good approximation to the theory of elasticity.

Solid elements in the shape of regular or irregular hexahedrons with 8 nodes (one in each corner) have been frequently applied. If such elements are used, it must be recognized that irregular elements may cause considerable errors, and that the accuracy obtained by regular elements depends on the displacement functions and the integration approach applied (several variants of this element exist). In the same manner, irregular quadrilateral elements with 4 nodes for plane stress or plane strain should be used with care.

Solid elements with 20 nodes are more robust concerning the element shape, and generally give better results. This is even more the case for solid elements with 27 nodes; see Section 8.7 in (Zienkiewicz and Taylor, 1989).

In present usage in offshore structures solid elements are preferred, not because detailed linear stresses are needed, but because the modelling of a complex geometry and the application of water pressure on external surfaces is facilitated using solid elements. Substructuring is used to make the modelling more modular. The responses are stored in the database as stresses in the super-convergent (Barlow) points.

The largest finite element calculations may involve more than one million degrees of displacement freedoms and requires the use of super-computers (CRAY YMP/464 has been used for the largest analyses) (Brekke, Åldstedt and Grosch, 1994).

In design, stresses and stress resultants are also needed at the boundary of the structural part, normally at the interface with an other structural part, and these values must be found by extrapolation from the internal points. The accuracy of this extrapolation (linear, parabolic) must be assessed, and when using the results it should be observed how the actual type of quantity would vary close to an edge. An example of an unfortunate effect of extrapolation is shown in connection with Fig. 4.8.

Irrespective of the choice of element type, the user should thoroughly study the information on the choice of element type and the modelling of the element mesh to understand the limitations of the program system (see examples in Section 4.2.5).

If the program manuals or previous applications of the same program do not provide sufficient documentation for an evaluation of accuracy, users must use test examples to provide sufficient insight in the obtainable accuracy.

A way of testing accuracy is to use the element chosen for a simple, but thorough representative, static model with a known analytic solution. For example, a clamped beam, a simply supported quadratic plate, an axial-symmetric tank or a spherical shell with simple boundary conditions. Examples are shown in Section 4.2.5. Accuracy assessments and test analyses must be included in the documentation.

The results in areas with irregular elements should always be scrutinized. If elements in such areas are generated automatically, the mesh must be studied and adjusted as necessary.

In some places irregular elements cannot be avoided, for instance in a transition zone between small and larger elements, or in cases of irregular geometry, for example at the interface between cell elements or in domes. It is important that irregular elements are placed in areas where the stresses are of secondary interest, or in areas with small stress gradients. Stresses in irregular elements should be used with care in design. Such cases should be subject to special assessment, by comparison with earlier accuracy studies of similar cases, or by analysing simple cases by the same element model for the local area.

Re-entrant corners give singular points and thus theoretically infinite stresses in an analysis according to the linear theory of elasticity. At such points the calculated stresses may get any value, depending on the element model. The case is not critical, since stress resultants, which are used in the design, are finite. However, by studies of accuracy it is necessary to be aware of the phenomenon.

Tools to support the study of the accuracy of finite element models are so far only implemented in commercial software to a limited extent. A check described in textbooks referred to above is the "patch test". If the test is satisfied, the solution converges to the correct result when the element size approaches zero. The test, however, says nothing about the accuracy for a certain element size and shape. Accuracy of stress resultants, not local stresses, is vital in design of concrete structures (Mathisen, Kvamsdal and Okstad, 1994), (Kvamsdal and Mathisen 1994).

When it is deemed necessary to account for the non-linear behaviour of reinforced concrete, particularly because of cracking, additional non-linear analyses are adapted for isolated parts, using results known from linear analyses at the boundaries. Such analyses are required in highly stressed intersections between shell type structural members and for slender shell panels, where geometrical non-linearities may also be important.

4.2.2 Program systems

A number of program systems are commercially available. It is necessary to choose a program system that has or gives access to relevant pre- and postprocessors, including load generation procedures for the relevant types of loads. Postprocessing is discussed in Chapter 5, Section 5.3.2, where also relevant postprocessors are listed.

4.2.3 Modelling and element meshes for shell surfaces

The isolated structural parts usually have simple geometries such as plane plates, circular cylinders, spherical domes and circular ring beams. By modelling for the use of a shell program, a reference surface in the middle of the thickness is chosen, resulting in a mathematical model with surfaces which can easily be generated automatically. The procedure becomes more complex if the thickness varies and a simple mathematical expression is chosen for the concrete form side. The most correct reference surface for a static analysis is a middle surface, and a deviation between the middle surface and a reference surface creates normal forces forming a narrow angle with the middle surface. Even though the angle is narrow, the effect can be significant; since shear forces and shear strengths are small compared to normal forces. Hence, a correction is needed, either of the automatically generated nodal point geometry, or of the results, especially those related to shear forces, for deviations between the reference surface and the middle surface. If an element model with solid elements is chosen, the modelling may be simplified, but the physical problem returns when shear forces and axial forces are to be computed from the stresses; see Chapter 5 (Design).

The choice of element size depends on the element types and the gradients of the quantities which are to be described; in the present case this means how the stress resultants in shell- and plate-shaped structural parts vary over the surfaces.

The structural parts are joined together in such a way that the bearing function is complex and the gradients are not accurately known until the analyses have been completed. Nevertheless, knowledge about gradients for simpler cases with known analytical solutions provides good support.

For a one-way slab with a uniform load the gradients only depend on the width spanned. The element size may thus be chosen in relation to the width (example: tri-cell walls in Condeep platforms).

For singly and doubly curved shell surfaces the loads may be considered to be carried by membrane forces, with the qualification that the membrane state is disturbed by edges, which in our case are mainly horizontal or vertical. The edge disturbances materialize as bending moments and shear forces in the shell. These edge disturbances are described mathematically as damped sine and cosine functions and show large gradients. The element size close to the edge must be chosen in such a manner that the edge disturbances are sufficiently accurately described.

For a cylindrical tank with constant thickness and axial-symmetric loading, edge disturbances emerge from the horizontal edges (for a Condeep cell from the upper or lower domes, see also Chapter 3 (Simplified analyses)). The damping of edge disturbances, and hence the variation with the distance z from the edge is a function of the non-dimensional coordinate

$$z/\sqrt{(rt)} \text{ (r=radius, t=thickness)}$$

(The parameter $\sqrt{(rt)}$ is related to the elastic length as applied in Chapter 3, by a constant factor.) The variation is shown in Fig. 4.1 for a moment M and in Fig. 4.2 for a shear force Q acting at the edge.

To reduce uncertainties in the transition from analysis to design, it is also important to choose an element model that is suited for the subsequent design. This aspect is discussed more closely in Chapter 5.

As Figs 4.1 and 4.2 show, the edge disturbances are limited to a zone extending about $2z/\sqrt{(rt)}$ from the edge, and the large gradients are found within a distance of about $\sqrt{(rt)}$ from the edge (see also Fig. 4.5).

For edge disturbances for a spherical shell, the damping is approximately as for a cylindrical shell, when the radius of the sphere replaces the radius of the cylinder.

For an edge along a generator in a cylindrical shell, the damping is slower, and the situation more complex. The difference between a simple arch and an arch in a shell is mainly found in the restraining of tangential displacements of the arch in the shell, caused by the shell surface. The degree of restraining depends on the shell length measured along the generator, radius/thickness ratio, etc. The largest gradients occur for strong restraining. For our objective, namely an assessment of the element size, it is conservative to assume completely rigid tangential restraining. In that case the differential equation for the arch will be the same as for the axial-symmetric tank; the damping will be the same. Thus, the extension of the elements at a straight edge can be chosen equal to the extension of the elements in the direction of the generator at the curved edges.

4.2.4 Design brief

In order to have analysis approaches, organization, responsibility allocation and quality assurance thoroughly discussed and decided; a Design Brief should be worked out for the element analysis. The Design Brief should also describe error estimates. These documents may be separate or parts of a more general document. Design Briefs are discussed in more detail in Chapter 5.

4.2.5 Examples of accuracy studies

(a) Background for the examples

As a part of the investigation work after the loss of the Sleipner A platform (Holand, 1994) Statoil contracted SINTEF to carry out a number of studies of accuracy. A selection of examples from these studies is included here. The good accuracy demonstrated for the cylindrical walls is assumed to represent a normal accuracy obtained by finite element methods. The errors for the tri-cell walls are, however, particularly large, and illustrate the risks that may be connected with the application of finite element programs, and the necessity of a careful application of program systems.

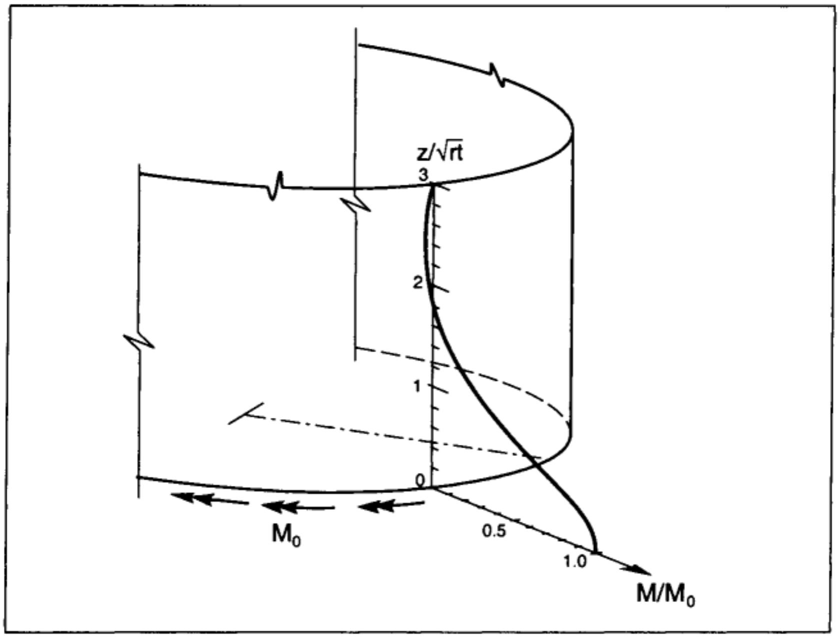


Fig. 4.1 Variation with distance from edge, of a moment M, acting at a curved edge of a cylindrical shell

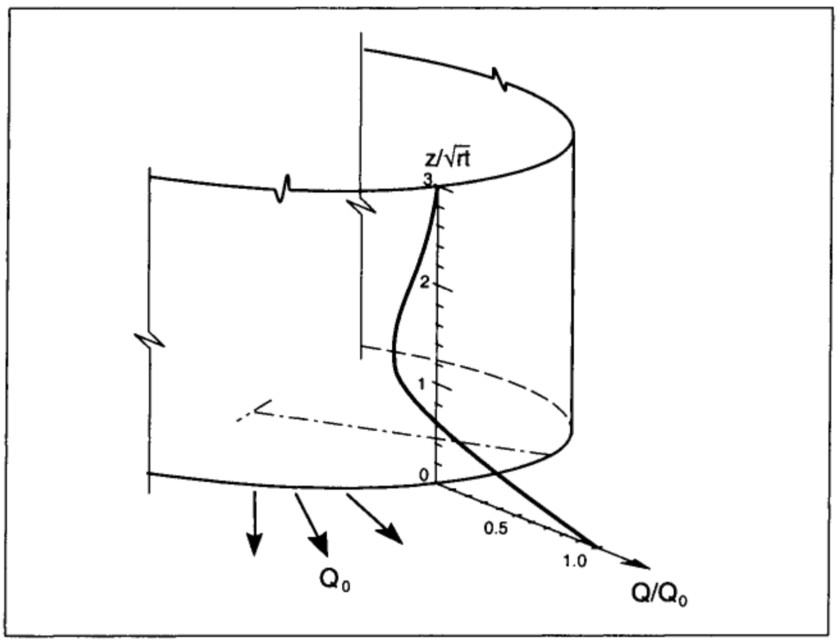


Fig. 4.2 Variation with distance from edge, of a shear force Q, acting at a curved edge of a cylindrical shell

(b) Cylindrical shell

Fig. 4.3 shows a cylindrical wall constituting a part of an axial-symmetric cell with external pressure. The wall is assumed to be clamped at the upper edge. By also clamping the lower edge, symmetry is obtained about a section at the middle of the height, as shown in Fig. 4.3. The model is reasonably close to the actual situation in an external cell in a Condeep platform and is assumed to be representative for a model case suited for an investigation of accuracy.

The shell has been modelled by solid elements, and an element model shown in Fig. 4.4. Each element has 8 nodes. Moments and shear forces computed by using the program system NASTRAN are shown in Fig. 4.5.

With the simple assumptions a differential equation may also be used for the analysis of the cell. Such an analysis gives a clamping moment of 1.75 MN and a shear force at the edge of 1.873 MN/m. The results from the element analysis (1.71 MN, respectively 1.849 MN/m,) are thus very close to the theoretically exact ones, and the model for the cylinder wall must be considered to be fully satisfactory.

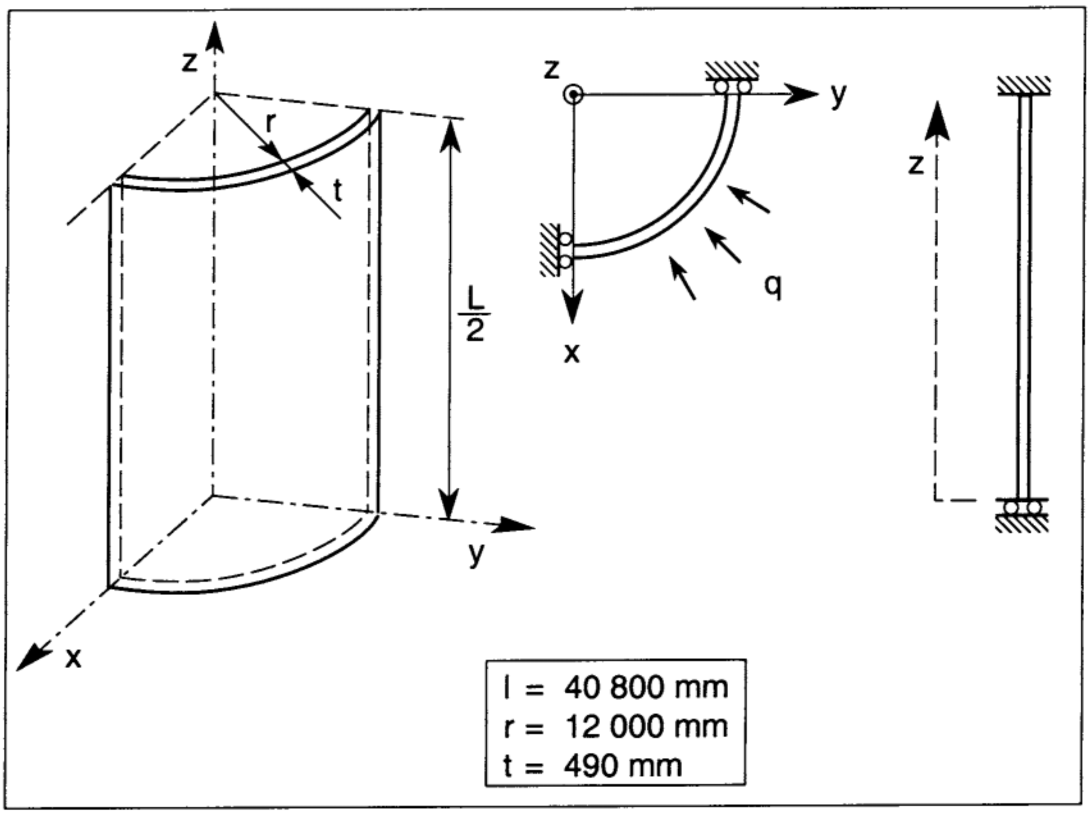


Fig. 4.3 Cylindrical wall. Geometry and edge conditions

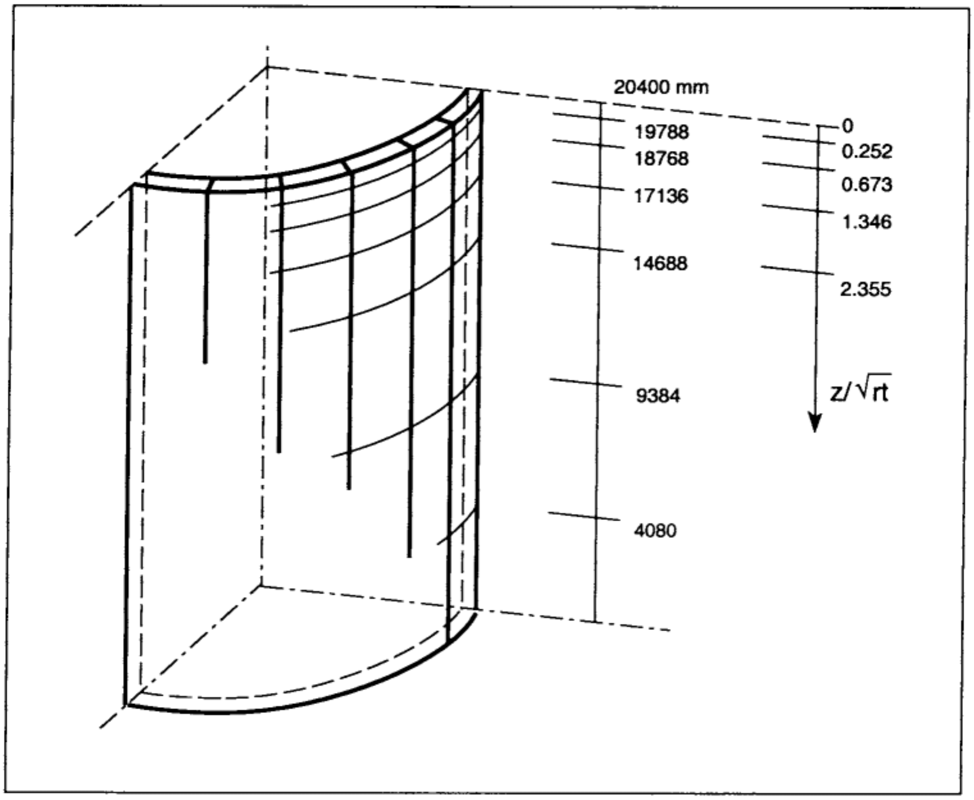


Fig. 4.4 Element model for cylindrical wall

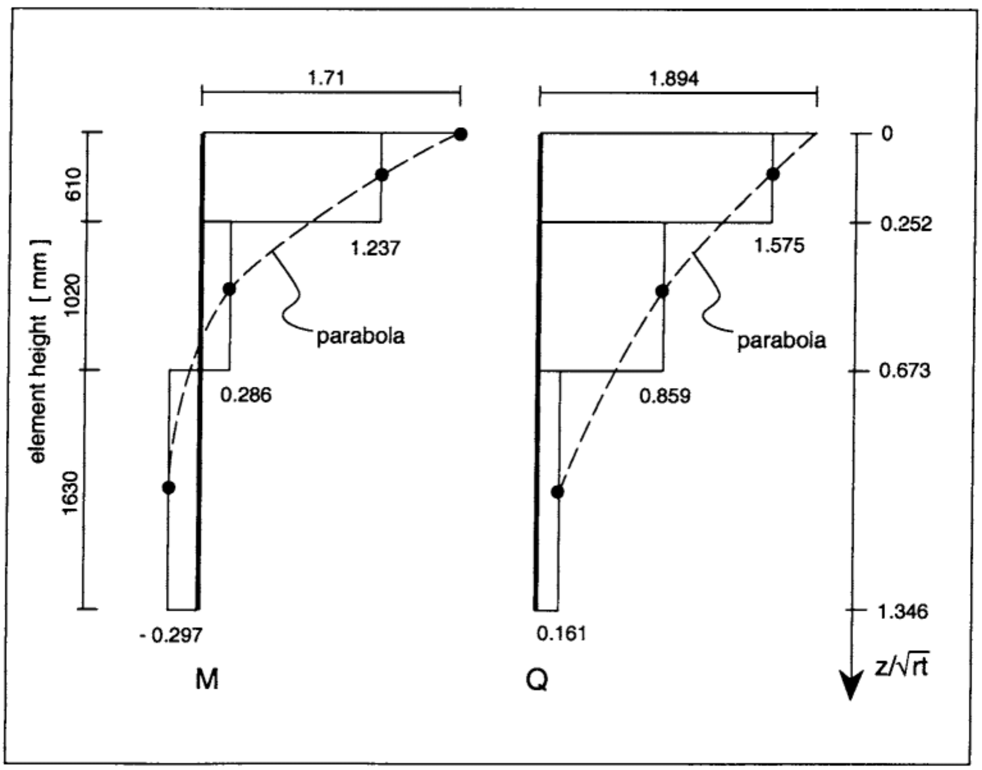


Fig. 4.5 Moments [MNm/m] and shear forces [MN/m] in cylindrical wall

(c) Plane cell wall

Fig. 4.6 shows a section of an element model as it has been modelled by a pre-processor. The section has been placed around a tri-cell and includes the tri-cell walls and adjacent walls. (A more complete description of a structure of this type is found in Chapter 3.) The accuracy obtained by this type of modelling will be investigated for the load case internal water pressure in the tri-cell. The problem is local and may be studied by analysing the local area included in the plane element model in Fig. 4.6. The pre-processor has produced the element model by dividing the inside as well as the outside of the cell wall in equal element side lengths. The procedure may seem logical, but the skew elements cause problems.

An analysis using NASTRAN yields bending moments and shear forces in the cell wall as shown in Figs. 4.7 and 4.8. Computed values in the centres of gravity of the elements are used as a basis and are extrapolated to the edge. The correct sum of moments in the middle of the span and at the support is $ql^2/8$, whereas the distribution between span and support depends on stiffness. As shown in Fig. 4.7 the analysis yields a moment sum, which is 15% too small.

The shear force varies linearly and is equal to $ql/2$ at the support. Here the result is still much worse, since the element at the support gives too small a value, and even more since the extrapolation by a parabola through points A, B and C is unfortunate. Depending on what point is considered, the shear force is 38 or 41% too small.

A change to regular elements as shown in Figs. 1.6 and 1.7 gives a moment sum as well as a shear force very close to the exact ones.

The conclusion is that the element in the NASTRAN version used involves the risk of large errors even for relatively moderate deviations from the rectangular shape, but also that such deviations may be revealed by simple investigations.

Much better stress resultants may be obtained by using a different interpretation of the results from the element analysis (Mathisen, Kvamsdal and Okstad, 1994). It is shown in the reference that the same example, with the element model as shown in Fig. 4.6, but with an interpretation of results based on virtual work, gives exact axial and shear forces and an error of only 7% in the moment in the critical section.

4.3 From linear analysis to design

The stress resultants found by the linear analysis are correct only if the material is homogeneous and linearly elastic. However, they are in any case a good approximation, since they satisfy equilibrium and also to a reasonable extent the relations between strains and stresses. It should, however, be observed that only the nodal forces satisfy equilibrium completely. When the program computes strains from nodal displacements, and later stresses and stress resultants from the strains, and even extrapolates the results to the boundary, the procedure is not uniquely defined. Also, there is no guarantee that these stress resultants satisfy equilibrium. For a good element model the deviations are small, but nevertheless, these uncertainties contribute to the needs for accuracy investigations as discussed in Section 4.2; see also examples in Section 4.2.5

The further design process in the ultimate limit state is not based on the theory of elasticity, but on the assumption that concrete cannot take tensile stresses. The process also uses design procedures that are analogous to those used in traditional concrete design; see Chapter 5. Even if design is based on assumptions that are different from those used in the analysis, the stress resultants are not corrected for this difference.

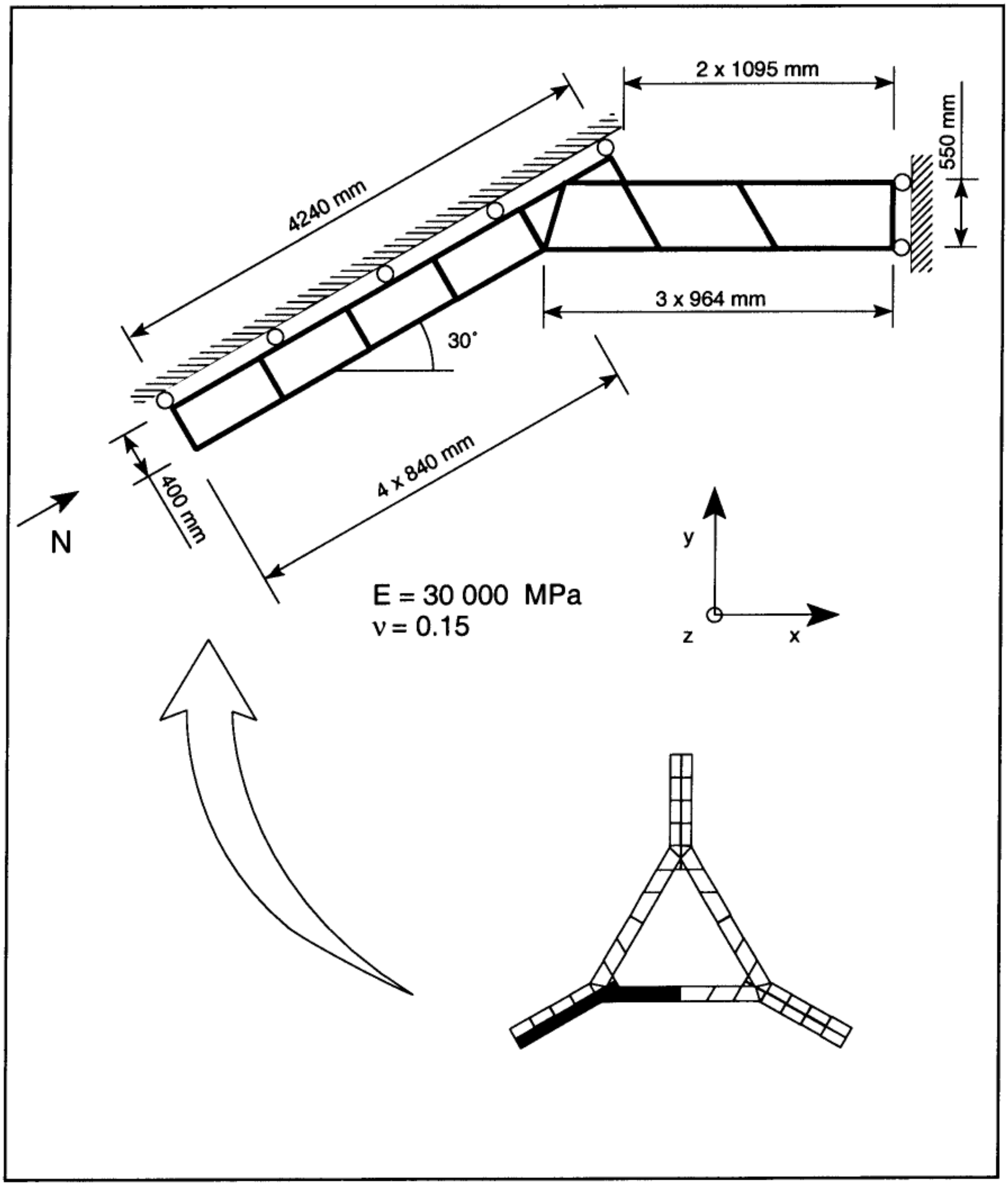


Fig. 4.6 Generated element model for tri-cell

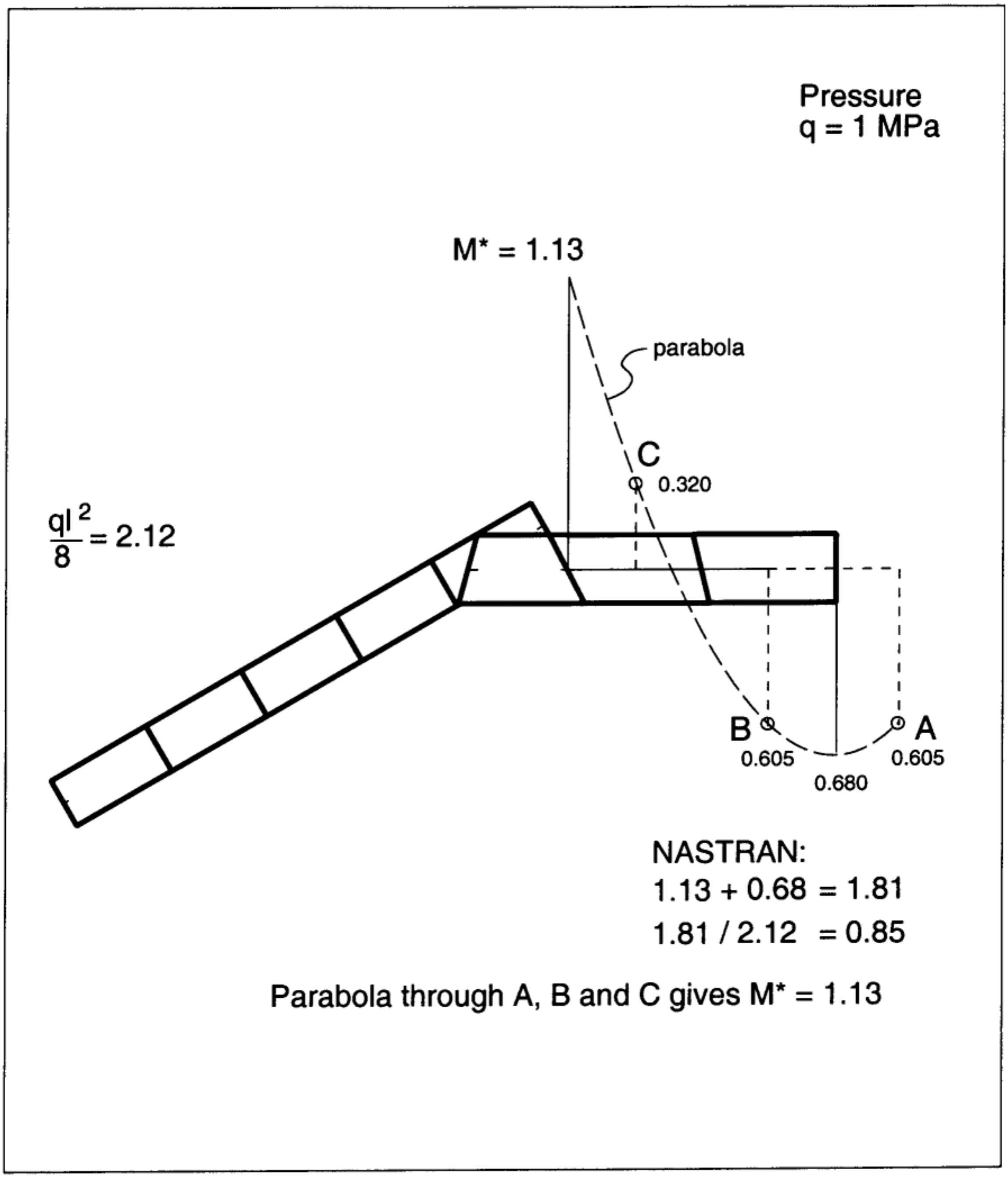


Fig. 4.7 Moment [MNm/m] in tri-cell wall with generated mesh

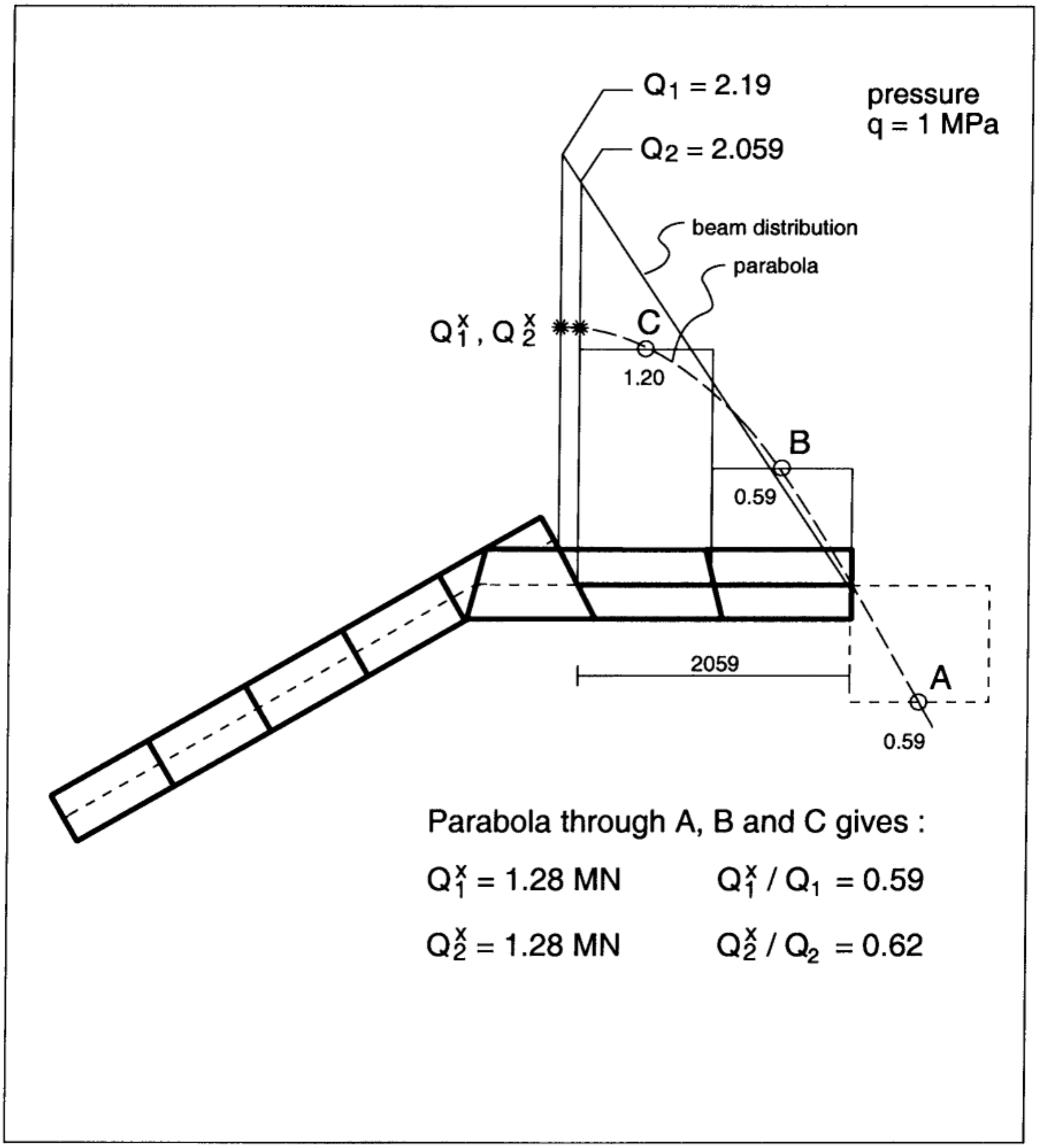


Fig. 4.8 Shear force [MN/m] in tri-cell wall with generated mesh

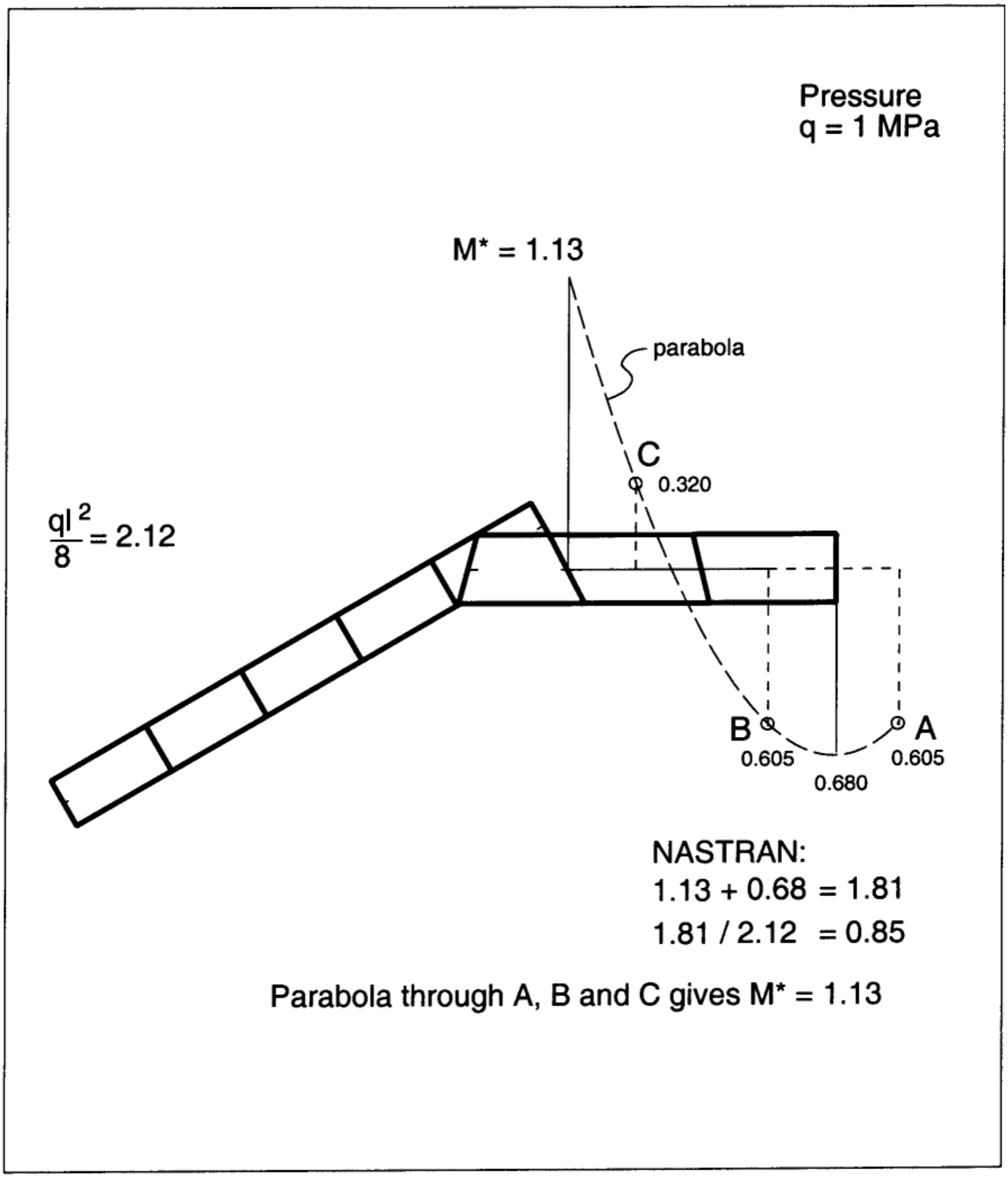


Fig. 4.9 Moment [MNm/m] in tri-cell wall with modified mesh

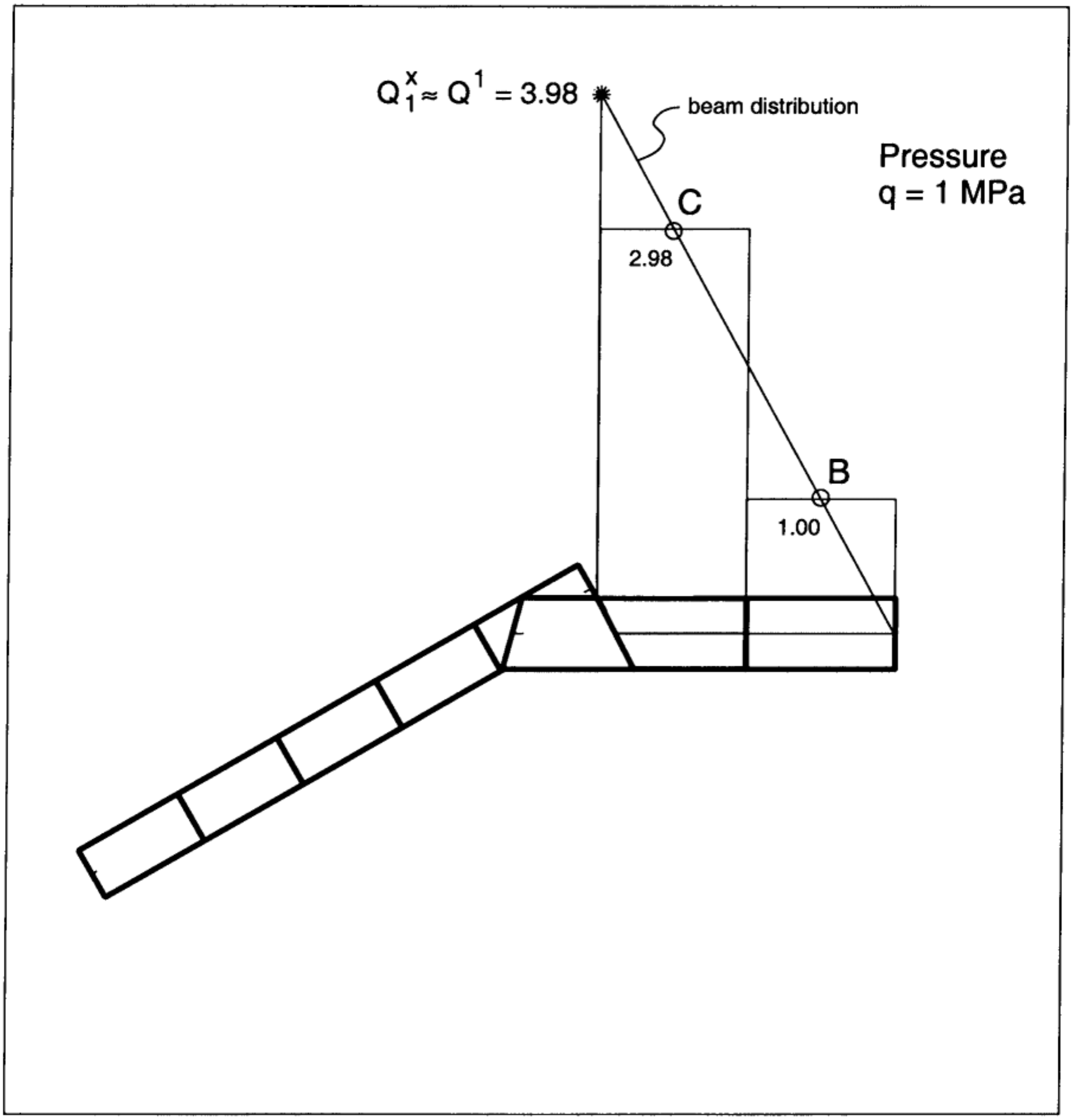


Fig. 4.10 Shear force [MN/m] in tri-cell wall with modified mesh

Normally the deviations caused by non-linear behaviour are small (in the case of statically determinate structures even zero), and the procedure is feasible. Nevertheless, there are cases where non-linear analyses are needed, see Section 4.5. If in doubt, displacements due to non-linear behaviour should be estimated and resulting additional moments and forces assessed.

4.4 Postprocessing

When the linear analysis for all chosen basic load cases is completed, the results are stored on data files for subsequent processing as described in Chapter 5. The transfer of data to the postprocessor, which may even be found in a different organization and be used by many others, is a critical process. This implies that the transfer must be subjected to quality assurance according to specified routines, which will prevent sources of error, such as incorrect files being used in the postprocessing. Moreover, the stress resultants from the postprocessing must be checked against the relevant loads.

The postprocessor should, in addition to its primary objective to be a design tool, also be organized as an efficient tool in checking that the results are correct. The postprocessor should, hence, include routines allowing diagrams to be drawn in a form which is suited to check the structural response. The requirement implies that the routines should be especially adapted to the needs of the design of reinforced concrete structures. The diagrams should include stress resultants in the relevant shell and plane sections (M_x, M_y, M_{xy}, N_x, N_y, N_{xy}, Q_x, Q_y) and stress resultant over larger portions, for example bending moments and shear forces in shafts (columns). Displacement diagrams are also valuable tools to understand the structural behaviour, and to check the results.

Generally, it may be said that assumptions inherent in the analysis model also are to be found as results in the output. A large amount of data is produced, and it is essential to sort out the critical load cases and concentrate diagrams and discussion of results on these critical cases. Such studies should be included in the documentation.

The part of the postprocessing that is directly related to the design of reinforced concrete is discussed in Chapter 5.

A main cause of concern about the linear finite element analysis is whether it produces a design that is too expensive, especially if the resulting amounts of reinforcement are unnecessarily high. The linear theory of elasticity often provides large local stress peaks in locations with irregular element geometry. The finer the element mesh, the more accurately local stress peaks are described by the finite element methods. Similarly, it may be questioned whether it is necessary to design for large local section forces caused by elastic restraints. In traditional concrete design such stress concentrations have often been neglected or reduced by smoothing the peaks over larger areas. Also for marine concrete structures it may be unnecessary to design for all stress peaks. Procedures must be evaluated in the modelling, for instance by reducing stiffness, or in design, by smoothing, which may be included in the postprocessor. It is difficult to give rules for this, but reductions must be restricted to stresses that are unnecessary for equilibrium, and smoothings made in such a manner that the total stress resultants satisfy equilibrium. They must, furthermore, be restricted to the ultimate limit states.

Such smoothings and reductions are obviously accepted in accidental situations, where local failure is accepted provided that no total structural failure results.

4.5 Non-linear analyses

4.5.1 Reasons for non-linear analyses

(a) Objectives

When it is deemed necessary to account for the non-linear behaviour of reinforced concrete, particularly because of cracking, additional non-linear analyses are adapted for isolated parts, using results known from linear analyses at the boundaries. Non-linear finite element methods are described e.g. in (Zienkiewicz, O.C. and Taylor, R.L. 1991).

In non-linear analyses the superposition principle is not valid, and analyses must be performed for each case separately. A consequence is that such analyses must be restricted to a few critical areas and special load cases.

Non-linear methods give significantly more complex numerical analyses than the linear ones. Hence, much more care must be exercised to avoid malfunction of such programs; for instance that the analyses are aborted too early by a convergence criterion that does not function as planned.

Reasons why linear analyses are not sufficient in some case may be divided into two categories: geometric non-linearities and material non-linearities.

(b) Geometric non-linearity

Geometric non-linearity occurs when the structure is so slender that displacements play a significant role for the static behaviour. Typical examples are the curved walls in cells and shafts in Condeep platforms. In these walls, displacements combined with deviations from ideal geometry and large axial forces in particular in the arch direction cause additional moments in the walls that are of a magnitude which must be considered in the design (risk of implosion). For the bending of a shaft as a long column, the axial loads may also give significant additional moments because of lateral deflections.

(c) Material non-linearities

Material non-linearity occurs in all design of reinforced concrete, for example when cracking of concrete and yielding of reinforcement is considered. Here, however, only the cases are considered where this causes significant changes in the load effects. Relevant locations are found where cell walls meet, in the transition from the vertical wall to the dome etc.

Non-linear analyses in such cases can contribute to giving improved understanding of the real behaviour of reinforced concrete, and may be applied directly in design. In such analyses, the need to consider multi-axial stresses and strains will often arise. Such considerations are generally outside the rules in codes and standards. Hence, a description is needed of the material models which the actual program applies, and the models must be assessed in relation to the requirements in codes and standards, in addition to research results and experience that are not implemented in rules. Related topics are discussed in Chapter 5.

4.5.2 Fracture mechanics

The present section could belong to the previous one, but the subject is nevertheless so special that it is discussed separately. Fracture mechanics considerations are needed when the formation and progression of cracks are particularly critical. An example is water pressure intruding in a crack as the crack progresses, a case which may occur where external curved walls in Condeep platforms meet, where there is risk of delamination (see Chapter 5), etc.

Fracture mechanics considerations are based on the recording of fracture energy in small specimens, and should be applied with care until the results from analyses have been calibrated with larger-scale tests.

4.5.3 Program systems

A number of program systems for non-linear analyses are available on the market. The following program systems have been applied for non-linear analyses of concrete platforms:

- Fenris (Det Norske Veritas Sesam AS, Høvik, Norway)
- Abaqus (Hibbitt, Karlsson & Sorensen, Pawtucket, RI, USA)
- Diana (Diana Analysis, Delft, The Netherlands)
- Solvia (Solvia Engineering AB, Linköping, Sweden).

For non-linear analyses a detailed study of the results is always needed to understand what the results tell us. Hence, the programs must be user-friendly in the sense that all assumptions are explained and the results are easily available as diagrams that are suitable for reinforced concrete.

4.6 Verification

4.6.1 Verification methods

Verification is the main topic in Chapter 7. Under the present heading only special issues related to the verification of finite element analyses are discussed. The following methods may be relevant:

- step by step checking of the analyses at hand (by document review) including check of discretization errors by error estimation analyses
- check of results (equilibrium, compatibility between displacements and between displacements and strains)
- check by independent simplified methods
- check by independent finite element analyses for all load cases or for critical load cases.

Checking that correct load combinations are included in the design load effects also belongs under this item.

When using finite element analyses, results are often presented in a form that is not easily accessible for verification. It is definitely unacceptable to include such results in an annex to the analysis report and refer to the annex. A summary section in the main report, where the annex is digested and the essence extracted, should supplement all annexes with detailed results. This procedure is also an important contribution to the internal control by the person who has performed the analyses to meet the requirement: “correct the first time”. As an example of guidelines for independent calculations the following guidelines by the Norwegian Petroleum Directorate (NPD, 1992) are quoted:

> "The design of structures or structural parts of significance to the overall safety should be verified by means of independent calculations. Such verification may be carried out by manual calculations or by computer calculations. When computer calculations are used, it is assumed that the person carrying out the verification uses another software programme than the designer. Software used in verifications should itself be verified for the purpose in question. The necessary calculations should be sufficiently accurate and extensive to clearly demonstrate that the dimensions are adequate".

4.6.2 Checklists

A checklist for verification of element analyses must be adapted to the verification procedures proposed in Chapter 7. Relevant key words are:

- qualification requirements
- linear/non-linear analyses
- program system
- element type
- element mesh
- input data
- accuracy of element model (checking report)
- load vectors
- transfer between pre-processor and main analysis, storing
- transfer of data to postprocessor
- retrieval of correct data (make certain that the last corrected data are used)
- transfer of analysis results to design (often: stresses to stress resultants)
- numerical accuracy

Special items for numerical accuracy are:

- precision level
- tolerances (frequently user controlled)
- condition numbers
- residual forces
- checking capabilities in the program
- what is default and what can be controlled by the user?

Special key words for non-linear analyses are:

- material models for concrete and reinforcement
- cracking criteria
- strain assumptions (for example plane stress, plane strain)
- modelling of reinforcement bars (for example individual bars or smeared)

- connection of individual bars to nodes
- water pressure in cracks
- geometric deviations
- convergence criteria
- fracture mechanics analysis

4.6.3 Qualification requirements

Qualifications must be required in two areas when finite element analyses are applied.

One area relates to engineering. The person responsible for element types and models, and for evaluation of results, must understand shell theory and finite element methods and have extensive knowledge of important restrictions in the programs that are to be used. The person must also have sufficient knowledge of all neighbouring disciplines, without needing to be a specialist in, for instance, wave loads or soil mechanics.

The other area relates to computing. The person responsible for the computer runs must understand the various risks of errors, for instance caused by numeric problems.

It is hardly appropriate to require that these two qualification areas are to be mastered by one person. It is also necessary to require that there is a fully qualified deputy for the responsible person, in case of illness, or in hectic periods requiring work far beyond normal working hours.

References

Bathe, K.J. (1982) *Finite Element Procedures in Engineering Analysis*. Prentice Hall, Englewood Cliffs, N.J.

Brekke, D.-E., Åldstedt, E. and Grosch, H.: Design of Offshore Concrete Structure Based on Postprocessing of Results from Finite Element Analysis (FEA), *Proceedings of the Fourth International Offshore and Polar Engineering Conference,* Osaka, Japan, April 10–15, 1994.

Cook, R.D., Malkus, D.S. and Plesha, M.E. (1989) *Concepts and Applications of Finite Element Analysis.* John Wiley and Sons, New York.

Crisfield, M.A. (1991) *Non-linear Finite Element Analysis of Solids and Structures, Vol. 1: Essentials.* John Wiley and Sons, Chichester.

Hinton, E. (1992) *Introduction to Non-linear Finite Element Analysis.* NAFEMS Publications, NAFEMS, Birniehill, East Kilbride, Glasgow G75 0QU.

Holand, I. (1994) The Loss of the Sleipner Condeep Platform. *First Diana Conference on Computational Mechanics,* Delft, The Netherlands, October.

NAFEMS (1991). A *Finite Element Primer.* National Agency for Finite Element Methods and Standards, NAFEMS Publications, Birniehill, East Kilbride, Glasgow G75 0QU.

Norwegian Petroleum Directorate (1992) *Regulations concerning loadbearing structures in the petroleum activities, stipulated by the Norwegian Petroleum Directorate, Stavanger, Norway.*

Mathisen, K.M., Kvamsdal, T. and Okstad, K.M. (1994) Techniques for Reliable Calculation of Sectional Forces in Concrete Structures Based on Finite Element Computations. Department of Structural Engineering, The Norwegian Institute of Technology.

Kvamsdal, T. and Mathisen, K.M. (1994) Reliable Recovery of Stress Resultants. *First Diana Conference on Computational Mechanics,* Delft, The Netherlands.

Saabye Ottosen, N. and Petersson, H. (1992) *Introduction to the Finite Element Method.* Prentice Hall, 1992.

Zienkiewicz, O.C. and Taylor, R.L. (1989) *The Finite Element Method, 4th Ed, Vol. 1 Basic Formulation and Linear Problems,* McGraw-Hill, London.

Zienkiewicz, O.C. and Taylor, R.L. (1991) *The Finite Element Method, 4th Ed, Vol. 2: Solid and Fluid Mechanics, Dynamics and Non-Linearity.* McGraw Hill, London.

5 Design

Erik Thorenfeldt, SINTEF

5.1 Typical structures and structural parts

Typical offshore concrete structures are discussed in Chapter 1, see Figs. 1.1, 1.2, 1.3 and 1.4. A sketch of a typical Condeep structure is shown in Fig. 2.1 and a sketch of a tension leg platform in Fig. 2.2. Typical structural parts and loadings appear for Condeeps from Figs. 3.4, 3.5 and 3.11, and for a floater from Figs. 3.12 and 3.13.

The structures are mainly cell structures, which in principle are composed of slabs, plates and shell elements. Simple massive beams, columns and frames occur relatively seldomly. Columns and frames as parts of the main structure usually have cross sections in the form of hollow cylinders or rectangular boxes which are designed locally as slab/plates/shell elements.

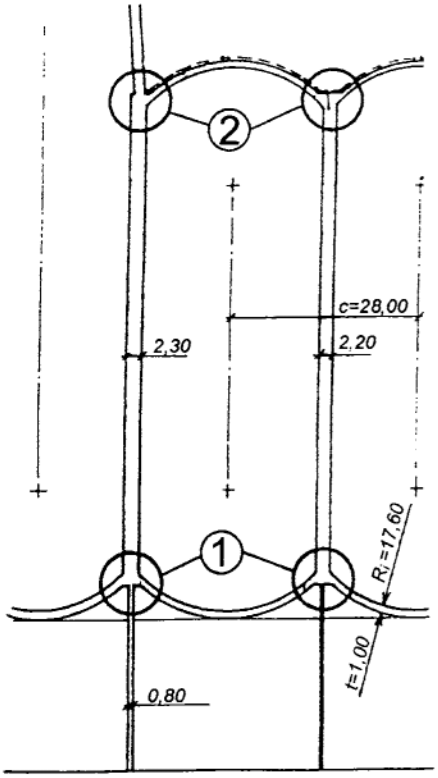
A Condeep structure is usually divided into skirts, lower domes, cell walls, upper domes and shafts (see Fig. 2.1). A floating platform will comprise other typical structural parts: pontoons, cylindrical columns and box beams.

Design of offshore concrete structures is in many respects similar to the design of large structures in general. The typical characteristics are the complexity caused by the numerous disciplines involved, among them

- soil mechanics
- loads from wind, waves and current
- accidental actions
- dynamic forces
- dynamic structural response
- non-linearities.

This Chapter 5 mainly discusses the design of the concrete structure itself when the analyses have been completed and the load effects determined. The main emphasis is placed on typical aspects which are especially important for producing a safe design.

In all types of platform structures the intersections between the different shell and plate structures represent critical parts of the structure and the design. Some important intersections between the different structural parts of a Condeep platform are shown in Fig. 5.1.



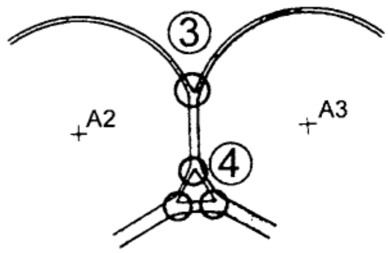


Fig. 5.1 Typical intersection regions (nodes) in a cell structure (Gullfaks-C)

1. Lower ring beam between the cell wall, lower dome and skirt
2. Upper ring beam between the cell wall, upper dome (and shaft)
3. Intersection between outer caisson cells
4. Intersection between joint cell wall and tri-cell walls

5.2 Design documents

5.2.1 Categories of design documents

The basis of design will be given in regulations, rules, standards, and specifications and will be a part of the contract for the design of the concrete structure. The specific basic documents to be applied will be decided by the client to a certain extent.

In addition to standards and specifications, the design may be based on existing practice. This will typically be the case when the existing specifications are supplemented by design methods worked out during the design of previous platforms approved by the client and possibly by a government agency connected with petroleum affairs.

In order to provide easy access to the necessary information for the personnel in the design teams and thereby enabling them to produce a consistent design, the most important information from the general documents is collected in a main document. This document is here named Design Basis. Where convenient, this document will give references to other basic documents.

In addition to the general design basis it will be practical to prepare documents for each activity in the design, which thoroughly discuss the application of the design basis and gather supplementary information. The activities within a large project may typically be: load analysis, dynamic analysis, static analysis, post processing, design of main structural parts, design of special parts for attachment of mechanical equipment, temporary structures for use in the construction phase, etc. These documents are named Design Briefs.

The Design Basis and Design Briefs are management documents for all design work to be performed within the project and its part activities. To obtain safe accomplishment of the project it is required that these documents be worked out and presented to the client for approval at an early stage.

5.2.2 Design basis

(a) Topics in Design Basis

The Design Basis will usually address the following topics:

- The client's most important functional requirements
- Reference to rules, regulations, standards and specifications
- Possible deviations from standards
- Design principles and limit states
- Temporary and permanent construction phases
- Loads, load combinations and load factors
- Material coefficients
- Materials and material parameters
- General reinforcement detailing
- Design assumptions and criteria
- Design procedures and methods
- Interface areas.

The above list only gives typical topics; it is not complete and should be supplemented as needed.

(b) Reference to rules, regulations, standards and specifications

The contents of the Design Basis document will be based on updated rules and specifications, but also refer to good practice developed by practical experience in the design of concrete platforms. The document also to some degree expresses the design philosophy of the project.

As an example, national rules and specifications used as main references for a Design Basis in Norway are found in (NPD, 1992), (NBR, 1990), (NBR, 1998) and (Statoil, 1992). Although (Statoil, 1992) is a company specification, it is often used as basis for the design, also by other clients. The client will then usually prepare his own supplementary specification which will be included in the design contract.

In addition to the above-mentioned national standards and specifications a varying number of recognised national and international specifications/standards, guidelines, research reports, and published articles will form the basis for the content of the Design Basis. Among the international standards for concrete, which are sometimes referred to, the following are mentioned: (CEB-FIP, 1993) and (CEN, 1991).

If, at some point, the rules put forward in the Design Basis deviate from the rules in the references, this should be clearly stated. All documents used in the preparation of the Design Basis should be listed in a reference list.

(c) The client's functional requirements

The client's functional requirements are usually specified in a separate document, where the conditions for management and use of the structure are described. In addition to a reference to this document it would be practical to quote for example:

- Maximum and minimum deck weight with centre of gravity
- Environmental loads
- Lifetime of the structure
- Water depths at the field
- The orientation of the structure.

(d) Design principles

The structures are usually designed according to the partial safety factor method. Under certain conditions the safety of the structure may also be assessed on the basis of probabilistic methods with specified safety indices. This approach is mainly used in connection with special accidental loadings.

In special cases a testing of structural parts may also be applied. It should be explicitly stated in the Design Basis document if such methods are to be applied. The structures are usually designed in the following limit states:

- Ultimate limit state ULS
- Serviceability limit state SLS
- Fatigue limit state FLS
- Accidental limit state (progressive collapse) PLS.

Appropriate criteria are given for each limit state. FLS and PLS are according to international terminology special cases of ultimate limit states, but since fatigue and accidental actions are particularly important for offshore structures, it is found convenient to use special identifiers for these ultimate limit states.

(e) Lifespan phases

The lifespan of the concrete structure from the start of the construction to removal of the structure from the field may be divided in several phases. Statoil (Statoil, 1992) distinguishes between temporary and permanent phases. It may be convenient to subdivide as follows:

- Construction phase (temporary) includes the building of the structure. This phase is often further subdivided.
- Transport phases (temporary) include all transport of parts of the structure or the complete structure until it is at the field, ready for installation.
- Installation phase (temporary) includes the time for installation of the structure in its correct position according to the specification of the client.
- Operational phase (permanent) includes the time from completed installation to removal of the structure.
- Removal (temporary) includes the task of removing the structure from the field.

Different sub-phases during construction will often be decisive for the design of traditional platform structures. These include floating out of dock or different floating phases during further construction and especially the almost complete submersion of the structure for deck-mating.

(f) Loads, load combinations and load factors

The loads acting on an offshore concrete structure have different characteristics. As an example, the Norwegian Petroleum Directorate (NPD, 1992) categorizes loads as permanent loads (P), live loads/variable functional loads (L), environmental loads (E), deformation loads (D), and accidental loads (A). Loads with categorization and detailed specifications concerning establishment of characteristic values will be provided by the client. As an example, see (Statoil, 1992).

Usually, permanent loads from self-weight and water pressure combined with environmental loads due to waves and wind will have a dominating influence on the design of the concrete structure. Determination of the characteristic environmental loads are based on observations at the site and the calculation of wind and sea states, according to (NPD, 1992) with 100 years return period. For serviceability limit state criteria or for temporary states shorter return periods are used, such as 1 year.

In order to determine the static equivalent design load on the basis of dynamic environmental loading, separate analyses are performed which take account of the stochastic variation of the loads and response of the structure. The designing wave load may therefore take different values for different parts of the structure.

Table 5.1 Example of load factors and combinations of loads (NPD, 1992)

Load Category	Limit state				
	Ultimate Comb. a	Ultimate Comb. b	Service	Fatigue	Accidental [4]
P	1.3 [1),2)]	1.0	1.0	1.0	1.0
L	1.3 [1)]	1.0	1.0	1.0	1.0
D	1.0 [3)]	1.0 [3)]	1.0	1.0	1.0
E	0.7	1.3	1.0	1.0	1.0/0.0
A					0.0/1.0

1. If the loads and load effects can be decided with high accuracy, the Norwegian Petroleum Directorate may allow the use of load factor 1.2.
2. The load factor for permanent loads is set to 1.0, if this is unfavourable.
3. For deformation loads from prestressing, national or regional standards may prescribe other values.
4. In the accidental limit state (PLS) it is to be checked that the damage due to accidental loading remains local. After local damage the structure should still be able to resist defined environmental conditions without extensive failure, free drifting, capsizing, sinking or extensive damage to the environment.

General requirements concerning loads, and especially which types of accidental loads are to be considered (ship collisions, falling objects, explosions, fire, earthquake, loss of internal pressure, erroneous trimming of the ballast, etc.) are to be given in the Design Basis. Furthermore, the extent to which analysis of the consequences of local damage is required, with possible flooding of parts of the cell structure, must also be specified.

Design load combinations and load factors should be based on analysis of the uncertainty of the load effects from combinations of typical loads acting on offshore structures, and will be specified in national or international regulations, or by the client. An example is found in Table 5.1.

(g) Material safety factors

Material safety factors will also be specified in national or international regulations. As an example, material factors used for offshore structures in Norway are given in Table 5.2.

Table 5.2 Example of material factors to be applied in design (NPD, 1992)

Limit state:	Ultimate	Service	Fatigue	Accidental
Concrete	1.25	1.00	1.00	1.00
Reinforcement	1.15	1.00	1.00	1.00

In (Statoil, 1992) material factors according to Table 5.2 are specified, but it is required that factors according to national standards shall be applied when they are higher.

(h) Material properties

High performance of the construction materials is generally required. For concrete in particular, low permeability is needed to satisfy the required durability and water-tightness of the structure. These requirements result in demands for low water/cement ratios and corresponding high concrete strength.

The importance of the self-weight of temporarily or permanently floating structures will often result in a demand for a high strength/density ratio of the concrete. Strength classes (according to NS 3473 (NBR, 1998) C65-C75 for normal density concrete and LC55-LC65 for lightweight aggregate concrete (where numbers refer to 28 days cube strength) are often used. The tendency during the development of offshore concrete structures has been increased strength/density ratios. The above strength classes are in principle covered by NS 3473, but it is recommended and partly required, that the mechanical properties are determined by testing.

Prior to application of new concrete types, the testing should include not only the usual mechanical properties, such as the ratio of the compressive strength of cubes and cylinders, E-modulus, creep coefficients, stress/strain diagram and tensile strength, but also fracture mechanics properties. To some extent, it is also recommended to test reinforced structural elements to verify the expected composite action of a new concrete type with reinforcement in terms of anchorage, shear strength, etc.

The strength classes according to NS 3473 are defined on the basis of such pre-testing with possible adjustments of the characteristic material properties to be used in design.

Similar specifications and documentation of the characteristic properties are required for reinforcement and prestressing steel.

(i) Reinforcement principles
The Design Basis document should outline the basic principles for the reinforcement system in order to ensure a unified detailing of the reinforcement in the whole structure. This will typically comprise the required minimum reinforcement, maximum spacing, standard bending diameters, methods and standard dimensions of splices and anchorages, limitations of maximum reinforcement density; use of bundled bars, etc. The prestressing system with standard cable dimensions will also be specified in the Design Basis.

(j) Design assumptions and criteria
The main parts of a concrete platform are classified in a high safety class taking into account that a failure situation may result in catastrophic consequences with high risk for loss of human lives. The extent of control measures is to be evaluated especially. Regarding control of the design and construction, reference is made to Chapters 6 and 7. Examples of relevant specifications are:

For the serviceability limit state:

- design exposure class (with corresponding concrete cover and crack widths)
- structural requirements to ensure strict water tightness
- criteria concerning vibrations and displacements, especially for shaft structures.

For the fatigue limit state: load distribution spectra and lifetime factors.

For dimension tolerances:

- thickness of each structural part
- deviations from the intended centre line of the components
- concrete cover
- position of the reinforcement.
- deviations from the ideal middle plane of the structure (as a basis for the design of slender shell structures).

(k) Design procedures and methods
Procedures and methods which are not uniquely described in the reference documents should be specified in the design basis. Examples may be:

- Load effects calculated by linear finite element analysis may be used as the main basis of design of the concrete structure
- Detail design of regular sections of the structure performed by automatic post-processing of the analysis results
- The method to be used in transverse shear capacity control
- Effect of water pressure in cracks
- Practicable simplifications and approximations for design for restraint forces due to imposed deformations
- Analysis and design for inward buckling (implosion)

- Additional design tasks which are not naturally included in the general design of the structure, like embedded steel structures, crane support structures, tube penetrations, temporary block-outs, etc.
- interface areas to other main parts of the concrete structure and to other design disciplines requiring procedures regarding management of interface information and co-ordination. Practical experience has shown that it is a considerable challenge to handle all the information necessary to satisfy all the different requirements in such interface areas, and that this is a source of possible design errors.

An example of the last item is that an offshore oil production platform will be equipped with a large number of tubes which are originally planned and designed by designers in other disciplines. The connection of such tube systems to the concrete structure will often require specific limitations of the load effects (deformations) in certain regions of the concrete structure. The technical solution to such problems should be worked out in close co-operation between process equipment and concrete structure designers to ensure that the combined structure performs as planned and the integrity of the concrete structure is taken care of.

5.2.3 Design briefs

(a) Needs for design briefs

In addition to the Design Basis document, a series of sub-documents in the form of design briefs for each main activity, such as load analysis, structural FEM analysis and detail design of each main part of the structure should be worked out as part of the Design Briefs. The basis for the Design Brief for a part of the concrete structure will be:

- relevant results obtained in the concept development phase
- drawings of the geometry of the part
- relevant parts of the Design Basis explaining how the loads are carried by the structure
- construction phases and limit states the structural part is to be designed for.

The Design Brief will be an extension and further detailing of the general topics in the Design Basis. The outline of the Design Briefs should be similar, but the topics to be discussed may vary depending on the type of activity or type of structure. The Design Brief will also include a description of how the design work is to be performed. It is often experienced that questions of a character where clarification with the client is necessary arise during the preparation of the Design Brief. The document will therefore also be helpful in clarification of all important questions in due time before starting the work-intensive production of design documents and drawings.

The following list of typical topics to be discussed in the design brief is not complete and should be supplemented in each case:

- Define documents and drawings to be produced within the activity
- Depict and describe the configuration of the structural part
- Describe interfaces to other parts and disciplines
- Indicate key data of load effects and geometry

- Describe the structural system
- Describe relevant construction phases and load combinations
- Indicate all construction phases and limit states to be checked
- Describe the load effects included in the global element analyses
- Describe analyses methods for load effects not included in the global analyses
- Describe design methods and design sections to be applied
- Reinforcement system and standard detailing, prestressing system and configuration
- Criticality and sensitivity assessment
- Indicate the relevant post-processing files
- Discuss the implementation of the quality assurance system for the particular task.

Documentation

A comprehensive list of documents and drawings necessary for the documentation of the design should be worked out. This list represents a survey of the work to be carried out and may be used to check that all design tasks are included in the work plan. It is also recommended that the document hierarchy is depicted. The purpose of this illustration is to show the relation between the design documents and the regulatory documents providing the basis for the design.

(b) Configuration and key data

Descriptions and drawings showing the configuration of the total structure and the actual structural part in particular, including the main construction phases and working joints, will provide the necessary comprehension and may serve as a reference for descriptions and evaluations performed during the detail design.

The key data for geometry and loads will typically comprise the concrete volume, anticipated amount of reinforcement and prestressing steel, buoyancy volumes, cell areas, resulting beam forces in shafts, global forces to be transferred to the sea bottom, etc. This information facilitates the designers' assessment of the sensitivity of the detail design, i.e. the importance of changes and adjustments of the loads or dimensions occurring during the detail design process.

(c) Interfaces to other structural parts and disciplines

Interfaces between different design tasks and how information should be handled are usually described in separate documents. Further discussion is included in the design brief if necessary. When the design of large structures is sub-divided and allocated to several separate design teams, the design criteria of one part of the structure must be fully compatible with the neighbouring parts, for example co-ordination of the use of prestressing in different main parts of the structure.

A table of important deadline dates for delivery of input data necessary for the design process to progress according to plan may be included. In addition, notice should be given in due time during the process.

(d) Structural system

The global structural system is described in brief. An explanation of how the loads will be carried by the actual part of the structure is emphasized. This information is important in order to provide the detail designers with a general view on the design task and help them in understanding the structural behaviour.

(e) Construction phases and load combinations

The design loads are established by scaling the unit load cases used in the analysis by multiplication with scaling factors for the characteristic value and load factors according to the actual limit state and load combination. Each load combination consists of a number of unit load cases which are scaled and added by linear superposition.

Equilibrium Load Cases consist of a set of active loads (self weight, self weight and functional loads on deck structure, weight of equipment, weight of ballasting, water pressure and environmental loads) and reaction loads (reactive soil pressure on ground based structures in operational phase or buoyancy forces and reaction forces in tethers in floating phases).

In the operational phase the structure will be designed for possible variation of the deck weight, variation of the effect of prestressing and the effect of waves and wind in various directions. Furthermore, the possibilities of different possible distributions of the soil reactions are usually accounted for. Assume that a design section in the structure is to be designed in Ultimate Limit State for:

- 12 different wave directions
- 2 deck weights (max and min)
- 2 soil pressure distributions
- 2 effects of prestressing (max and min)
- 2 load combinations (a and b in ultimate limit state).

The example results in 192 load cases which are to be checked. The number of load cases increases considerably in dynamic and fatigue response calculations.

As mentioned in Section 5.2.2, the lifetime of the structure consists of several phases. The total number of phases will mainly depend on the number of sub-phases during construction. The transportation, installation, operation and removal phases will exist for all typical offshore structures.

Due to the large number of load combinations and construction phases it is practical to assess which combinations/phases will certainly not be decisive in the design and can therefore be excluded. This evaluation is commonly based on simplified calculation methods of the same type as also used in the verification of the design (see Chapters 3 and 7). The results of these calculations are included in a separate load-phase document.

An abstract of this document explaining for which construction phases design calculations are to be made and which load combinations and limit states are to be checked in each phase shall be included in the Design Brief. The principles for establishment of the design load combinations should also be included. However, the detailed combination of unit load cases used in the analysis will be described in separate referenced documents.

All special load combinations which are not included in the global analysis must be described and explained in detail, to avoid time-consuming discussions at a later stage. Typical examples are combinations with local loads such as ship collisions and impact from falling objects, local implosion or other non-linear effects.

(f) Analysis and design methods

The general design methods are described at a suitable detail level in the Design Brief and/or with reference to the Design Basis. A complete list of all relevant analysis result files to be used in the post-processing design of the actual part of the structure should be worked out. Identification of the person responsible for the verification of the analysis and reference to the verification report should also be stated here.

The element mesh used for the actual part of the structure is described and depicted with clear indication of the areas of the structure where the results of the analysis are assumed not to be sufficiently accurate and modification is needed. All elements where the analysis results will not be used as a basis for the post-processing should be identified in the Design Basis.

Reference is made to other documents where the general use of linear element analysis and the automatic post-processor based design are described in detail. In the Design Brief, all design sections to be used by the post-processor for calculation of the section forces based on integration of the analysis output stresses, are depicted. The selection of design sections among all accessible integration points in the analysis should be explained and justified.

It is often necessary to select a supplementary design section which does not correspond to the integration points in the analysis. The method of establishing the section forces in such cases should be explained, especially when the forces in sections at the border of structural intersections are determined by extrapolation from the forces in neighbouring design sections.

Supplementary analysis and design programs and manual design methods to be applied in the design of areas with concentrated loads or complex geometry, where direct post-processing of the global analysis results are not applicable, should be described in detail. Typical examples are the design of intersection areas and analysis/design for implosion and impact from ship collision and falling objects. Furthermore, relevant design methods to be applied within the category Miscellaneous Design (see Section 5.2.2.11) should be described.

It is recommended that all relevant supplementary design methods including assumptions, limitations and how the design results are to be co-ordinated with the results of the general post-processing, are clarified and approved by the client's revision of the Design Brief at an early stage.

(g) Reinforcement system and detailing

General principles for detailing of the reinforcement are described in the Design Basis. These principles are adapted to the main configuration of the actual structural members and further specified in the Design Brief. The reinforcement system is to be fully defined here. The reinforcement system includes definition of the number of reinforcement layers with directions relative to the defined principal directions of the structural part and position relative to the concrete surface. Furthermore, standard diameters for the main and shear reinforcement and the maximum number of bars in reinforcement bundles are to be stated.

The reinforcement system is input data to the post-processor and defines the standard amounts to be used in the design iterations performed by the processor to obtain the necessary practical reinforcement. The reinforcement system will also provide the basis for the generation of general reinforcement drawings showing the selected standard amounts and the overlap splicing system for the reinforcement in slip-formed structures.

Typical reinforcement configuration drawings of typical intersection areas should also be prepared to ensure efficiency and constructability in areas with complex geometry.

It is also practical to define standard anchorage and lap splicing lengths dependent on the reinforcement dimensions and other influencing parameters according to the concrete design rules. The transverse reinforcement in the form of ties, stirrups and T-headed bars are important parts of the detailing. Especially when lap-spliced U-shaped stirrups or T-headed bars are applied, the criteria and conditions for use should be specified.

(h) Prestressing system

The prestressing system is described in the Design Basis. In the Design Brief further detailing including the distribution and staggering of the cables and position and typical detailing of anchorages are described. The effect of unit loads from prestressing is included in the FE analysis. The applied method for the scaling of the prestressing force to the correct initial prestressing force and further scaling to take into account the time dependent loss of prestress is described. It will often be necessary to combine the effects of prestressing in several construction stages.

(i) Criticality assessment

The aim of the criticality assessment is to point out areas of the structure which may be especially critical with respect to the constructability, functionality and safety and hence, need extra resources in design, construction and verification to ensure the reliability of the structure. Typical examples are intersection areas, construction joints, areas with dense reinforcement, shaft top with deck support structure, etc.

(j) Sensitivity assessment

In order to establish a sound basis for the final design it is necessary to assess the sensitivity of the structure with respect to possible changes in the anticipated loads or load effects, e.g.:

- What is the consequence of increased wave heights?
- Will cracking of the concrete with locally reduced stiffness result in significant changes in the dynamic response of redistribution of forces?
- How much will possible deviations from the calculated membrane forces influence the shear strength of the structural members?

The sensitivity assessment should result in consistent modifications of the design to achieve the necessary robustness of the structure to resist such possible deviations from the original design assumptions. The necessary extra safety margins to be included in the design should be agreed upon with the client if not already included in the contract.

(k) General processing files, quality assurance

The organization of files in the postprocessor is depicted, with a list of the general processing files, input and result data files. This listing provides a general survey of the files and facilitates the documentation of the design. The general quality assurance system of the project will be described in separate documents; see also Chapter 6. It is, however, recommended to include a description of how the quality assurance system is to be implemented in the actual design activity. The quality assurance of the input data files to the postprocessor should be mentioned, particularly the implementation of changes of the input files, the persons authorized to make such changes and the procedures established to assure that the correct input files with the latest changes are implemented in the design. An updated revision list with logging of all file changes must be available. The revision log must include the dates of implementation, the cause of the changes and the person responsible. The revision list of each file may conveniently be included in the heading.

5.3 Design procedures

5.3.1 Design process

In Sections 5.4 and 5.5 the design activities, i.e. the design and detailing of the main concrete structural parts are discussed. For further discussion of the analysis, reference is made to Chapters 3 and 4.

All personnel performing technical tasks must know the content of the Design Basis and understand the application of this document in their own work. Furthermore, the personnel must be familiar with the content and application of the Design Brief for the actual part of structure where they are involved in the design. Fig. 5.2 shows a simplified flow chart of the design process based on the use of a postprocessor.

The design of concrete sections is based on integrated load effects from the finite element analysis; see also Chapter 3. Due to the large number of load cases and sections to be checked, a high degree of automation of the design process is necessary. As far as possible a postprocessor is used as a design tool.

It is important to be aware of the inherent limitations of the validity of a section-by-section design based on forces from linear elastic analysis. The linear analysis cannot simulate all important load effects with satisfactory accuracy. Local analysis and manual design must therefore supplement the analysis. The results of the supplementary design are added to or substitute the results from the postprocessor.

An automatic design process represents a demanding challenge to the designer regarding a thorough assessment of the results. Independent simplified calculations must be done to verify that the calculated amounts of reinforcement and utilization of the concrete sections are reasonable. See also Chapter 3 (Simplified analysis) and Chapter 6 (Verification).

5.3.2 Postprocessor

This section gives a brief description of the postprocessor and the limitations in the use of such a program. The function of the postprocessor is not, however, explained in detail. A more extensive description of the design of offshore concrete structures based on processing of results from finite element analysis is given in (Brekke, Åldstedt and Grosch, 1994). Further details may be found in the user's manuals for the respective postprocessing programs.

The detail design is performed in a set of representative sections throughout the structure. The structure may conveniently be divided in horizontal sections (H.S) and vertical radial sections at constant angles (f-sections or F.S). A simple example showing the design sections in a cylindrical structure is shown in Fig. 5.3. The design points are determined by the intersections of the horizontal and vertical sections.

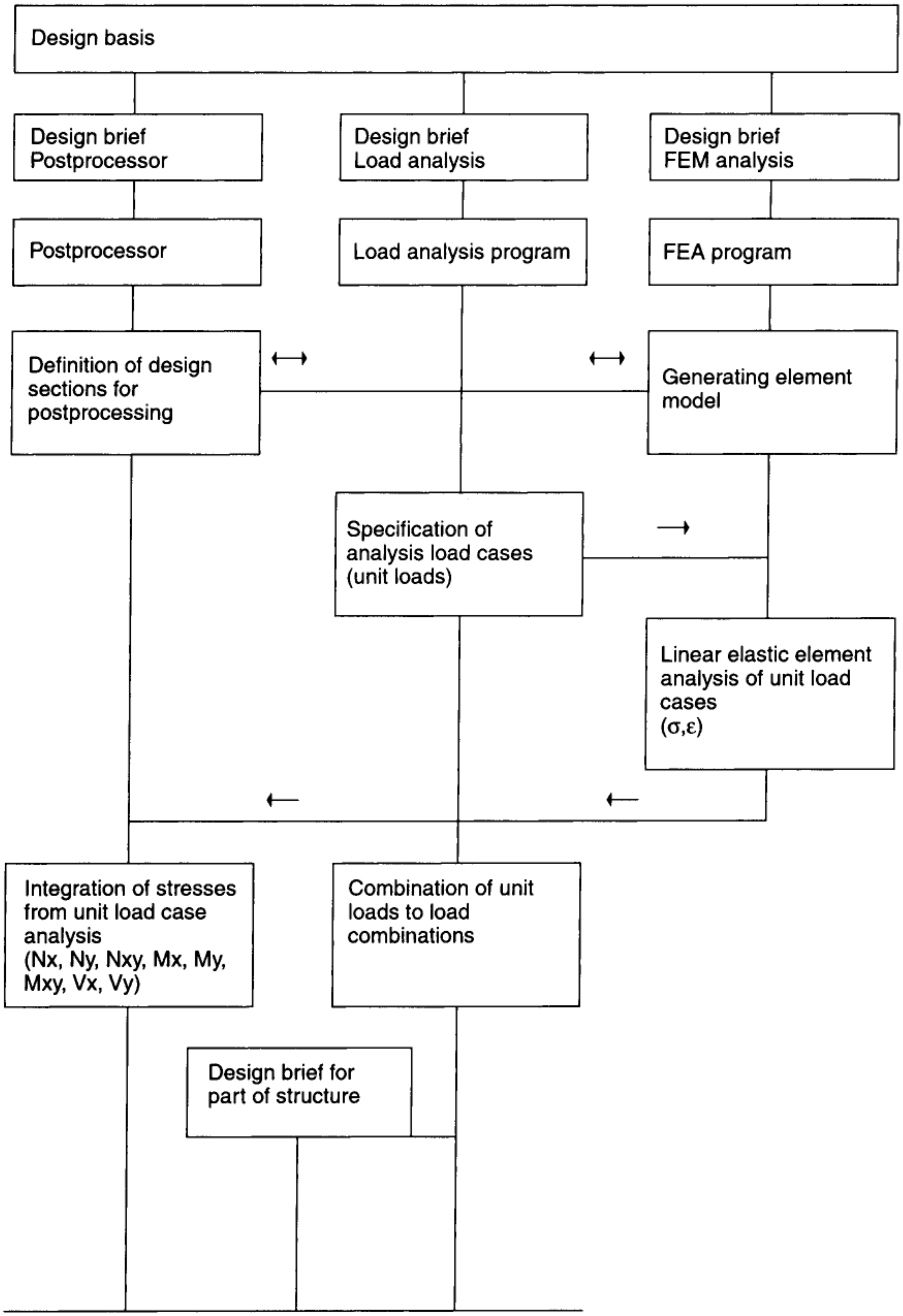


Fig. 5.2 a Simplified flow chart for the design process. Complete structure. Iterations not included

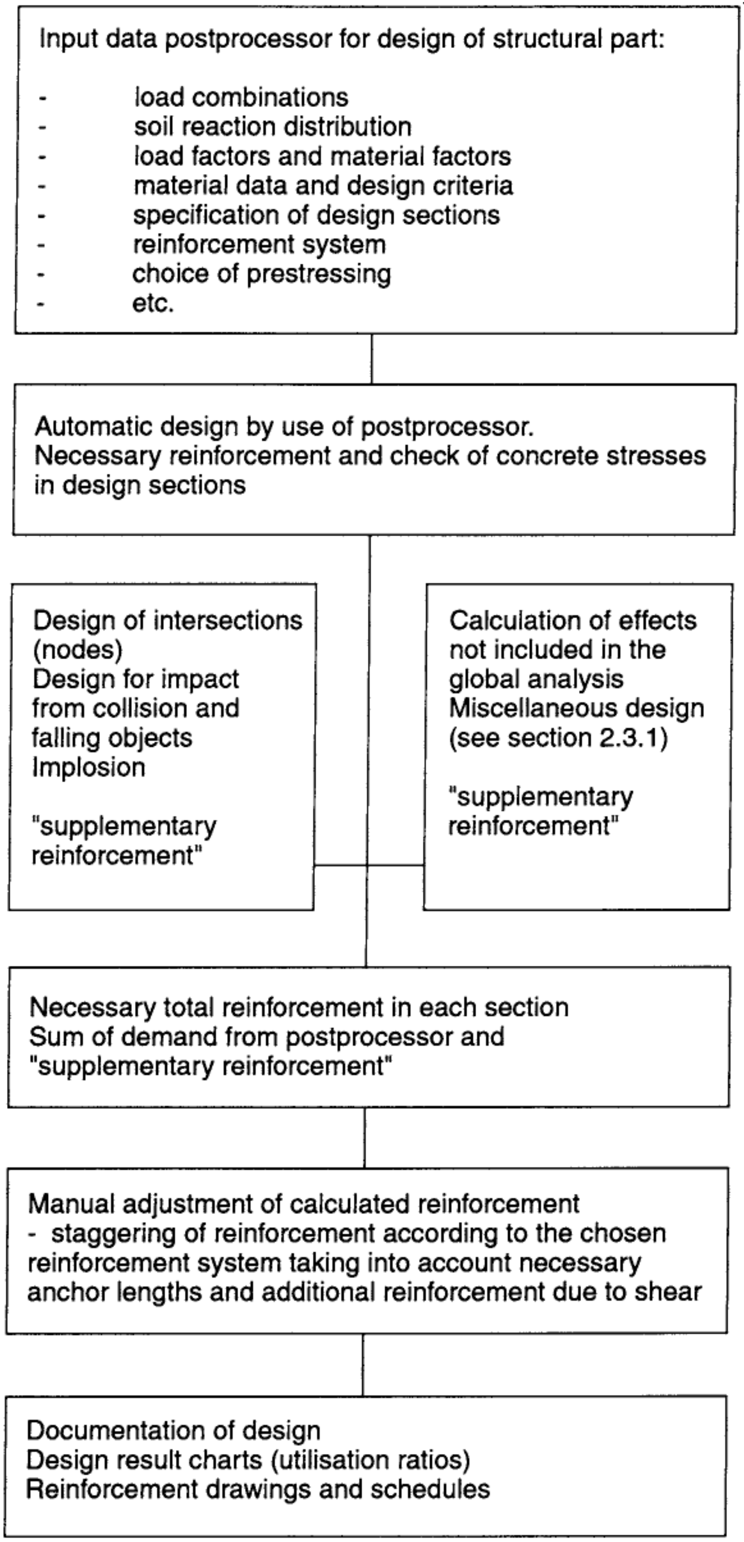


Fig. 5.2 b Simplified flow chart for the design process for part of structure. Iterations not included

The number and position of the integration points (Gauss-points), where the stresses may be integrated by the postprocessor, is determined by the choice of the finite element model and the type of elements. The choice of the element mesh for the analysis also decides the possible locations of the design points. Good communication between the analysis team and the team responsible for the postprocessing is required to ensure that the practical use of the analysis results in the detail design is considered in the element mesh development.

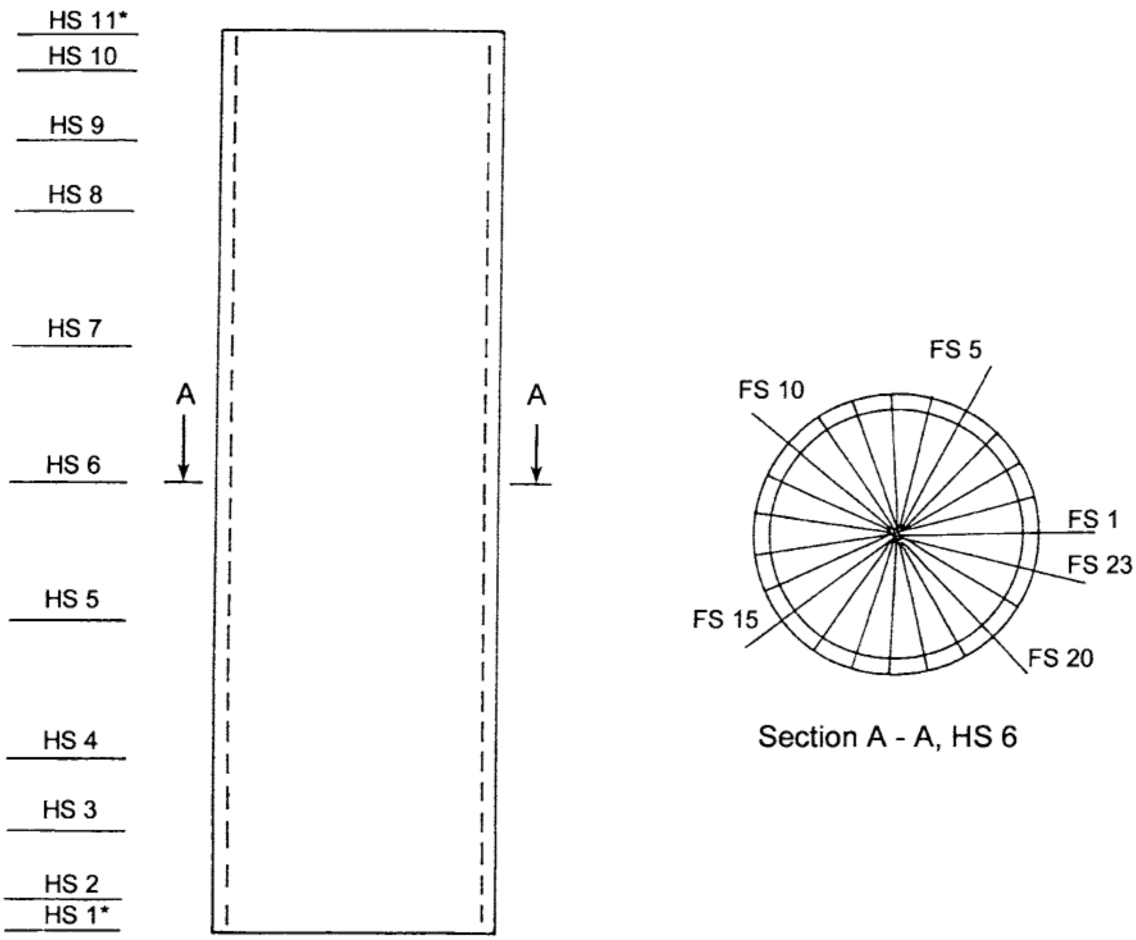


Fig. 5.3 Division of a cylindrical shell in design sections.
* The section forces in the border sections are determined by extrapolation

The general requirements concerning the element modelling are discussed in Chapter 3.

The element size should be reduced towards the borders with high stress gradients to achieve sufficient accuracy. Rectangular elements should be used as close to the borders of the structures as possible. The use of skew elements with reduced accuracy and complex interpretation of the section forces is often unavoidable in intersection regions between the regular structural main parts (see Fig. 5.1).

Forces at the borders of the intersection regions are often needed in practical design. The simplest way is to define a design section at the border. Usually these sections do not correspond to the integration points of the elements. The section forces are therefore often extrapolated from the results at integration points further into the regular part of the structure. Extrapolation methods are discussed in Chapter 3.

Acceptably small analysis errors in regular design points are easily unacceptably enhanced by extrapolation. The accuracy must be closely validated. All regions of the structure where the analysis is expected to give an inaccurate result should be documented in a separate report. This report should be used actively in the design process to ensure that supplementary analysis of these regions is carried out.

In regions with varying shell thickness it is especially important to choose the position and direction of the design section in order to obtain representative section forces. Sections normal to the middle plane of the elements are usually preferable (see Chapter 3).

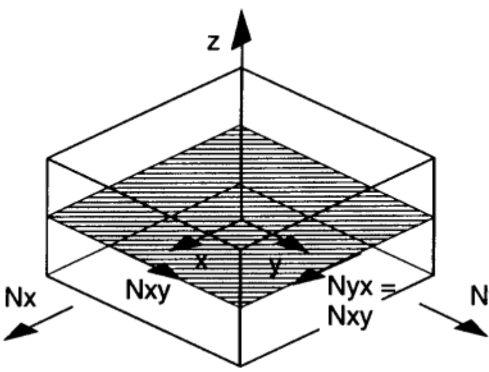


Fig. 5.4 Membrane forces (N_x, N_y, N_{xy}).

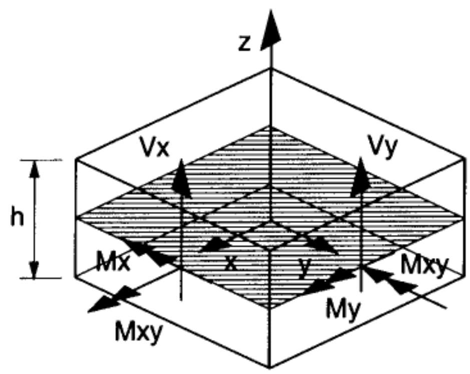


Fig. 5.5 Slab section forces (M_x, M_y, M_{xy}, V_x, V_y).

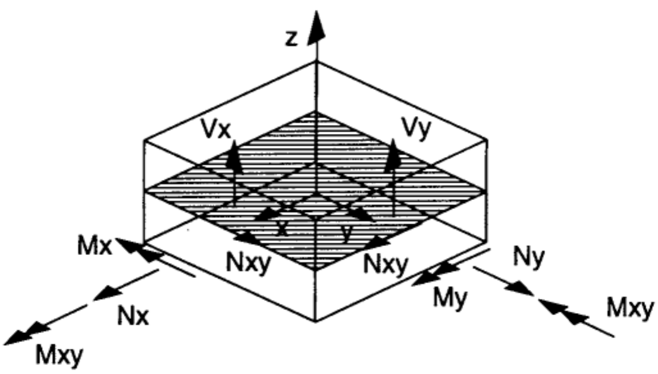


Fig. 5.6 Shell section forces (N_x, N_y, N_{xy}, M_x, M_y, M_{xy}, V_x, V_y).

The design is based on the shell section forces at each design point. The shell section forces can be regarded as a combination of in-plane membrane forces and slab section forces (bending moments, torsional moment and out-of-plane shear forces).

This is illustrated in Figs. 5.4–5.6 by application of the forces on two faces of a finite element.

In reality it is the forces acting in different directions in defined single points (represented by a line through the thickness) which are the basis of the design.

The postprocessor divides the sections in a series of concrete layers (lamellas) and defines the direction and position of each reinforcement layer. The design for membrane forces and moments is based on strain compatibility with the assumption that plane sections remain plane. The non-linear material properties of concrete and reinforcement are taken into account. The processor calculates the response of the section-to-section forces by an incremental iterative solution procedure. If the design criteria are not met, the processor automatically increases the reinforcement in predefined steps and finally checks that the design strengths of the concrete lamellas are not exceeded.

The out-of-plane shear forces are not included in the response calculation. The shear capacity is checked analytically in accordance with the methods in relevant concrete design codes or standards, usually by a simplified method.

The water pressure is regarded as a load at the surface of the structure in the global analysis. The effect of water pressure penetrating into cracks is therefore calculated by a special routine in the postprocessor (see Section 5.4.3).

The iterative design based on response calculation followed by an incremental increase of the reinforcement and recalculation in several steps is a time-consuming process. The calculation time can be reduced by the choice of reasonable starting conditions in terms of reinforcement amounts and initial strain.

A practical method to establish reasonable reinforcement amounts at an early stage is to start the processing with a few selected load combinations which are assumed to be decisive. Based on the result of the concept study phase it is often possible to point out the load combinations and load cases which will decide the necessary reinforcement in the major parts of the structure. The required minimum reinforcement in the respective structural members will be a convenient initial choice for this pre-processing. The preliminary results will still be at hand within a reasonably short time due to the few load cases. Experience shows that this preprocessing will provide a good basis for the selection of the initial reinforcement amounts in the final processing of the vast number of load cases in the final design.

The necessary reinforcement in each single design point will vary according to the variation of the section forces. For practical reasons it is necessary to simplify the reinforcement. The possibility of defining regions of the structure with constant reinforcement amount as input data is therefore a necessary feature of the post-processor program. It is often practical to define different constant reinforcement regions for each principal reinforcement direction and layer. Separate procedures are developed for the definition of the staggering of the shear reinforcement.

Due to the large amount of input data to be handled in the design of a complex structure, a strict accountancy of the use of the postprocessor is necessary. The limited number of persons authorized to make changes in the input data should be listed in the Design Brief. An updated revision list with logging of all file changes must be available. The revision log must include the dates of implementation, the cause of the changes and the person responsible. The revision list of each file may conveniently be included in the heading. It is also essential that all result files clearly state all input data. The key word is traceability.

The postprocessor manuals should provide the designer with easy access to all necessary information for an optimal utilization of the design tool. The manuals must include the basic design formula and criteria and clear statements regarding the limitations of the validity of the program. The manuals will conveniently comprise a general part followed by design specifications with optional design criteria adaptable to different kinds of structures.

During the design process it may be necessary to change the design parameters, e.g. the concrete strength class. It is therefore important that the processor is prepared for a flexible implementation of such changes. The designer will sometimes also change the design parameters in order to assess the robustness/sensitivity or criticality of the design.

The postprocessor should be prepared for use together with the most commonly used finite element programs. It will be advantageous if it also is easily adapted to other FE programs. A flexible postprocessor in this respect will provide a freedom of choice of the analysis program that is best suited for the present task.

The postprocessor will utilize a lot of information concerning the concrete structure already existing in the finite element program employed in the analysis of the structure. The postprocessor should be able to retrieve all relevant information from the analysis result files without time-consuming manual data input. The print and plot facilities of the postprocessor program should provide simple selection of optional result presentations in the form of result tables and/or graphical presentations.

In addition to automatically generated result tables containing the main results only, it should be possible to easily generate more detailed information tables. Such tables may concern specified sections containing, e.g. section geometry data, load combination, sectional forces, strain result vector, stresses in concrete, normal and shear reinforcement and summarized design results in terms of Utilization Ratios (UR).

The design load combination will generally consist of several basic load cases. Easy access to the listing of all basic load cases with scale factors and the corresponding contribution to the section forces are necessary processor facilities to enable the designer to perform simple checking of the section forces. This is very important in order to achieve correct and reasonable results.

Plotting of the distribution of forces and reinforcement amounts with corresponding capacity along the structure should be emphasized. Graphical presentations will often be the most effective means for assessment of the design results.

The processor program should have a low user threshold with easy access to the input and output files. In large projects new personnel will frequently join the design team without experience in the use of the design tool. It is therefore important that inexperienced personnel can quickly grasp the details of the standard use of the program.

The postprocessor should be flexible as regards the adaptation to various types of hardware. Preferably it will be adaptable to large computers, workstations and Personal Computers. It ought to be the extent of the design task, not the structure of the postprocessor, which decides the necessary hardware.

It is, however, important that sufficient computer capacity is available. The time consumption running the processor should not have too much influence on the progress of the design work. It should at least be possible to start running a fairly large design task, e.g. a spherical shell, at the end of a workday and receive the result file ready for assessment and further adaptation the next morning.

Periodically the design tool will be used intensively. Often several large design jobs will be performed during the same period within strict time limits. It is important to establish efficient support functions by computer experts. Possible errors or defects occurring during the intensive design periods must be corrected without delay. The support team should also be readily available to answer questions concerning the advanced use of the postprocessor beyond what is naturally included in the manuals.

Especially in the start-up phase of the detail design process it will be necessary to perform considerable manual operations, e.g. to get the processor operative, and there will be a relatively high risk that human errors will occur. This again emphasizes the importance of a responsible and critical use of the automatic design tool. The results should be critically assessed and not be regarded as an authorized answer to correct design.

All software to be deployed in detailed design of marine concrete structures is to be verified for this application (NPD, 1992).

The ISO standard (ISO13819 Part 3) will, when available, for design refer to NS 3473 as a standard that covers relevant conditions (see Chapter 1, Section 1.6.2). It will still be advantageous for further international application if the postprocessor also has the possibility of performing the design checking according to other standards, e.g. the British Standard (BS), Canadian Standard (Can Standard) or the American ACI codes, and Eurocode 2. Design according to other codes is already implemented in some of the existing postprocessors. In any case, the postprocessor program should be prepared for easy implementation of optional design codes.

5.3.3 Effect of water pressure in cracked concrete

(a) General

Marine structures may be exposed to very high water pressure. For example, the water pressure at 250 m depth corresponds to a load of 250 tonnes per square metre, or a surface stress of 2.5 MPa. The design tensile strength of C65 concrete according to NS 3473 also equals 2.5 MPa. This implies that existing structures are exposed to water pressure of the same order of magnitude as the tensile strength of the concrete.

In a concrete body with unsealed surfaces exposed to high water pressure the internal pore pressure will gradually increase. Ordinary concrete contains pores in a wide range of relative sizes with different fluid surface stresses. It is therefore difficult to measure directly a representative “effective” pore pressure in the material. The notion of the effective pore pressure is therefore based on the interpretation of the observed influence on strain and strength of concrete exposed to high water pressure.

Measured strain and strength of 100 mm concrete cubes with or without sealing of the surface exposed to high water pressure (800 m water head) show that the build-up of pore pressure inside the concrete is strongly material- and time-dependent (Bjerkeli, 1990). The results indicate that a near 100% effective pore pressure can develop in small specimens of ordinary porous concrete (C25—C35) in a few days. For high strength concrete (strength class higher than C65) the pore pressure development will take months, and the maximum effective pore pressure will finally be about 60–90%. For high-strength lightweight aggregate concrete the eventual pore pressure development may take a very long time, probably due to the combination of low permeability of the high strength mortar and large pore volumes in the lightweight aggregate.

For thick structures with the actual concrete types used in marine structures the development of pore pressure will generally take a long time. In structures exposed to water pressure on one side, the pore pressure will probably stabilize with a gradually decreasing pore pressure from the exposed outside towards the inside, but the pore pressure gradient is not necessarily linear.

If the structure has visible cracks with open communication with the exposed surface, it is naturally assumed that the water will penetrate into the crack with a constant water pressure equal to the external pressure action on the crack surfaces. The effect of the water pressure will be a local wedge effect which will affect the state of equilibrium of the cracked section of the structure.

Obviously, the effect of high water pressure in cracks must be taken into account, due to tensile stresses induced by direct loading or restraining of imposed deformations. It is a matter of discussion to which extent it is also necessary to take into account the effect of possible water pressure in arbitrary internal cracks without direct communication with the exposed surface, due to local stresses in the interior of the concrete sections. A design philosophy based on the assumption that cracks with full water pressure can occur in arbitrary locations and directions anywhere in the concrete members will consequently lead to extreme situations where, for example, the spalling off of the concrete cover is unavoidable.

(b) Calculation of section forces

In a generally formulated clause in NS 3473 it is suggested that the water pressure in the structure should be taken into account. In clause 9.4.7 it is stated:

> Potential fluid or gas pressure in pores or cracks shall be accounted for in structures exposed to direct pressure from fluid or gas in the calculation of forces and moments if this increases the action effects or reduces the capacity.

For thick shell structures it will generally be significant for the calculation of the section forces in the structure, whether the water pressure load is assumed to act on a presumed watertight surface, or the pressure is assumed to act deeper in the structure, e.g. at the middle plane of the shell.

In regular cases with external water pressure the penetration of water into the concrete will lead to reduced forces on the structure. For example, the horizontal force in the hoop direction of a cylindrical platform shaft with external water pressure will decrease approximately proportionally to the ratio between the middle and the outer diameter of the shaft. Also the moments and the shear forces at the shell borders will usually decrease (as opposed to a pressure vessel with internal pressure where the section forces tend to increase).

It is therefore conservative to assume that the water pressure acts on the outer surface. Furthermore, when the structure is modelled with volume elements it will be more complicated to depart from the usual finite element method routine for application of distributed load on the element surfaces.

In most cases it is therefore accepted that the water pressure is applied on the surface in the structural analysis. It is, however, important to be aware of this assumption and take into account the effects of water pressure in cracks and pores by the capacity check of each single section and local areas of the structure.

(c) Design for axial forces and moments by water pressure loads

In ultimate limit state design it is usually assumed that concrete cannot transfer tensile resultants. By calculation of the resistance to actual forces and moments it is assumed that the tensile strength of the concrete is zero and that cracks will occur in the tensile zones of the structure. The capacity check of a section is at first calculated without water pressure. If the tensile face is exposed to water pressure the wedging effect of water pressure in the crack is taken into account as an external tensile force calculated as the resultant of a constant pressure on the crack surface, and a new state of equilibrium is calculated. The tensile force resultant contributes to the moment as well as the axial force in the section.

In the general case, where the sections are checked by a postprocessor, the section is subdivided in several layers. The tensile forces from the water pressure are applied in each layer with direction normal to a calculated crack direction, i.e. in the principal tensile strain direction. In general, this direction will vary from layer to layer. For practical reasons the water pressure is applied in all layers where the minimum compressive stress is less than the water pressure. The consequence of this is that elements that are not subjected to significant biaxial moments with equal signs, or biaxial membrane compression, may be subjected to tensile stresses from the water pressure in all layers but with varying direction.

Application of water pressure load as tension in the principal strain direction of the layers will generally increase the necessary reinforcement and appears to be a safe method for calculation of the effects of water pressure on the axial force and moments in the ultimate limit state. Naturally the importance of the water pressure varies with the force of the water pressure. The relative influence on the reinforcement demand is more significant in areas of the structure with small reinforcement amounts near the required minimum reinforcement.

(d) Design for shear force due to water pressure load

Design for shear force in slabs and shells is usually based on the "simplified method" according to NS 3473. The method includes a considerable "concrete contribution", which increases significantly in structures with axial compression force, and a contribution from shear reinforcement based on the standard truss model with 45 degree compression diagonals. It is usually accepted that shear reinforcement (stirrups) can be omitted in areas where the concrete contribution to the shear strength is sufficient.

Calculation of necessary stirrup reinforcement in loaded areas near the supports and design rules for necessary shear reinforcement are based on the usual cases where the load is applied by compression against the top side of the structure with respect to the direction of the load and the support forces are transferred by compression against the under side. If the loads or support reactions are imposed on the structure in such a way that internal tensile forces occur in the direction of the load, it is required that these forces are carried by the reinforcement.

When the loads are caused by compression from fluid (or gas), the pressure may penetrate into shear cracks and influence the transfer of loads to the structure in a way which is comparable to suspended (hanging) loads. The transfer of water pressure load will, however, be distributed through the thickness of the structure and thereby not be quite as unfavourable as suspended loads. On the other hand, it may be argued that the concrete contribution to the shear strength may be further reduced if water pressure of the same order of magnitude as the tensile strength of the concrete is allowed to penetrate into the structure. This particularly concerns the upper limit of the concret e contribution in the design formula in NS3473 which is clearly related to the criteria that the principal tensile stress at the centre of the section equals the tensile strength of the concrete. Activation of shear reinforcement presupposes that inclined shear cracks exists. Such cracks, however, do not have direct communication with the water-exposed surface.

Fig. 5.7 shows the required shear capacity related to the shear force diagram of a member with constant distributed load. The stapled capacity diagram for members with suspended load well above the shear force diagram is recommended also for structures with water pressure loading. No reduction is made for loads near the support. This means that the maximum shear force at the edge of the support should be accounted for.

As mentioned in Section 5.4.3, the necessary tensile reinforcement is increased due to the effect of water pressure normal to the bending cracks. If the main tensile reinforcement is staggered, the effect of inclined shear cracks is considered by extending the reinforcement a length equal to the internal lever arm (z) with an additional (full) anchorage length beyond the section where the actual part of the reinforcement may theoretically be terminated.

According to the simplified method in NS3473 a compressive axial force will increase the shear capacity considerably. It is recommended that the axial force calculated by the global analysis with water pressure at the external surface is conservatively reduced by a force resultant corresponding to full water pressure in the section area.

The upper limit of the shear capacity formula will often be decisive for structural members with high compressive axial force. When high compressive stresses exist in the principal shear direction of a slab or shell or, maybe more often, normal to this direction, the tensile strength in the third principal direction will decrease. A full utilization of the shear strength according to the simplified method for members without shear reinforcement may in such cases turn out to be non-conservative.

Application of the alternative truss model method for shear design will result in a general need for shear reinforcement in all members subjected to transverse shear. This seems unnecessaryly conservative for slabs and shells in general.

Methods based on the modified compression field theory (MCFT) (Collins and Michell, 1991) represent a further option which is also introduced in the so-called general method in NS3473. This method is primarily intended for members with in-plane shear, but is also applicable for beams and slabs with shear reinforcement.

Based on the notion of the mean tensile strength of cracked (reinforced) concrete and the ability of shear transfer in cracks, the method also features a "concrete contribution" to the shear strength, but the method is apparently not sufficiently well verified to be applied in the shear design of members without shear reinforcement. The need for minimum shear reinforcement in platform structures is further discussed in Section 5.4.3.

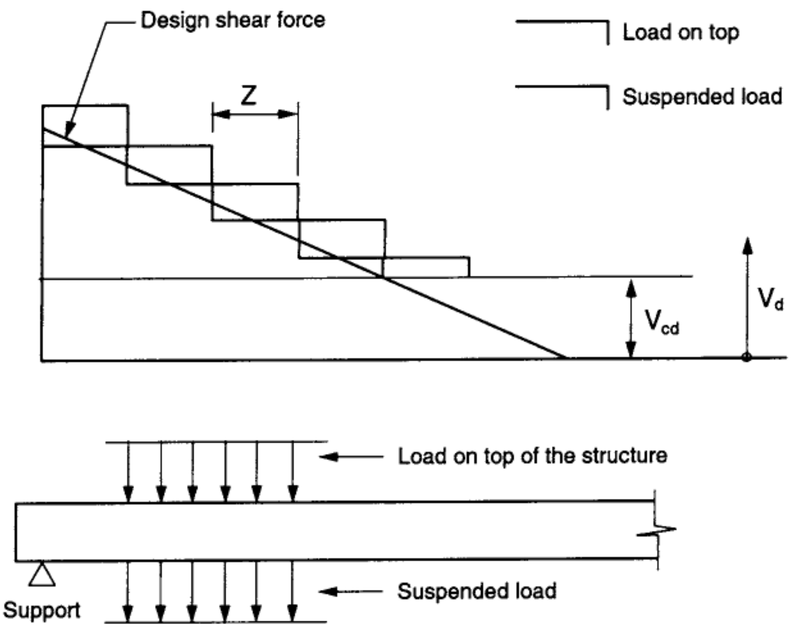


Fig. 5.7 Curtailment of shear reinforcement by distributed load. Simplified method

(e) Water pressure in the nodes of the structure

The possibility that transverse loads from external water pressure will be transferred to the structure in shear cracks is indirectly taken into account in the design of regular members. The transverse component of the pressure on the inclined crack surfaces will substitute the transverse load at the outer surface in the same area.

However, when water pressure penetrates into cracks in areas with discontinuous geometry, such as the sharp inner corner of the outer intersections between two main cells in the caisson, or the internal corner of a tri-cell between the main cells in a Condeep structure, an extra transverse load in the form of a wedge-effect will occur. This load will add to the load on the exposed surface taken into account in the analysis. This effect is illustrated in Fig. 5.8.

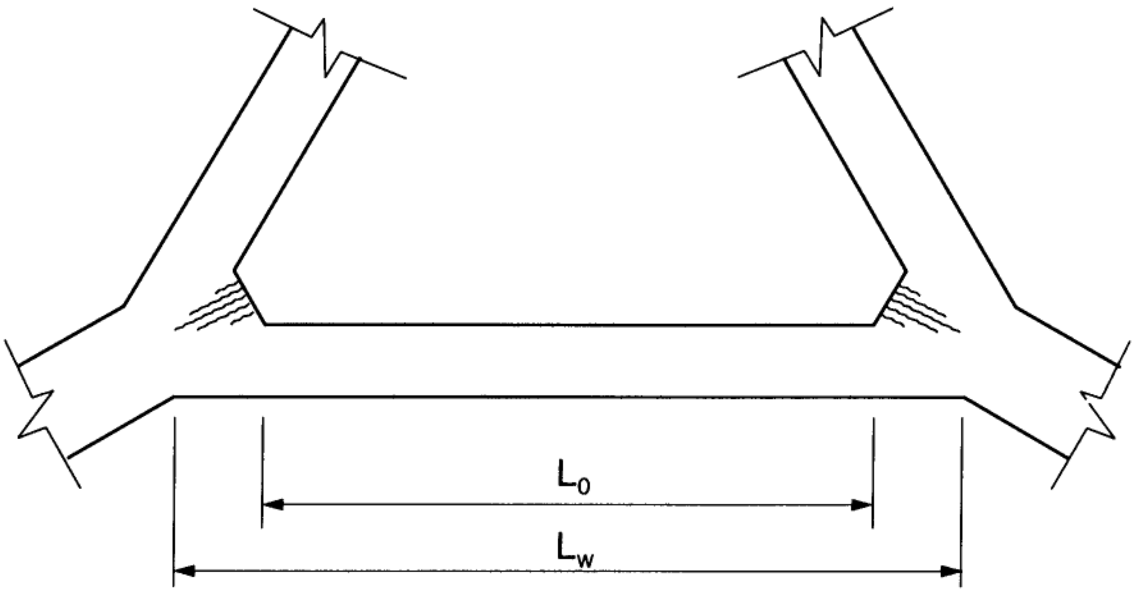


Fig. 5.8 Increase of the loaded area due to water pressure in cracks

The design of such intersection areas is often based on simplified strut and tie models. The resultant of water pressure in cracks will often be added directly to the needed capacity of the transverse tension ties. It is therefore especially important that the wedge action of the water pressure is taken into account in such discontinuity regions. Realizing that the forces in the corner areas are highly statically indeterminate and depend on both the local and global stiffness and load distribution, it is necessary to estimate the possible depth of cracking with water penetration on the basis of a conservative estimate of the tensile strains.

(f) General de-lamination

For structural members with little or no reinforcement in the interior of the sections, the question arises whether high water pressure in assumed small cracks might lead to unstable crack propagation in the structure. High water pressure in cracks parallel to the surface of a shell structure may, as the ultimate consequence, lead to separation of the structure in independent layers. A frequent calculation model considers a shell as consisting of several interacting layers. The possible loss of the composite action of the layers due to water pressure in cracks is therefore often called de-lamination.

In a cylindrical shell, e.g. a platform shaft, with an external water pressure load, radial compression stresses increase from zero at the internal surface to a value equal to the water pressure at the outer exposed surface. Even if a 100% effective pore pressure with a corresponding pressure gradient from the outside to the inside exists in the concrete, the effect will be limited to a full compensation of the former radial compressive stresses. The build-up of a corresponding water pressure in arbitrary internal cracks will, in this case, not lead to any tendency for crack propagation.

The risk of de-lamination is therefore linked to the possible existence of relatively open communication paths for the water to enter into the concrete structure where high water pressure can occur with sufficient water supply to enhance the widening and propagation of existing cracks. Permeable zones due to imperfect working joints, inhomogeneity of the concrete, zones with congested reinforcement or embedded steel parts or more regular cracking due to load-effects, may facilitate water transport.

Also imperfect bonding between ducts for prestressing steel and the surrounding concrete may provide continuous passageways for water. This is enhanced in cases where tensile cracking along the duct occurs due to high compression stresses normal to the duct before mortar injection, or due to excessive injection pressure.

If cracks with water overpressure occur in unreinforced zones, the question of possible crack propagation is a fracture mechanics problem. This has been modelled and analysed by non-linear FE analysis, but the fracture criteria when water pressure is a driving force in the crack, is not well known. In its extreme consequence, de-lamination may cause failure of the structure. Unfortunate build-up of high water pressure in cracks parallel to the inner surface at the level of the innermost layer of reinforcement may easily lead to local failure with loss of the concrete cover. This type of failure cannot be prevented by reinforcement. The failure consequences are, however, limited.

It will be much more serious if water pressure in cracks near the middle plane of the shell would cause complete delamination with transfer of the full water pressure load to the inner half of the shell.

Thick shell structures exposed to water pressure of a magnitude comparable to the effective tensile strength of the concrete in the actual three-dimensional stress state should be equipped with transverse (shear) reinforcement. This is especially needed where it is possible to point out probable water passageways for water such as cracks, working joints, embedded steel, or prestressing ducts.

5.3.4 Discontinuity areas

(a) Design methods

Standard methods and rules for the design of regular sections of slabs, walls and shells, which are included in the design routines in design programs (postprocessors) are not sufficient in regions of the structure with discontinuity of geometry or loading. The design calculations may then be based on simplified force models that satisfy the equilibrium conditions. The models might be simple strut-and-tie models or more complex truss models or stress fields adapted to the actual geometry of the region in accordance with the standard or code used. The description below refers to Norwegian Standard (NS3473, 1992), where design principles for regions where the assumption that plane sections remain plane is invalid are:

- The main principle is that the concrete is assumed to transfer compression forces by assumed concentrated struts or more distributed compression fields with a series of compression struts between cracks. The main tensile forces are transferred by reinforcement with directions in conformity with the force model. The reinforcement is anchored safely at, or beyond the assumed nodes of the model.
- Full utilization of the design strength of the reinforcement ties may be assumed only when tensile strains corresponding to the yield strain of the reinforcement can be accepted.

- The effective cross section of the compression struts or compression fields is to be assumed in accordance with recognized calculation models. The design compressive strength of the concrete in the struts is to take into account the effect of the cracking parallel to the principal compressive strain direction. A modified design strength dependent on the transverse strain is given, e.g. in NS3473. This implies that the transverse strains must be controlled and limited. Usually a simplified assessment of the transverse strain as a basis for the design of the distributed reinforcement directed approximately normal to the struts is required.
- The transverse reinforcement must be able to resist tensile forces due to possible deviations from the assumed compression field. Insufficient transverse reinforcement may lead to splitting of the compression struts with a shear failure through the compression field as a possible consequence.
- The compression capacity may be decisive in concentrated joints. Simplified rules, especially applicable for concentrated loads and supports are given, e.g. in NS 3473. In joints where one or more tensile ties will be anchored, it is important that the reinforcement is safely anchored in, or behind, the joint area to ensure a safe transfer of forces between the tensile ties and the compression struts.

A main challenge as regards the use of simplified force models is to develop models in sufficient conformity with general strain compatibility requirements. If there is no recognized calculation model for the member and states of stress in question, the geometry of the model may be developed on the basis of load testing or theoretical analysis based on strain compatibility of members with similar or comparable geometry. Analysis of the member by linear finite element methods may give valuable support for the choice of force directions and force distribution in statically indeterminate models. Linear analysis will, however, not give realistic results in cases where the cracking of the concrete will significantly change the flow of forces in the area, e.g. in members with tensile stresses around sharp internal corners.

Calculation models for typical discontinuity regions may be verified and modified by non-linear finite element analyses where the reinforcement units and the cracking of the concrete are modelled. It is, however, important to be aware of the limitation of the validity of current practically applicable non-linear analysis methods where, for example, an unrealistic modelling of perfect bond between concrete and steel is usually applied. For important, typical intersection areas, load testing of the prototype or scaled down physical specimens may be required.

According to NS3473, force models should be applied in order to determine internal forces at distances less than the effective depth (d) from the support or from concentrated loads. The provision is especially intended to ensure safe transfer of forces to concentrated supports. In shell structure intersection regions with more gradual transition into corner areas, often with indirect support of one structural member on the other, it is important to define the sections where the shear capacity verification should change from the simplified method to a force model (truss, strut and tie or compression field model). An example is shown in Fig. 5.9.

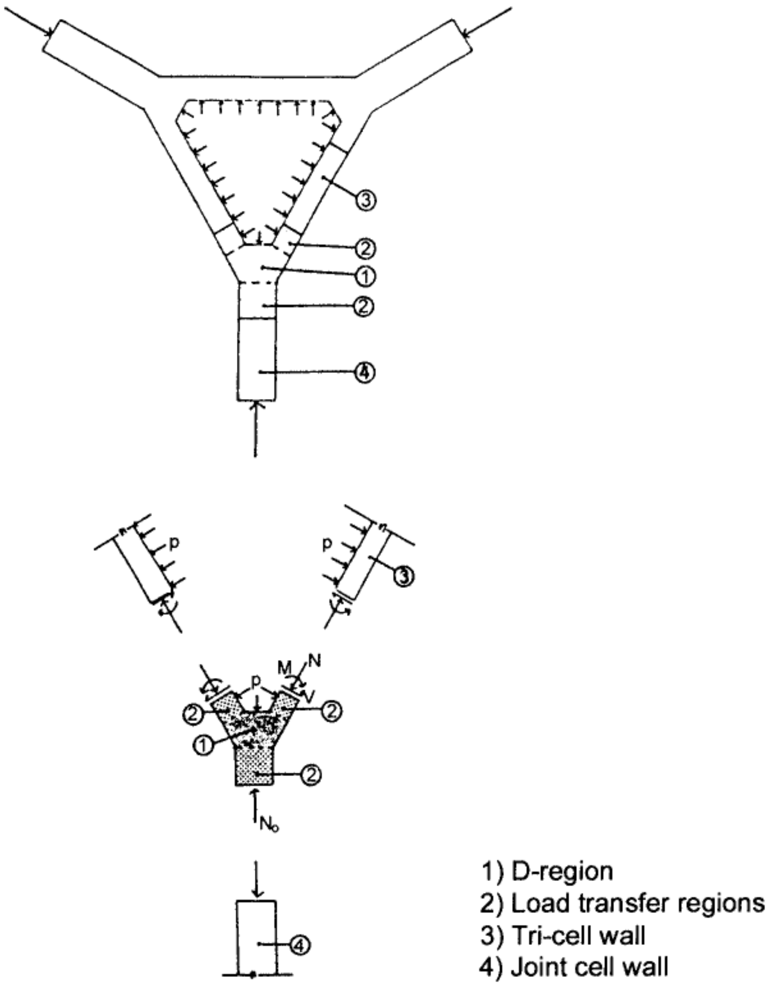


Fig. 5.9 Definition of discontinuity regions. Tri-cell corner

(b) Example. Corner area in Sleipner A1 Condeep platform

Failure in the corner area in the Condeep structure Sleipner A1 was the reason for the loss of the platform. Hence, the joint between the tri-cell walls and the joint cell wall in this structure has been tested in full scale (Jakobsen *et al.,* 1993) and thoroughly analysed. Possible models for the design of 60-degree corners with opening moment, shear force and axial forces are discussed with reference to the results for reconstruction of Sleipner A1.

Simple models for the transfer of pure opening moment in wall corners are shown in Fig. 5.10. The capacity of 90-degree corner with the simple reinforcement as shown in Fig. 5.10 a) will be very limited. The main reason is that the compression diagonal in the corner will in reality not be rectilinear as depicted. Tensile stresses directed along the intersection diagonal between the inner and outer corners will cause cracking parallel to the compression diagonal in the corner when the tensile strength of the concrete is exceeded. After cracking, the unbalanced force resultant on the concrete body outside the crack will cause a push-off failure of the corner. With the exception of thin wall corners with moderate stresses and low reinforcement ratios, it will be necessary to apply diagonal links as indicated in principle in Fig. 5.12 b). The necessary reinforcement detailing will be strongly scale dependent.

A pure bending moment is transferred more naturally by the simple reinforcement with almost constant internal lever arm around the 60-degree corner shown in Fig. 5.10 c). The tensile force resultant of the inside reinforcements is naturally intersecting the deviation angles of the compression struts. However, to avoid excessive width of the tensile crack occurring in the inner corner, a transverse reinforcement crossing the inner corner as shown in Fig. 5.10 d) is recommended.

Fig. 5.12 shows corners with significant shear forces and corresponding balancing axial forces in addition to the opening bending moments. In the 90-degree corner in Fig. 5.11 a) the shear force in one of the adjacent walls may be picked up by the main tensile reinforcement in the opposite wall. Supplementary reinforcement is not necessary, however, the anchorage of the tensile reinforcement close to the outside compression face is increasingly important.

The shear force in the 60-degree corner in Fig. 5.11 b) cannot be transferred directly to the main reinforcement in the opposite wall. This reinforcement is slanting 30 degrees to the "wrong" side compared to the principal tensile direction of the stresses at the axis of the adjacent wall. Safe transfer of the shear forces depends entirely on the transverse reinforcement A suspension support of the walls is established by the transverse corner reinforcement. The support function of the reinforcement is enhanced if the corner is equipped with a haunch with the reinforcement placed in the haunch as shown in Fig. 5.11 b). In this case the reinforcement will also take over the transfer of the tensile force due to the opening corner moment.

The transverse reinforcement crossing the inner corner is also recommended for 90-degree corners to minimize the width of the corner crack.

The intersections between the tri-cell walls and the joint main-cell wall in a Condeep structure with high water pressure externally and inside the tri-cells will be subjected to large axial compression forces in the walls in addition to moment and shear in the tri-cell corners. The axial forces are often dominating, i.e. the components normal to the joint wall of the axial forces and the shear forces in the tri-cell walls result in transverse compression in the theoretical node point of the intersection. This is often the case also when additional wedge forces due to water pressure in cracks are taken into account. Large axial forces compared to the opening moment will also tend to minimize the tension forces in the reinforcement at the inside of the tri-cell walls near the inner corner.

Due to the thickness of the walls, the inside faces will, however, meet at the inner corner at a considerable distance from the theoretical intersection point of the wall centre-lines. The ratio of the thickness of the tri-cell wall to the joint wall will increase this distance. Due to the strain gradient in the corner, large transversal tensile stresses may occur near the inner corner in spite of the large axial compression forces. The transverse stress resultant will increase radically by the introduction of a corner haunch.

In the simplified force model of the Sleipner A1 tri-cell corner shown in Fig. 5.12 a), the component normal to the tri-cell wall of the tensile force (F_{sv}) in the transverse reinforcement in the haunch must equilibrate the full shear force of the tri-cell wall. This reinforcement must also transfer the resulting tensile forces due to opening moment/axial force and additional wedge forces due to water pressure in cracks in the haunch. Supplementary distributed transverse reinforcement is necessary to balance water pressure on cracks deeper into the intersection region.

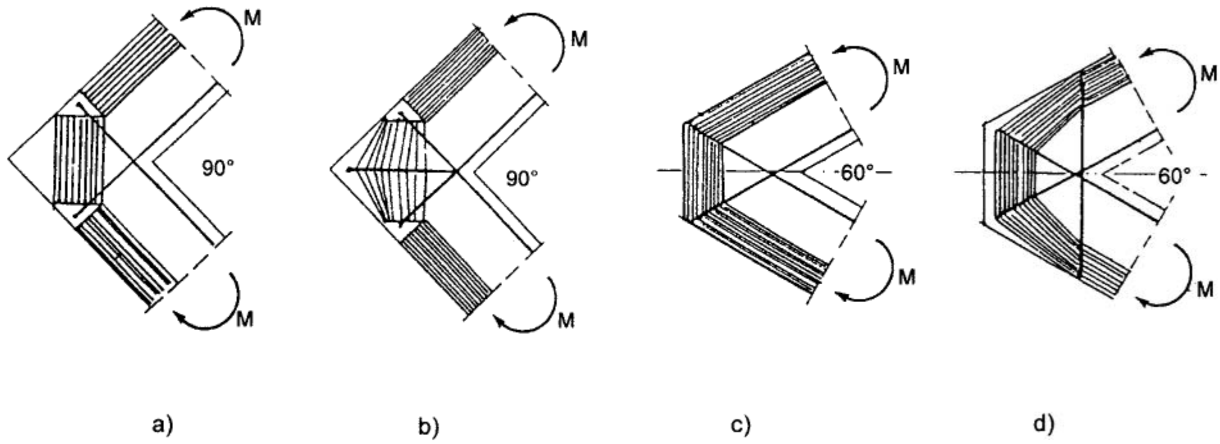


Fig. 5.10 Corners with pure opening moment

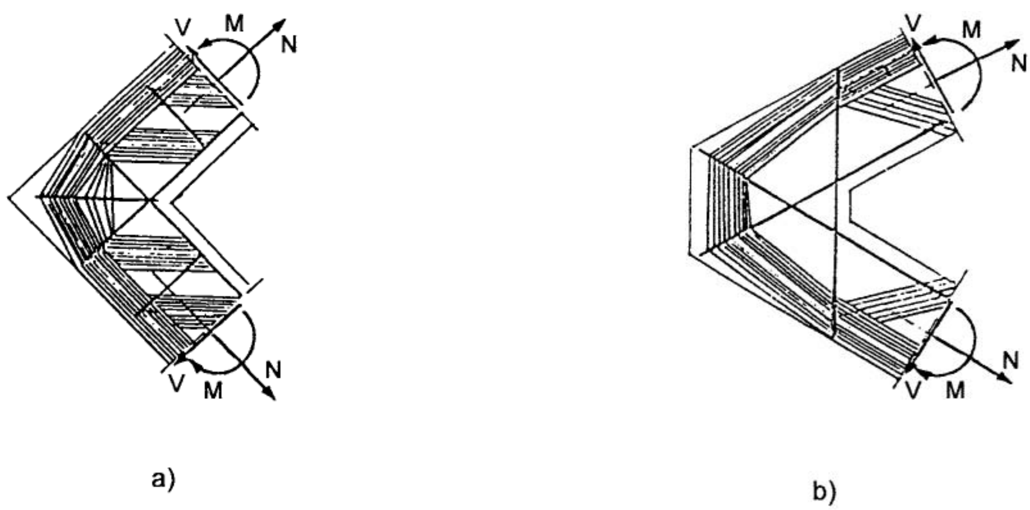


Fig. 5.11 Corners with opening moment, shear force and axial force

The problems of establishing a consistent model when the transverse reinforcement is too short are indicated in Fig. 5.12 b). Especially region A, with little or no links between the wall faces, will be over-stressed. The tensile crack occurring behind the anchor plate of the T-headed bar will initiate the development of a shear failure through the corner area.

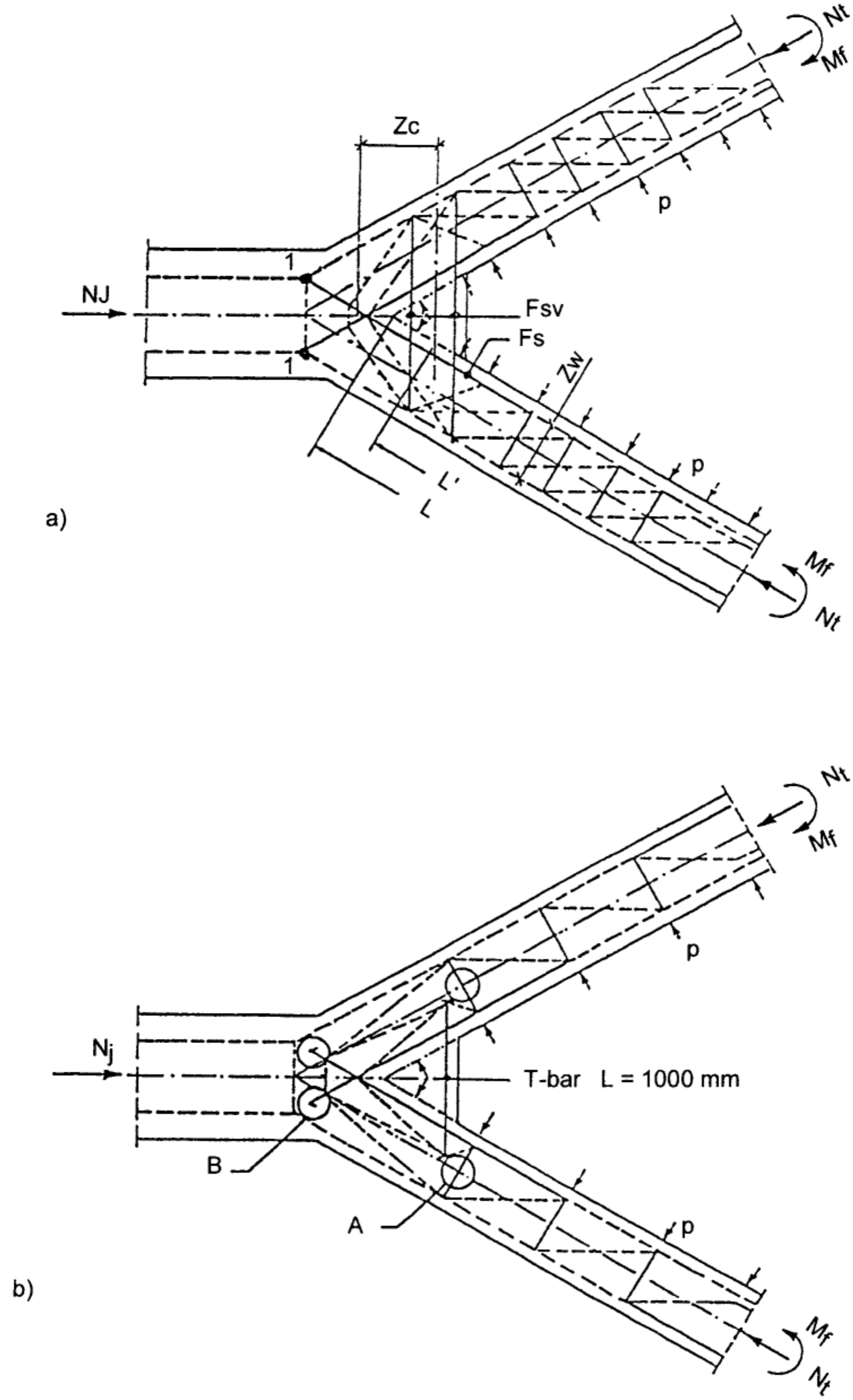


Fig. 5.12 Force models for the tri-cell corner of Sleipner A1

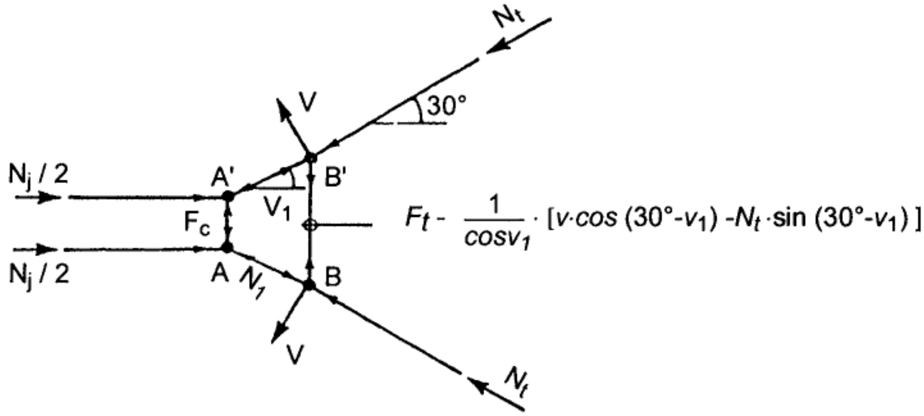


Fig. 5.13 Interaction of shear, axial forces and transverse forces in tri-cell corner

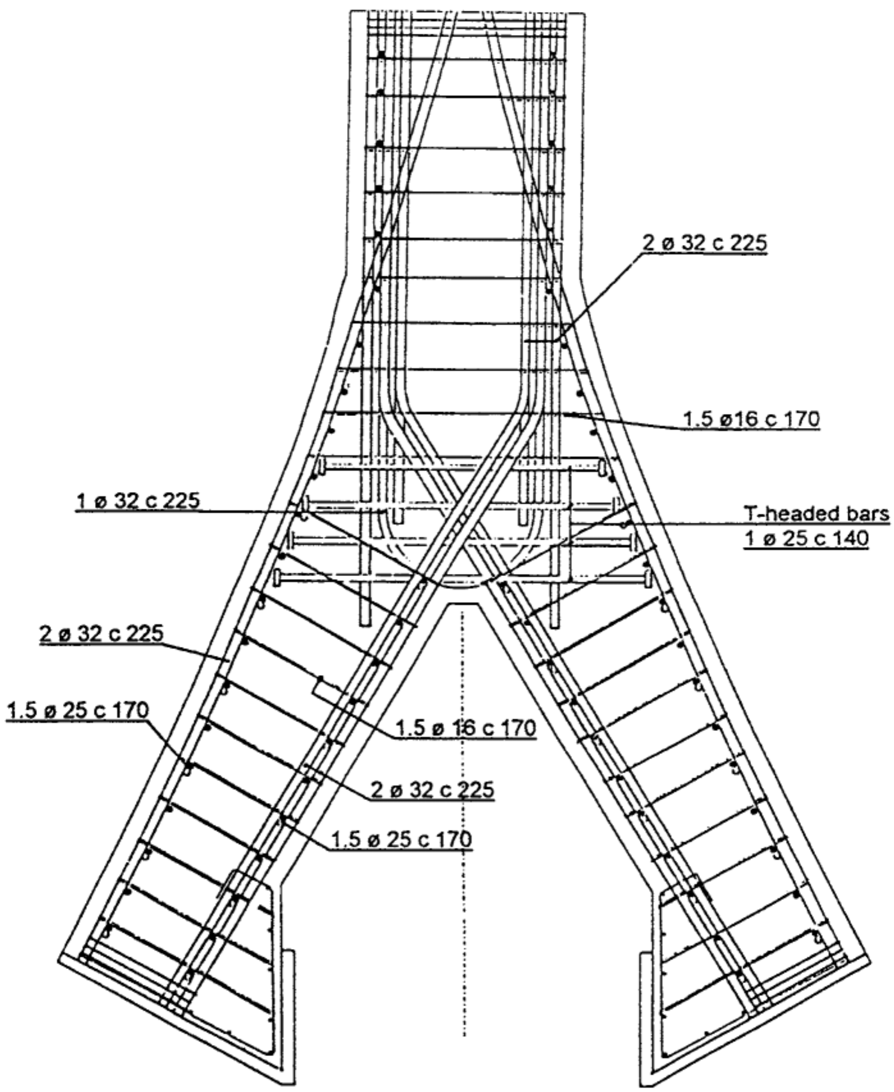


Fig. 5.14 Reinforcement in Y-shaped test specimen. Sleipner A2

The necessary transverse reinforcement may be somewhat reduced by a further development of the force model in Fig. 5.12 a). The main question is to which extent the shear forces can be carried by a change of direction of the compression struts in the corner area. The interaction of the shear and normal forces in the corner is indicated in Fig. 5.13. When the normal forces are large, the transverse force (F_t) will decrease considerably by a small change of direction of the normal force at point B.

The anchorage of the transverse reinforcement as close as possible to the opposite surface is a condition for the model to work as conceived. Tensile stresses in the concrete behind the end of the transverse reinforcement may initiate shear failure if the reinforcement is too short. Detailing of the transverse reinforcement with due consideration of practical placing of the reinforcement in slip-form construction is very important.

A series of small diameter bars with hooks around the outer horizontal reinforcement is probably the best structural solution, but requires strict tolerances and many different reinforcement units. Large diameter transverse bars anchored by overlap parallel to outer layer horizontal reinforcement are possible, but difficult to place. Anchor plates in the form of right-angle T-heads will interfere with the concrete cover if anchored outside the outer layer of the longitudinal reinforcement as desired. A combination of more than one type of reinforcement may be practical. Fig. 5.14 shows reinforcement detail sketch with T-headed bars combined with thinner hooked links.

The Y-shaped specimen shown was load-tested as part of the development of reinforcement details for the new Sleipner platform. The test proved satisfactory performance with a surplus capacity without any sign of shear failure. The test confirmed that large tensile forces occur in the corner, however; in the actual case the number of layers of T-bars probably could have been reduced without significant change of the load bearing capacity.

5.3.5 Ship collision and falling objects

The design of concrete structures for ship collisions and impact from falling objects will mainly imply the design of walls and shells for various kinds of punching effects. The probability of occurrence of collision loads exceeding certain limits determines whether the design is performed according to the ultimate or accidental limit state criteria.

By ship collisions, the magnitude and distribution of a static equivalent load is mainly determined by the speed, mass and deformability of the colliding vessel itself. Impact mechanics and impact loads are dealt with in Chapter 3, Section 3.6.

As a rule the ordinary design formula for the resistance to local concentrated static loads (punching shear capacity) are applied. Shear reinforcement is often required in structures exposed to ship collisions between specified levels above and below the operational mean water level. In the case of collision with large ships, the global resistance of the structure will often be decisive.

Falling objects are usually regarded as stiff bodies with a specified kinetic energy hitting the concrete structure at a concentrated contact surface. The impact of falling objects with high velocity and small cross section may penetrate into the concrete and eventually lead to direct perforation of the concrete shell structure.

The exposed upper domes of a Condeep structure or the pontoons of a floating concrete structure are often protected against moderate impacts from falling objects by a layer of low-strength lightweight aggregate concrete. In this case the load may be determined by the dynamic crushing strength of the protective concrete, and the penetration depth by equating the impact energy and the internal work. The resistance of the main structure is related to the static punching shear resistance with increased material strengths due to the high strain rates. The existing rules may not be fully applicable for extremely concentrated loads.

5.3.6 Fatigue

Fatigue of the longitudinal or shear reinforcement due to the large number of load cycles from the wave action may be decisive for certain parts of concrete platform structures. Fatigue of concrete in compression is seldom decisive.

Design rules concerning fatigue of reinforced and prestressed concrete are constantly developing. Recommended references are CEB-FIP Model Code (CEB-FIP Model Code 1990) and Norwegian Standard (NS3473, 1992). Safety against fatigue failure may be differentiated dependent on the basis of failure consequences and inspection accessibility. An example of safety differentiation is found in the regulations by the Norwegian Petroleum Directorate (NPD, 1992), where the number of cycles during the assumed service life is multiplied by a specified fatigue factor.

5.3.7 Prestressing

Several advantages can be achieved by the prestressing of structures. Prestressing may be necessary in certain parts of the structure to comply with specified requirements regarding water-tightness or limitation of crack widths to avoid corrosion of the reinforcement. Prestressing is also applied to decrease the stress range in the ordinary reinforcement in structures subjected to fatigue-load cycles. Prestressed structures are to a larger extent performing in the un-cracked state, with larger stiffness and better conformity between the linear analysis and the design. The use of high strength prestressed reinforcement substituting a larger amount of ordinary reinforcement will decrease the weight of the structure, which will be advantageous in highly weight-sensitive floating structures.

On the other hand, prestressing of areas of the structure, which will be subjected to large compression stresses by reversal of the direction of the load, might give undesirable additional compressive stresses.

The degree of prestressing of offshore structures is often presented as the percentage of the load effect of the characteristic wave action which is counteracted by the prestressing without tensile stresses in the critical section.

The effect of prestressing is usually taken into account by a basic load case in the global analysis. The time dependent loss of prestress is taken into account by determination of an approximate single loss factor for the actual part of the structure.

The needed prestressing force depends on the load effects. The optimal choice of degree of prestressing is discussed and decided in each single case. The necessary prestressing is decided mainly by the requirements regarding durability and water-tightness.

By the checking of maximum allowable crack widths to ensure durability without corrosion, it is usually accepted to calculate the load effects (normal force and moments) of waves occurring 100 times during the service life of the structure. These load effects will be about 75% of the effect of the characteristic wave with 100-year return period.

The usual water-tightness criterion for important buoyancy compartments is that resulting membrane tension should be avoided in service state. An important question is for which wave load (return period) this requirement should be satisfied.

If the same load level as for the durability requirement is applied, there is a risk that structures designed for water-tightness may experience cracking through the full sections due to seldom loads, but still with a high probability of occurrence during the service life of the structure. Cracks through the section represent a weakness zone with potential water leakage also after the closure of the crack.

Prestressing capable of resisting the axial forces due to the full characteristic (100-year) wave action without cracks through the section is recommended for structures where the tightness of the structure during the full service life is emphasized. It seems, however, reasonable to take into account a safe portion in the tensile strength of the concrete when designing for such rare wave loads.

The requirements in NS3473 regarding the maximum allowable stresses in the prestressing reinforcement may also influence the effectiveness of the prestressing.

In the section of NS3473 concerning the Serviceability Limit State it is stated:

> The stresses in the prestressed reinforcement shall for no combination of actions exceed 0.8 f_y, alternatively 0.8 f_{02}.
> During prestressing, however, stresses up to 0.85 f_y, alternatively 0.85 f_{02}, may be permitted provided it is documented that this does not harm the steel, and if the prestressing force is measured directly by accurate equipment.

The background for these requirements is mainly to prevent excessive loss of prestress in the service state. This may occur if the steel is stressed significantly above the proportionality limit or exposed to such high sustained stresses that the relaxation of the steel increases considerably.

Because of the gradual decrease of the stresses in the prestressing steel with time due to the general prestressing losses, the above requirements will usually be decisive during prestressing. However, the stress in the prestressing steel may exceed the initial prestress in special cases with especially high service loads. This may be the case if the load effects of the full characteristic 100-year wave are applied in the Serviceability Limit State. The required limitation of the service stresses may then be decisive for the prestressing steel demand and the possible choice of the prestressing level.

When designing for such seldom loads, it seems acceptable to allow stresses up to 0.85 f_y, alternatively 0.85 f_{02} also in service states of short duration, provided that inelastic strain in the prestressing steel is compensated for by initial prestressing to the same level.

The passive anchors of the prestressing cables may be placed internally or at the surface of the structure. The active prestressing anchor must be accessible from the outside and may be placed in recesses or outside ribs. The use of recesses is the most practical method in slip-form construction.

The anchor recesses will disturb to some extent the flow of forces and the general reinforcement in the structure. The recesses are to be well distributed to avoid continuous weakness zones in the structure. The recesses are grouted when the prestressing is finished, but the compression strength of the grouted section is still somewhat reduced. The reduced compressive and tensile strength loss is to be compensated by additional reinforcement if necessary.

It is important to calculate the tensile forces due to the deviation of the general forces around anchor zones and where the prestressing cables themselves change direction, and provide the necessary reinforcement. Walls and shells are to be equipped with both transversal and extra longitudinal reinforcement in the anchorage zones.

5.4 Reinforcement

5.4.1 Basis for reinforcement design

All reinforcement is to comply with the requirements in relevant codes and the corresponding product standards. The required reinforcement amount calculated by the postprocessor and supplementary manual calculations is the basis for the preparation of reinforcement drawings and schedules.

The reinforcement systems used for offshore structures are principally the same as for onshore structures. The main practical differences from ordinary structures are the large dimensions and high loads, requiring particularly heavy reinforcement.

Furthermore, large parts of concrete platforms are slip-formed, e.g. skirts, cell walls and shafts. The comments concerning practical detailing and reinforcement systems in Section 5.4.3 are therefore mainly related to slip-form construction.

The reinforcement production documents comprise reinforcement schedules, special listings of reinforcement for slip-form construction, "reinforcement keys" etc, in addition to ordinary reinforcement drawings.

5.4.2 Minimum surface reinforcement

The primary purpose of the reinforcement is to transfer internal tensile forces. The commonly used models for calculation of the capacity of concrete structures presuppose that the structure is sufficiently reinforced to establish a stable system of inner forces even after the tensile strength of the concrete is exceeded and tensile cracks occur. The reinforcement must be able to distribute cracks without yielding.

The reinforcement ratio necessary to meet this general requirement depends on the tensile strength of the concrete and the distribution of the tensile stresses at cracking. The largest reinforcement percentage is required when the cracking is caused by axial tension.

According to NS3473 it is allowed to assess the necessary minimum reinforcement in each actual case. Clause 17.4.1:

> In each individual case, the actual structure and state of stress shall be taken into consideration when determining the minimum reinforcement.

By this assessment of the crack distributing ability of the reinforcement it is to be taken into consideration that tension forces due to restraining of imposed deformations may influence the state of stress significantly in minimum reinforced areas. Pure axial tension, however, occurs more rarely.

A reduced percentage of surface reinforcement may still give satisfactory crack distribution in thick structures. The main reason is that the tension forces transferred by reinforcement bonds introduce unevenly distributed tensile stresses in the concrete section.

The required minimum reinforcement is increased in areas of the structure exposed to high water pressure. The reinforcement must resist the additional resultant of the water pressure at the crack surface.

The minimum reinforcement in walls and shells according to the NPD-guidelines (NPD, 1992) takes into account the thickness of the structure and the water pressure. The required minimum amount at each face and in each main direction is as follows:

$$A_{s\,min}=k\,A_c\,(ft_k+av)f_{tk}/f_{sk}$$

where k is a factor varying from 0.4 for wall thickness 300 mm to 0.25 for wall thickness > 800 mm. (interpolation for intermediate wall thicknesses)

v is the actual water pressure

α is a factor, which is taken as 1.0 if the relevance of lower values is not documented

A_c is the area of the concrete section

f_{tk} is the tensile strength of the concrete

f_{sk} is the yield strength of the reinforcement.

The theoretical k-value is 0.5 if the stress at cracking is constant through the thickness and ftk is the actual tensile strength. Crack distribution is therefore not guaranteed by the required minimum reinforcement if this situation can occur.

Prestressed reinforcement in injected ducts only contributes slightly to the distribution of cracks.

The general requirement regarding compressive reinforcement is not so clear. Standard requirements have been related to general assessment of the safety of reinforced concrete structures. The increased safety of sections consisting of two materials with uncorrelated strength variables may be regarded as a condition for the use of material safety coefficients for reinforced concrete. The risk that tension may occur in unexpected directions, e.g. due to accidental loads, is a further argument for requiring a minimum amount of reinforcement. The direction of the internal forces in shells and wall members of offshore structures subjected to environmental loads may vary considerably. Equal minimum reinforcement at both faces and in both main directions is therefore usually required.

5.4.3 Minimum transverse reinforcement

Simplification of the reinforcement by omitting transverse reinforcement is accepted in ordinary thin slabs and walls in building structures. Slabs, as opposed to beams, may be designed without shear reinforcement if the shear capacity calculated according to the simplified method is sufficient without stirrups.

This has also been good practice for ordinary shell structures. A minimum amount of shear reinforcement is required only when the shear reinforcement is assumed to contribute to the shear capacity of the shell. The usual requirement: A_s min=0.2 A_c f_{tk} /f_{sk} is identical to the requirement for slabs in NS3473. For concrete strength C75 and reinforcement yield strength

500 MPa this corresponds to, for example, Ø12 mm reinforcement links with spacing of 270 mm in both main directions.

There are several special reasons for requiring a general minimum transversal reinforcement in walls and shells in large marine structures. Some of these reasons are:

- Scale factors for thick structures
- The use of concrete with high strength and brittleness
- Water pressure in pores and cracks
- Frequently lap sliced reinforcement
- High bi-axial compression stresses.

The use of a general minimum amount of transverse reinforcement is recommended in thick structures exposed to high water pressure. Specific general requirements are lacking.

5.4.4 Reinforcement layout and detailing

The detailing of the reinforcement in structures composed of intersecting cells satisfying the requirements regarding the continuity and safe anchorage of the reinforcement and, at the same time, the desired simplicity of production and placing of the reinforcement, is a challenging design task.

Based on the calculated theoretically necessary amount of reinforcement in a limited number of design points, a practical curtailment of the reinforcement is to be chosen which satisfies the required amount in all sections by a reasonably simple variation of the reinforcement in different sectors and levels.

The necessary extension of the reinforcement in order to cover the additional tension force due to the effect of transverse shear and the necessary anchorage length beyond the theoretical points where a part of the reinforcement may be terminated, are to be considered by the curtailment of the longitudinal reinforcement.

The curtailment of the transverse reinforcement is related to the distribution of the shear force. The recommended shear reinforcement curtailment for suspended loading is to be chosen where the shear force is due to water pressure, which may penetrate into cracks; see Fig. 5.7.

Continuity of the reinforcement with effective splicing and anchorage should be emphasized. However, due to the large dimensions of the structure and the rather short practical lengths of the reinforcement bars in slipform construction, the reinforcement will be frequently lap spliced throughout the whole structure, also in areas with high utilization of the capacity. It is therefore utterly important to stagger the reinforcement and distribute the splices as well as possible, preferably with not more that 1/3 of the reinforcement spliced in the same section. The lap splices in the outer layers are to be secured by stirrups.

The reinforcement amounts are usually increasing towards the structural main nodes in the intersections between the structural members. Realistic models of the force flow in the node areas are necessary and helpful as a basis for the detailing of the reinforcement in such areas. It is especially important to get a clear picture of which parts of the reinforcement should be made continuous and which parts should be anchored and terminated in the node area. The reinforcement is to be shaped in accordance with the analytical model and anchored safely at the assumed local joints.

The general requirements concerning the local positioning of the reinforcement is stated in NS3473 as follows:

- Reinforcement shall be placed in such a way that concreting will not be obstructed and so that sufficient bond anchorage, corrosion protection and fire resistance will be achieved.
- The positions of ribbed bars may be designed in accordance with the given minimum spacings without regard to the ribs, but the actual outer dimensions shall be taken into account when calculating clearance for placing of reinforcement and execution of the concreting.
- The positioning of the reinforcement shall be designed so that the given requirements to the concrete cover can be obtained in compliance with the specified tolerances.

The minimum theoretical clearance between single bars or bundles according to (NS 3473, 1992) is 2 $Ø_e$ (or 1.5 $Ø_e$ in lap splicing areas), where $Ø_e$ is the equivalent diameter of bundles. Bundles are widely used in large structures, but more than 2 bars in each bundle (3 bars at the splices) is avoided as far as possible.

It is important to take into account the actual outer dimensions of ribbed bars when reinforcement in several layers is used. Production of realistic detail drawings (and sometimes prototype tests) of intersection regions is necessary to avoid obstructions during the construction work; (see Fig. 5.14).

Fig. 5.15 shows a simplified sketch of the reinforcement in a slipformed cell wall. The reinforcement must be directed vertically and horizontally due to the yokes carrying the formwork and due to the continuous lifting operation. The horizontal “hoop” reinforcement must be inserted beneath the yokes. The hoop reinforcement is preferably placed in the outer layer outside the vertical reinforcement. The placing of more than one reinforcement layer on each side of the curved wall is difficult. The second horizontal layer must in this case be placed inside the inner vertical layer, but handling of the curved bars in the internal of the wall is still difficult, and the practical bar lengths will be very limited. Reinforcement in several layers is easier to accomplish in plane walls with straight bars.

The horizontal reinforcement is indicated on the drawings with the required constant spacing of the bundles. The required vertical reinforcement intensity is to be recalculated and indicated by the equivalent number of bundles between the yokes. The diameter of the shaft structures will typically vary with the height levels. The distance between the yokes and the number of reinforcement bundles will vary accordingly.

The vertical reinforcement is placed inside guidance racks attached to the slipform construction. The vertical position and a reasonable distribution of the bundles are thereby secured. Vertical bars are placed as close as possible to the yokes to minimize the inevitable increased spacing due to the width of the yokes (typically 200–250 mm). The designer must take into account the necessary practical adjustment of the reinforcement in slipform construction by the choice of a reasonably large theoretical spacing in order to avoid violation of the minimum spacing requirement in practice. The spacing of the vertical bars can be adjusted (if needed) where the hoop reinforcement bars are tied to the vertical reinforcement.

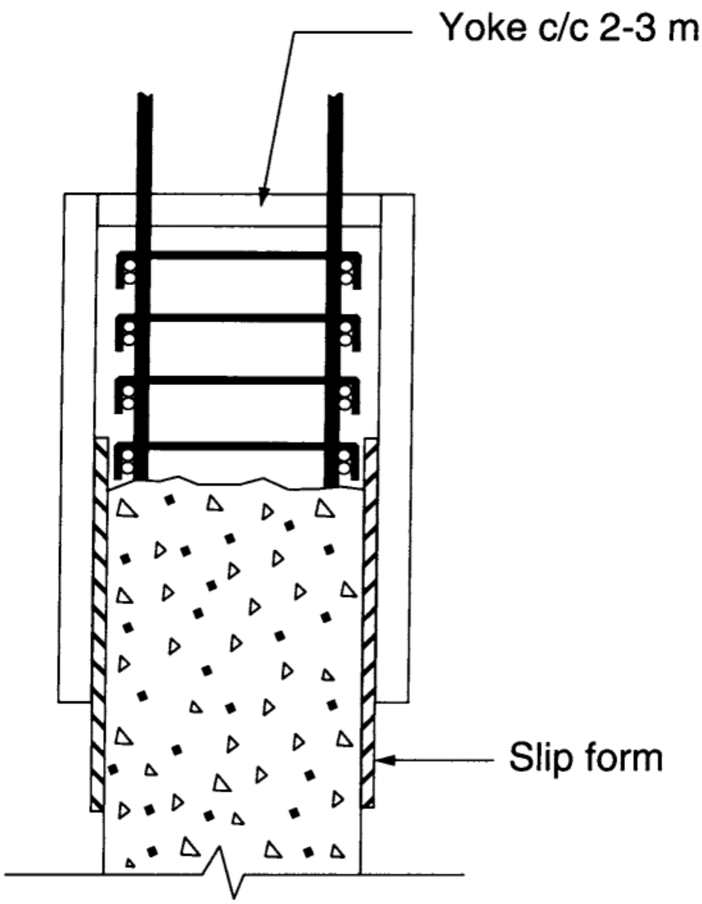


Fig. 5.15 Simplified reinforcement of slipformed wall

Strict tolerances on the spacing of the vertical bundles are required at working joints. The necessary full co-ordination of the reinforcement in adjacent structural parts requires high quality production secured by operative quality assurance systems.

The concrete cover with its assumed tolerances is taken into account in the design. The reinforcement is to be placed within the prescribed tolerances. “Spacers” are usually attached to the top edge of the formwork to obtain the correct minimum cover of the outer layer of the horizontal reinforcement, and thereby also securing the position of the outer layer of the vertical reinforcement. Additional spacers between the reinforcement layers are to be prescribed if more than one layer of vertical reinforcement is used.

By slipform production of structural members with varying dimensions it is important to take precautions to avoid the possibility that the reinforcement is forcing the formwork out of the correct position and direction. The opposite effect that the slipform is guiding the reinforcement, is the desired situation.

Simplification of the reinforcement with equal amounts in reasonably large sectors is preferred in slipform construction. The continuous progress of concreting without delay is the mandatory consideration.

The reinforcement system and detailing suggested by the designer should be discussed with the responsible personnel at the building site before the finalizing of the reinforcement production documents. This concerns primarily the personnel responsible for the reinforcement, but the personnel responsible for the concreting should also be consulted, especially when the density of the reinforcement will require special workability of the concrete. In that case it may also be necessary to determine the position of the concreting tubes in advance.

Thorough discussions with the responsible personnel at the site and adjustment of the reinforcement in order to simplify the execution, but without violation of the required continuity and safe anchorage of the reinforcement, will certainly result in more optimal reinforcement solutions. The staff on site has generally the best knowledge of the problems occurring during the execution of the reinforcement. Some of the points of importance regarding the mounting of the reinforcement are:

- Practical lengths and shapes of the reinforcement (handling weight limits, simplicity of production and installation)
- The number of regions with different reinforcement (a large number of variants will generally increase the risk of errors and delay)
- Practical and clear production documents (drawings, lists, etc).

It is often necessary to increase the number of workers drastically within the limited time periods when the large main parts of the structures are reinforced and concreted. This is especially the case when executing large slipform constructions on continuous shift-work basis. In addition to the proper training of all personnel on site, the best guarantee for a successful result is the choice of a simple reinforcement without unnecessary large number of reinforcement variants.

Reinforcement shapes or amounts are not to be changed on site without the explicit approval by the responsible designer. All changes are to be reported, registered and evaluated. The changes are finally summarized in updated drawings showing the final structure as built.

References

Bjerkeli, L.M. (1990) Water Pressure on Concrete Structures. Dr.ing Thesis **1990:31,** Division of Concrete Structures, The Norwegian Institute of Technology.

Brekke, D.-E., Åldstedt, E. and Grosch, H. (1994) Design of Offshore Concrete Structures Based on Postprocessing of Results from Finite Element Analysis (FEA): Methods, Limitations and Accuracy. *Proceedings of the Fourth International Offshore and Polar Conference,* ISOPE, Osaka, Japan, pp. 318–28.

CEB-FIP. Model Code 1990 (1993). Thomas Telford Services, London

CEN (Comité Européen du Normalisation) (1991) *European Prestandard* ENV 1992–1–1.Eurocode 2. Design of concrete structures .

Collins, M.P. and Mitchell, D. (1991) *Prestressed Concrete Structures.* Prentice-Hall.

ISO standard 13819 Part 3 (to appear, will cover the entire engineering process for offshore concrete structures).

Jakobsen, B., Gausel, E., Stemland, H. and Tomaszewicz, A., (1993) Large-scale tests on cell wall joints of a concrete base structure. *High Strength Concrete 1993 Proceedings. Lillehammer, Norway.*

Norwegian Council for Building Standardisation, NBR (1998), *Norwegian Standard* NS 3473 Concrete Structures, Design rules, 4th edition, Oslo, Norway, 1992 (in English), 5th edition 1998 (English edition in print).

Norwegian Council for Building Standardisation (NBR) (1990) *Norwegian Standard NS* 3479 Design of Structures. Design Loads. 3rd Edition (In Norwegian).

Norwegian Petroleum Directorate (NPD) (1992) *Regulations Concerning Loadbearing Structures in the Petroleum Activities.* Including guidelines for structural design of concrete structures, stipulated by NPD, Stavanger, Norway.

Statoil (1992) N-SD-001. *Specification for Design.* Structural Design for Offshore Installations.

6 Quality assurance

Erik Jersin, SINTEF

6.1 Introduction

6.1.1 Purpose and limitations of Chapter 6

The purpose of this chapter is to give a general view of the basic demands for Quality Assurance and throw light on what these imply when it comes to engineering, design and dimensioning of concrete structures at sea.

The description particularly concentrates on important elements, considering the scope of this book. Consequently, this chapter does not give a complete picture of which requirements should be fulfilled, for instance to fully satisfy *ISO 9001:1994 Quality Systems. Model for quality assurance in design, development, production, installation and servicing.* Thus, among others the following important elements in ISO 9001:1994 have only been dealt with superficially:

- 4.1 Management responsibility
- 4.5 Document and data control
- 4.13 Control of non-conforming products
- 4.14 Corrective and preventive action
- 4.16 Control of quality records

In addition, Quality Assurance related to the development and use of *software* has not been dealt with here. Particular standards for the above-mentioned exist, e.g. *ISO 9000–3:1992 Quality Management and Quality Assurance standards, Part 3: Guidelines for the application of ISO 9001 in development, supply and maintenance of software.*

6.1.2 Special features of engineering and design of concrete structures at sea

In relation to Quality Assurance there are several conditions that characterize engineering and design of concrete structures (Gravity Base Structures—GBS) at sea:

- Improved calculation models and larger computer capacity has led to more reliable calculations and better utilisation of loadbearing structures than previously. Consequently, reduced safety factors have been accepted. However, attention should be drawn to the fact that the introduction of higher and more complex technology, generally speaking, does increase the vulnerability to errors, faults and other nonconformities. Thus, to maintain the same level of safety even better conformance to specifications during engineering, design, dimensioning and construction is required.

- The use of rather complicated computer systems for analysis, calculation and dimensioning implies a certain risk of reduced awareness of the importance of keeping a general view on the construction.
- Short time limits often make it necessary to carry out simultaneously, activities that previously were carried out in series. Concurrent engineering thus leads to a number of iterations and greater demand for managing the changes.
- A considerable number of certain types of GBS's, e.g. Condeep, have been built. Thus there is a danger of reduced attention during the engineering, design and construction processes. When a task or a project comes to be looked upon as a routine job, it is tempting to use younger personnel with less experience. However, if one does not make sure that senior personnel with sufficient experience attend to the control functions, there is a serious risk of jeopardising the quality.
- After the Internal Control and Quality Management/Quality Assurance concepts were brought into focus during the last decade, there has been a shift in attention from the product (the structure) to the processes and procedures that generate the product. The strategy behind this is clear and logical enough. In general, it will be wise to focus on *the prevention* of mistakes, deviations and nonconformities as opposed to inspection and rework at later stages. Thus, by focusing on the organizing, planning, procedures and management systems instead of employing extensive checks of details in the final product, greater profit can be achieved. However, the question could be raised, whether this trend has gone too far during the last decade. Thus, a present challenge could be to find the right balance between (system) control and (product) inspection.

6.1.3 Vocabulary

In the discussion of Quality Assurance in this chapter, the latest version of ISO's vocabulary standard in this field has been used *(ISO 8402:1994 Quality Management and Quality Assurance. Vocabulary).*

In general, it should be noted that in everyday speech one would often come across particular, usually trade-related terms. These could have occurred more or less by chance, and can be a mixture of different languages, e.g. Norwegian, English and American. Such terms should not be used uncritically, because they often lead to confusion rather than improved communication. Two examples are given below.

Design Review is a well-established term in most trades and commonly used in Quality Assurance standards. Within some companies, the term *Engineering Technical Audit* is often used, as well. The content of this term in fact is so similar to Design Review that the need for it could be questioned. Problems that may be experienced in making the Design Reviews effective are hardly solved simply by introducing another term.

Likewise, *Quality Audit* is another well-established term. The term *Implementation Review* is sometimes used to denote a Quality Audit carried out prior to Contract Award. The right term would be, for example, *Pre Contract Quality System Audit.*

Moreover, one should be aware of the fact that the ISO vocabulary is being revised from time to time. In addition, new standards and guides are more or less continuously being drawn up. A need for defining new terms or re-defining previously used terms may arise. Because it usually takes several years to revise an existing ISO-standard and even longer to publish a new one, terminology in itself can be a problem.

The conclusion is that it would be wise to follow the ISO 8402 strictly, avoid self-invented terms, if possible, and by all means define necessary additional terms carefully.

6.1.4 Further reading

The following textbooks could be recommended for more detailed information:

Carrubba, E.R. and Gordon, R.D. *Product Assurance Principles. Integrating Design Assurance and Quality Assurance.* McGraw-Hilll Book Company, New York, 1988. ISBN 0-07-010148-5.

Evans, James R. and Lindsay, William M. *The management and Control of Quality.* West Publishing Company, St. Paul, 1989. ISBN 0-314-47285-1.

Fox, M.J. *A Quality Auditing Manual*. Technical Communications (Publishing) Ltd., Letchworth, 1992. ISBN 0-946655-62-6.

Jersin, Erik. *Kvalitetsstyring—Kvalitetssikring—Kvalitetskontroll.* TAPIR, Trondheim, 1985. (In Norwegian only.) ISBN 82-519-0596-6.

Juran, J.M. *Quality Control Handbook.* McGraw-Hill, New York, 1988, ISBN 0-07-033176-6.

Lock, Dennis (ed.). *Handbook of Quality Management.* Grower Publishing Company Limited, Hants, 1990. ISBN 0-566-02770-4.

Onnias, Athuro. *The Language of Total Quality.* TPOK Publications on Quality. Castellamonte (To), 1992. ISBN 88-86073-00-3.

Robinson, Charles B. *How To Make The Most Of Every Audit: An Etiquette Handbook For Auditing.* ASQC Quality Press, Milwaukee, 1992. ISBN 0-87389-158-9.

Rothery, Brian. *ISO 14 000 and ISO 9000.* Gower Publishing Company, The Netherlands, 1995. ISBN 0-566-0764-89.

6.2 Basic requirements for quality assurance

6.2.1 Reference requirements

The basic Quality Assurance requirements that regulate the activities of offshore operators and contractors at the Norwegian continental shelf have been laid down in the following documents:

- *ISO 9001:1994 Quality Systems. Model for quality assurance in design, development, production, installation and servicing.*
- *The Petroleum Act* and corresponding regulations, particularly *The Norwegian Petroleum Directorate (NPD): Regulations concerning the licensee's internal control in petroleum*

activities on the Norwegian Continental Shelf with comments ("The Internal Control Regulation")

In addition, several guides are in frequent use. These are not regulatory documents, but offer guidelines. Particular attention should be drawn to the following:

- *ISO 9004–1:1994 Quality management and quality system elements. Part 1: Guidelines.*
- *ISO 9000–3:1992 Quality management and quality assurance standards. Part 3: Guidelines for the application of ISO 9001 to the development, supply and maintenance of software.*
- *ISO 10011:1992 Guidelines for auditing quality systems. Parts 1, 2 and 3.*
- *ISO 9004–4:1995 Quality management and quality system elements. Part 4: Guidelines for quality improvement.*

Note that "The ISO 9000-family" is being revised at present. The target dates for publishing the revised standards are year 2000–2001. However, there is no reason to assume that the actual quality assurance requirements in the revised standards will be significantly changed.

6.2.2 The three basic principles of Quality Assurance

In view of the large number of documents that exist when it comes to Quality Management (QM) and Quality Assurance (QA), it is important to keep in mind the following three simple and basic QA principles:

- ***Prevent***
- ***Detect***
- ***Correct***

These principles in fact form the three-pronged basic strategy of all commonly used QA standards and guidelines to day. Table 6.1 gives a broad outline of how the most usual Quality Assurance requirements can be grouped accordingly.
Each element is described in more detail below.

Table 6.1. The basic principles of Quality Assurance and some corresponding requirements

a. PREVENT *NONCONFORMITIES*	- All relevant parts of the organization has to be comprised by the QA efforts - The product requirements shall be unambiguous - Qualification requirements shall be established for personnel as well as processes - Responsibilities shall be clearly defined - All product requirements and changes shall be recorded - The quality system shall be documented
b. DETECT *NONCONFORMITIES*	- Results shall be checked against requirements and recorded - The quality system shall be audited - The processes and/or final product shall be properly verified
c. CORRECT *NONCONFORMITIES*	- Nonconformities shall be treated in a prescribed and systematic way - Recurrences shall be prevented

(a) Prevent

In modern Quality Assurance, the main consideration is to prevent defects and nonconformities from occurring in the first place. This is because it would always be profitable to do things "right the first time", as opposed to correcting mistakes and nonconformities later on. In addition, prevention increases the chances of the final product being correct, since there is always a risk that inherent defects and nonconformities will not be detected at all, or not until it is too late. Note that the term "product" may include service, hardware, processed materials, software or a combination thereof (ISO 8402). In this chapter, "product" also includes intermediate results of the engineering, design and construction process, i.e. calculations, drawings, specifications and other documents.

The most important preventive activity during engineering and design, in addition to good organizing and planning, is to make sure that the key personnel working on the project have sufficient and suitable competence (knowledge and experience).

(b) Detect

In general, it should be appreciated that modern technology and the interaction between technology and man, is not well enough developed to prevent all defects and nonconformities from occurring. Man is often considered to be the weakest joint in the chain. This goes for many products and businesses. It is therefore important to realise that defects and nonconformities certainly *will* occur from time to time, almost regardless of how much effort is put into preventing them.[1] Efficient checks, inspections, audits and reviews must therefore be organized to ensure that the nonconformities are discovered and as quickly as possible, before they lead to serious consequences in terms of quality or safety problems, or financial losses.

During engineering, design and dimensioning, such checks should be carried out at different times and by means of various techniques. As mentioned above, the checks would have two objectives. One is to verify the *processes,* i.e. that the quality control and quality assurance systems are effectively planned and implemented. To this end Quality (System) Audits should be carried out. The other objective is to check the *output,* i.e. the results of the different activities during engineering, design and dimensioning. To this end Self-Checks, Discipline Checks (DC), Inter Discipline Checks (IDC) and Third Party Verifications are vital.

(c) Correct

As soon as a nonconformity has been detected, it must be assessed and a decision about how it should be treated further must be made. It is important that the decision is made at the right level in the organization. "Right level" here usually means the lowest level that has a complete view of the consequences of the decision. Usually, the decision concerning nonconformity in the engineering and design phase will be to correct or revise the relevant documents, calculations, etc. to satisfy the requirements. However, sometimes it may be relevant to apply for a *deviation permit,* with or without rework. This would often imply a need for approval by the authorities. In any case, the procedure for treatment of nonconformities should be well defined beforehand. To ensure verifiability and possibility of supervision of nonconformities, the decisions must be properly recorded as well.

Having treated the nonconformity in a justifiable manner, it is important to analyse *why* it

[1] The nuclear, space and aviation industries put down more efforts and resources to prevent failures than any other trade. Still, serious accidents do occur and unfortunately will continue to occur from time to time in the future.

occurred, i.e. to find the root cause, and implement the necessary measures to prevent recurrences. Such measures are usually called *Corrective Actions.* The analysis also includes an evaluation of the present nonconformity in view of previous ones, in order to discover and correct prospective unfortunate trends as early as possible. If, for example, a significantly increasing number of defects have been detected on drawings after they have been distributed, this could be a reason for tightening up the Self-Check requirement on the designers. Alternatively, the drawing software programs should be re-checked (verified).

6.3 Quality assurance in engineering and design of concrete structures

6.3.1 The ISO requirements for Quality Assurance

ISO 9001 describes a series of general requirements to the quality system in engineering and design, construction, installation and servicing. In this chapter we presuppose that such a basic system has been implemented. This means that the ISO requirements regarding determination of Quality Policy, responsibility and authority, etc. are fulfilled and in addition that procedures exist for document control, nonconformity treatment, corrective actions, quality records, internal quality audits, training, etc. In this chapter, we will therefore concentrate on those elements that are particularly relevant in engineering and design of concrete structures.

The general requirements for Quality Assurance in engineering and design are described in the following ISO standards:

- ISO 9001:1994, item 4.4
- ISO 9004–1:1994, item 8[2].

In general, the ISO 8402:1994 terms have been used. In the cases where these deviate from the terms that are used among concrete professionals, the latter have been used.

6.3.2 Some important QA-elements and QA-tools in engineering and design

"QA-elements" in this context means the procedures, etc. that are required for the Quality System to satisfy the 20 main requirements in the ISO 9001:1994 standard. These elements have been given the numbers 4.1 to 4.20 in the standard. "QA-tools" are used as a less precise term for certain specific techniques and tools that will contribute to fulfilling the requirements effectively.

Some of those elements and tools have been described in brief below, namely qualifying key personnel, Self-Check, Discipline Check (DC), Inter Discipline Check (IDC), Third Party Verification, Design Review (DR), Hazard and Operability Studies (HAZOP), Worst-Case Analysis and Quality Audit.

Fuller descriptions are given in Appendices B-H.

[2]Note that ISO 9004–1 is a Guide; it is not intended as a contract requirement.

(a) Qualifying key personnel

In design of concrete structures, as in other high technology projects, one of the most important preventive measures is ensuring that fully qualified personnel are assigned to the key positions. Before anyone is appointed, it should be made clear which basic qualifications and what type of experience, for example from similar projects, are required. Subsequently, the most suitable candidates should be selected based on CVs, references and interviews. Finally, the candidates should be asked to outline how they will attack the task, and the answers evaluated.

(b) Self-Check

All analyses, calculations and documents should be thoroughly self-checked, that is to say checked by the person or persons who have carried out the work, before the results are passed on to others. As mentioned, the policy should be to make everything *right the first time.*

Fig. 6.1 shows the relation between Self-Checks and the other elements in the verification process.[3] When Fig. 6.1 is read vertically, the order of the different checks appears. Thus, all documents and drawings should first be subject to a Self-Check, i.e. a check by the same person who is carrying out the task. If a nonconformity (NC) to the specification for that particular task is observed it should be corrected at once, i.e. before the document or drawing is passed on. The relevance of the subsequent checks depends on the criticality of the document or drawing. If the task has no criticality classification at all, the result would pass on to a Design Review (if required) or directly for approval. If classified I, II or III (see Table 6.2) the document or drawing would be subject to a *Discipline Check* at a suitable level. Documents or drawings of Criticality II and III would in addition be subject to an *Inter Discipline Check* (if relevant). The most critical results (Criticality III) would finally be subject to a *Third Party Verification* as well.

If nonconformity is detected during these checks, the document or drawing will, of course, have to be corrected or re-worked. In Fig. 6.1 this is indicated by the dotted feed-back loops.

Design Review, Discipline Check, Inter Discipline Check and Third Party Verification are more closely examined below and in Appendices B—E.

(c) Criticality

The amount or level of quality assurance of documents and drawings should reflect their importance (level of criticality), which could be decided by a qualified person or by a Design Review. The criticality should be clearly stated in the document. This is to ensure that the level of attention coincides with the document's importance in relation to quality, safety, environment and/or economy. Three levels of criticality are described in Table 6.2. In engineering and design of Gravity Base Structures (GBS's) a major part of the technical calculations, documents and drawings will in principle be of Criticality III. However, as indicated in Fig. 6.1, there are still several possibilities for tailoring the amount of quality control to a suitable level.

(d) Discipline check (DC)

A Discipline Check is an inspection to ensure that the technical documentation satisfies all internal and external requirements within one's own discipline before further distribution and use. In order to be acceptable, a Discipline Check has to be *independent,* i.e. carried out by a competent person or body other than the one(s) that drew up the documents, or were responsible for or involved in the process in some other way. This goes for the other types of

[3] The term *verification* is used here as a joint term for several types of checks that aim at providing evidence that specified requirements has been fulfilled.

verification as well, except for Self-Check. However, as mentioned above, the necessary level of independence may vary according to the criticality of the document or drawing.

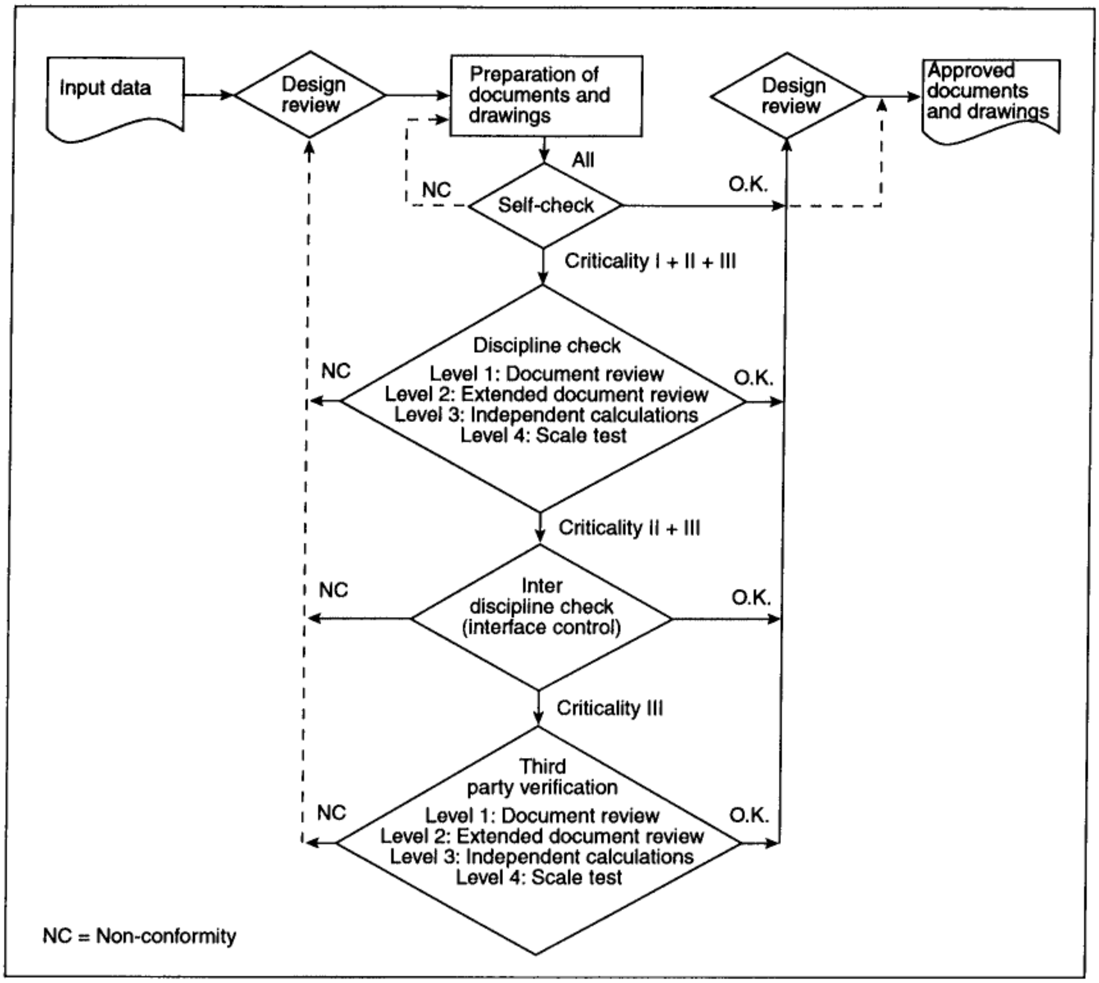


Fig. 6.1. The verification process in engineering and design

The reason for the principle of independent checks is that, according to many years of experience, it is usually much easier to discover mistakes, defects and weaknesses when it comes to other's work than one's own. In addition, those who carry out the checks should not be under any influence or pressure that could have a negative impact on quality. Such a pressure would, for example, exist if the person(s) carrying out the checks would also be held responsible for lost time or additional costs resulting from detecting and correcting errors. Discipline Check (as well as Third Party Verification; see Fig. 6.1) can be carried out on four different levels:

Level 1: Document Review
Level 2: Extended Document Review
Level 3: Independent Calculation
Level 4: Scale Test

For a further description of Discipline Check, as well as the four above-mentioned levels, see Appendix B and Chapter 7, Verification of design.

Table 6.2. Suggested criticality classification of documents and drawings[4]

Level of criticality	Explanation
III	Defects and flaws will or can lead to loss of human lives, major environmental consequences or major economic losses. Changes may have a great impact on other disciplines.
II	Defects and flaws will or can lead to personnel injuries, moderate environmental damage or moderate economic losses. Changes may have an impact on other disciplines.
I	Defects and flaws will probably not lead to damage or losses of great importance. Changes will have little or no impact on other disciplines.

(e) Inter discipline check (IDC)

An Inter Discipline Check is an inspection and a review of technical documentation to ensure that it fulfils all internal and external requirements and considerations for other (i.e. interfacing) technical disciplines before further distribution and use. The check should be carried out by competent persons from the relevant other disciplines.

An Inter Discipline Check is an independent check. It comes after, and in addition to, Self-Check and Discipline Check (see Fig. 6.1). It is carried out only when an interface to other disciplines exists.

The Inter Discipline Check has been further described in Appendix C.

(f) Third party verification/External verification

A Third Party or External Verification of documents and drawings consists of Document Review (Level 1), Extended Document Review (Level 2), Independent Calculations (Level 3) or Scale Test (Level 4) carried out by a different company or organization than the one responsible for the executed work (Fig. 6.1). Third Party Verification may also include external Quality Audits to verify that the quality system is effective.

From the authorities' point of view, the main aim of Third Party Verification is to obtain objective evidence that the requirements have been met. The operator could therefore be instructed to carry it out. From the point of view of the operator and the personnel involved in engineering and design, it is important that the Third Party Verification is synchronised with the design work. The reason behind this is that the verification can and should give effective support and current corrections to design. This has been further explained in Chapter 7, Verification of design.

[4] Note that several publications and companies use the opposite order, i.e. Level I is the most critical, Level III the least critical. The reason for the suggested order in this book is to match the Levels 1–4 in Fig. 6.1

Third Party Verification can be costly and should therefore only be used to identify and verify the most critical parts of the structure (Criticality III, see Table 6.2). For further description, also see Appendix D.

(g) Design review (DR)

Briefly explained, a Design Review is a systematic, independent review of the design and the way it is documented. In general, Design Reviews should at least be carried out between the main activities or phases of the project (see Fig. 6.2). The general aim is to optimalize the design solutions. The reviews should be carried out by groups of persons who have the relevant technical background and experience, among other things, in order to obtain synergetic effects and make the most of the available competence.

For further description of Design Review, see Appendix E.

(h) Hazard and operability studies—HAZOP

A HAZOP is a formal, systematic and critical review of different parts of a system, design, plant or structure in order to identify potential problems regarding safety and operability, so that risk-reducing actions can be implemented. The analysis could be a complete risk analysis or a pre-study for more detailed studies of critical parts of a structure. HAZOP can be carried out both during pre-engineering and detail design. The analyses could also be carried out during operations, e.g. in connection with maintenance or modifications of the design or the operational procedures. For further description, see Appendix F.

(i) Worst-case analysis

A Worst-case analysis is a systematic analysis and evaluation of the consequence of the worst possible input data, occurrences and combinations of occurrences, to personnel, the environment and assets.

The purposes of a Worst-Case Analysis are two-sided (for further description, see Appendix G):

- To verify that safety and other important functions are maintained under abnormal loads, foreseeable abuse and abnormal human stress.
- To ensure that decisions on *not* designing and dimensioning for such extreme conditions are made on the right (i.e. high enough) level in the organization.

(j) Quality audit/Quality system audit

A Quality (System) Audit is, simply put, a systematic and independent review or investigation to ensure that the Quality System is properly designed and implemented. Audits should be carried out according to written procedures.

Quality Audits have been greatly emphasised as an important management tool during the last decades. Requirements have been made to ensure that the audits are carried out both in NPD's Internal Regulations and in ISO 9001. However, it can be argued that practice within this area has developed in a somewhat unfortunate direction. Especially within offshore applications, attention has been drawn primarily to audits of written quality system descriptions and procedures. The question could be raised, whether this may have been at the sacrifice of attention on the quality of the actual product.

For further description of Quality Audits, see Appendix H.

6.3.3 Quality Assurance elements related to the main activities in engineering and design

Fig. 6.2 gives a general view of the main activities and documents in the engineering and design phase.

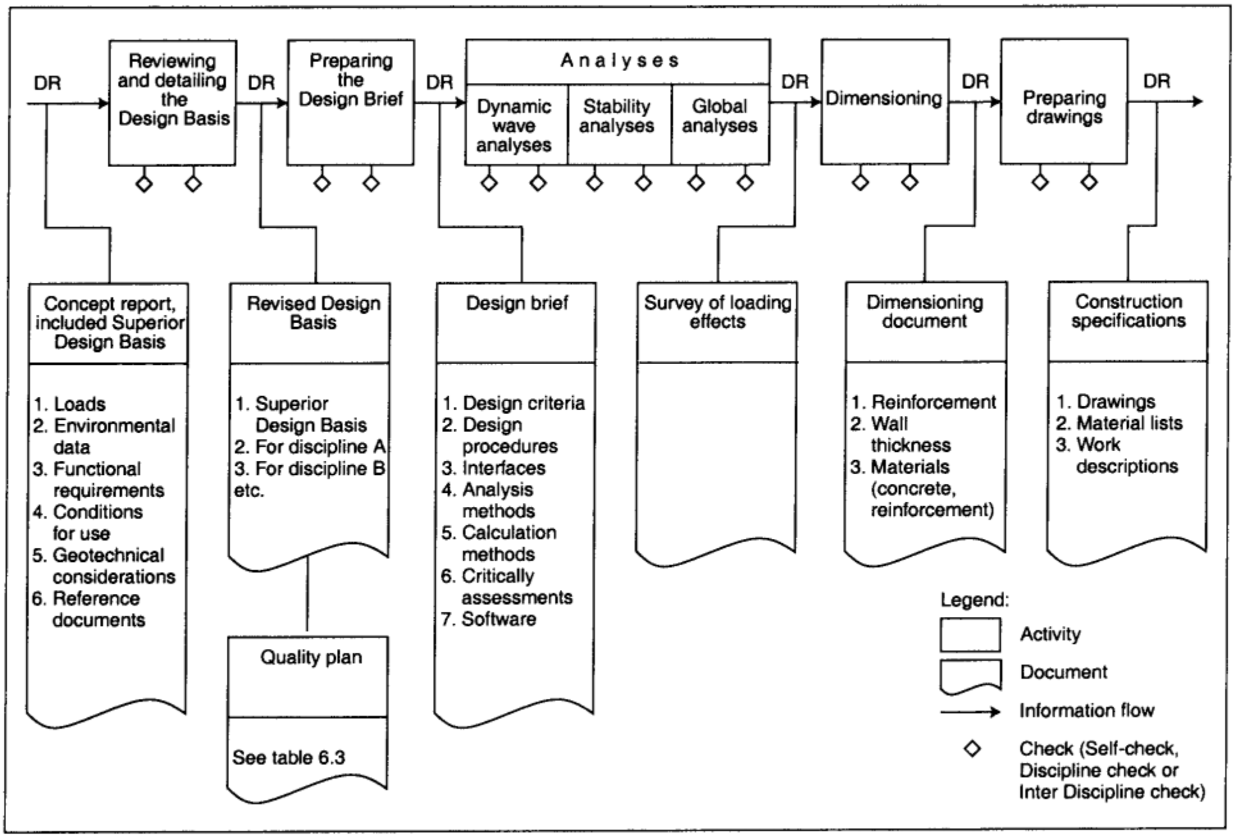


Fig. 6.2 Main activities and documents in engineering and design of marine concrete structures (DR=Design Review).

The first activity is Review and detailing of Design Basis (Fig. 6.2). This is done on the basis of the Concept Report from the previous phase (Concept Defining). The Concept Report contains the superior Design Basis with separate chapters regarding loads, environmental data, functional requirements, conditions for use, geotechnical conditions and reference documents. During this stage, a further analysis of the conditions for the structure, and an in-depth review of the consequences for the different disciplines, are carried out.

A general requirement to the results of all the activities during design, is that the documents must be correct and attend to all necessary regards to clarity, unambiguity and suitability for further use, including traceability and verifiability. This means that the configuration control and the document control must be satisfactory, as well. In relation to this, one should also realise that the iterative character of the engineering and design process makes it very important to be in total control of all changes that are made on the way. "Total control" here means that all consequences of the proposed changes should be considered thoroughly and evaluated at the right organizational level before they are implemented. This implies among other things that the person(s), who worked out the original requirement or the previous solution, should have an opportunity to assess the consequences of the proposed changes, before they are accepted and implemented.

When a change has been accepted, everyone who is in position of the original documents must be informed of the change at once.

During the review and detailing of the Design Basis for the different disciplines, Self-Checks, Discipline Checks, and Inter Discipline Checks should be carried out. This is indicated with small parallelograms in Fig. 6.2.

A (Project) Quality Plan should be prepared at the same time as the review and detailing of the Design Basis (see Fig. 6.2). The Quality Plan describes the particular quality assurance activities that are to be carried out in the project to satisfy the requirements, and what resources should be used for this. Table 6.3 gives an example of the layout of a Quality Plan for the engineering and design phase.

The Quality Plan should be rooted in the operator's Quality System and be co-ordinated with or constitute part of the Project Plan. The Quality Plan for the complete project should include the following elements as well (ref.: *ISO 9004–1:1994,* item 5.3.3 and *ISO 9004–5 Guidelines for quality plans):*

- Description of aims for quality
- Distribution of responsibility and authority in different project phases
- Procedures, methods and work instructions that are to be employed

Table 6.3. General layout of a Quality Plan during engineering and design (example)

Method Activity	Discipline Check (DC) 1. Document Review 2. Extended Document Review 3. Independent Calculations 4. Scale Test	Inter Discipline Check (IDC)	Third Party Verification 1. Document Review 2. Extended Document Review 3. Independent Calculations 4. Scale Test	Design Review	HAZOP	Worst-case Analysis	Quality Audit
Reviewing and detailing Design Basis • Risk Analysis							
Preparing Design Brief							
Analyses • Dynamic Wave Analysis • Stability Analysis • Global Analysis							
Dimensioning							
Preparing Drawings							
Testing • Laboratory testing • Full scale testing							

- Programs for testing, reviews, inspections and audits in the different phases
- System for change and modification of the Quality Plan as the project proceeds
- Other initiatives that are to be carried out in order to reach the goals.

The Design Brief (see Fig. 6.2) is prepared on the basis of the revised Design Basis. The Design Brief lays down the Design Criteria further, together with the design procedures, important interfaces and preferred methods for analysis and calculations. Other important parts are the results of criticality evaluations for the individual parts of the structure, and a specification of the software programs accepted for use.

During development of the Design Brief, the same quality elements as in the treatment of the Design Basis should be employed, that means *Self-Check, Discipline Check, Inter Discipline Check* and *Design Review.* Since the extent of the documents is steadily increasing as the project progresses, it is evident that the checks require more and more resources. To some extent, different kinds of competence are needed, too.

The analyses are carried out on the basis of the Design Brief. They consist of Dynamic Wave Analysis, Stability Analysis and Global Analysis.

Dynamic Wave Analysis results in dimensioning waves for different parts of the structure, hydrodynamic loads, masses and damping. The results are used as basis for the Global Analysis.

Stability Analysis gives an answer to whether the structure has the necessary stability against capsizing during the construction process, submerging, tow-out, installation, operation and service. Both floating stability and geotechnical stability has to be assessed.

Global Analysis gives an answer to what kind of strain the different parts of the structure is exposed to during submerging, tow-out, installation and operation.

The three above-mentioned analyses together make up the basis for the later dimensioning, and are therefore critical for the structure's safety and fitness for use. Related to Figs. 6.1 and 6.2, there is a need for a more comprehensive quality assurance than previously, in this phase of the project. In addition to Self-Check, Discipline Check and possibly Inter Discipline Check, *Third Party Verifications* would therefore be needed and hence required by the authorities.

The dimensioning is carried out on the basis of a general view of the load effects. During dimensioning, the length and location of the reinforcement, the final wall thickness and the quality of the concrete and the reinforcement are determined.

The result of the dimensioning is entered in the *Dimensioning Document.* This should be subject to *Self-Check, Discipline Check* and possibly *Design Review* before the drawing work starts.

The drawing work will result in drawings, material lists and descriptions that are necessary for the construction to start (see Fig. 6.2). All sketches and specifications should be subject to *Self-Check* and suitable independent verification. A *Design Review ("Engineering Readiness Review")* should be completed before release for construction and fabrication.

7 Verification of design

Tore H.Søreide, Reinertsen Engineering

7.1 Introduction

The purpose of this description of verification is to present a procedure for control for detailed design of offshore structures. For exemplification, Chapter 7 refers to government regulations from the Norwegian Petroleum Directorate (NPD, 1997) as well as company specifications from the Norwegian oil company Statoil (Statoil, 1991). Both documents require design verification to be part of the quality assurance. The major activities of verification are outlined below, together with the basis for verification in the form of authority and company specifications.

Chapter 6 has presented the overall system for quality assurance, where design verification is an integrated part. The objective of verification is to guarantee that the final product is in accordance with the Government regulations and Company specifications. For engineering, the outcome is in the form of drawings and specifications for fabrication.

The NPD regulations (NPD, 1997) give detailed requirements concerning verification. Section 7.2 discusses these requirements as well as the corresponding verification activities. Depending on their importance for the quality and safety of the final structure, the various documents need different types of control. Section 7.3 presents four levels of verification that involve document control as well as prototype testing.

Section 7.4 considers a system for external verification. Special emphasis is given to the view that the verification should ideally also be a support for engineering, so that time coordination becomes important. Alternative means of organizing verification are presented in Section 7.5, while Section 7.6 considers administrative services coupled to the work on verification. These services include budgeting and a set-up for reporting and communication between the engineering and verification bodies on unsolved topics.

Qualification requirements for the engineers participating in the verification work are dealt with in Section 7.7. Special effort should be made to guarantee that there are particularly competent people in the technical lead position within the verification activity. An essential function of the technical leader is to sort out the major technical questions for follow-up. Section 7.8 exemplifies the scope of work involved and how the verification job may be planned. Fig. 7.1 illustrates the importance of co-ordination in time between engineering and verification. It is clear that for the verification to affect design prior to fabrication, the comments from verification should reach engineering within the timeframe when the problem is being addressed. If not, it is often difficult for engineering to react to the comments from verification, and a negative atmosphere of dialogue arises.

7.2 Norwegian Petroleum Directorate requirements

The NPD regulations on loadbearing structures (NPD, 1997) formally states the independence between the engineering and verification activities within the same project, emphasizing a third-party verification. The operator is to consider the amount of verification that is related to the critical aspects of the structural part under engineering.

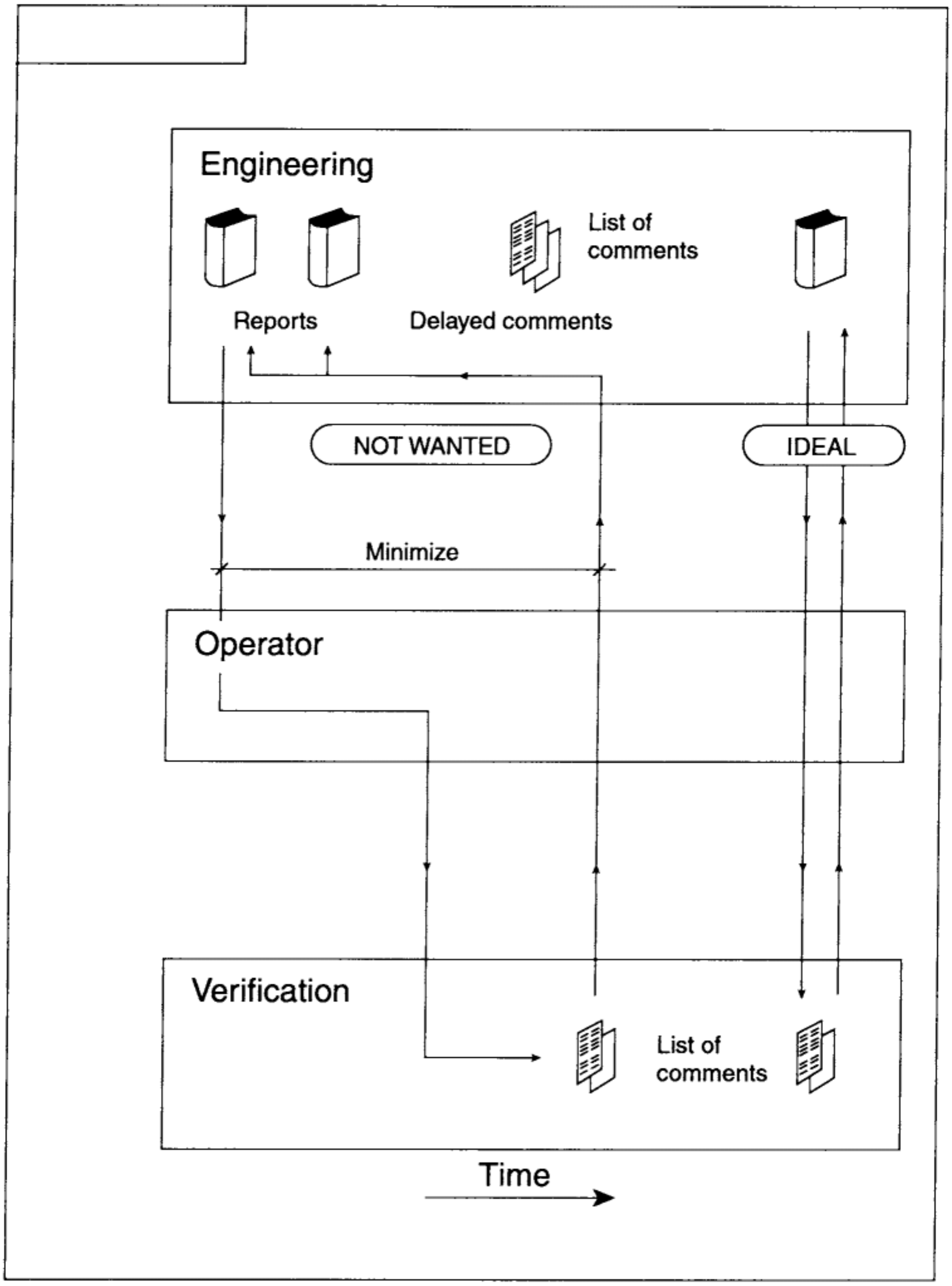


Fig. 7.1 Time delay from engineering to verification

NPD provides a detailed list of verification activities. These are areas of verification that all engineering on offshore loadbearing structures should undergo. Among the essential areas listed are:

- Design Basis in accordance with guidelines
- Qualifications by personnel in engineering
- Organization of engineering
- Documentation and testing of computer programs
- Load modelling
- Response analysis
- Capacity control
- Tolerances in design
- Drawings in accordance with design calculations
- Design and forming of details.

For parts of the structure that are essential for its integrity, verification should be carried out as an independent analysis. In general, NPD states that verification may be in the form of a combination of document review and analysis. The term "verification" is to cover all types of control, document review as well as independent analyses.

Requirements on the qualifications of personnel depend on their function in the engineering team. The requirements differ from the technical supervisor down to the design engineer. It is essential that the lead personnel have held positions of project experience, from which insight into behaviour of structures is gained. Further, the scheme of engineering activities must be uniquely defined, and experience as well as basic theoretical knowledge within structural mechanics is necessary to ensure that the model is representative of the real structure. Among critical areas in the present design of offshore structures are the modelling for stress analysis and the selection of relevant design wave situations for different areas of the structure. Special problems also arise in the postprocessing phase, related to design of heavily loaded areas.

Documentation and testing of analysis programs should be related to the planned use of the program. The purpose of such control is twofold, in the sense that the program as well as the user shall be tested. The verification of computer programs includes problem relevant examples to be analysed, as well as documentation of the correctness of the results. The user of the program is to perform this verification. In parallel, a description is to be implemented in the Design Brief document, in which the major analysis activities to come up in engineering are outlined, including the scope of work and budget. For the case of non-linear analysis, special attention is to be given to personnel qualifications and program verification.

The engineering activities on load modelling, response analysis and capacity control constitute the major design calculations. The load modelling discipline brings the Design Basis specifications into the analysis. Response analysis provides the stresses and includes static as well as dynamic analyses, see also Chapter 3. Classification of the loads, as related to the response characteristics of the structure, is vital for a correct scheme of response analysis.

The activity related to capacity control implies combining section forces from separate load effects up to design load situations, as a basis for dimension control.

It is clear that the series of load modelling, response analysis and capacity control is the vital basis for the later fabrication drawings, and as such all three activities should be subject to a high level of verification, primarily independent analysis.

The NPD requirement (NPD, 1997) regarding tolerances and inaccuracies as part of verification, relates to the check that the choice of material safety factor in design is in accordance with the Design Basis specifications, as well as with the system of control during fabrication. For critical areas with complex geometry, the verification of capacity should include a sensitivity study, in which upper limits for deviations are considered, even beyond the tolerances specified. Non-linear analysis is often an alternative to linear elastic analyses for complex stress flows, representing a more realistic simulation of stress redistribution.

The verification of fabrication drawings is to pay full attention to the control so that the drawn structure is in accordance with the input and the results of the engineering calculations. Major elements for control are the dimensions given, as well as reinforcement amounts and locations. The amount of pretensioning specified for fabrication should also be checked against the corresponding load modelling.

The term “D-regions” (D for Discontinuity) is used to locate areas for design that the global analysis model does not cover, and where special calculations of capacity are needed. The verification of these areas either is made by a local finite element model, or, in some cases, where ultimate capacity is to be verified, by a strut and tie model. The verification of D-regions puts special requirements on the personnel. Experience in practical design may be needed to be able to determine the flow of forces in and out of the detail in question.

Section 7.8 gives some proposals on verification activities that follow from the NPD regulations. However, the complete set of verification activities depends on the problem faced, and, thus, a verification plan should be initiated at the start of all engineering projects.

7.3 Levels of verification

7.3.1 Choice of levels

As discussed above, the level of verification depends on how critical the actual engineering activity is. As a general rule, the major analysis elements are given top priority. It is convenient to classify the verification into the following four levels:

Level 1: Document Review
Level 2: Extended Document Review
Level 3: Independent Analysis
Level 4: Scaled Model Tests

A description of objectives and content is given below for the above four classes of verification.

7.3.2 Level 1: Document review

The verification by document review is the simplest type of control, and as such implies a critical evaluation of the document from engineering according to project specific checklists, and reporting on questions that arise. A general rule is that the relevance of the analysis model is checked, together with input for analysis and values of load effects from the analysis and corresponding strength parameters. The evaluation of the analysis model is to be based upon separate considerations concerning load-carrying behaviour, including possible dynamic effects.

The process of document review is at least to be applied for all basic documents in analysis and design procedures (Design Basis and Design Brief). For these documents, the document review level 1 of control is made even for documents that are very critical.

7.3.3 Level 2: Extended document review

The extended document review form of verification implies level 1 control of a document, supplemented by a check of calculations. The control is still related to a specific document from engineering, however, essential calculations are checked either by simplified hand techniques or by local finite element models.

All control calculations are to be stored together with the document. They should be clear and easy to follow in the case of later technical discussions. Normally, the level 2 control calculations are not reported separately.

Level 2 control is to be made for all major documents on analysis and design, that are not covered by independent analyses in design.

7.3.4 Level 3: Independent analysis

The verification by independent analysis is a completely separate analysis of the total structural system, or alternatively parts of it. It is a general rule in practice that independent analyses are made by a different program system than the one applied in engineering.

The different runs on independent analyses are to be co-ordinated, so that in total they cover the main activities of analysis and design. The independent verification analyses are to be reported by separate verification documents, and comparisons with engineering results are to be included.

7.3.5 Level 4: Scaled model test

The experimental verification by scaled model tests normally deals with critical details in the structure, in most cases also in reduced scale. The objective is to verify analysis models on capacity, or to check the feasibility of fabrication. Model tests are normally supplements to the verification of calculations, as described in previous chapters. Strict requirements are to be put on planning, execution and evaluation of such model tests, especially on the effects of scaling. Test laboratories thus normally should carry out this type of verification.

7.3.6 Choice of verification level

An activity plan for the verification is to be made, as well as scope of work for each activity. The extent of control, together with computer programs for control calculations are outlined here, also see Sections 7.6 and 7.8.

In general, the following choice of verification levels should be made:

Level 1: Design Basis document
Design Brief document

	Loads
	Criteria for capacity
Level 2:	Secondary calculations
	Internal verification
	Finite element models
	Local capacity controls
	Interface analysis/capacity control
Level 3:	Load models
	Design waves
	Global stress analysis
	Load combination
	Capacity control
Level 4:	Global response (wind and waves)
	Capacity of vital details
	Fabrication feasibility of details in full scale (reinforcement).

7.4 External verification

7.4.1 Alternatives in external verification

The present section deals with external (third party) verification of detail design, the objective being to phase the verification work into the project organization. In addition, a set-up for third-party verification is outlined that enables engineering to profit from the verification work.

Alternative 1: Operator controlled verification
Fig. 7.2 illustrates the organization of the design project, including one box for engineering, one for operator and finally a separate box for third-party verification. The solid lines show the usual progression, in which all communication to third-party verification and back goes via the operator. The system proposed also puts special requirements on the communication between operator and engineering, as well as from operator to third party verification.

The above set-up for engineering control is based upon the system control by the operator, an activity that makes the need for a technical team by the operator. The third-party verificator together with the operator now comes out as one unit in discussion with engineering. The operator also sorts out the engineering activities for third-party verification, and decides the extent of verification. In practice, the verificator comes up with a budget for the work, which is then the subject for discussions with the operator.

The lists of comments as well as the checklists from third-party verification are sent to the operator for evaluation and possible completion. The final comments to engineering are made by the operator. The operator also runs the subsequent technical discussions with engineering, as well as follow-up of non-conformances.

It emerges that the above organization of verification is both time consuming and expensive.

The operator gets a key role in the clarification effort on comments from verification. The heavy involvement of the operator in the technical discussions between engineering and verification also represents a large risk for time delays, and the gains from verification to engineering may disappear.

Alternative 2: Direct communication between engineering and third party verification
An alternative system for verification of engineering is illustrated in Fig. 7.3, involving direct communication between engineering and verification. The formal responsibility is still by the operator, as well as the task of discipline co-ordination. The main difference from the scheme in Fig. 7.2 is that direct technical discussions now take place between engineering and verification.

7.4.2 Choice of alternative

The criteria for the selection of system for third-party verification are as follows:
Criterion 1: Technical qualifications
The sum of experience and theoretical basis by the operator and the third-party verificator for alternative 1, is to be compared to the qualifications by the verificator for alternative 2. The main emphasis is laid upon key persons for the verification, their formal education as well as experience from relevant projects.
Criterion 2: Plan for third-party verification
A scope of work for verification is to be set out, including an activity plan and time schedule for reporting back to engineering. In the selection of company for third-party verification, the above activity description should be a major criterion.
Criterion 3: Cost
Based on an estimate for man-hours a budget is to be made for the verification work. The operator may choose a split contract based on separate activities, or a type of framework agreement, with an upper limit on costs.

As seen from the above alternatives, the type and size of project are quite often decisive for the choice of system for verification. In the case of operator supplement to third party verification, the alternative in Fig. 7.3 should be chosen.

For both alternatives 1 and 2, the verificator is to report to the operator the work done. In the case of alternative 2, lists of non-conformances are to be sent in parallel to engineering and to the operator, and the operator may supplement this by comments.

7.5 Internal verification

The internal verification of results from engineering is to be included in the plan for verification, including personnel, level of control as well as technical subject for control.

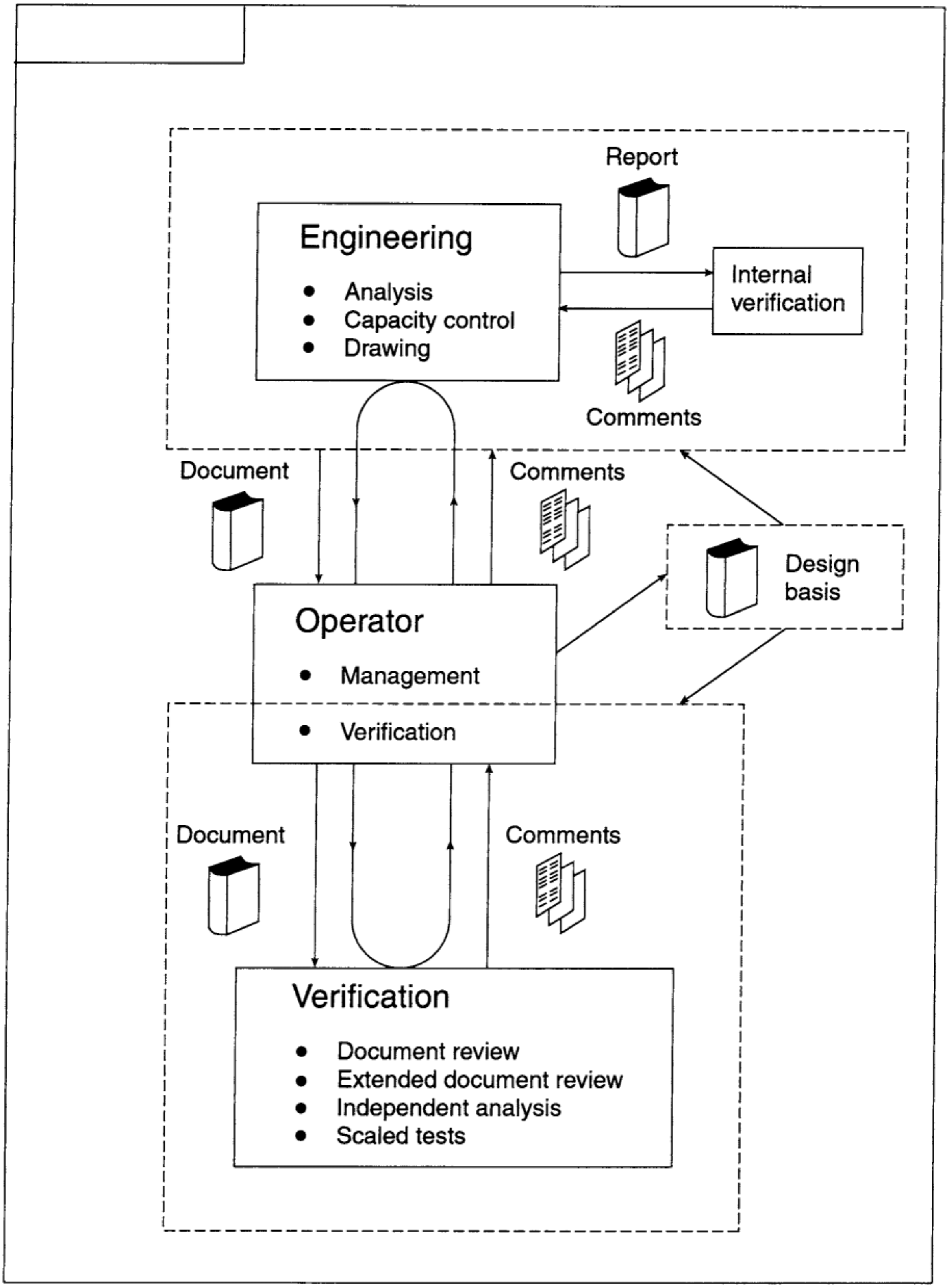


Fig. 7.2 Alternative 1: Operator controlled verification

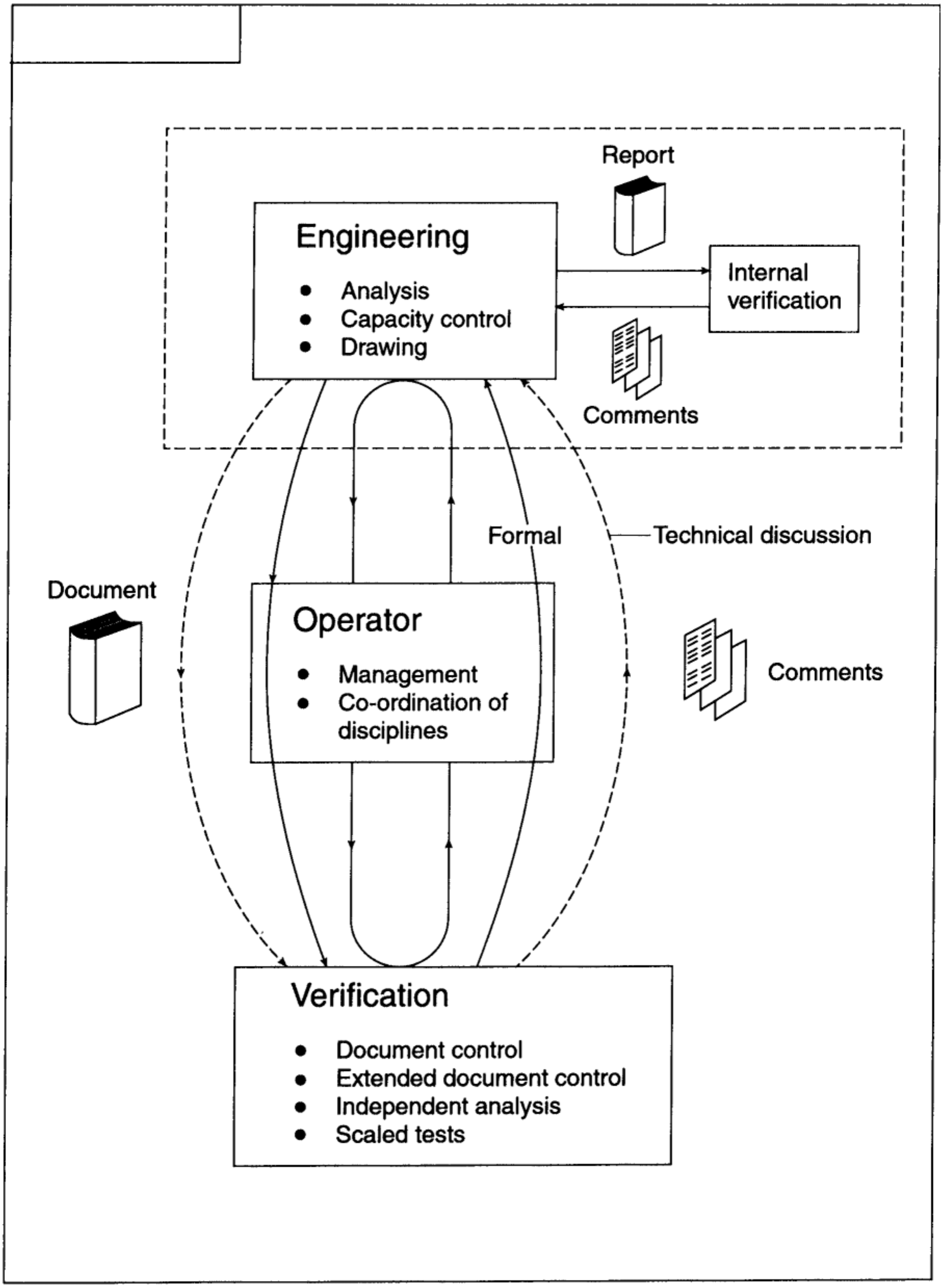


Fig. 7.3 Alternative 2: Direct communication between engineering and third party verification

Figs. 7.2 and 7.3 each shows a box for internal verification in engineering. The plan for verification should co-ordinate this activity with the overall set-up for the third-party verification. The most common system is to have a separate group in engineering, that functions like a group for third-party verification. All comments and the follow-up of comments from internal verification are then to be documented.

The alternative is that well-qualified personnel come into the engineering team with their major effort on control. Communication to engineering is easier, and internal verification in engineering can now also become a daily support for the design work. It is, however, still essential to keep the formal requirements intact, such as reporting of comments and documentation of the follow-up, even though the personnel doing the internal verification are in close contact with engineering.

Both the above alternatives involve the internal verification being an independent activity from engineering, so that the personnel for verification themselves do not have responsibility for design work.

7.6 Budgeting, reporting and follow-up of non-conformances

7.6.1 Budgeting

This section presents a system for the budgeting of the third-party verification activities. It also describes forms of reporting by verification as well as follow-up of non-conformances.

Prior to the start of verification, a detailed plan is to be made, taking into account the control activities in verification as well as the treatment of non-conformances.

For the case of a split contract for third-party verification, or alternatively with a framework agreement, a budget for the verification work is to be made. The budget schemes contain the scope of work for each verification activity, together with planned man-hours and personnel.

A similar set-up should also be made for planning additional work during verification, either as an extension of existing activities, or by making a new scope of work.

Any exceedance of the budget has to be reported immediately and be subject to verification. Possible modifications of scope of work, so as to reduce the costs, need acceptance by the operator.

7.6.2 Reporting of non-conformances

The company responsible for verification is to produce a system for reporting on comments and non-conformances, covering all four levels of verification. As far as possible, effort should be made to keep the same form of reporting for level 1 and level 2 controls. The document on non-conformances should have a standardized front page, see Fig. 7.4, together with a list for reporting and follow-up comments, Fig. 7.5.

The report on non-conformances is to be signed by the person with technical responsibility for the verification work. A checklist should follow each document from engineering, and produce the major technical issues in the report. The engineer doing the verification work, should pay special attention to the checklist, and finally make their own evaluation of the completion of the different technical subjects.

The comments from verification are to follow the document. In Fig. 7.5 the list from verification contains the reference to engineering document, numbering of non-conformances as well as a technical description of each comment, possibly also supplemented by sketches. A column is also included for reporting on the follow-up, in which a signature confirms and dates the verification of the corrective action, when agreement between engineering and verification is obtained. Each list on non-conformances is to be checked and signed by the technical supervisor for verification.

CLIENT: **PROJECT TITLE** **VERIFICATION OF ENGINEERING**
VERIFICATION REPORT
DETAIL ENGINEERING
Verification report for:
Activity no.:
Date: **Sign. originator/Lead engineer**
Date: **Sign. Project manager**

Fig. 7.4 Front page on non-conformance report

CLIENT:

PROJECT TITLE

NON-CONFORMANCE HANDLING

Page:
Per date:
Sign:

DOCUMENT

No.	Non-conformance	Non-conf. reported date:	Corrective action performed:	Corrective action date:	Verified date Sign:

Fig. 7.5 List of non-conformances

7.7 Requirements concerning qualifications

7.7.1 Documentation of qualifications

Necessary qualifications are to be documented for the persons making the verification, in the form of formal education as well as relevant experience.

7.7.2 Requirements for technical leader for verification

A technical responsible person is to be identified by the third-party verificator. The primary function of the technical leader is to evaluate all comments from verification to engineering, assuring that relevant comments are forwarded. A wide theoretical basis is necessary to cover global behavioural effects on the structure, as well as local problem areas. As a minimum, ten years of experience in structural design should also be required in order to lead the verification of larger projects.

The function of the technical leader is to evaluate the technical activities presented for verification and distribute these among the engineers for control. To follow up the control work, knowledge is also needed concerning special analyses on load effects, response analysis and design. The technical leader for verification is to check all comments that arise during the control work and sort out the essential non-conformances that are to be forwarded. The comments also ought to be ranked with respect to critical aspects, making the discussion with engineering rational.

At certain milestones, the technical leader for verification is to report on the status of progress, highlighting on the critical items that are still not cleared from the list of non-conformances. Evaluations are then to be made whether the items still remaining require a redesign and thereby a stop in fabrication.

The technical leader represents the verification in all meetings with engineering and the operator. Qualifications in technical presentations, also in foreign languages, are a requirement in larger projects.

7.7.3 Requirements for engineers

For the verification engineer in general, good basis within structural mechanics is necessary, including theory concerning load effects, stress analysis as well as capacity control. It is essential that tasks for the verification engineers are related to their qualifications. It is the responsibility of the technical leader for verification to let the engineers have the relevant documents for control.

In the case of level 3 control by independent analysis, experience in the use of computer programs is required. Further, structure behaviour knowledge is needed for making self control of the level 3 analysis results, prior to reporting on possible non-conformances.

7.8 Scope of verification activity

7.8.1 Planning of verification

The present section outlines the plans for some major activities within verification, the objective being to come up with major issues in the verification of documents from engineering. It is, however, to be emphasized that the activity schemes below do not represent the complete set of verification activities for a structure design project.

7.8.2 Verification of global analysis model

The control of the global model for stress analysis has its main objective to verify that the finite element mesh represents the load carrying behaviour of the structure. The verification is also to identify areas where the model is not representative for the real structure behaviour and where extra effort on design needs to be taken.

The control of the model follows the level 2 scheme of verification. The document on global model by engineering is to have plots on section stress resultants and verification ought to check these with simplified techniques. Tests are to be made on the accuracy of the model in areas with high degrees of element distortions, ending up in a documentation of the feasibility of the analysis model to handle certain complicated areas.

As a supplement to the document control of the global analysis model, an independent analysis of stress results is to be performed by verification, applying an independent computer program. The modelling should include an evaluation of areas where separate analyses are needed (D-regions).

The independent analysis model is related to the model from engineering, together with comparisons on stresses for design. A separate document is made on the verification of analysis model.

7.8.3 Verification of design waves

The main objective of the present design wave verification is to assure that the waves for design calculations are representative for the load effects and the response characteristics of the structure.

The engineering evaluation of design waves is checked, involving the hydrodynamic model, unit amplitude waves, stochastic response analysis with extreme value estimates, as well as choice of wave directions and range of wave periods.

For the hydrodynamic model, panel sizes are related to the shortest wave. The selection of wave periods is related to the Design Basis document on environmental data, and the natural periods of the structure system.

The models for stress analysis are considered with major effort placed on element types that are related to geometry, stiffness, mass and damping.

For the global rigid-body model on hydrodynamics, effort is to be placed on the control of wave directions and periods. The wavelengths should also be related to the geometry of the structure. Critical waves for essential load effects are sorted out by simplified frame models.

For the stochastic analysis, the choice of wave input parameters is considered in parallel with the Design Basis document and the NPD regulations (NPD, 1997). Extreme value estimates from the stochastic analysis are compared with the hand-calculated responses, see Chapter 3. In general, the design wave period should be close to the predefined critical sea state peak period. Scaling of the amplitude of the design wave should also be given special attention, The document control of verification, as described above, is a level 2 control with simple supplementary calculations. Special attention is to be placed on the selection of design waves, as these are fictive regular waves, representing the critical sea states for different responses in the structure.

7.8.4 Verification of loads in global model

The purpose of the present activity on load modelling verification is to check the application of unit load cases, together with the scaling of responses into design combinations.

The basic load cases in the engineering analysis model are related to the different stages of construction, installation and operation. Wave loading, together with current, wave drift and wind are checked against the Design Basis document, supplemented by estimates on forces. For the dynamic effects from waves, special care ought to be taken for the modelling of inertia forces due to rigid-body motion of the structure.

Load effects from pretensioning are to be checked against the actual routing of cables. Evaluation is also to be made on possible modifications of the pretensioning system during fabrication in heavily loaded areas to cover possible redesign.

The independent global model by verification also includes the load modelling referred to above in Section 7.8.2.

7.8.5 Verification of design loads combinations

The activity on verification of design load combinations is essential in the way that it covers the latest stage of calculations before fabrication drawings. Time co-ordination is important for the present verification activity.

For different areas of the structure, design load combinations are taken out of the automatic process of capacity control. Plots of section forces are part of this information from engineering. Then there are also stress resultants in the form of shell forces as well as global stress resultants for columns and shafts. Often, verification is given direct access to the engineering files.

Based upon critical load combinations, estimates of stress resultants are made by simplified calculations. The structure system depends on the stage of fabrication, installation or operation.

For cases where first order waves come into a design load combination, relevance to Section 7.8.3 on design wave selection is appropriate. However, in combination with other load types, correlation between the different load effects is to be taken into account, and thus the critical single wave does not necessarily come out as the worst wave for the combined load effect.

The selection of design waves is based upon scaling of regular wave responses to fit the extremes of a stochastic analysis. The process of design wave generation is made for a certain number of structure areas. The control of design waves from engineering should also include a check of the correct design wave being applied for the structure part for which it is valid. Based on the independent analysis control of stress resultants, a separate verification is made of capacity and reinforcement amounts.

References

Norwegian Petroleum Directorate (NPD), (1997) *Acts, Regulations and Provisions for the Petroleum Activity.* Norwegian Petroleum Directorate, Stavanger, Norway.

Statoil (1991) *Structural Design for Offshore Installations, Specification for Design.* N-SD-001, Statoil, Stavanger, Norway.

Appendices

Appendix A Discipline Activity Model

Fixed platforms—Concrete substructures checklists

The objective of Appendix A is to present possible checklists for design of offshore concrete substructures. Such checklists could be useful tools for the practical design to avoid important steps being missed out during the different stages of the design process.

The following key relates to checklists A1 through A5:

Checklist Key		
√	-	Required at the stage indicated.
[√]	-	Optional item which may be required depending upon field development type.
P	-	Preliminary design data which must be estimated or assumed, based on experience, to provide a suitable basis for the work.
i	-	Significant input to project documents.
←	-	Input from other disciplines (as identified).
→	-	Output to other disciplines (as identified).

A1. Fixed platform—Concrete substructure design data checklist (3 pages)

Field Development Phase			**1**		**2**		**3**
	Study Stage Description:		Prospect Evaluation	Field Evaluation	Feasibility Study	Field Development	Concept Definition
	Estimate Class Required:		Class A (O of M Cost)[5]	Class B (± 40%)	Class C (± 30%)	Class D (± 20%)	Class E (± 15%)
Design Data (Design Basis)	Inter-Discipline						
1. Waterdepth	←	Marine Technology					
• datum			√	√	√	√	√
• tolerances					[√]	√	√
2. Environmental data (in place)	←	Marine Technology					
• waves (storm)				P	P/[√]	√	√
• waves (fatigue)					P/[√]	√	√
• wind				P	P/[√]	√	√
• current				P	P/[√]	√	√
• marine growth					P/[√]	√	√
• earthquake							
3. Environmental data (Temporary condition)	←	Marine Technology					
• waves					P	√	√
• current					P	√	√
• wind					P	√	√
4. Geotechnical data	←	Geotechnical					
• seabed survey				P	P/[√]	P/[√]	√
• soil report				P	P/[√]	[√]	√
• seismic data					P	P/[√]	√

[5] Order of Magnitude

A1. Fixed platform—Concrete substructure design data checklist (3 pages)

Field Development Phase:			**1**		**2**		**3**
	Study Stage Description:		Prospect Evaluation	Field Evaluation	Feasibility Study	Field Development	Concept Definition
	Estimate Class Required:		Class A (O of M Cost)	Class B (± 40%)	Class C (± 30%)	Class D (± 20%)	Class E (± 15%)
Design Data (Design Basis)	Inter-Discipline						
5. Topsides	↔	Topsides					
• size and configuration					√	√	√
• orientation					√	√	√
• weight			P	P	√	√	√
• uncertainties in weight est.					[√]	√	√
• support points and stability elevations					[√]	√	√
6. Risers, J-tubes and pipelines							
• number and size	←	Projects	P	P	√	√	√
• location	←	Topsides			P/[√]	√	√
• termination points	↔	Topsides				√	√
7. Caissons							
• number and size	↔	Process/ Mechanical	P	P	√	√	√
• location	↔	Topsides			P/[√]	√	√
• lower cut-off elevation	↔	Process/ Mechanical			P/[√]	√	√
8. Conductors	↔	Reservoirs/ Projects/ Operations					
• number and diameter			P	P	√	√	√
• spacing					P/[√]	√	√
• location/array					P/[√]	√	√
• cooling						[√]	√

A1. Fixed platform—Concrete substructure design data checklist (3 pages)

Field Development Phase:			**1**		**2**		**3**
	Study Stage Description:		Prospect Evaluation	Field Evaluation	Feasibility Study	Field Development	Concept Definition
	Estimate Class Required:		Class A (O of M Cost)	Class B (± 40%)	Class C (± 30%)	Class D (± 20%)	Class E (± 15%)
Design Data (Design Basis)	Inter-Discipline						
9. Safety	←	Safety/ Topsides					
• Ship Impact loads/energy					P/[√]	√	√
• Shaft Requirements							
• Access to sea							√
10. Drilling Template (option)	↔	Subsea					
• Number of wells				P	√	√	√
11. Field layout	←	Projects/ Pipeline Topsides					
• Neighbouring installations					[√]	√	√
• Pipeline locations					√	√	√
12. Materials	←	Materials					
• Size/grade limitations				√	√	√	√
• Reinforcement principles						[√]	√
• Concrete reinforcement requirements							√
• The platform's mechanical equipment						√	√
13. Miscellaneous	←	Trans-port Process					
• Bulk storage requirements			P	P	P/[√]	√	√
• Oil storage requirements			P	P	P/[√]	√	√
14. Facility Design Life	←	Operations		P	P/[√]	√	√
15. Mechanical Equipment							
• Ballast water						√	√
• Seawater intake						√	√

A2. Fixed platform—Concrete substructure activity checklist (2 pages)

Field Development Phase:			**1**		**2**		**3**
	Study Stage Description:		Prospect Evaluation	Field Evaluation	Feasibility Study	Field Development	Concept Definition
Activity	Estimate Class Required: Inter-Discipline		Class A (O of M Cost)	Class B (± 40%)	Class C (± 30%)	Class D (± 20%)	Class E (± 15%)
1. Produce design premise/ briefs					[√]	[√]	√
2. Review of existing similar developments			√	√	√	√	√
3. Selection of structure/ installation philosophy			√	√	√	√	√
4. Configuration studies	←	Topsides					
• Overall dimensions			[√]	√	√	√	√
• Number of shafts			[√]	√	√	√	√
• Number of storage cells			[√]	√	√	√	√
• Shaft configuration			[√]	√	√	√	√
• Mechanical equipment				[√]	√	√	√
5. Preliminary sizing				[√]	√		
6. Foundation studies				[√]	√	√	√
• Stability					√	√	√
• Settlements						√	√
7. Detailed geometry							√
8. In-place analysis							
• Nat. Freq./mode stages						√	√
• ULS (100 year wave)					[√]	√	√
• FLS (Fatigue)					[√]	√	√
• SLS (Deflectrans)						[√]	√
• Modeltests						[√]	√
9. Installation studies							
• Deck transfer					√	√	√
• Tow to site					√	√	√
• Ballasting					√	√	√
• Modeltests							[√]

A2. Fixed platform—Concrete substructure activity checklist (2 pages)

Field Development Phase:			**1**		**2**		**3**
	Study Stage Description:		Prospect Evaluation	Field Evaluation	Feasibility Study	Field Development	Concept Definition
	Estimate Class Required:		Class A (O of M Cost)	Class B (± 40%)	Class C (± 30%)	Class D (± 20%)	Class E (± 15%)
Activity	Inter-Discipline						
10. PLS analysis (Accidental) • Ship collision • Extreme wave • Earthquake • Dropped objects • Fire • Progressive collapse						 [√] [√] [√] [√] [√] [√]	 √ √ √ √ √ √
11. Prepare drawings				√	√	√	√
12. Design of typical details and appurtenances						[√]	√
13. Overall Weight Estimate			√	√	[√]		
14. Weight Take Off (incl. foundations) • Volume Estimate					[√]	√	√
15. Prepare Material Take Off for Long Lead Items							[√]
16. Prepare Input to report			√	√	√	√	√
17. Topsides/Substructure Interface Study					[√]	√	√
18. Fabrication yards • Limitations of location • Yard limitations					 [√] √	 √ √	 √ √
19. Installation • Installation period				 √	 √	 √	 √
20. Removal				√	√	√	√
21. Project plans				[√]	√	√	√

A3. Fixed platform—Concrete Substructure Design Tools Checklist (1 page)

Field Development Phase:			1		2		3
	Study Stage Description:		Prospect Evalu-ation	Field Evalu-ation	Feasibi-lity Study	Field Devel-opment	Con-cept Def-inition
	Estimate Class Required:		Class A (O of M Cost)	Class B (± 40%)	Class C (± 30%)	Class D (± 20%)	Class E (± 15%)
Design Tool	Inter-Discipline						
1. Prospect Evaluation/Weight Estimating Software			√	√	√	√	√
2. 3D Structural Analysis program					[√]	√	√
3. Hydrodynamic (wave load) program					[√]	√	√
4. Hydrostatic collapse program						[√]	√
5. Natural frequency program					[√]	√	√
6. Dynamic response (wave/ seismic) program					[√]	√	√
7. Fatigue Analysis					[√]	√	√
8. Design Code checking programs					[√]	√	√
9. Soil-structure interaction program							[√]
10. Non-linear (progressive collapse) program (optional)							[√]
11. Geotechnical Software					√	√	√
12. Floating Stability Analysis Software					√	√	√

A4. Fixed platform—Concrete Substructure Special Study Checklist (1 page)

Field Development Phase:			**1**		**2**		**3**
Special Study	Study Stage Description:		Prospect Evaluation	Field Evaluation	Feasibility Study	Field Development	Concept Definition
	Estimate Class Required:		Class A (O of M Cost)	Class B (± 40%)	Class C (± 30%)	Class D (± 20%)	Class E (± 15%)
	Inter-Discipline						
GENERAL 1. Riser pipestress analysis						[√]	√
2. J-tube flowline pull-in							[√]
3. Bulk storage					[√]	√	√
4. Oil Storage					[√]	√	√
5. Docking Studies					[√]	√	√
6. Scouring/Sand Movement Study						[√]	[√]
7. Cuttings Disposal/Shale Chute Location Study							[√]
8. Evaluation of potential GBS/Topsides transfer sites	←	Projects/ Structural	[√]	√	√	√	√
9. Mechanical equipment • Ballast system • Grouting system • Cooling of conductors						 √ [√]	 √ √ √

A5. Fixed platform—Concrete Substructure Deliverables Checklist (1 page)

Field Development Phase:			**1**		**2**		**3**
Deliverable	Study Stage Description: Estimate Class Required: Inter-Discipline		Prospect Evaluation Class A (O of M Cost)	Field Evaluation Class B (± 40%)	Feasibility Study Class C (± 30%)	Field Development Class D (± 20%)	Concept Definition Class E (± 15%)
1. Design Premise/briefs	→	PLT			[√]	[√]	√
2. Configuration Study Report	→	Projects	[√]	√	√	√	√
3. Foundation Study Report	→	PLT		[√]	√	√	√
4. Inplace analysis results	→	PLT			[√]	√	√
5. Installation Study Reports	→	Marine Operations			√	√	√
6. Accident analysis results	→	PLT				[√]	√
7. Mechanical equipment	→	PLT			[√]	√	√
8. Drawings	→	Projects		√	√	√	√
9. Weight Estimate report	→	Projects	√	√	√	√	√
10. Material Take Off	→	Procurement					[√]
11. Contribution to Study Report	→	Projects	i	i	i	i	i
12. Topsides/substructure interface report	→	All Disciplines			[√]	√	√
13. Construction and Installation Study Report	→	Construction/ Marine Operations			[√]	√	√
14. Riser pipestress analysis results	→	PLT					[√]
15. J Tube/flowline installation report	→	Pipelines					[√]
16. Storage facilities study report	→	Operations			[√]	√	√
17. Docking study report	→	Marine Operations			[√]	√	√
18. Scouring/sand movement study report	→	PLT				[√]	[√]
19. Cuttings disposal/shale chute study report	→	Operations					[√]
20. Removal report	→					√	√
21. Modeltests	→	PLT				[√]	√
22. Project plans	→	PUT PRT		[√]	√	√	√
23. Project costs	→	PUT PRT	√	√	√	√	√
24. Checklist for next phase	→	PLT			√	√	√

A6. Fixed platform—Concrete Substructure Document Quality Control (1 page)

The following key relates to the document quality control checklist:

Checking Level Key
Recommended Checking Levels:
1 - Self Check + Discipline Internal Check + Interdiscipline Check + Approval
2 - Self Check + Discipline Internal Check + Review and Approval
3 - Self Check + Review and Approval
[3] - Checking level if deliverable is included at this stage

The following table relates to the document's criticality

Criticality	Definition
I	Vital importance that quality of document is high
II	Important that quality of document is high
III	Moderat requirement to quality of document
IIII	This document is of less importance to other disciplines

Field Development Phase:		1		2		3
Document / Deliverable		Study Stage Description / Checking Levels				
Type	Criticality	Prospect Evaluation	Field Evaluation	Feasibi-lity Study	Field Devel-opment	Concept Definition
1. Design Premise/briefs	I			[3]	[2]	2
2. Configuration Study Report	I	[3]	2	2	1	1
3. Foundation Study report	I		[3]	3	2	2
4. Inplace analysis results	I			[3]	3	2
5. Installation Study reports	I			3	2	1
6. Accident analysis results	II				[3]	1
7. Mechanical equipment	II			[3]	2	1
8. Drawings	I		3	2	1	1
9. Weight Estimate report	I	3	2	1	1	1
10. Material Take Off	I					[2]
11. Contribution to Study Report	I	3	3	2	1	1
12. Topsides/substructure interface report	II			[3]	2	1
13. Construction and Installation Study Report	II			[3]	2	1
14. Riser pipestress analysis results	II					[3]
15. J Tube/flowline installation report	II					[2]
16. Storage facilities study report	I			[3]	2	1
17. Docking study report	II			[3]	2	2
18. Scouring/sand movement study report	III				[3]	[2]
19. Cuttings disposal/shale chute study report	III					[3]
20. Removal report	I				2	1
21. Model tests	II				[3]	1
22. Project plans	I		[3]	2	1	1

Appendix B Discipline Check (DC)

Definition: Discipline Check is a check to ensure that technical documents satisfy all internal and external requirements within one's own discipline before further distribution and use. A competent person, other than the one(s) that drew up the documents should carry out the check.

Purpose: To prevent defects and additional work in later phases due to the documents not being based on the correct assumptions or limitations, or that they contain technical or formal defects or weaknesses.

Limitations:

1. Discipline Check does not guarantee that the interfaces to other disciplines are attended to. For this purpose, *Inter Discipline Check* (IDC) is used.

2. Discipline Check can be carried out by a person (or several persons) who has the same, or a higher level of competence than the person(s) who worked out the documents. DC adds safety in comparison to Self-Check, but does not in principle, due to the above-mentioned reason, attend to other defects or weaknesses than those that could have been discovered by a more detailed Self Check.

Reference : Requirements

* ISO 9001, item 4.1.2.2
* ISO 9001, item 4.4
* ISO 9001, item 4.5.2
* Norwegian Petroleum Directorate (NPD): Regulations for structural design of loadbearing structures

Principles:

1. The individual Discipline Manager is responsible for carrying DC into effect within his own area of responsibility before the documents are distributed to others. If consultants are used the relevant Discipline Manager, if necessary supported by the QA department, is also responsible for ensuring that DC has been carried out at the consultant's.

2. DC is carried out by checking that all internal and external requirements have been attended to, particularly the following:
   a) Accordance with the last, valid version of the *Design Basis (Design Baseline),* which lays down the loads, environmental data and other functional requirements, conditions for use, geotechnical conditions, reference documents, etc. that will make up the basis for the engineering and design work.
   b) Accordance with the last, valid version of the *Design Brief,* which describes how the engineering and design work should be carried out (that is to say what important analyses,

calculations and checks are to be carried out), the *Design Criteria,* the design procedures and the interfaces.

c) Check to ascertain that all analyses and calculations are based on the correct conditions and limitations, that the applied software has been approved of for its current use, that all analyses and calculations are carried out correctly and that the results seem reasonable.

d) Check of all analyses and calculations by means of alternative methods and software, if required. This goes especially for, but is not limited to:
   * critical parts of the structure (e.g. tri-cells)
   * cases where new (i.e. unverified) methods or solutions are used

e) Check to ascertain that relevant experience from other projects has been taken into consideration.

f) Check to ensure that all interfaces within the discipline has been attended to.

g) Check to ensure that all formal requirements to the documents have been met, including identification, readability, clarity, neatness, references, dates and signatures.

h) Check to ascertain that all drawings are in accordance with the calculations, that geometrical measures are in mutual accordance, that critical measures and issues have been especially marked (“flagged”) and that the structure is construction- and service friendly.

i) Check to ensure that interfaces to other disciplines has been attended to, as far as possible. (See in addition, Inter Discipline Check).

Records: All inspected documents and completed checklists should be dated and signed by the person(s) who carried out the check.

References:
* ISO 9001, item 4.4
* ISO 9004–1, item 8

Appendix C Inter Discipline Check (IDC)

Definition: Check to ensure that technical documentation satisfies all internal and external requirements with regards to other technical disciplines, before further distribution and use.

Purpose: Preventing defects, unfortunate solutions or extra work from occurring, due to lack of consideration of interfaces to other disciplines.

Limitations: Inter Discipline Check in principle presupposes that *Discipline Check* (DC) has already been carried out. If this is not the case, special initiatives are required, to meet the purpose of IDC.

For IDC to be effective, it is important that the relevant disciplines give the work the necessary priority. The individual Discipline Manager is responsible for this.

Reference Requirements:

* ISO 9001, item 4.1.2.2
* ISO 9001, item 4.4
* ISO 9001, item 4.5.2
* Norwegian Petroleum Directorate (NPD): structural design of loadbearing structures Regulations for

Principles:

1. The individual Discipline Manager is responsible for sending the documents to other disciplines for IDC, in accordance with the project's Quality Plan and the prevailing procedures and distribution list. Normally, *Discipline Check* should have been carried out first. If IDC is to be carried out in parallel to DC, this should be made evident, and necessary initiatives carried out to prevent mistakes and problems from occurring.

   A general Checklist, information about possible particular conditions that should be assessed and a fixed deadline for comments should accompany the documents. The internal procedures regarding document control must be followed regarding dispatch, recording and filing throughout the IDC-process.

2. The Discipline Manager's responsibility also applies when consultants work out the documents.

3. The personnel carrying out the IDC should follow the Checklist and make comments. The Checklist should contain, but are not restricted to, possible conflicts, problems and indistinctness regarding:

   • interface to other disciplines
   • construction techniques and project progress

- contract and internal decisions
- possible particular requirements
- formal requirements to the documents (identification, readability, clarity, references, dates and signatures)

4. The Checklist, Distribution list and possible document pages with comments are dated, signed and further distributed in accordance with agreed procedures.

5. The Discipline Managers receive the comments and evaluate their relevance before passing them on to the person who worked out the documents. The Discipline Manager must approve of the implementation, possibly the neglecting of the ideas and comments. Disagreements regarding technical issues should be solved at meetings with the involved parts. If agreement cannot be reached, the case should be sent one level up in the organization to be decided on.

6. If the comments lead to great changes, a new IDC must be carried out.

Records: All documents and filled in checklists should be signed and dated by the one(s) who carried out the check.

References:
* ISO 9001, item 4.4
* ISO 9004–1, item 8

Appendix D Verification

Definitions:

Verification
Confirmation by examination and provision of objective evidence that specified requirements have been fulfilled.
[ISO 8402:1994]

Notes:

1. In engineering, design and dimensioning, verification concerns the process of examining the result of a given activity, to establish conformance with the stated requirements for that activity. Verification is thus a joint term for several elements of Quality Assurance which concern different types of internal and external reviews, checks, inspections, tests, alternative calculations, surveillance's and quality audits.

2. The term "verified" is used to designate the corresponding status.

Objective evidence
Information, which can be proved to be true, based on facts obtained through observation, measurement, test or other means.

Independence
The verification is independent when personnel other than those who are directly responsible for the work and the results that are to be verified carry it out. Furthermore, the personnel should not report to the same manager and they should be free from any pressure that may influence on their judgement.

Internal verification
Verification carried out by own employees.

External Verification/Third Party's Verification
Verification carried out by personnel employed by and reporting to another organization/body.

Purpose:

The purposes of all types of verification are the following:

1. Preventing defects and failures in the final product or service, as well as preventing additional work and costs due to nonconformities being discovered at a later point in time.

2. Providing evidence of, and thus greater confidence in, that the requirements have been met, and that the product will be well fit for use.

Limitations:

1. The final proof of the product's fitness for use can only be obtained by real use. The applied technology and the verifying personnel's competence in each case limit the value of the verification activities.

2. The degree of independence is often of vital importance to the confidence in the results of the verification. *Internal verification* is necessary, but not always sufficient, for instance with respect to the authorities and one's own management. Usually, the reason behind this is not suspecting occurrences of conscious actions or omissions, but rather the fact that the ability of someone discovering defects in his/her own work is inferior to someone "from outside". Besides, one cannot ignore the fact that the power of judgement could be impaired due to stress, for instance regarding time, money or the mere knowledge about potential consequences of discovering defects and nonconformities. An example of the latter would be if costly analyses or calculations had to be redone if errors were discovered.

<u>Example 1.</u> The value of a simulation depends on how well the computer program has been tested (verified) for similar tasks previously, and that the operator handles the data and the program correctly and unaffected by the desired result.

<u>Example 2.</u> A *Design Review* could be an efficient means of ensuring that the previous experience and the total competence of the organization are conveyed to the structure. However, a condition for this is that persons who have the adequate competence and experience carry out the review, and that sufficient time has been allocated for the purpose.

<u>Example 3.</u> When calculations which have been carried out by means of Finite Element Analysis (FEA) are to be verified, a different method should be applied, or at least a different program. The reason behind this is to avoid the same (systematic) mistakes from being repeated in the verification. In addition, it is presupposed that possible nonconformities, which have been discovered by the verification, will be subject to an in-depth analysis and assessment. If such nonconformity were explained away, for instance by claiming that a coarser model was used in the verification compared to in the original calculations, the verification would give a false feeling of safety.

Reference requirements:

* ISO 9001, item 4.1.2.1
* ISO 9001, item 4.1.2.2
* ISO 9001, item 4.4.7
* ISO 9001, item 4.4.8
* Norwegian Petroleum Directorate (NPD): Regulations for structural design of loadbearing structures

Principles:

1. Both internal and external verification should in principle be *independent,* that means carried out by personnel other than the person(s) who is (are) directly responsible for the work that is to be verified.

2. Regular verification should follow predetermined procedures.

3. Independent, external experts are often used in the verification of high-technology activities or products. These experts should in principle be free to choose their own methods. Neither the project team nor others should direct the verification process in detail, for two reasons. First, because this often creates confusion regarding authority (who is responsible for what) and second, because it can reduce the confidence in the results of the verification. The verifying party in addition ought to have a major impact on which activities or results should be subject to (third party) verification.

4. Verification during engineering, design and dimensioning can be carried out on four different levels (see Fig. 6.1):
   Level 1: Document Review
   Level 2: Extended Document Review
   Level 3: Independent Calculations
   Level 4: Scale Test(s).

<u>Document Review</u> involves a check to ascertain that all documents from the engineering and design phase (calculations, specifications, drawings, technical reports, etc.) are available, and impeccable, unambiguous and fit for use. The review could be carried out on all or some of the documents, depending on the criticality.

Document Review is not likely to reveal more basic defects, e.g. due to the use of inferior methods.

<u>Extended Document Review</u> implies normal Document Review supplied with checks of selected items. Those checks should be documented and filed together with the original document.

<u>Independent Calculations</u> should be carried out if the consequences of potential defects or nonconformities are major. The highest level of safety, and thus the greatest confidence, is achieved if a different method, computer program, computer, etc. is used. However, this is more time consuming. It is also required that methods, computer programs, etc. that are used during verification are themselves verified for the current application. Simple analyses and manual calculations can in some cases be an efficient, cheap and sufficient verification of the results.

Independent calculations should concentrate on the most critical parts of the structure.

<u>Scale Test(s)</u> imply that selected parts of the structure are built, usually on a smaller scale, and loaded or otherwise tested, under controlled conditions. The purpose could be to verify in practice that the structure is able to take the loads it has been dimensioned for, with the given margins for safety, and/or that it can be constructed and inspected with the

presupposed methods, tools, dimensions and materials. Due to practical and economic reasons, such tests with large concrete structures usually cannot be carried out on a full scale (1:1), When evaluating the results, it is of vital importance to take possible scale effects into consideration.

References:

* ISO 9004–1, item 8.5
* ISO 9001, item 4.4.7
* ISO 9001, item 4.4.8

Appendix E Design Review (DR)

Definition: Documented, comprehensive and systematic examination of a design to evaluate its capability to fulfil the requirements for quality, identify problems, if any, and propose the development of solutions. [ISO 8402:1994]

Purpose: Exploiting the total experience from previous projects to achieve an optimal structure. It is, of course, particularly important to prevent serious faults, which can lead to structural breakdown.

Limitations:

1. Design Review is not alone sufficient to ensure a high quality structure.
2. The benefits of DR depends on the participants in the review, particularly their competence, experience from similar projects and structures, creativity and ability to identify potential problems.
3. The Design Review is usually of advisory character. The effect therefore also depends on the project leader having the will and ability to take the advice into consideration.

Reference requirements:

- NPD: Regulations for structural design of loadbearing structures
- NPD: Regulations concerning the licensee's internal control in petroleum activities
- ISO 9001, item 4.4.6
- NPD: Regulations concerning implementation and use of risk analyses in petroleum activities

Principles:

1. Design Reviews can in principle be carried out in all phases during engineering, design, dimensioning, construction and fabrication. The reviews should be included in the Project Plan or Quality Plan. Ad-hoc-like reviews should be arranged if considerable changes in the functional specifications arise, or if needed due to particular problems.

   To characterise the individual reviews further, supplementary designations are often used, for instance:

   - *Preparatory Design Review*
   - *Following-up Design Review*
   - *Design Review of the Design Basis (Design Baseline)*
   - *Design Review of the Design Brief*

- *Design Review of the Design Criteria*
- *Civil Design Review*
- *Design Review of the Shaft Design*
- *Design Review of tri-cell reinforcement, etc.*
- *Mechanical Design Review*
- *Engineering Readiness Review* (often used before permission to start the construction work is granted), etc.

2. The project leader, or the person he/she has authorised, is responsible for preparing, summoning and leading the meetings, in co-operation with the QA department. The participants, sometimes called the Design Council, should be persons with relevant competence regarding the themes that are to be dealt with. Consultants, the authorities and/or contractors may be invited, when appropriate. The necessary documents to be reviewed should always have been studied in advance.

3. Those themes which should be dealt with could include the structure's fitness for use, constructability, testability, strength, reliability, maintainability, safety, environmental considerations, life cycle costs, etc.

   The result of DR could be:

   - calling attention to nonconformities, weaknesses and potential problems with the proposed solution, for instance in light of experience from other projects
   - proposals for improvement
   - confirmation of favourable choices for solutions
   - agreement on the need for other reviews.

4. Minutes of the meetings should be drawn up. It is important to give *the reasons* for the advice and recommendations, as well. If the project manager chooses <u>not</u> to follow the advice, the reason for this should also be recorded.

References:

- ISO 9001, item 4.4.6
- ISO 9004–1, item 8.4
- ISO 9004–1, item 8.6
- ISO 9004–1, item 8.7

Appendix F Hazard and Operability Analysis (HAZOP)

Definition: Formal, systematic and critical review of different parts of a system, design or structure in order to identify potential safety and operational problems.

Purpose: Identifying possible safety and/or operational problems that may arise during construction, operation and maintenance of a process plant or a structure. The analysis could be a complete risk analysis or a prestudy for later, more detailed studies of certain critical parts of a plant or structure.

Limitations:

1. Any non-predicted hazard will not be part of the analysis.

2. The results depend on the competence of the analysis group.

3. HAZOP does not take human failures and common cause failures into account.

4. It is difficult to identify component failures and environmental effects as reasons for nonconformity.

Reference requirements:

- NPD: Regulations for structural design of loadbearing structures
- NPD: Regulations concerning the licensee's internal control in petroleum activities
- NPD: Regulations concerning implementation and use of risk analyses in the petroleum activities

Principles: HAZOP may be carried out as part of pre-engineering and/or detail design, as well as in connection with maintenance and/or modifications of the structure, operation procedures, etc.

The analysis can be divided into 5 steps:

1. Define the purpose of the study, the methodology and the time schedule.

2. Select the members of the analysis group.

3. Prepare the analysis work.

4. Carry out the analysis.

5. Record the results.

The analysis group should be made up of persons with different backgrounds, who have special competence within their own field. The group usually consists of a leader, a secretary plus 4–6 additional persons, depending on the size and the complexity of the object of the analysis. If there is a need for it, other persons can be brought in. The work is mainly carried out in a series of brainstorming meetings, conducted by the leader. The process demands much from the leader of the working group; he or she ought to have previous experience with HAZOP.

The analysis is normally done on the basis of drawings, construction procedures, etc.

The analysis work starts by clarifying the purpose and the normal condition of the elements that are to be evaluated. The work is directed by the HAZOP-leader by means of a series of guiding words and checklists. Examples of guiding words are “none”, “bigger than”, “smaller than”, etc. Every guiding word is used on different structure elements at the specific items that are to be examined. By means of these guiding words, possible nonconformances from the normal condition in every structural element are identified, as well as the reason(s) behind nonconformity and the consequences. The analysis should be carried out for different conditions.

The results should be verifiable and recorded by means of a HAZOP Report Form. The form should as a minimum contain a column for guiding words, nonconformances, causes, consequences, recommenda tions and comments. The latter could, for example, be questions for the project manager, recommendations regarding changes in design or comments about particular risks that ought to be dealt with in special procedures.

References:

* NS 5814:1991 Requirements to risk analyses
* Rausand, Marvin, 1991: Risikoanalyse—Veiledning til NS 5814, Tapir, Trondheim. (This is a guide to the Norwegian Standard NS 5814.)

Appendix G Worst-Case Analysis

Definition: Systematic analysis and assessment of the consequences of the worst possible input data, occurrences and combinations of occurrences for people, the environment and assets.

Purposes:

1. To verify that safety and other important functions are maintained under abnormal loads, foreseeable abuse and abnormal human stress.

2. To ensure that possible decisions on *not* dimensioning for such extreme conditions are made on the right (i.e. a high enough) level in the organization.

Limitations: In worst-case analyses, only those occurrences and combinations of occurrences which have been identified beforehand, and assessed to be of current interest to such analyses, are dealt with.

Reference requirements:

* NPD: Regulations for structural design of loadbearing structures

Principles:

1. Worst-case analyses should be carried out for critical design parameters to obtain a general view of the consequences for personnel, the environment and material values due to extreme stresses that may occur in connection with testing, operation and maintenance. The environment may cause such extreme stresses, by single occurrences or by combinations of occurrences. The assessment may start out with the normal condition of the installation. Alternatively, potential nonconformity's can form the starting point. For example, the normal condition would be that all reinforcement has been installed as planned, while a potential nonconformity might be that 5% of the reinforcement is lacking or seriously corroded in critical sections of the structure.

2. In order for worst-case analyses to be carried out, there ought to be a certain probability for the occurrence(s) to happen. The identification of those occurrences and conditions should be based on a systematic review of the installation.

3. The worst possible occurrences might be analysed one by one, or as a combination of occurrences. Such combinations should be possible, although not very likely to happen at the same time. The analysis of combinations could be by means of a systematic review of different scenarios, e.g. in the form of a matrix.

4. Worst-case analyses do not use a particular technique or method; it is rather a philosophy for finding out how robust the installation

is under the influence of extreme conditions and stresses. The principle can be used both in risk analyses and also for instance in stress calculations and dimensioning.

5. The analyses should be documented in a verifiable manner. In addition the basis for the choice of the analysed occurrences should be recorded.

6. The results could for example be expressed like this: “The structure is robust against the influence”, “Risk reducing action ought to be carried out” or “The probability of the occurrences happening is too small—no action is required”.

7. The result of the analysis should form a basis for decisions.

Examples:

a) Calculation of the structure’s ability to withstand the 10.000-year wave.
b) Calculation of the structure’s stability during an earthquake combined with a hurricane and insufficient maintenance.
c) Assessment of the consequences if all uncertainties in the dimensioning would be pulling in the same, unfortunate direction and at the same time 5% of the reinforcement would be lacking or seriously corroded in critical sections.
d) Assessment of the consequences, if one of the structural elements should be torn off (as in the “Alexander L. Kielland”[6] case).

[6] “Alexander L.Kielland” was a semi-submersible flotel which capsized in the Northern Sea on March 27, 1980. The direct/immediate cause of the loss was that one of the 5 platform legs was torn off, due to extremely heavy weather conditions.

Appendix H Quality Audit/Quality System Audit

Definitions:

Quality Audit

Systematic and independent examination to determine whether quality activities and related results comply with planned arrangements and whether these arrangements are implemented effectively and are suitable to achieve objectives. [ISO 8402:1994]

Audit Program

General view of the planned audits for a particular period of time.

Audit Plan

Detailed plan for the carrying out of a particular audit.

Observation

A statement of fact made during an audit and substantiated by objective evidence. [ISO 10011–1:1992]

Objective Evidence

Qualitative or quantitative information, records or statements of facts pertaining to the quality of an item or service or to the existence and implementation of a quality system element, which is based on observation, measurement or test and which can be verified. [ISO 10011–1:1992], see also Appendix 3 Verification.

Nonconformity

Non-fulfilment of specified requirements. [ISO 8402:1994]

Recommendation

The audit team's proposal for improvement of the auditee's systems.

Purposes:

Quality System Audit is carried out with one or more of the following intentions in mind:

a) Ascertain whether the elements in the Quality System comply with the requirements of the company and the authorities, as a basis for pre-qualification of a supplier or contractor or as a part of the follow-up of a contract.
b) Assess how effective the Quality System is implemented when it comes to meeting the goals for quality.
c) Give the auditee an opportunity to improve the Quality System.

Limitations:

1. Quality Audits can and should contribute to a safe and efficient project execution, but it can never guarantee that nonconformity will not occur or that it is detected in time. The situation could be compared to the car-driving test (as basis for issuing a driving licence); the test is a means of ensuring safe driving, but it cannot guarantee that the driver will never cause accidents.

2. The audit will in practice be based on spot tests, which, of course, affects the reliability.

3. The effectiveness of the audit depends on the competence of the auditor(s) and the auditee's will and power to participate and cooperate, both during the audit and with regard to implementing corrective actions.

Reference requirements:

- NPD: Regulations for structural design of loadbearing structures
- NPD: Regulations concerning the licensee's internal control in petroleum activities
- ISO 9001, item 4.17

Principles:

1. Regular quality audits are done in accordance with a drawn up *audit program.* Audits could also be caused by significant changes in the quality system or the quality of delivered products or services, or in order to follow up Corrective Action Requests (CAR). When selecting the areas to be audited, emphasis should be put on how critical the activity is when it comes to the safety and fitness for use of the product or service and the vulnerability regarding nonconformities.

2. The Quality Audit typically applies to, but is not limited to, the quality system or elements thereof and to processes, products and services. Such audits may be called, respectively:
   * Quality System Audit
   * Process Quality Audit
   * Product Quality Audit
   * Service Quality Audit, etc.

   Quality System Audit is the one most frequently used. The basic principles are, however, the same for all of them. Quality Audit can also be termed according to when in the engineering and design or construction process it is carried out, e.g. in connection with pre-qualifying of supplier, or before commissioning. (The latter is often called *Implementation Quality Audit* or sometimes *Implementation Review*.)

3. Audits should be carried out in accordance with ISO 10011–1, –2 and –3; *Guidelines for auditing quality systems.* The standards give guidelines for the following conditions:
   a) The responsibilities and tasks of the audit team and audit leader.
   b) The qualification requirements to the auditors and the audit leader.
   c) Planning, preparation, carrying out and reporting the audit.
   d) Follow-up of Corrective Action Requests.

Records: The Quality Audit should be reported and recorded in accordance with the guidelines in ISO 10011–1. The audit is not "closed" until it is verified and duly signed to show that all the Corrective Action Requests have been properly dealt with.

References:

* ISO 9004–1, item 5.4
* ISO 10011–1, –2, –3 Guidelines for auditing quality systems;
  Part 1: Auditing
  Part 2: Qualification criteria for quality system auditors
  Part 3: Management of audit programmes