OFFSHORE CONCRETE STRUCTURES

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SUMMARY

The design methods of offshore concrete structures have developed considerably since the pioneering years of the late sixties and early seventies and the petroleum industry have gained more than 20 years experience with the use of concrete gravity substructures for oil and gas producing platforms.

Aspects of the operational experience and the main steps in the development of such structures are presented and lines are drawn to today's construction of concrete structures for the Bass Strait.

The market for the large concrete structures in the North Sea is presently very meagre. The paper considers some of the challenges facing the industry when trying develop new competitive concepts and design methods.

The design procedure is presented in general terms with a run through of the postprocessing methods employed in today's design. Typical aspects of construction are also shown.

INTRODUCTION

The beginning of 1994 has seen the construction start of two concrete gravity base structures for oil production platforms offshore Australia. They will be installed in the Bass Strait in 1996 and are the first of their kind to be built on the southern hemisphere. Until now substructures in Australia have all been made of steel, typically as conventional Jacket structures or single tower gravity based steel structures. A larger platform with Production Facilities and Living Quarters will generally require a jacket while for small, remotely operated platforms, the latter may suffice.

The choice of concrete substructure for the latest development in Bass Strait proves that concrete is a competitive alternative to steel for either type of platform. The main argument for the concrete solution is the ability of the structure to carry its own weight and the topload in a floating condition from the construction site inshore to the actual field location.

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EXISTING CGS' IN THE NORTH SEA

There are at present more than 26 concrete gravity support structures for oil or gas producing platforms in the North Sea. The first was installed in 1973 (Ekofisk One) The last to be installed was Sleipner A in 1993. In 1995 the Troll platform will be installed in the Norwegian Sector of the North Sea. 1995 will also see the installation of the first Tension Leg Platform (TLP) with a hull made of light weight aggregate concrete and a similar structure (a concrete floater) with catenary anchoring system.

Listed in Table 1 are the main Offshore Concrete Structures and the year of installation. The water depth for the CGS' installed in the North Sea vary from 40 m to 303m. Topside weight range from a few thousand tons to more than 50.000 tons The Heidrun TLP has a design topside weight of 60.000 tons. Some of the concrete structures have oil storage. Typical for the structures built in Norway is that the topside structure, or part of it, is installed without any lifting equipment. A mating method is utilised which require deep waters, typically 150 meters.

OPERATIONAL EXPERIENCE

Det Norske Veritas (DNV) have performed inspection of 11 concrete substructures in the North Sea representing 65 platform years in service[1]. No deterioration of the structural concrete was revealed other than what was directly attributable to accidental impacts from falling objects or ships operating near the platform. Concrete surfaces seemed in excellent condition without cracks or defects.

Where repairs have been required this has been performed satisfactorily. Generally no maintenance is likely other than renewal of sacrificial anodes. It is worth noting that in some cases the consumption of anodes has exceeded what was estimated in the design. The general design assumption for corrosion protection is that anodes should last the platforms lifetime. The indications are that the designers have underestimated the current drainage to reinforcement in contact with embedment plates, penetrations and skirts.

During the regular inspections the following constitute the main findings:

- debris and drill mud on top of and around caisson
- marine growth in splash zone
- corrosion in splash zone
- registered cracks have healed
- pH-value generally around 5 (should stay above 4)
- epoxy membranes applied on concrete in splash zone remains generally in good condition; signs of spalling in some cases.

The offshore installations are regularly undergoing major or minor modifications. These could typically be prompted by changes in production yielding e.g.:

- increased topside weight
- altered centre of gravity of topside
- joining up of new well heads or fields
- general upgrading of equipment

In some cases such modifications have involved changes to the concrete substructures, such as attaching new clamps along shaft walls. Additional local loads have not posed any design problems and coring of bolt holes in the shaft walls and caisson have been performed successfully.

Modifying the offshore installations to account for production from new developments has also meant documenting the platforms ability to withstand an extended lifetime. DNV have performed lifetime extension studies for several structures in the North Sea.

The design lifetime for structures being built today is typically 25 to 60 years. The substructure for the Troll Gas development is being designed for a 70 year lifetime.

CHALLENGES IN DESIGN OF CGS

The main challenge in the development of a CGS will always be competitiveness. With many available production sites/yards for steel structures, alternative development solutions in the form of jackets, jack-ups, floating production systems or subsea completion may prove more economic. The advantage of a concrete gravity base solution may prove itself for one or several of the following reasons:

- client require oil storage facility
- soil conditions are unfavourable (expensive piling required for a jacket solution)
- lack of reliable weather windows limits/prevents offshore lifting/piling operations
- high local content (building industry)
- low maintenance cost
- topside weight capacity

It should be remembered that the construction cost of a CGS itself may only be 10% or less of the total platform installation. The economy lies very much in the advantages the buoyant structure provides during construction and installation. This is typically the case for the two structures being built in Port Kembla.

CONCEPT DEVELOPMENT

Over the years contractors and consultants around Europe have developed a wide range of concrete gravity platform concepts. These are general solutions developed for typical applications. The two structures being built in Port Kembla for installation in Bass Strait in 1996 are based on the same general idea of a large base supporting slender load carrying tower(s) extended through the water plane (Fig. 1). The stiffness of the caisson base is provided by a cell structure. These cells can have various shapes depending on the shape of the caisson and the tower configuration. The Norwegian CONDEEP is made up of cylindrical pressure vessels, or cells, with dome shaped ends to allow the concrete structure to withstand the extreme water pressures exerted on the walls during the kingpin mating operation.

The original CONDEEP had 19 of these cells and 3 shafts. Over the years this concept has been adjusted to fit various soil conditions and client's technical specifications and today "Condeeps" in the North Sea range from the Monotower type with 7 cells around one central shaft (Draugen) to the wide based Statfjord B with 4 shafts on top of 24 cells.

The concrete structures developed for the British and Dutch sector of the North Sea have generally had straight walled cells as these concepts do no rely on deep submergence.

In the ice resistant gravity structure for the Hibernia development offshore Canada a system of triangular cells is employed. The tooth-shaped exterior ensures that the enormous ice loads are taken as pure compression in the concrete walls. The thick outer walls will also resist the hydrostatic pressure during deck mating.

Once field specific data are known and the client can establish his requirements for a platform installation the designers may scale and adjust their concepts. This phase of the development relies on the designers experience and knowledge of the concept. Adjustments to any of the main dimensions will inevitably upset the basic load assumptions and the hydrostatic stability calculations. It is important that the concept has some tolerance to changes. More than once a project has been delayed due to revised wave loads, updated geotechnical data or the clients wish to increase topload.

OPTIMISATION

Development of oil and gas fields in the North Sea involves large investments over 2 -3 years for a 25 to 50 years production. Installation of the structures offshore may only take place in very definite weather windows, typically July, August. Any delay during design and construction may lead to the project being delayed a whole year. Hence the project schedule will generally not allow much optimisation. The tendency is rather to allow too short time for the designers to reach a level where the construction can start without changes being expected.

Today's advanced computer software enables the designer to analyse and post-process enormous amount of data. Rather than taking advantage of this by optimising the design and performing additional verification the designer and the verifying body are expected to perform their work in less time.

The challenge must therefor be to convince the clients and the contractors of the importance of granting ample time for design and verification.

PLANNING

Planning the design of a CGS requires delivery dates for some basic, mainly client (i.e. oil company) provided information, the most critical being weight and COG of the topsides, oil storage capacity (if any), soil properties at the installation site, water depth and environmental conditions (waves and wind). The tradition in Norway is award an EPCI contract for the design, construction, transportation and installation of the CGS including the deck mating operation and to let the detail design and construction develop consecutively. This has the advantage that building may start as soon as the first results from the analyses/post-processing are available and it allows a close co-operation between the designers and the site personnel in the design period. This means that the solutions presented by the engineers on the drawings are already "ratified" by the work force on site. The disadvantage, particularly from a designers and verifiers point of view, is that throughout the project the design is then very much driven by the construction schedule.

Constructing a CGS is a work intensive exercise. A project may typically employ 400 to 600 workers. Keeping everyone working effectively requires experienced planners and exact knowledge of the different activities involved. The logistics of crane capacity, on- and off- site transport, storage etc. is very complicated. Maintaining an experienced core work force is crucial.

ANALYSIS

The design of a CGS inevitably involves the use of Finite Element Analyses. The caisson structure is highly indeterminate and the shafts interact with the topside steel structure. In addition the structure interacts with the supporting soil on site and may even go through various critical floating phases with wave action or uneven ballasting loads. Before establishing a FE model the geometry must be fixed (ref. main scantling approval) and all major loads should have been determined.

The analyses tools have developed tremendously since 1969, when DNV first started using the newly developed codes later to be known as SESAM. Today's software allows running of solid element analyses covering the complete CGS' and topsides. 3 - 4 weeks computer runs are reduced to 24 - 48 hours. This has in fact given the designers the possibility to rerun the analysis or part of it if the geometry or loads changes.

Over the years the non-linear behaviour of concrete (i.e. cracking) has been accounted for in the design in an approximate and unsatisfactory way. The main challenge now is to enable the finite element models to include investigation of the non-linear behaviour of parts of the concrete structure.

POSTPROCESSING

"Postprocessing" is the term used for transforming the not-so-useful stresses (nodal or Gaussian) given by the FE-analyses, into section forces, thereafter sorting the results to find the worst combination of forces for any section and lastly perform code checks of selected (or all) limit states according to the specified codes.

In the SESAM suite of programs CONCODE provides all these facilities with code checking being performed according to various standards (NS3473, DNV-FOI). As for the analysis software above, the challenge within postprocessing is to develop routines which will handle non-linear problems. A large JIP-project (INDACS) aiming to do just this is presently under way in Norway.

DETAILING

The drawings are the designers final product and the basis for all work performed on site. Even with experienced designers and draftsmen there is now way a satisfactory result can be obtained without the co-operation of the contractor. His construction methods, skills, experience and ideas are vital input to reinforcement layout, prestressing systems and geometry details.

The continuous challenge within the scope of detailing is to find ways of presenting the design in a way which produce the least amount of drawings which are easy to check (i.e. provide the necessary information) and prevents mistakes being made on site.

CAD is fully accepted within production of reinforcement drawings, but there is still a long way to go before 3-D representation of reinforcement in a CGS structure is common. Up till now this facility has only been used to investigate areas with congested reinforcement and known conflicts between reinforcement and mechanical items.

JOINT DESIGN (D-REGIONS)

The postprocessing systems can only handle sections of the structure where the assumption of plane sections remaining plane is valid. This assumption is true for slab and shell sections away from intersections. The intersections, however, may be critical, especially from a compression stress point of view. Similarly, areas with large concentrated loads and correspondingly thick sections cannot be properly analysed and designed with linear methods. The development of software for non-linear analyses and design of such sections is part of the JIP-project mentioned earlier.

MAIN DESIGN PARAMETERS AND LOADING CONDITIONS

Some parameters are established early in the development phase of a new offshore platform and remain virtually unchanged throughout the design process. Typical examples would be:

- number of well slots
- oil storage capacity
- wave height

It is a challenge for the developer, viz. the oil company to establish and fix all important parameters as early as possible, thereby preventing costly changes late in the design period. Parameters that may change during the design process:

- new soil data or data interpretation
- topside weight and centre of gravity
- ballast level in cells during marine operations
- design specifications

Alternatively well defined tolerances may be included in the concept where experience shows that changes will occur. The cost implications must be clarified, however.

A CGS has two main interfaces: upwards against the topside and downwards against the sea bottom. Any changes in the deck weight or the soil characteristics will inevitably have serious consequences for the CGS design. Experience shows that most changes and discussions are related to these two areas.

TOPSIDE WEIGHT AND STRUCTURAL SYSTEM

The concept phase and the pre-engineering phase will determine the optimum support conditions for a chosen layout of facilities and will yield an estimated total weight of topsides. This process is crucial in deriving the optimal solution, technically and economically. The alternative structural systems are typically:

- MSF (Module Support Frame) or Integrated Deck
- MSB (Module Support Beams)
- One, two, three or four column support

The soil conditions may influence the choice of superstructure: e.g. a low bearing capacity may require a wide caisson which will provide the basis for multiple shafts. Similarly for structures at large water depths it is important to keep the splash zone area to a minimum to reduce the effect of wave forces. Hence, a Monotower structure may be the optimum solution.

FUNCTIONAL REQUIREMENTS

The shafts may be used to house various facilities. Typically fire-water pumps, water/diesel storage tanks and permanent ballast water/oil export pumps may be located inside a one shaft (utility), while well conductors (oil or gas), J-tubes and hydrocarbon risers are located in the other(s) (drill). One drill shaft may typically have slots for up to 24 conductors. Running the conductors, risers etc. inside the shaft protects them from the extreme wave action and possible impacts from ships operating near the platforms.

The size, number of and orientation of conductors determine the minimum required internal diameter of the drill shafts. In order not to attract too large wave loads, however, the diameter of the shafts near the water line (splash zone) should be kept to a minimum.

A feature unique to the concrete substructure is their ability to hold large amounts of oil in storage while waiting for tankers to bring it ashore. The temperature effect of the distribution and storage of hot oil within the structure must, however, be taken into account in the design and the effect of this may be significant.

SOIL CONDITIONS

With the installation of the Gullfaks C platform in 1989 and the Draugen platform in 1993 it has been demonstrated that a CGS may be supported on a wide range of soil conditions. The platform's site specific characteristics are achieved by adjusting the total base area or the depth of penetrating skirts. An indication of how the various soil conditions in the North Sea affect the CGS-concepts is given by the following examples:

	Sleipner A	Troll (1995)
Soil cond.:	Hard sand	Soft clay
Base area:	12,000 m^2	16,600 m^2
Skirt depth:	(1.0 m*)	36.0 m
Water depth:	82.5 m	303 m

* grout formwork

The recent installation of the Goodwin "A" platform and earlier the North Rankin "A" platform has clearly demonstrated the problems related to the calcareous soils on the Australian North West Shelf. These sediments represent a challenge to offshore foundation engineers due to the complicated interaction between pile and soil as driving of piles takes place. The pile bearing capacity has turned out to be much lower than what was previously anticipated.

For the design of a gravity based structure this problem is of little relevance. The horizontal and not the vertical resistance will normally govern the foundation bearing capacity, and the critical failure mode is one of shallow sliding. On a location with soil conditions resembling those of the Rankin "A" it has been shown [3] that 6 m deep skirts would suffice. Penetration of the skirts would not pose any problem.

WATER DEPTH

When specifying water depth for a fixed offshore installation, account should be made for any settlement or subsidence.

The problem of subsidence due to the extraction of hydrocarbons from the underground formations was not recognised when the Ekofisk Field was established in the mid seventies midway between England, Denmark and Norway. Subsequently, the operator has had to extend the jacket legs by 6 meters and construct a new concrete barrier around the concrete oil storage tank to maintain the level of safety of the installations. Further measures are necessary and a number of the old platforms will be replaced by new ones in the near future.

ENVIRONMENTAL CONDITIONS

The specification for environmental condition should cover the following elements:

- Wave height
- Current
- Wind speed
- Seismic activity
- Erosion

OIL STORAGE

Implementing oil storage in the structure requires special consideration in the design procedure. The effect of temperature may increase the amount of reinforcement considerably and the method of implementing the load effect in the design should be clearly stated in the specifications.

The main objection to having a 600.000 bbls (typical) oil storage under water offshore is the inherent danger this poses to the environment. However, the only known leak from an oil storage of this kind in the North

Sea was caused by a 10 tonnes pipe section falling freely through the water and hitting the roof of one of the oil storage tanks. The punched-through concrete section remained in place due to the reinforcement and the hole was repaired successfully.

There are different methods for oil storage under water and each method requires separate considerations in the design of the concrete structure and the mechanical systems.

- Dry oil
  Air pressure (atmospheric) above the oil surface. Any (minor) leak will only lead to flow of water into the tanks due to the outside excess water pressure.
- Wet oil
  Water below the oil; oil drained from the top as water is pumped in underneath, or vice versa. The typical method in the North Sea, normally in combination with underpressure.
- Underpressure
  A wet oil system where the possibility of oil leakage to the surrounding environment is prevented by maintaining an underpressure within the cells compared to the outside water pressure. Experience from the North Sea is that this an expensive mechanical system to maintain and the oil companies prefer to pay for the additional cost of prestressing and reinforcement to satisfy the requirements for water tightness.

CONSTRUCTION FACILITIES

It is a widespread misconception that construction of a CGS requires deep sheltered waters. The ongoing construction of the West Tuna and Bream B platforms in Port Kembla proves otherwise. The depth of the casting basin is 7.6 meters and the minimum depth of the towing channel out of the harbour is 15 meters. As the West Tuna structure is towed to the Bass Strait it will carry the complete top load of 7000 tonnes.

The construction facilities will, however, be one of the parameters governing the shape, total cost and the construction schedule and data needs to be established on the following:

- Dry dock facility - water depth
- Towing channel

In the search for a suitable CGS construction site in Western Australia several locations have been considered feasible.

DESIGN PROCESS

GENERAL

This chapter presents a typical design process for an offshore structure, but the design process will depend on:

- Client requirements
- Mandating regulations and design codes
- Consultant qualifications/resources
- Structure (new concept, new materials,)
- Loads (environmental conditions, topside weight)

With respect to the development and design of an offshore substructure it is important to recognise the following:

- Environmental loads are complex
- Geometry of structure is complex
- Consequences of damage/failure are extreme
- Amount of reinforcement may exceed 1000kg/m3 locally

- Dynamic behaviour of structure may be a critical factor in the development and design.

FEASIBILITY STUDY

The feasibility study will normally involve the following:

- Hand calculations
- Experience
- Evaluations

It will document the main construction phases, stability during tow and resistance to the ultimate loads in operation. To prevent any waste of time and effort through the subsequent phases it is important that all basic parameters and design results are verified.

CONCEPT DEVELOPMENT

During the further concept development the following will normally be produced:

- Hand calculations
- (Simple) Finite Element Analyses checks
- Main scantling (wall thickness', typical reinforcement and prestressing amounts)
- Cost estimates and schedules

The Conceptual Design should determine all main dimensions, such as caisson overall length and width, typical internal wall layout, wall thickness (roughly) and should possibly also give estimates on amount of prestressing.

PRE-ENGINEERING

- FE-analysis (shell elements)
- Updated (optimised) dimensions
- Typical layout of reinf. and prestressing
- Details to show feasibility in congested areas
- Material take off
- Construction considerations

The Preliminary Design phase may involve use of simplified Finite Element Analyses to check critical sections of the structure and provide estimates of required reinforcement. If necessary wall thickness may be revised or haunches may be introduced. The use of local thickness variations is sensible from a concrete weight point of view, but complicates construction. It should be emphasised that the global effects on a structure of this size may be of prime importance and can only be properly estimated by use of FEA.

DETAIL ENGINEERING

The scope for the Detailed Design phase is (mainly) to perform Code Checks for all (relevant) sections of the structure, provide required amount of reinforcement and detail the reinforcement on drawings. In order to perform a reliable code check the stresses in each particular part of the structure must be known. Although the structures in question may be considered simple, the combination of global and local effects is complicated without the proper tools. Finite Element Analysis and well proven software for postprocessing is considered a must.

Local analysis of a straight wall for the maximum occurring hydrostatic pressure may give the required minimum wall thickness at mid span. At the supports, however, global effects will typically cause disturbances requiring heavy reinforcement or increased concrete cross-sectional area.

The aim must be to let the results from the preliminary design form the basis for a FE model to be prepared right from the start of the detail design phase.

This section does not cover parallel running activities, such as wave analysis, topside designs, marine operation analysis etc.

The key activities that forms the basis for the detail design is listed below:

- Design Basis and Design Briefs
  - * Global Analyses
  - * Local Analyses
  - * Non-linear Analyses
  - * D-region Analyses
  - * Postprocessing
  - * Hand calculations
- Analyses
  - * FEM Manual
  - * FE modelling and load application
  - * FEA Go Ahead
  - * Analysis runs
- Postprocessing
  - * Preparation of drawings
  - * Preparation of post processing input files
  - * (Analysis runs)
  - * Completion of post-files
  - * Code checking
- Detailing
  - * Detailing of reinforcement and reinforcement drawings
- Documentation

A comprehensive outline of the postprocessing activity is presented in Appendix I. It should be noted that it is necessary to establish a reinforcement system prior to code checking. Size and position of reinforcement are important parts of the input.

CONSTRUCTION

Slipforming is the optimal construction for concrete structures of this size and shape. This should be recognised by the designer and taken into account when detailing the reinforcement for caisson walls and shaft walls.

The designer must also account for forces that will develop during construction due to temperature effects, prestressing loads or creep and shrinkage of concrete as it cures.

TRANSPORTATION

The actual marine operations are not covered here. It is, however important to not the different forces that may develop in the structure during the various transportation phases due to loads e.g.:

- Uneven ballasting
- Accidental flooding of cells
- Mooring and towing forces
- Impact loads

Some of these loads may also give rise to built-in stresses that should be added to the stresses developing in the subsequent phases.

INSTALLATION

The installation phase will not normally be governing for the CGS-structure, but local loads on skirt or guiding dowels should be considered.

OPERATION

The temporary phases may be governing the amount of reinforcement in parts of the structure. It must be emphasised, however, that it is the operation phase that requires the structure to be durable, and without cracks, to withstand dynamic loads and fatigue failure through 40 to 70 years and to resist the action of the 100 year wave and possibly earthquakes.

REMOVAL

It is a general requirement of the Norwegian Petroleum Directorate that all structures shall be removed once the production from the field has seized. The oil company should present a feasible method for removal and the design should include relevant load cases.

The structures being built for the Bass Straight are designed to be re-floated and towed to final resting place.

RECENT DEVELOPMENTS

This paper has covered aspects of design, construction, installation and operation of Gravity Base Structures, i.e.: structures that are positioned on the seabed and obtain their stability through self weight and or skirts.

Currently two floating concrete structures are being completed in Norway, a Tension Leg Platform (TLP) for the Heidrun field and a catenary anchored platform for the oil production on the Troll field. These structures have brought the use of concrete into a new dimension with a series of challenges with respect to both design and construction. The basic principles, however, are still the same as those applied for a CGS.

REFERENCES:

[1] Fjeld, S. and Røland, B. (1982

In-Service Experience with Eleven Offshore Concrete Structures

OTC 4358, Houston)

[2] Røland, B. (1987)

Case Histories of damages and Repair to Concrete Platforms Seminar on concrete offshore platforms, ISMES, Bergamo, Italy

[3] Røland, B and Høeg, K. (1993) Design and Construction of Offshore Gravity Platforms on Calcareous Sediments

Beryl A	1975	Frigg TCP2	1977	Ravensburn North	1991?
Brent B	1975	Frigg TP1	1975	Schwedeneck	1983
Brent C	1978	Gullfaks A	1986	Schwedeneck	1983
Brent D	1976	Gullfaks B	1987	Sleipner A	1993
Cormorant	1978	Gullfaks C	1989	Statfjord A	1977
Draugen	1993	Heidrun TLP	1995	Statfjord B	1981
Dunlin		MCP01	1976	Statfjord C	1984
Ekofisk One	1973	NAM F3	1992?	Troll	(1995)
Frigg CDP1	1975	Ninian Central	1978	Troll Oil	(1995)
		Oseberg A	1988		

Table 1 Offshore Concrete Structures and Year of Installation

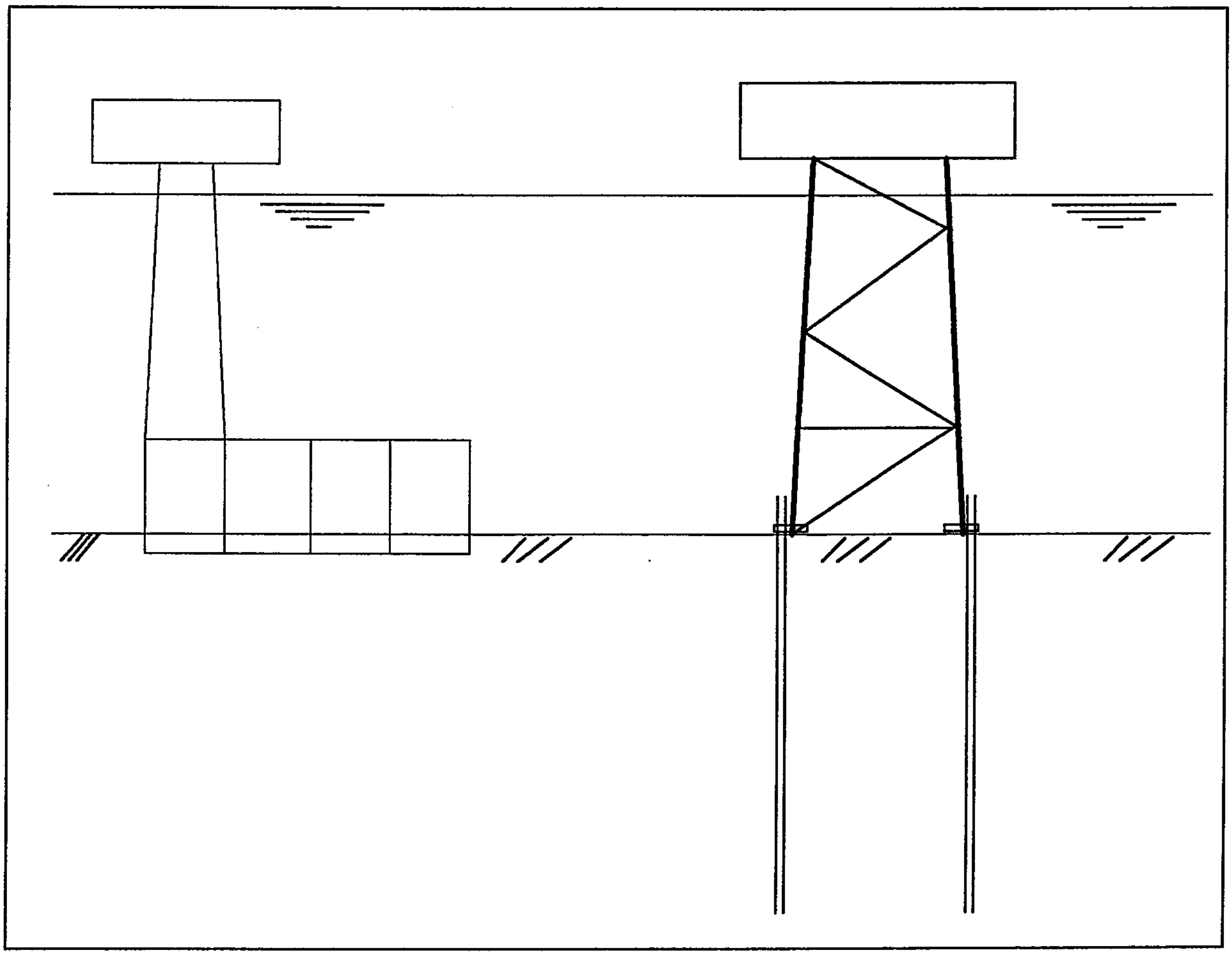

Figure 1 Concrete Gravity Structure and Piled Jacket Structure

APPENDIX I

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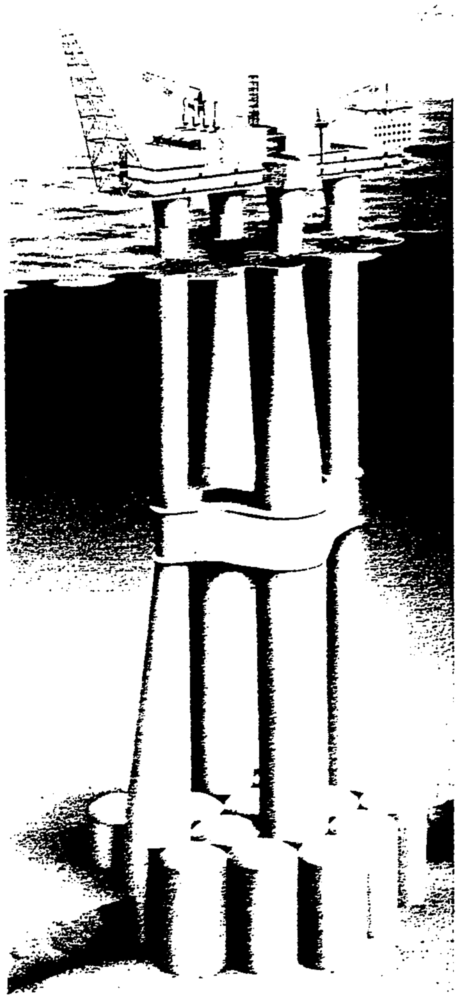
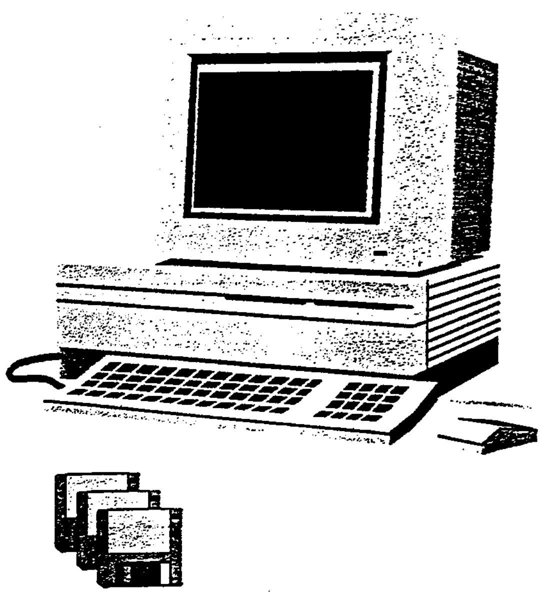




CONCODE
Presentation

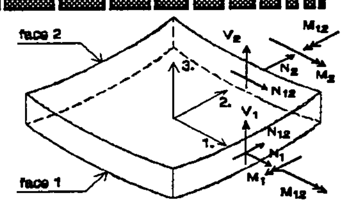
CONCODE System Overview

I.I

- ***DEVELOPED BY:*** *Dr. techn. Olav Olsen a.s*
- ***MARKETED BY:*** *Det Norske Veritas Sesam a.s*
- ***USER INTERFACE:***
  - *current:* — *line mode* — *(DOS, UNIX, VMS,..)*
  - *developing:* — *graphics mode* — *(Windows, Motif)*
- ***RUN:***
  - *batch*
  - *interactive*
- ***DATABASES:***
  - *integration data (sup.el., elem., gausspt., integration constants,...)*
  - *design data (sections, geometry, forces, reinforcement, utitilization ratio,...)*
- ***STAFF: (development, quality assurance, user support)***
  - *Kjell Fiskum, M.Sc. - Jonny Nilsson, M.Sc.*
  - *Arild Fiskum, Eng. - Sidsel Wang Skudal, Eng.*
- ***QUALITY ASSURANCE:***
  - *test battery*
  - *independent systemtests executed by the Department of Structural Technology*
  - *aiming for ISO 9001 certification within 1994*

CONCODE Main Features

I.I



- ***Part One Main Tasks***
  - ***Locate sections through elements and create section geometry***
  - ***Create extrapolated sections***
  - ***Transform/integrate stresses from FE analysis result files into shell stress resultants***
  - ***FE analysis verification (plots)***
- ***Part Two Main Tasks***
  - ***Combine stress resultants (ordinary loads and imposed deformations)***
  - ***Sort out worst combinations***
  - ***Calculate necessary reinforcement***
  - ***Calculate utilization ratios for concrete and reinf. (ULS,SLS,PLS,FLS)***
  - ***Generate plots/tables for verification and documentation***

CONCODE Development -94

I.I

- *new integration method (SECFOR)*
- *non-linear analyses (INDACS)*
- *shear design (Collins/Adebar) and BS811O/DNV rules*

- *design database*

- *support for design of D-regions*
- *design of beam sections and solids*
- *calculation of prestressing loads*

- *graphical user interface (Windows and Motif)*
- *new plotting features*

- *connection to CAD program for reinforcement dwg. (ConMic)*

I.I

Projects utilizing CONCODE 1.1

- *A number of concept studies, pre- and detail engineering projects since 1980*

- *Detail engineering projects:*
  - *Gullfaks A*
  - *Gullfaks B*
  - *Gullfaks C*
  - *Oseberg A*
  - *Sleipner A*
  - *Draugen*
  - *Troll Oil*

- *Pre-engineering projects:*
  - *Troll GBS*
  - *Heidrun TLP*

Customers 1.1

- *Conoco, Houston*

- *Veritec, Norway*

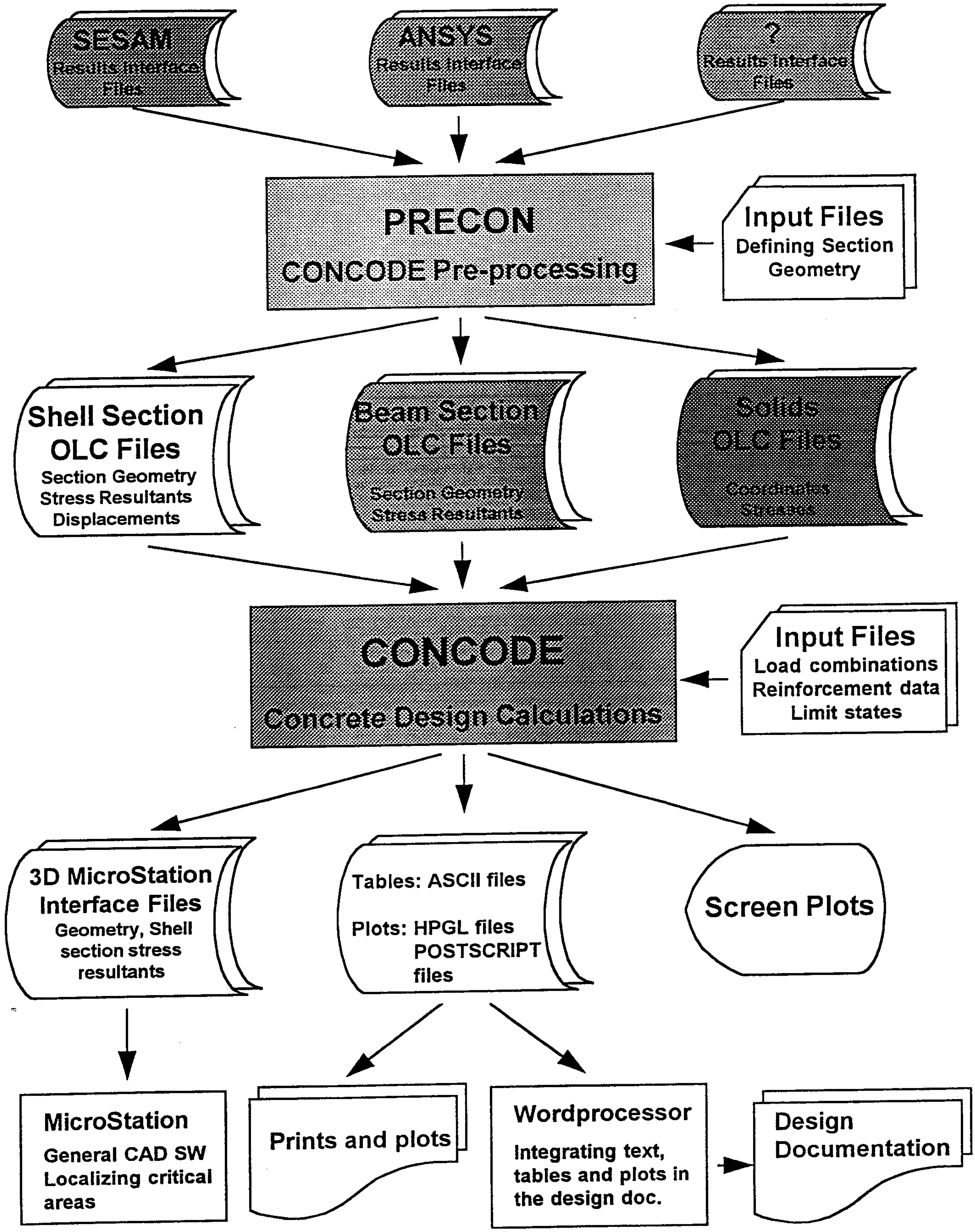
- *Kværner Engineering, Norway*

- *Exxon, Houston*

- *Ove Arup & Partners, London (trial evaluation)*

CONCODE Design Flow

1.1



INPUT 1.1

- *material - and design parameters*
- *design sections*
- *reinforcement alternatives*
- *reinforcement distributions*
- *load combinations*
- *limit states, water pressure*
- *design mode*
- *wanted output (tables, plots)*

OUTPUT 1.1

- *geometry*
- *forces/moments*
- *displacements*
- *necessary reinforcement (points/areas)*
- *utilization of concrete and reinforcement stresses*
- *crack widths*
- *water tightness*
- *miner's sum*

MATERIALS I.I

- ***normal weight concrete***
- ***light weight concrete***
- ***ordinary reinforcement***
- ***prestressed reinforcement***

LOAD TYPES I.I

- ***ordinary loads***
- ***imposed deformations***
- ***turn/return wave loads (+w, -w)***
- ***complex wave loads (real, imag.)***
- ***water pressure in cracks***

LOAD COMBINATIONS 1.1

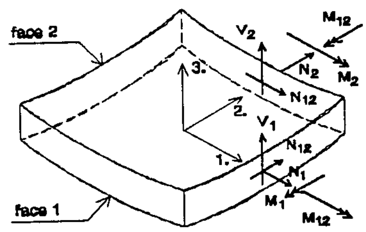
- *equilibrium comb.* ELC
- *basic comb.* BAS
- *sort comb.* SOR

LOAD EFFECTS (origin) 1.1

- *from analysis through PRECON* OLC
- *through input statements* ILC
- *through prestressing reinforcement* PLC

LOAD EFFECTS *(shell section)* I.I

- *N1, N2, N12:* *membran forces* *(F/L)*
- *M1, M2, M12:* *plate moments* *(FL/L)*
- *V1, V2:* *out of plane shear forces* *(F/L)*



DESIGN CALCULATION METHODS I.I

- *in plane design:* *mathematical, non-linear equations*
- *shear design:* *empirical formulae*

DESIGN MODES

1.1

- *verification (fixed reinforcement)*
- *design (determines neccessary reinforcement)*
  - *independent sections*
  - *user-defined areas*

DESIGN CODES

1.1

- *Current:*
  - *NS 3473 nov 1989*
  - *NPD regulations*
  - *BS 8110*
- *May 1994:*
  - *DNV rules*

LIMIT STATES SUPPORTED I.I

- *ultimate* (ULS)
- *progressive collapse* (PLS)
- *serviceability* (SLS)
- *fatigue* (FLS)
- *water tightness criteria* (WTC)

DESIGN CHECKS I.I

- *ULS/PLS:* *stresses in concrete and reinforcement*
- *SLS:* *stresses, crack widths, water tightness*
- *FLS:* *miner's sum*

CONCODE System
Quality Assurance I.I

- *Theory manual*
- *Technical manual*

- *Users's manual*
- *User course*

- *Systemtest Design*
- *Systemtest Geometry and Stress Resultants*
- *Test Battery*

SUMMARY I.I

- *a complete solution for verification/design of concrete shell structures*
- *a proven solution during approx. 10 years /# platforms*
- *a extensively verified solution*
- *close cooperation with an advanced concrete design environment: OO ++*
- *close cooperation with an advanced FEM environment: DNV Sesam*
- *close cooperation with an advanced software house: DNV Sesam*
- *strong commitment to keep **CONCODE** a leading design tool*
- *strong commitment to serve clients / projects.*

Plotting features

I.I

- **Simplified plots:**
  - *shell forces*
  - *utilization ratioes*
  - *reinforcement distribution*
  - *design sections*

Plotting features (cont.)

I.I

- **Section geometry plots:**
  - *shell forces*
  - *principal forces and moments*
  - *beam forces*
  - *stresses*
  - *geometry with axes*
  - *displacements*
  - *utilization ratioes*
  - *D-regions*

Element plots 1.1

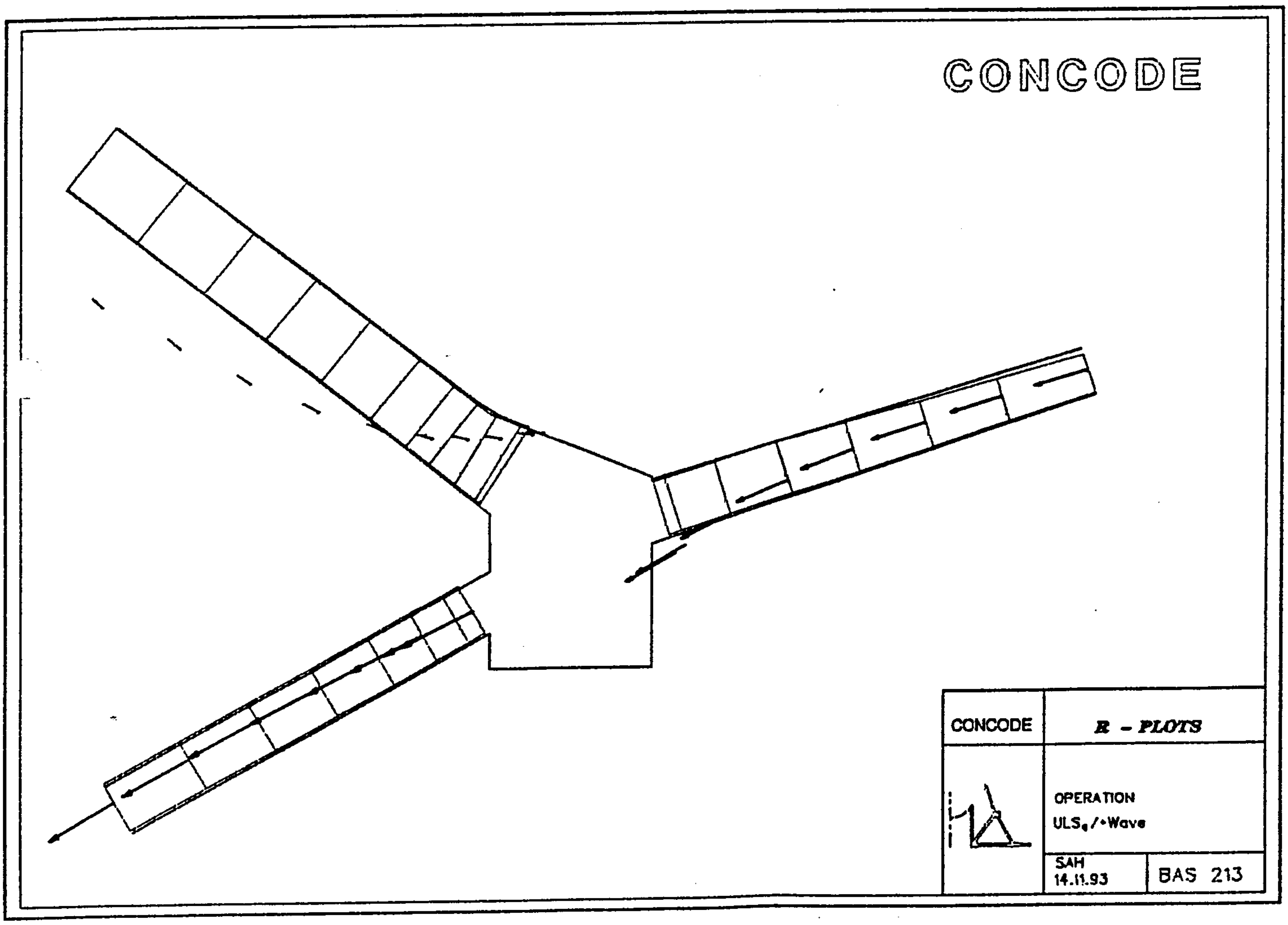
- **Undeformed node geometry with the following options:**
  - node no.
  - element no.
  - gausspt. mark w/wo no.
  - deformed geometry w/wo values
  - element nodal forces w/wo values
  - gausspt. stresses w/wo values (comp., principal stresses)
  - displacements, stresses and forces may be plotted for OLC, ELC and BAS

Section plots 1.1

- **Section geometry with the following options:**
  - thickness
  - 123-axes
  - F- and H- no.
  - section forces w/wo values (comp., principal forces/moments)
  - section forces may be plotted for OLC, ELC and BAS

R-PLOT 1.1

- *R-PLOT for arbitrary beam sections*
  - *interpretation of structural response*
  - *selection of determinant loadcases/areas*
  - *design of ringbeams*
- *Freebody with response and attaced outer loads*
  - *truss modelling*
  - *equilibrium consideration*
  - *verification of analyses*
- *Relative deformations for defined loadcombinations*
  - *truss modelling*
  - *interpretation of structural response*

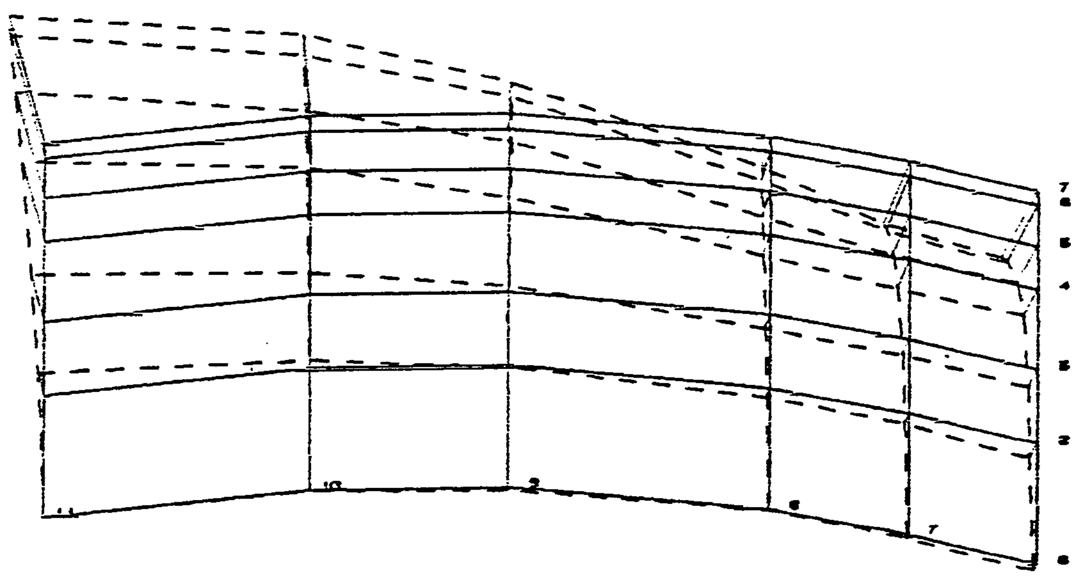


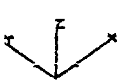
PART : SK1	CELL : A5	SIGN. JN	Nov 17 1998 12131147	PAGE
STRESS	OLC 1 - 312	CHECK	DATE	

MAX : 312. F.S8 H.S7

MIN : -1679. F.S11 H.S7

SEC		FORCE	PART	CELL
F.S5	H.S1	33.	SK1	A5
F.S6	H.S3	241.	SK1	A5
F.S6	H.S5	522.	SK1	A5
F.S6	H.S7	312.	SK1	A5
F.S7	H.S2	157.	SK1	A5
F.S7	H.S4	407.	SK1	A5
F.S7	H.S6	756.	SK1	A5
F.S8	H.S1	22.	SK1	A5
F.S8	H.S3	198.	SK1	A5
F.S8	H.S5	565.	SK1	A5
F.S8	H.S7	420.	SK1	A5
F.S9	H.S2	25.	SK1	A5
F.S9	H.S4	-197.	SK1	A5
F.S9	H.S6	-407.	SK1	A5
F.S10	H.S1	20.	SK1	A5
F.S10	H.S3	-285.	SK1	A5
F.S10	H.S5	-785.	SK1	A5
F.S10	H.S7	-1054.	SK1	A5
F.S11	H.S2	-288.	SK1	A5
F.S11	H.S4	-1054.	SK1	A5
F.S11	H.S6	-1622.	SK1	A5





PART : SK1 CELL : A3	SIGN. JN	Nov 17 1995 12151147	PAGE
UTILIZATION Y1 (MM2/M) LIMIT STATE ULS	CHECK	DATE	

MAX : 131. F.S14 H.S2

MIN : 56. F.S7 H.S2

SEC		UR	REF	LOAD
F.S1	H.S2	77.	2182.	OLC1
F.S2	H.S2	75.	2182.	OLC1
F.S3	H.S2	67.	2182.	BAS1000
F.S4	H.32	87.	2182.	BAS1000
F.S5	H.32	66.	2182.	OLC3
F.S6	H.S2	58.	2182.	OLC3
F.S7	H.S2	67.	2182.	BAS1000
F.S8	H.S2	71.	2182.	OLC1
F.S9	H.S2	90.	2182.	OLC1
F.S10	H.S2	102.	2182.	OLC1
F.S11	H.S2	115.	2182.	OLC1
F.S12	H.S2	124.	2182.	OLC1
F.S13	H.S2	129.	2182.	OLC1
F.S14	H.S2	131.	2182.	OLC1
F.S15	H.S2	122.	2182.	OLC1
F.S16	H.S2	115.	2182.	OLC1
F.S17	H.S2	96.	2182.	OLC1
F.S18	H.S2	84.	2182.	OLC1
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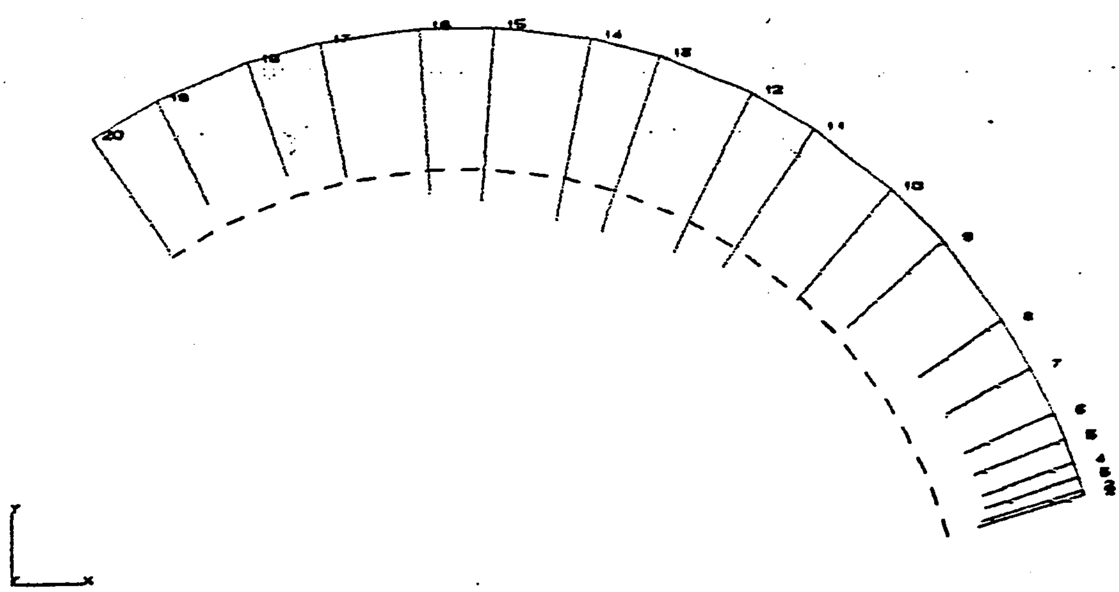
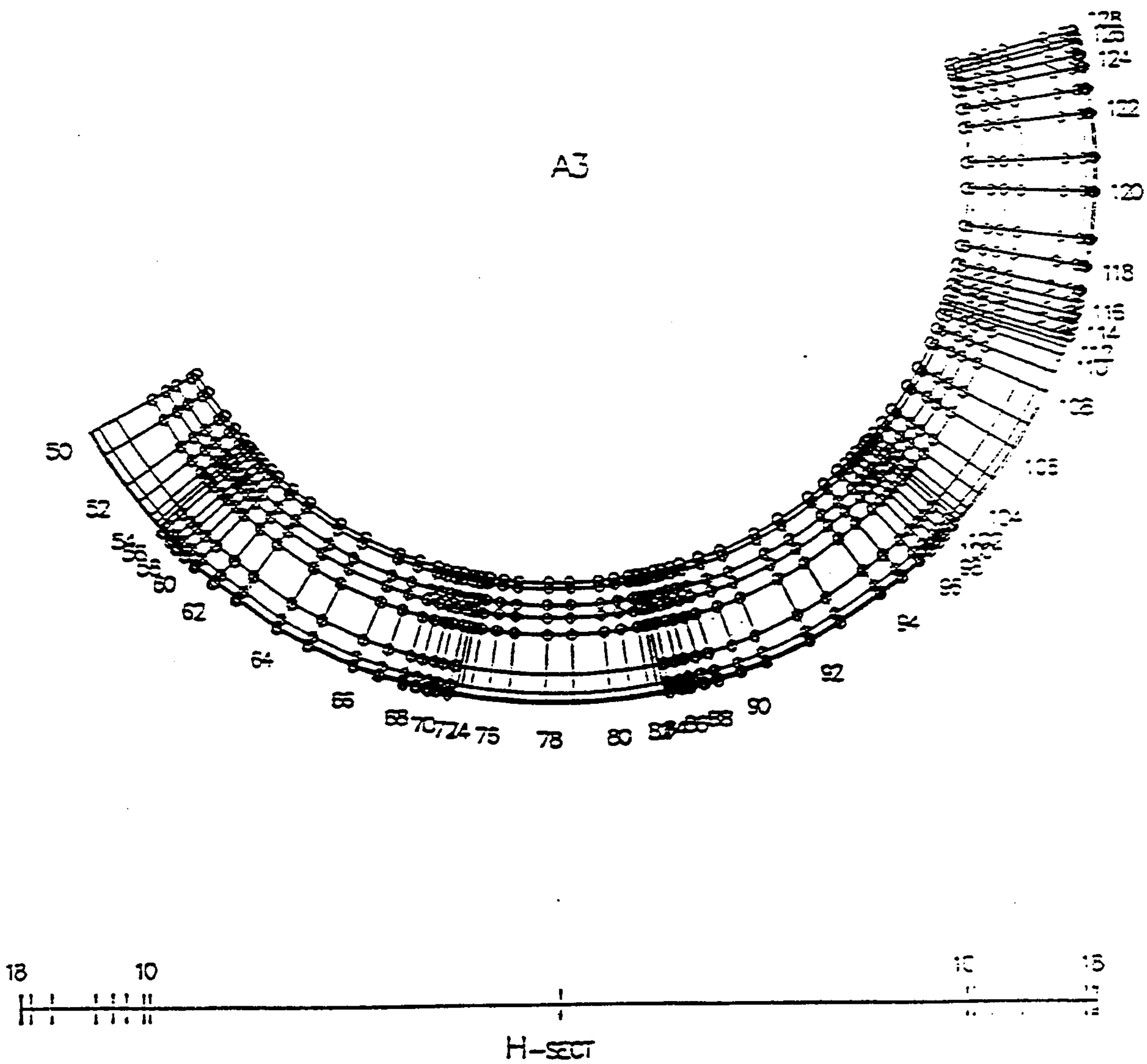
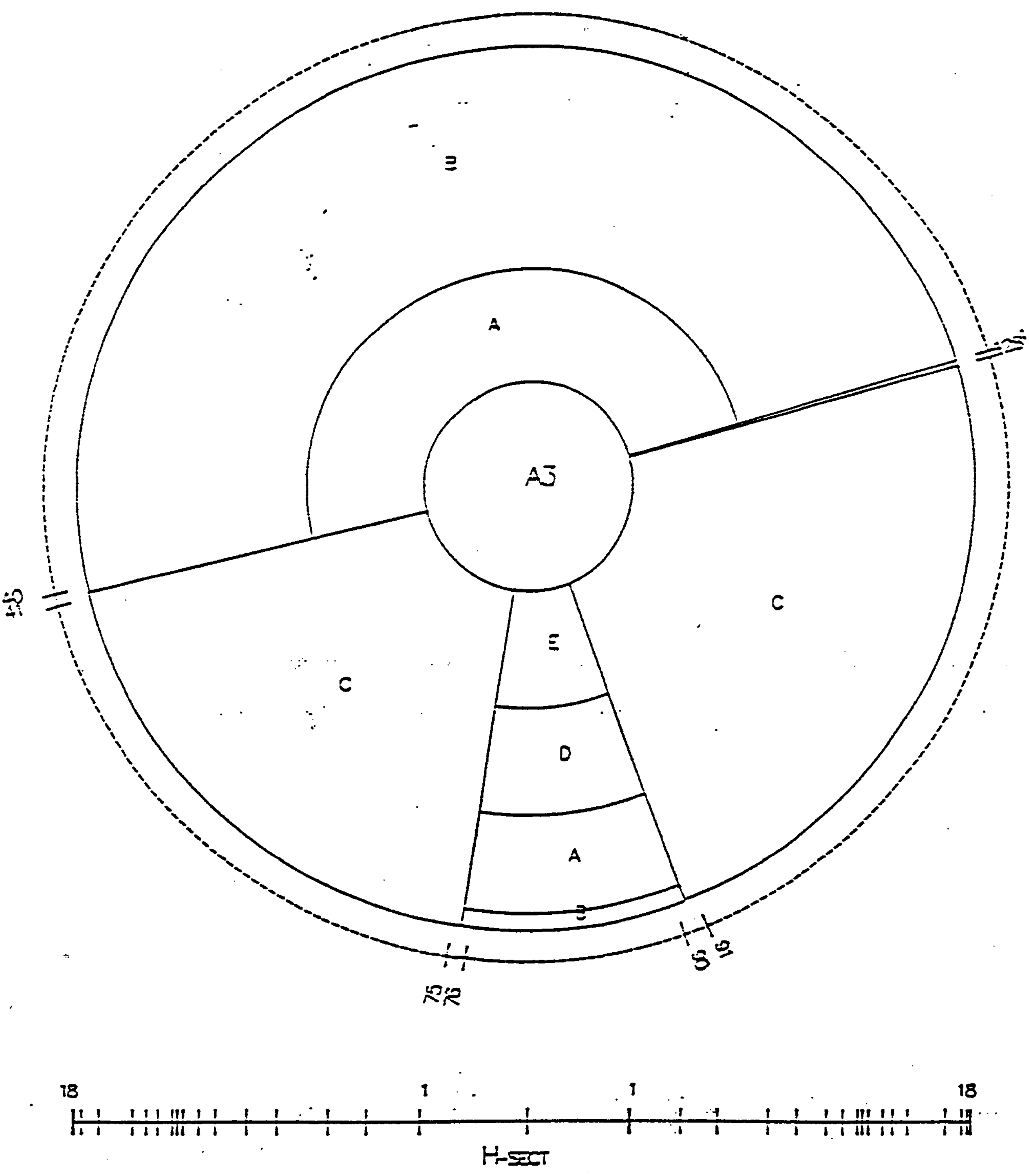
F.S20	H.S2	96.	2182.	OLC1



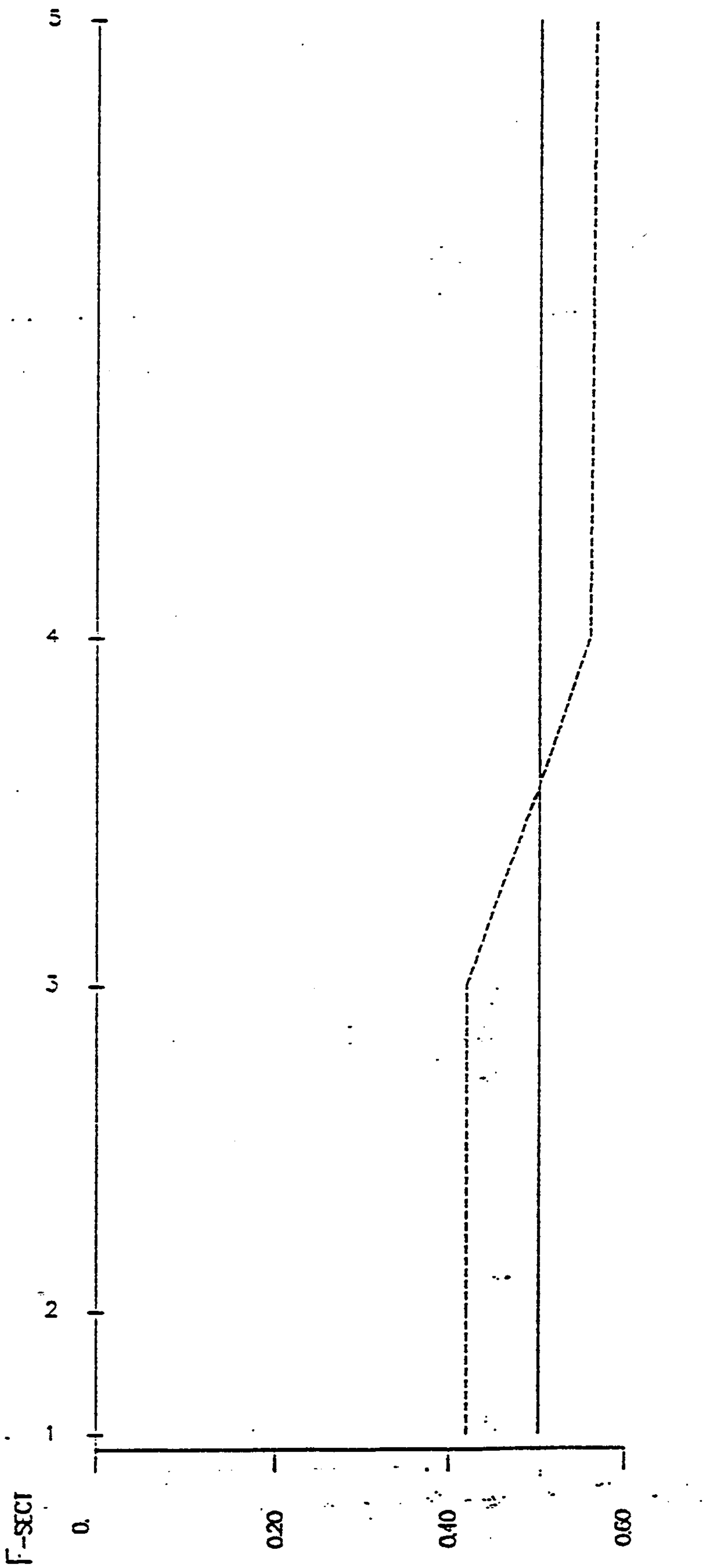
DEPLO TEST	SGN.	DATE	PAGE
PART : LDO CELL : A3	CHECK	DATE	
LOCATION OF DESIGN SECTIONS			



DEPLO TEST	SIGN. -	DATE	PAGE
PART : LDO CELL : A3	CHECK	DATE	
X2-REINFORCEMENT DISTRIBUTION (MM2/M)			



DEPLO TEST PART : SKI CELL : A3 H.S 1- 5 UTILIZATION W1 (MM) LIMIT STATE SLS	SIGN.	DATE	PAGE
	CHECK	DATE	



UR D/O	(MM)	LOAD-ID	SECT-ID
113.00	0.50	BAS 1	H.S 1
112.00	0.50	BAS 1	H.S 1
84.00	0.50	BAS 1	H.S 1
84.00	0.50	BAS 1	H.S 1
84.00	0.50	BAS 1	H.S 1

PLOT15