ode

McALPINE Design Group

TOTAL

TOTAL OIL MARINE LTD

RE-ANALYSIS OF MCP-01

FINAL REPORT

NOVEMBER 1994

OFFSHORE DESIGN ENGINEERING LIMITED
6 Spring Gardens
Citadel Place,
Tinworth Street
London SE11 5EH
Tel: 071 793 9444
Fax. 071 793 9072

McALPINE DESIGN GROUP
Eaton Court
Maylands Avenue
Hemel Hempstead
Herts HP2 7TR
Tel: 0442 233444
Fax: 0442 230024

McALPINE
Design Group

TOTAL OIL MARINE LTD

RE-ANALYSIS OF MCP-01:

FINAL REPORT

OFFSHORE DESIGN ENGINEERING LIMITED
6 Spring Gardens
Citadel Place,
Tinworth Street
London SE11 5EH
Tel: 071 793 9444
Fax. 071 793 9072

McALPINE DESIGN GROUP
Eaton Court
Maylands Avenue
Hemel Hempstead
Herts HP2 7TR
Tel: 0442 233444
Fax: 0442 230024

NOVEMBER 1994

FOREWARD

This document is divided into two parts.

PART 1

Part 1 is the report on the structural re-analysis performed between August 1993 and February 1994. The report contains results and commentary on the re-analysis.

PART 2

Part 2 is the Background Document, prepared between December 1992 and January 1993 as part of the process of drafting the re-analysis specification. It contains background information on the original design, construction and installation as well as details of subsequent modifications and inspections.

PART 1

RE-ANALYSIS REPORT

PROJECT:
TOTAL OIL MARINE
MCP-01 PLATFORM

McALPINE
Design Group

RE-ANALYSIS OF MCP-01

FINAL REPORT

DOCUMENT NO. 3401-A-M-002-2	ODE JOB No. 340

DATE	REV NO.	STATUS	ISSUED BY	APPROVED BY
3/11/94	2	FINAL	PW	PRG
15/7/94	1	DRAFT FINAL	PW	PRG
25/2/94	0	FIRST ISSUE	PW	PRG

CONTENTS

APPENDICES

APPENDIX E (CON'T)

APPENDIX H - BOAT IMPACT

APPENDIX I - LOCAL MODELS

EXECUTIVE SUMMARY

GENERAL

The concrete gravity base platform MCP-01 has been re-analysed on behalf of Total Oil Marine plc by a joint venture of Offshore Design Engineering Ltd and the McAlpine Design Group.

The original certification for MCP-01 by Lloyds in 1975 was for 20 years, and thus the primary objective of the re-analysis was to verify that the design life of the platform may be safely extended by another 20 years to 2015; secondary objectives were to:

- provide a tool for integrity matters;
- provide advice on structural aspects of future plans;
- allow suitable reaction in the event of an emergency;
- develop safe and cost effective inspection philosophy;
- operate weight control system;
- perform disinvestment studies.

ACTIVITIES PERFORMED

The re-analysis was broken down into the following main activities:

a) Data Collection

A comprehensive data collection exercise was performed which involved the assembly of all relevant drawings and documentation held by TOTAL, Doris and ODE into a project library, and their registration on a dedicated database.

b) FE Model

A large 12000 element (57000 active degrees of freedom) model was built using the FE program ANSYS (Rev. 5.0) that covered the concrete sub-structure, and the main and manifold deck beams and columns. The model was based on the as-built drawings. Four noded shell elements and 3 noded beam elements were used for the walls/slabs and beams/columns respectively. Two noded spring elements were used to model the soil. As the overall objective was to provide an economic model consistent with acceptable accuracy, it was necessary to supplement the main model with six local models consisting of 3D bricks and plate elements: tunnel; base slab; tunnel/interior wall; lobel wall/diaphragm; star to circular shaft; and base slab/lobed walls. The total active DOF in these local models is 120000.

The FE model has been validated for stiffness by comparison with site measured natural frequentcies which have not changed significantly over a number of years. The natural frequencies, computed with a simple 2 degree of freedom model using FE model results and original 1975 soilestiffness, are 5 - 7.5% less than site measured natural frequencies. This is considered to be a good level of agreement and conservative from an assessment point of view. A lower bound soil stiffness has been used in the model in order to conservatively assess the forces in the structure.

c) Loads

All loads on the structure were re-calculated; these included topsides, structure deadweight, wind, snow & ice, wave, prestress and temperature. In particular the topsides load was re-evaluated in relation to the consequences of demanning; and the wave loading was re-calculated by Doris Engineering on the basis of revised environmental criteria supplied by MAREX Ltd and the long term marine growth profile from the 1991 AURIS report.

The on-bottom weight of the structure has been regularised from 205,000t at original installation in 1976 to approximately 214,000t in 1994, by taking into account the addition of topsides modules in the late '70's and early '80's, re-appraisal of the ballast loads and the effects of the demanning project. The total weight (dead and live) of topsides equipment at present is estimated to be 14,700t.

Due to the reduced extreme (100 year) wave height (26.4m compared to 29.0m), the mud-line moment and shear have reduced by 23% and 7% respectively.

d) Code Checks

The finite elements were grouped into so called "Design Groups" which represented zones of the structure with similar thicknesses and reinforcement details: 42 for the plate elements and 87 for the beam elements. Twenty four basic loadcases were formulated and combined to give 156 loads combinations for detailed analysis. Results for these were then filtered by the in-house program MEP (Minimum Enveloping Program) in order to obtain for each design group the load combinations that contained 26 key parameters (axial force, shear force, principal stress etc). The associated sets of forces were then checked against the relevant section properties in accordance with BS 8110 using the W. S. Atkins program CONCRETE for both ULS and SLS conditions. Results were taken from the local models, where the main FE model was not considered to be insufficiently accurate.

It was concluded that the sub-structure and the deck met current code requirements for both operational and extreme conditions with a few exceptions. These were a small number of "hot spots", particularly in the submerged zone which do not satisfy the DEn Guidelines for crack widths or for shear capacities (up to 20 - 30% overstressed).

However, with respect to crack widths there is no evidence from existing offshore structures that cracking in the submerged zone has resulted in significant corrosion of reinforcement. Thus durability is considered to be acceptable provided that adequate inspection is maintained for a selection of areas where calculated crack widths are outwith the code limits.

Shear capacity evaluation varies significantly between the codes of practice and the shear stresses found would be acceptable under the Norwegian Code (NS3473) or with shear enhancement factors. Thus it can be concluded that these areas of structure are adequate.

e) Fatigue Checks

The fatigue strength of the concrete sub-structure was reviewed in order to ensure that the fatigue life of all parts was in excess of 40 years (1975-2015). An initial review was performed in accordance with the simplified approach of the DEn Guidance Notes, but in virtually all design groups stresses exceeded the limits. A more detailed analysis was therefore performed using the MAREX environmental data and the fatigue option of the program CONCRETE. This showed, that even with very conservative assumptions, only six out of 42 design groups have fatigue lives less than 100 years, the lowest being 49 years which occurs for design group 21 (external diaphragm). Fatigue performance was therefore considered acceptable.

f) Boat Impact

The most credible boat impact scenario for the Jarlan Wall according to the DEn Guidelines was determined to be a 11/14 MJ collision corresponding to a 5000t supply boat (bow & stern impacts). Force/indentation characteristics for typical supply boats were taken from DNv guidelines and the equivalent pressures applied to a non-linear local model of a typical 15m wide x 10m deep section of the Jarlan Wall. In addition a number of hand checks were also performed. The results indicated that apart from some local spalling there would be no punching shear, direct shear or bending type failures to the Jarlan Wall for the 11/14MJ collision. The local model is now available to investigate any future boat impacts.

g) Foundation Status

Factors of safety, against sliding, overturning and bearing capacity were re-calculated using the increased on-bottom weight and reduced wave loading. It was found that all factors of safety were in excess of the minimum allowables with the most critical being a bearing factor of safety of 1.83 (minimum allowable 1.5). A summary of safety factors is presented in Table ES1.

Parameter	Factor of Safety
Sliding	1.88
Overturning	4.05
Bearing	1.83

Table ES1 - Foundation Stability Safety Factors

No detailed FE analysis of the foundation soils was performed as foundation loads have not increased significantly and no large settlements have occured since installation. In addition frequency monitoring has also shown the foundations to be behaving normally.

CONCLUSIONS

Overall the results of the re-analysis indicate that the Platform (deck and sub-structure) has sufficient strength, fatigue life and foundation stability for another 20 years of operation subject to the following:

i) Durability, with respect to crack width criteria, in certain localised areas fall short of the DEn Guidelines requirements but it is considered that provided adequate inspection is maintained, structural integrity up to 2015 will not be compromised;

ii) Stress levels are relatively high over some parts of the structure and thus deterioration or damage would be particularly of concern if these parts were affected.

Figures ES1 to 4 show areas of the structure (sub-structure and deck) that have high stress levels, large crack widths or low fatigue lives. Diagrams showing calculated crack widths for the sub-structure are presented in Appendix E.

It should be noted that these conclusions assume that structural deterioration, in particular rebar and/or cable corrosion, has not occurred to any great extent.

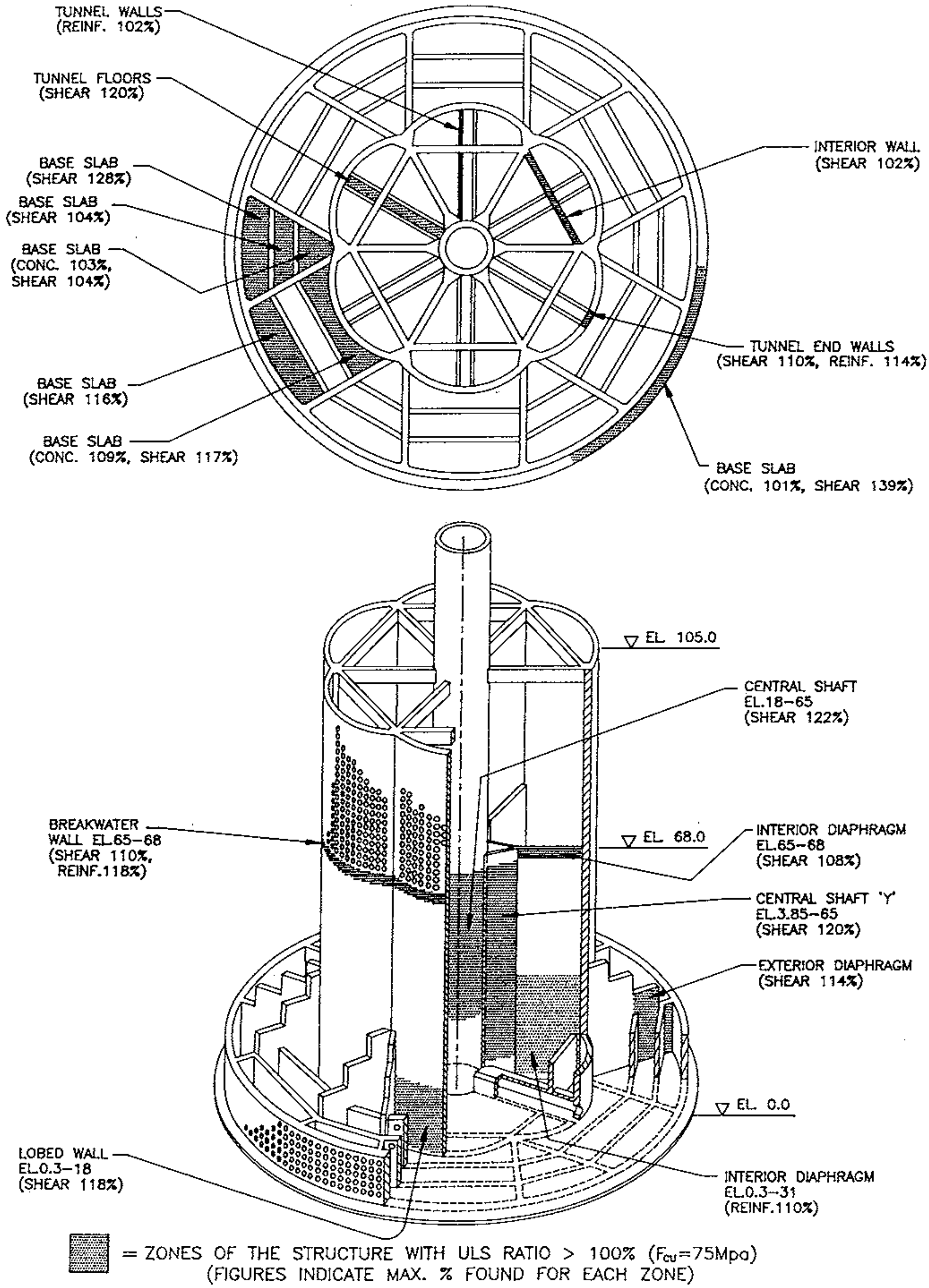
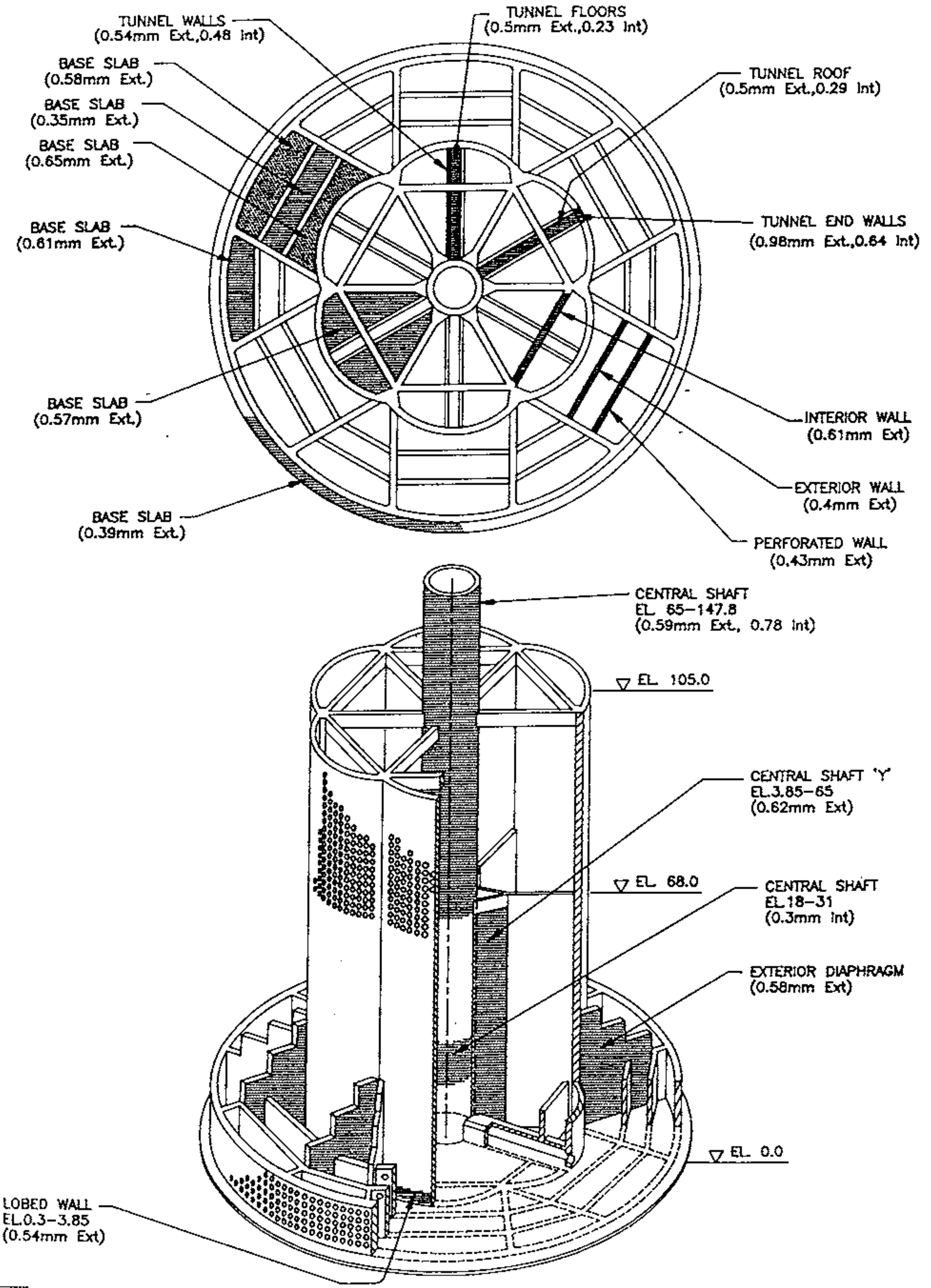


FIGURE ES1 – Areas of the Substructure with High Stress Levels (ULS)



= ZONES WITH CALCULATED CRACK WIDTHS EXCEEDING CODE (0.3mm EXT., 0.1mm INT.)
(FIGURES INDICATE MAXIMUM CRACK WIDTHS FOUND FOR EACH ZONE)

Figure ES2 – Areas of the Sub-Structure with Large Crack Widths

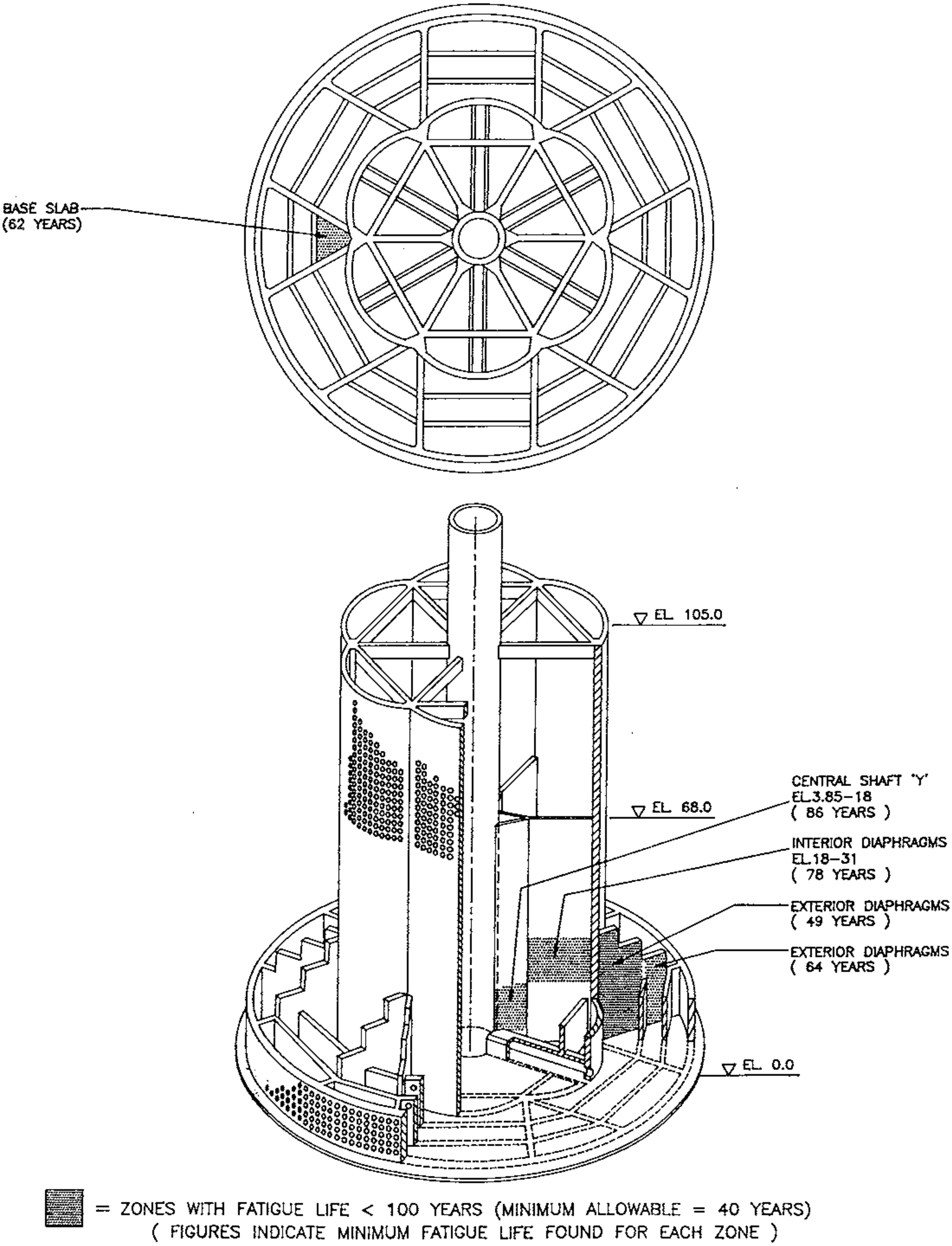


Figure ES3 – Areas of the Sub-Structure with Low Fatigue Lives

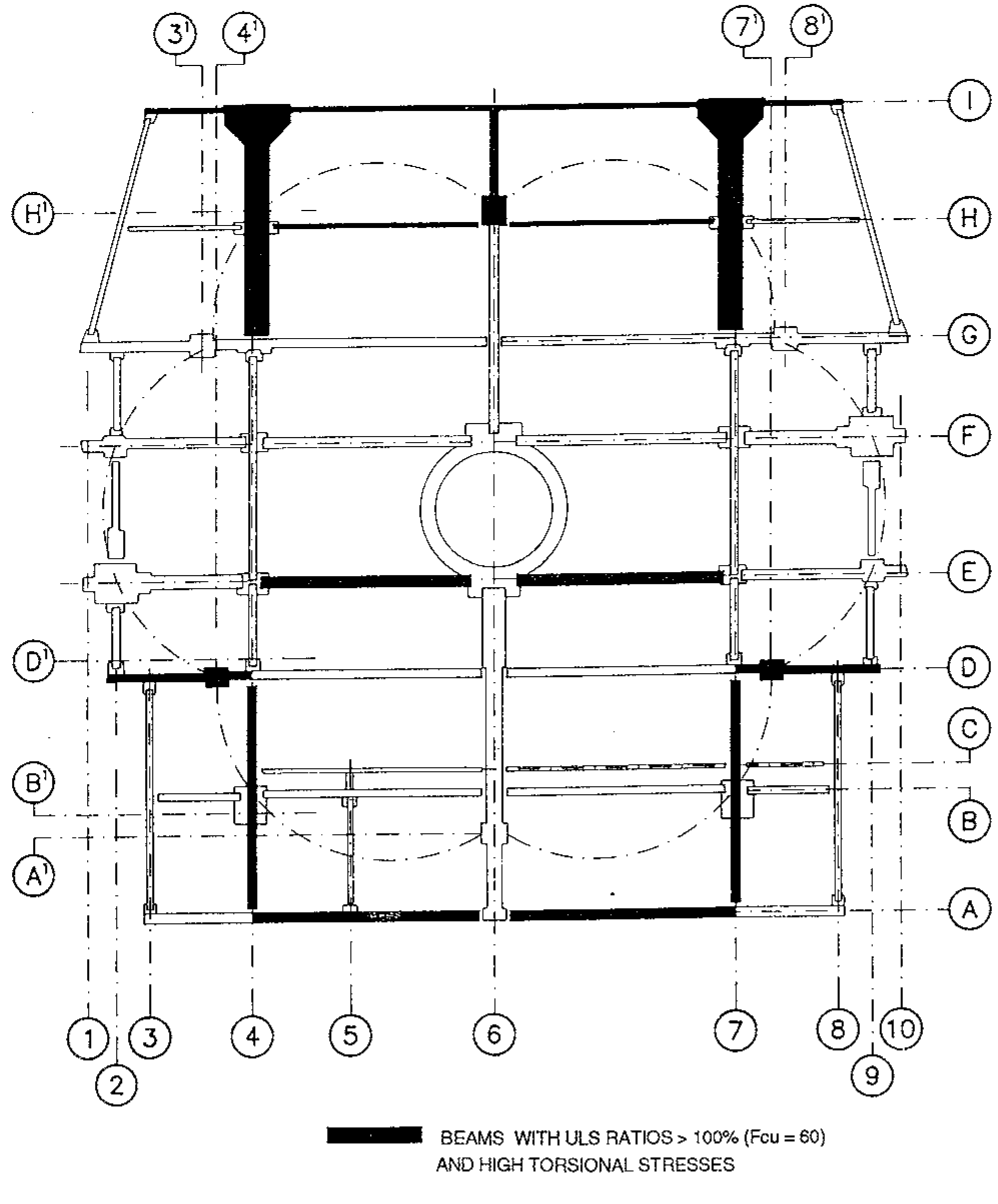


Figure ES4 - Deck Beams with High Stress Levels

1.0 INTRODUCTION

This report presents the results of the structural re-analysis of MCP-01 performed on behalf of Total Oil Marine PLC by a joint venture of Offshore Design Engineering Ltd and the McAlpine Design Group.

The re-analysis was performed in accordance with ODE document nos. 331-S-M-002 (Specification for the Re-analysis of Concrete Structures), and 3311-S-M-001 (Background Report on Specification for Re-analysis of Concrete Structure) and covered the concrete sub-structure, the concrete deck beams (main and manifold), concrete manifold deck columns and the steel tubular deck columns.

This report contains a descriptive report and commentary on the work performed during the re-analysis project.

The Appendix contains lists of calculation notes and available relevant documentation, details of the FE models, and stress/code check plots.

Approximately 20 files of calculations were generated during the project and these are available separately.

2.0 OBJECTIVES OF RE-ANALYSIS

The original certification for MCP-01 by Lloyds in 1975 was for 20 years; however Total Oil Marine wish to extend the platform life by another 20 years to 2015 by gaining re-certification.

The primary objective of the re-analysis is therefore to verify that the design life of the MCP-01 platform may be safely extended by another 20 years.

The platform was demanned at the end of 1992 and there is no overnight accommodation (except for emergencies). The platform is remotely controlled from St Fergus and there are only three active gas lines; 2 no. 32" running through tunnels A, C, D and F and the 18" Texaco pipeline (formerly the "Oxy Riser").

Secondary objectives are therefore to:

- provide a tool for integrity matters;
- provide advice on structural aspects for future plans;
- allow suitable reaction in the event of any emergency;
- develop safe and cost effective inspection philosophies;
- operate weight control system;
- perform disinvestment studies.

3.0 DESCRIPTION OF MCP-01

The MCP-01 manifold and compression platform, is located in block 14/9 halfway along the route of the gas pipelines from the Frigg field on the UK/Norwegian dividing line in the North Sea to the St Fergus terminal on the Scottish coast.

The concrete structure was ordered in January 1975 by Total Oil Marine from Howard-Doris, and was built at the Skanska Doris site in Stromsad, Sweden. The structure was towed and placed in its final location in June 1976.

The pipelines from the Frigg field to St Fergus were connected to the risers installed in the structure in summer 1976. A third pipeline, from the Occidental field, was connected to the external riser during the summer of 1978.

The platform location is shown in Figure 3A. The concrete structure comprises:

- A circular raft foundation, designed to transmit the structure self-weight and the environmental forces to the sea bed. It is composed of a slab, stiffened by a series of walls arranged on a circumferential and radial pattern.

- A lobate structure, which supports the deck, and acted as a bouyancy unit during the installation of the structure. This volume also contains the sand ballast, necessary to provide stability of the structure on the sea bed. The upper part of the lobate wall is perforated to dissipate wave energy on the Jarlan principle.

- A main deck of precast concrete beam supported from steel columns standing on the lobate wall and the central concrete shaft, and a concrete manifold deck above part of the main deck.

The general arrangement of the structure is shown in Appendix A, Figure A3.

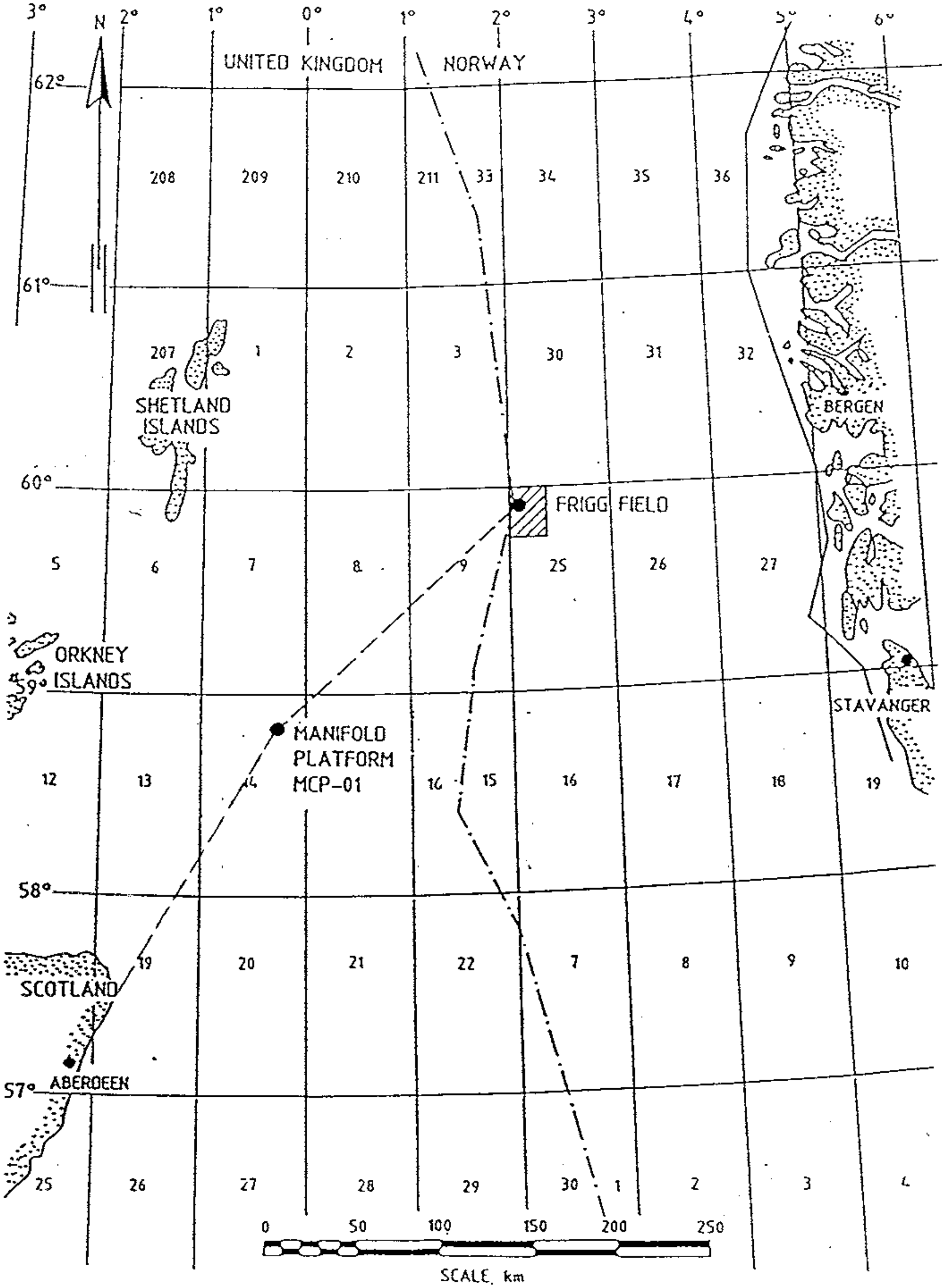


Figure 3A - Platform Location

4.0 FINITE ELEMENT MODELS (FEM)

4.1 GENERAL

The detailed analysis of the MCP01 gravity based structure (GBS) has been performed using a finite element model containing 11328 elements and 57000 active degrees of freedom. The model represents the base slab, antiscour wall (AW), perforated wall (PW), exterior wall (EW), interior wall (IW), interior diaphgram (ID), exterior diaphragm (ED), lobed wall (LW), jarlan wall (JW), tunnels, struts, and spider beams, main deck and manifold deck beams. The model does not include non-structural concrete, installation aids, riser pipes, and crane slabs, and pipe anchorage points. Outline details of the model are shown in Appendix B, Figure B5.

ANSYS finite element program (Revision 5.0) has been used to analyse the structure. In particular, four noded shell elements have been used for walls, three noded beam elements are used for beams (deck as well as spider beams in the substructure), two noded link elements are used for local strut-and tie models, and two noded spring elements are used to model the supporting soil. The element output results are explained in the following sections.

Considerable care was taken in the selection of elements and in the density of the mesh. The mesh density for shell elements has been derived from a combination of various factors mentioned below:

a) Parametric studies relating to each part of the structure in order to determine an acceptable level of 'localised' modelling error. The acceptable error level varies for each component of the structure depending on its importance.

b) Element aspect ratio (maximum side length divided by the minimum side length) have been kept to about 7 to comply with the acceptance limits obtained from standard parametric studies for flat and curved surfaces.

c) The tapering angle of the mesh has been limited to about 15 degrees.

d) Element volume ratio (defined as element volume to effective modulus) at the junctions has been limited to the range 0.5 to 2.0 to achieve a gradual change in stress resultants.

A single element has been chosen to represent the concrete slab through its thickness for the entire structure. The shape of these elements has been controlled. Distorted elements have been minimised and are limited to base slab, intersection of the base slab with the lobed wall, and at shaft intersection from star model to circular model. However, in built shape tests by ANSYS have been made for all elements.

4.2 MODELLING ASSUMPTIONS

a) Linear elastic analysis has been carried out, since this is considered sufficient to assess the global forces in the structure. However, in the places where there is a significant cracking, re-distribution may occur and generally non-linearity is inevitable. The results from linear analysis are considered to be conservative in such areas. For boat impact loading a non-linear analysis is appropriate and therefore has been carried out.

b) Centre line dimensions have generally been used to model the structure. The influence of this on the wave loading has been found to be insignificant.

c) Wall junctions have been modelled to get reasonable representation of the stiffness of the joint in the global model. Local models have been used to assess the stresses at such areas.

d) All the base slab elements are assumed to be in one horizontal plane in the global FE model. However a step change in base slab at the lobed wall interface has been investigated through a local model.

e) No support has been assumed under the central core of the GBS. This is conservative because it gives generally higher stresses in the base slab compared to support under the entire structure. Also, the scour zone in the outer core has not been modelled since it is considered to be insignificant, approximately 1% of the annular area (ref. ODE Doc 87-281).

Both effects could be subject to future studies.

f) No temporary load conditions are analysed, which may produce locked in stresses in the structure. Also the effect of construction/prestressing sequence has not been considered. This may generate secondary stresses in the structure. However such effects have been considered in the code checks by conservatively increasing the prestress losses.

g) Soil has been modelled using translational and rotational springs. This is considered to be sufficient for the assessment of structure forces. No detailed assessment of soil stresses has been made from the FE results. However global foundation stability has been assessed in section 7.3.

h) Global Analysis is not intended to provide accurate results at areas of obvious stress concentrations, however a smooth transition in these areas has been achieved in the model. Local models were used to obtain stresses in these areas.

4.3 GEOMETRY

Table 4.3 lists the element thicknesses used in the FE model. These have been derived from as built GA drawings of the structure.

Base Slab	650 to 1150mm
Antiscour Wall	750
Perforated Wall	550
Interior Wall	750
Exterior Wall	550
Interior Diaphragm	550 to 1000
Exterior Diaphragm	900 to 1400
Lobed Walls	550
Tunnel Roof	800
Tunnel Side Walls	500
Tunnel End Wall	2500
Central Shaft	
a) +0.0 to +60.0m	800
b) +60.0m to +146.0m	600
Jarlan Wall	1200
Struts/Beams	
a) +65.0m	1000*1200
b) +105.0m	1000*1600
Radial Beams/Y-Beams	1000*4000

Table 4.3 - Key Dimensions in mm

4.3.1 Base slab

The base slab 102.0 m in diameter is modelled with 1284 elements. The mesh is relatively coarse for the slab outside the the lobed wall. However the slab is spanning one way in these areas and the local discretization error is not expected to be significant. In addition, simple hand calculations followed by a finer mesh of the base slab have also been used to validate the results from the global analysis. The base slab is assumed to be in one horizontal plane, namely, a step change in cross section of the slab particularly at the intersection of base with lobed wall has not been modelled. However, this has been considered in a separated detailed model. Various parts of the base slab have thicknesses ranging from 600 mm to 1150 mm. Each node of the base slab outside the central core has been supported by a vertical soil spring and horizontal springs in locations. Soil modelling details are given in Section 4.6.

4.3.2 Walls

Antiscour wall, perforated wall and Jarlan wall have perforations in them. Therefore, the modelling of these areas have been kept relatively simple by reducing the stiffness of the walls. The reduction in the Youngs modulus of these walls depends on the ligament ratio (defined as hole spacing divided by the hole diameter). Average or smoothed stress resultants have been assessed by this method and therefore, the analysis is not intended to show localised stress concentration at the openings.

These walls have been modelled with four noded shell elements. Average area of each element is about 8 m^2. All other walls (ID,ED,IW,LW, and EW) have been modelled in the usual manner. At level 65.0m the meshing density has been increased because of a step change in cross section of the lobed wall. Also, at the interface of the columns with the Jarlan wall, density of the mesh has been increased to give a gradual change in stresses.

4.3.3 Central Shaft

The central shaft of diameter 9.6m (centerline) is modelled with shell elements. The enclosed angle of the element has been limited to about 15 degrees. The straight walls which form a star model for the shaft have also been modelled using shell elements. Each wing of the star is modelled with 5 elements. The transition zone between star shape of the shaft to the circular shape has been achieved by warping the elements at the junction. However, in order to get reasonable results, the warping factor (defined as component of the vector from the first node to the fourth node parallel to the element normal divided by the thickness of the elements) is limited to 0.1. However, detailed assessment of localised stresses has been made using a local model (see Section 11.5). The density of the mesh has been made coarse towards the top end of the shaft because of the absence of the wave load.

4.3.4 Tunnels

The tunnels act as continuous box beams. All the walls of the tunnels are spanning one way. Therefore relatively coarse mesh has been used for these elements. At the tunnel end wall (enclosure) the opening for the pipe has not been modelled in the global model. However, the tunnel behaviour has been studied using a detailed model. The details of the local model have been presented in Section 11.3.

4.3.5 Struts/Radial Beams

Struts at level +65.0, and +105.0m are modelled as 3D beam elements. Although they were originally designed as struts, the rebar/prestressing detail at the ends appears to be a moment connection. To be conservative, the joints are not modelled as pin joints in the global FE model. The radial beams at +105.0m are modelled with 3D elements. Ten divisions have been used to model the spider beams. The number of divisions has been dependent on the profile of the prestressing cables and the modelling accuracy.

4.3.6 Columns

Composite columns supporting the deck have been modelled with 3D beam elements. The stiffness of the columns has been computed using Snell's law of composites. The pinned connection between the column and the Jarlan wall has been modelled using strut and tie model. This involves linking the line elements (of the column) to the Jarlan wall shell elements by the use of link elements as shown in Appendix B, Figure B1. The intersection of the columns and concrete deck beams are modelled as moment transfer connections.

4.3.7 Deck beams

The deck beams (both main deck as well as manifold deck) have been modelled using 1100 3D beam elements. The original design assumes that they are simply supported. However, after detailed examination of the rebar and the prestressing at the connection, it was decided to use them with moment transfer connections. The number of elements along the length for each of these beams has been decided based on the modelling accuracy and the prestressing profile. Massless rigid links (i.e. moment transfer connections) have been used to connect the deck to the central shaft.

4.3.8 Junctions

The junctions of the typical walls shown in Appendix B, Figure B2, have been modelled using tapered elements. A similar approach has been considered for modelling the *nodes* of the structure. Although this type of modelling is not representative of the local nature of stress concentrations, it predicts the overall global stiffness of the walls reasonably accurately. A detailed model of such typical junctions has been studied using 3D brick elements in Section 11.6. The bending and membrane behaviour of the wall are modelled with tapered elements.

4.4 ELEMENT DESCRIPTION

4.4.1 Four Noded Shell Element (SHELL 63 in ANSYS)

The element has 4 nodes and each node has 6 degrees of freedom. The element is isoparametric with extra displacement shapes. Each node can have a different thickness. The element output contains three in-plane forces (NX,NY, and NXY), three moments (MX,MY, and MXY) and out-of-plane shear forces (VXZ, and VYZ). In deriving the out-of-plane shear forces a constant shear stress has been assumed through the thickness. Generally, this element has been used in the global model for modelling curved as well as straight walls/slabs.

4.4.2 Three Noded Beam Element (BEAM 44 in ANSYS)

The element has three nodes, two nodes defining the connectivity, and third node defining the orientation of the beam. Linear tapering of the beam can be accommodated in the element. Each of the two connecting nodes has 6 degrees of freedom. These include three translations and three rotations. The element output contains nodal forces (FX,FY, and FZ), and nodal moments (MX,MY and MZ). Thus bending, torsional and shear behaviour could be analysed. Shear deformations have been included. This element is mainly used to model the deck, struts at 65m level and radial beams at 105m level.

4.4.3 Link Element (LINK 8)

Two noded link elements have no bending capability. They act as strut/tie elements. Thus they have one degrees of freedom for each node. The element output contains axial force only.

These elements are used to model the strut and tie behaviour near the deck column/jarlan wall interface.

4.4.4 Spring Element (LINK 14)

These are two noded elements having one degree of freedom each node. The element output contains either reaction force or reaction moment. These are used to model the foundation.

4.4.5 Eight noded Shell Element (SHELL 93)

This element is similar to SHELL 63 except that it has mid-side nodes. Each node has 6 DOF. The element output consists of axial forces, bending moments, in-plane and out-of-plane shear forces. The out-of-plane shear stress varies linearly through the thickness. The transverse shear stresses are assumed to be constant through the thickness. This element was used in local models to enable to extract the results directly into the CONCRETE post-processor.

4.4.6 Eight Noded Brick (SOLID 73)

This element is used in modelling the intersections particularly the *node*. Each of the eight nodes has three translational degrees of freedom and three rotational degrees of freedom. The element output consists of the forces in X, Y and Z directions.

4.4.7 Eight Noded Non-linear Reinforced Concrete Element (SOLID 65)

This element has 8 nodes with 3 DOF for each node. It is a non-linear element. The material non-linearity is built into the element. The rebar (up to three directions) is modelled as a smeared layer at its correct depth in the element. The concrete is capable of cracking in tension, and crushing in compression. Thus the element allows plastic deformation. This element has been used to model the boat impact.

4.4.8 Eight Noded Shell (SHELL 43)

This element is similar to SHELL 63 except that it has extra shape functions.

4.5 MATERIAL PROPERTIES

Materials properties have been derived from the background document (Doc No: 3311-S-M-001).

Concrete

a)	Modulus of elasticity (E)	35 E6 kPa
b)	Poissons Ratio	0.2
c)	Shear Modulus(G)	14.6 E6 kPa
d)	Composite (steel casing) modulus	42E6 kPa

Steel

a) Modulus of Elasticity 205E6 kPa

Reduction of Elastic constants to account for holes (based on published data for closed form solutions):

		Reduction in E	Reduction in G	Effective Poisson's Ratio
a)	Antiscour Wall	0.623	0.463	0.197
b)	Perforated Wall	0.412	0.176	0.157
c)	Jarlan Wall	0.465	0.241	0.161

4.6 MODELLING OF SOIL

The soil supports for the structure have been modelled using spring elements. As shown in Appendix B, Figure B3, the supports are provided outside the central core. Therefore, no contact has been assumed for the central core. A detailed assessment of the stiffness of the soil has been carried out both using empirical rules and published data. The modulus of elasticity of the soil has been taken as 90 MPa (average value used in original 1975 design) and the Poissons ratio as 0.35. Static stiffness coefficients have been evaluated for a ring foundation (based on published data) and are summarised below:

Vertical stiffness	10430 MN/m
Translational stiffness	7740 MN/m
Rotational stiffness	18.16 E6 MNm/rad
Torsional stiffness	23.66 E6 MNm/rad

Figure B3 shows the arrangement of the linear as well as rotational springs. The vertical springs contribute to the rocking and yawing stiffness, therefore only balance needs to be applied as rotational springs. The translational springs contribute to the torsional stiffness of the GBS hence the balance needs to be applied as torsional springs.

In the model there are about 760 nodes in the outer core of the base slab. Vertical spring support has been used for all these nodes. Horizontal springs are used in NS and EW directions at 20 nodes of the base slab as shown in the figure.

4.7 DESIGN GROUPS

The FE model of the sub-structure has been divided into 42 design groups consisting of shell/ plate elements. The number of design groups for the beams (deck beams and sub-structure beams) is 87. Design group is defined as those set of elements containing similar geometrical properties, and reinforcement/prestressing characteristics. Detailed description of the design groups is given in Section 7.0. The design groups exclude *nodes.*

4.8 METHOD OF LOAD APPLICATION

Loads applied on the model include element loads, and nodal loads. The element loads mostly are used to apply the still-water condition loads , namely, hydrostatic, ballast, prestress, and dead loads. Wave loading from diffraction analysis has been transferred to the FE model in terms of nodal loads and moments.

4.9 VALIDATION OF MODEL

The validation of the model has been carried out in two stages. Only the first stage is described here. It includes validation/calibration of the model results with field monitored data. Second stage involves verification of the analysis for a given set of loadings with hand calculated results. This is included in Section 6.

4.9.1 Sway Frequencies

The sway frequencies of the structure *inter alia* depends mainly on the soil stiffness, and the structure stiffness. Therefore, it is necessary to perform a coupled 3 DOF modal analysis to determine the natural frequencies of the structure. Appendix B, Figure B4, shows the model for dynamic analysis. The soil stiffness from Section 4.6 is used in the model. The structure stiffness has been obtained from parametric studies. This was achieved by running the model with lateral forces and computing the displacement at various levels. From this displacement the stiffness of the sub-structure, the stiffness of the deck have been obtained. The other variables for the dynamic analysis include, the mass moment of inertia of the structure (includes component from soil, structure mass, and the hydrodynamic mass). Table 4.9.1 presents results from the 3 DOF modal analysis. Thus the computed natural period of the structure is 0.73 Hz as against an observed average natural period of 0.75 Hz which shows very good agreement

Table 4.9.1 - Frequencies in Hertz

Effect of Added Mass

$Kx = 7740\text{x}10^3$ kN/M (soil stiffness) $K_{x\theta} = 18.16\text{x}10^9$ kNM/rad (soil stiffness)
$K_n = 2110\text{x}10^3$ kN/M (structure stiffness) $J_\theta = 400\text{x}10^6$ tm^2 (mass moment of Intertia of structure)

M_2 = 20000t (Deck wt)

	f_1 (mode 1)	f_2 (mode 2)	f_3 (mode 3)
M_1 = 300,000t (weight of	0.70	0.85	2.39
M_1 = 250,000t structure)	0.72	0.91	2.38
M_1 = 200,000t	0.72	1.0	2.48

Effect of added mass moment of Inertia

K_x, K_{xe}, K_h, M_2 as above

M_1 = 300,000 t (weight of structure incl. hydrodynamic mass)

	f_1 (mode 1)	f_2 (mode 2)	f_3 (mode 3)
$J_e = 200 \times 10^6$ tm²	0.70	0.85	2.39
$J_e = 400 \times 10^6$ tm²	0.73	0.92	2.99

Effect of Soil

$K_h = 2160 \times 10^3$ $J_e = 400 \times 10^6$ tm²
M_1 = 300,000 t M_2 = 20,000T

Case (a) $K_h = 7{,}740 \times 10^3$ kN/m $K_{xe} = 18.16 \times 10^9$ kNM/rad
Case (b) $K_x = 10{,}000 \times 10^3$ kN/M $K_{xe} = 27 \times 10^9$ kNM/rad

	f_1 (mode 1)	f_2 (mode 2)	f_3 (mode 3)
Case (a)	0.70	0.85	2.39
Case (b)	0.82	1.01	2.46

The FE model has been validated for stiffness by comparison with site measured natural frequencies which have not changed significantly over a number of years. The natural frequencies, computed with a simple 2 degrees of freedom model using FE model results and original 1975 soil stiffness, are 5 - 7.5% less than site measured natural frequencies. This is considered to be a good level of agreement and conservative from an assessment point of view. A lower bound soil stiffness has been used in the model in order to conversatively assess the forces in the structure.

They also confirm that the original soil stiffness used in 1975 is satisfactory for the assessment of forces in the structure.

4.9.2 Relative Displacement Ratios

In order to get confidence in the global stiffness of the sub-structure, a separate set of calculations have been prepared. For the lateral forces (both in NS and EW directions) on the structure, the deck displacement and the base displacement in sway motion have been compared. Typical analysis results are presented in Table 4.9.2 showing ratios (deck and base) ranging from 10 to 12. This compares very well with average values of 8.0 to 12.0 observed in monitoring. This shows that the global structure stiffness assessed from our model , particularly the sub-structure is underestimated by 10 to 15%.

Table 4.9.2 Relative Displacement Ratios

Loading in EW Direction (Displacements in m, rotations in radians)

	X	Y	Z	ROTX	ROTY	ROTZ
(a) Base Slab Node	0.143E-1	0.341E-4	-0.35E-1	-0.207E-5	0.107E-7	0.488E-6
(b) Deck Node	0.176	-0.277E-3	-0.342E-3	-0.764E-5	0.126E-2	0.130E-3
(c) Top of C/S	0.124	-0.672E-4	-0.162E-4	0.109E-4	0.164E-2	-0.100E-4

Ratio $\frac{(b)\text{-}x}{(a)\text{-}x}$ 12.4

Ratio $\frac{(c)\text{-}x}{(b)\text{-}x}$ 1.22

Loading in NS Direction

	X	Y	Z	ROTX	ROTY	ROTZ
(a) Base Slab Node	0.26E-51	0.146E-14	0.296E-1	0.102E-2	0.994E-6	-0.811E-6
(b) Deck Node	-0.37E-3	-0.187E-3	-0.318E-1	-0.89E-3	0.535E-3	-0.208E-6
c) Top of C/S	0.588E-3	0.22095	0.847E-2	0.173E-2	-0.755E-5	0.265E-5

Ratio $\frac{(b)\text{-}x}{(a)\text{-}x}$ 12.8

Ratio $\frac{(c)\text{-}x}{(b)\text{-}x}$ 1.18

Also, for a typical lateral loading the deck displacements have been compared with the displacements at the top of the shaft. Typical results are presented in Table 4.9.2. Analysis gave a ratio of shaft displacement to deck displacement of 1.2 to 1.4. This again compares with average observed values of 1.2 to 1.4 (In NS and EW).

The analysis also predicted about 36mm displacement in EW direction and about 50mm in the NS direction for an operating wave of 15.4m height. This again compares with observed displacement of 31mm in EW direction (2mm per one metre of wave height is quoted in the reports. Therefore for an operating wave height of 15.4 m, the expected displacement is 31mm). This shows that the structure stiffness, the deck stiffness, and finally the soil stiffness are satisfactory in the model.

4.9.3 Torsional Frequency

The analysis reveals the torsional mass moment of inertia of the structure. Added to this are the components from the hydrodynamic mass and the ballast mass. Coupling of the soils and structure could occur. Therefore, it was necessary to use a 2 DOF model to calculate the coupled torsional frequency. Hand calculations showed that the total torsional mass moment of inertia is between 200E6 and 300E6 tm^2. The average torsional stiffness of the structure is 4700 E6 kNm/rad. Coupled 2 DOF dynamic analysis reveals that the first coupled torsional frequency is between 0.71 Hz and 0.75 Hz. This compares very well with the observed torsional frequency of 0.75 Hz.

5.0 LOADING

All loading on the Platform was re-assessed as part of the re-analysis. This included:

- Dead & Live Loads
- Marine Growth
- Wind
- Snow & Ice
- Wave & Current Details
- Prestress
- Temperature

These were combined into 16 basic combinations that covered operational and extreme waves for both ULS and SLS conditions.

5.1 DEAD AND LIVE LOADS

5.1.1 Topsides Modules and Equipment

As part of the re-analysis, the weight (both dead and live) of all topsides modules and equipment has been completely re-assessed. In particular the effects of the change in status of various modules on demanning have been taken into account. Loads were re-assessed on the basis of documentation from the original installation, subsequent modifications (including the demanning project) and discussions with TOTAL personnel.

The current total dead and live load of topsides equipment were calculated to be respectively 12969t and 1722t (total 14691t). A breakdown of these figures is given in Table 5.1.1.

The equivalent weights from the 1986 weight control report, were respectively 14711t and 756t (total 15467t).

It is considered that the discrepancy is due to the effects of demanning, rationalisation of dead loads and the application of more representative live loads on the laydown areas.

The recalculated loads were applied to the FE model as either UDL's or point loads. Module reactions were calculated from the revised module weights and the original reaction distribution shown on the following drawings:

Main Deck - North Area Loads at Level 123.0	MP-5009-M4-15-01
Main Deck - South Area Loads at Level 123.0	MP-5009-M4-15-02
Main Deck - Loads on Underdeck	MP-5009-M4-15-03
Main Deck - Loads at Level 133.0	MP-5009-M4-15-04

The steel-deck plating over the northern half of the deck was included with the topsides modules and equipment. The weight of this steel decking was calculated to be approximately 2 kN/m2, which was cross checked against the weight of the individual panels as installed and found to be in agreement. The 2kN/m^2 was then apportioned to beams in primarily the E-W direction. The total weight of secondary deck steel applied was 644t.

DESCRIPTION	DEAD LOAD, t	LIVE LOAD,t	TOTAL WT., t	COMMENTS
COMPRESSION MODULE	1518	100	1618	decomissioned
COMPRESSION MODULE	1518	100	1618	decomissioned
QUARTERS(ELQ) & HELIDECK	1513	62	1575	0.5KN/M2 LL on 4 Floors(emergency only)
LOAD REPARTITION STRUCTURE	1224	115	1339	2.5KN/M2 LL Laydown area
GENERATORS MODULE	937	100	1037	Includes switchgear, power(2 gen), passage control room, constr. offices
SEPARATION MODULE	858	81	939	5 KN/M2 LL
MAIN DECK SEC STEEL 1.10/D.I	644		644	
UTILITIES	554	40	594	40T LL Potable water
MANIFOLD PHASE 1	558	20	578	decomissioned nominal LL only
MANIFOLD PHASE 2	558	20	578	as above
INTERFACE-PIPING	361		361	
NORTH EAST LAYDOWN		350	350	5KN/M2 LL
EAST PEDESTAL CRANE	234	20	254	capacity reduced to 10T
OCCIDENTAL MODULE	148	100	248	
WEST PEDESTAL CRANE	234		234	
PERM LIVING QUARTERS	192	23	215	1.5kN/m2 on 1 floor
SUMP ,SLOP TANKS	66	142	208	
DEISEL OIL TANK 4-6/C-D	66	126	192	reduced requirement
6-7/H-I	183	0	183	
DIVING SYSTEM(PERM)	75	75	150	
FLARE BOOM	145		145	
VALVE MANIFOLD SKID	122		122	
EMERGENCY GEN MODULE	106	10	116	
INTERFACE -STRUCTURAL	110		110	
DEISEL OIL TANK 3-4/C-D	40	54	94	as above
4-6/H-I	91	0	91	
TELECOMS TOWER	86		86	
PUMPHOUSE A	77		77	Includes caisson&riser
PUMPHOUSE B	77		77	
UNDERDECK BASKET GH/4,6	49	25	74	1.5KN/M2 LL(storage only)
UNDERDECK BASKET FG/4,6	46	25	71	1.5KN/M2 LL(storage only)
NORTH CONTAINER LAYDOWN		70	70	5KN/M2 LL
RECREATION ROOM	67	0	67	
LOCKER ROOM	40	15	55	Includes PCP mods
UNDERDECK BASKET GH/1,4	31	15	46	1.5KN/M2 LL(storage only)
LIFEBOATS1&2	44		44	
COLD STORE	27	15	42	
FIRE PUMP MODULE	40	0	40	
DIVING SYSTEM(MOBILE)	38	0	38	
SPARE PARTS	16	9	25	
NEW LIFEBOAT	22		22	From Doris lifeboat calcs
A-B/3-5	21	0	21	
NITROGEN GENERATION	18	2	20	
EAST LAYDOWN	15	5	20	
BLOWDOWN CONTROL	20	0	20	
GAS CONDENSATE	20	0	20	
UTILITY GAS SKID	20	0	20	
WEST FLOTEL LANDING	19		19	Refer PCP weight control
EAST FLOTEL LANDING	19		19	Assumed the same as west flotel
CENTRAL STAIRWAY	16		16	
FUEL GAS PACKAGE(GEN)	15	1	16	
TURBINE EXHAUSTS	15		15	
FUEL GAS PACKAGE(COMP)	13	1	14	
FUEL GAS PACKAGE(COMP)	13	1	14	
INST AIR COMPRESSOR ROOM	10		10	
UNDERDECK STORAGE TANK	10	0	10	
PILOT GAS SKID	10	0	10	As given by TOM
TOTAL	12969	1722	14691	

Table 5.1.1. - Total Dead and Live Load

5.1.2 Sub-Structure

The volume of concrete in the sub-structure, including the central shaft to +147.8m, was re-calculated from the as-built drawings and found to be approximately 54,000m³, which compares well to the value of 53,400m³ given in document D5237 for the sub-structure less the central shaft between 105 and 147.8. Assuming a dry density of reinforced concrete of 2.6t/m³, a submerged density of 1.6t/m³ and allowing for the volumes displaced by the tunnels and shafts, this gave a submerged weight for the structure of 83,324t for a water depth of 93m. A breakdown of these figures is given in Table 5.1.2.A.

WATER DEPTH : 93

CONCRETE DENSITY : 2.6

COMPONENT	MIN. EL.	MAX EL.	CONC.VOL M3	VOL.DISPL. M2	DRY WT., T	SUBMERGED %	SUBMERGED WT.
ANTISCOUR WALL	0	1	2864	2864	7446.4	100	4582.4
PERFORATED WALL	0	1	1824	1824	4742.4	100	2918.4
EXTERIOR WALL	0	1	1826	1826	4747.6	100	2921.6
INTERIOR WALL	0	1	1417	1417	3684.2	100	2267.2
LOBED WALL	1.3	15	1393	1393	3621.8	100	2228.8
NODES	1.65	15	1100	1100	2860	100	1760
EXTERIOR DIAPHRAGMS	0	1	6443	6443	16751.8	100	10308.8
INTERIOR DIAPHRAGMS	0	1	4809	4809	12503.4	100	7694.4
SHAFT	1.3	68	1642	6336.5	4269.2	100	-2087.3
SHAFT	68	93	615.75	2375	1600.95	100	-774.05
BASE SLAB	0	1	6900	6900	17940	100	11040
TUNNELS	0	1	1001.5	2490	2603.9	100	113.9
LOBED WALL	15	65	5082	5082	13213.2	100	8131.2
NODES	15	31	1470.3	1470.3	3822.78	100	2352.48
NODES	31	68	2960	2960	7696	100	4736
INTERIOR DIAPHRAGM	65	68	324	324	842.4	100	518.4
BEAMS	65	68	560	560	1456	100	896
LOBED WALL	65	68	669	669	1739.4	100	1070.4
LOBED WALL	68	93	4075	4075	10595	100	6520
NODES	68	93	1575	1575	4095	100	2520
LOBED WALL	93	105	2117	2117	5504.2	0	5504.2
NODES	93	105	756	756	1965.6	0	1965.6
BEAMS	105	105	762	762	1981.2	0	1981.2
SHAFT	93	105	296	1140	769.6	0	769.6
PEDESTALS	105	105	200	200	520	0	520
SHAFT	105	147.8	1094	4066	2844.4	0	2844.4
TOTAL:			53776	65534	139816		83324

Table 5.1.2A - Sub-Structure Weight

Variation in water depth has a small effect on the submerged weight, as shown in Table 5.1.2.B, and there is only a 1140t decrease in weight as the water depth increases from 92.47m to 95.90m.

Water Depth	Submerged Wt.,t
92.47	83494
93.00	83324
94.00	82989
95.90	82354

Table 5.1.2B - Variation of submerged Weight with Water Depth

The dead-weight of the sub-structure was calculated automatically by ANSYS based on the element thicknesses. This was then factored to give the appropriate submerged weight for the water depth under consideration.

5.1.3 Ballast

The ballast was placed in the structure in two stages:

- prior to towing
- at the final site

These ballast quantities were re-calculated using the following densities:

Item	Density t/m^3	Ref Doc No
Wet Sand	1.60	D1077
Saturated Sand	1.90	D1077
Submerged Sand	0.88	D1161
Concrete Slab	2.38	D1077

These figures gave recalculated values of 25086t for the ballast placed prior to towing and 76820t for ballast placed offshore.

However, the unsubmerged weight of sand and the concrete slab placed before installation is quoted as 53,623t in document D1077, and the submerged weight as 30,000t and 25,800t in documents D5237 and D1022 respectively. It was therefore decided to apply a conservative value of 25,800t in the re-analysis.

The value of ballast placed offshore was calculated to be 87,295m^3 equivalent to a submerged weight of 76,820t. However the volume of sand recorded in document D1146 as being placed offshore was 91,047m^3, which was established by measurement of the elevation of the top of the ballast and allows for 10% loss through the breakwater wall from the amount pumped. This is equivalent to a submerged weight of 80,120t assuming a density of 0.88t/m^2. It was therefore decided to apply 80,120t in the re-analysis.

The total submerged weight of ballast applied in the re-analysis was 105,920t. A comparison of the ballast weights is shown in Table 5.1.3.

	Document No.			Re-analysis	
	D5237	D1022	D1146	Calculated	Applied
TOWING BALLAST	30000	25800		25086	25800
OFFSHORE BALLAST	86000		80120	76819	80120
TOTAL :	116000	N/A	N/A	101905	105920

Table 5.1.3 - Ballast Comparison

The ballast load was applied to the FE model as pressures as indicated in Figure 5.1.3. A uniform pressure of $44t/m^2$ was applied to the base slab and a triangular pressure distribution, varying linearily between $0t/m^2$ at +50.0 and $26.4t/m^2$ at the base slab, was applied to the lobated wall and the central shaft. The basic load case generated was then factored to give the correct overall ballast weight.

No silo effects were included as the ballast was totally submerged.

5.1.4 Deck Structure

5.1.4.1 Main Deck Beams

The deadweight of the concrete deck beams was re-calculated assuming a concrete density of $2.6t/m^3$. This gave a total weight of 5962t, which compared to the originally (1975) calculated weight of 6800t. It was decided conservatively to apply the weight of 6800t. The deck beam weight was applied to the FE model as UDL's.

5.1.4.2 Manifold Deck

The weight of the concrete manifold beams and manifold columns was re-calculated as for the main deck beams. The calculated weight was 873t compared to the originally (1975) calculated weight of 1000t. It was decided conservatively to apply a weight of 1000t. The beam weights were applied as UDL's and the column weights as point loads to the FE model.

5.1.4.3 Main Deck Columns

The weight of the main deck columns was re-calculated assuming a concrete density of $2.6t/m^3$. This gave a total weight of 1082t, compared to the originally (1975) calculated weight of 1400t. It was decided conservatively to apply a weight of 1400t. This was applied as point loads at the relevant locations.

5.1.5 On-Bottom Weight

The on-bottom weight corresponding to the weights calculated in Sections 5.1.1 to 5.1.4 was compared to the originally specified value of 205,000t. The results of the comparison are shown in Table 5.1.4.

COMPONENT OF WEIGHT	RE-ANALYSIS		RE-ANALYSIS		ORIG. CALCS.	ORIG. CALCS.
	CALCULATED	APPLIED	CALCULATED	APPLIED	ref. D1161	+ rev LOADS
WATER DEPTH	92.47	92.47	95.9	95.9	94	94
SUB-STRUCTURE	83494	86570	82354	85670		
MAIN DECK BEAMS	5962	6800	5962	6800		
DECK COLUMNS	1082	1400	1082	1400		
MANIFOLD DECK/COLUMNS	873	1000	873	1000		
STEEL DECK PLATING	644	644	644	644		
TOWING BALLAST	25086	25800	25086	25800		
SUB-TOTAL:	117141	122214	116001	121314	121007	121007
OFFSHORE BALLAST	76819	80120	76819	80120	79154	80120
TOPSIDES DEAD LOAD	12325	12325	12325	12325	4839	12325
OXY RISER	130	130	130	130		130
ON BOTTOM WT.	206415	214739	205275	213889	205000	213582

NOTE :
1. Sub-structure weight is submerged weight and includes weight of concrete below 105.0m plus the whole of the central shaft.

Table 5.1.4. - Comparison of On-Bottom Weights

The comparison indicates that the applied on-bottom weights of 214,739t and 213,839t are comparable to the original value of 205,000t adjusted to 213,583t to allow for overballasting and increased topsides loading.

5.2 MARINE GROWTH

5.2.1 Methodology

The 1991 Marine Fouling assessment by AURIS provides the latest marine growth data, which presents current as well as predicted thicknesses for 1999.

The report states that marine growth on MCP-01 is now tending towards a long term equilibrium, i.e. the forecast for 1999 can be taken as the long term equilibrium values.

As soft marine growth compresses to varying degrees according to environmental conditions, thicknesses are presented for both extreme and operating conditions.

The report provides data for the west face only. It was assumed that thicknesses will be similar for other external faces. No data is presented for inside the Jarlan holes. However it was assumed that thicknesses would be the same as for the external face. This was considered to be conservative as marine growth is likely to be less thick inside the holes due to the increased water velocities.

Forecast long-term thicknesses were used in the re-analysis. Marine growth was not considered in the original design.

5.2.2 Effect on Wave Loads

The most significant effect of marine growth on wave loading is due to the increased drag and restriction at the Jarlan holes.

Average marine growth thicknesses were therefore calculated for every 5 metre band of the Jarlan wall between 105.0 and +65.00 based on the forecast 1999 values from the AURIS report. These are presented on Table 5.2.2 for both operating and extreme conditions.

5 METRE BANDS		TOTAL THICKNESS	
TOP ABOVE SEABED	BOTTOM ABOVE SEABED	OPERATIONAL (mm)	EXTREME (mm)
105	100	0	0
100	95	0	0
95	90	44	38
90	85	45	38
85	80	47	38
80	75	53	39
75	70	65	41
70	65	76	42

Table 5.2.2. Predicted Marine Growth Thicknesses

5.2.3. Weight of Marine Growth

It was assumed that the weight of marine growth was insiginificant compared to the overall sub-structure deadweight. This is because the thickness of hard growth is minimal and the more abundant soft growth has a neutral buoyancy.

5.3 WIND

5.3.1 Methodology

The topsides wind loads on the platform have been re-assessed, using the wind speeds given in Marex Report 1167 Volume 1 and taking into consideration any revision to the topsides configuration.

Shape coefficients were taken from DnV Appendix B and CP3 Chapter 5.

Wind forces were calculated for each of the four cardinal directions and then vectored for intermediate directions.

Module reactions were calculated assuming that the modules were rigid and that there were no torsional effects, with support dimensions being taken from ODE drawings MP5009-15.01 & 15.02.

In addition to the modules, wind loads were calculated for the concrete deck beams, the Jarlan wall and the central shaft between the underside of the deck and the top of the Jarlan wall.

5.3.2 Wind Speeds

The following 3 second gust wind speeds at el. +146.2m have been used for the re-analysis:

Extreme	:	53.8 m/s
Operating	:	37.7 m/s

The 3 second gust, although not relevant for large modules, has conservatively been taken. Similarily the wind speed has been taken at el +146.2m (top of helideck).

Directionality of wind speed has also conservatively been ignored.

Wind forces for the operating case have been calculated by factoring those calculated for the extreme case by 0.7, which corresponds to the ratio of the squares of the respective wind speeds.

Extreme and operating wind speeds (one minute gust) at +10m above sea level used in the original design were 53m/s and 36m/s respectively.

5.3.3 Results

The wind loads calculated for each component are shown in Table 5.3.3 for each of the four cardinal directions. The maximum wind force calculated was 587 tonnes for an easterly wind and represents less than 0.9% of the extreme wave base shear. Thus the omission of wind forces in the original sub-structure design would appear to be justified.

DESCRIPTION	EXTREME WIND LOADS, KN											
	EAST			WEST			NORTH			SOUTH		
	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz	Fx	Fy	Fz
LRS MODULE	-1505	2296	-1470	1274	-2296	1365	-119	973	1617	119	-973	-1617
OXY RISER MODULE	-66	0	0	0	0	0	0	-88	0	0	0	0
MCC GENERATOR	0	0	0	467	0	0	0	0	0	0	322	0
EAST VENT STACK FORCES	-345	0	0	345	0	0	0	-325	0	0	325	0
UTILITIES MODULE	-184.8	0	0	0	0	0	0	-1107.6	0	0	0	0
QUARTERS MODULE	-240	0	0	0	0	0	0	0	0	0	1299	0
MANIFOLD LEVEL 133	-166	0	1	166	0	1	0	-700	0	0	700	0
JARLAN WALL	-1108	0	0	1108	0	0	0	-1108	0	0	1108	0
DECK BEAMS	-415	0	0	415	0	0	0	-415	0	0	415	0
CENTRAL CORE	-325	0	0	325	0	0	0	-325	0	0	325	0
PEDESTAL CRANES	-1510	0	0	1510	0	0	0	-1510	0	0	1510	0
TOTAL:	-5865	2296	-1469	5610	-2296	1366	-119	-4606	1617	119	5031	-1617

Table 5.3.3 - Wind Load Summary

5.4 SNOW & ICE

The Department of Energy guidance notes give maximum extreme values of snow and ice loads. However, they state that maximum accumulations of snow and ice for 50 year return periods are very difficult to predict.

In view of this and the fact that maximum loads were expected to be low compared to overall dead loads, it was decided to apply the loading as a UDL over the whole deck area.

The maximum thickness of wet snow was taken as 200mm with a density of 100 kg/m^3, giving a UDL of 0.2 kN/m^2. This was conservatively increased to 0.5kN/m^2 to allow for any accumulations on vertical surfaces.

The UDL representing the snow and ice was then applied as equivalent line loads to the concrete deck beams, giving a total snow and ice load of 197 tonnes.

Snow and ice loads on the top of the Jarlan wall were ignored.

5.5 WAVE AND CURRENT LOADING

5.5.1 Background

The hydrodynamic force induced by waves on a fixed structure is usually considered as the sum of the drag force and the diffraction force. However for structures whose diameter is large relative to the wave length, the diffraction force can represent up to 95% of the total wave force. In these cases, calculation of the hydrodynamic force is carried out by means of diffraction analysis, using computer programs such as DIODORE.

In the case of MCP-01, although overall the diffraction force is dominating, the drag force of the Jarlan wall cannot be neglected. Calculation of the latter requires specialist techniques and for MCP-01 an in-house computer program called LAHOULA was used.

In addition, as the meshing used for the FE model was very different to that defined to compute the hydrodynamic forces, it was necessary to use the interface program DIOFEC to transform the hydrodynamic pressures into forces to be applied to the nodes of the FE model.

The methodology is described in detail in Section 5.5.3.

5.5.2 Design Criteria

With reference to MAREX report No. 1167 the following criteria were selected for the re-analysis.

5.5.2.1 Sea Levels

The maximum and minimum seawater level for different environmental conditions given by MAREX were as follows:

Condition	SWL
100 year maximum	95.92
HAT	95.26
LAT	93.00
100 year minimum	92.47

Maximum and minimum seawater levels for the re-analysis were therefore conservatively taken as follows for both extreme and operating conditions:

Condition	SWL
maximum	95.92m
minimum	92.47m

5.5.2.2 Wave Height and Periods

For the extreme (storm) condition, corresponding to a return period of 100 years, the following wave height and periods were taken:

H_{max} = 26.4m
T_{max} in a range between 13.5 sec to 18.1 sec

For the operating condition, corresponding to a return period of one month, the following wave height and periods were taken:

H_{max} = 15.5m
T_{max} in a range from 10.3 sec to13.8 sec

It should be noted that the operating wave characteristics were extrapolated from data given in the MAREX report.

5.5.2.3 Wave Incidences

The hydrodynamic analysis was performed for 7 wave incidences (30°, 120°, 210°,300°,315°,330°,345°). Directionality of wave height was conservatively ignored.

Wave incidence definition is shown in Figure 5.5.2.3.

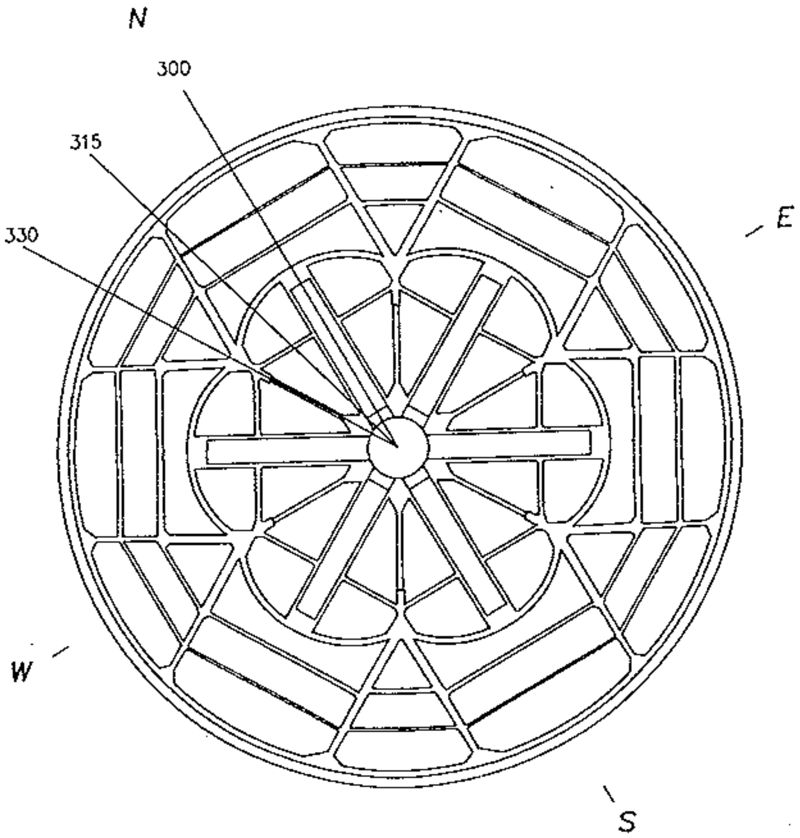


Figure 5.5.2.3 - Wave Incidences

5.5.2.4 Phase Angles

In order to find the most critical phase angle for each wave type, it was decided that hydrodynamic pressures would be calculated at 15° phase angle intervals.

Phase angle definition is shown in Figure 5.5.2.4.

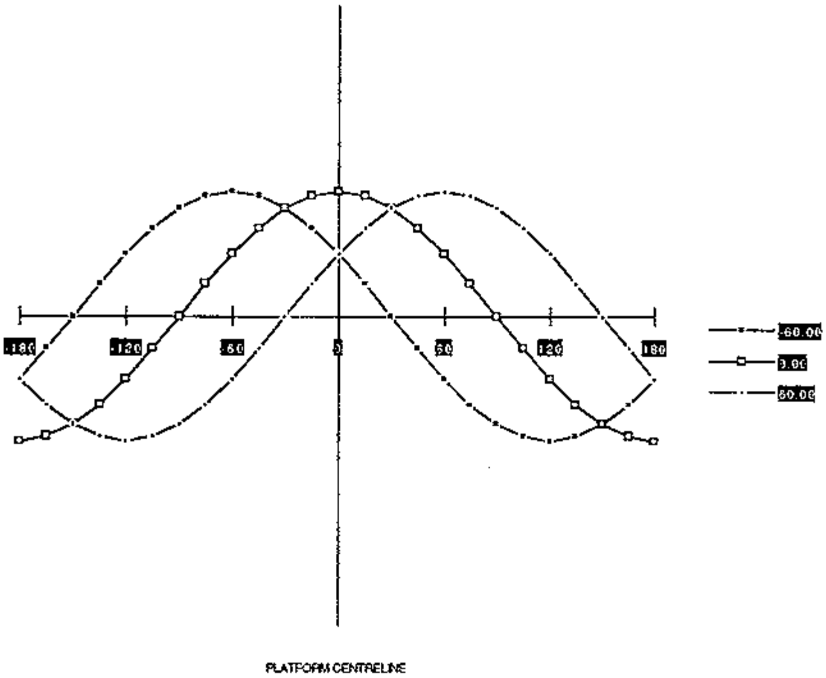


Figure 5.5.2.4 - Phase Angle Definition

5.5.3 Methodology

5.5.3.1 Outline

Hydrodynamically the platform was considered to comprise of 5 parts:

Part	**Location**
1.	the lower part up to 15m above sea bed;
2.	the lobated wall from level +15m to the lower level of the Jarlan wall + 65.0m;
3.	the Jarlan wall;
4.	the central shaft within the lobated wall;
5.	radial beams and deck columns.

The hydrodynamic forces were determined by DIODORE for parts 1 and 2, by LAHOULA for part 3, by McCamy-Fuchs formulation for part 4 using a reduced wave height, and by HOULJACK for part 5. The methodology is illustrated diagramatically in Figure 5.5.3.A. with full details for each part being given in Sections 5.5.3.2 to 5.5.3.5.

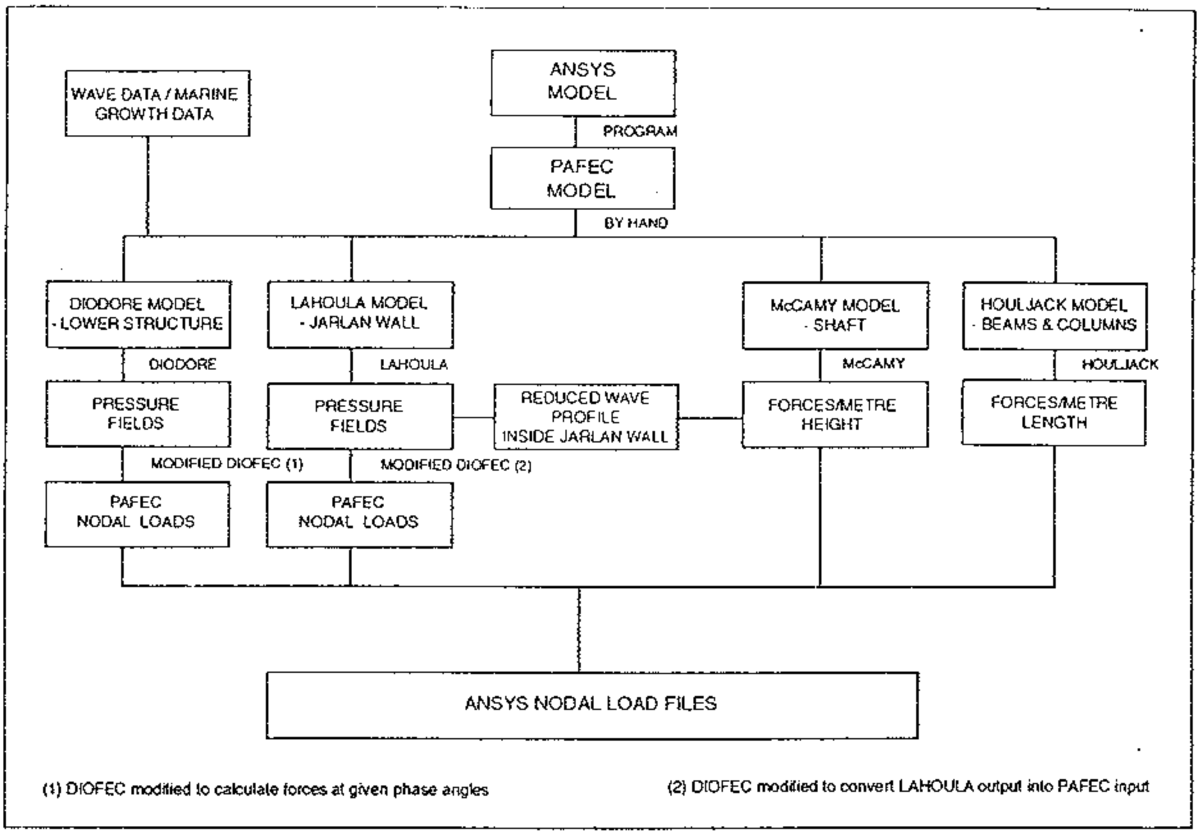


Figure 5.5.3.A - Wave Loading Methodology

The objective of the hydrodynamic analysis was to calculate the maximum hydrodynamic forces acting on the sub-structure that covered variations in:

- water depths;
- wave heights;
- wave periods;
- phase angles;
- wave incidences.

5.5.3.2 Lower part of the platform and lobated wall (Parts 1 and 2)

Hydrodynamic forces on the lower part of the platform and the lobated wall were calculated by applying regular Airy's waves to a DIODORE model.

This model was characterised by the following two major features:

- the perforated wall and the anti-scour wall were replaced by solid walls;
- a "dummy" horizontal slab (roof) enclosed the space between the anti-scour wall and the lobated wall at level +15m.

These features were necessary due to the impossibility of representing in DIODORE a perforated wall and the difficulty of representing satisfactorily the pressure distribution within the complex and small region (relative to wave lengths) located at the base of the structure between the anti-scour wall, and the lobated wall.

Details of the DIODORE model are shown in Figure 5.5.3.2.

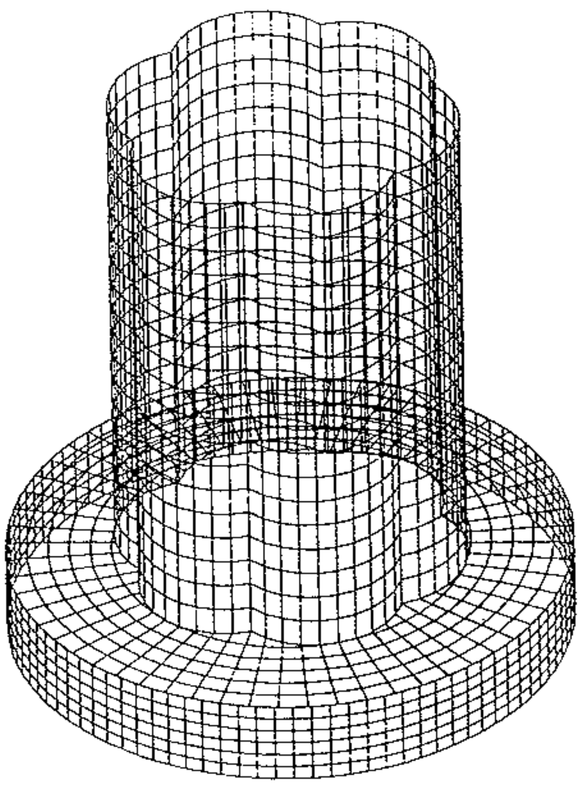


Figure 5.5.3.2 - DIODORE Model

It should be noted that the Jarlan wall above 65m was modelled to take into account its influence on the structure below and that no hydrodynamic forces were calculated for the exterior diaphragms or the lobated wall between 0 and +15m, as forces on these elements are considered to be small compared to total forces.

DIODORE was run for a 1 metre wave amplitude for both maximum and minimum water levels and for a range of periods (10.3 to 18.1 seconds) that covered minimum and maximum periods for both operating and extreme conditions.

The hydrodynamic pressures were then factored for the relevant wave height, and real/imaginary parts combined appropriately to give hydrodynamic pressures at 15° phase angle increments.

Finally, hydrodynamic pressures for selected wave cases were transferred to the FE model using DIOFEC, with pressures on the roof at level +15m being transferred to the corresponding nodes on the base slab in the FE model.

5.5.3.3 Jarlan Wall (Part 3)

The pressure distribution around the Jarlan wall was computed by applying regular Airy's waves to a LAHOULA model. The model consisted of an equivalent cylinder of 60m diameter with an infinite height.

LAHOULA computes wave kinematics, water elevations and pressure distribution around a porous vertical cylinder. The methodology can be summarised as follows:

- both incident wave field and diffracted wave field are written in polar co-ordinates (Bessel's series expansions);
- out of the cylinder, the potential is assumed to be the sum of the incident flow and a part of a diffracted wave flow;
- within the cylinder the potential is expressed in a series expansion similar to incident potential one, with unknown complex transmission co-efficients;
- the mean flux through the perforated wall is equal to zero;
- the wave kinematics around two cylindrical surfaces, located respectively at a small distance from the outer and inner face of the Jarlan wall, are related by a non-linear damping equation taking into account the energy losses due to friction in the holes, jet mixing and viscous damping;
- the non-linear equations are linearized and solved by a least square method, so all reflection and transmission co-efficients are determined.

The program was calibrated against the original model test results from 1975.

For the re-analysis of MCP-01, the following thicknesses of marine growth in the Jarlan holes have been taken:

Storm condition	30mm
Operating condition	42mm

As the model is a cylinder, the same resultant is expected whatever the wave incidence, and therefore only one wave incidence has been considered. The analysis was therefore performed for:

- 2 water depths;
- 2 wave heights;
- 8 wave periods (10.3, 11.8, 12.6, 13.8, 15.0, 16.5, 18.1 seconds);
- 24 phase angles.

A specially developed version of the DIOFEC program was then used to transform the pressure into nodal forces and moments for application to the FE model.

5.5.3.4 Central Shaft (Part 4)

The horizontal force per unit length of shaft was computed according to McCamy-Fuch's formulation, using the in-house program MCAMY, for regular Airy's waves with a reduced wave height in order to take into account the protective effect of the Jarlan wall.

Reduced wave heights corresponding to both extreme and operating waves were calculated as follows from the LAHOULA results:

Hmax	**Reduced Height**
26.4m	25.5m
15.5m	15.1m

It should be noted that as the space between the lobated wall and the shaft is filled with sand up to level +50m, no hydrodynamic pressure was calculated below this level.

The horizontal force at each level was then distributed by hand to the corresponding nodes of the FE model.

The analysis was only performed for the selected wave cases as the calculated forces only represented a small proportion of the overall wave force.

5.5.3.5 Radial Beams and Deck Columns (Part 5)

The hydrodynamic forces applied to the radial beams and deck columns were computed using the in-house computer program HOULJACK for regular Airy's waves. HOULJACK determines the water particle velocity and then applies Morrisons equation to calculate the force on the member.

It was assumed that the deck columns were subjected to the full wave height whilst the radial and strut beams inside the Jarlan wall were only subjected to the reduced wave heights as detailed in Section 5.5.3.4.

5.5.4 Selection of Critical Cases

The various combinations of wave height, period, phase and water depth analysed hydrodynamically are shown in Table 5.5.4

WAVE TYPE	WAVE HT	WATER LEVEL	PERIOD	PHASE		DIRECTION
Extreme	**26.4**	**95.90**	13.8	-165	15	**300**
			15.0	-150	30	**315**
			16.5	-135	45	**330**
			18.1	-120	60	345
				-105	75	30
				-90	90	120
				-75	105	210
				-60	120	
				-45	135	(7 No.)
				-30	150	
				-15	165	
				0	180	
	26.4	**92.47**	13.8	-165	15	**300**
			15.0	-150	30	**315**
			16.5	-135	45	**330**
			18.1	-120	60	345
				-105	75	30
				-90	90	120
				-75	105	210
				-60	120	
				-45	135	(7 No.)
				-30	150	
				-15	165	
				0	180	
Operational	**15.5**	**95.90**	10.3	-165	15	**300**
			11.8	-150	30	**315**
			12.6	-135	45	**330**
			13.8	-120	60	345
				-105	75	30
				-90	90	120
				-75	105	210
				-60	120	
				-45	135	(7 No.)
				-30	150	
				-15	165	
				0	180	
	15.5	**92.47**	10.3	-165	15	**300**
			11.8	-150	30	**315**
			12.6	-135	45	**330**
			13.8	-120	60	345
				-105	75	30
				-90	90	120
				-75	105	210
				-60	120	
				-45	135	(7 No.)
				-30	150	
				-15	165	
				0	180	

BOLD indicates wave cases selected for application to FE model

Table 5.5.4 - Summary of Diodore/Lahoula Runs for Wave Case Pre-Selection

From these combinations, it was necessary to select the most unfavourable for application to the FE model according to the following criteria:

- maximum base shear;
- maximum overturning moment;
- maximum pressure on Jarlan wall;

Due to the slight non-symmetric nature of the structure, it was decided to select wave incidences of 300, 315 and 330 degrees. Equivalent incidences in other quadrants were not considered necessary as the C of G of the Topsides was found to be very central.

With respect to period, it was decided that only the longest period for each wave height would be used, as it was found that the longest gave rise to the most critical load combinations.

The critical phase angle(s) for each wave height, water depth and incidence combination with respect to maximum base shear and overturning moment were then determined. It was found that a phase angle of -60° was the most critical for both criteria as illustrated in Figure 5.5.4A, which shows the variation of overturning moment with phase angle for extreme wave, incidence 300° and water depth 95.9m. This equates to the phase angle of 300° used in the original analysis of 1975.

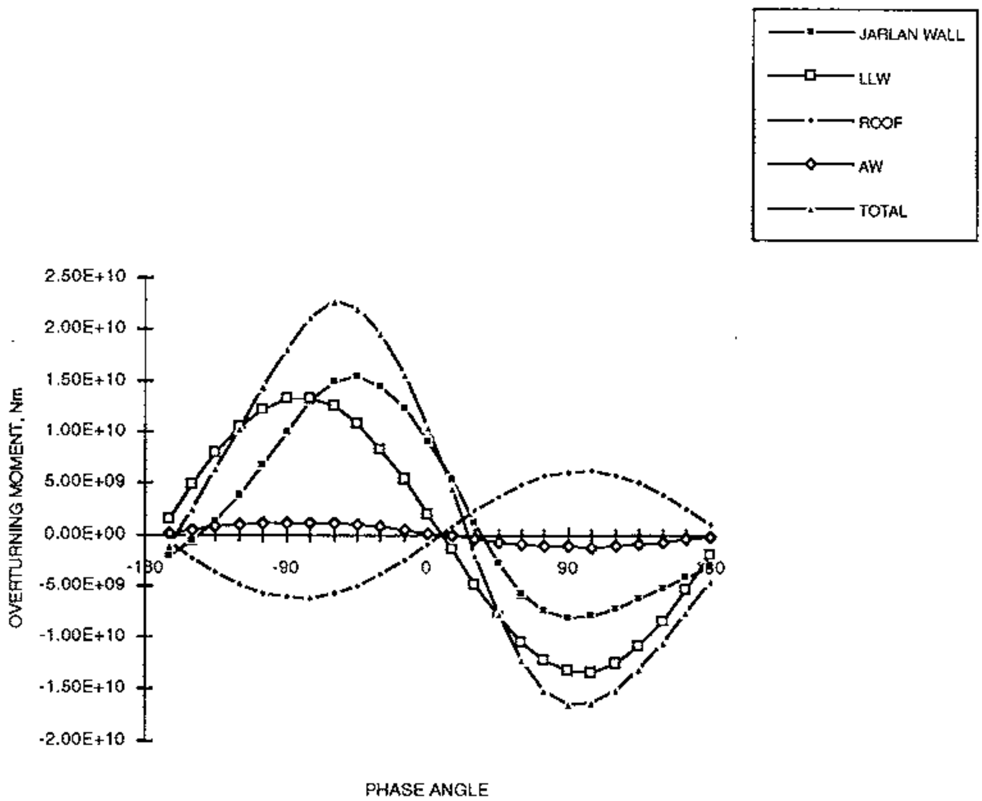


Figure 5.5.4A - Variation of O/T Moment with Phase Angle

The critical phase angle(s) for each wave height, water depth and incidence combination with respect to maximum pressure on the lobated wall were then determined. It was found that a phase angle of -60° was the most critical as shown in Table 5.6.4D for an extreme wave of 26.4m and water depth of 95.9m.

Phase Angle	Max Pressure (kN/m^2)
0	50
30	18
60	-32
90	-55
120	-61
150	-54
180	-34
210	-14
240	31
270	62
300	80
330	75

Table 5.5.4D - Lobated Wall Pressures

The critical phase angle(s) for the deck columns and radial/strut beams were determined with respect to the force resultant for the extreme wave at maximum water depth, with a period of 18.1 seconds and an incidence angle of 300°. It was found that a phase angle of -15° was the most critical. No selection was performed for the operating wave as the deck columns and radial / strut beams were not wetted in this case.

5.5.5 Results Summary

The results from the individual hydrodynamic analyses were combined to give global overturning moments and base shears for the 18 selected wave cases as shown in Table 5.5.5.A

It can be seen that the wave incidence has minimal effect, but that increasing water depth marginally decreases base shear due to a higher proportion of the wave over-topping the breakwater wall.

CASE No.	WAVE HT. (M)	WATER LEVEL (M)	PERIOD (SEC)	PHASE (DEG)	INCIDENCE (DEG)	BASE SHEAR (T)	O/T MOMENT (TM)
1	26.4	95.9	18.1	-60	300	6.52E+04	2.49E+06
2	26.4	95.9	18.1	-15	300	3.53E+04	1.69E+06
3	26.4	95.9	18.1	-60	315	6.52E+04	2.49E+06
4	26.4	95.9	18.1	-15	315	3.53E+04	1.69E+06
5	26.4	95.9	18.1	-60	330	6.52E+04	2.49E+06
6	26.4	95.9	18.1	-15	330	3.53E+04	1.69E+06
7	26.4	92.47	18.1	-60	300	6.66E+04	2.40E+06
8	26.4	92.47	18.1	-15	300	3.73E+04	1.78E+06
9	26.4	92.47	18.1	-60	315	6.66E+04	2.40E+06
10	26.4	92.47	18.1	-15	315	3.73E+04	1.78E+06
11	26.4	92.47	18.1	-60	330	6.66E+04	2.40E+06
12	26.4	92.47	18.1	-15	330	3.73E+04	1.78E+06
13	15.5	95.9	13.8	-60	300	3.25E+04	1.34E+06
14	15.5	95.9	13.8	-60	315	3.25E+04	1.39E+06
15	15.5	95.9	13.8	-60	330	3.25E+04	1.34E+06
16	15.5	92.47	13.8	-60	300	3.37E+04	1.32E+06
17	15.5	92.47	13.8	-60	315	3.37E+04	1.32E+06
18	15.5	92.47	13.8	-60	330	3.37E+04	1.32E+06

Table 5.5.5A - Wave Load Summary

The results were also compared with the 1975 analysis and model tests as shown in Table 5.5.5B. It was noted that extreme overturning moment had decreased by 23% and the extreme base shear by 7%. The corresponding figures for the operational wave were 20% and 4.5% respectively.

	EXTREME			OPERATIONAL		
	ORIGINAL	MODEL TEST	RE-ANAL	ORIGINAL	MODEL TEST	RE-ANAL
Hmax, m	29	29	26.4	18	18	15.5
Period, sec	16	16	18.1	12.5	12.5	13.8
Phase, degrees	300	300	-60	300	300	-60
Water depth, m	97.1	97.1	95.9	97.1	97.1	95.9
incidence			300			
Jarlan wall:						
Fh1, T	1.97E+04		1.74E+04	9.34E+03		8.79E+03
M1, Tm	1.71E+06		1.49E+06	8.20E+05		7.49E+05
LLW:						
Fh2, T	4.07E+04		3.02E+04	2.00E+04		1.61E+04
M2, Tm	1.52E+06		1.25E+06	8.50E+05		6.89E+05
Central shaft:						
Fh3, T	3.11E+03		1.65E+03	1.80E+03		1.11E+03
M3, Tm	3.00E+05		1.43E+05	1.10E+05		9.14E+04
Antiscour wall:						
Fh4, T	5.67E+03		1.48E+04	2.15E+03		5.94E+03
M4, Tm	4.50E+04		1.11E+05	1.60E+04		4.49E+04
Roof:						
Fv5,T	-1.95E+04		-1.71E+04	-7.30E+03		-5.65E+03
M5,Tm	-4.40E+05		-5.77E+05	-1.60E+05		-2.58E+05
Total Horiz. force, T	**6.92E+04**	**6.76E+04**	**6.41E+04**	**3.33E+04**	**2.80E+04**	**3.19E4**
Total Vert. force, T	-1.95E+04		-1.71E+04	-7.30E+03		-5.65E+03
Total o/t moment, Tm	**3.13E+06**	**3.15E+06**	**2.42E+06**	**1.64E+06**	**1.50E+06**	**1.32E+06**

Table 5.5.5B - Comparison with 1975 Results

5.5.6 Current

The effect of current on the structure was allowed for by applying a factor to the hydrodynamic pressures calculated by DIODORE for the selected wave cases. DIODORE only calculates the inertia forces and ignores any drag effects. Therefore to take into account the loads induced by current and drag, the horizontal component of water particle velocity has to be calculated and added to the current velocity so as to determine an equivalent drag force (composed of current and wave effects). A factor, by which to adjust the DIODORE inertia loads, is then calculated as (drag load + inertia load) / (inertia load).

The factor for the MCP-01 re-analysis was based on the 100 year current profile which was taken as varying between 0 m/s on the seabed and 1.57 m/s at el.+95.9m. A range of wave periods (13.5 to 18.1 sec) were analysed and an average wave particle velocity determined. The 100 year factor was calculated to be 1.05, which was then conservatively applied to the operating condition as well.

5.6 PRE-STRESS

The post-tensioned pre-stressing cables on MCP-01 consist of 12 or 24 no. 15.2mm strands (12T15 or 24T15) with a minimum breaking strength of 25.4t per strand (nominal area 143mm^2). This is equivalent to a stress of 178 kg/mm^2 (1742 N/mm^2).

The originally specified jack pressures corresponded to cable forces of 249t (12T15) or 484t (24T15), with the equivalent stress in the steel being 145 kg/mm^2 (1422 N/mm^2).

Prestressing forces in the deck beams were modelled as a series of UDL's with balancing end forces and moments based on an average tendon profile.

Prestressing forces in the sub-structure were modelled as equivalent UDL's that generated the relevant section forces.

For both cases, average losses due to creep, friction etc., were calculated from the original design calculations. For the 12T15 cables, forces of 174.3t (30% loss) and 180t (27.7% loss) were taken for the deck beams and sub-structure respectively. For the 24T15 cables in the sub-structure a force of 350t was taken, corresponding to a 27.7.% loss. These have been checked against site measurements and are considered acceptable.

Losses were taken as being constant along the members as it was considered that this would result in a maximum error of about +/- 5% at any particular section. This was thought to be small when compared with the uncertainty of the actual estimate of losses, especially creep.

Prestress loads in the deck beams were calculated using a customised spreadsheet that calculated average tendon profiles, UDL's based on radius of curvature and end forces based on anchorage angles.

5.7 TEMPERATURE DIFFERENTIALS

In order to represent the potential thermal gradient between the inside and outside of the central shaft, a temperature difference of 10°C was specified between the outer and inner faces. This corresponded to a minimum seawater temperature of 5°C and a maximum internal shaft termpature of 15°C.

5.8 COMBINATIONS

The basic load cases described in Sections 5.1 to 5.7 were combined into the load combinations as summarised in Table 5.8. The combinations and load factors are based upon those in the DEn Guidance Notes.

Load Condition	ULS												SLS	
Load Combination	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Load Type:														
Sub-Structure Dead	1.2	1.2	1.2	1.2	1.2	0.9	1.2	0.9	1.2	0.9	1.2	0.9	1.0	1.0
Deck Dead	1.2	1.2	1.2	1.2	1.2	0.9	1.2	0.9	1.2	0.9	1.2	0.9	1.0	1.0
Live*	1.6	1.6	1.6	1.6	1.2	-	1.2	-	1.2	-	1.2	-	1.0	1.0
Hydrostatic LAT	0.9	-	0.9	-	0.9	0.9	-	-	0.9	0.9	-	-	1.0	-
Hydrostatic HAT	-	1.2	-	1.2	-	-	1.2	1.2	-	-	1.2	1.2	-	1.0
Prestress	1.1	1.1	0.9	0.9	1.1	1.1	1.1	1.1	0.9	0.9	0.9	0.9	1.0	1.0
Wave/Wind Operating	1.4	1.4	1.4	1.4	-	-	-	-	-	-	-	-	1.0	1.0
Wave/Wind Extreme	-	-	-	-	1.2	1.4	1.2	1.4	1.2	1.4	1.2	1.4	-	-
No of Wave Cases	6	6	6	6	12	12	12	12	12	12	12	12	6	6

* includes snow and ice loads

Table 5.8 - Load Combination Summary

For each of the 16 load combinations, there were either 6 or 12 associated wave cases depending on whether one or two phase angles were investigated. This gave a total of 132 load combinations to be analysed, which are detailed in Appendix C1.

6.0 ANALYSIS VERIFICATION AND OUTPUT

6.1 GENERAL

Simple hand calculations have been carried out for a typical wave load combination and the predicted computer results have been compared. This gives us a degree of confidence in the behaviour of each of the structural element. Principal stresses have been taken as a means of comparison for this purpose.

Moreover, global equilibrium and the local equilibrium of the entire structure for each of the load applied has been internally calculated by ANSYS. Reactions have been compared with the applied loads and the out-of-balance if any has been noted. In addition to in-house quality procedures (McA Quality Procedures DP14 and MS023), other checks such as ill-conditioning, data errors as described in CIRIA report (TN 133) have also been carried out.

Also, the deck model on it self has been analyzed. A pinned connection has been assumed at the base of the columns for this type of analysis as shown in Appendix D, Figure D1. The objective of the model is to calculate reactions and check the equlibrium of the deck. Also, the results have been compared with hand calculations. The stiffness of the deck has also been assessed by applying lateral forces.

6.2 REACTION SUMMARIES

The global analysis of the model has been carried out for six still water condition loadings and 18 basic wave/wind loading conditions. This was necessary to keep the input/output files to a manageable size. Also, the advantage of this method is that we can easily verify each basic load set for equilibrium. Loads from the analysis of the 18 basic wave/wind loading conditions are summarized in Table 6.2A and 6.2B. The original is at the centre of the platform and the centre line of the base slab.

These loads compare very well with the applied loading. The out-of-balance is small except for the case of prestressing (+/- 5%). The out of balance for prestressing has been investigated and is considered acceptable (+/- 10% of applied load).

Direction	Phase	Water Depth	Fx (EW) x 10^6 kN	Fy (NS) x 10^6 kN	Fz (VERT) kN
300	-60	LAT	-0.2764	-0.1564	59216
315	-60	LAT	0.2243	-0.2218	59243
330	-60	LAT	0.1594	-0.2723	58297
300	-60	HAT	0.2636	-0.1504	55385
315	-60	HAT	-0.2158	-0.2133	55393
330	-60	HAT	-0.1533	-0.2618	55448

Table 6.2A - Reaction Summary for Operational Wave Loading

Direction	Phase	Water Depth	Fx (EW) x 10^6 kN	Fy (NS) x 10^6 kN	Fz (VERT) x 10^6 kN
300	15	LAT	-0.1736	0.3088	0.2886
300	-60	LAT	-0.3137	0.5505	0.1746
315	15	LAT	-0.2537	0.2569	0.2892
315	-60	LAT	-0.4449	0.4504	0.1745
330	15	LAT	-0.3127	0.1817	0.2895
330	-60	LAT	-0.5959	0.3196	0.1746
300	15	HAT	-0.7108	0.6172	0.4601
300	-60	HAT	-0.3069	0.5398	0.1673
315	15	HAT	-0.2424	0.2457	0.2841
315	-60	HAT	-0.4352	0.4386	0.1664
330	15	HAT	-0.2754	0.2035	0.2844
330	-60	HAT	-0.5341	0.3105	0.1644

Table 6.2B - Reaction Summary for Extreme Wave Loading

Moreover, from 'deck only' analysis the reactions for different load cases are summarized in the Table 6.3C. These results are computer with a load factor of 1.0 for SLS computations. Total bending moment and anial force resulting from dead, live, equipment, snow/ice and wind have been presented in the table. These loads again compare fairly well with the applied loading on the deck.

	Fx (EW) kN	Fy (NS) kN	Fz (VERT) kN
Dead Load	0	0	87003
Equipment Load	100	-10	124900
Live Load	0	0	17705
Wind East	2577	-2299	1449
Wind North	105	1663	-1617
Wind South	-119	-1673	-1617
Wind West	+2251	2296	-1366
Snow & Ice	0	0	2023
Prestress	-985	-226	-791

Table 6.2C - Reaction Summary for "Deck Only" Analysis

6.3 TYPICAL RESULTS

The results for a particular combination have been plotted in terms of principal stresses. These are shown in Appendix D, Figures D2 to D12 for various parts of the structure. The stresses are in kPa and S1, S2 and S3 correspond to the three principal stresses. Typically one of them is zero or near zero for each of the elements. These results reveal the 'hotspots'. These include the intersection of walls, nodes, and tunnel opening, and sudden change of cross-sectional areas. The results from the global analysis are however not-applicable in these areas because of modelling errors. A method has been demonstrated in the following sections to deal with such areas. A summary of average stress is presented in Table 6.3A. These results show that the stresses are within acceptable limits except at the points of obvious stress concentrations. The hand calculated results have been computed from various strut/tie models and closed form solutions for panels.

Away from localized hot-spots, hand calculated results agree very well with FE results. This shows that the behaviour of each of the structural component has been captured fairly well in the model.

Base Slab	Max Principal Stress MPa		Design Condition
	FE	Hand Calc.	
Inner Core	-10.0 3.0	-10.0 1.0	Ballast & Hydrostatic
Outer Core	-11.0 4.5	-21.0 2.8	Wave Pressure
Antiscour Wall	-19.0	-22.0	Wave Pressure
Exterior Diaphragm	-28.0 6.0	-34.6 8.0	Wave Pressure
Interior Wall	-22.0	-22.0	Wave Pressure
Lobed Wall	-18.0	-24.0	Wave Pressure
Jarlan Wall	-5.0	-70.0	Wave Pressure
Central Shaft	-9.0	-9.5	Wave Pressure

Table 6.3A - Typical Stresses in Elements
(Compressive Stresses Negative)

The results from 'deck only' analysis have been presented in the form of bending moment and shear force, axial force diagrams presented in Appendix D, Figures D3 to D17. Typical results are summarised in Table 6.3B . These results are computed with a load factor of 1.0 for SLS computations. Total Bending moment and axial force resulting from dead, live, equipment, snow/ice and wind have been presented in the table. From the results, given in the table, it can be concluded that the beams are more or less in compression.

Main Deck Beams	Total M (kNm)	P (kN)	Top Fibre Stresses MPa	Bottom Fibre Stresses MPa
4GI	10364	-29724	-8.0	-5.4
E1-4 to E4-8	7711	-44585	-14.0	-10.0
6FI	4187	-37155	-12.0	-9.0
G4-7	35665	-44600	-22.0	0.0
7AD	35300	-5200	-16.0	-3.0

Table 6.3B - Typical Moments and Stresses in Deck
(Compressive Stresses Negative)

7.0 CODE CHECKS

7.1 METHODOLOGY

7.1.1 Design Groups

The plate and beam elements used in the ANSYS model to represent the sub-structure and the deck were grouped together into so called "Design Groups", which represented zones of the structure with similar thicknesses and reinforcement details. This gave a total of 129 design groups: 42 for the sub-structure and 87 for the deck and sub-structure beams. The location of the design groups is shown in Appendix E, Figures E1 & E2.

For each design group, section details, including thickness, reinforcement and prestressing cable areas and cover etc, were abstracted from the as-built drawings. For groups where details varied, values which would give the minimum resistance were taken.

7.1.2 MEP

The forces and moments output from ANSYS for the 132 load combinations, described in Section 5.8, were filtered by the in-house developed software MEP (Minimum/maximum Enveloping Program) in order to obtain for each design group, the critical load combinations that contained the minimum or maximum of one or more of the following 13 parameters:

N_{xx} (in-plane force)
N_{yy} (in-plane force)
N_{xy} (in-plane force)
M_{xx} (in-plane moment)
M_{yy} (in-plane moment)
Mxy (in-plane moment)

N_1 (principal force)
N_2 (principal force)
M_1 (principal moment)
M_2 (principal moment)

V_{xy} (transverse shear)
V_{yz} (transverse shear

σ_1 (principal skin stress + face)
σ_2 (principal skin stress - face)
V (principal transverse shear)

Thus for each design group there was a maximum of 26 critical combinations to check for both ULS and SLS conditions.

7.1.3 CONCRETE Program

The critical load combinations for each design group were checked against the relevant section details in accordance with BS8110 using the W.S. Atkins Program CONCRETE.

The CONCRETE program has been developed to efficiently check concrete structures against codes of practice and industry guidelines. The program can analyse prestressed and reinforced concrete slabs, plates and shells with symmetric and/or asymmetric reinforcements, subjected to either unaxial or multi-axial stress fields.

Two methods are available to solve a loaded slab for concrete fibre strains and reinforcement steel stresses, the strip method and the layered method.

The simpler BS8110 **strip method** can be used where the loads are primarily in one direction and there is no significant in-plane shear or torsion.

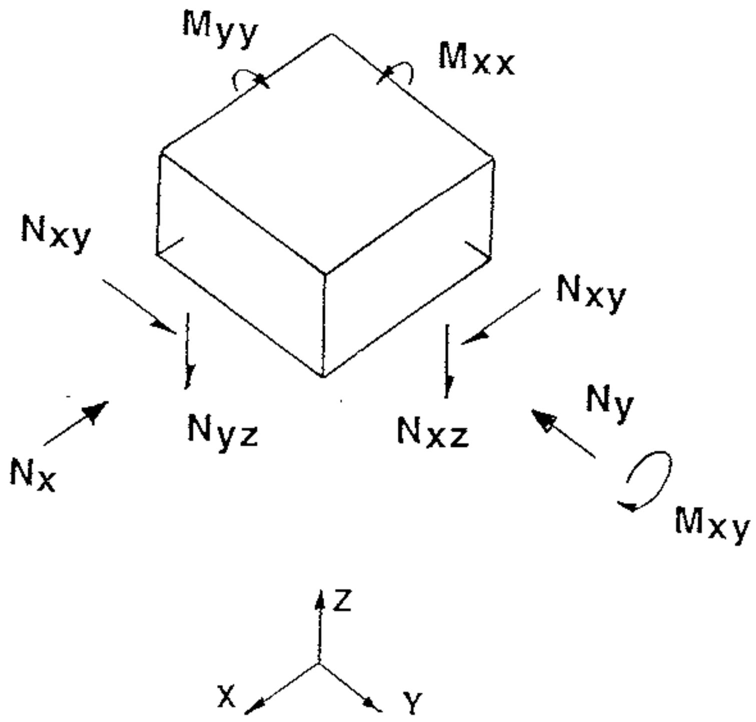
The more sophisticated finite **layered method** is capable of solving concrete slabs under a general state of stress.

Both methods allow the user to define reinforcement and prestressing tendons at any depth and angle for each section under analysis.

The pattern of loading on any unit width of slab/plate or shell can comprise axial loads, bending moments and out of plane shear. In general the loading is represented by the following eight load components:

Nx	-	Axial load per unit width in the X-direction;
Ny	-	Axial load per unit width in the Y-direction;
Nxy	-	In plane shear force per unit width of slab;
Mx	-	Bending moment per unit width about the Y-axis;
My	-	Bending moment per unit width about the X-axis;
M_{xy}	-	Torsional moment per unit width of slab;
N_{xz}	-	Out of plane shear force per unit width acting on the X-Z plane of slab.
N_{yz}	-	Out of plane shear force per unit width acting on the Y-Z plane of slab.

The above forces for the unit width of slab are shown diagrammatically in Figure 7.1.3.



NOTE:

i) Tensile forces are positive
ii) Positive moments cause tensile stresses in bottom fibres

Figure 7.1.3 - Forces/Moment Description

7.1.4 Key Results Description

The key ULS results extracted from the CONCRETE results were concrete, shear, reinforcement and prestress utilisation ratios, and the key SLS results extracted were concrete utilisation and crack width.

Concrete utilisation (ULS) is the extreme fibre compressive strain in the concrete compared to the crushing strain expressed as a percentage.

Concrete utilisation (SLS) is the extreme fibre compressive stress in the concrete compared to the allowable expressed as a percentage.

Shear utilisation (ULS) is the applied force at a section compared to the available capacity expressed as a percentage.

Reinforcement utilisation (ULS) is the maximum strain in the reinforcement compared to the yield strain expressed as a percentage.

Prestress utilisation (ULS) is the maximum strain in the cables taking into account the initial prestressing force and the additional strain from the cable acting as reinforcement, compared to the yield strain expressed as a percentage.

Crack width (SLS) is a theoretical maximum concrete crack width calculated for the applied forces and moments.

Two values of concrete cube strength (fcu) were taken - 60 MPa and 75 MPa. 60 MPa corresponds to the 28 day cube strength from tests made during the original construction and 75 MPa corresponds to the 60 MPa with a 1.25 ageing factor applied, which seems will be justified in view of some recent measured cable strengths in the range of 90 - 110 MPa. For further discussion on cube strength reference should be made to Section 6.1 of the Background Document in Part 2.

The results tabulated in Section 7.2 are maximum values per design group and for most cases the average values are considerably lower.

Where a local model has been developed, the results have been extracted from that local model.

Full code check results for each design group are given in Files 8 to 12 of the Calculation Notes.

7.2.3 SAND Program

SCALE suite (Structural Calculation Ensemble) of SAND package developed by Fitzroy Computers in U.K. has been used for assessment of Torson to BS 8110 in deck beams. It is a simple spread-sheet type of software used in design of structural elements, mainly beams.

7.2 RESULTS SUMMARY

7.2.1 Deck Beams and Columns

7.2.1.1 ULS Results

The ULS results for the beams and columns are summarised in Table 7.2.1.1.

Bending and initial shear checks to BS8110 for the deck beams and manifold deck columns have been performed using the strip option in CONCRETE and concrete strengths (fcu) of 60 and 75 MPa.

The maximum concrete utilisation ratio for the strut/radial beams of 111% occurs for the design group 82 (strut beam at +103.0m) with fcu + 60 MPa but reduces to 89% if fcu is increased to 75 MPa.

The maximum concrete utilisation ratio for the deck beams is 126% for design group 25 with fcu = + 60 MPa but reduces to 101% if fcu is increased to 75 MPa.

In parallel combined shear and torsion checks to BS8110 were performed using the program SAND. Six design groups (15, 16, 31, 34, 40, 41) failed this initial check and required more detailed hand checks, the results of which indicated satisfactory performance.

The steel encased main deck columns were checked using both ACI (1970) and BS 5400 part 5. The maximum interaction ratio calculated according to ACI was 0.053 and the utilization ratio derived from BS 5400 was 33%.

Plots of utilisation ratios are presented in Appendix E, Figures E8 to E11 derived from BS 5400 was 33%.

DG	LOCATION	fcu = 60 Conc. Util. %	fcu = 75 Conc. Util. %	fcu = 60 Shear Util %	fcu = 60 Column Interaction Ratio %	fcu = 60 Shear/Torsion	COMMENTS/STATUS
1	I3.6/6.8	116	93	22	N/A	OK	OK
2	H2.4	38	30	15	N/A	OK	OK
3	H7.9	26	21	12	N/A	OK	OK
4	H4.6/6.7	117	94	55	N/A	OK	OK
5	G1.4	29	23	34	N/A	OK	OK
6	G7.10	35	28	33	N/A	OK	OK
7	G4.6/G6.7	40	32	26	N/A	OK	OK
8	F1.4	12	10	8	N/A	OK	OK
9	F4.6	34	27	23	N/A	OK	OK
10	F6.7	51	41	30	N/A	OK	OK
11	F7.10	44	35	27	N/A	OK	OK
12	E1.4	33	26	44	N/A	OK	OK
13	E7.10	48	38	35	N/A	OK	OK
14	E4.6/E6.7	117	94	50	N/A	OK	OK
15	D2.4	32	26	29	N/A	OK with further checks	OK
16	D7.9	34	27	28	N/A	OK with further checks	OK
17	D4.6/6.7	47	38	38	N/A	OK	OK
18	C4.7	22	18	26	N/A	OK	OK
19	C7.8	20	16	13	N/A	OK	OK
20	B3.4	92	74	22	N/A	OK	OK
21	B7.8	30	24	15	N/A	OK	OK
22	B4.7	68	54	33	N/A	OK	OK
23	A3.4	90	72	17	N/A	OK	OK
24	A7.8	36	29	29	N/A	OK	OK
25	A4.6/A6.7	126	101	42	N/A	OK	OK
26	1G.I	34	27	12	N/A	OK	OK
27	10G.I	14	11	12	N/A	OK	OK
28	2D.G	49	39	19	N/A	OK	OK
29	9D.G	34	27	15	N/A	OK	OK
30	3A.D	35	28	70	N/A	OK	OK
31	4A.D	92	74	46	N/A	OK with further checks	OK
32	4D.F	31	25	43	N/A	OK	OK
33	4F.G	39	31	24	N/A	OK	OK
34	4G.I	37	30	106	N/A	OK with further checks	OK
35	5A.C	34	27	61	N/A	OK	OK
36	6A.B	18	14	29	N/A	OK	OK
37	6B.D	90	72	54	N/A	OK	OK
38	6D.E	69	55	83	N/A	OK	OK (Shear checked by hand)
39	6E.H	26	21	46	N/A	OK	OK
40	6H.I	16	13	30	N/A	OK with further checks	OK
41	7A.D	74	59	68	N/A	OK with further checks	OK
42	7D.G	83	66	41	N/A	OK	OK
43	7G.I	32	26	105	N/A	OK	OK
44	8A.D	21	17	7	N/A	OK	OK
45	9D.E	16	13	9	N/A	OK	OK
46	9F.G	28	22	12	N/A	OK	OK
47	9E.F	32	26	15	N/A	OK	OK
48	COLUMN 1600	N/A	N/A	N/A	0.67	N/A	OK
49	COLUMN 1600	N/A	N/A	N/A	0.8	N/A	OK
50	COLUMN 1600	N/A	N/A	N/A	4.1	N/A	OK
51	COLUMN 1600	N/A	N/A	N/A	4.1	N/A	OK
52	COLUMN 1600	N/A	N/A	N/A	0.9	N/A	OK
53	COLUMN 1600	N/A	N/A	N/A	5.3	N/A	OK
54	COLUMN 1600	N/A	N/A	N/A	0.7	N/A	OK
55	COLUMN 1600	N/A	N/A	N/A	3.6	N/A	OK
56	COLUMN 1600	N/A	N/A	N/A	1.9	N/A	OK
57	COLUMN 2000	N/A	N/A	N/A	1.1	N/A	OK
58	COLUMN 2000	N/A	N/A	N/A	1.5	N/A	OK
59	COLUMN 2000	N/A	N/A	N/A	4.1	N/A	OK
60	COLUMN 2000	N/A	N/A	N/A	1	N/A	OK
61	COLUMN 2000	N/A	N/A	N/A	2.3	N/A	OK

Continued Next Page

Table 7.2.1.1 - Beams & Columns ULS Condition 1/2

DG	LOCATION	fcu = 60 Conc. Util. %	fcu = 75 Conc. Util. %	fcu = 60 Shear Util %	fcu = 60 Column Interaction Ratio %	fcu = 60 Shear/Torsion	COMMENTS/STATUS
62	D'2-9		0		N/A	OK	OK
63	E'2-9	43	34	38	N/A	OK	OK
64	11-D'E	6	5	4	N/A	OK	OK
65	13-D'E	34	27	11	N/A	OK	OK
66	16-D'E	6	5	4	N/A	OK	OK
67	18-D'E	51	41	14	N/A	OK	OK
68	6-D'E	15	12	9	N/A	OK	OK
69	MANIFOLD COL.	6	5	1	N/A	OK	OK
70	MANIFOLD COL.	14	11	8	N/A	OK	OK
71	MANIFOLD COL.	13	10	7	N/A	OK	OK
72	MANIFOLD COL.	21	17	9	N/A	OK	OK
73	MANIFOLD COL.	14	11	0	N/A	OK	OK
74	MANIFOLD COL.	42	34	16	N/A	OK	OK
75	MANIFOLD COL.	N/A	N/A	N/A	N/A	N/A	Not a column/beam
76	MANIFOLD COL.	6	5	2	N/A	OK	OK
77	Radial 103	69	55	32	N/A	OK	OK
78	Radial 103	69	55	32	N/A	OK	OK
79	Radial 103	64	51	31	N/A	OK	OK
80	Radial 103	63	50	30	N/A	OK	OK
81	Struts 103	55	44	20	N/A	OK	OK
82	Struts 103	111	89	20	N/A	OK	OK
83	Struts 103	87	70	20	N/A	OK	OK
84	Struts 103	54	43	65	N/A	OK	Further checks required
85	Struts 65	101	81	18	N/A	OK	OK
86	Struts 65	43	34	16	N/A	OK	OK
87	Struts 65	21	17	10	N/A	OK	OK
	MAXIMUM	126	101	106	5.3		

Table 7.2.1.1 - Beams & Columns ULS Condition 2/2

7.2.1.2 SLS Results

The SLS results for the beams and columns are summarised in Table 7.2.1.2.

SLS checks to BS8110 have been performed on all beams and manifold deck columns using the strip option in CONCRETE for concrete strengths of 60 and 75 MPa. The generally advantageous effects of tension stiffening have been included.

No SLS checks have been performed on the main deck columns as they are steel encased.

Crack widths are within the allowable value for the atmospheric zone of 0.1mm with the exception of the manifold deck beam D'2-9, (0.13 mm).

The maximum overall concrete utilisation ratio of 129% occurs for manifold deck beam D'2-9, (dg 62), and strut beams (dg 84) with fcu = 60 MPa but reduces to 103% if fcu is increased to 75 MPa.

DG	LOCATION	fcu=60MPa		fcu=75MPa		COMMENTS/STATUS
		Crack Width	Conc. Util. %	Crack Width	Conc. Util. %	
1	I3.6/6.8	0.07	106	0.07	84.8	OK
2	H2.4	0	78	0	62.4	OK
3	H7.9	0	78	0	62.4	OK
4	H4.6/6.7	0	104	0	83.2	OK
5	G1.4	0	88	0	70.4	OK
6	G7.10	0	88	0	70.4	OK
7	G4.6/G6.7	0	88	0	70.4	OK
8	F1.4	0	88	0	70.4	OK
9	F4.6	0	88	0	70.4	OK
10	F6.7	0	88	0	70.4	OK
11	F7.10	0	88	0	70.4	OK
12	E1.4	0	88	0	70.4	OK
13	E7.10	0	88	0	70.4	OK
14	E4.6/E6.7	0.05	119	0.05	95.2	OK
15	D2.4	0.03	98	0.03	78.4	OK
16	D7.9	0.03	98	0.03	78.4	OK
17	D4.6/6.7	0.02	101	0.02	80.8	OK
18	C4.7	0	98	0	78.4	OK
19	C7.8	0	98	0	78.4	OK
20	B3.4	0	98	0	78.4	OK
21	B7.8	0	98	0	78.4	OK
22	B4.7	0	98	0	78.4	OK
23	A3.4	0	98	0	78.4	OK
24	A7.8	0	98	0	78.4	OK
25	A4.6/A6.7	0	98	0	78.4	OK
26	1G.I	0	98	0	78.4	OK
27	10G.I	0	98	0	78.4	OK
28	2D.G	0	98	0	78.4	OK
29	9D.G	0	98	0	78.4	OK
30	3A.D	0	98	0	78.4	OK
31	4A.D	0	98	0	78.4	OK
32	4D.F	0	116	0	92.8	OK
33	4F.G	0	106	0	84.8	OK
34	4G.I	0	96	0	76.8	OK
35	5A.C	0	96	0	76.8	OK
36	6A.B	0	96	0	76.8	OK
37	6B.D	0	96	0	76.8	OK
38	6D.E	0	96	0	76.8	OK
39	6E.H	0	96	0	76.8	OK
40	6H.I	0	96	0	76.8	OK
41	7A.D	0	96	0	76.8	OK
42	7D.G	0	116	0	92.8	OK
43	7G.I	0	84	0	67.2	OK
44	8A.D	0	84	0	67.2	OK
45	9D.E	0	84	0	67.2	OK
46	9F.G	0	84	0	67.2	OK
47	9E.F	0	84	0	67.2	OK
48	COLUMN 1600	N/A	N/A	N/A	N/A	Columns are steel encased
49	COLUMN 1600	N/A	N/A	N/A	N/A	Columns are steel encased
50	COLUMN 1600	N/A	N/A	N/A	N/A	Columns are steel encased
51	COLUMN 1600	N/A	N/A	N/A	N/A	Columns are steel encased
52	COLUMN 1600	N/A	N/A	N/A	N/A	OK (Columns are steel encased)
53	COLUMN 1600	N/A	N/A	N/A	N/A	OK (Columns are steel encased)

Table 7.2.1.2 - Beams & Columns SLS Condition 1/2

DG	LOCATION	fcu=60MPa		fcu=75MPa		COMMENTS/STATUS
		Crack Width	Conc. Util. %	Crack Width	Conc. Util. %	
54	COLUMN 1600	N/A	N/A	N/A	N/A	OK (Columns are steel encased)
55	COLUMN 1600	N/A	N/A	N/A	N/A	OK (Columns are steel encased)
56	COLUMN 1600	N/A	N/A	N/A	N/A	OK (Columns are steel encased)
57	COLUMN 2000	N/A	N/A	N/A	N/A	OK (Columns are steel encased)
58	COLUMN 2000	N/A	N/A	N/A	N/A	OK (Columns are steel encased)
59	COLUMN 2000	N/A	N/A	N/A	N/A	OK (Columns are steel encased)
60	COLUMN 2000	N/A	N/A	N/A	N/A	OK (Columns are steel encased)
61	COLUMN 2000	N/A	N/A	N/A	N/A	OK (Columns are steel encased)
62	D'2-9	0.13	129	0.13	103.2	Accept
63	E'2-9	0	80	0	64	OK
64	11-D'E	0	60	0	48	OK
65	13-D'E	0	77	0	61.6	OK
66	16-D'E	0	60	0	48	OK
67	18-D'E	0	84	0	67.2	OK
68	6-D'E	0	67	0	53.6	OK
69	MANIFOLD COL.	0	8	0	6.4	OK
70	MANIFOLD COL.	0	15	0	12	OK
71	MANIFOLD COL.	0	22	0	17.6	OK
72	MANIFOLD COL.	0.02	23	0.02	18.4	OK
73	MANIFOLD COL.	0.03	15	0.03	12	OK
74	MANIFOLD COL.	0.02	40	0.02	32	OK
75	MANIFOLD COL.	N/A	N/A	N/A	N/A	Not a column
76	MANIFOLD COL.	0	15	0	12	OK
77	Radial 103	0	70	0	56	OK
78	Radial 103	0	70	0	56	OK
79	Radial 103	0	70	0	56	OK
80	Radial 103	0	70	0	56	OK
81	Struts 103	0.30	103	0.30	82	Accept
82	Struts 103	0	101	0	81	Accept
83	Struts 103	0	102	0	82	Accept
84	Struts 103	0	129	0	103.2	Accept
85	Struts 65	0.13	94	0.13	75.2	OK
86	Struts 65	0.13	94	0.13	75.2	OK
87	Struts 65	0.13	94	0.13	75.2	OK
	MAXIMUM	0.30	129	0.30	103.2	

Table 7.2.1.2 - Beams & Columns SLS Condition 2/2

7.2.2 Sub-Structure

For all code checking on the sub-structure, the generally advantageous effects of tension stiffening have been included and the effects of water pressure acting within a crack ignored.

Because of relatively low percentages and high surface areas of reinforcement in the structure, the use of tension stiffening in the concrete is considered acceptable.

It is also believed that the pore pressures in concrete (particularly sub-merged zone) have reached a steady state condition whereby water penetration depths equal to half to quarter thickness of the structural member have now been reached. Therefore, water pressure in the cracks will be counteracted by internal pore water pressure and hence further opening of the cracks is not possible. It is therefore considered appropriate not to include water pressure in the cracks for the structure.

Results, where possible, have been taken from the main model using the MEP routine. However, for some groups this was considered to give erroneous results as the modelling contained non-representative "hot spots", which were then picked up by the MEPprogram and presented as the results for that group. This was remedied by either taking the results from the relevant local model for that design group or by conducting an element by element code check omitting the MEPstage. This then allowed extreme results to be discarded. Full details of the local models are presented in Section 11.0.

7.2.2.1 ULS Results (Sub Structure)

The ULS results for the sub-structure are summarised in Table 7.2.2.1 and are based on bending and shear checks to BS8110 performed using the layered option in CONCRETE, with concrete strengths of 60MPa and 75MPa.

The maximum concrete utilisation ratio of 136% occurs for design group 6 with fcu = 60, but reduces to 109% if fcu is increased to 75. The only other areas where the utilisation ratio for fcu = 60 exceeds 100% is in the base slab for design groups 9 and 7, where the ratios are 109% and 129% respectively.

The maximum shear utilisation ratio of 139% occurs for design group 1. With BS8110 shear enhancement this value reduces to about 87%. Thus shear enhancement factors (hand adjusted since it is not automatic in the CONCRETE program) given in BS8110 would certainly improve the results presented in the table. In all there are 22 design groups where the shear utilisation ratio exceeds 100%. It is considered that this is due to the typically more stringent requirements of BS8110 compared to the originally used ACI code. Appendix 1B of the Background Document No: 3311-S-M-001-0 presents a comparison which concludes that in certain circumstances BS8110 requires up to 47% more shear steel. Shear capacity evaluation varies significantly between the codes of practices and the shear stresses found would be acceptable under the Norwegian and other codes.

The maximum reinforcement utilisation ratio of 118% occurs for design group 37 (breakwater wall). The only other design groups where the reinforcement utilisation exceeds 100% are 19 and 24, where the ratios are 110% and 102% respectively.

The maximum prestressing cable utilisation ratio of 51% occurs for design group 20 (outer internal diaphragm 0.3 - 18.0).

The base slab local model (design groups 1 to 10) revealed concrete utilisations (Fcu = 60) in the range 109% to 136%. However these results are with a conservative load factor of 1.4. If we use 0.9 still water loads plus 1.2 times extreme wave load, this results in a effective load factor of 1.2. Therefore the ULS results for the base slab are to be factored by a ratio of: 1.2/1.4 (=0.87). With the application of this factor, the base slab concrete and shear utilisation ratios are satisfactory.

DG	LOCATION	fcu = 60	fcu = 75				COMMENTS
		conc. util %	conc. util %	shear util %	reinf. util. %	prest. util. %	
42	Int Diaphragm 65-68	26	21	108	54	40	Main model with node area excluded
41	Central shaft (Y) 65-68	58	46	77	99	48	Main model
40	Central shaft 127.6-147.8	4	3	71	14	41	Main model with shear stress at node refined
39	Central shaft 65-127.6	31	25	86	66	n/a	1101C Local Model
38	Breakwater wall 68-105	16	13	99	61	46	Node area excluded
37	Breakwater wall 65-68	30	24	110	118	49	Node area excluded
36	Int Diaphragm (outer) 31-65	36	29	96	51	46	Node area excluded
35	Int Diaphragm 31-65	41	33	39	79	47	Node area excluded
34	Central shaft 31-65	28	22	118	56	41	Main model
33	Central shaft (Y) 31-65	27	22	111	55	39	Main model
32	Int Diaphragm (outer)18-31	26	21	94	44	41	Main model with shear stress at node refined
31	Int Diaphragm (inner)18-31	48	38	103	78	40	Main model with shear stress at node refined
30	Central shaft 30-31	45	36	122	87	40	Main model
29	Central shaft (Y) 18-31	32	26	105	96	40	Main model
28	Lobed wall 18-65	46	37	79	86	48	Main model
27	Central Shaft (Y) 3.85-18	23	18	120	47	44	Main model
26	Lobed wall 3.85-18.0	37	30	107	94	41	Main model
25	Tunnel roof	36	29	64	77	n/a	Tunnel local model
24	Tunnel walls	45	36	96	102	48	Tunnel local model
23	Ext Diaphragm	87	70	114	48	46	
22	Ext Diaphragm	50	40	102	81	44	
21	Ext Diaphragm	21	17	74	35	41	
20	Int Diaphragm (outer) 0.3-18	27	22	94	96	51	
19	Int Diaphragm (inner) 0.3-18	55	44	61	110	50	Main model
18	Central shaft 0.3-18	33	26	2	59	n/a	
17	Central Shaft (Y) 0.3-3.85	26	21	95	43	41	Main model
16	Interior wall	27	22	102	49	42	
15	Lower lobed wall 0.3-3.85	35	28	118	97	39	1101D Local model
14	Tunnel end wall	18	14	110	114	n/a	Tunnel local model
13	Exterior wall	32	26	100	65	40	
12	Perforated wall	15	12	94	43	41	
11	Antiscour wall	41	33	85	87	34	
10	Base slab (bottom of central shaft)	25	20	48	43	39	Base slab local model
9	Base slab	109	87	78	99	41	Base slab local model
8	Base slab (tunnel floor)	30	24	120	49	39	1101D/ Base slab local model
7	Base slab	129	103	104	100	40	Base slab local model
6	Base slab	136	109	117	94	48	Base slab local model
5	Base slab	75	60	104	66	39	Base slab local model
4	Base slab	50	40	96	79	40	Base slab local model
3	Base slab	76	61	128	91	37	Base slab local model
2	Base slab	40	32	116	96	42	Base slab local model
1	Base slab	126	101	139	93	42	Base slab local model
	MAXIMUM	136	109	139	118	51	

Note: Results are from main model unless stated otherwise.

Table 7.2.2.1 - Sub-Structure ULS Condition

7.2.2.2 SLS Results (Sub-Structure)

The SLS results for the sub-structure are summarised in Table 7.2.2.2 and are based on checks to BS8110, performed using the layered option in CONCRETE, with concrete strengths of 60 and 75 MPa.

A maximum concrete utilisation ratio of 129% occurs for design groups 41, 39, 33, 31 and 29 with fcu = 60, but reduces to 103% if fcu is increased to 75. The only other design groups where concrete utilisation ratios exceeded 100% for fcu = 75MPa were 35, 30, 17 and 3, where the values were 102%, 101%, 102% and 102% respectively.

Crack widths are presented for both faces of design groups where conditions differ on each side (i.e. atmospheric/submerged). Zero crack width indicates that the section is in compression. The maximum external crack width found was 0.98mm and the maximum internal crack width found was 0.78mm. These compare with respective allowables from the DEn Guidance notes of 0.3mm and 0.1mm. It was found that increasing fcu from 60MPa to 75MPa had minimal effect on crack widths.

It should be noted that these are maximum crack widths for the groups concerned and that the average in most cost cases is considerably lower.

Plots of utilisation ratios are presented in Appendix E, Figures E3 to E7.

DG	LOCATION	Allow. Crack Width	fcu=60		fcu=75		COMMENTS
	External:		conc. util %	crack width	conc. util %	crack width	
42	Int Diaphragm 65-68	0.3	102	0.20	82	0.20	
41	Central shaft (Y) 65-68	0.3	129	0.01	103	0.01	
40	Central shaft 127.6-147.8	0.3	25	0.39	20	0.39	
39	Central shaft 65-127.6	0.3	77	0.59	62	0.59	1101C Local model
38	Breakwater wall 68-105	0.1	129	0.03	103	0.03	Non-representative elements removed
37	Breakwater wall 65-68	0.3	58	0.09	46	0.09	
36	Int Diaphragm (outer) 31-65	0.3	83	0.05	66	0.05	
35	Int Diaphragm 31-65	0.3	128	0.00	102	0.00	
34	Central shaft 31-65	0.3	88	0.10	70	0.10	1101C Local model
33	Central shaft (Y) 31-65	0.3	97	0.50	78	0.50	
32	Int Diaphragm (outer)18-31	0.3	88	0.00	70	0.00	
31	Int Diaphragm (inner)18-31	0.3	129	0.00	103	0.00	
30	Central shaft 30-31	0.3	99	0.04	79	0.04	1101C Local model
29	Central shaft (Y) 18-31	0.3	101	0.62	81	0.62	
28	Lobed wall 18-65	0.3	115	0.18	92	0.18	Non-representative elements removed
27	Central Shaft (Y) 3.85-18	0.3	55	0.48	44	0.48	
26	Lobed wall 3.85-18.0	0.3	93	0.14	74	0.14	
25	Tunnel roof	0.3	99	0.50	79	0.50	Tunnel local model
24	Tunnel walls	0.3	109	0.54	87	0.54	Tunnel local model
23	Ext Diaphragm	0.3	80	0.00	64	0.00	
22	Ext Diaphragm	0.3	117	0.53	94	0.53	Non-representative elements removed
21	Ext Diaphragm	0.3	119	0.58	95	0.58	
20	Int Diaphragm (outer) 0.3-18	0.3	69	0.00	55	0.00	
19	Int Diaphragm (inner) 0.3-18	0.3	120	0.00	96	0.00	Non-representative elements removed
18	Central shaft 0.3-18	0.3	24	0.00	19	0.00	
17	Central Shaft (Y) 0.3-3.85	0.3	128	0.06	102	0.06	1101A Local model
16	Interior wall	0.3	102	0.61	82	0.61	
15	Lower lobed wall 0.3-3.85	0.3	105	0.54	84	0.54	1101D Local model
14	Tunnel end wall	0.3	43	0.98	34	0.98	Tunnel local model
13	Exterior wall	0.3	61	0.40	49	0.40	Non-representative elements removed
12	Perforated wall	0.3	48	0.43	38	0.43	Non-representative elements removed
11	Antiscour wall	0.3	60	0.02	48	0.02	
10	Base slab (bottom of central	0.3	70	0.06	56	0.06	Base slab local model
9	Base slab	0.3	89	0.57	71	0.57	Base slab local model
8	Base slab (tunnel floor)	0.3	66	0.50	53	0.50	1101D/base slab local models
7	Base slab	0.3	53	0.09	42	0.09	Base slab local model
6	Base slab	0.3	118	0.65	94	0.65	Base slab local model
5	Base slab	0.3	51	0.09	41	0.09	Base slab local model
4	Base slab	0.3	94	0.35	75	0.35	Base slab local model
3	Base slab	0.3	128	0.61	102	0.61	Base slab local model
2	Base slab	0.3	123	0.58	98	0.58	Base slab local model
1	Base slab	0.3	129	0.39	103	0.39	Base slab local model
	MAXIMUM (external)		129	0.98	103	0.98	
	Internal:						
40	Central shaft 127.6-147.8	0.1	31	0.27	25	0.27	
39	Central shaft 65-127.6	0.1	57	0.78	46	0.78	1101C Local model
34	Central shaft 31-65	0.1	111	0.00	89	0.00	1101C Local model
30	Central shaft 30-31	0.1	126	0.30	101	0.30	1101C Local model
25	Tunnel roof	0.1	91	0.29	73	0.29	Tunnel local model
24	Tunnel walls	0.1	111	0.48	89	0.48	Tunnel local model
18	Central shaft 0.3-18	0.1	50	0.02	40	0.02	
14	Tunnel end wall	0.1	83	0.64	66	0.64	Tunnel local model
8	Base slab (tunnel floor)	0.1	89	0.23	71	0.23	Base slab local model
	MAXIMUM (Internal)		126	0.78	101	0.78	

Table 7.2.2.2 - Sub-Structure SLS Condition

7.3 FOUNDATION STATUS

The foundation status of MCP-01 has been reassessed using the same methodology as detailed in the following documents:

i) Total Oil Marine - MCP-01 Foundation Study No 86/8 (ODE Doc No 87-281).

ii) Review of MCP-01 Foundation Status (Total Doc No MPI/TIR/86/03).

Factors of safety against sliding, overturning and bearing capacity, as well as minimum soil pressure have been recalculated. These are shown in Table 7.3, together with results from previous analyses.

Values have been re-calculated for extreme and operating waves as well as maximum and minimum on-bottom weights. The minimum on-bottom weight of 205,275t is considered to be an extreme lower bound based on recalculated ballast and sub-structure weights. The on-bottom weight in reality is considered to be close or equal to the maximum of 214,739t depending on water depth.

		Orig. design 1975	Doc. No. 87-281	RE-ANALYSIS Extreme Max. Fv	Min Fv	RE-ANALYSIS Operating Max. Fv	Min Fv
FOUNDATION AREA	m2	5722	5650	5650	5650	5650	5650
INERTIA OF BASE	m4	4.69E+06	4.58E+06	4.58E+06	4.58E+06	4.58E+06	4.58E+06
ON BOTTOM WEIGHT	t	205000	214879	214739	205275	214739	205275
BASE SHEAR	t	67600	69747	65700	65700	3.18E+04	3.18E+04
OVERTURNING MOMENT	t m	3.15E+06	3.31E+06	2.56E+06	2.56E+06	1.30E+06	1.30E+06
DIAMETER	m	101	101	101	101	101	101
ANGLE OF FRICTION (1)	deg	31	31	31	31	31	31
SUBMERGED DENSITY OF SOIL (1)	t/m3	1.05	1.05	1.05	1.05	1.05	1.05
BEARING CAPACITY FACTOR (1)		26	26	26	26	26	26
ρ (1)		0.53	0.53	0.53	0.53	0.53	0.53
λ (1)		0.47	0.47	0.47	0.47	0.47	0.47
SHAPE FACTOR (1)		0.31	0.31	0.31	0.31	0.31	0.31
μ (1)		0.304	0.305	0.236	0.247	0.120	0.125
θ_1 (1)		1.17	1.17	1.01	1.04	0.71	0.72
θ_2 (1)		0.74	0.74	0.09	0.30	0.00	0.00
ECCENTRICITY FACTOR (1)		0.468	0.467	0.572	0.553	0.797	0.786
INCLINATION FACTOR (1)		0.269	0.276	0.300	0.281	0.579	0.563
Qult (1)	t/m2	53.8	55.0	73.3	66.5	197.1	189.3
RESISTANCE TO SLIDING	t	123100	129032	128948	123265	128948	123265
OVERTURNING RESTORING MOMENT	t m	10352500	10851390	10844320	10366388	10844320	10366388
BEARING CAPACITY	t	307850	310507	414172	375927	1113499	1069679
FACTOR OF SAFETY, SLIDING (2)		1.82	1.85	1.96	1.88	4.05	3.88
FACTOR OF SAFETY, OVERTURNING		3.29	3.28	4.24	4.05	8.34	7.97
FACTOR OF SAFETY, BEARING CAPACITY (2)		1.50	1.45	1.93	1.83	5.19	5.21
MIN. SOIL PRESSURE	t/m2	1.9	1.5	9.8	8.1	23.7	22.0

* Allowable=1.5 (extreme), 2.0 (operating)

Table 7.3 - Foundation Status

The most critical parameters were found to be the Bearing Capacity and Sliding Factors of Safety for the extreme conditions, with values of 1.96 and 1.93 for maximum on-bottom weight, and 1.88 and 1.83 for minimum on-bottom weight. These are in excess of the figure of 1.5 required by the DEn Guidance Notes. The equivalent figures for the operating case are 4.05 and 5.19, and 3.88 and 5.21, which are well in excess of the specified value of 2.0.

Factors of safety against overturning are 4.00 and 7.9 for the extreme and operating conditions respectively.

The minimum soil pressure calculated was 8.1 t/m² in excess of the 0 t/m² necessary for no uplift. There is therefore always a positive pressure on the base.

8.0 FATIGUE

8.1 INTRODUCTION

A major part of the re-analysis was to demonstrate that MCP-01 has sufficient additional fatigue life to enable re-certification to 2015, i.e. a total fatigue life of at least 40 years.

The fatigue strength of the concrete sub-structure has therefore been reviewed. No review of the fatigue strength has been performed for the deck as this is not considered to be fatigue sensitive.

The fatigue review of the sub-structure has been conducted in two stages; an initial review and a more detailed analysis.

8.2 INITIAL REVIEW

The initial stage was to perform an outline review of stress levels in accordance with the simplified approach of the DEn Guidance Notes section 23.2.9, which states that with load factors of 1.0, no further checks are necessary if:

(1) for straight high yield reinforcing bars to BS 4449: 1978 and BS 4461: 1978 the maximum tensile stress range does not exceed 140 N/mm^2. For loads having more than 1 million cycles the maximum tensile stress range is not greater than 35 N/mm^2.

(2) for concrete the maximum resultant compressive stress does not exceed 0.33 f_{cu}. No significant membrane tensile stresses exist and flexural tensile stresses are limited to 0.20 $\sqrt{f_{cu}}$.

(3) the detailing of reinforcement in areas of significant cyclic loading, with respect to bends, laps, butt welds and mechanical connectors satisfies the recommendations given in 23.2.9(b).

Code check results for the SLS condition (load factor = 1.0) were reviewed and it was found that for the majority of design groups, one or more of the above limitations was exceeded. It was therefore decided that in view of this and the fact that an extended design life of 40 years was required, that a more detailed fatigue analysis should be performed.

8.3 DETAILED ANALYSIS

8.3.1 Methodology

A detailed analysis methodology was developed that included the principles of cumulative damage and the variation in stress range with wave height. The methodology in outline was as follows:

(1) Determine operational wave case that is considered to cause maximum stresses in sub-structure.

(2) Apply these wave loads on their own to the structure with load factors = 1.0.

(3) For the maximum principal stresses in each design group, find the associated element and section forces / moments. The assumptions are that:

   (i) Principal stresses are directly related to the associated stresses in the concrete and reinforcement and hence the element with maximum principal stress will also have maximum reinforcement and concrete stresses.

   (ii) The stress range can be taken as twice the maximum principal stress.

(4) Using the values of forces/moments determined in 3, determine the relationship that relates section forces to wave height. (Dynamic magnification has shown to be insignificant and therefore response should be linear throughout the wave spectrum).

(5) Using the MAREX environmental conditions report, determine 8 ranges of wave heights and expected frequencies.

(6) For each design group calculate equivalent section forces for each wave height range using the relationship determined in 4.

(7) Determine static (still water) section forces / moments for each design group.

(8) Specify appropriate S-N curve for reinforcement, cables and concrete.

(9) Run fatigue check in WS Atkins program CONCRETE for each design group, using section forces / moments and frequencies from 5, 6 and 7.

8.3.2 Fatigue Checks in CONCRETE

CONCRETE uses cumulative damage calculations with a deterministic approach to determine the fatigue life of both concrete and steel components under cyclic loading. Whilst the program was not developed for any one code or set of rules, the fatigue checks followed satisfy the requirements of several documents, including DEn Guidance Notes, DnV, and NS 3473.

The calculation of fatigue damage and fatigue life are fairly conventional and follow the Miner's Rule approach. Damage in each component for each set of cyclic loads is obtained from material characteristics (S-N curves) and acting stresses, before damage is summed and inverted to give a fatigue life.

Cyclic loading data on the structure may be represented in one of two ways:

(i) each cycle may be represented as a load time history, with slab loads N_X, N_y, N_{xy}, M_X, M_y, M_{xy}, N_{xz}, N_{xy}) provided at two or more time steps through the cycle (to provide a maximum/minimum range of stress);

(ii) each cycle may be considered to be harmonic and may therefore be represented in a complex form as static, real (0° phase) and imaginary (90° phase) loads. These loads may then be combined to generate the variation of load through the load cycle.

For the MCP-01 re-analyses (ii) was used with the imaginary part set to zero.

In assessing the cumulative damage to the reinforcement and prestress, the following trilinear curve from the Concrete-in-the-Oceans programme was used:

(a) Stress ranges between 400 Nmm^{-2} and 235 Nmm^{-2}:

$$\text{Log } N = 19.62 - 6.0 \text{ Log } \sigma$$

(b) Stress ranges between 235 Nmm^{-2} and 65 Nmm^{-2}:

$$\text{Log } N = 12.04 - 2.8 \text{ Log } \sigma$$

(c) Stress ranges below 65 Nmm^{-2}:

$$\text{Log } N = 15.65 - 4.8 \text{ Log } \sigma$$

For concrete subjected to compression/compression cycling, the following DnV S-N curve was adopted:

$$\text{Log}_{10}\text{N} - ccfact.\frac{(1 - S_{max} / (\alpha f_{cck} / \gamma_m))}{(1 - S_{min} / (\alpha f_{cck} / \gamma_m))}$$

where:

N	number of cycles to failure under constant amplitude loading from S_{min} to S_{max};
ccfact	taken as 10 to 12.7 for MCP-01;
S_{max}	maximum compressive stress (Nmm^{-2});
S_{min}	minimum compressive stress (Nmm^{-2});
f_{cck}	concrete cylinder strength (Nmm^{-2});
γ_m	material partial safety factor for concrete;
α	flexural gradient coefficient.

For concrete subjected to compressive/tension cycling the S-N curve proposed by Waagaard was used:

$$\mathrm{Log}_{10}N = ctfact.(1 - S_{max} / (\alpha f_{cck} / \gamma_m))$$

where:

ctfact taken as 8 for MCP-01.

The steel and concrete S-N curves used for MCP-01are shown in Figure 8.3.2.

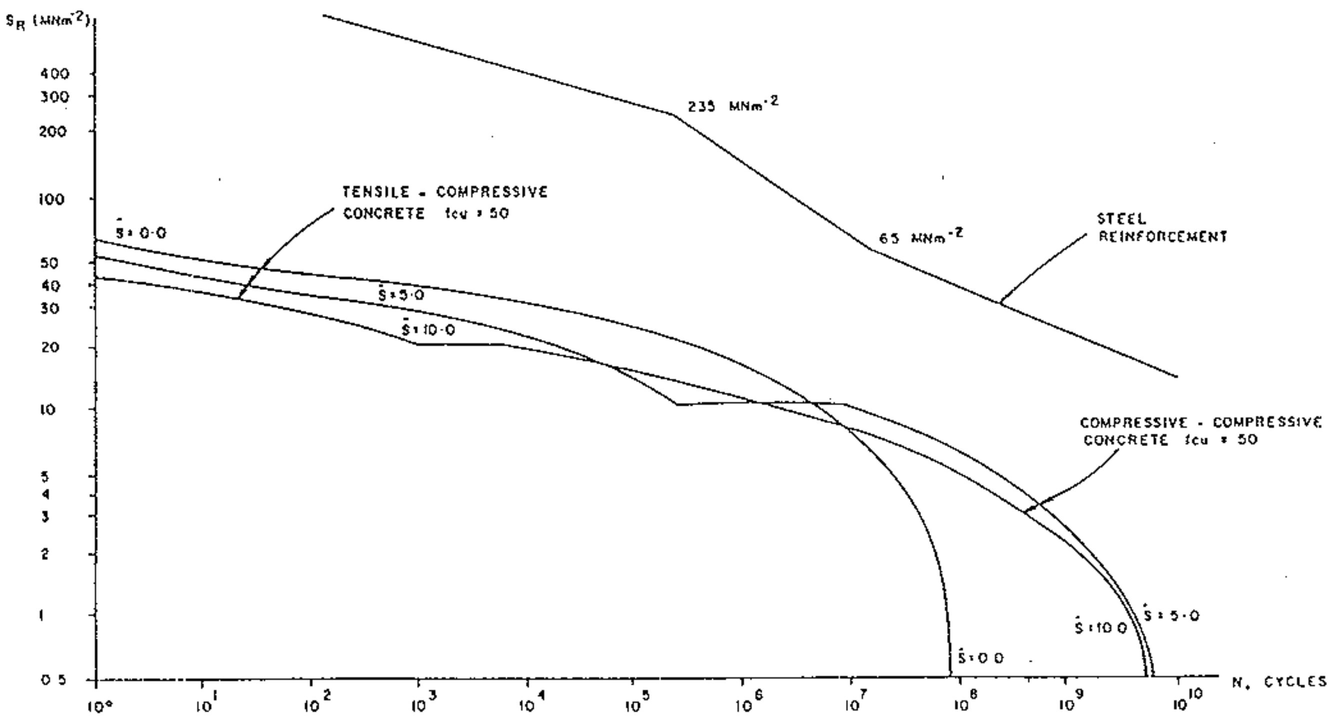


Figure 8.3.2 - S-N Curves for Steel and Concrete

8.3.3 Results

For each design group, the element with the maximum in-plane principal stress was identified under operating conditions together with associated forces and moments. For the same element, forces and moments for still water and extreme conditions were also extracted.

It was then assumed that the relationship between stress range (and force/moments) followed the following equation:

$$\Delta\sigma = KH^{\alpha}$$

where:

H Wave height
$\Delta\sigma$ Stress range (or force/moment range)
k, α Constants

The operating wave height is 15.5m and the extreme wave height is 26.4m, giving a ratio of operating to extreme of 0.587.

From the extracted principal stresses, the average value of the ratio of operating principal stress to extreme principal is 0.52. This was considered to be very close to the wave height ratio of 0.587 and therefore a linear relationship between wave height and principal stress could be assumed, i.e. $\alpha = 1.0$. The proportion of operating wave forces / moments is shown in Table 8.3.3A for all the selected wave height ranges.

From the MAREX environmental report the overall range of wave heights was divided into 8 separate sub-ranges and the number of annual occurrences calculated as shown in Table 8.3.3A. Waves with a return period of over a year were ignored as it was considered that high cycle low amplitude waves are more critical for fatigue than low cycle high amplitude waves.

Wave Height	No. of Waves per Year	Proportion of Operating Wave
0.0-0.9	3035041	0.0-0.06
1.0-1.9	1879560	0.06-0.12
2.0-4.9	1036599	0.12-0.32
5.0-7.9	65939	0.32-0.51
8.0-10.9	5654	0.51-0.70
11.0-13.9	461	0.70-0.90
14.0-16.9	79	0.90-1.09
17.0-19.9	9	1.09-1.28

Table 8.3.3A - Wave Frequencies

Equivalent force / moments sets were then evaluated for each design group and input into CONCRETE. Fatigue lives for each of the 42 design groups were calculated for concrete, reinforcement and cables respectively and are summarised in Table 8.3.3B.

DG	LOCATION	PREDICTED FATIGUE LIFE (YEARS)		
		concrete	rebar	prestress
1	Base slab	231	>1000	>1000
2	Base slab	721	>1000	>1000
3	Base slab	581	>1000	>1000
4	Base slab	281	>1000	no cables
5	Base slab	230	>1000	no cables
6	Base slab	281	>1000	no cables
7	Base slab	270	62	no cables
8	Base slab (tunnel floor)	252	>1000	no cables
9	Base slab	186	>1000	>1000
10	Base slab (bottom of central shaft)	332	>1000	>1000
11	Antiscour wall	178	>1000	>1000
12	Perforated wall	263	>1000	>1000
13	Exterior wall	120	>1000	>1000
14	Tunnel end wall	283	>1000	no cables
15	Lower lobed wall 0.3-3.85	332	>1000	
16	Interior wall	270	>1000	>1000
17	Central Shaft (Y) 0.3 - 3.85	298	>1000	
18	Central shaft 0.3-18	291	>1000	>1000
19	Int Diaphragm (inner) 0.3-18	120	>1000	>1000
20	Int Diaphragm (outer) 0.3-18	262	>1000	>1000
21	Ext Diaphragm	49	>1000	>1000
22	Ext Diaphragm	64	194	311
23	Ext Diaphragm	141	>1000	>1000
24	Tunnel walls *	n/a	n/a	n/a
25	Tunnel roof *	n/a	n/a	n/a
26	Lobed wall 3.85-18	155	>1000	>1000
27	Central Shaft (Y) 3.85-18	86	502	>1000
28	Lobed wall 18-65	105	>1000	>1000
29	Central shaft (Y) 18-31	248	224	>1000
30	Central shaft 18-31	257	>1000	>1000
31	Int Diaphragm (inner) 18-31	78	>1000	>1000
32	Int Diaphragm (outer) 18-31	164	>1000	>1000
33	Central shaft (Y) 31-65	231	>1000	>1000
34	Central shaft 31-65	254	>1000	>1000
35	Int Diaphragm (inner) 31-65	86	>1000	>1000
36	Int Diaphragm (outer) 31-65	230	>1000	>1000
37	Breakwater wall 65-68	113	509	>1000
38	Breakwater wall 68-105	199	>1000	>1000
39	Central shaft 65-127.6	249	>1000	n/a
40	Central shaft 127.6-147.8	309	>1000	>1000
41	Central Shaft (Y) 65-68	265	>1000	>1000
42	Int Diaphragm 65-68	161	>1000	>1000
	MINIMUM	49	62	311

* Not subject to cyclic loading

Table 8.3.3B - Fatigue Life Summary

9.0 DYNAMIC ANALYSES

The dynamic response of both the whole structure and the individual components were investigated to ensure that there were no significant effects.

The natural period of the whole structure was calculated by hand and compared to the value of 1.4 sec measured on site by Fugro McCelland over a number of years.

The natural period of a number of structural components considered to be sensitive to dynamic effects were also calculated by hand.

For each, dynamic magnification factors were calculated assuming a lower bound extreme wave period of 10 seconds. The maximum dynamic magnification factor found was 1.02 which occurred for the structure as a whole. It was therefore concluded that dynamic effects could be ignored in the re-analysis. A summary of the results of the analysis are shown in Table 9.0

Location	**Frequency * (Hertz)**	**Period (Seconds)**	**Dynamic Magnification**
Structure as a whole	0.71	1.4	1.02
Columns	4.41	0.22	1
Central Shaft	5	0.2	1
Lobed Wall	5	0.2	1
Beams level 10	5.5	0.18	1
Perforated Wall	13.6	0.08	1
Exterior Wall	16.7	0.06	1
Antiscour Wall	21	0.05	1

* calculated in air

Table 9.0 - Dynamic Magnification Factors (Wave Period = 10 Seconds)

10.0 BOAT IMPACT

10.1 OBJECTIVE

An investigation into the effects of likely boat impact scenarios was conducted to both provide an assessment of the damage that may be caused by a boat impact and also to develop a computer model with which TOTAL can investigate any future collisions.

10.2 SCENARIO EVALUATION

A range of boat impact scenarios were investigated to determine the most credible and critical with respect to the Jarlan wall. The main variables considered were:

- impact energy
- location of impact
- type of impact (bow/side etc)
- probability of impact

The most likely impacting vessel was determined to be a typical 5000t supply boat, with an impact speed of 2 m/s. With added mass coefficients of 1.4 and 1.1 for sideways and bow collisions respectively, this gave impact energies of 14 and 11 MJ.

The point of impact, in accordance with the DnV rules, was considered to range from 10m below LAT to 13m above HAT, corresponding to elevations of +85.26m and +106.0m. The latter indicated that impact on a deck column was possible, but this was disregarded as the scenario was considered unlikely.

The direction of impact was considered to be radial, i.e. towards the centre of the platform.

In addition to the 11/14 MJ case, a 40 MJ collision was investigated as an upper bound energy case. The selected bow and sideways impact scenarios are illustrated in Figure 10.2 for a 5000t supply boat.

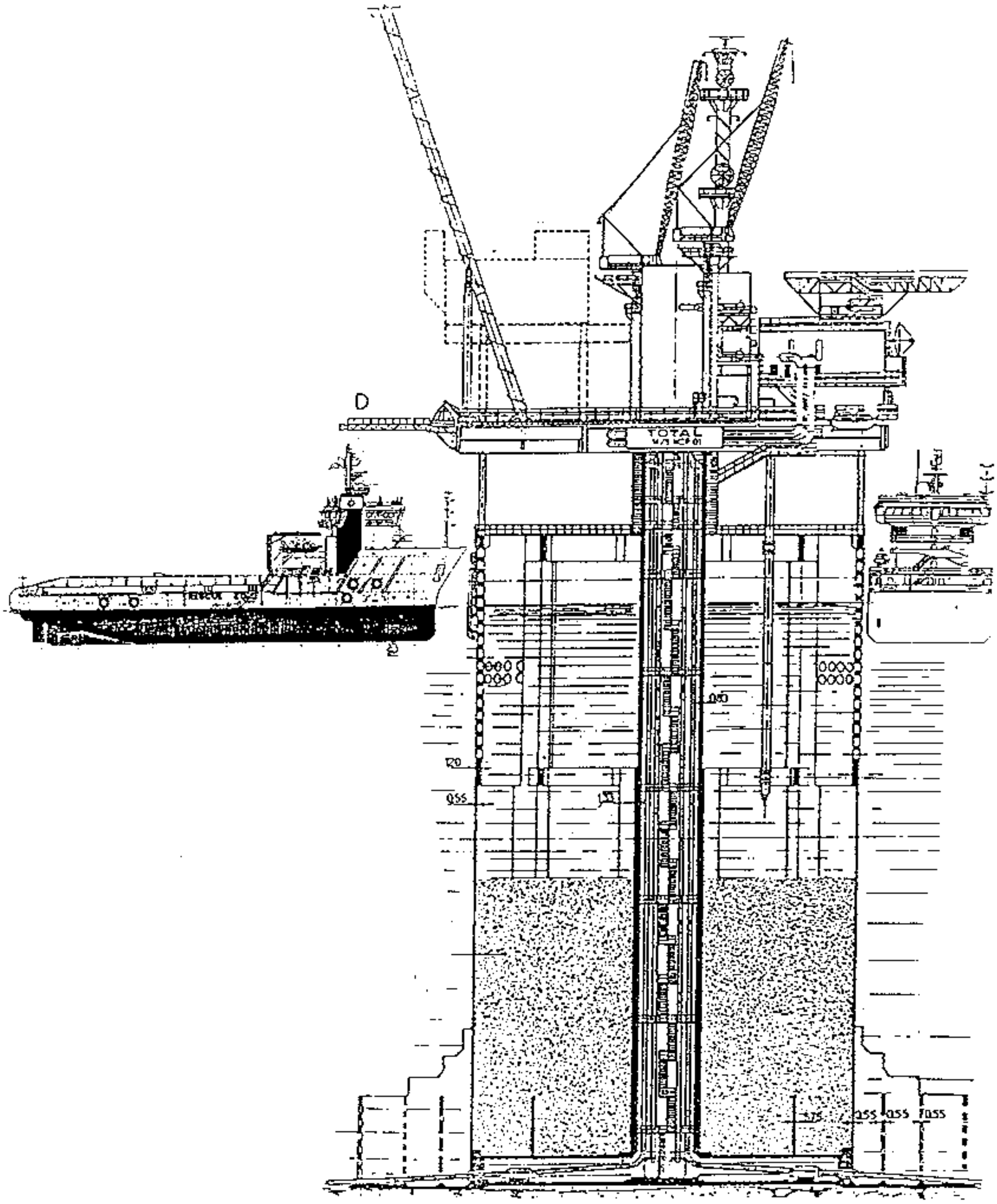


Figure 10.2 - Selected Impact Scenarios

10.3 FORCE CALCULATION

As the stiffness of the Jarlan wall is very large in comparison to that of the impacting vessel, it was considered that the majority of the impact energy would be absorbed by the vessel, and that the impact force applied to the wall could therefore be related to the force/ indentation characteristics of the vessel.

The vessel crushing strength can be calculated from the relevant deck and hull drawings. Drawings and force/indentation curves of a typical supply vessel are given in DnV Recommended Practice Note RP D205, and these were used to calculate the depth of indentation for each energy level.

From the indentation depth calculated and a knowledge of the ships geometry, the area of impact was calculated for bow, stern and side impacts.

The calculated impact forces and impacted areas are shown in Table 10.3.

Type	Energy (MJ)	Indentation (m)	Max Force (kN)	Contact Area m^2	Perimeter (m)	Contact Pressure KN/m^2	Punching Shear kN/m
Bow	11	1.5	13000	6.27	12.1	2080	1074
Stern	11	0.8	21200	27.00	24.2	795	876
Side	14	0.7	29000	27.70	21.4	1046	1355
Bow	40	3.0	36000	17.10	20.4	2107	1765

Table 10.3 - Impact Scenario Comparison

Although the contact pressure and the punching shear for the 40 MJ bow collision were higher than the 11 MJ bow collision case, this case was considered to be unlikely and it was not investigated any further in this instance.

10.4 LOCAL MODEL FOR BOAT IMPACT

10.4.1 General

From the aforementioned loads, simple hand calculations have been performed on the Jarlan wall for the assessment of impact loads. Published results have been used to calculate the capacities of the section. However, they do not show the amount of local damage. Hence, it has been necessary to model the local behaviour using non-linear analysis. Although, the impact forces do not cause severe distress to the structure as a whole, it could produce local damage. In order to assess the local damage, a local model has been prepared using Concrete elements. The model as detailed in Figure H1, Appendix H consists of 2000 concrete elements, and 432 prestressing elements. The total DOF in the model is 4160. The rebars have been modelled in 'smeared' fashion. Equivalent area of the rebars in a given direction have been input to the element. The prestressing has been modelling using discrete elements (using STIF 8). The circular openings in the jarlan wall have been idealised to a square shape in the model. A suitable location for selecting the local model would be to get maximum damage with minimum stiffness. Therefore, the impact has been applied in the central zone of the jarlan wall within 10m below LAT. The concrete properties include crushing strength of 60 MPa and tensile strength of 2.8MPa.

10.4.2 Initial Assessment From Hand Calculations

For bow impact, the predominant failure mode, if any, would be in punching. However, for stern/side impact, predominant failure mode is in bending. The capacity of the Jarlan wall for punching has been estimated to be 54 MN. This depends on the beadth of contact, amount of prestressing, and geometrical parameters of the wall. It can be shown that the FOS against most credible punching is 2.0. The bending and direct shear capacity of the slab has been assessed from the CONCRETE program. Progressive limit state (PLS) type of calculations have been performed on the wall. The results show the concrete stresses are with in 80% of ultimate, but local spalling could occur.

10.4.2 SUBMODELLING

In order to create the boundary conditions for the local model, it is necessary to run the global model. Therefore, a suitable load combination has been identified. Maximum impact force from Section 10.3 has been added to the chosen combination. From the results of the analysis the boundary conditions have been applied to the local model. The size of the local model has been chosen in order to justify the elastic boundaries for the non-linear model.

10.4.3 RESULTS

Appendix H, Figures H2 to H3 show the deflected shape of the wall and the principal stresses in the wall for the 11MJ bow impact. These reveal that the maximum concrete compressive stress under the bow impact is about 25 MPa. No crushing of concrete is observed. The inner fibre has cracked with tensile stresses developing to 6.5 MPa. This also reveals that the reinforcement has not yielded. Also, the stress in the prestressing tendon is 50% of yield stress. Also, from the cracked gauss point location, crack depth has been identified as 0.45m. These results show significant damage can not be expected for the structure under the circumstances. The structure performance is satisfactory under the application of these loads.

However, as a part of the sensitivity study, the load magnitude was increased to twice the value used in the previous analysis. The results revealed that the concrete compressive stresses have gone up by a factor of 1.9 and principal tensile stresses have gone up by a factor of 1.6 compared to those presented in Figures H2 & H3. The local stiffness could be reduced considerably under this type of Impact. The implications of such a loss of wall (particularly re-distribution of forces) or damage on the global analysis has not been addressed currently.

11.0 LOCAL MODELS

11.1 GENERAL

The local models have been prepared after identifying the 'hotspots' revealed in the global analysis. Also, the local models are prepared for areas where global modelling is not coarse enough to predict the local results accurately. These local models are described below. Only load combination 10122 (operating wave at LAT, direction 315°) has been applied as it is considered to be generally the most onerous. ULS results have been calculated by factoring the SLS results by 1.4. Submodelling technique in ANSYS have been used where necessary. A total of 6 local models were prepared. Number of elements in all such local models is 12300 with total active 120000 DOF. The local models were as follows:

- Tunnel
- Base Slab
- Tunnel / Interior Wall
- Lobed Wall / Diaphragms
- Star to Circular Shaft
- Base Slab/Lobed Walls

11.2 BASE SLAB

As mentioned, the base slab model is coarse in certain areas particularly near the outer core. Therefore, a detailed model using 8 noded shell elements has been prepared for a quarter slab. The model consists of 1150 elements with about 10000 DOF. The objective of the analysis is to compare the results of the local model with the global model and run code checks for each of the elements from the detailed model. The pressure on the base slab both in extreme and operating conditions have been calculated and are applied for the base slab. The prinicpal stress results are presented in Appendix I, Figure I1.

11.2.1 SLS/ULS Checks

The results from the analysis were read in to CONCRETE post processor. All 10 original design groups consisting of 1150 elements. Detailed assessment has been carried out for design groups 1 to 10 for SLS and ULS conditions. For ULS conditions conservatively a load factor of 1.4 has been used. For SLS conditions, crack pattern and the concrete utilisation pattern have been plotted. These results identify the critical areas and reveal a more detailed picture of the crack pattern in the base slab. Results are tabulated with the main model results in Section 7.2.

11.3 TUNNELS

A local model for the tunnel has been prepared using 8 noded shell elements as shown in Appendix I, Figure I2. The model consists of 1100 elements with 18000 DOF. The tunnel is subjected to ballast, hydrostatic loading in addition to the self weight. The wave loading on the tunnel is not significant except at the tunnel enclosure. The opening in the tunnel end wall has also been taken into account.

11.3.1 SLS/ULS Checks

From the analysis, the results in terms of forces and moments were read into CONCRETE for post-processing. All the 1100 elements were processed for detailed assessment. These have been classified into four original design groups. These cover design groups 8,14,24 and 25. Both SLS and ULS checks have been carried out. Conservatively a load factor of 1.4 has been used for ULS conditions. The detailed analysis reveals as that at the entrance of the tunnel into the shaft is relatively highly stressed. Also, at the tunnel enclosure (known as caisson wall in inspection reports), cracking outwith code requirements has been revealed. The location and magnitude of crack matches with those in the Inspection reports.

11.4 OPENING OF TUNNEL INTO SHAFT (1101A)

Because of the tunnel opening into shaft, the stresses near the opening are not accurate in the global model. Therefore, it was necessary to investigate this area in depth. The design groups covered in the detailed model include 17,19 and 27 from the main model.

The model consists of 2944 eight noded shell elements with approximately 54000 DOF. The loads on the local model include dead load, prestressing load, ballast, hydrodynamic loads. The 'submodelling' technique has been employed to get the boundary conditions for the model. Details of the model are shown in Appendix I, Figure I3.

11.4.1 SLS/ULS Checks

The results from the analysis were read into the CONCRETE Program. A detailed assessment was carried out for design groups 17, 19 and 27 for both ULS and SLS conditions. For the ULS condition a load factor of 1.4 has been conservatively taken. Results are tabulated with the main model results in Section 7.2.

11.5 CHANGE OF CROSS-SECTION OF SHAFT FROM STAR TO CIRCULAR

The stresses in the circular walls and straight walls of the shaft are not accurate in the global model. Also, at level +65.0m the shape of the shaft changes from star model to circular shape. This means that the change in stiffness of the shaft at the point of maximum wave loading. Therefore, it was necessary to examine the results further using a local model. The total no of elements in the model is 2100 corresponding to active 50000 DOFs. The loads on the sub-model include dead load, prestressing, wave pressure, ballast loads, and hydrostatic loads. The boundary conditions have been obtained from the global model.

Details of the model are shown in Appendix I, Figure I4 to I7.

11.5.1 SLS/ULS checks

The results from the analysis were read directly into CONCRETE post-processor. Detailed assessment has been made for the original design groups. Both SLS and ULS conditions have been analysed. A load factor of 1.4 is used in examining the ULS conditions.

11.6 NODE/DIAPHRAGM INTERFACE

At level 65m there are three major geometrical changes in the structure. The diaphragm wall increases in thickness, there is a step change in cross section for the lobed wall and there is also a thick concrete *node*. Although the stiffness has been correctly assessed in global analysis, detailed evaluation of stresses locally is necessary. A local model using SOLID 73 has been prepared for examining the results in detail. The total no. of elements are 2552 with 20000 DOF. Six elements through thickness have been used. Distorted elements have been minimized. The aspect ratio has been limited to about 4.0. The loading applied on the local model include dead, prestress, and wave pressure. The prestressing has been modelled using discrete elements (2 noded LINK 8). The wave loading from the global model (calculated on c/l dimensions) has been transferred on to the outer surface of the local model. Cut boundary displacement method has been applied to compute the nodal boundary conditions. Details of the model are shown in Appendix I, Figures I8 to I11.

11.6.1 Stress Review

Plots of principal stresses for the node are shown in Appendix I, Figure I9. These indicate that the node is generally in compression with the exception of the strut beam intersection (red area at centre of the model). The maximum in-plane compressive stress found was -43 N/mm^2. This is a local effect and the concrete is in triaxial compression and is considered acceptable. However, this was again in the region of the strut beam intersection and in general stresses were between -5.0 and 10.0 N/mm^2. These were considered acceptable.

11.7 BASE SLAB-LOBED WALL INTERFACE

The base slab has a step change in cross section at the junction with the lobed wall. The horizontal forces introduce additional bending in the lobed walls and the slab. Therefore, a detailed examination of these areas has been carried out in a local analysis. The local model has a combination of solid bricks and shell elements. Brick elements are used to model the base and 589 shell elements are used to model the lobed wall. The design groups covered in this model are 8,10,15,16, and 28. It consists of 1696 no brick elements and 589 shell elements. The total no of DOFs is 18677. The loading applied on the local model includes dead, prestressing and wave loads. Sub-modelling technique has been empoyed to find the boundary conditions for the model. The stresses from the brick elements are converted to concrete and steel stresses. The results from shell elements are directly read into CONCRETE post-processor. Both SLS and ULS conditions have been checked.

Details of the model are shown in Appendix I, Figures I12 and I13.

11.7.1 SLS/ULS Checks

The results from the analysis were read into the CONCRETE post processor program. A detailed assessment was carried out for design groups 15 and 26. For the ULS condition a load factor of 1.4 has been conservatively taken. Results are tabluated with the main model results in Section 7.2.

12.0 CONCLUSIONS

12.1 GENERAL

Sections 12.2 to 12.6 summarise the main conclusions from the re-analysis. It should be noted that these conclusions assume that structural deterioration has not occurred to any great extent, and that no rebar or cable corrosion is evident.

12.1.2 Loads

All loads on the structure have been completely re-assessed. The most significant changes with respect to the original 1975 analysis concern the weight of the topsides equipment and the wave loading.

Present topsides weight is estimated to be approximately 12969t dead-weight plus 1722t live load. This represents a small decrease with respect to the 1986 weight control report due to the effects of demanning, dead load rationalisation and the application of more representative live loads on the laydown areas.

The extreme wave height is now estimated to be 26.4m compared to 29.0m used in the original analysis. This has had the effect of decreasing overturning moment and base shear by 23% and 7% respectively. In the operational case, these figures are 20% and 4.5% respectively.

12.1.3 Code Checks

Both ULS and SLS checks have been performed for all design groups in the deck and sub-structure. These checks were performed in accordance with BS8110 rather than ACI 318-71 and FIP-CEB used in the original design.

The maximum utilisation ratio for the deck beams was 103% assuming a value of fcu of 75 MPa. Crack width (0.13mm), were all within the allowable limit of 0.1mm.

For the radial and strut beams at levels 65.0m and 105.0m, the maximum ULS concrete utilisation ratio was 88% (fcu = 75 MPa) and crack widths, with 4 exceptions (dg 81 (0.3mm), dg 85, 86, 87 (0.13mm)) were within the allowable limits of 0.3 and 0.1mm for beams at +65.0 and +105.0 respectively.

For the sub-structure ULS, concrete utilisation ratios were all below 100% with fcu = 75 MPa with 4 exceptions (dg 24 (125%), dg 7 (107%), dg 6 (109%), and dg 1 (101%)). There is no doubt that the different parts of structure have different concrete strengths and the variation from the test results is quite large. However, if plastic theory for the determination of ultimate moments is used the results may be acceptable.

Shear utilisation exceeds 100% for 22 design groups with the maximum of 128% occuring for dg 3, but it was noted that BS 8110 is more stringent for shear checks than the originally used ACI code. Shear capacity evaluation varies significantly between the codes of practice and the shear stresses found would be acceptable under the Norwegian codes or with shear enhancement factors. BS 8110 rules for shear do not strictly cover high strength concrete and are strictly applicable to beam type of structures only. Thus it is considered that the higher shear stresses can be accepted.

Reinforcement utilisation ratios were all below 100% with 4 exceptions (dg 14 (144%), dg 37 (118%), dg 19 (110%) and dg 24 (102%). Many areas of the structure are under reinforced and the quantities of reinforcement vary from 112 to 156 kg/m^3 in the sub-structure and 190 kg/m^3 in the deck. The ductility of the structure for ultimate loads is much greater in this type of structure compared to those consisting of 300 to 400 kg/m^3. Therefore, in the event of failure, the structure may still demonstrate enough ductility to take plastic deformations and re-distribute the internal forces before a local collapse mechanism could occur. From this point of view, although a local mechanism may develop, in general the structure can still carry the loads safely.

Crack width checks were divided between atmospheric (internal) and submerged (external) faces. Allowable crack widths for the internal and external faces are 0.1mm and 0.3mm according to the DEn Guidance Notes. It was found that for the internal face, 7 out of the 9 design groups had crack widths in excess of 0.1mm with a maximum value of 0.78mm occuring for dg 39. For the external face it was found that 16 out of the 42 design groups had crack widths in excess of 0.3mm with a maximum value of 0.63mm occuring for dg 16. However it should be noted that these crack widths are maximum values for a particular design group and that average values are much lower.

In recent years, a considerable effort has been put into establishing what relationship, if any, exists between crack widths and corrosion. The general conclusion from such studies is that small crack widths (say less than 0.6mm) very rarely pose any particular corrosion risk. whatever the nature of the environment. Therefore, these crack widths, particularly in the submerged zone where oxygen levels are low, appear to be acceptable. However, in the atmospheric zone dynamic cracking may occur and therefore it is essential to maintain inspection.

Durability is thus considered to be acceptable provided that adequate inspection is maintained in the areas with excessive crack widths.

12.1.4 Fatigue Checks

A detailed fatigue check was conducted that met the requirements of the DEn Guidelines. The minimum fatigue life found for any design group was 49 years for concrete and 62 years for reinforcement which was in excess of the 40 year projected life of the platform. It was thus concluded that the platform has adequate fatigue life in order to continue operations until 2015.

12.1.5 Boat Impact

A range of boat impact scenarios concerning a 5000t supply boat were investigated. These covered both 11/14MJ collisions. The impact was assumed to occur on the Jarlan Wall. A non-linear analysis and hand calculations concluded that no significant damage would be caused to the structure. However this might result in local spalling of concrete. The effect of a larger impact has not been addressed in the study, but it is anticipated that a re-distribution of stresses may occur, thus limiting damage.

12.1.6 Foundation Status

Factors of safety against sliding, overturning and bearing capacity were re-assessed taking into account the effects of increased topsides load and decreased wave loads. The most critical factor of safety was a bearing capacity of 1.83 compared to a minimum allowable of 1.5. This showed an increase from the corresponding value of 1.45 calculated in the 1987 re-appraisal. It was thus concluded that foundation status was satisfactory.

PART 1

APPENDICES A TO I

INDEX TO ALL COLOUR FIGURES:

-ve stress	:	Compressive stresses
+ve stress	:	Tensile stresses
⟵⟶	:	Tensile principle stress vectors in black, green and blue relate to S1, S2 and S3 vectors respectively.
→⟵	:	Compressive principle stress vectors; black, green and blue relate to S1, S2 and S3 vectors respectively.
S1, S2 and S3	:	Principle stresses in three directions.
TOP	:	Stresses at the top surface of a shell element.
BOT	:	Stresses at the bottom surface of a shell element.
SMX/MX	:	Maximum value.
SMN/MN	:	Minimum value.
NOAVG	:	Stresses are not averaged over adjacent elements or at interface nodes.
AVG	:	Stresses are averaged over adjacent element or at interface nodes.

APPENDIX A - MISCELLANEOUS

APPENDIX A1

ODE CALCULATION NOTES:

TITLE	CTR NO
Re-assessment of Topsides Loads	301
Marine Growth Appraisal	302
Wind Loads	303
Snow and Ice Loads	304
Wave and Current Loads	305
Substructure and Ballast Deadweights	306
Prestress Loads	307
Temperature Loads	308
Load Combinations	309
Material Properties	401
Dynamic Effects	703
Calculation Boat Impact Loads	802

MACDG FILES AND NOTES:

Project Documents:

NO	TITLE	CTR NO
FILE A	Background Documents	
FILE B	Project Proposal	
FILE C	Main Project File	
FILE D	ODE Supplied Documents	
FILE E	Drawings	
FILE F	Drawings	
FILE G	Draft Report	1201
FILE H	References	

Calculation Files:

NO	TITLE	CTR NO
FILE 1	Topsides Modelling	502
FILE 2	Substructure Modelling	101 to 105/ 501 to 506/ 601/602
FILE 3	Wave Loads from Doris	305
FILE 4	Design Groups	201 / 202 / 203
FILE 5	Code Checks Validation	
FILE 6	Ship-Impact Analysis	801 / 802 / 902
FILE 7	Additional Work	1501 to 1504
FILE 8	Sub-Structure Code Checks (SLS)	603

Calculation Files (Con't):

NO:	TITLE	CTR NO:
FILE 9	Sub-Structure Code Checks (ULS)	601
FILE 10	Sub-Structure Code Checks (Fatigue)	504
FILE 11	Beams Code Checks (SLS)	603
FILE 12	Beams Code Checks (ULS)	603
FILE 13	Local Model of Base Slab	1101D
FILE 14	Local Model of Tunnel	1101E
FILE 15	Local Model 1101A	1101A
FILE 16	Local Model 1101B	1101B
FILE 17	Local Model 1101C	1101C
FILE 18	Local Model 1101D	1101D

MCP-01 RE-ANALYSIS DOCUMENT REGISTER

Date 10 JAN 94

DOC TITLE	SOURCE	SOURCE DOC NO	PROJECT DOC NO	ISSUE DATE	DATE RE
MCP-01 STRUCTURAL MONITORING	FUGRO	92/3047/R02	1074	4/93	12/93
MCP-01 STRUCTURAL MONITORING	FUGRO	92/3047/R04	1073	7/93	12/93
MCP-01 STRUCTURAL MONITORING REPORTS	FUGRO	92/3047/R03	1072	4/93	12/93
MCP-01 STRUCTURAL MONITORING REPORTS	FUGRO	92/3047/R01	1071	12/92	12/93
MCP-01 LIFEBOAT STUDY	DORIS	DE 1653	1070		SEP 93
MCP-01 PLATFORM CONVERSION PROJECT WEIGHT REPORT VOLUME 3	FOSTER WHEELER	0167MPMTN001	1069	24/9/92	7/7/93
MCP-01 PLATFORM CONVERSION PROJECT WEIGHT REPORT VOL 2	FOSTER WHEELER	0167MPMTN001	1068	24/8/92	7/7/93
MCP-01 PLATFORM CONVERSION PROJECT WEIGHT REPORT VOLUME 1	FOSTER WHEELER	0167MPMTN001	1067	24/8/92	7/7/93
WEIGHT CONTROL REPORT NO 1 MANIFOLD PH2 MODULE 01	TOTAL		1066		7/7/93
MCP-01 TOPSIDES WEIGHT CONTROL REPORT-1 JULY 1986	TOTAL		1065		7/7/93
MCP-01 1985 STATUS REPORT FOR THE PRIMARY CONCRETE STRUCTURE	TOTAL		1064		7/7/93
MCP-01 UNDERWATER CONCRETE INSPECTION 1992	TOTAL		1063		7/7/93
SUMMARY OF INSPECTION REPORTS ON FOUNDATION AND SCOUR SURVEY 1986-1992	TOTAL		1062		7/7/93
LIST OF UNDERWATER INSPECTION REPORTS 1977-1992 LIST FO UNDERWATER INSPECTION REPORTS	TOTAL		1061		7/7/93

MCP-01 RE-ANALYSIS DOCUMENT REGISTER

Date 10 JAN 94

DOC TITLE	SOURCE	SOURCE DOC NO	PROJECT DOC NO	ISSUE DATE	DATE RE
BROWN & ROOT STEEL DECK DRAWINGS	BROWN&ROOT		1060		
MAIN DECK LOADS	CCE	MP5009M4	1059	8/82	2/7/93
DECK LOADINGS	TOTAL		1058	9/1985	2/7/93
ENVIRONMENTAL CONDITIONS MCP01 VOLUME 2-OPERATIONAL CRITERIA	MAREX	1167	1057	11/5/93	2/7/93
ENVIRONMENTAL CONDITIONS FOR MCPO1 VOLUME 1 DESIGN CRITERIA	MAREX	1167	1056	26/2/93	2/7/93
1991 MARINE FOULING ASSESSMENT	AURIS		1055	4/93	2/7/93
MANIFOLD COMPRESSION PLATFORM	TOTAL		1054		
EQUIPMENT LOCATION DRAWINGS	TOTAL	MP2790S00010000	1053	5/10/90	2/7/93
MCPO1 FOUNDATIONS DESIGN	DORIS	MP2D1055	1052	7/1975	
1982 MCP01 STRUCTURAL SURVEY UNDERWATER INSPECTION OF FOUNDATIONS	TOTAL	241D12/ILH	1051	3/1983	
LRS SUPPORT CERTIFICATION FILE	HOWARD DORIS		1050		
INSTALLATION STUDY MCPO1	HEEREMA	LS/1.50.00 REVA	1049	MARCH 1981	
MCPO1 COMPRESSION PROJECT PERMANENT LIVING QUARTERS	HOWARD DORIS	MP5023M6CN102	1048		
CPO1 STRUCTURAL DESIGN REPORT VOLUME 2 UTILITIES MODULE	BROWN AND ROOT		1047		
UNDERDECK DIVING SYSTEM FOR THE MCP-01 PLATFORM	HOWARD DORIS		1046		

MCP-01 RE-ANALYSIS DOCUMENT REGISTER

Date 10 JAN 94

DOC TITLE	SOURCE	SOURCE DOC NO	PROJECT DOC NO	ISSUE DATE	DATE RE
VOLUME 1					
MCPO1 FOUNDATION STUDY	CDE	87-281	1045		
COMPRESSION PROJECT MCPO1 CONTRACT C-5009 STRUCTURAL DESIGN REPORT VOL3 AND 4	CDE	MP5009 M4CL31	1044	AUG 1982	
COMPRESSION PROJECT MCPO1 CONTRACT C-5009 STRUCTURAL DESIGN REPORT VOL2 PART1	CDE	MP C5009M4CL31	1043	AUG 1982	
OXY RISER	DORIS	VOLUME 6	1042		
18" OXY RISER DESIGN MANUAL VOLS 1-3	TOTAL		1041		
MAREX PROPOSAL FOR ENVIRONMENTAL DESIGN CRITERIA	MAREX	P3883	1040	29/6/92	4/11/92
BEHAVIOUR OF THE CONCRETE STRUCTURE AND ITS FOUNDATION SOIL TAKING INTO ACCOUNT MARINE GROWTH	DORIS	CGD D1850 REV B	1039	24/282	2/12/92
EHAVIOUR OF THE CONCRETE STRUCTURE AND ITS FOUNDATION SOIL TAKING INTO ACCOUNT MARINE GROWTH	DORIS	CG-D D1588	1038		20/11/92
REVIEW OF MCP01 FOUNDATION STATUS		MPI/TIR/86/03	1037		20/11/92
EVALUATION OF FOUNDATION STABILITY		NGI 87314-1	1036		4/11/92
MCPO1 FOUNDATION STUDY STUDY 86/8	CDE	ODE 87-281 REV2	1035	4/11/92	
SPECIFICATION FOR INSPECTION OF MCPO1	DORIS	2135	1034	JULY 1982	
'CPO1 UNDERWATER INSPECTION ,AN/FEB 1992	MCALPINE		1033	18/3/92	10/12/92
MCP01 UNDERWATER INSPECTION OF CONCRETE SUB-STRUCTURE OCTOBER NOVEMBER 1991	MCALPINE		1032	16/12/91	10/12/92

MCP-01 RE-ANALYSIS DOCUMENT REGISTER

Date 10 JAN 94

DOC TITLE	SOURCE	SOURCE DOC NO	PROJECT DOC NO	ISSUE DATE	DATE RE
DURABILITY ASSESSMENT OF CONCRETE SAMPLES FROM MCP01 PLATFORM	MCALPINE		1031	5/12/89	10/12/92
ATMOSPHERIC INSPECTION YEARBOOK 1989	MCALPINE		1030	JAN 89	10/12/92
ATMOSPHERIC INSPECTION YEARBOOK 1988	MCALPINE		1029	DEC1988	10/12/92
ATMOSPHERIC INSPECTION YEARBOOK 1987	MCALPINE		1028	APRIL 1987	10/12/92
ATMOSPHERIC INSPECTION 1987 YEARBOOK	MCALPINE		1027	JULY 1986	10/12/92
ASSESSMENT OF CURRENT CONDITION	MCALPIN E		1026	11/7/85	10/12/85
ATMOSPHERIC INSPECTION YEARBOOK 1985	MCALPINE		1025	JUNE 1985	10/12/92
ATMOSPHERIC INSPECTION 1984	MCALPINE		1024	AUG 1984	10/12/92
ATMOSPHERIC INSPECTION YEARBOOK 1990	MCALPINE		1023	MAR 1991	8/12/92
ATMOSPHERIC INSPECTION YEARBOOK 1991	MCALPINE		1022	FEB 1991	8/12/92
APPRAISAL OF THE 1982-85 INSPECTION PROGRAMME	ODE	ODE 86-250 REVA	1021	JULY 86	
APPRAISAL OF THE 1978-1981 INSPECTION PROGRAMME	DORIS	D 2109	1020	JULY 1982	2/12/92
COMMENTS ABOUT THE REPORTS OF THE INSPECTION PERFORMED 1977-78 APPENDIX C TO 5237	DORIS	5235 REV 1	1019	2/79	10/11/92
1982 MCP01 STRUCTURAL SURVEY UNDERWATER INSPECTION OF FOUNDATIONS-EXTRACTS		241 D12/ILH	1018	MARCH 1983	

MCP-01 RE-ANALYSIS DOCUMENT REGISTER

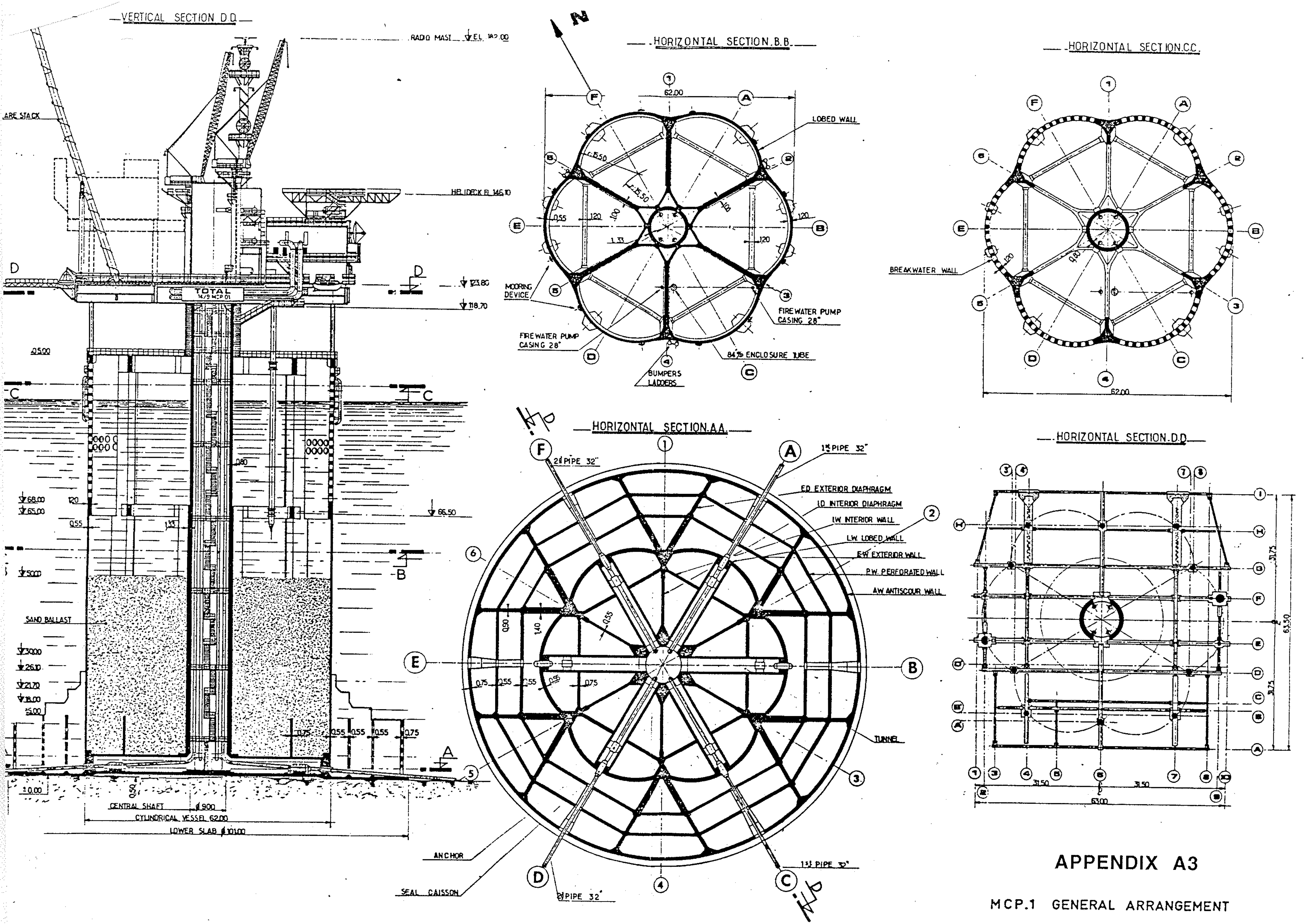
Date 10 JAN 94

DOC TITLE	SOURCE	SOURCE DOC NO	PROJECT DOC NO	ISSUE DATE	DATE RE
PRIMARY CONCRETE STRUCTURE ANOMALY LOCATIONS (CONCRETE SUB-STRUCTURE DEFECTS VOL 6)	TOTAL MARINE	DS 1-406	1017		
STUCTURAL MONITORING SYSTEM JUNE TO AUGUST 1982	STRUCTURAL MON	290	1016	10/82	20/11/82
SPECIFICATION FOR THE INSPECTION OF FRIGG MCPO1 STRUCTURE	DORIS	5237	1015	MARCH 1979	10/11/92
VOLUME 4 SECTION 4	HOWARD DORIS		1014		
VOLUME 7 SECTION 1	HOWARD DORIS		1013		
VOLUME 7 SECTION 21	HOWARD DORIS		1012		
BASIC DESIGN DATA VOLUME 1	DORIS	21946	1011		
CONCRETE SUB-STRUCTURE CABLES & REINFORCEMENT	DORIS	VOLUME 2	1010		
CONCRETE SUBSTRUCTURE G.A.s	DORIS	VOLUME 1	1009		
COMPARISON ACI 318 & BS8110	CCE		1008	NOV 92	
CALCULATION NOTES	DORIS	VOLUME 4/4	1007		
CALCULATION NOTES	DORIS	VOLUME 3/4	1006		
CALCULATION NOTES	DORIS	VOLUME 2/4	1005		
CALCULATION NOTES	DORIS	VOLUME 1/4	1004		

MCP-01 RE-ANALYSIS DOCUMENT REGISTER

Date 10 JAN 94

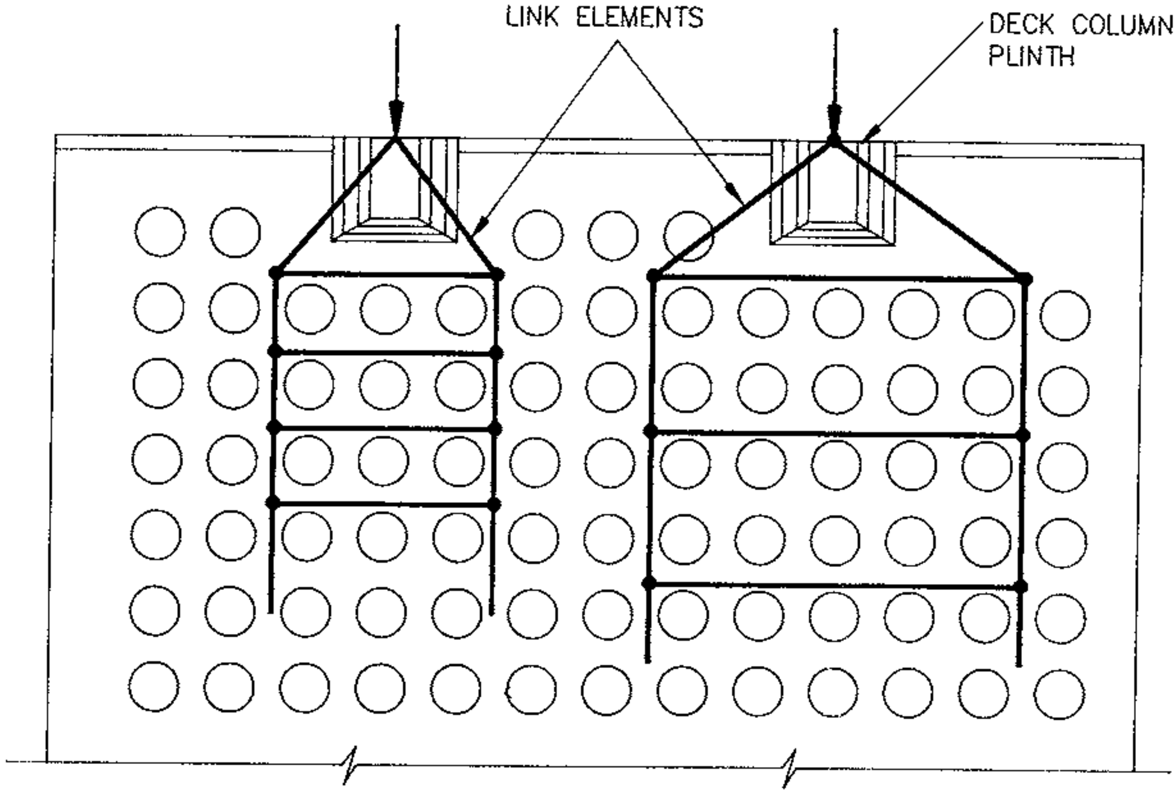
DOC TITLE	SOURCE	SOURCE DOC NO	PROJECT DOC NO	ISSUE DATE	DATE RE
DECK LOADS	DORIS	VOLUME 7	1003		
COMPRESSION PROJECT	DORIS	VOLUME 4	1002		
CONCRETE DECK BEAMS - GA & REINF CABLES	DORIS	VOLUME 3	1001		



APPENDIX A3

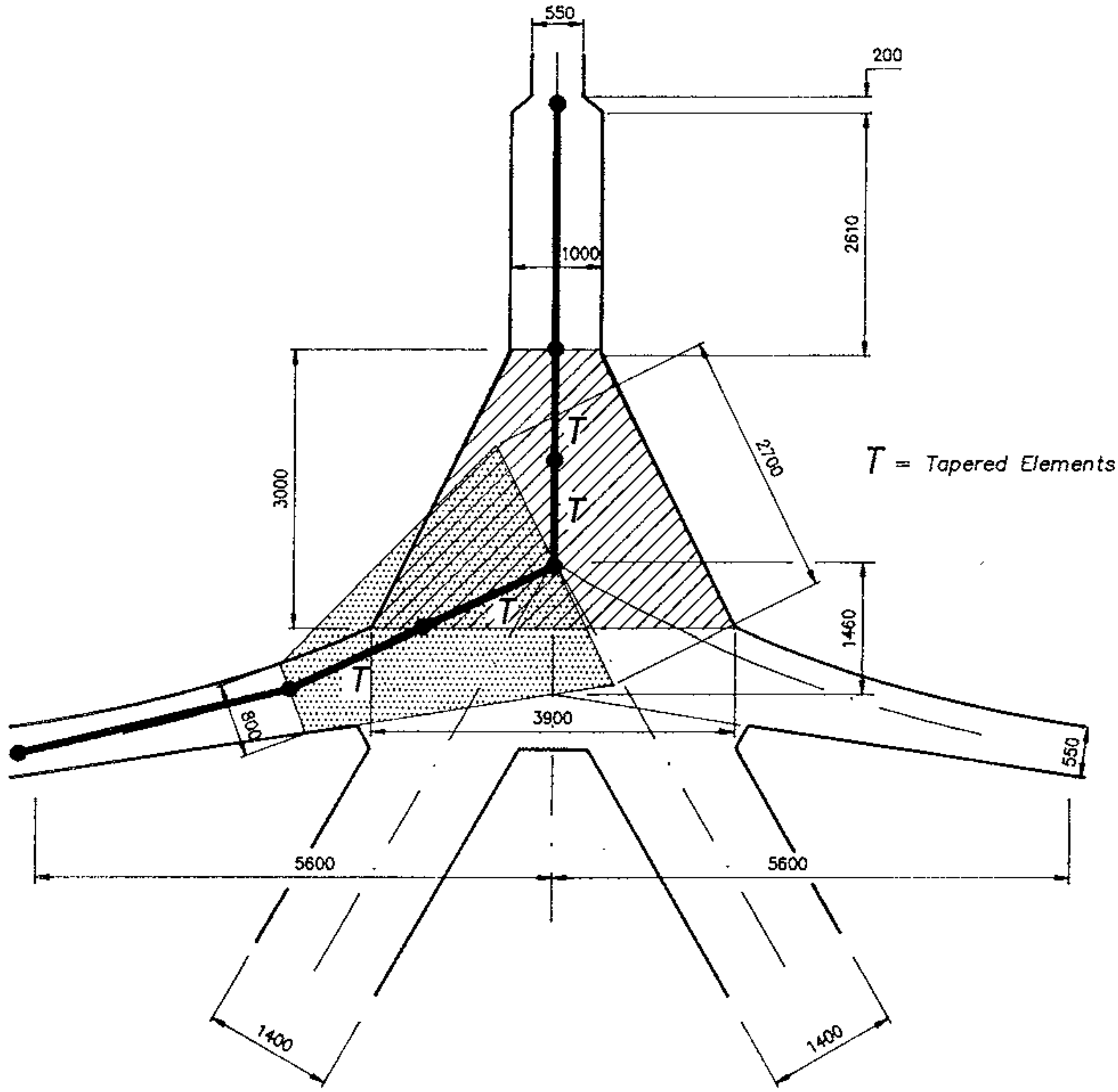
MCP.1 GENERAL ARRANGEMENT

APPENDIX B FE MODEL DETAILS



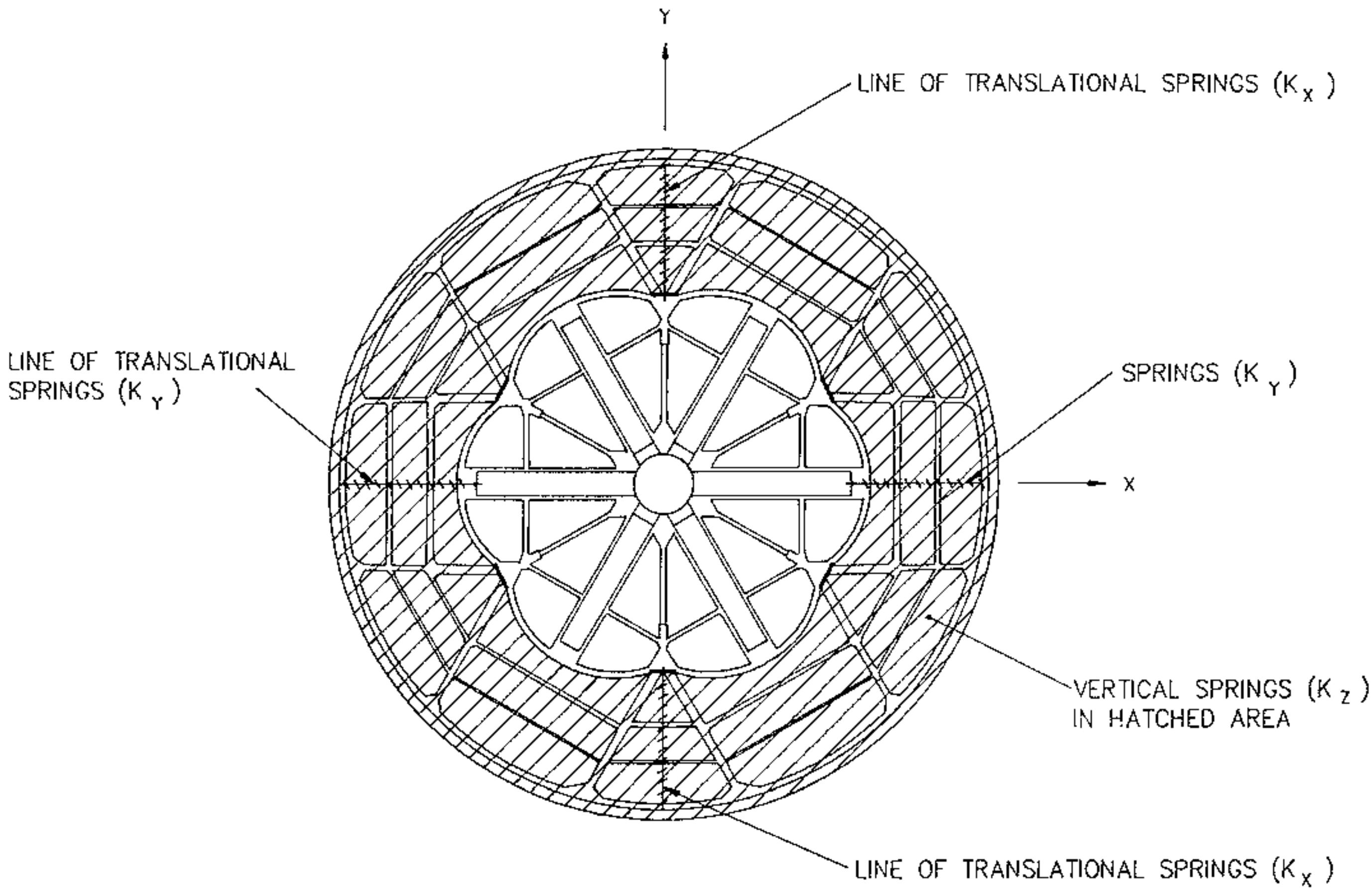
DECK COLUMN/JARLAN WALL INTERFACE
STRUT/TIE MODELS

FIGURE B1



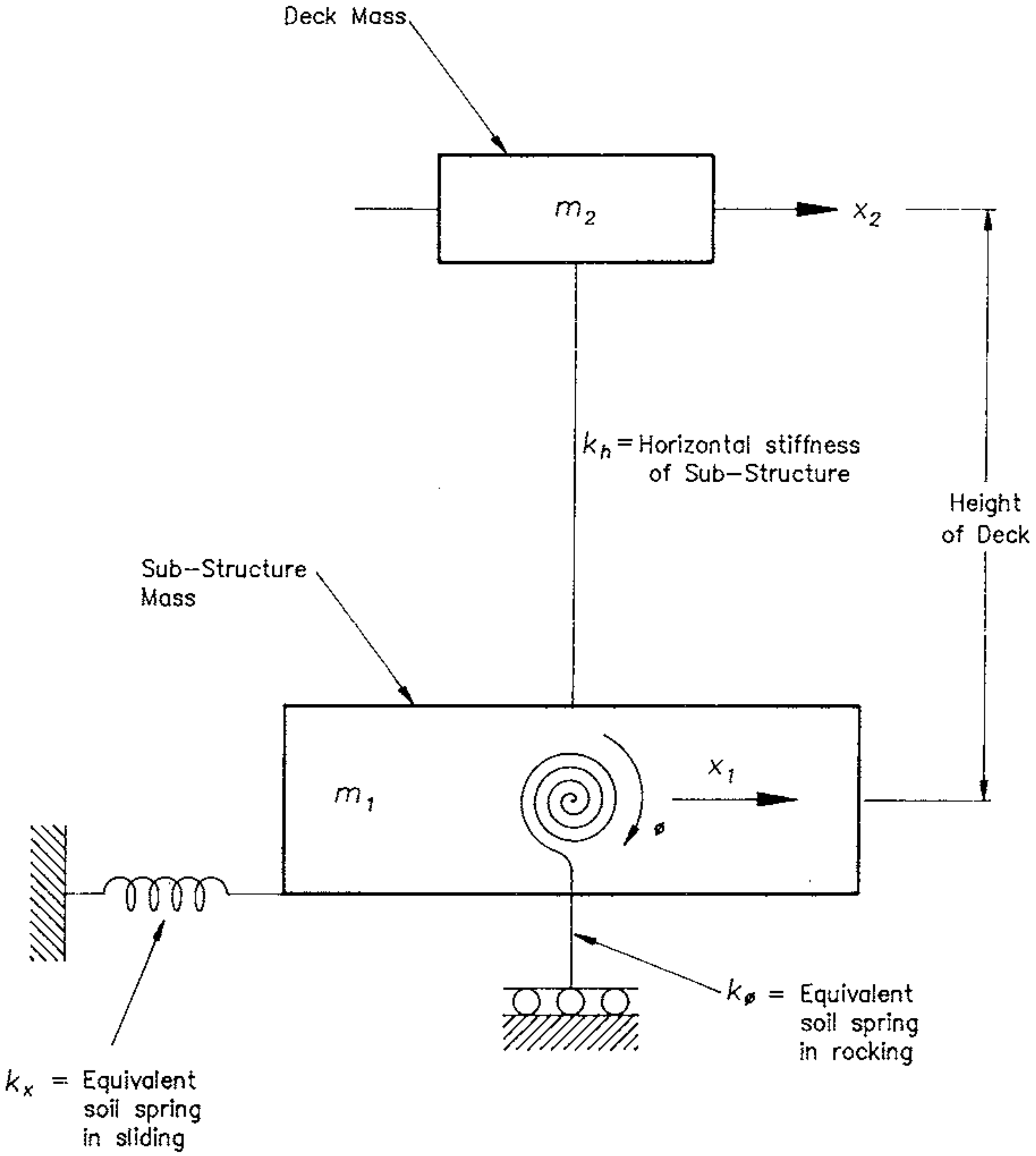
TAPERED ELEMENTS AT TYPICAL JUNCTIONS

FIGURE B2



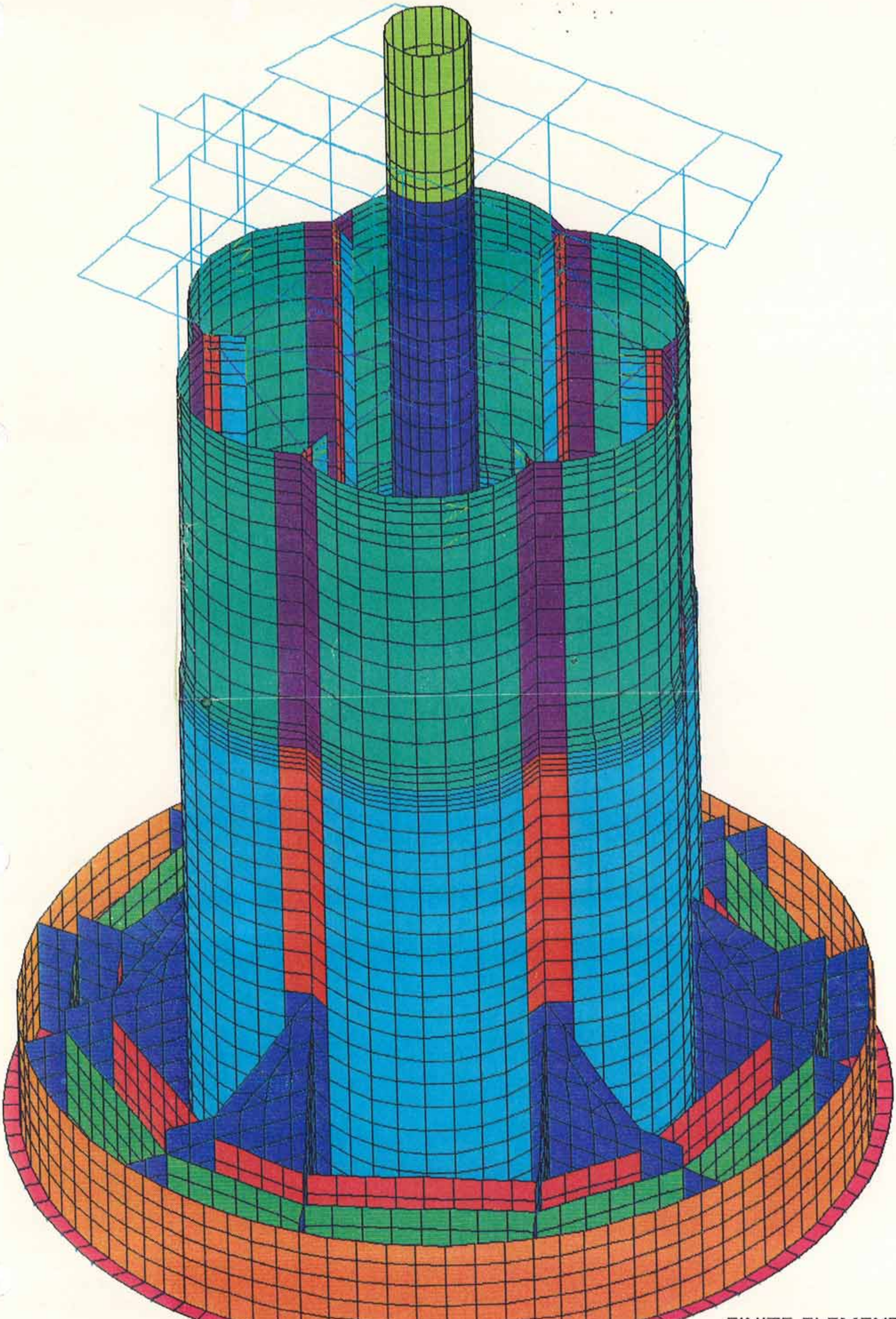
SOIL SUPPORTS

FIGURE B3



DYNAMIC ANALYSIS MODEL

FIGURE B4



FINITE ELEMENT MODEL

FIGURE B5

APPENDIX C - LOADS

C1 Load combinations

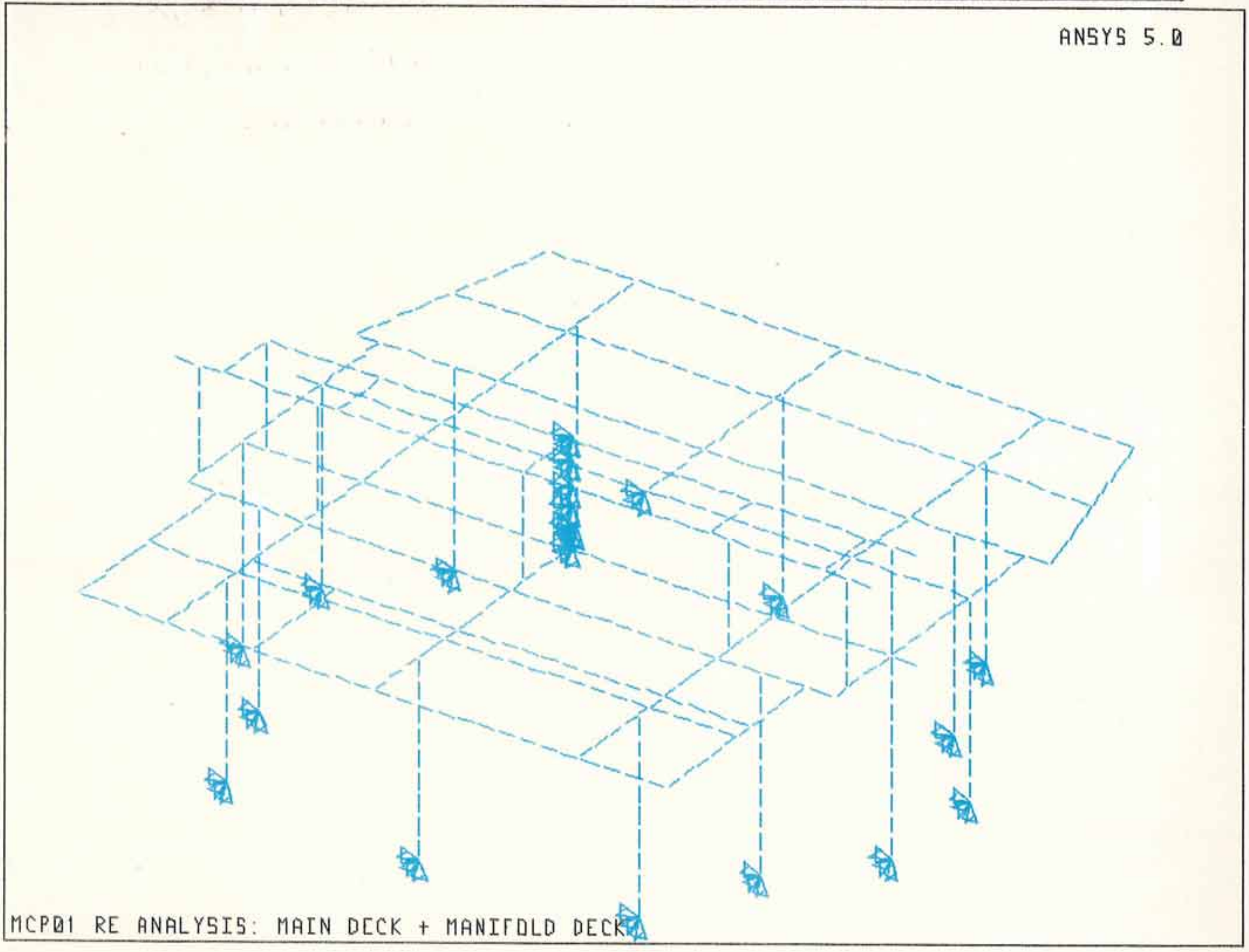
REF	COND	LCOMB	DESCRIPTION
	ULS	101	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 300 PHASE 60)
		102	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 315 PHASE 60)
		103	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 330 PHASE 60)
		104	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 300 PHASE 60)
		105	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 315 PHASE 60)
		106	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 330 PHASE 60)
2	ULS	107	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 300 PHASE 60)
		108	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 315 PHASE 60)
		109	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 330 PHASE 60)
		1010	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 300 PHASE 60)
		1011	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 315 PHASE 60)
		1012	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 330 PHASE 60)
3	ULS	1013	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 300 PHASE 60)
		1014	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 315 PHASE 60)
		1015	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 330 PHASE 60)
		1016	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 300 PHASE 60)
		1017	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 315 PHASE 60)
		1018	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 330 PHASE 60)
4	ULS	1019	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 300 PHASE 60)
		1020	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 315 PHASE 60)
		1021	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 330 PHASE 60)
		1022	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 300 PHASE 60)
		1023	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 315 PHASE 60)
		1024	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 330 PHASE 60)
5	ULS	1025	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE 15)
		1026	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE -60)
		1027	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE 15)
		1028	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE -60)
		1029	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE 15)
		1030	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE -60)
		1031	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE 15)
		1032	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE -60)
		1033	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE 15)
		1034	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE -60)
		1035	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE 15)
		1036	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE -60)
6	ULS	1037	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE 15)
		1038	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE -60)
		1039	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE 15)
		1040	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE -60)
		1041	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE 15)
		1042	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE -60)
		1043	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE 15)
		1044	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE -60)
		1045	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE 15)
		1046	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE -60)
		1047	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE 15)
		1048	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE -60)
7	ULS	1049	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE 15)
		1050	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE -60)
		1051	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE 15)
		1052	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE -60)
		1053	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE 15)
		1054	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE -60)
		1055	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE 15)
		1056	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE -60)
		1057	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE 15)
		1058	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE -60)
		1059	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE 15)
		1060	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE -60)
8	ULS	1061	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE 15)
		1062	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE -60)
		1063	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE 15)
		1064	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE -60)
		1065	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE 15)
		1066	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE -60)
		1067	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE 15)
		1068	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE -60)

REF	COND	LCOMB	DESCRIPTION
	ULS	101	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 300 PHASE 60)
		102	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 315 PHASE 60)
		103	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 330 PHASE 60)
		104	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 300 PHASE 60)
		105	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 315 PHASE 60)
		106	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 330 PHASE 60)
2	ULS	107	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 300 PHASE 60)
		108	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 315 PHASE 60)
		109	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT LAT (DIR 330 PHASE 60)
		1010	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 300 PHASE 60)
		1011	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 315 PHASE 60)
		1012	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + OPERATING WAVE AT HAT (DIR 330 PHASE 60)
3	ULS	1013	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 300 PHASE 60)
		1014	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 315 PHASE 60)
		1015	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 330 PHASE 60)
		1016	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 300 PHASE 60)
		1017	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 315 PHASE 60)
		1018	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 330 PHASE 60)
4	ULS	1019	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 300 PHASE 60)
		1020	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 315 PHASE 60)
		1021	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT LAT (DIR 330 PHASE 60)
		1022	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 300 PHASE 60)
		1023	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 315 PHASE 60)
		1024	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MIN PRESTRESS + OPERATING WAVE AT HAT (DIR 330 PHASE 60)
5	ULS	1025	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE 15)
		1026	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE -60)
		1027	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE 15)
		1028	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE -60)
		1029	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE 15)
		1030	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE -60)
		1031	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE 15)
		1032	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE -60)
		1033	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE 15)
		1034	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE -60)
		1035	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE 15)
		1036	MAX DEAD + MAX LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE -60)
6	ULS	1037	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE 15)
		1038	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE -60)
		1039	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE 15)
		1040	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE -60)
		1041	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE 15)
		1042	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE -60)
		1043	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE 15)
		1044	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE -60)
		1045	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE 15)
		1046	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE -60)
		1047	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE 15)
		1048	MIN DEAD + ZERO LIVE + MIN LAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE -60)
7	ULS	1049	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE 15)
		1050	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE -60)
		1051	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE 15)
		1052	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE -60)
		1053	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE 15)
		1054	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE -60)
		1055	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE 15)
		1056	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE -60)
		1057	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE 15)
		1058	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 315 PHASE -60)
		1059	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE 15)
		1060	MAX DEAD + MAX LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 330 PHASE -60)
8	ULS	1061	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE 15)
		1062	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 300 PHASE -60)
		1063	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE 15)
		1064	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 315 PHASE -60)
		1065	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE 15)
		1066	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT LAT (DIR 330 PHASE -60)
		1067	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE 15)
		1068	MIN DEAD + ZERO LIVE + MAX HAT HYDROSTATIC+ MAX PRESTRESS + EXTREME WAVE AT HAT (DIR 300 PHASE -60)

		1069	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MAX PRESTRESS +	EXTREME WAVE AT HAT	(DIR 315 PHASE 15)
		1070	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MAX PRESTRESS +	EXTREME WAVE AT HAT	(DIR 315 PHASE -60)
		1071	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MAX PRESTRESS +	EXTREME WAVE AT HAT	(DIR 330 PHASE 15)
		1072	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MAX PRESTRESS +	EXTREME WAVE AT HAT	(DIR 330 PHASE -60)
9	ULS	1073	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 300 PHASE 15)
		1074	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 300 PHASE -60)
		1075	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 315 PHASE 15)
		1076	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 315 PHASE -60)
		1077	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 330 PHASE 15)
		1078	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 330 PHASE -60)
		1079	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 300 PHASE 15)
		1080	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 300 PHASE -60)
		1081	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 315 PHASE 15)
		1082	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 315 PHASE -60)
		1083	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 330 PHASE 15)
		1084	MAX DEAD +	MAX LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 330 PHASE -60)
10	ULS	1085	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 300 PHASE 15)
		1086	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 300 PHASE -60)
		1087	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 315 PHASE 15)
		1088	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 315 PHASE -60)
		1089	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 330 PHASE 15)
		1090	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 330 PHASE -60)
		1091	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 300 PHASE 15)
		1092	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 300 PHASE -60)
		1093	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 315 PHASE 15)
		1094	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 315 PHASE -60)
		1095	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 330 PHASE 15)
		1096	MIN DEAD +	ZERO LIVE +	MIN LAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 330 PHASE -60)
11	ULS	1097	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 300 PHASE 15)
		1098	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 300 PHASE -60)
		1099	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 315 PHASE 15)
		10100	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 315 PHASE -60)
		10101	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 330 PHASE 15)
		10102	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 330 PHASE -60)
		10103	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 300 PHASE 15)
		10104	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 300 PHASE -60)
		10105	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 315 PHASE 15)
		10106	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 315 PHASE -60)
		10107	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 330 PHASE 15)
		10108	MAX DEAD +	MAX LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 330 PHASE -60)
12	ULS	10109	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 300 PHASE 15)
		10110	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 300 PHASE -60)
		10111	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 315 PHASE 15)
		10112	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 315 PHASE -60)
		10113	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 330 PHASE 15)
		10114	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT LAT	(DIR 330 PHASE -60)
		10115	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 300 PHASE 15)
		10116	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 300 PHASE -60)
		10117	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 315 PHASE 15)
		10118	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 315 PHASE -60)
		10119	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 330 PHASE 15)
		10120	MIN DEAD +	ZERO LIVE +	MAX HAT HYDROSTATIC+	MIN PRESTRESS +	EXTREME WAVE AT HAT	(DIR 330 PHASE -60)
13	SLS	10121	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT LAT	(DIR 300 PHASE -60)
		10122	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT LAT	(DIR 315 PHASE -60)
		10123	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT LAT	(DIR 330 PHASE -60)
		10124	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT HAT	(DIR 300 PHASE -60)
		10125	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT HAT	(DIR 315 PHASE -60)
		10126	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT HAT	(DIR 330 PHASE -60)
14	SLS	10127	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT LAT	(DIR 300 PHASE -60)
		10128	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT LAT	(DIR 315 PHASE -60)
		10129	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT LAT	(DIR 330 PHASE -60)
		10130	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT HAT	(DIR 300 PHASE -60)
		10131	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT HAT	(DIR 315 PHASE -60)
		10132	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	OPERATING WAVE AT HAT	(DIR 330 PHASE -60)
15	SLS	10133	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT	(DIR 300 PHASE 15)
		10134	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT	(DIR 300 PHASE -60)
		10135	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT	(DIR 315 PHASE 15)
		10136	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT	(DIR 315 PHASE -60)
		10137	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT	(DIR 330 PHASE 15)
		10138	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT	(DIR 330 PHASE -60)

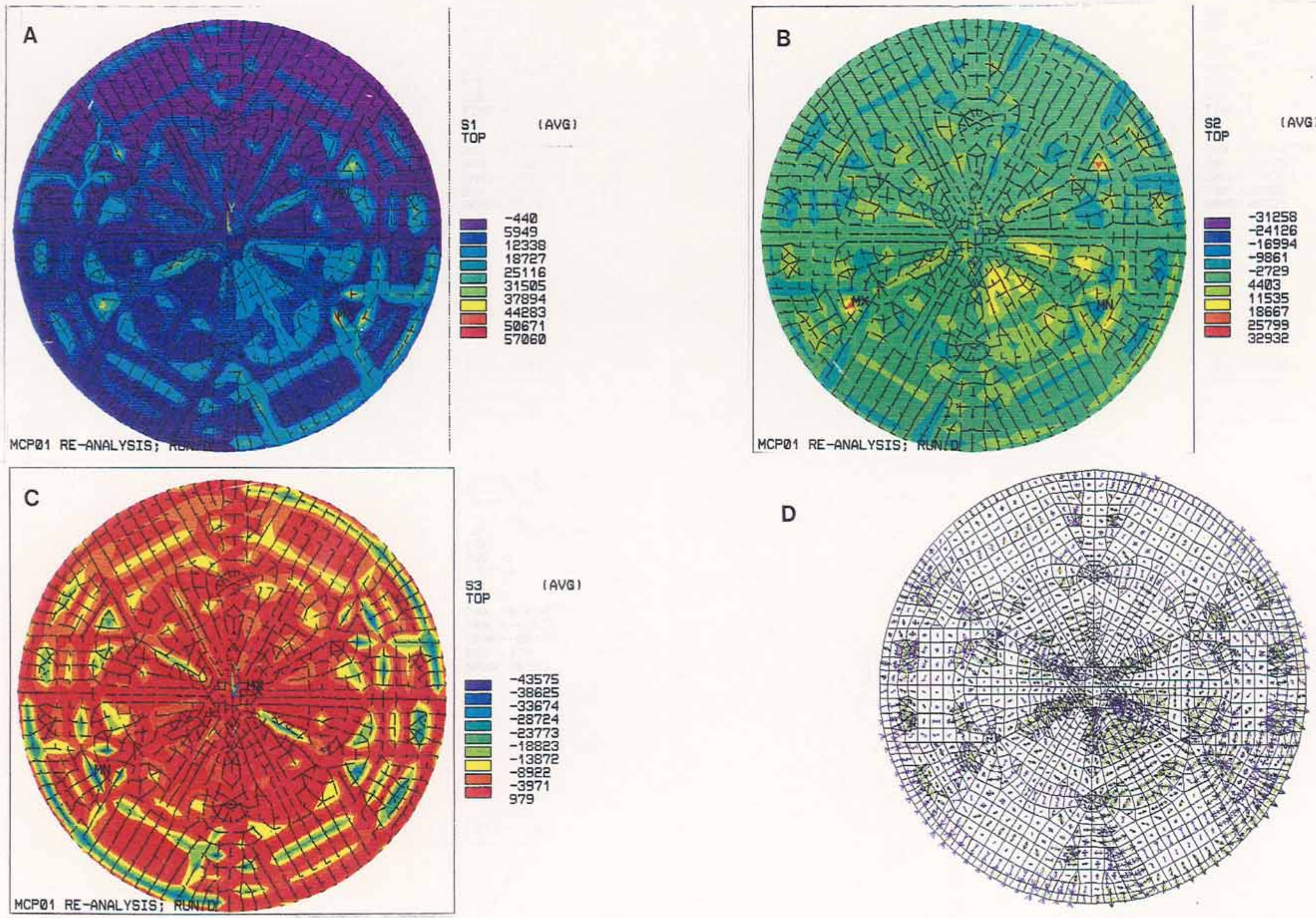
		10139	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 300 PHASE 15)
		10140	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 300 PHASE -60)
		10141	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 315 PHASE 15)
		10142	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 315 PHASE -60)
		10143	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 330 PHASE 15)
		10144	DEAD+	LIVE+	LAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 330 PHASE -60)
16	SLS	10145	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT (DIR 300 PHASE 15)
		10146	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT (DIR 300 PHASE -60)
		10147	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT (DIR 315 PHASE 15)
		10148	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT (DIR 315 PHASE -60)
		10149	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT (DIR 330 PHASE 15)
		10150	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT LAT (DIR 330 PHASE -60)
		10151	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 300 PHASE 15)
		10152	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 300 PHASE -60)
		10153	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 315 PHASE 15)
		10154	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 315 PHASE -60)
		10155	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 330 PHASE 15)
		10156	DEAD+	LIVE+	HAT HYDROSTATIC+	PRESTRESS+	EXTREME WAVE AT HAT (DIR 330 PHASE -60)

APPENDIX D - STRESS / BENDING MOMENT PLOTS



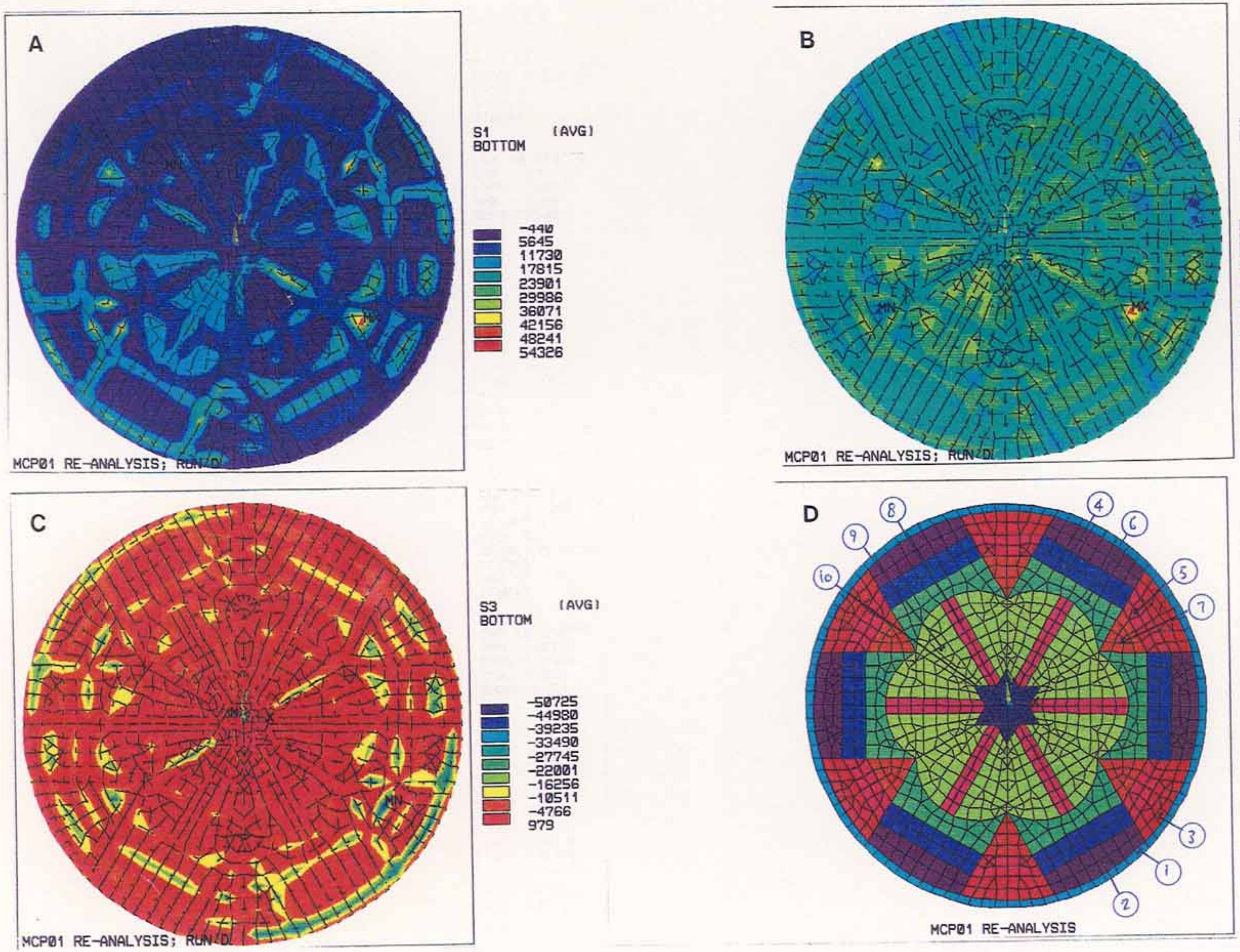
'DECK ONLY' ANALYSIS

FIGURE D1.



BASE SLAB
A) - C) PRINCIPAL STRESSES IN kPa
D) PRINCIPAL STRESS DIRECTIONS

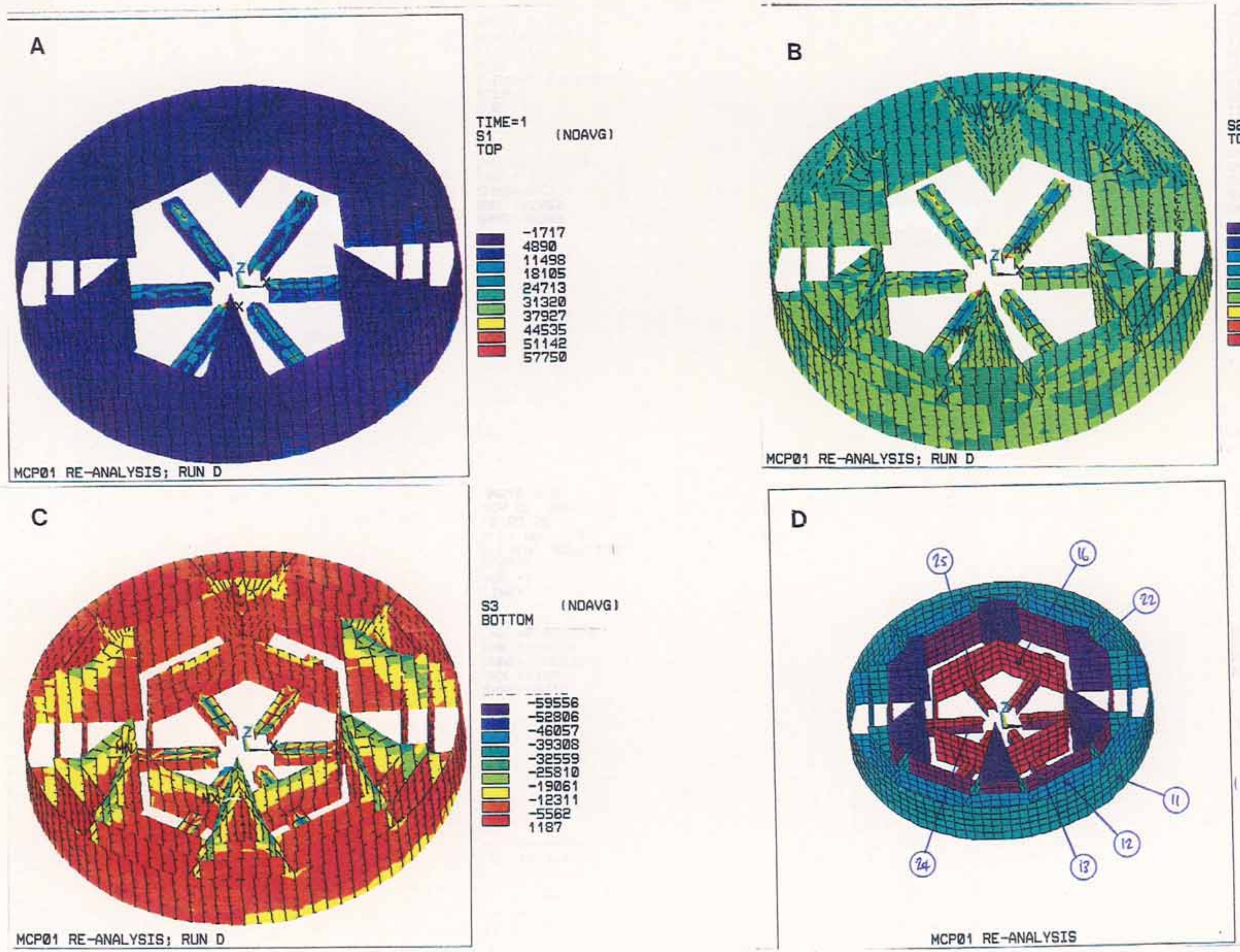
FIGURE D2



BASE SLAB

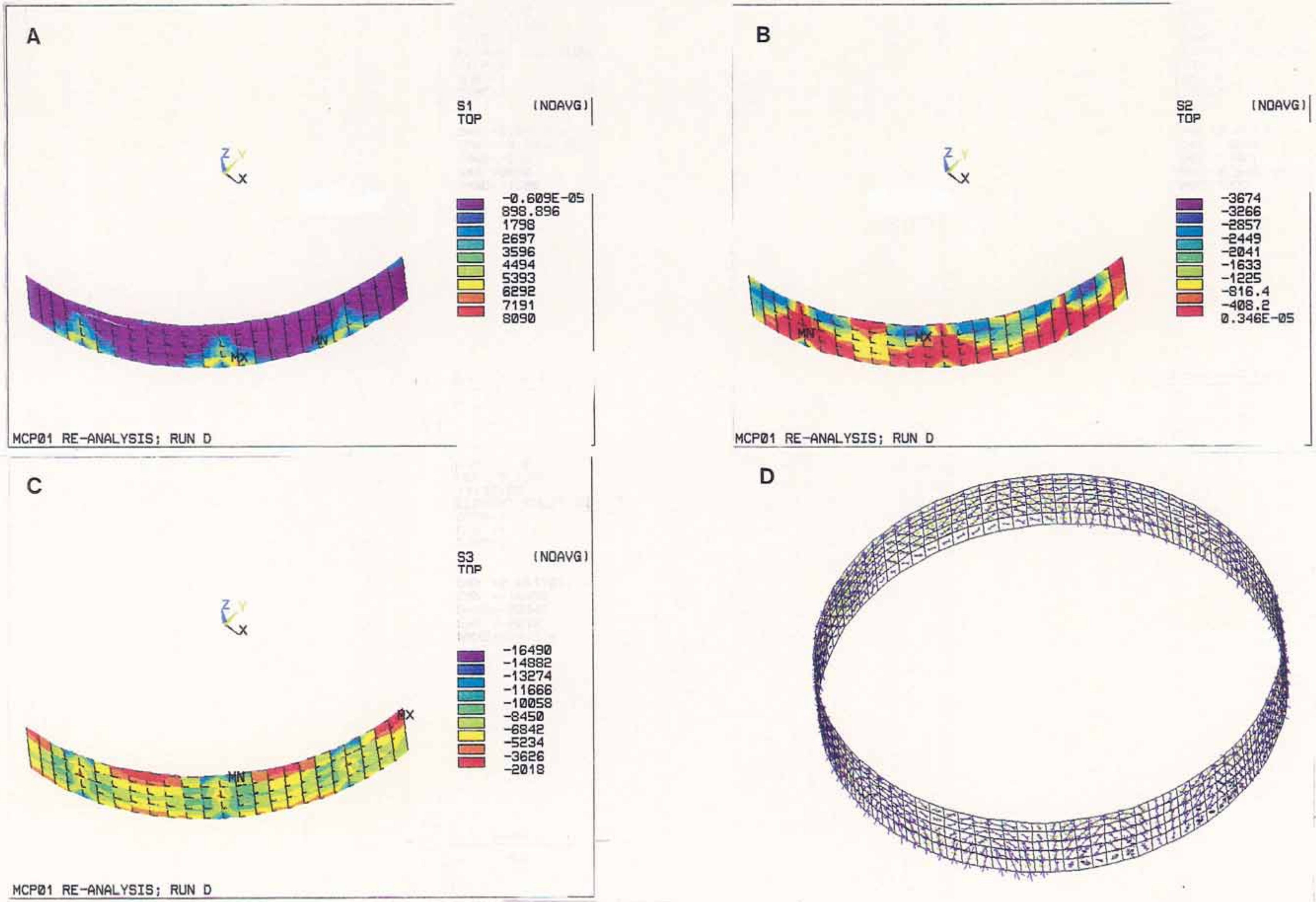
A) - C) PRINCIPAL STRESSES IN kPa
D) DESIGN GROUPS

FIGURE D3



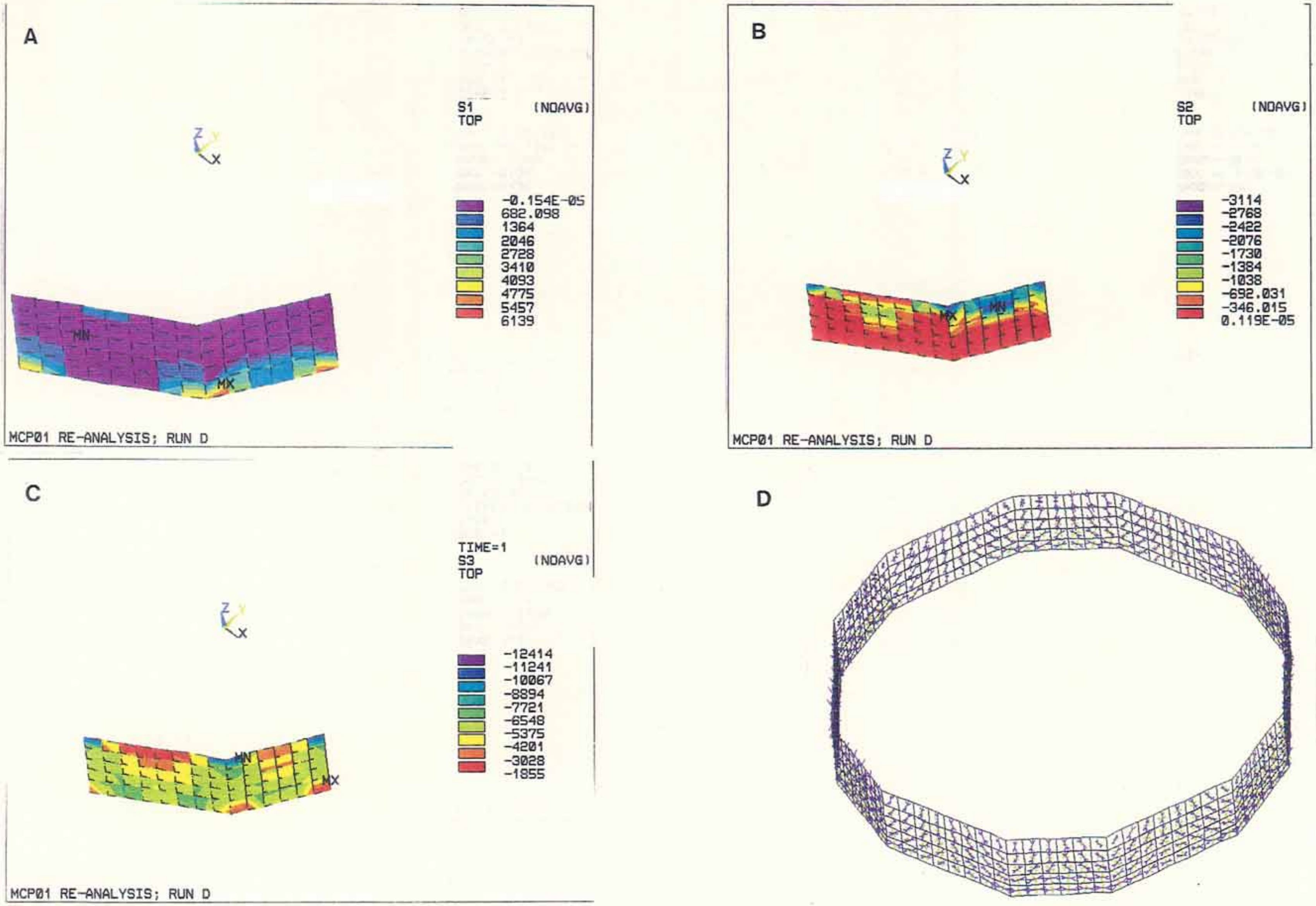
SUBSTRUCTURE WALLS TO +31.0M
A) - C) PRINCIPAL STRESSES IN kPa
D) DESIGN GROUPS

FIGURE D4



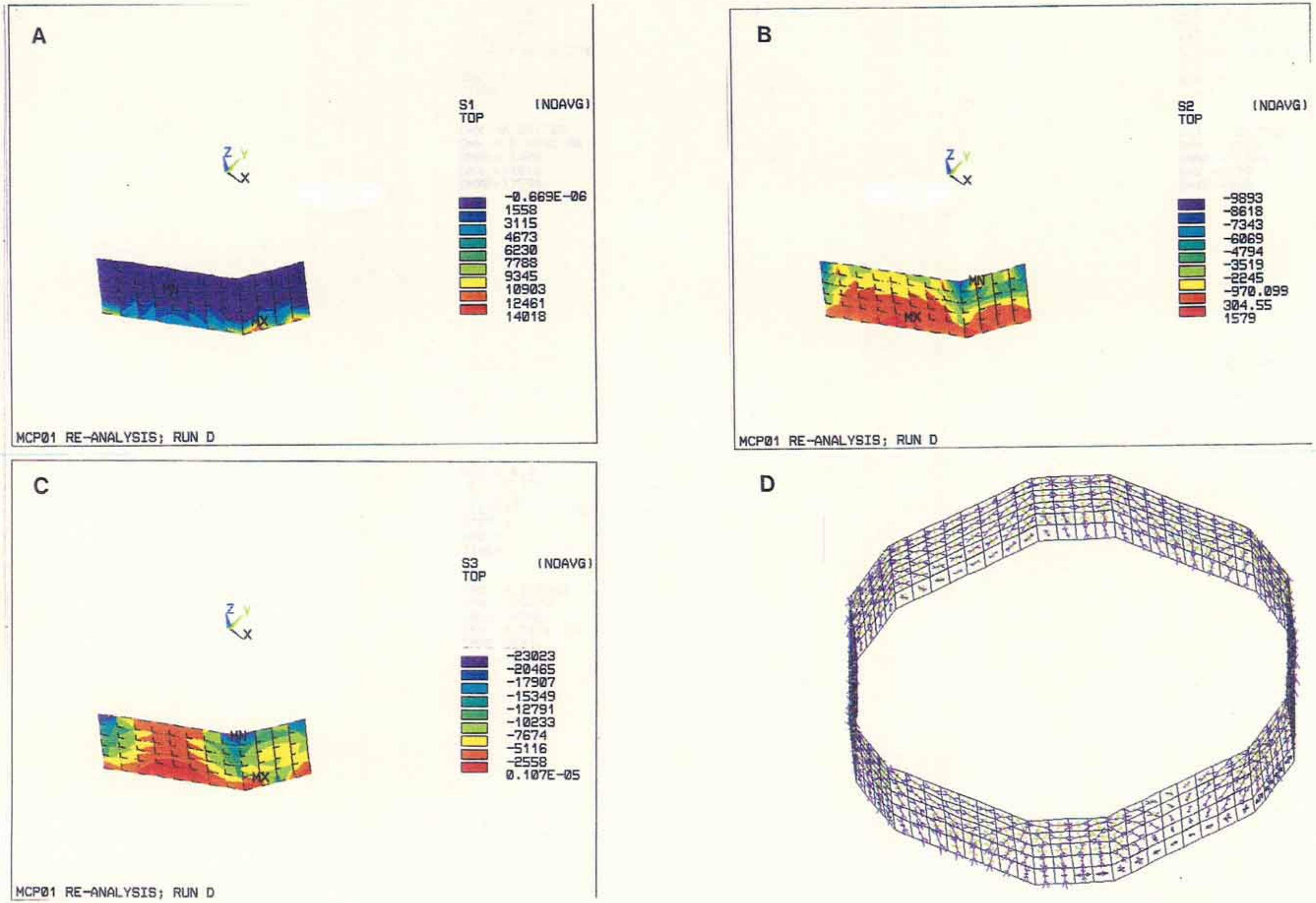
ANTISCOUR WALL
A) - C) PRINCIPAL STRESSES IN kPa
D) PRINCIPAL STRESS DIRECTIONS

FIGURE D5



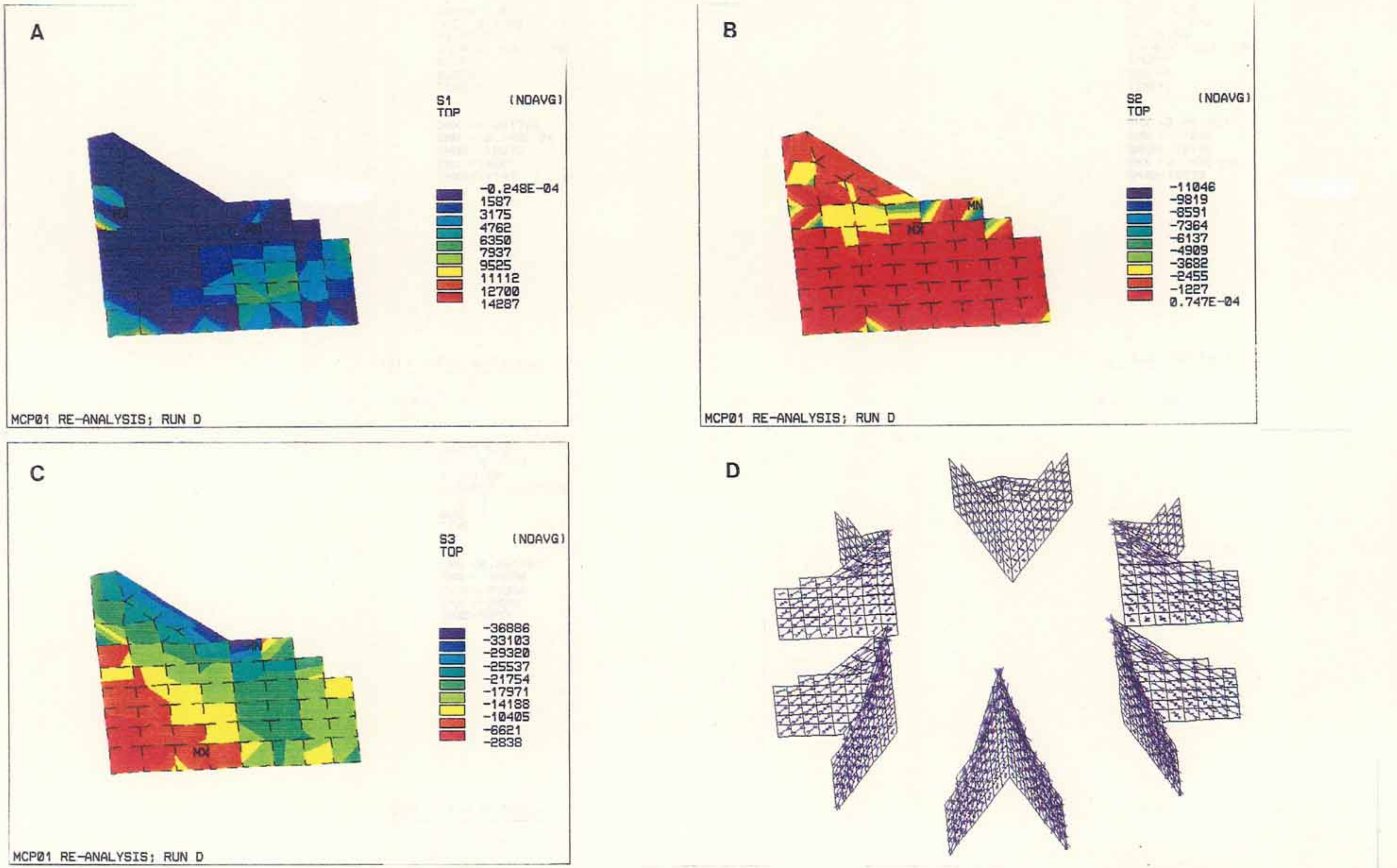
PERFORATED WALL
A) - C) PRINCIPAL STRESSES IN kPa
D) PRINCIPAL STRESS DIRECTIONS

FIGURE D6



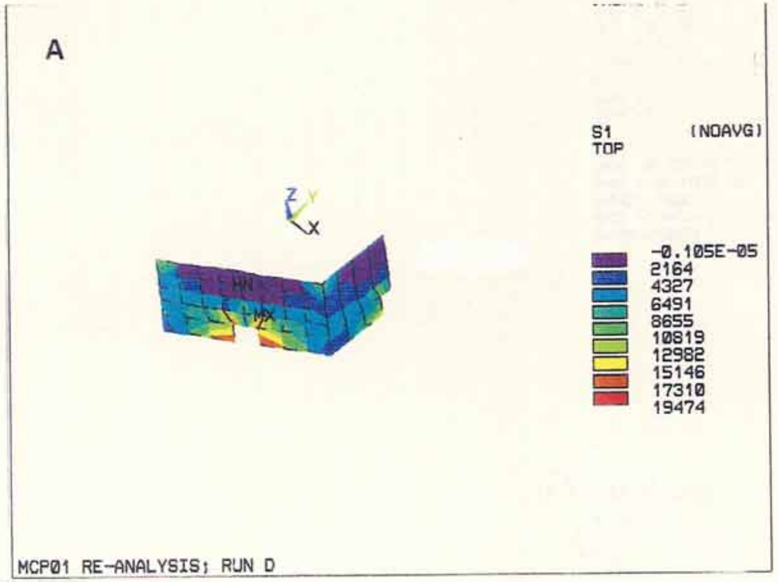
EXTERIOR WALL
A) - C) PRINCIPAL STRESSES IN kPa
D) PRINCIPAL STRESS DIRECTIONS

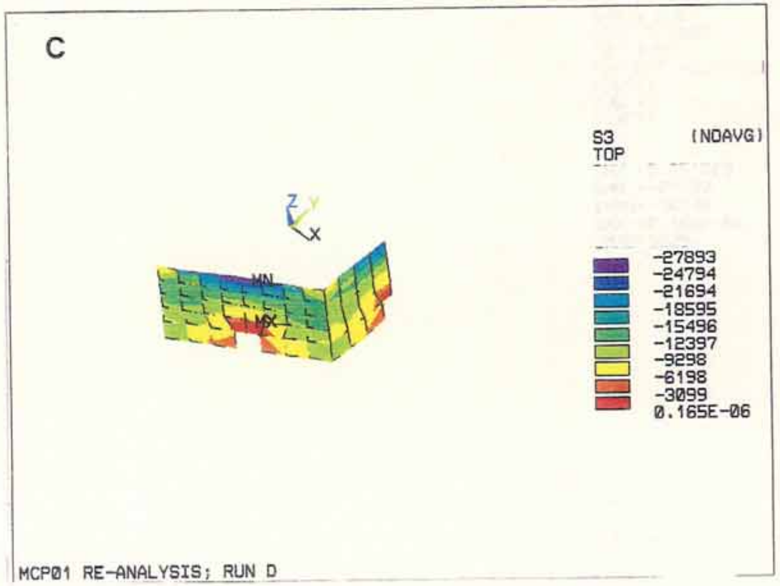
FIGURE D7

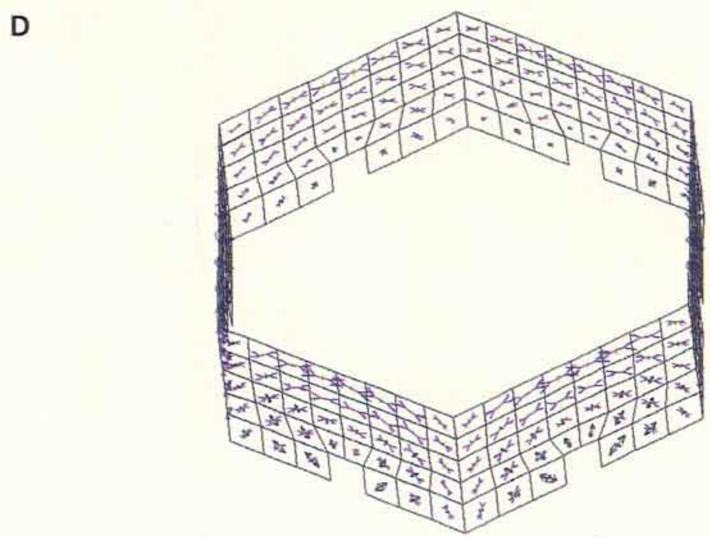


EXTERIOR DIAPHRAGM
A) - C) PRINCIPAL STRESSES IN kPa
D) PRINCIPAL STRESS DIRECTIONS

FIGURE D8

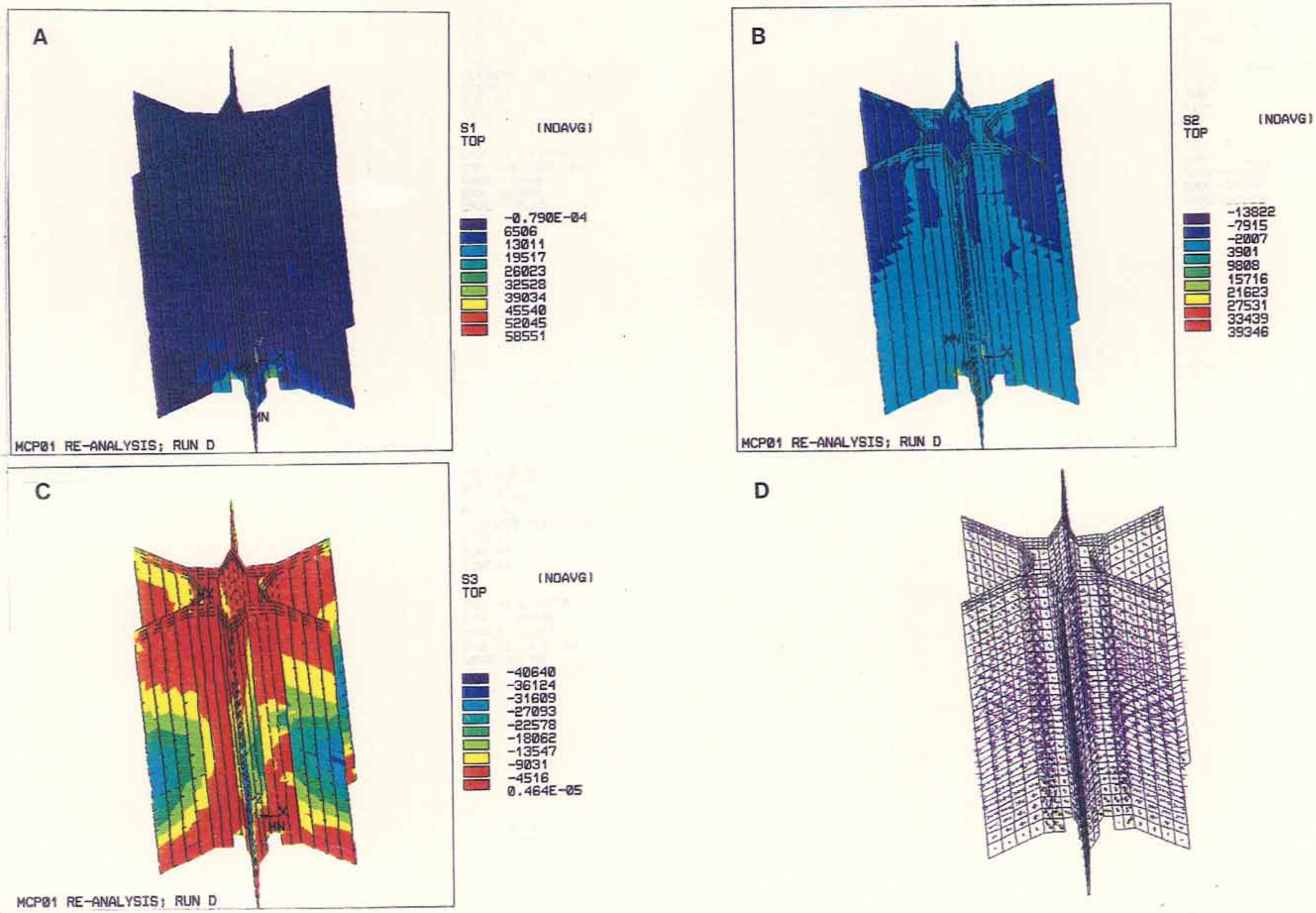






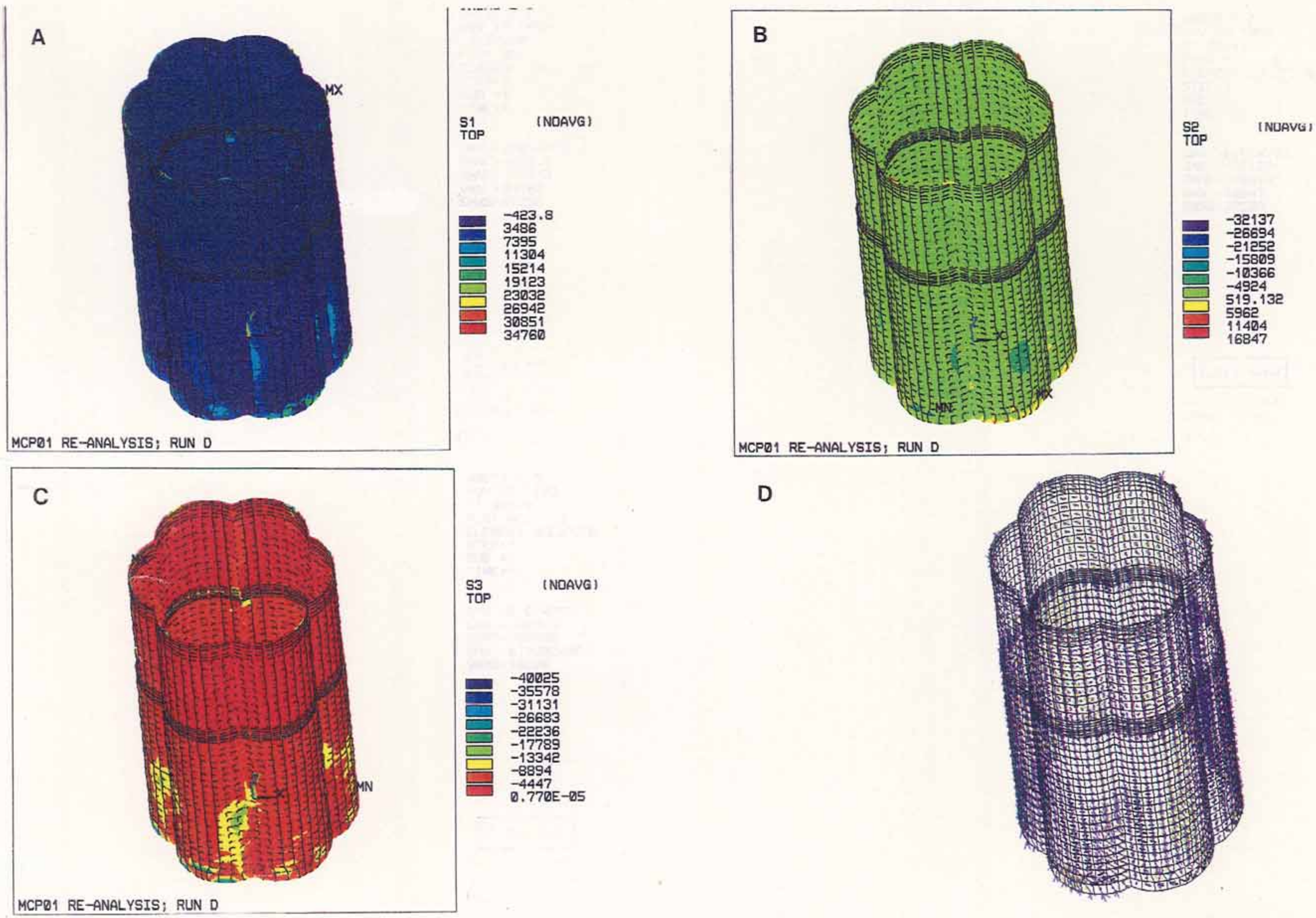
INTERIOR WALL
A) - C) PRINCIPAL STRESSES IN kPa
D) PRINCIPAL STRESS DIRECTIONS

FIGURE D9



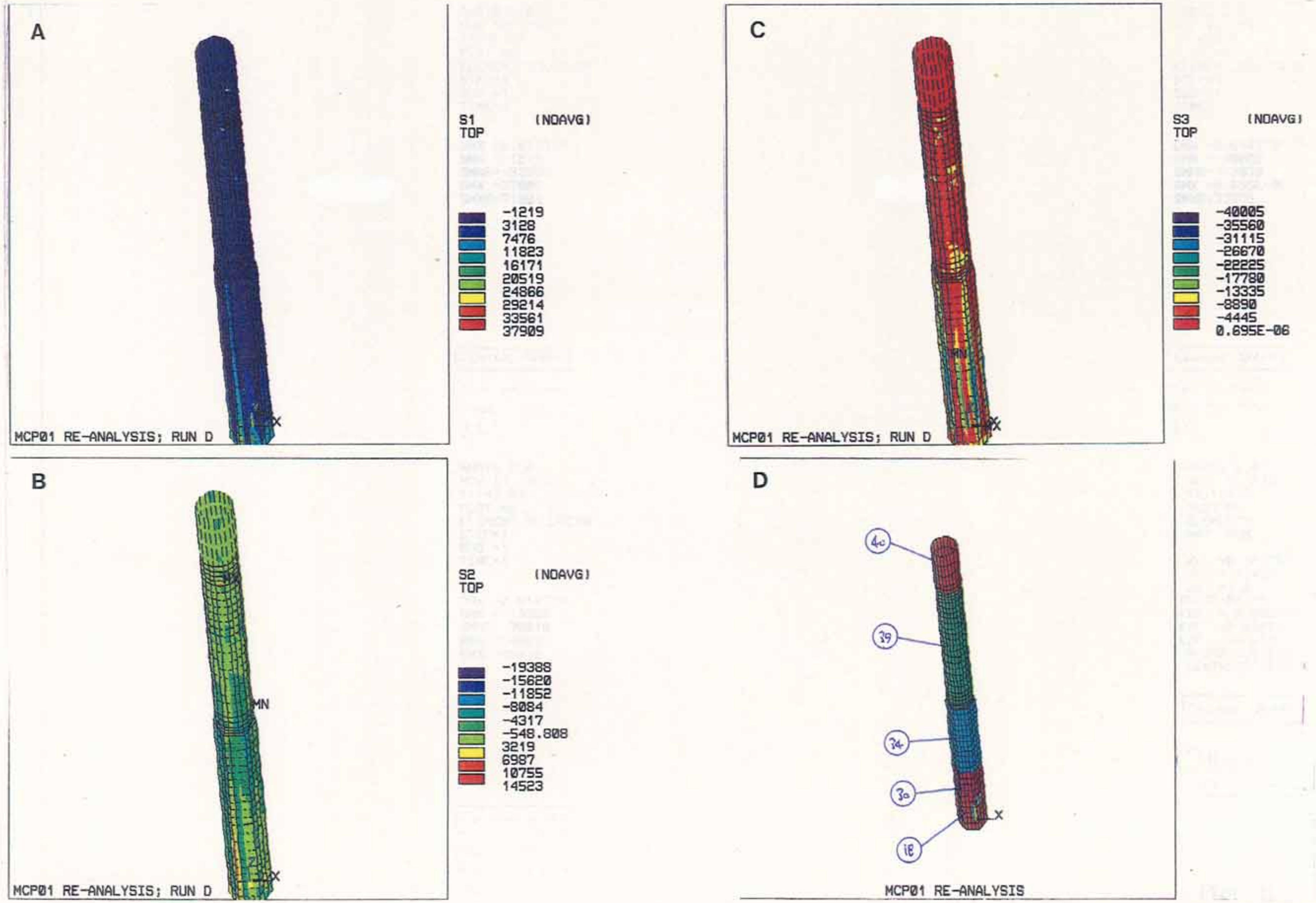
INTERIOR DIAPHRAGM
A) - C) PRINCIPAL STRESSES IN kPa
D) PRINCIPAL STRESS DIRECTIONS

FIGURE D10



LOBED/JARLAN WALLS
A) - C) PRINCIPAL STRESSES IN kPa
D) PRINCIPAL STRESS DIRECTIONS

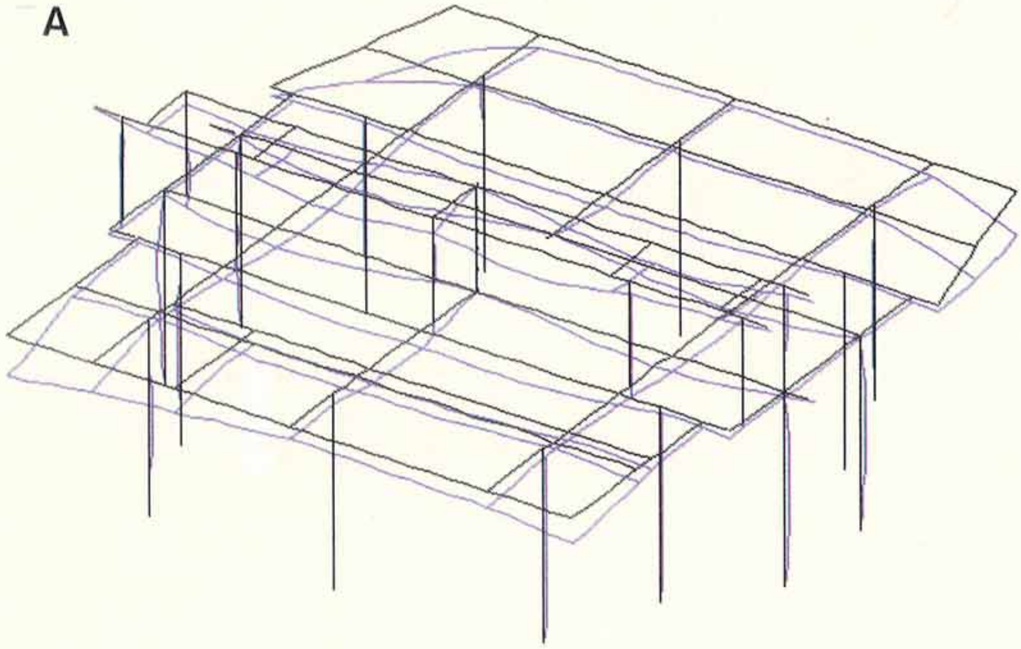
FIGURE D11



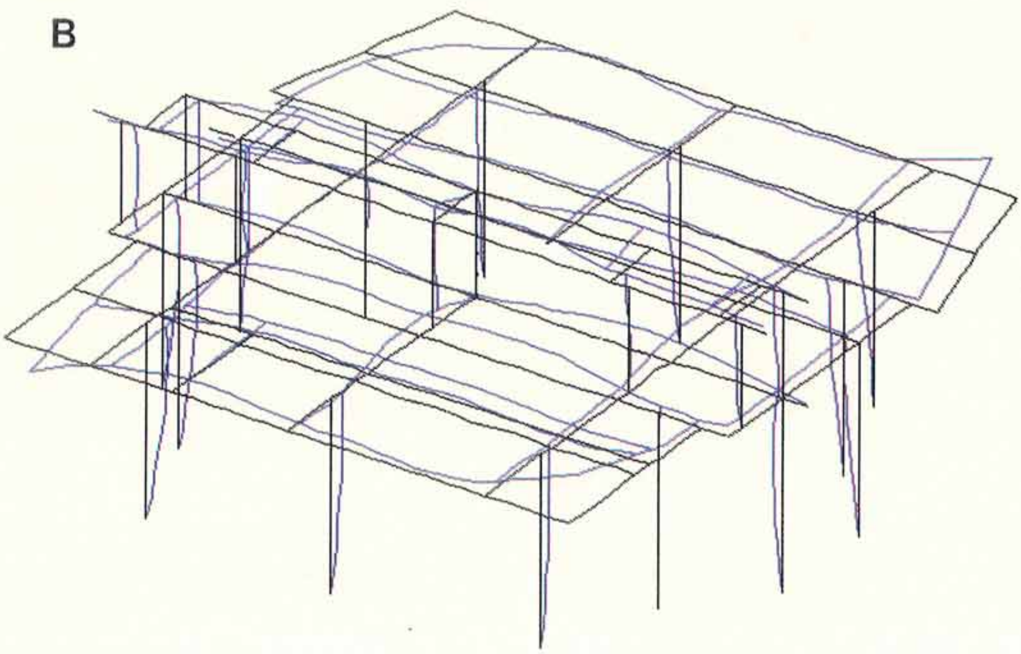
CENTRAL SHAFT
A) - C) PRINCIPAL STRESSES IN kPa
D) DESIGN GROUPS

FIGURE D12

A



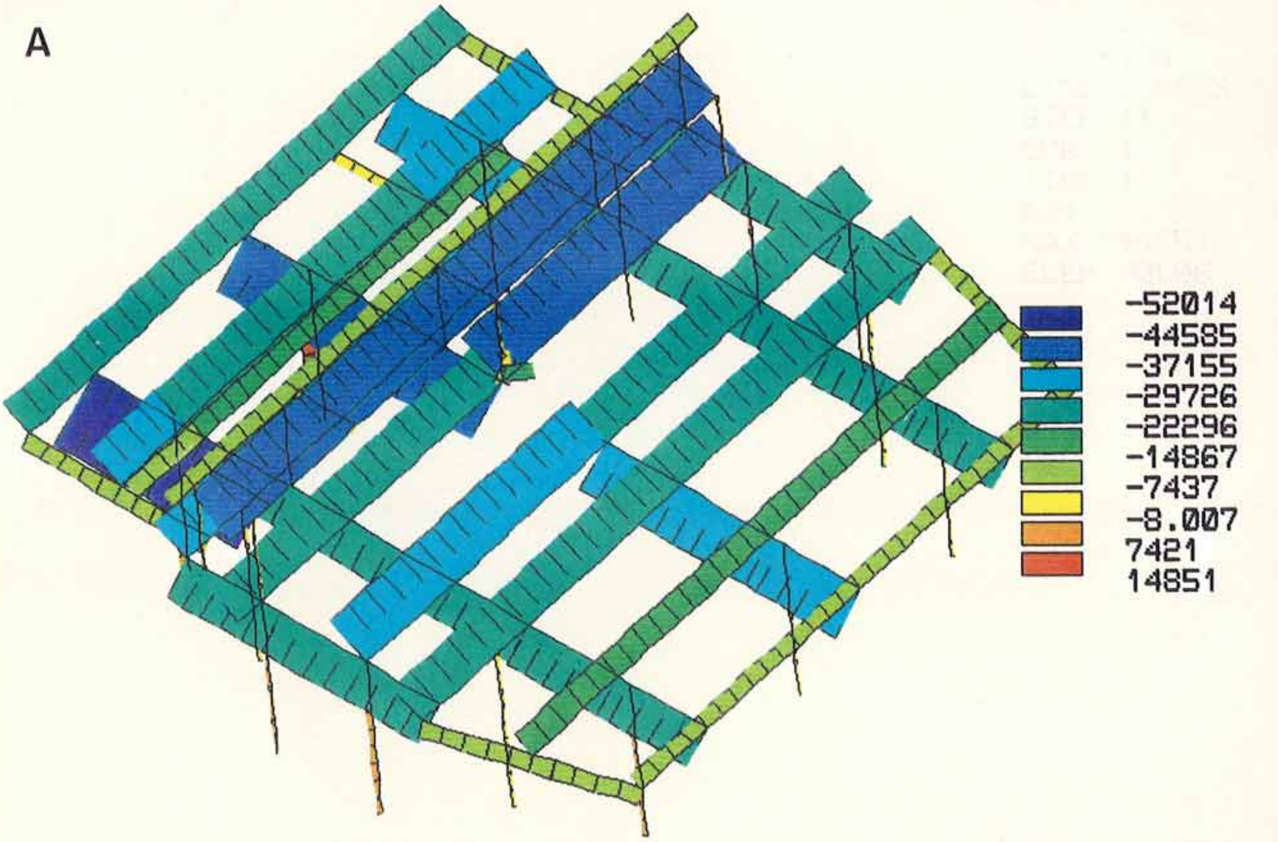
B



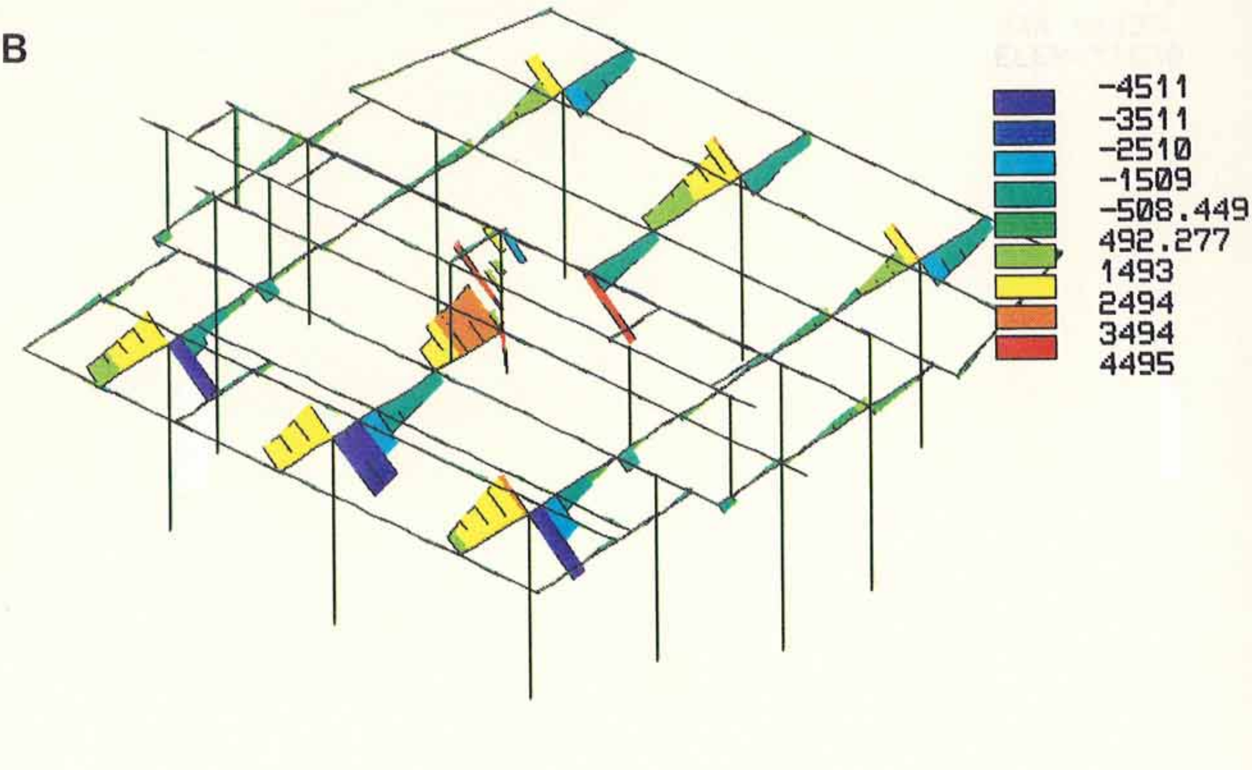
DECK ONLY ANALYSIS
A) DISPLACED SHAPE UNDER DEAD LOADS
B) DISPLACED SHAPE UNDER PRESTRESS

FIGURE D13

A

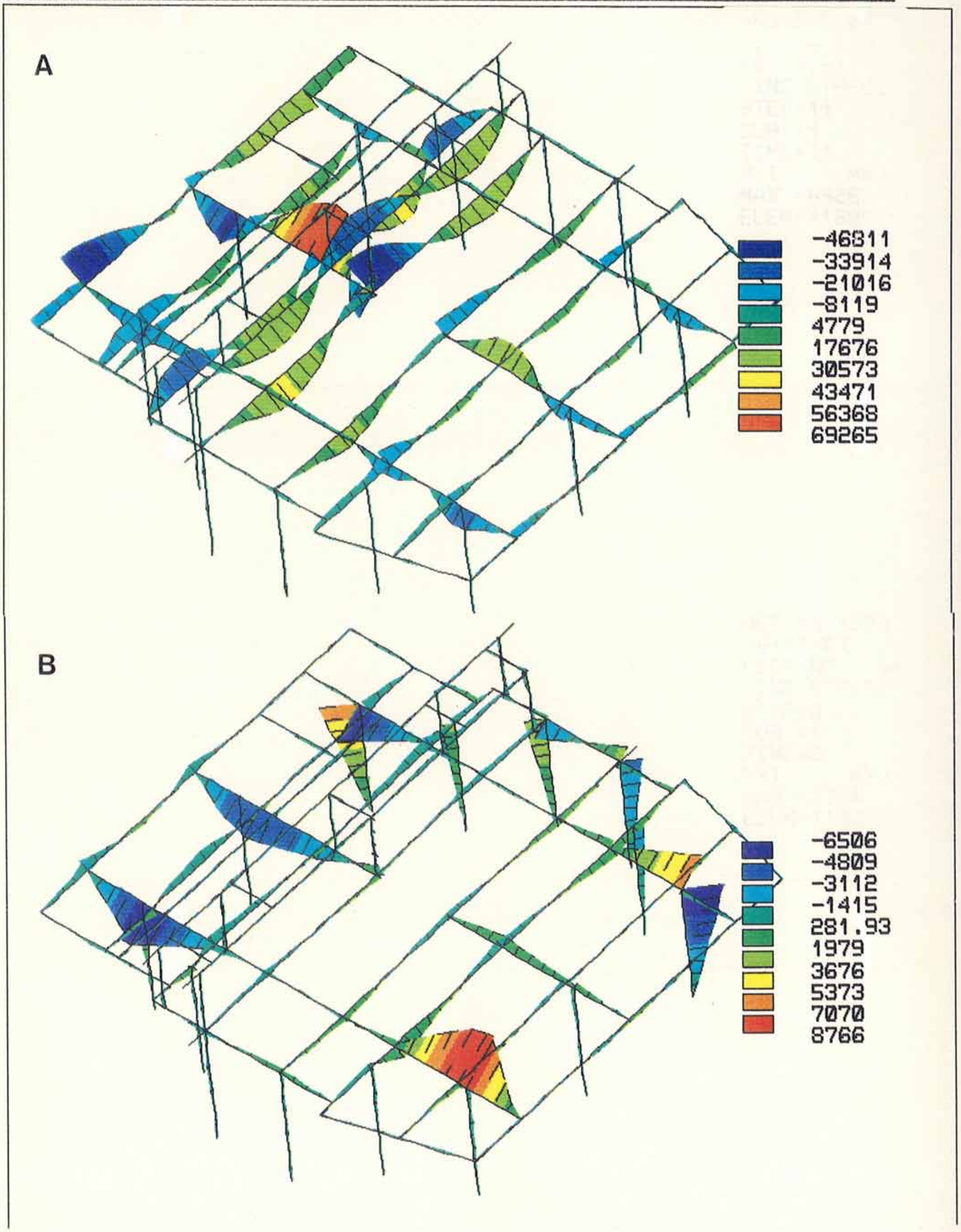


B



DECK ONLY ANALYSIS;
PRESTRESS LOADING
A) MEMBER AXIAL FORCE IN kN
B) VERTICAL SHEAR FORCE IN kN

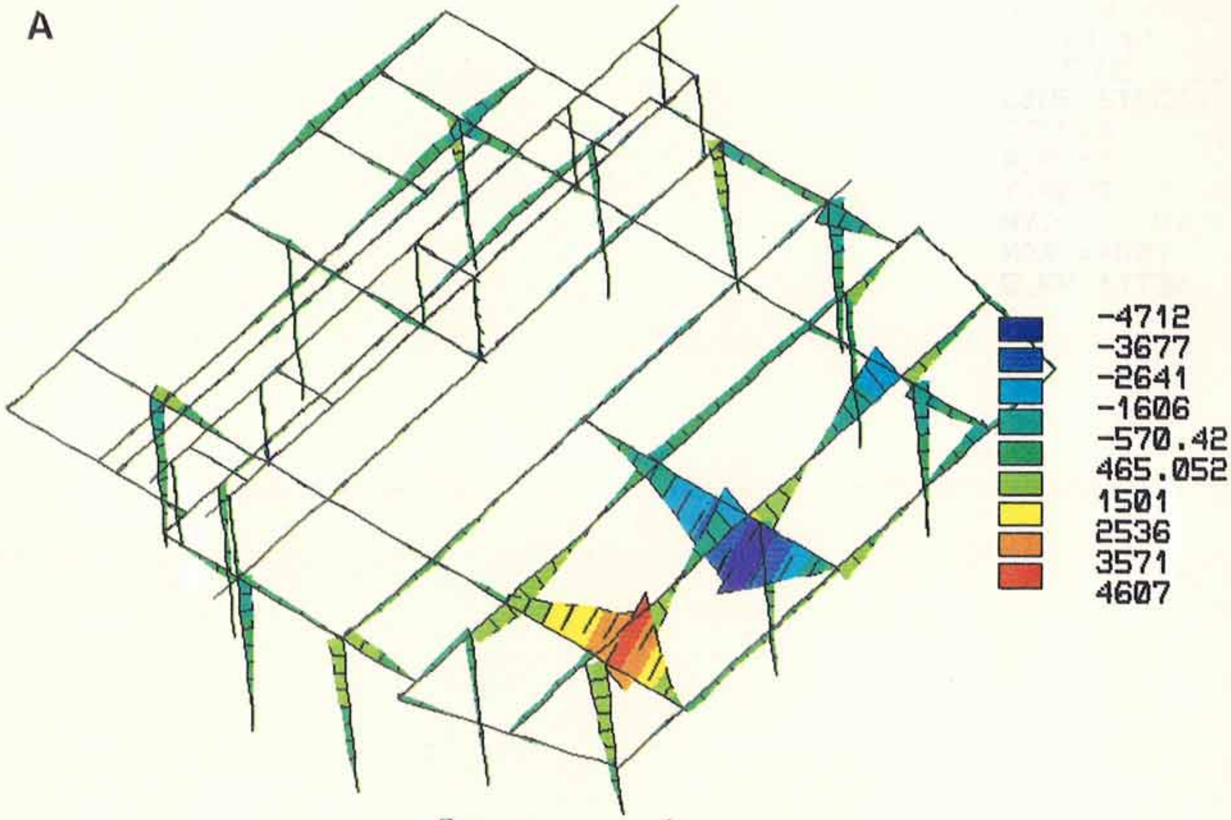
FIGURE D14



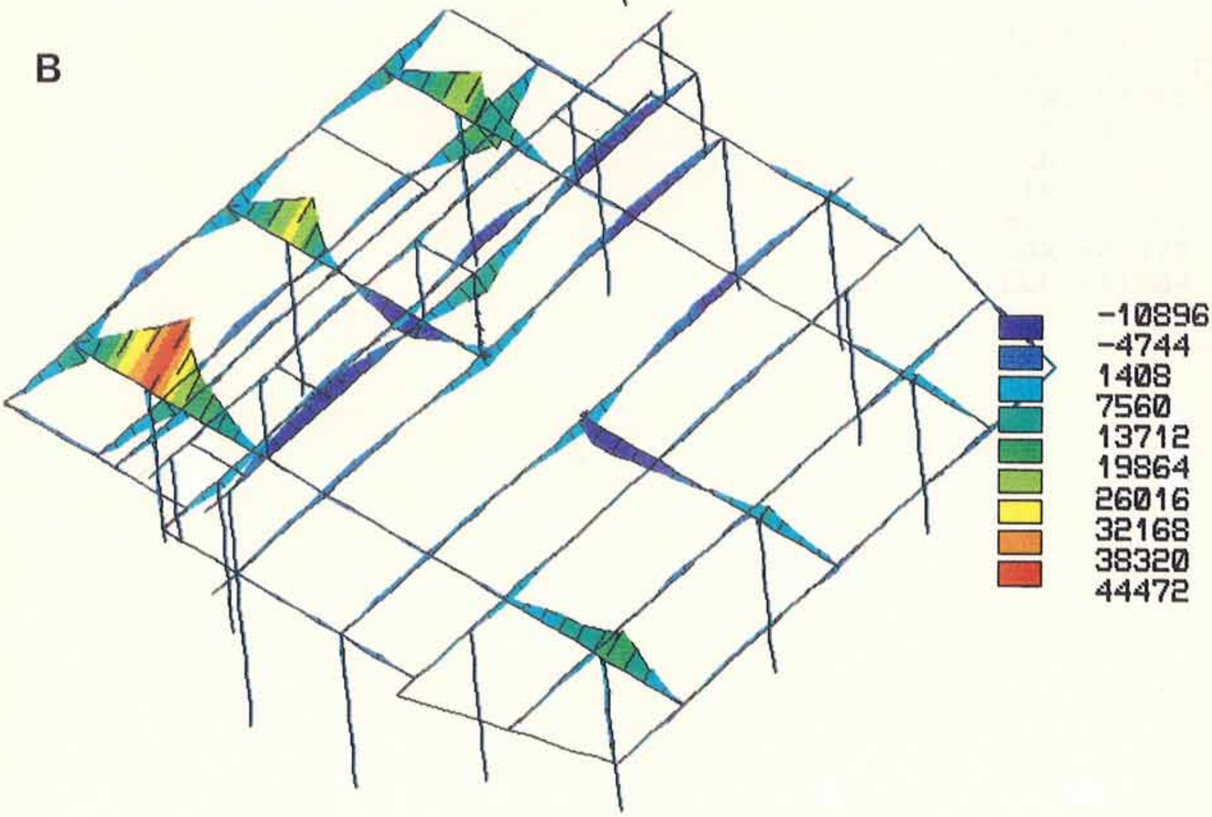
DECK ONLY ANALYSIS:
OUT-OF-PLANE BENDING MOMENT IN kNm
A) PRESTRESS LOADING
B) WIND LOADING FROM SOUTH

FIGURE D15

A

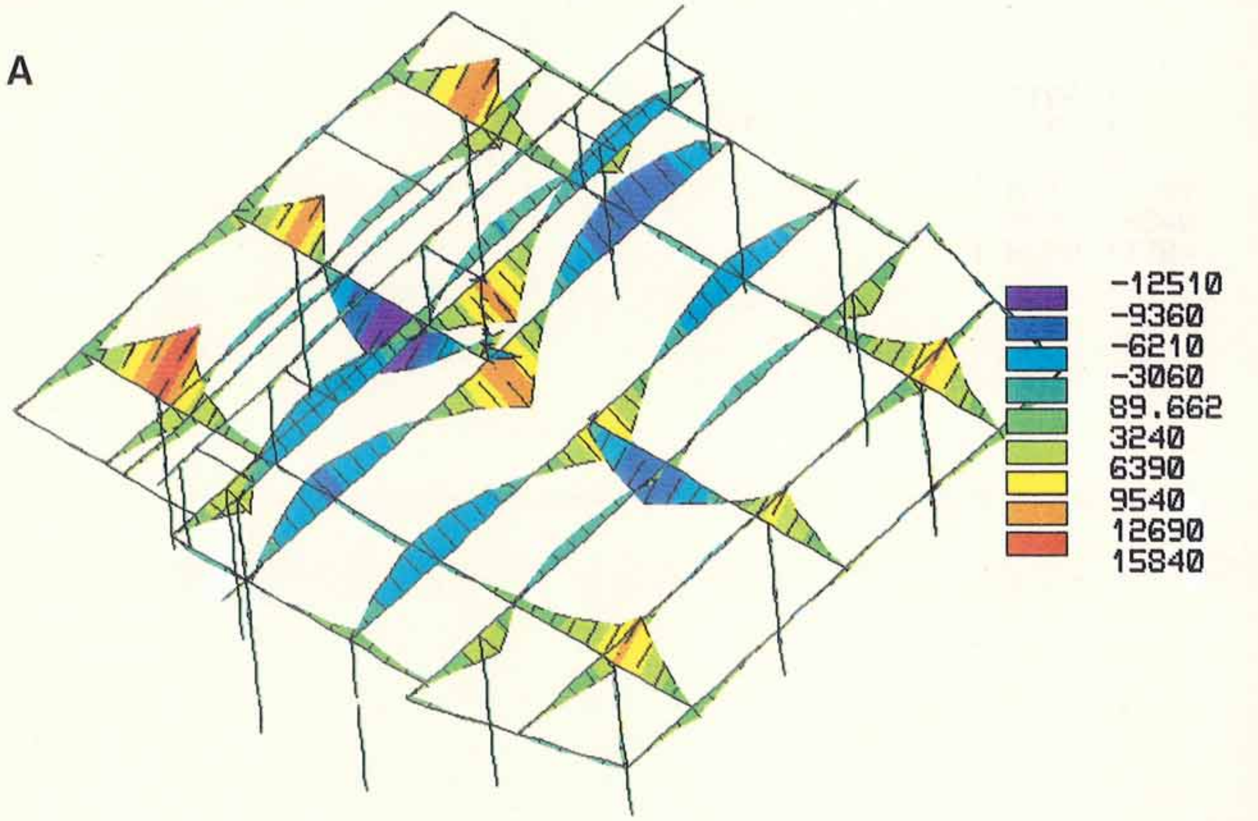


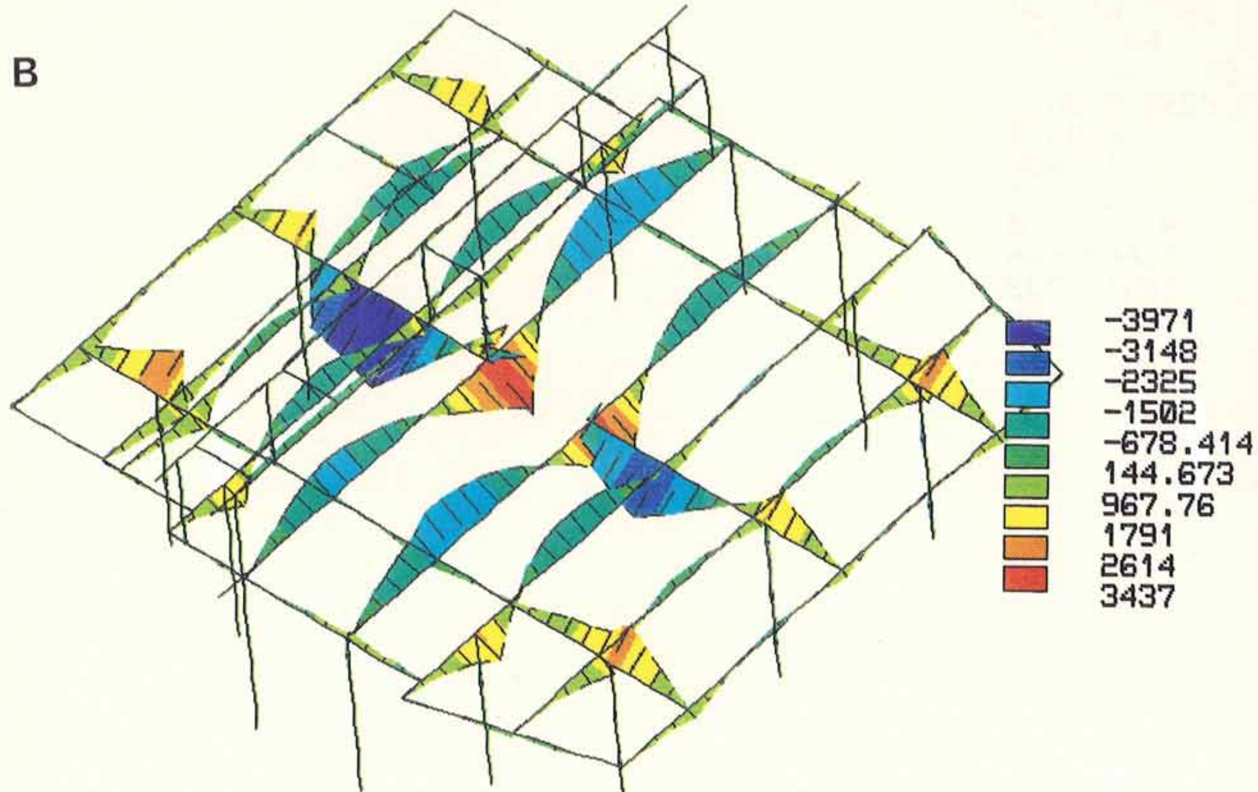
B



DECK ONLY ANALYSIS:
OUT-OF-PLANE BENDING MOMENT IN kNm
A) WIND LOADING FROM WEST
B) EQUIPMENT LOADING

FIGURE D16

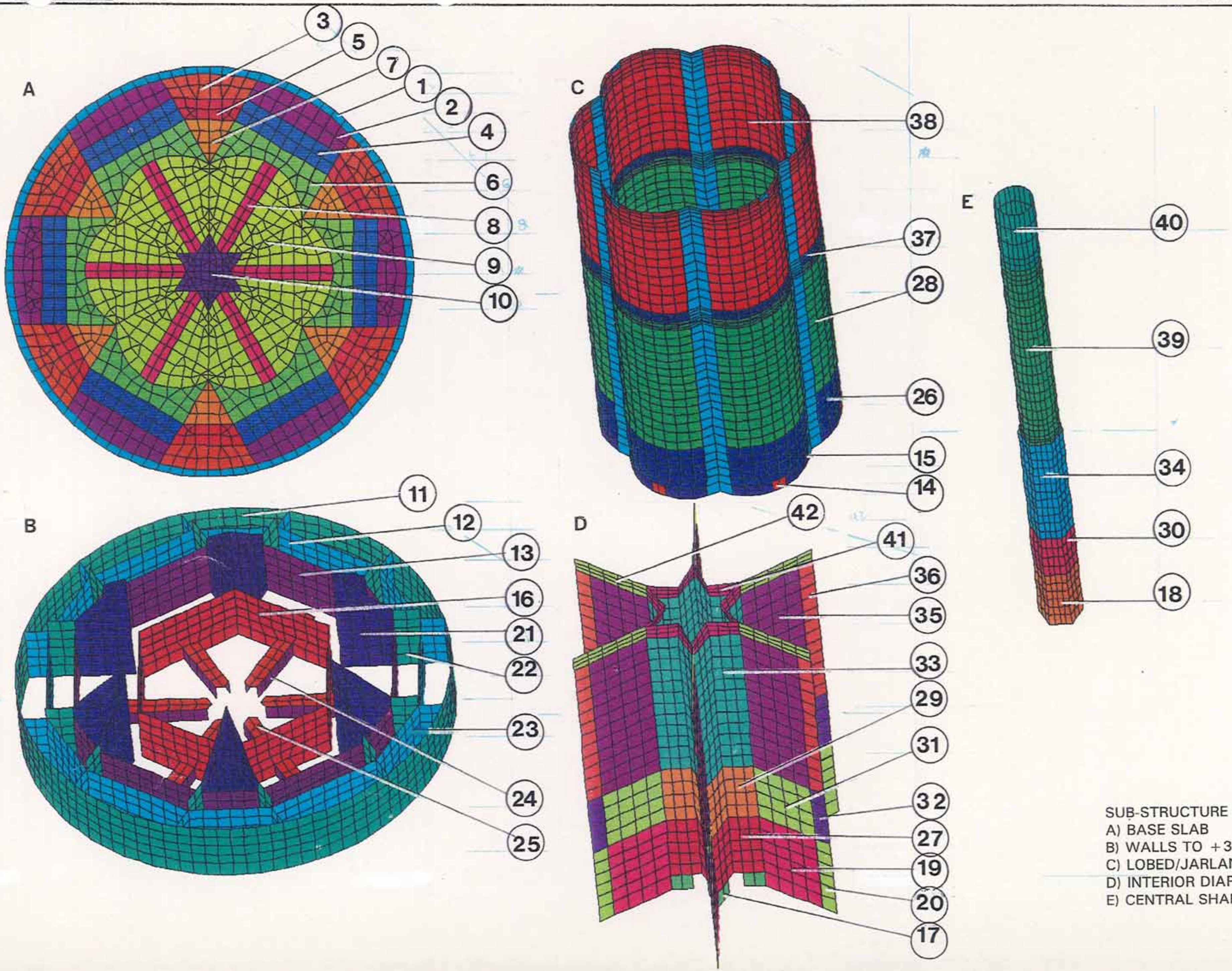




DECK ONLY ANALYSIS:
OUT-OF-PLANE BENDING MOMENT IN kNm
A) DEAD LOAD
B) WIND LOADING FROM SOUTH

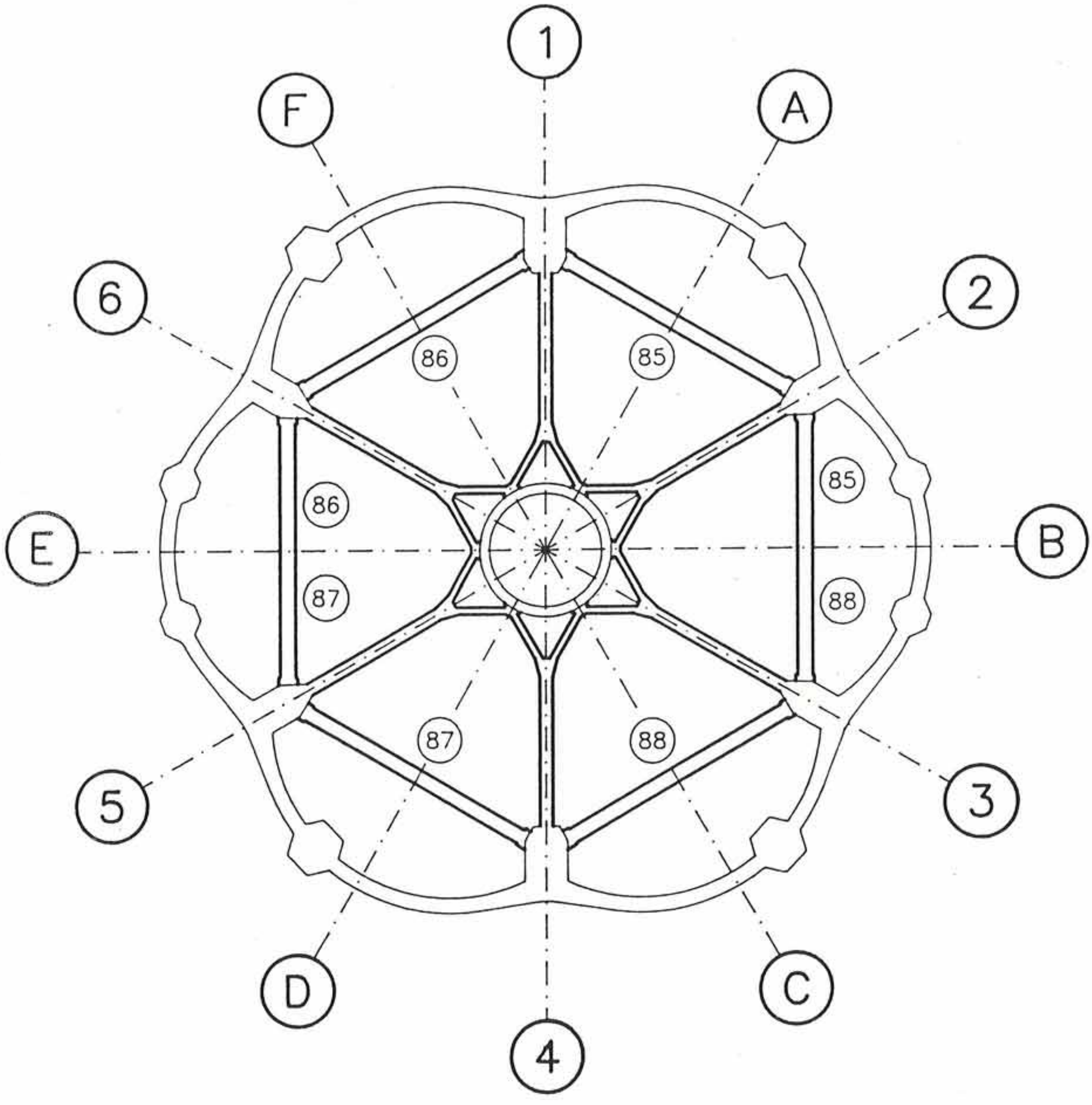
FIGURE D17

APPENDIX E - DESIGN GROUPS & CODE CHECK RESULTS



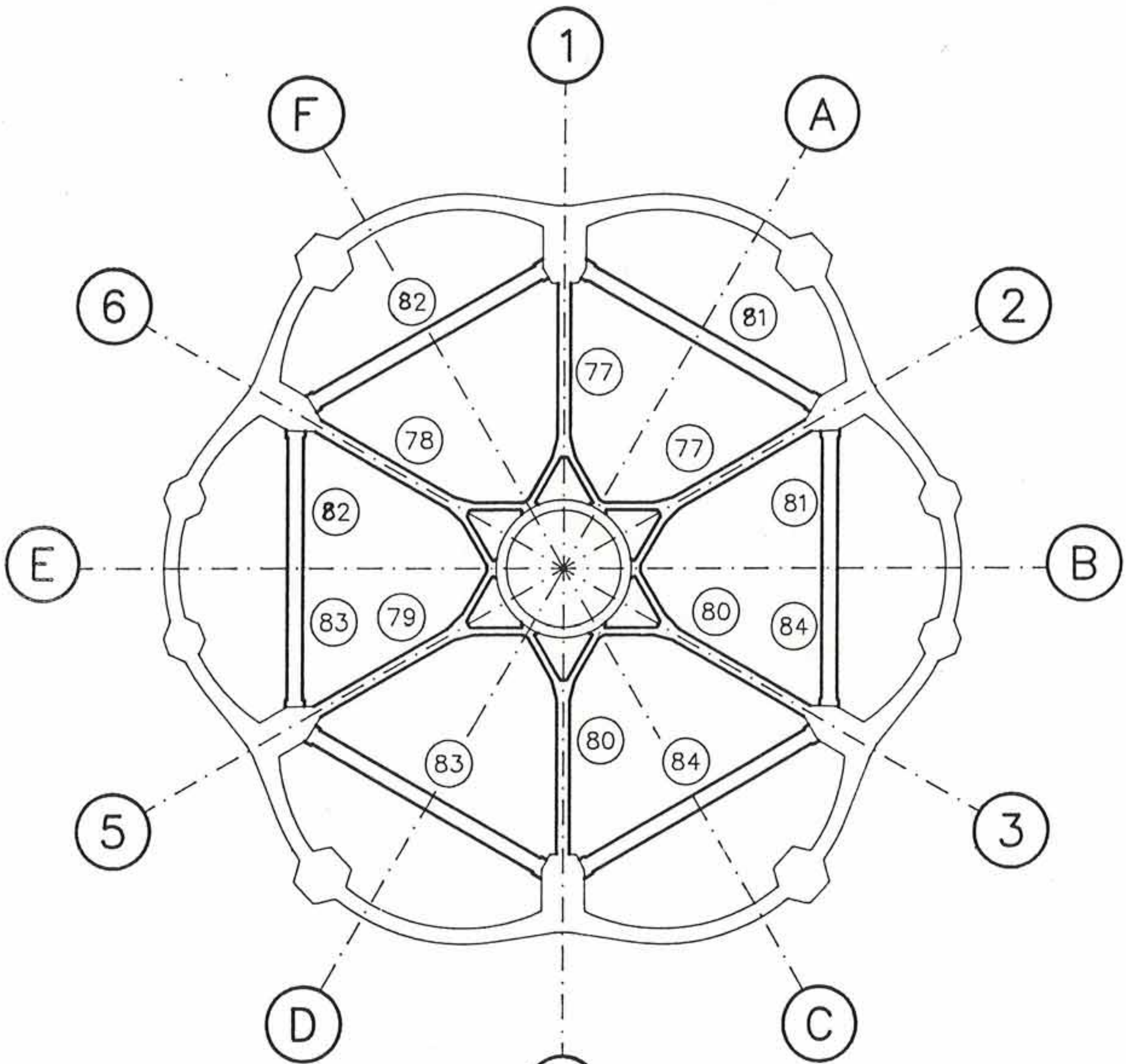
SUB-STRUCTURE DESIGN GROUPS:
A) BASE SLAB
B) WALLS TO +31.0M
C) LOBED/JARLAN WALL
D) INTERIOR DIAPHRAGM
E) CENTRAL SHAFT

FIGURE E1



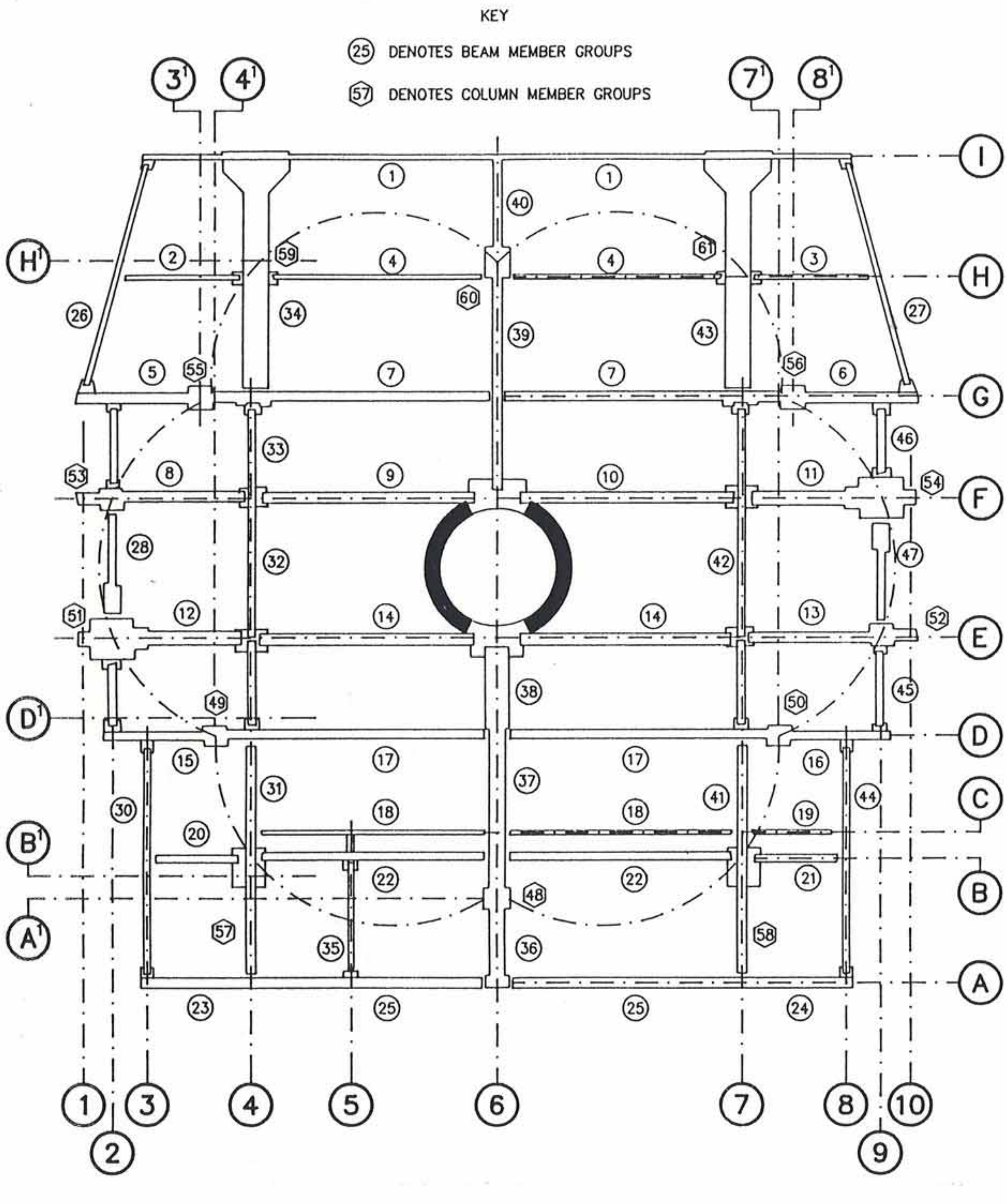
BEAM DESIGN GROUPS:
A) STRUTS AT +65.0M

FIGURE E2



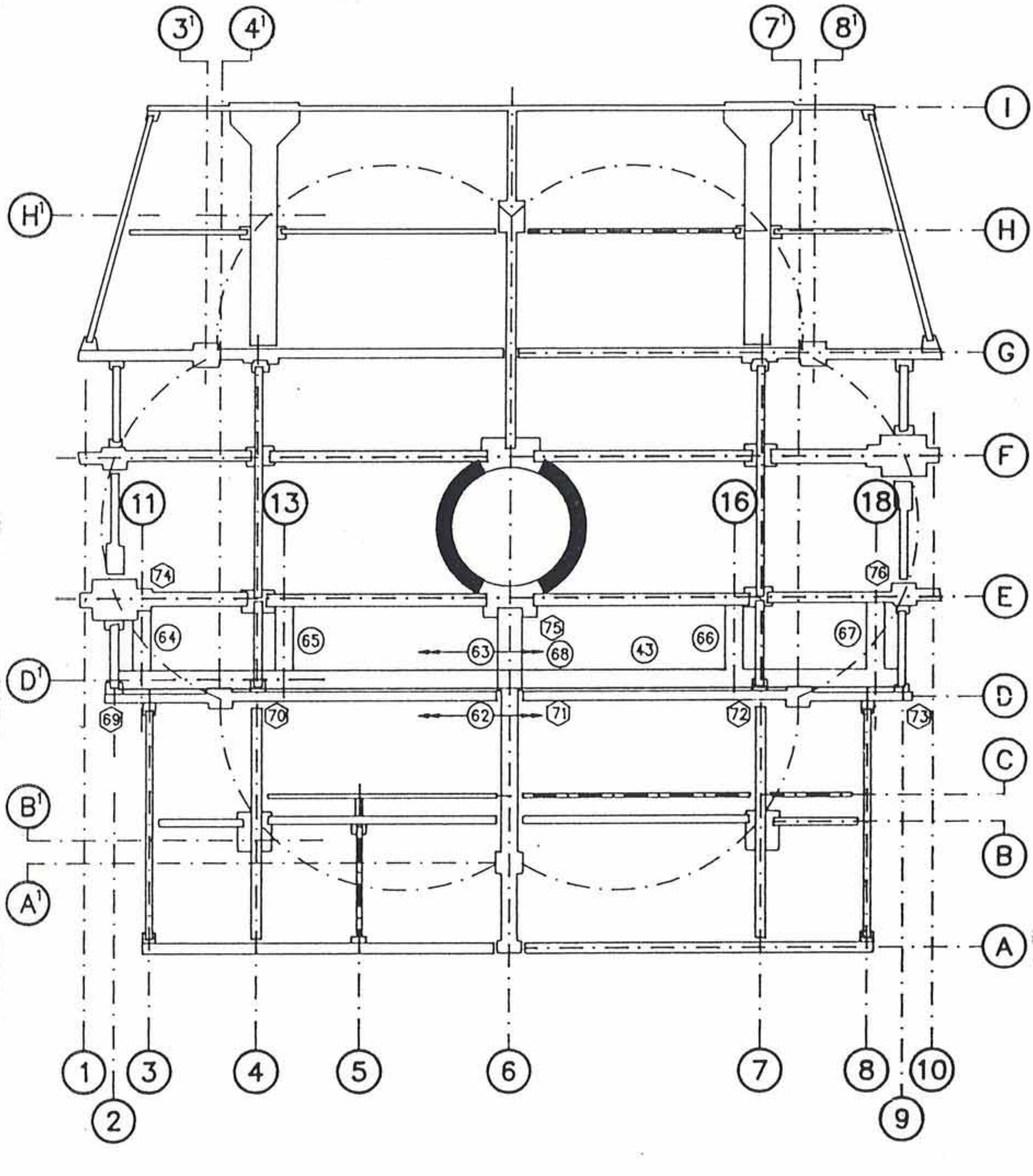
BEAM DESIGN GROUPS:
B) STRUTS/BEAMS AT +105.0M

FIGURE E2



BEAM DESIGN GROUPS:
C) MAIN DECK BEAMS/COLUMNS

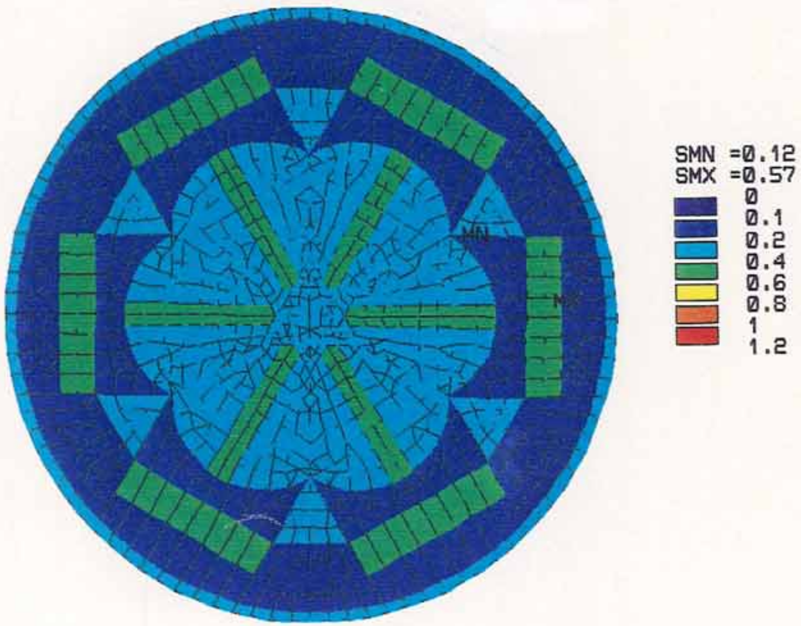
FIGURE E2



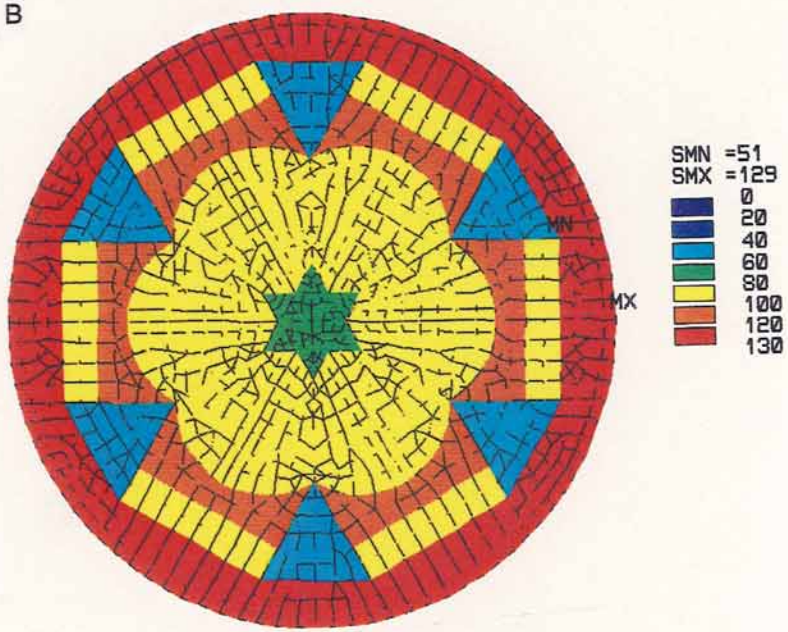
BEAM DESIGN GORUPS:
D) MANIFOLD DECK BEAMS

FIGURE E2

A

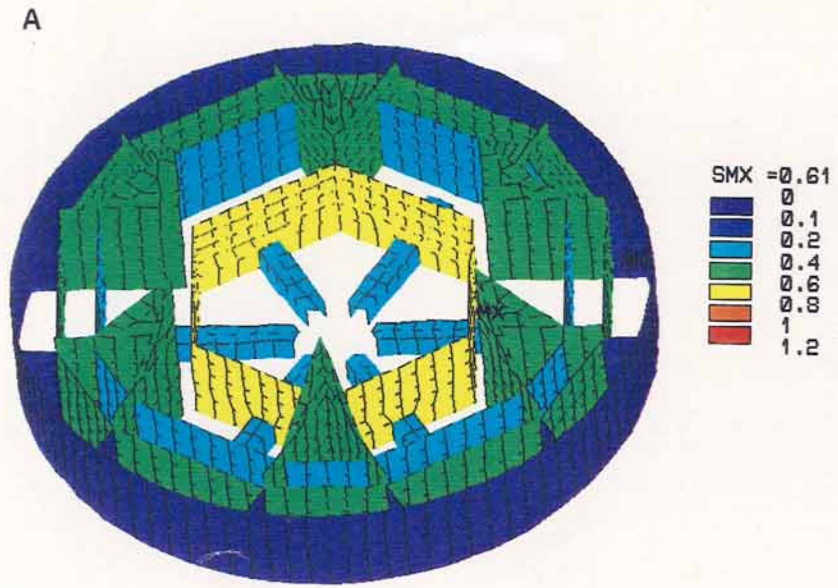


B



BASE SLAB:
A) CRACKWIDTH IN mm
B) CONCRETE UTILISATION
PERCENTAGE OF SLS STRESS (24 MPa)

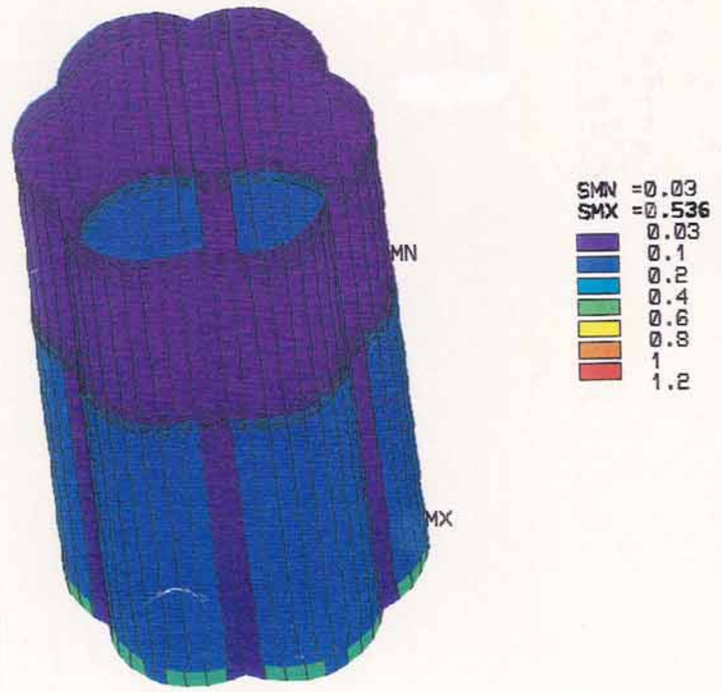
FIGURE E3



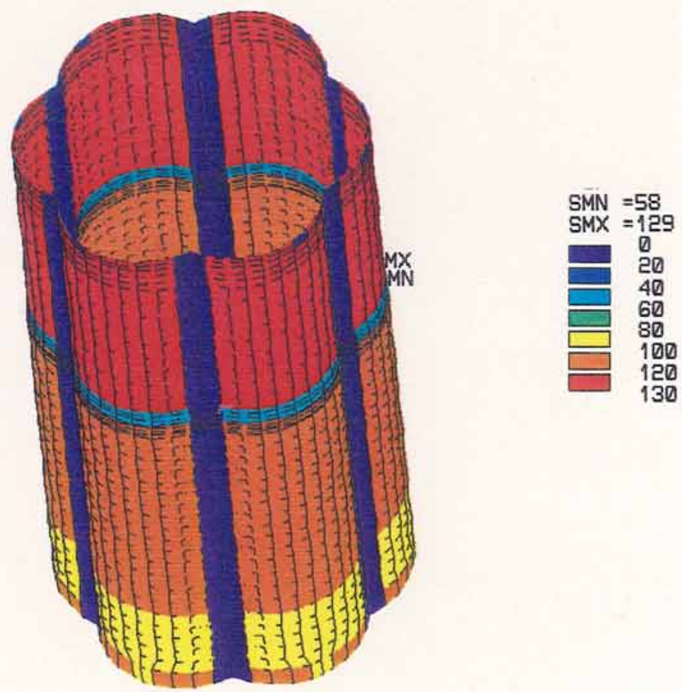
SUBSTRUCTURE WALLS TO +31.0M:
A) CRACKWIDTH IN mm
B) CONCRETE UTILISATION
PERCENTAGE OF SLS STRESS (24 MPa)

FIGURE E4

A



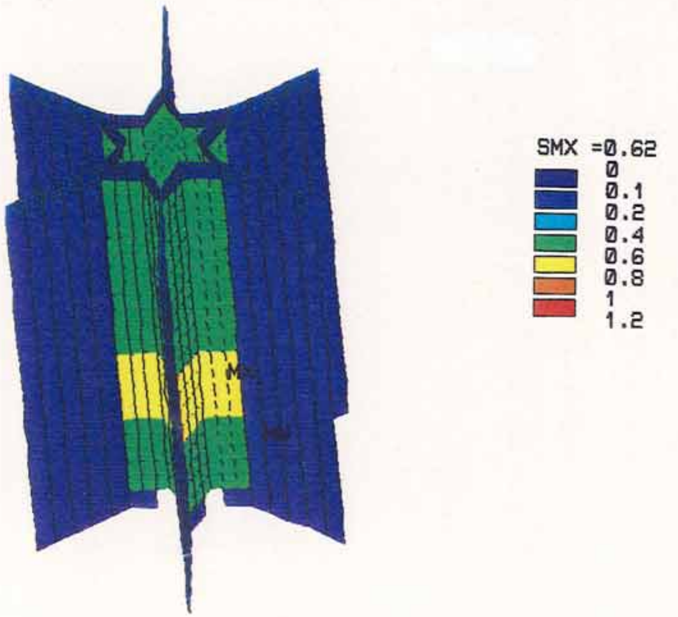
B



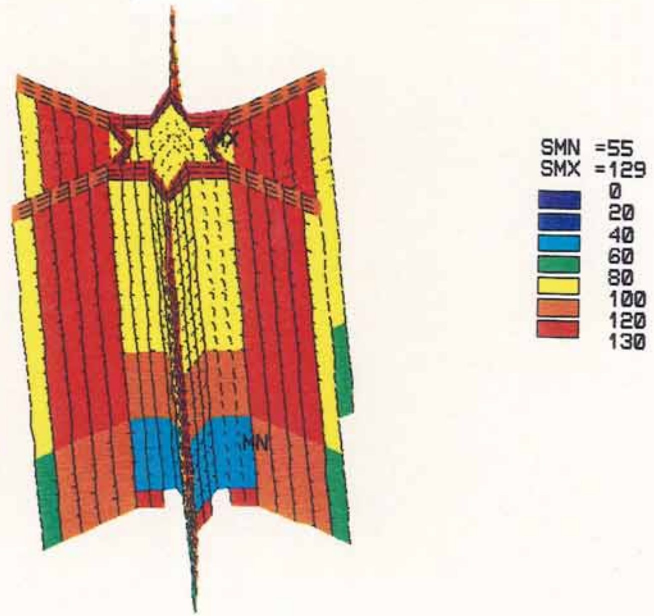
LOBED/JARLAN WALL:
A) CRACKWIDTH IN mm
B) CONCRETE UTILISATION
PERCENTAGE OF SLS STRESS (24 MPa)

FIGURE E5

A



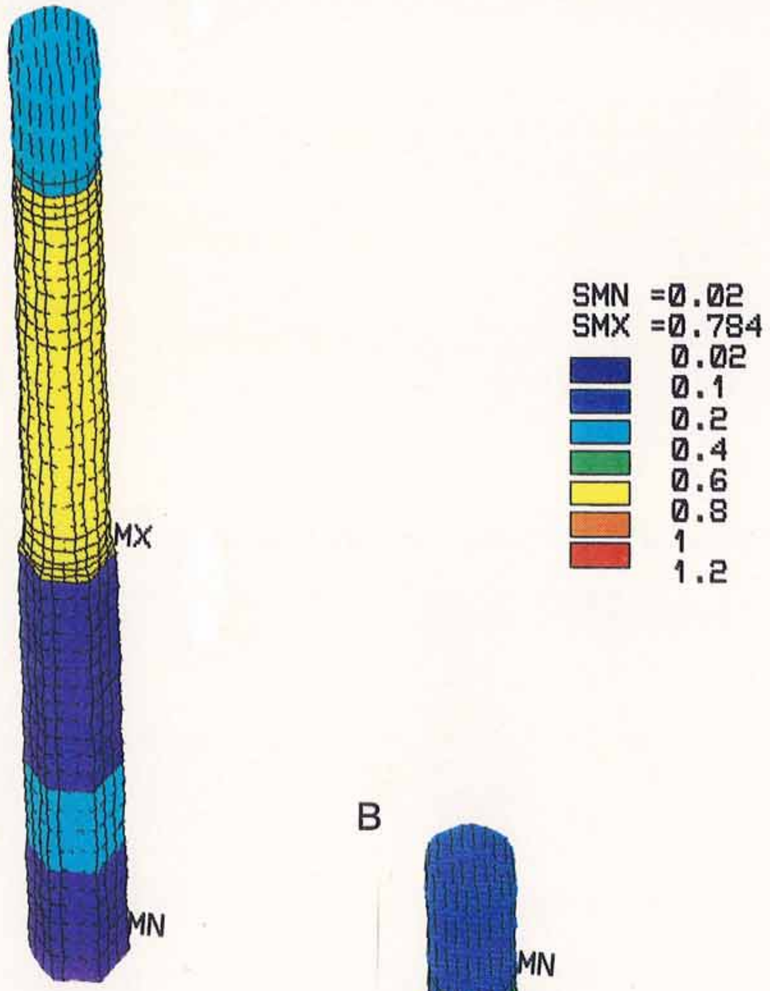
B



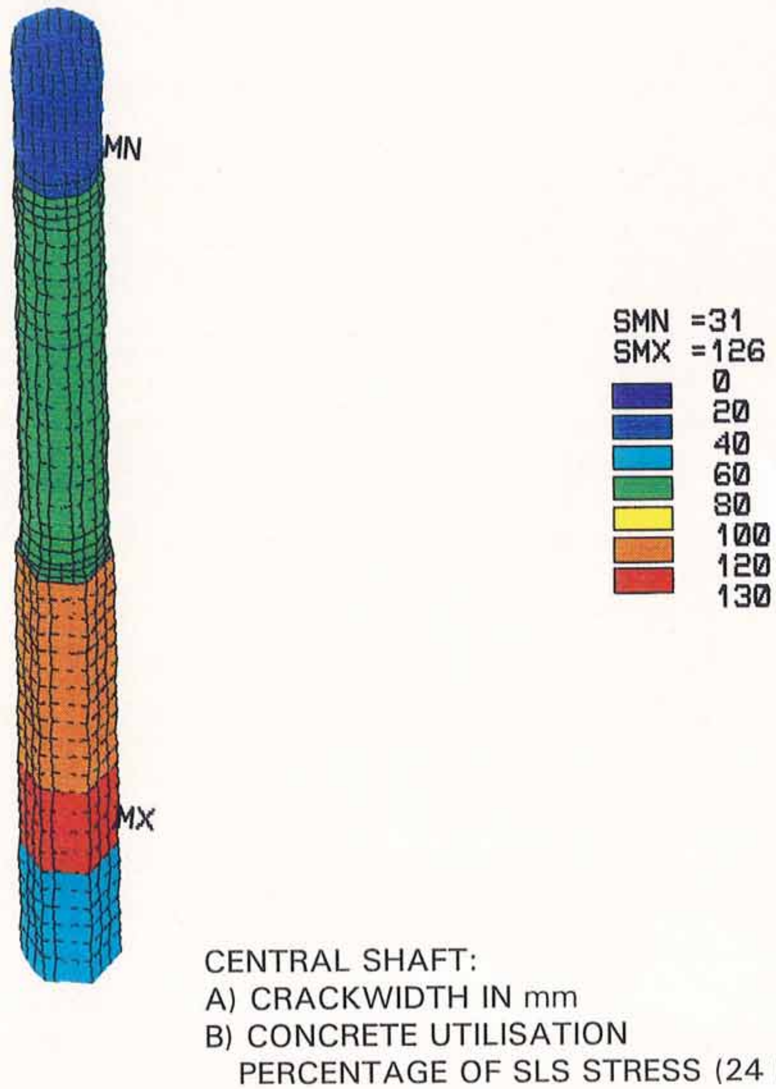
INTERIOR DIAPHRAGM:
A) CRACKWIDTH IN mm
B) CONCRETE UTILISATION
PERCENTAGE OF SLS STRESS (24 MPa)

FIGURE E6

A

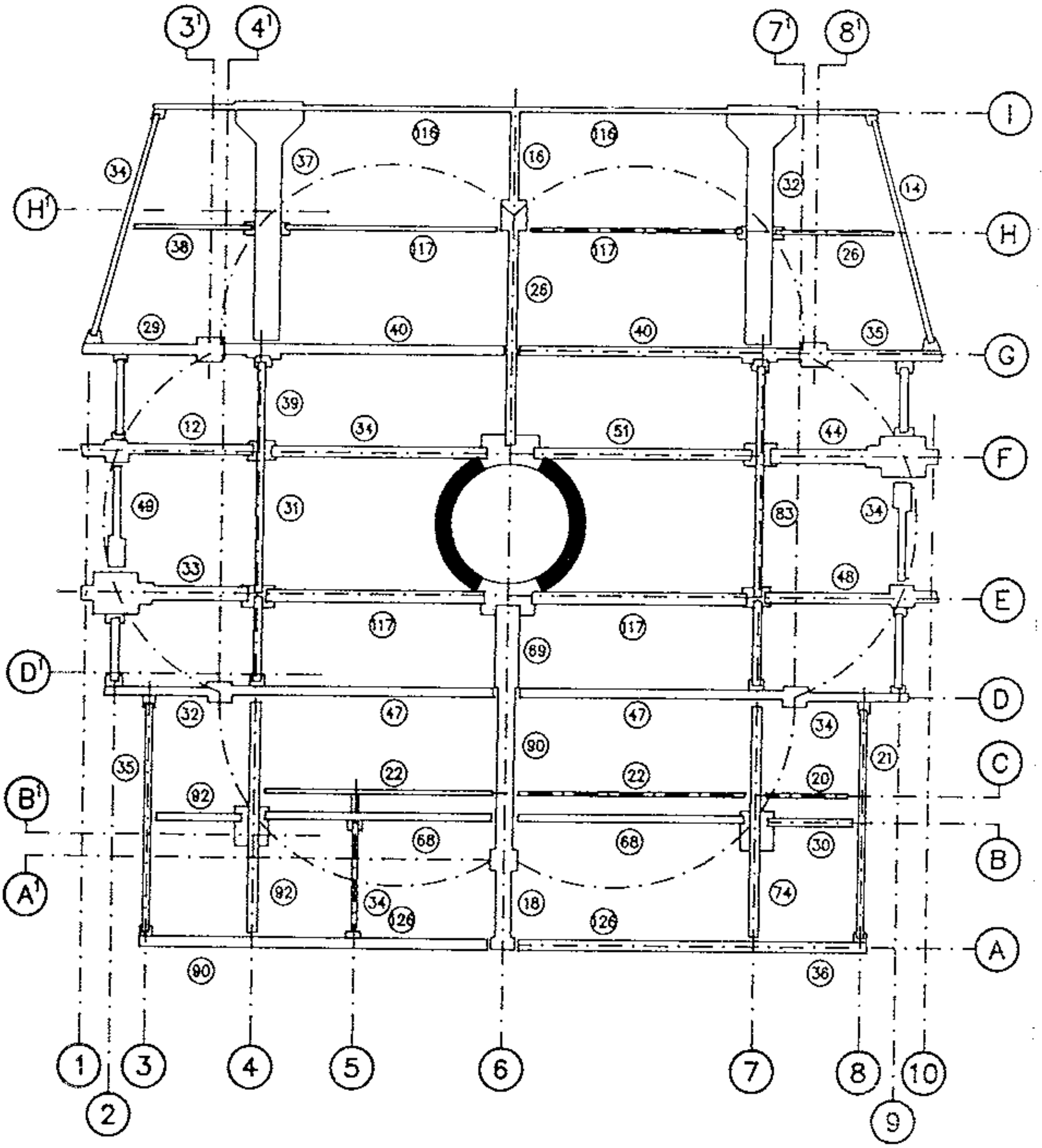


B



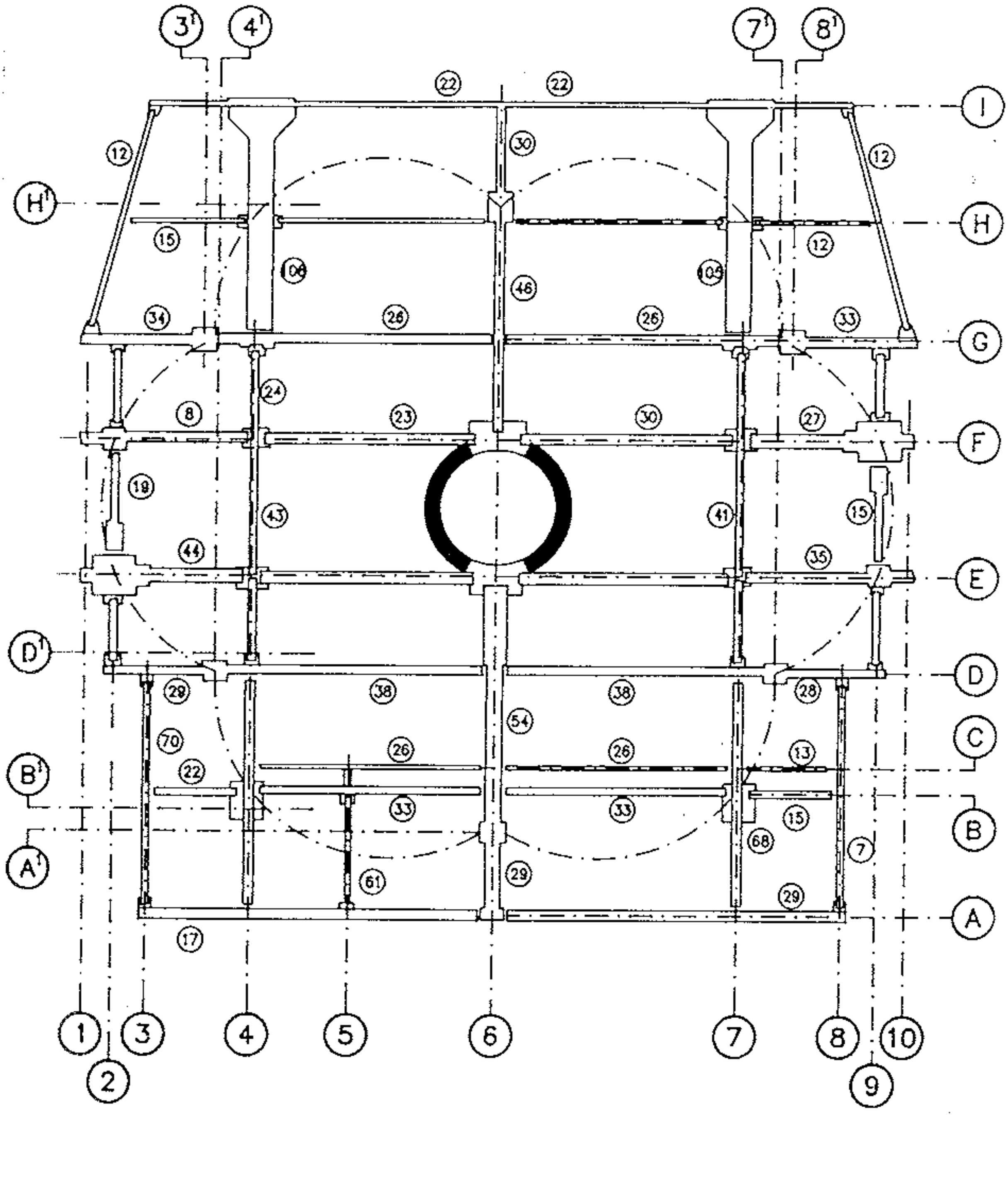
CENTRAL SHAFT:
A) CRACKWIDTH IN mm
B) CONCRETE UTILISATION
PERCENTAGE OF SLS STRESS (24 MPa)

FIGURE E7



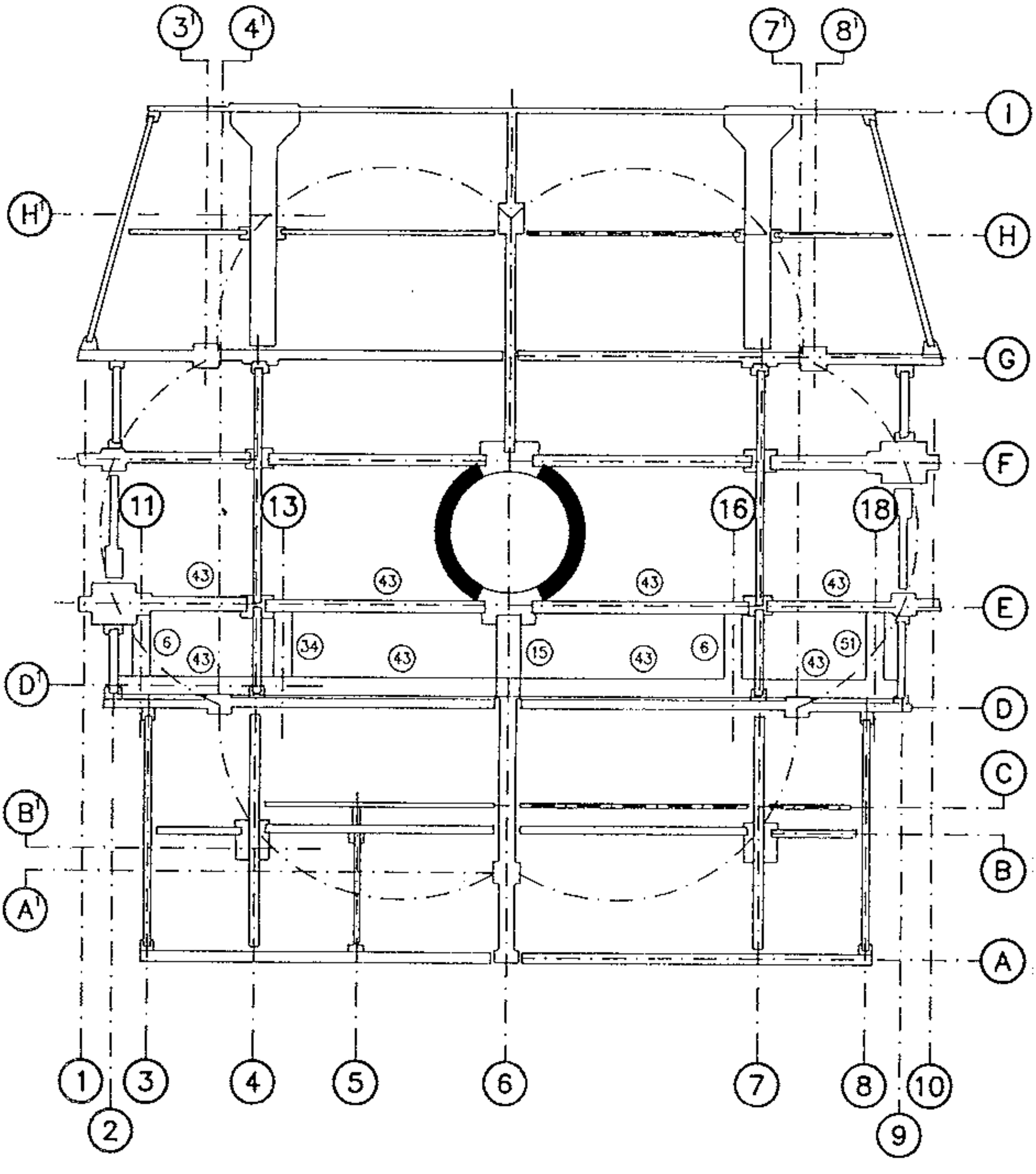
MAIN DECK BEAMS:
CONCRETE UTILISATION IN ULS

FIGURE E8



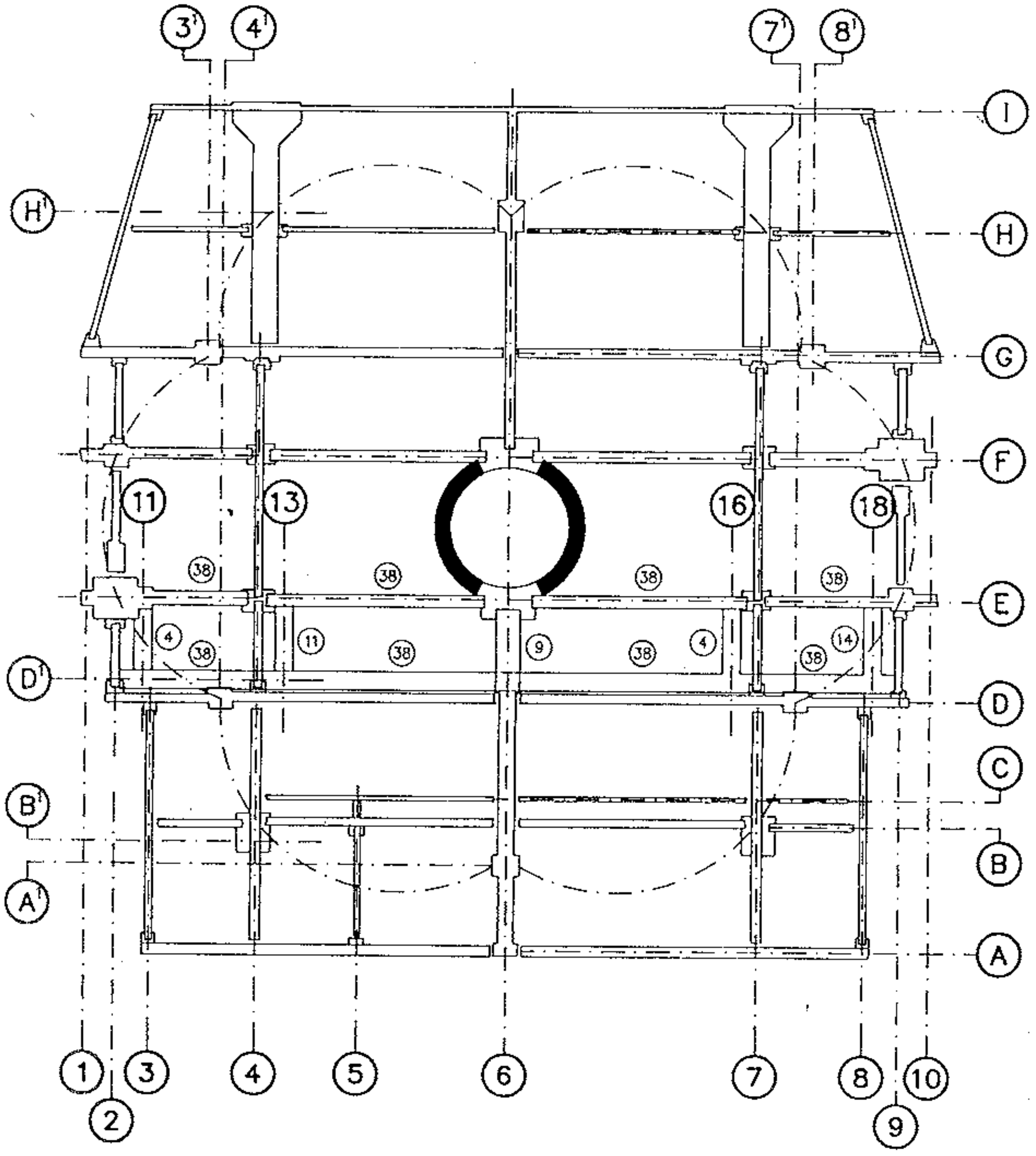
MAIN DECK BEAMS:
SHEAR UTILISATION
PRECENTAGE OF ULS SHEAR

FIGURE E9



MANIFOLD DECK BEAMS:
CONCRETE UTILISATION
PERCENTAGE OF SLS STRESS (24 MPa)

FIGURE E10



MANIFOLD DECK BEAMS:
SHEAR UTILISATION
PERCENTAGE OF ULS SHEAR

FIGURE E11

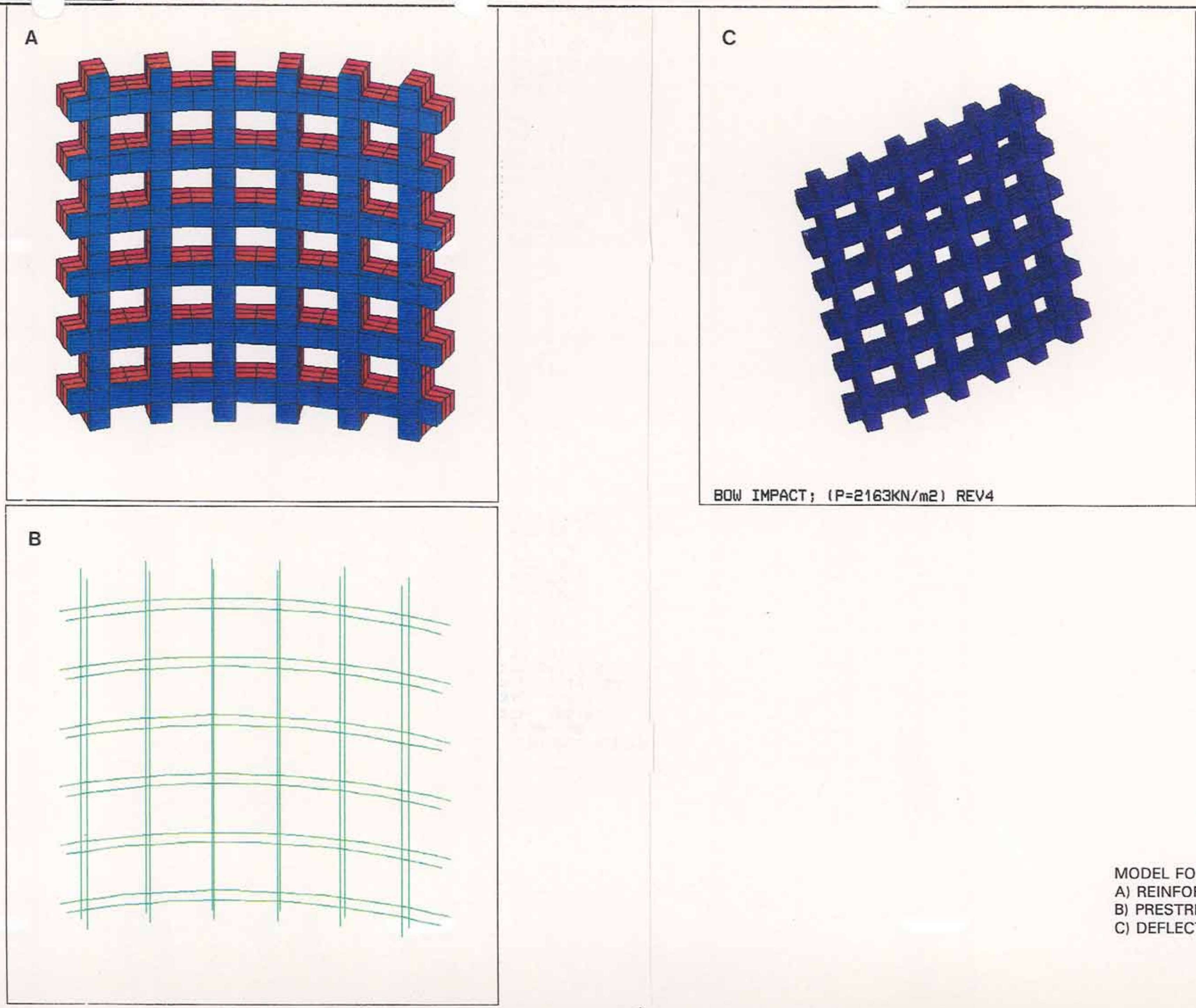
APPENDIX F

Not Used

APPENDIX G

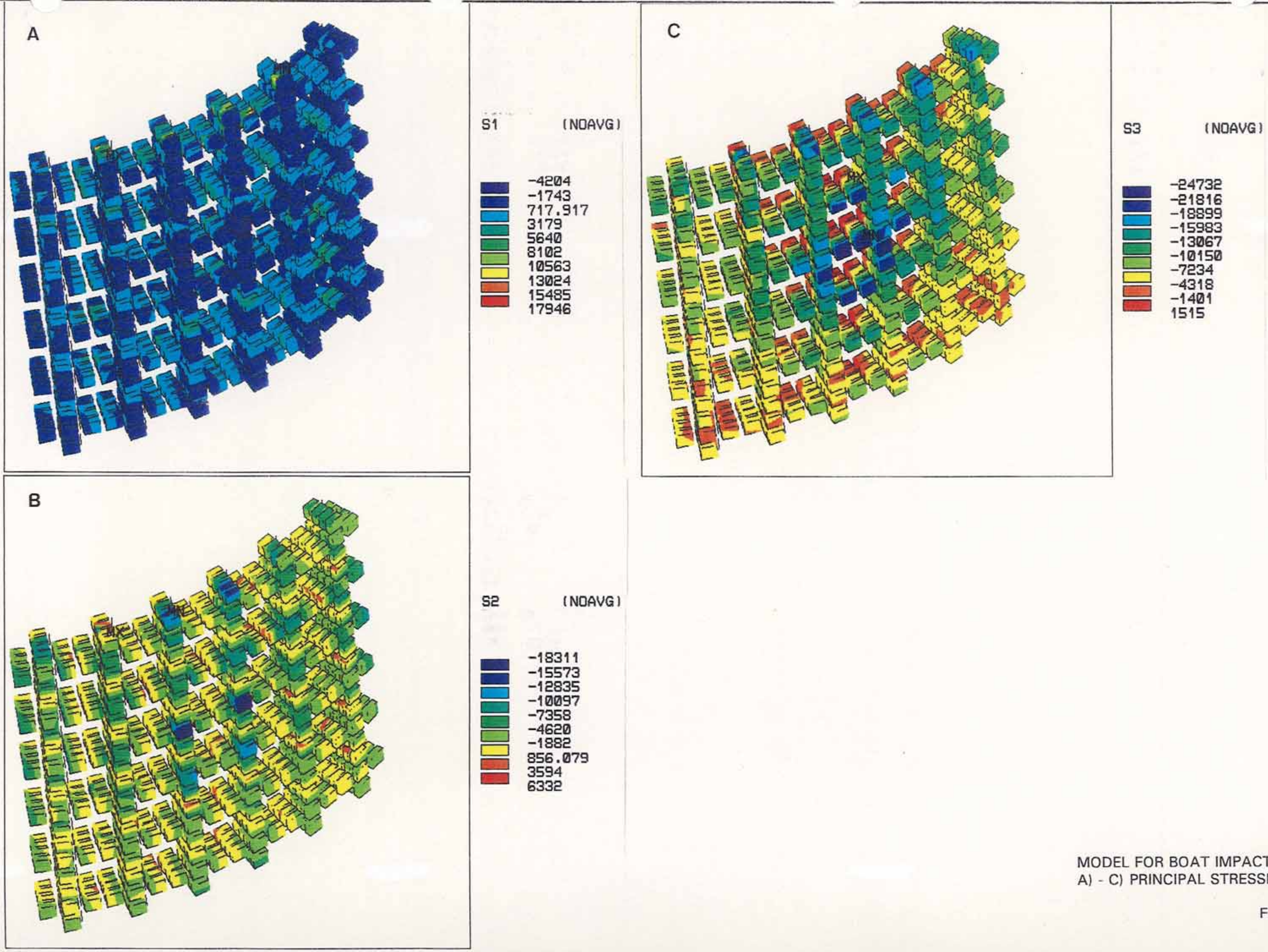
Not Used

APPENDIX H - BOAT IMPACT



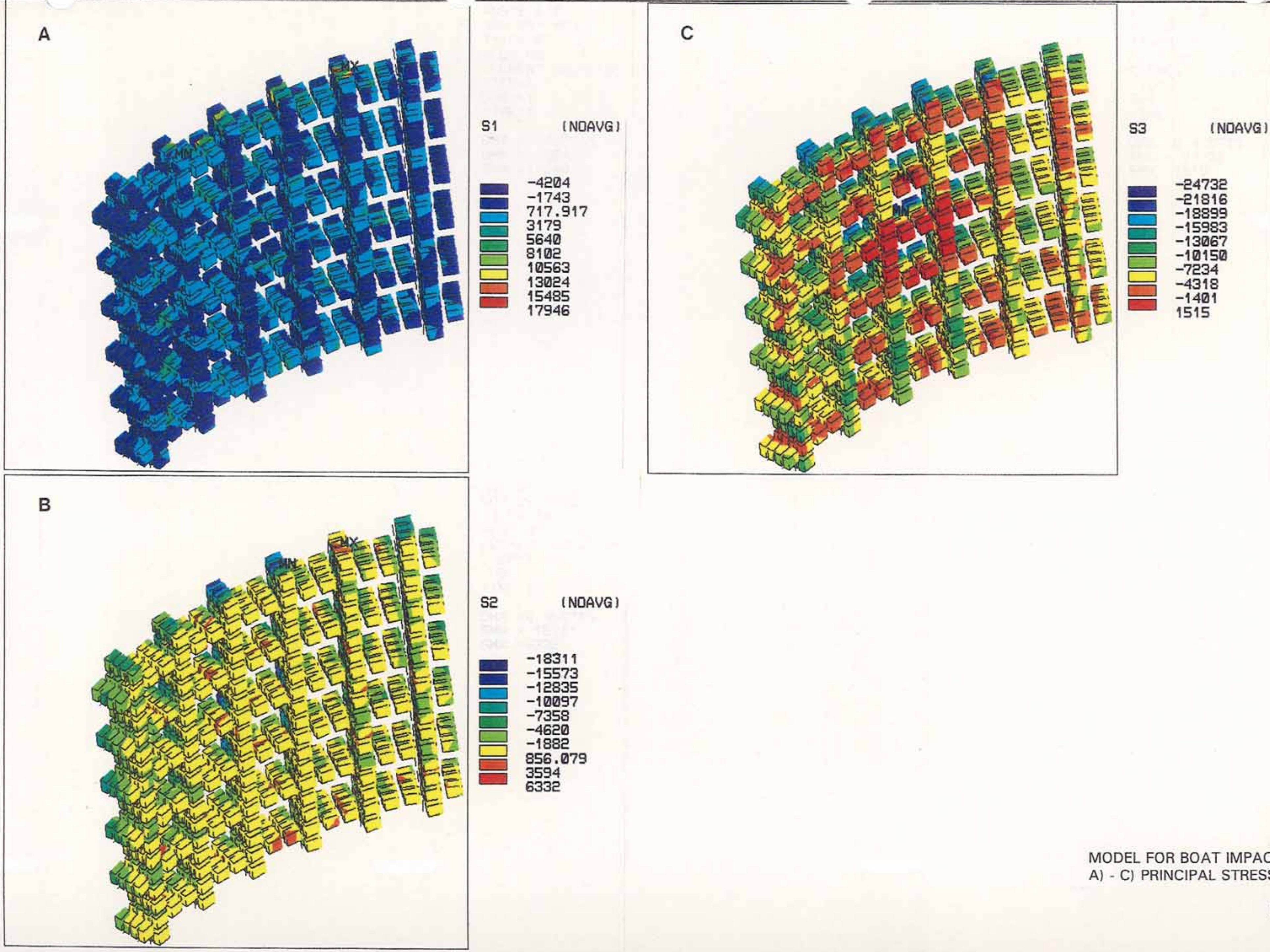
MODEL FOR BOAT IMPACT
A) REINFORCED CONCRETE ELEMENTS
B) PRESTRESSING LINK ELEMENTS
C) DEFLECTED SHAPE

FIGURE H1



MODEL FOR BOAT IMPACT
A) - C) PRINCIPAL STRESSES IN kPa

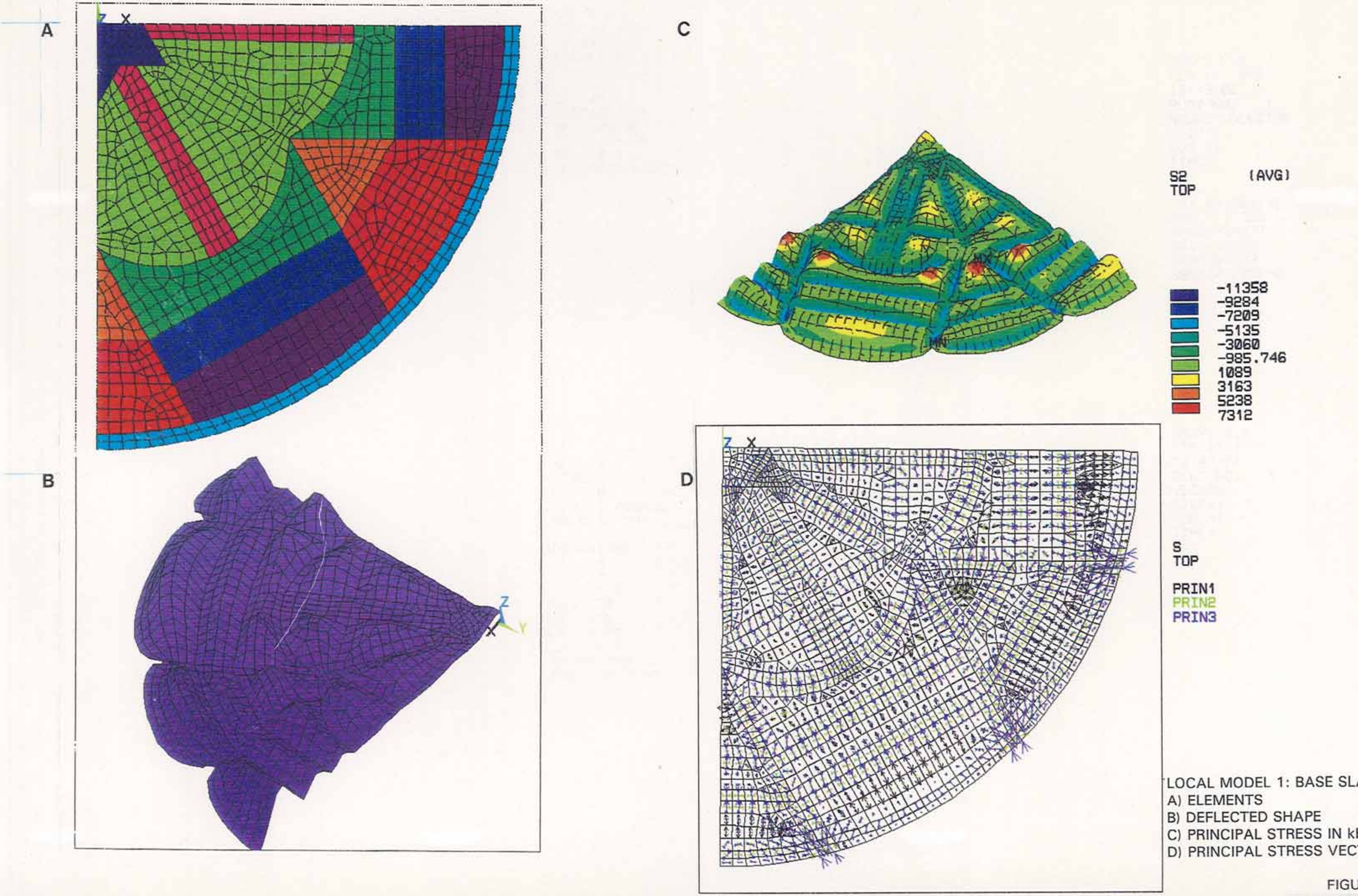
FIGURE H2



MODEL FOR BOAT IMPACT
A) - C) PRINCIPAL STRESSES IN kPa

FIGURE H3

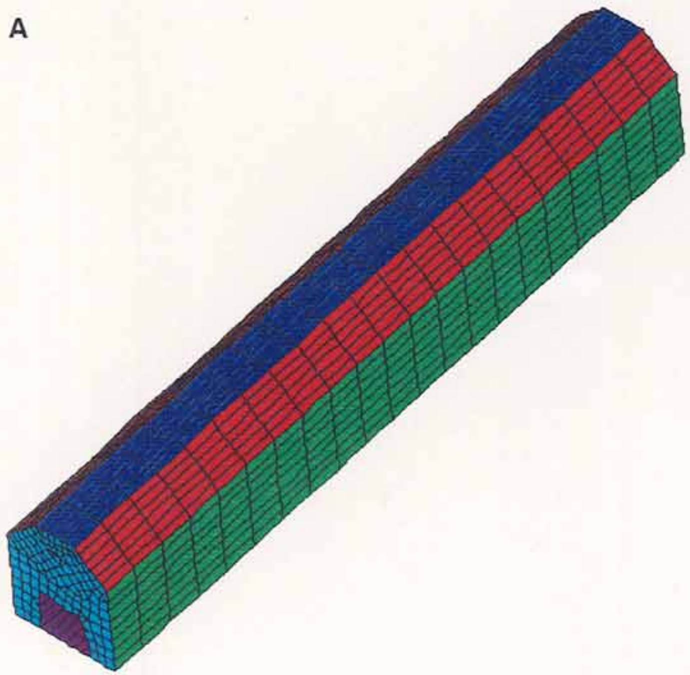
APPENDIX I - LOCAL MODELS



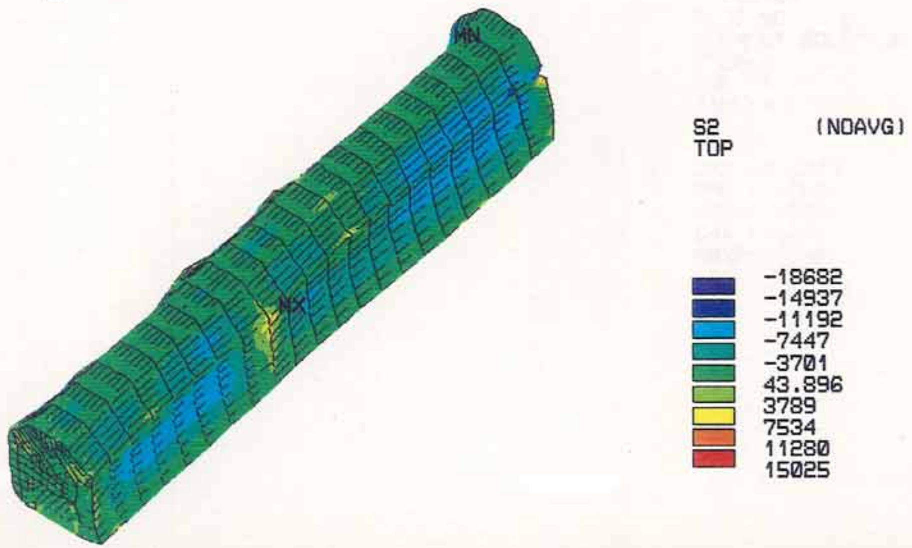
LOCAL MODEL 1: BASE SLAB
A) ELEMENTS
B) DEFLECTED SHAPE
C) PRINCIPAL STRESS IN kPa
D) PRINCIPAL STRESS VECTORS

FIGURE I1

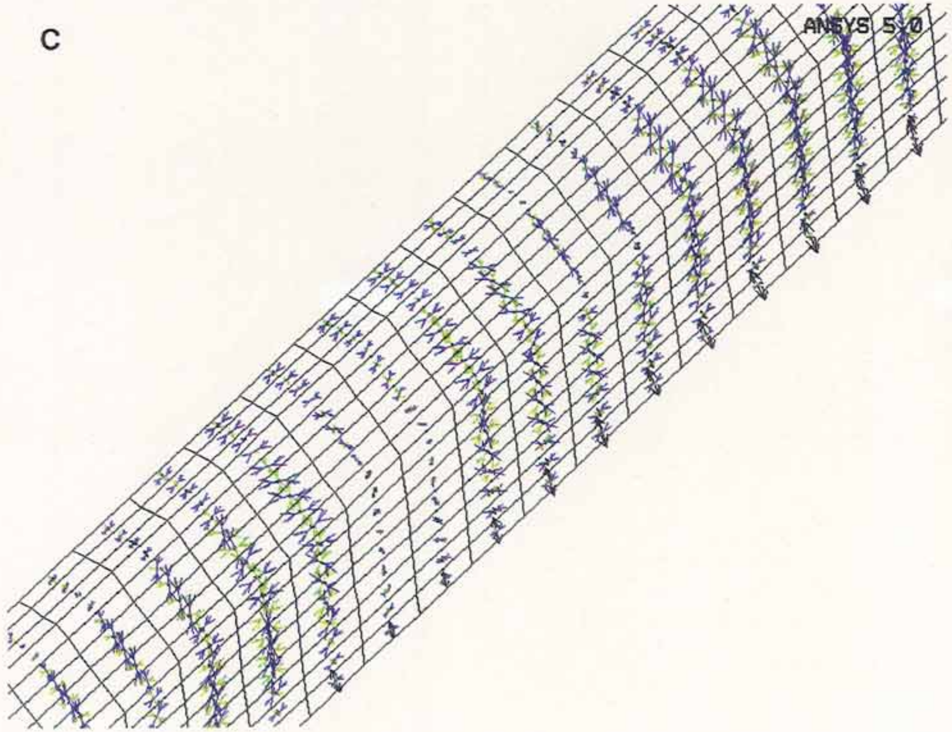
A



B

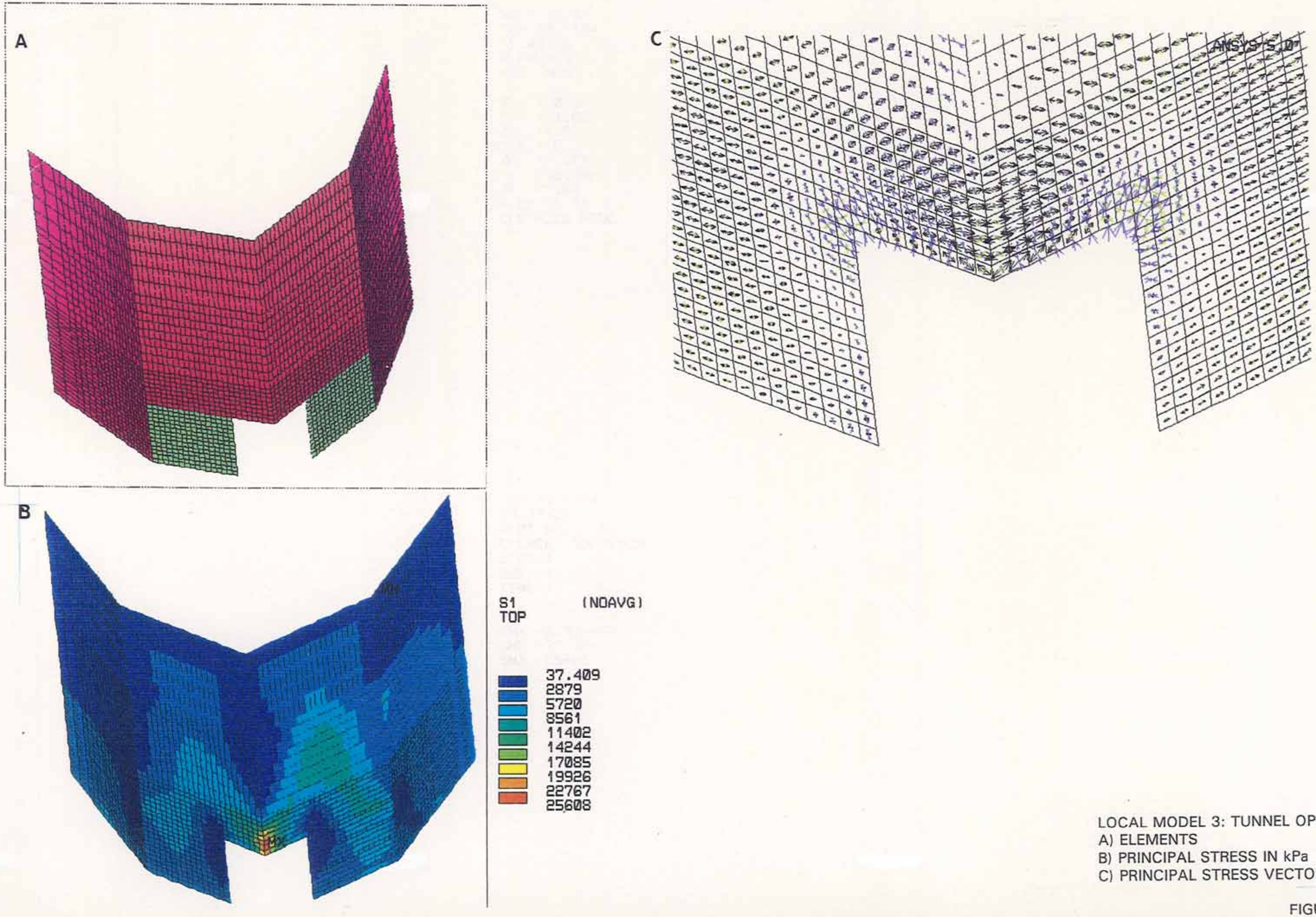


C



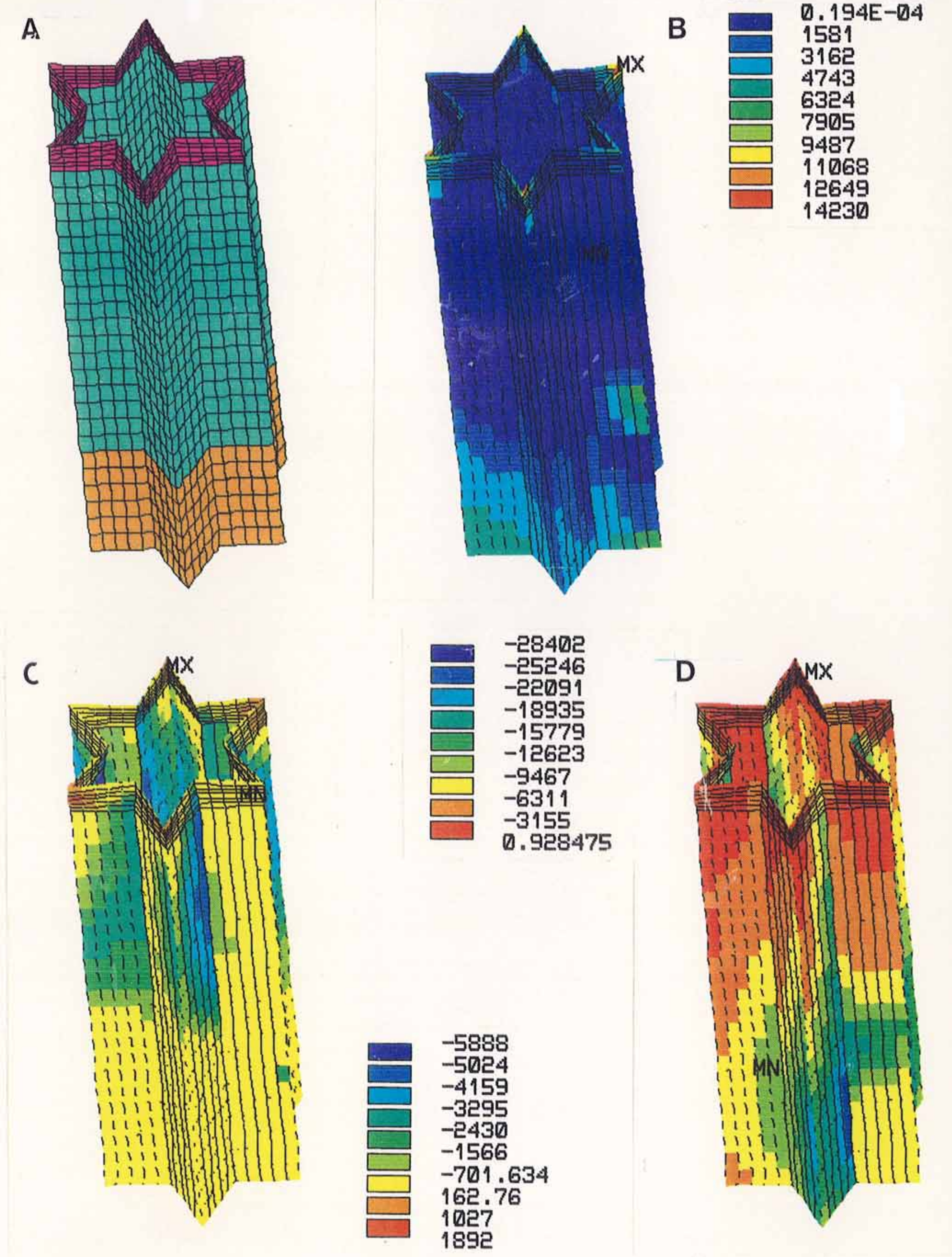
LOCAL MODEL 2: TUNNEL
A) ELEMENTS
B) PRINCIPAL STRESS IN kPa
C) PRINCIPAL STRESS DIRECTIONS

FIGURE I2



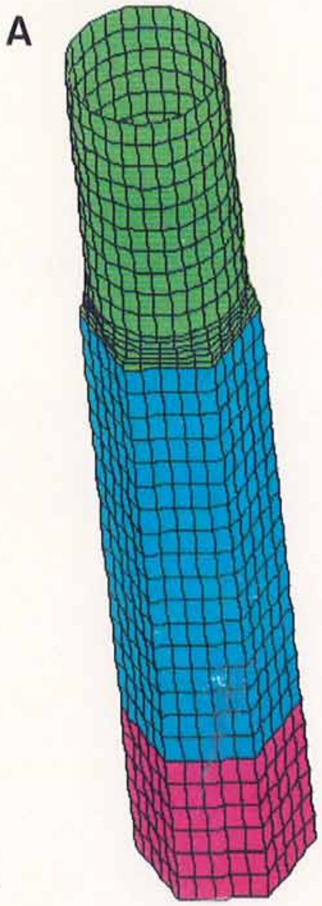
LOCAL MODEL 3: TUNNEL OPENING
A) ELEMENTS
B) PRINCIPAL STRESS IN kPa
C) PRINCIPAL STRESS VECTORS

FIGURE I3



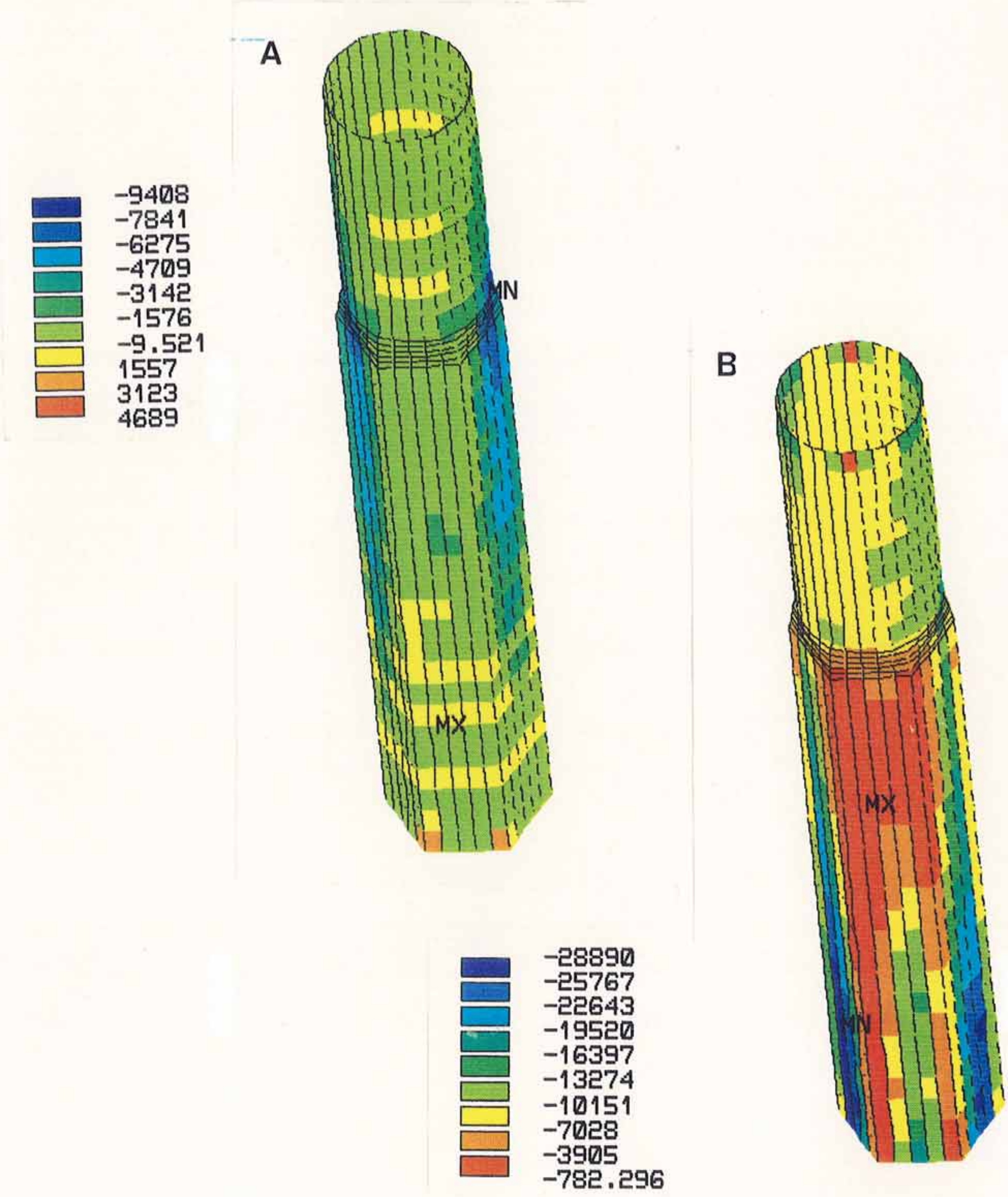
LOCAL MODEL 4: CENTRAL SHAFT
A) ELEMENTS
B) - D) PRINCIPAL STRESSES IN kPa

FIGURE 14



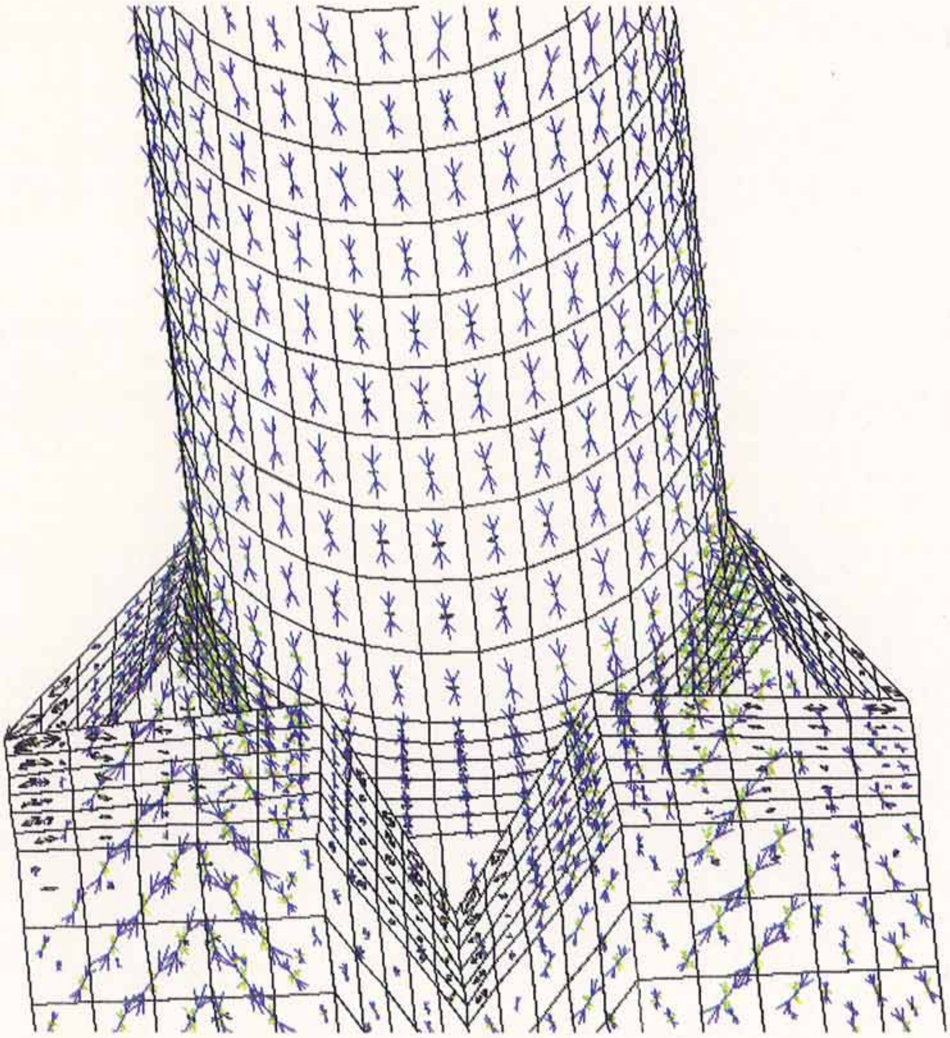
LOCAL MODEL 4: CENTRAL SHAFT
A) ELEMENTS
B) - D) PRINCIPAL STRESSES IN kPa

FIGURE 15



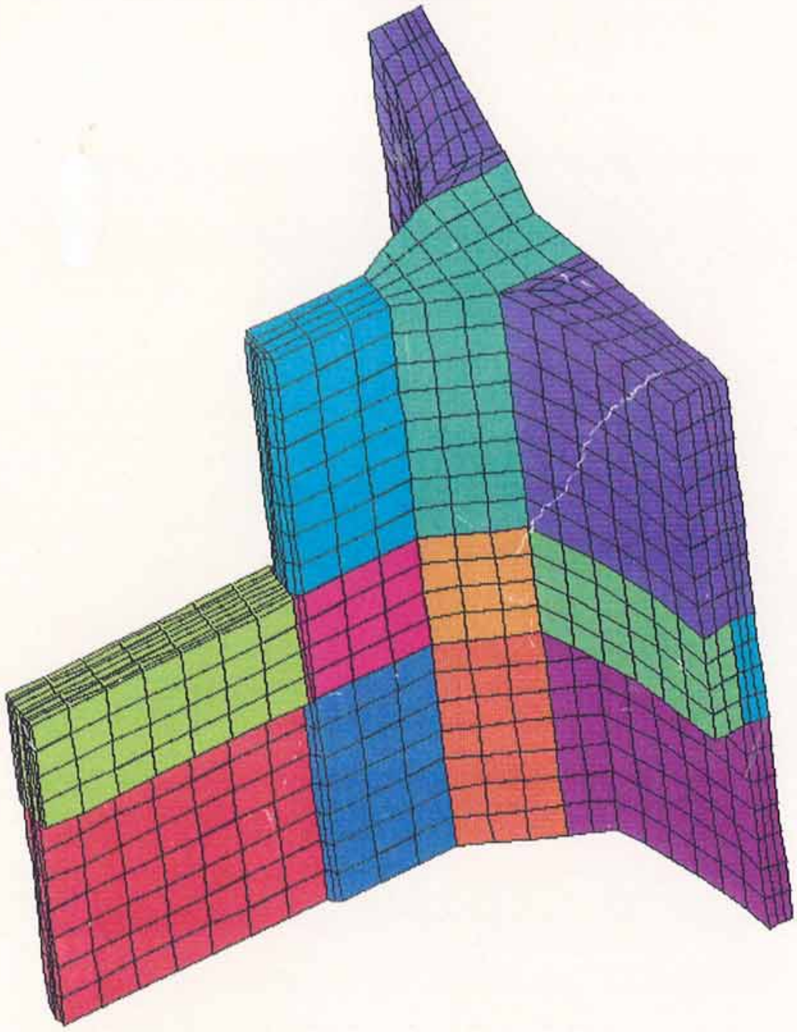
LOCAL MODEL 4: CENTRAL SHAFT
A)
B) PRINCIPAL STRESSES IN kPa

FIGURE I6



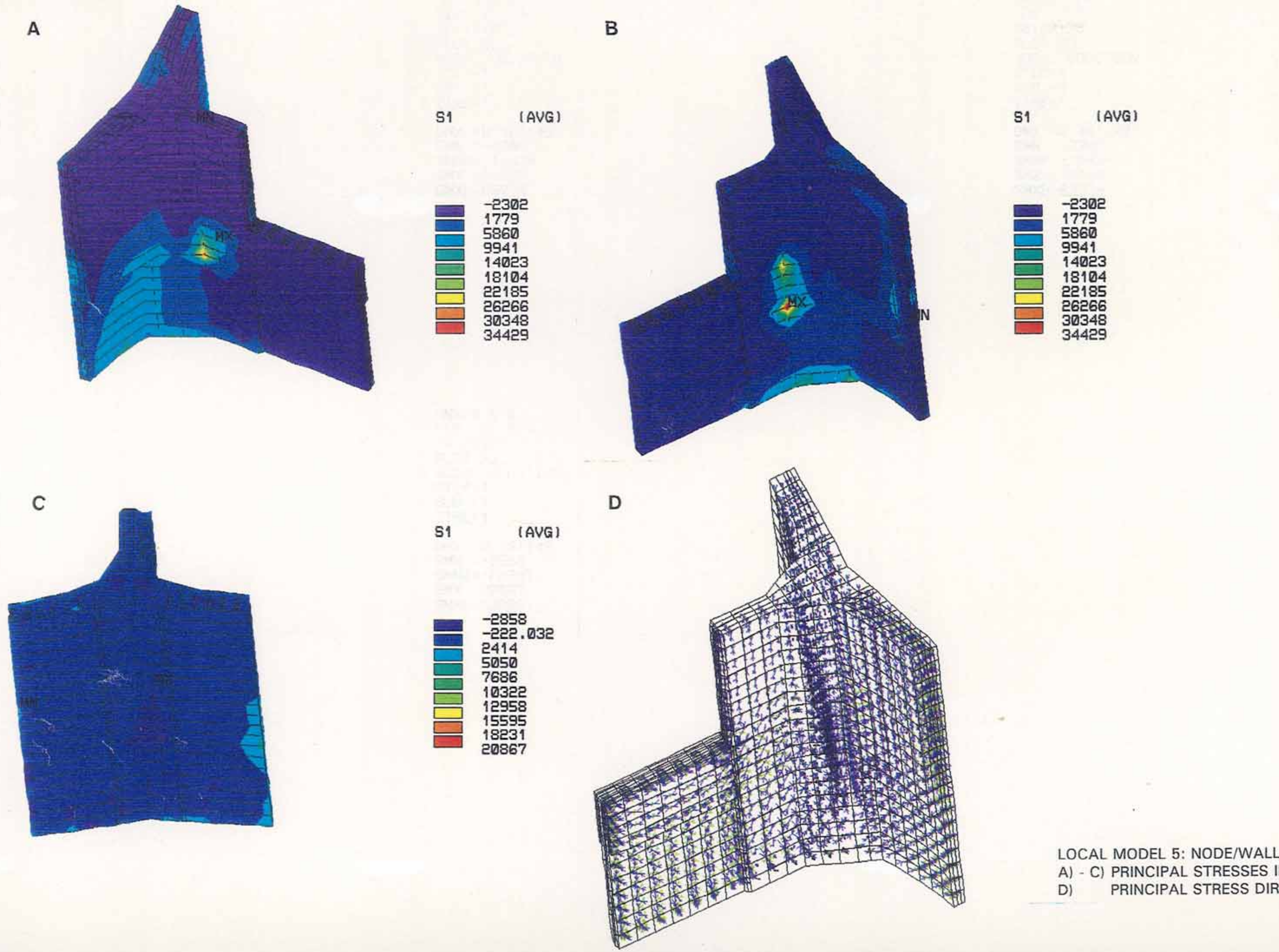
LOCAL MODEL 4: CENTRAL SHAFT
PRINCIPAL STRESS DIRECTIONS

FIGURE 17



LOCAL MODEL 5: NODE/WALL INTERFACE ELEMENTS

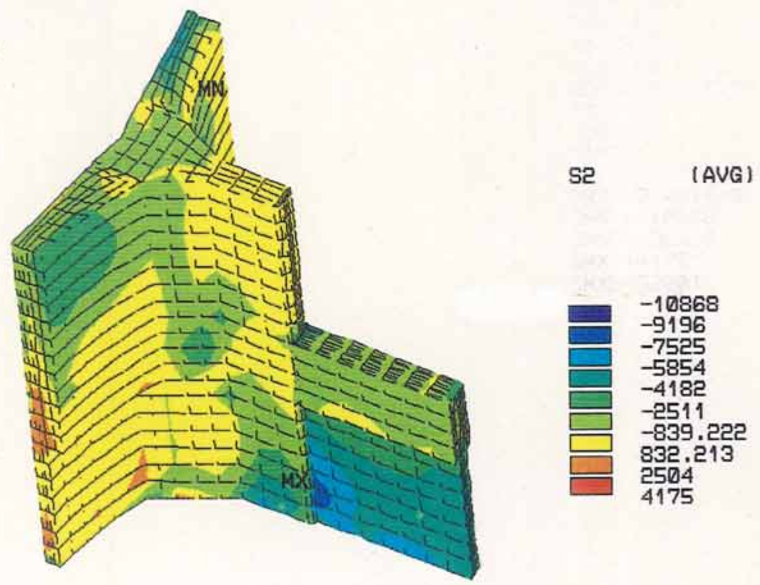
FIGURE 18



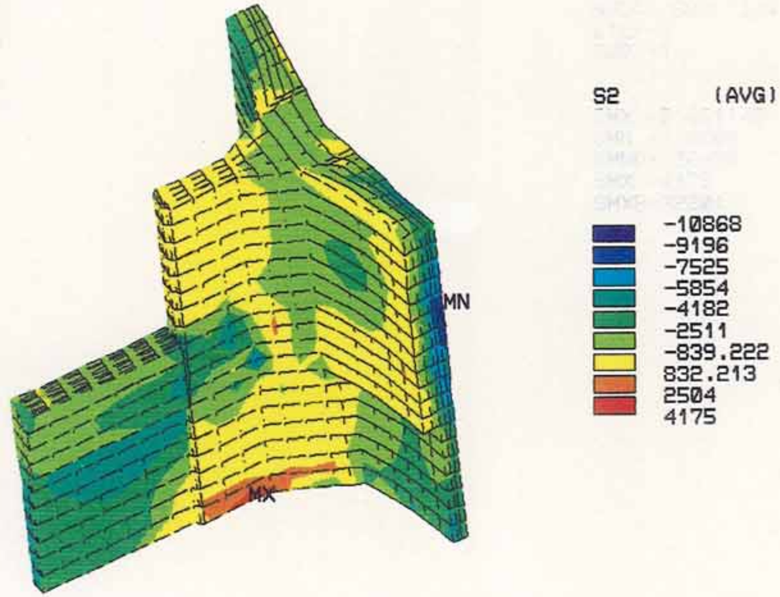
LOCAL MODEL 5: NODE/WALL INTERFACE
A) - C) PRINCIPAL STRESSES IN kPa
D) PRINCIPAL STRESS DIRECTIONS

FIGURE I9

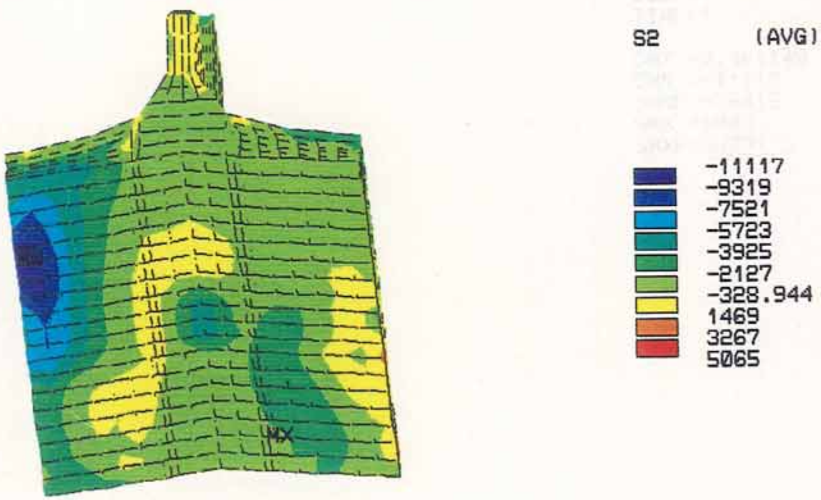
A



B

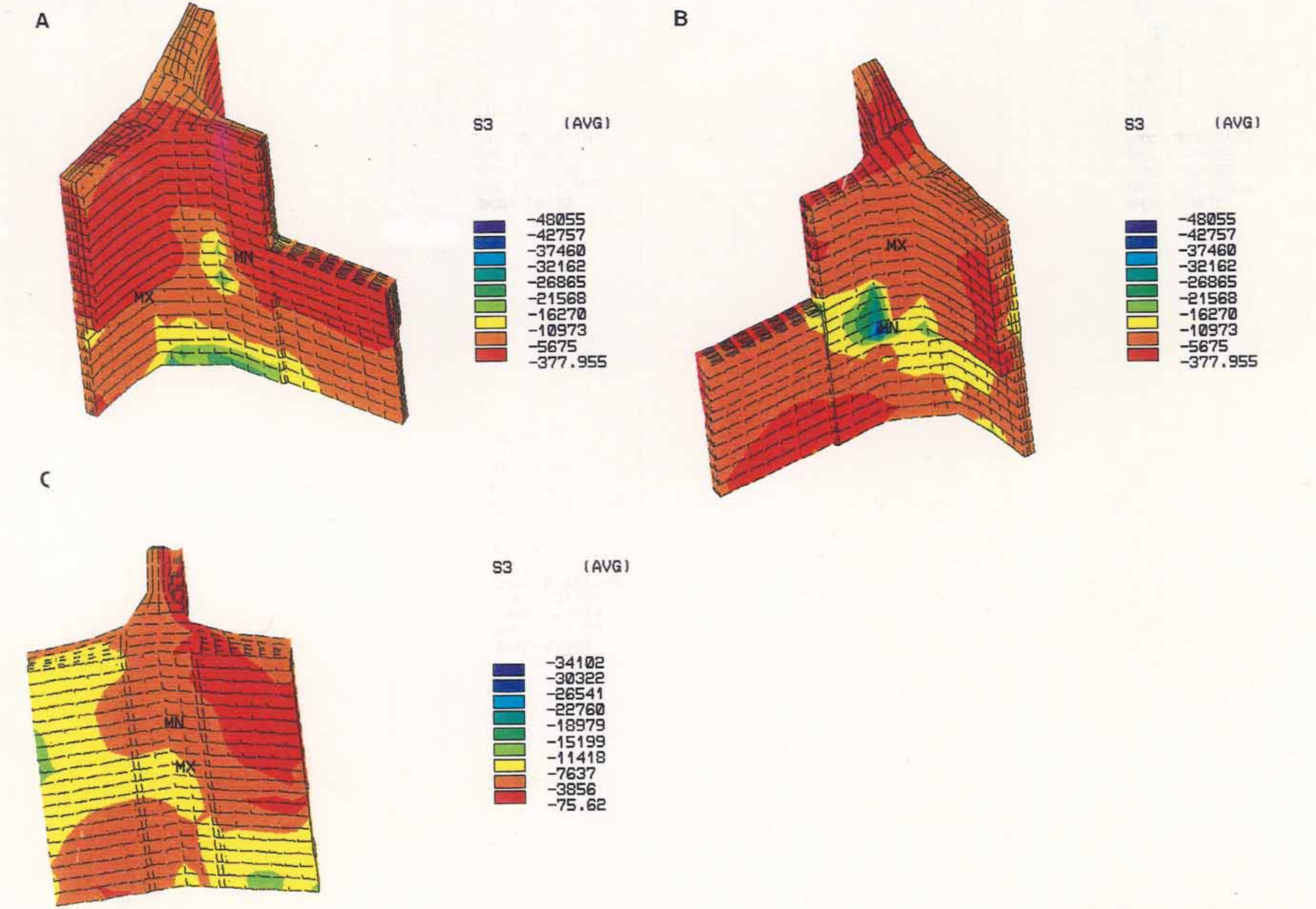


C



LOCAL MODEL 5: NODE/WALL INTERFACE
A) - C) PRINCIPAL STRESSES IN kPa

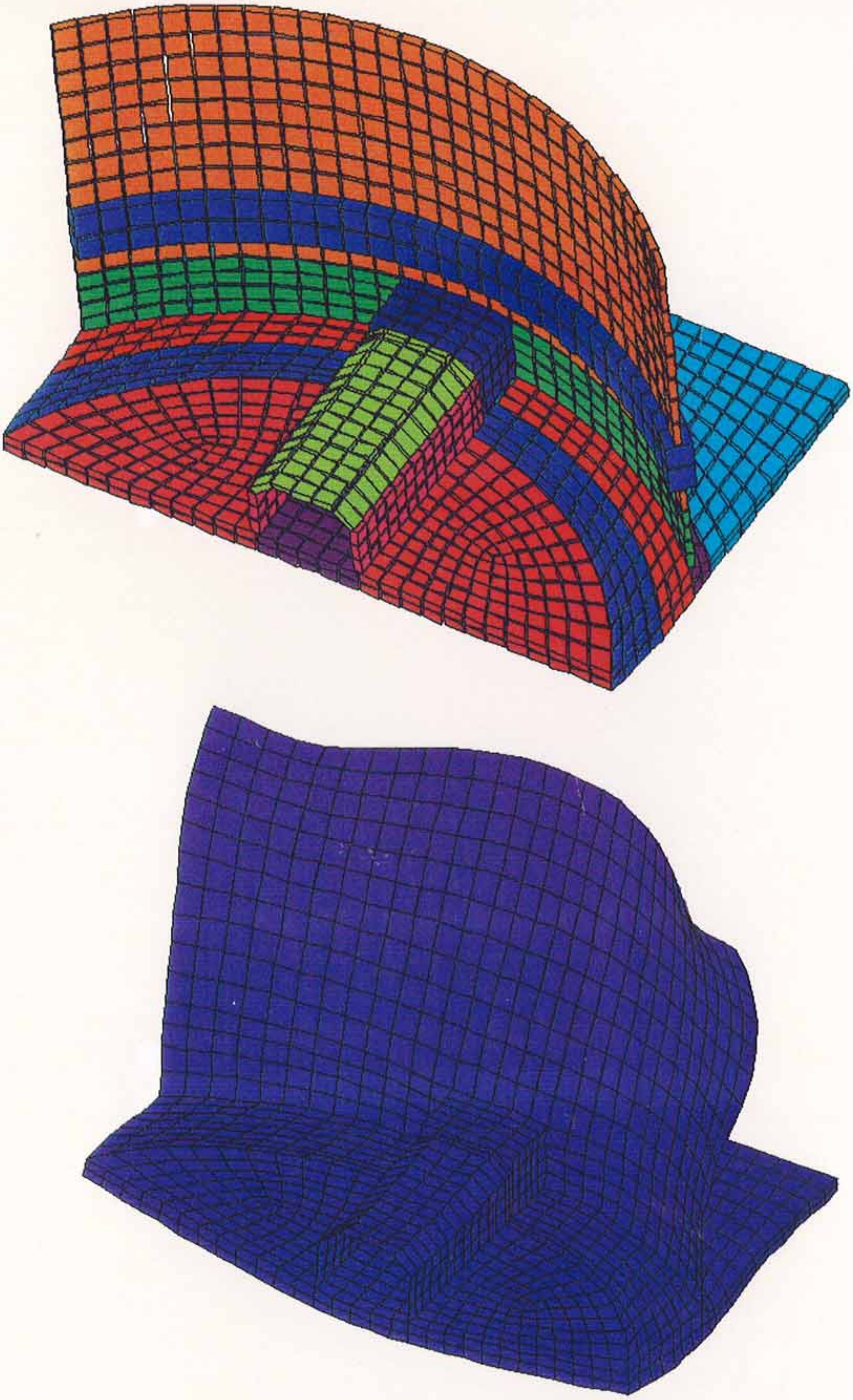
FIGURE I10



LOCAL MODEL 5: NODE/WALL INTERFACE
A) - C) PRINCIPAL STRESSES IN kPa

FIGURE I11

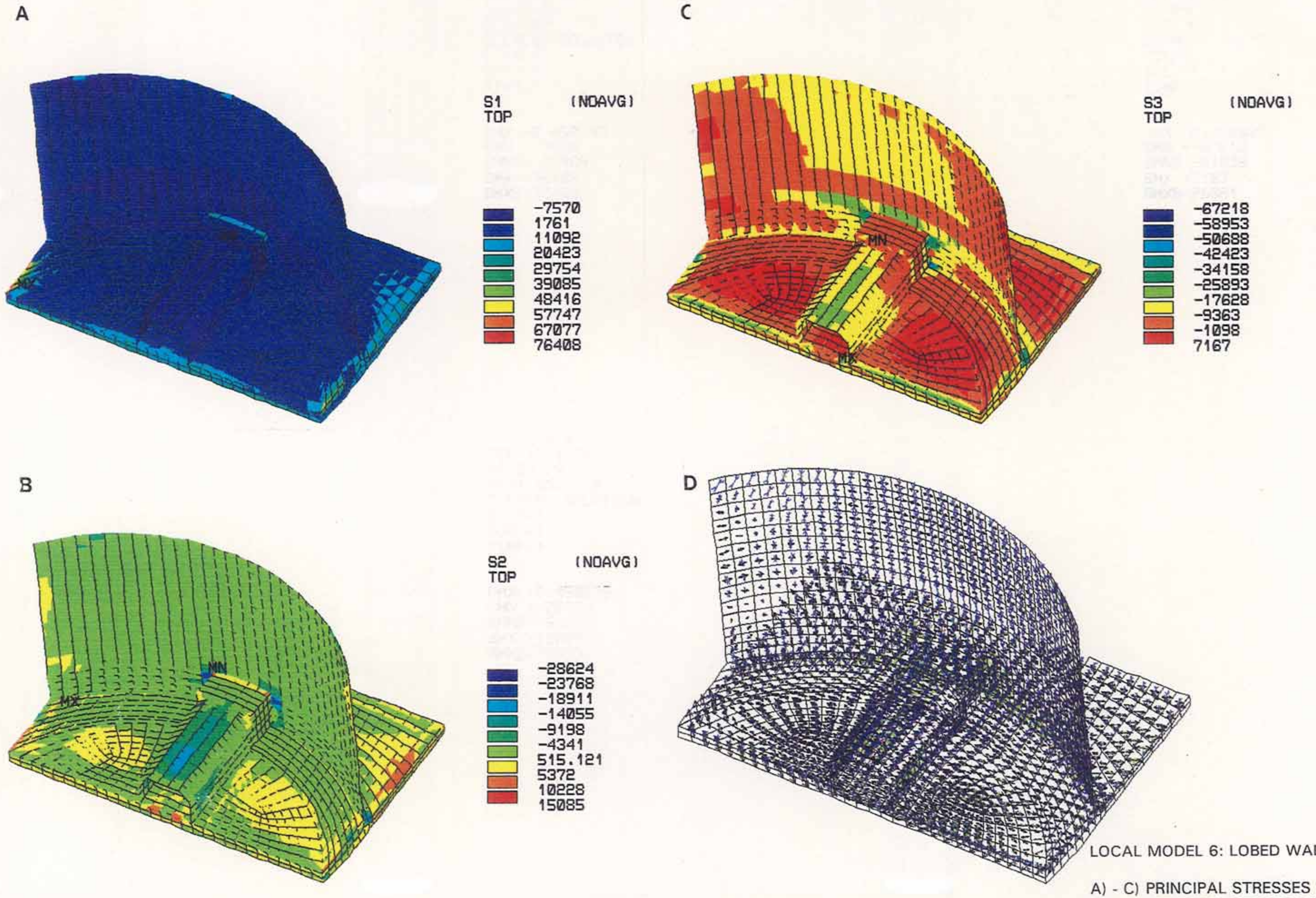
A



B

LOCAL MODEL 6: LOBED WALL/BASE SLAB
A) ELEMENTS
B) MAGNIFIED DISPLACED SHAPE

FIGURE I12



LOCAL MODEL 6: LOBED WALL/BASE SLAB

A) - C) PRINCIPAL STRESSES IN kPa
D) PRINCIPAL STRESS DIRECTIONS

FIGURE I13

2

PART 2

BACKGROUND DOCUMENT

PROJECT:
TOTAL OIL MARINE
MCP-01 PLATFORM

McALPINE
Design Group

BACKGROUND REPORT ON SPECIFICATION FOR RE-ANALYSIS OF CONCRETE STRUCTURE

DOCUMENT NO.	3311-S-M-001	ODE JOB No. 331
		Page 1 of 58

15/1/93	0	FIRST ISSUE	RGF RH	PW	DK
DATE	REV NO.	STATUS	ISSUED BY	CHECKED BY	APPROVED BY

CONTENTS

APPENDICES

1.0 OBJECTIVES AND REQUIREMENTS

1.1 INTRODUCTION

This document provides the background work to the initial sections of the specification for the re-analysis of MCP-01 which has been prepared by McAlpine Design Group and Offshore Design Engineering Ltd for Total Oil Marine Ltd. It gives details of the information available, the quality of the information, gaps in the information, selection of data for the re-analysis and likely methods of re-analysis.

1.2 TOTAL OBJECTIVES

The original certification for MCP-01 by Lloyds in 1975 was for 20 years. Total wish to extend the platform life by at least another 20 years to 2015 by gaining re-certification by Lloyds. This will be the first re-certification for a concrete platform in the UK sector.

As part of the re-certification process a re-analysis of the structure is required. Total intend to let a contract for this re-analysis during the early part of 1993. The objective of the current exercise is to write a specification for that re-analysis.

The re-analysis will cover the whole of the GBS, the deck support columns, and the concrete deck beams at +123.00 and +133.00.

The platform was demanned at the end of 1992 with maintenance teams flown in every 2 weeks or as necessary. There is no overnight accommodation (except for emergencies) and the platform will be remotely controlled from St Fergus. At present only one active gas line is anticipated.

However, Total are hoping to have as much flexibility as possible with regards to future developments in neighbouring fields, and the possibility of adding more equipment etc.

As well as providing a re-analysis specification, the current exercise has collated as much existing data as possible on the structure. This has been done by researching Total, ODE and Doris records. The objective was not to review relevant documents or drawings in detail but to list and assemble those that would be required in the re-analysis. This also enables gaps in the information required to be established.

In parallel with this work Total have requested MAREX Ltd to produce an environmental report for the site.

Aspects of the re-analysis that require detailed consideration include:

- Soils data/scouring
- Applicable design codes (is any retrospection allowed?)
- Environmental loading
- Wave loading on Jarlan wall and deck columns
- Marine growth and particularly effect on Jarlan holes
- Current topsides loading

- Highly stressed areas, e.g. external diaphragms
- Structural defects/deterioration

The McAlpine/ODE scope of work is contained in Appendix 1A.

1.3 LLOYDS' REQUIREMENTS

1.3.1 General Philosophy and Criteria

Lloyds generally expect re-analysis to be carried out to current editions of codes (in line with draft ISO code) and that an equivalent level of safety to that given by Department of Energy Guidelines and BS 8110 is demonstrated.

Code requirements are likely to be similar for manned platforms but account would be taken of demanning when assessing the safety of the platform. Acceptable levels of safety under serviceability would take into account current state of platform, operational conditions and level of inspection and monitoring.

1.3.2 Re-Analysis

1.3.2.1 Environmental Conditions

Lloyds would accept an analysis by MAREX based on long term data with minor checking. Environmental conditions based on limited data would require more extensive checking. Recent predictions tend to be lower than those used for original design. Lloyds would expect Total to propose design conditions for their agreement based on the environmental data. A 100 year extreme event would be considered to give sufficient safety for a 20 year life without further evaluation, whereas a 50 year event would need further evaluation together with load factors to establish that there was an adequate level of safety.

1.3.2.2 Loading

Modification of wave action due to the Jarlan wall needs to be allowed for and Lloyds would expect justification of the methods of load calculation. Doris have calibrated their modelling of loads against original model tests.

Lloyds would expect documentation defining existing topside loading.

Lloyds' main concern regarding accident loading would be ship impact for which a check of the effect of a defined boat would be expected.

Vulnerability of radial beams to damage by dropped objects may need consideration.

Lloyds have not considered seismic loading to date.

1.3.2.3 Soils

Lloyds are currently evaluating the soil pressure loading used by Doris for Hibernia, which is considered to be more realistic than using springs.

There has been some scour of the foundations, but this is not extensive and has been grouted. Lloyds would not expect it to be a concern.

1.3.2.4 Material Properties

Ageing of concrete might have detrimental effects (creep) as well as beneficial (strength gain). Account should be taken of areas where there are known to have been problems.

Leaking of cachetages has been repaired by resin injection and is accepted as being satisfactory. An assessment of the effect of some reduction in the level of prestressing might be considered.

1.3.2.5 FE Model

Lloyds would review the modelling and would expect justification that the model represents the structure adequately (e.g. at changes of section).

1.3.3 Lloyds' Approval

Total should submit a design specification and a sheet of design parameters to Lloyds for their agreement prior to carrying out the re-analysis. Lloyds would also like the opportunity to comment on the calculation of wave loads.

For approval Lloyds would review the whole analysis and carry out some independent calculations, for example by checking right through one area of the structure.

1.4 CODES AND STANDARDS

The original design was carried out to the following standards:

ACI 318-71	Building Code Requirements for Reinforced Concrete
FIP - CEB	Recommendations for the Design and Construction of Concrete Sea Structures, 2nd edition, 1974

Lloyds have indicated that it should be demonstrated that the codes used in the re-analysis give an equivalent level of safety to that contained in the current edition of the DEn Guidance Notes. The Guidance Notes in turn then refer to BS 8110 with respect to concrete design. This requirement of Lloyds would suggest that the easiest approach is to base the re-analysis on the current edition of the Guidance Notes.

A comparison between the original and current codes is given in Appendix 1B. This concludes that the differences in section capacity calculated by BS 8110 and ACI 318 are due to the different material characteristics and safety factors applied rather than in the application of the equations governing equilibrium.

The major loss in calculated allowable capacity due to the change in codes will be in shear. For this type of structure the reduction in shear capacity will mainly affect slabs with differential pressures across them, such as the base slab.

However, the present Guidance Notes do not specify how the problem of principal stresses in directions other than that of the main reinforcement should be treated. The DnV rules on the other hand do recommend suitable methods.

The original structural analysis of the base slab, radial diaphragms and circular external walls was carried out by hand using yield line theory, deep beam or cantilever corbel analogies. The arches of the lobate Jarlan wall have been analysed using a grillage of beam element models.

Reflecting the developments in computer analysis and finite element modelling a more detailed analytical model will be used. The concrete substructure will be modelled as membrane finite elements, capable of carrying both inplane and out of plane axial, bending and shear forces. The output from this type of analysis will be either principal stresses or orthogonal stresses aligned with the main reinforcement directions.

Clearly this will require a different design technique to be employed. A design procedure for reinforcing against principal stresses at angles to the reinforcement has been developed.

Differences in the required reinforcement and prestressing levels will not be large where the principal stress directions coincide with the reinforcement directions.

In areas of high shear (deep beams and corbels) vertical and horizontal prestress has been applied to overcome the potential shear cracking, again a substantial difference in calculated capacity should not occur if the safety margins anticipated in the code are maintained.

A further problem to be addressed is how to handle the high peak stresses that will occur in the FE analysis at geometric discontinuities.

2.0 DOCUMENTS

2.1 CURRENT TOPSIDES DRAWINGS AND REPORTS

Key General Arrangement and Loading drawings are listed below.

Other drawings listed will be needed to clarify the exact geometry and give details of reinforcement, prestressing and connection between precast members.

Key Drawings and Reports for Topsides

Weight Control Report	Not available
Structural Design Report	MP-5009-M4-CL-31 Vol 2 Part 1
Main deck General Layout	A1-MPZ-QD-4001
Main deck - North Area Loads at Level 123.0	MP-5009-M4-15-01
Main deck - South Area Loads at Level 123.0	MP-5009-M4-15-02
Main deck - Loads on underdeck	MP-5009-M4-15-03
main deck - Loads at Level 133.0	MP-5009-M4-15-04
Occidental Riser Vertical Trussed Beam GA	A1-OR-30-301
Occidental Riser Vertical Trussed Beam GA	A1-OR-30-302
Load Repartition Structure	Title Blocks illegible
LRS Transverse Trusses	Title Blocks illegible

Key Drawings For Topsides Not Currently Available

- Current equipment plot beams and elevations
- Drawings MP-5035-M4-15-1025 to 1028 for LRS
- Drawing ????-00-4014 for column types

Supplementary Drawings For Topsides

Main Deck Panels A1-MP2-QD-6309 to 6364
Underdeck baskets MP2-5009-M4-15-140 to 158
(cannot read No.s on drawings)
Columns A1-MP2-QD-6103 to 6121
Load Repartition Structure (Note title blocks are illegible)
MP2-5009-M4-15-11 to 15,40,41,60,70
MP2-5009-M4-15-120 to 129
Occidental Riser A1-OR-30-303 to 330

2.2 SUB-STRUCTURE DRAWINGS AND CALCULATIONS

Substructure drawings are listed in Volume 7 Section 1 of the Concrete Structure Design Manual. The drawing groups are listed below; individual drawing titles may be found in the complete list, and the drawings are in Volumes 8 and 9.

Sub division	from number	Group	Designation
QU	7001	Manifold	Formwork Drawings
QU	7101	Central Shaft	Formwork Drawings
QU	7201	Manifold	Steel and Cable Drawings
QU	7301	Central Shaft	Steel and Cable Drawings
QU	7401	Manifold	Steel Schedules
QU	7501	Central Shaft	Steel and Cable Drawings
QU	7601	Manifold	Cable Schedule
QG	1051	General	Formwork Drawings
QG	1999	General	List of Documents
QR	2001	Raft Foundation	Formwork Drawings
QR	2201	Raft Foundation	Cable Drawings
QR	2301	Raft Foundation	Steel Drawings
QR	2401	Raft Foundation	Steel Schedules
QR	2601	Raft Foundation	Cables Schedules
QR	2901	Raft Foundation	Scheme Plans
QV	3001	Cylindrical Vessel	Formwork Drawings
QV	3101	Cylindrical Vessel	Formwork Drawings
QV	3201	Cylindrical Vessel	Cables Drawings
QV	3301	Cylindrical Vessel	Steel Drawings
QV	3401	Cylindrical Vessel	Steel Schedules
QV	3601	Cylindrical Vessel	Cables Schedules
QV	3901	Cylindrical Vessel	Scheme Plans
QD	4001	Decks	Formwork Drawings
QD	4201	Decks	Cables Drawings
QD	4301	Decks	Steel Drawings
QD	4401	Decks	Steel Schedules
QD	4601	Decks	Cables Schedules
QD	4901	Decks	Scheme Plans

Calculations are also listed in Volume 7 Section 1 of the Concrete Structure Design Manual. The calculation groups are listed below; titles of individual calculations may be found in the complete list.

Sub division	from number	Group	Designation
QG	1800	General	Design Calculations
QG	1900	General	Computer Calculations

Equipment Drawings.are listed in Volume 7, Section 2 of the Concrete Structure Design Manual. The drawing groups are listed below; individual drawing titles may be found from the complete list and the drawings are in Volumes 10 and 11.

Sub division	from number	Group	Designation
QC	6104	Deck	Guide Pump North
QD	6100	Deck	Columns
QD	6200	Deck	Base Plates
QD	6300	Deck	Steel Decking, Fixtures
QF	1000	Platform	General
QM	5000	Platform	Seal Caisson Plate
QM	6000	Platform	Central Shaft
M	1000	Platform	General
M	6000	Platform	General
M	6200	Platform	Ballasting System
M	6300	Platform	Plug
M	6500	Platform	Echo Sounder
R	6000	Platform	Liaison Pipe Riser
QT	6100	Platform	Central Shaft
QT	6200	Platform	Upper Deck
QT	6600	Platform	Steel Decking

2.3 DESIGN REPORTS

The following are the relevant design reports:

21946 Concrete Structure Design Manual Volume 1, Howard Doris

D1013 Technical Specification for Reinforced and Prestressed Concrete, Doris, April 1975 (contained in Document No. 21946).

D1077 Revised Construction Procedure, Doris, December 1975 (contained in Document No. 21946).

D1161 Sand Ballasting Report of MCP-01 Platform, Doris, 26 August 1976.

2.4 INSPECTION REPORTS

The relevant inspection reports are listed below. McAlpine hold copies of the detailed reports and data sheets for the atmospheric inspection. For the underwater inspection only the reports listed below are held by McAlpine or ODE.

D5237 Specification for Inspection of Frigg MCP-01 Structure, Doris, March 1979.

5235 Comments about the reports of the Inspection performed 1977-78 (Appendix C to 5237) Revision 1 Feb 1979, C G Doris.

D2109 Appraisal of the 1982-85 Inspection Programme ODE.

86/230 Appraisal of the 1982-85 Inspection Programme ODE.

Atmospheric Inspection, Year Book 1984, McAlpine Sea Services
Atmospheric Inspection, Year Book 1985, McAlpine Sea Services
Atmospheric Inspection, Year Book 1986, McAlpine Sea Services
Atmospheric Inspection, Year Book 1987, McAlpine Sea Services
Atmospheric Inspection, Year Book 1988, McAlpine Sea Services
Atmospheric Inspection, Year Book 1989, McAlpine Sea Services
Atmospheric Inspection, Year Book 1990, McAlpine Sea Services
Atmospheric Inspection, Year Book 1991, McAlpine Sea Services

DS1 - DS406 Primary Concrete Structure Anomaly Locations, Drawings

MCP-01 Underwater Inspection October/November 1991, McAlpine Sea Services, 16 December 1991.

MCP-01 Underwater Inspection Jan-Feb 1992, McAlpine Sea Services, 18 March 1992.

Durability Assessment of Concrete Samples from MCP-01 Platform.

Contents list of Inspection Reports for Atmospheric Inspection of Primary Concrete Structure.

The extent of observed scouring up to 1986 is summarised in the following report:

87-281 MCP-01 Foundation Study, Study No. 86/8, Offshore Design Engineering, April 1987.

Reports on the repair carried out and subsequent inspection of the foundation have not been seen. These are required to determine what account should be taken of scour in sensitivity studies.

Recent reports on the extent of marine growth have not been seen and are required for estimation of weight and wave loading.

3.0 SOILS DATA

3.1 SOIL DATA AVAILABLE

3.1.1 Original Site Investigations

Two separate soil investigations were performed for MCP-01. The first in 1972 consisted of three boreholes, five cone penetration tests and laboratory testing. The second in 1974 consisted of seven boreholes, five cone penetration tests and laboratory testing. These are reported in NGI reports 72 007-4 and 74 015-3 respectively (Appendix A).

The 1972 site investigations and some of the 1974 investigations lie outside the foundation of the platform as installed (Figure 3.1). Boreholes and cone penetration tests are summarised in Table .31.

TABLE 3.1 SUMMARY OF BOREHOLES AND CPTs

Number	Date	Depth (m)	Comment
Boreholes			
B1A	1972	70.50	Outside foundation
B1B	1972	17.29	Outside foundation
B3 East	1974	29.89	Outside foundation
B3 Centre (1)	1974	16.29	Within foundation
B3 25m West	1974	30.00	Within foundation
B3 14m West	1974	8.09	Within foundation
B3 West	1974	15.29	Within foundation
B3 Centre (2)	1974	4.29	Within foundation
B3 North	1974	30.79	Outside foundation
CPTs			
CPT 8	1972	2.69	Outside foundation
CPT 9	1972	9.69	Outside foundation
CPT 10	1972	8.00	Outside foundation
CPT 11	1972	6.50	Outside foundation
B3 Centre	1974	13.09	Within foundation
B3 South	1974	18.59	Within foundation
B3 West	1974	13.00	Within foundation
B3 East	1974	19.19	Outside foundation
B3 North	1974	19.00	Outside foundation
B3 15m North	1972	12.50	Within foundation
B3 15m South	1974	18.39	Within foundation
B3 25m East	1974	10.69	Within foundation
B3 25m West	1974	11.39	Within foundation
B3 14m West	1974	17.39	Within foundation

The site investigations indicate that the foundation soil consists of 60m of fine to medium sand overlying a hard clay (encountered in boring B1A). The upper 2 to 3 metres are a uniform medium sand overlying a uniform fine sand with a high relative density. There are local lenses of clay and silt which do not extend over the complete foundation area.

Laboratory testing included classification tests, consolidation and simple shear tests on clay, and triaxial tests on sand.

3.1.2 Subsequent Investigation

No site investigations into soil properties have been carried out since installation of the platform.

Inspection of the platform revealed some scouring of foundation between 1982 and 1986. The inspection results are summarised in Appendix B of ODE report 87/281 (see Appendix 3A) and the extent of scour summarised in Table 3.2.

TABLE 3.2 EXTENT OF SCOUR

Year	Total area of scour (m^2)
1982	5.25
1983	None reported
1984	8.51
1985	36.0
1986	55.6

Remedial works were subsequently carried out but reports of this work have not been seen nor reports of subsequent inspection to establish the effectiveness of the remedial works.

3.2 INTERPRETATION OF DATA

Parameters for the original design were derived from the site investigation data by Doris (Report MP2 D1055). Subsequent stability analyses were carried out to take account of the effects of scour and marine growth (see Appendix 3A) and these studies reconsidered the soil parameters. The original design assumed an angle of friction of 36° based on the triaxial tests carried out. Later studies suggest that, if anything, this is conservative and that a higher angle of friction would be expected for plane strain conditions which are more appropriate to the foundation analyses.

3.3 SOIL PARAMETERS FOR RE-ANALYSIS

3.3.1 Proposed Method of Determination

There are three options for determination of soil parameters:

i. Use values from original design;

ii. Derive new values from original field data;

iii. Derive new values from new field data.

It is considered that the original soil data from the 1974 site investigation should be used as the basis for determining soil parameters. It is proposed that these data are reviewed and new soil parameters determined taking account of the work carried out in 1987 by NGI. It is not considered necessary to carry out any field work. These conclusions are discussed further below.

3.3.2 Original Design Values

The soil parameters used for original foundation design were (D1055):

-	angle of friction	36°	
-	submerged unit weight	1.05 t/m^3	
-	water content	20%	
-	angle of friction between sand and concrete	31°	
-	Young's Modulus	9000 t/m^2	static
		15000 t/m^2	dynamic

There are a number of reasons why these data may not be appropriate for re-analysis.

3.3.2.1 Variation of soil conditions.

The soil conditions vary with depth, being divided into 4 layers above the clay layer at 60m. However, the variation is not great and apart from the top 2 to 3m conditions are very uniform in the top 25m. Variations across the foundation are very minor. The effect of these small variations on structural performance is not considered likely to be significant.

3.3.2.2 Appropriateness of Friction Angle

The value of 36° is based on undrained triaxial tests. For the expected relative density of the sand this may be an underestimate. Plane strain conditions are more appropriate to behaviour and would have a higher angle of friction. The value used in the original design is therefore considered to be conservative.

3.3.2.3 Lack of Field Data

The only permeability test was carried out on a grab sample in 1982. No data was presented on Young's Modulus; the values used were based on analytic expressions.

3.3.2.4 Changes due to Consolidation

Consolidation of the foundation soils during the 20 year life may have changed the soil parameters from the original parameters. As there is very little clay in the upper layers of the soil this will not be a major effect and will have little effect on structural behaviour. Changes in soil parameters could be determined either by modelling of consolidation or by obtaining fresh field data. The expense of either of these is not justified as the effect will be so small and it is proposed to use original soil data.

3.3.3 Recalculation of Soil Parameters

Recalculation of soil parameters based on original field data would allow the most appropriate values for angle of friction, Young's Modulus and other parameters to be determined taking account of current knowledge. An assessment of soil parameters was carried out by NGI in 1987 (Report 87 314-1) including comparison with similar soils at other sites and it is suggested that this report is taken account of. It is considered that derivation of parameters from available site data and by comparison with data from other sites will give sufficient accuracy for the purpose of re-analysis and will improve confidence in values compared to original design.

3.3.4 Obtaining New Field Data

Obtaining new field data would allow changes in parameters due to consolidation to be assessed and would give better values for permeability and Young's Modulus for which there is no useful original data. However, the expense of obtaining these data is not considered to be justified as values adequate for re-analysis can be derived from existing data.

3.4 DATA AND PROCEDURE FOR DETERMINING SOIL PARAMETERS

3.4.1 Data To Be Used

The report on the 1974 site investigation (NGI 74015-3) should be used for obtaining basic data. This report includes some information from the 1972 site investigation.

Data from the following boreholes and cpts which lie within the foundation area should be used:

Boreholes:	B3 Centre (1) B3 25m West B3 14m West B3 West B3 Centre (2)
CPTs:	B3 Centre B3 South B3 West B3 15m North B3 15m South B3 25m East B3 25m West B3 14m West

For information below 30m depth, borehole B1A from the 1972 investigation should be used.

3.4.2 Procedure for Determination of Parameters

Soil parameters should be determined from the original data listed above and taking account of NGI report 87 314-1. Recognised current methods of calculation should be adopted.

3.5 METHOD OF MODELLING SOIL

3.5.1 Requirements

The representation of the soil in the structural model must correctly model the support conditions experienced by the structure in terms of reactions and displacements.

The foundation comprises a circular base slab of 101m diameter with a central section raised by 500mm.

Factors which must be taken into account for MCP-01 are:

- soil stiffness (including variation with depth);
- scouring of foundation;
- degree of soil contact in central recess.

3.5.2 Soil Stiffness

The soil conditions are substantially uniform to a depth of 60m and it is considered that lenses of clay or silt will have no significant effect on stiffness.

Detailed representation of soil variations beneath the foundation is therefore not necessary to obtain a sufficiently accurate determination of soil stiffness. It is considered that the level of computation required does not justify finite element modelling of the soil.

3.5.3 Scour

The maximum amount of scour reported in 1986 was 56m^2 which is equivalent to 1% of foundation area. The maximum horizontal extent of the scour was 4.3m beneath the foundation. The areas of scour were filled with grout and stone subsequently placed around the platform (details not known at present). Subsequent inspections of the foundations should be made available to verify the effectiveness of the remedial works and to establsish whether any further scouring has taken place.

It is proposed that no scour is assumed for the analysis but that a reduction in bearing area is included as part of sensitivity studies. This reduction should reflect the greatest amount of scour which has been observed locally as the full bearing pressure may not have been developed beneath areas of scour which have been repaired.

3.5.4 Central Recess

The central recess is 500mm higher than the outer ring and the structure was designed on the basis that the recess slab is not in contact with the soil. The effect of contact between structure and soil in this area is to provide a larger bearing area and to reduce edge pressures. Under extreme loading with full contact over the central area uplift at the edge of the foundation may occur.

Contact of the central recess slab with the seabed soil might have resulted from:

- heave of soil after installation;
- settlement of structure.

Heave of soil due to foundation pressures, if it has occurred, would be greatest close to the edge of the bearing area, i.e. at perimeter of central access.

If contact has been made between the soil and the slab due to heave the contact pressures are still likely to be less than under the outer ring due to the disturbed nature of the soil.

Settlement measurements have not been taken before 1986 and therefore it is difficult to estimate what settlement has taken place. An initial settlement of 170 - 220mm and additional long term settlement of not more than 100mm was anticipated in the design (document D1055). Air gap measurements in 1986 suggest that the air gap has reduced by 225 to 355mm from the design value (Document MPI/TIR/86/03). However, level measurements in the tunnels show no appreciable differential settlement.

It seems therefore that full contact between the central recess and the soil is unlikely. It may be possible to drill through the base slab to establish whether there is contact with the soil. However, this operation would entail drilling against a 94m heat of water and would be costly, involve some risk and may not be reliable. It is not considered that the expense and risks are justified.

3.5.5 Modelling of Soil

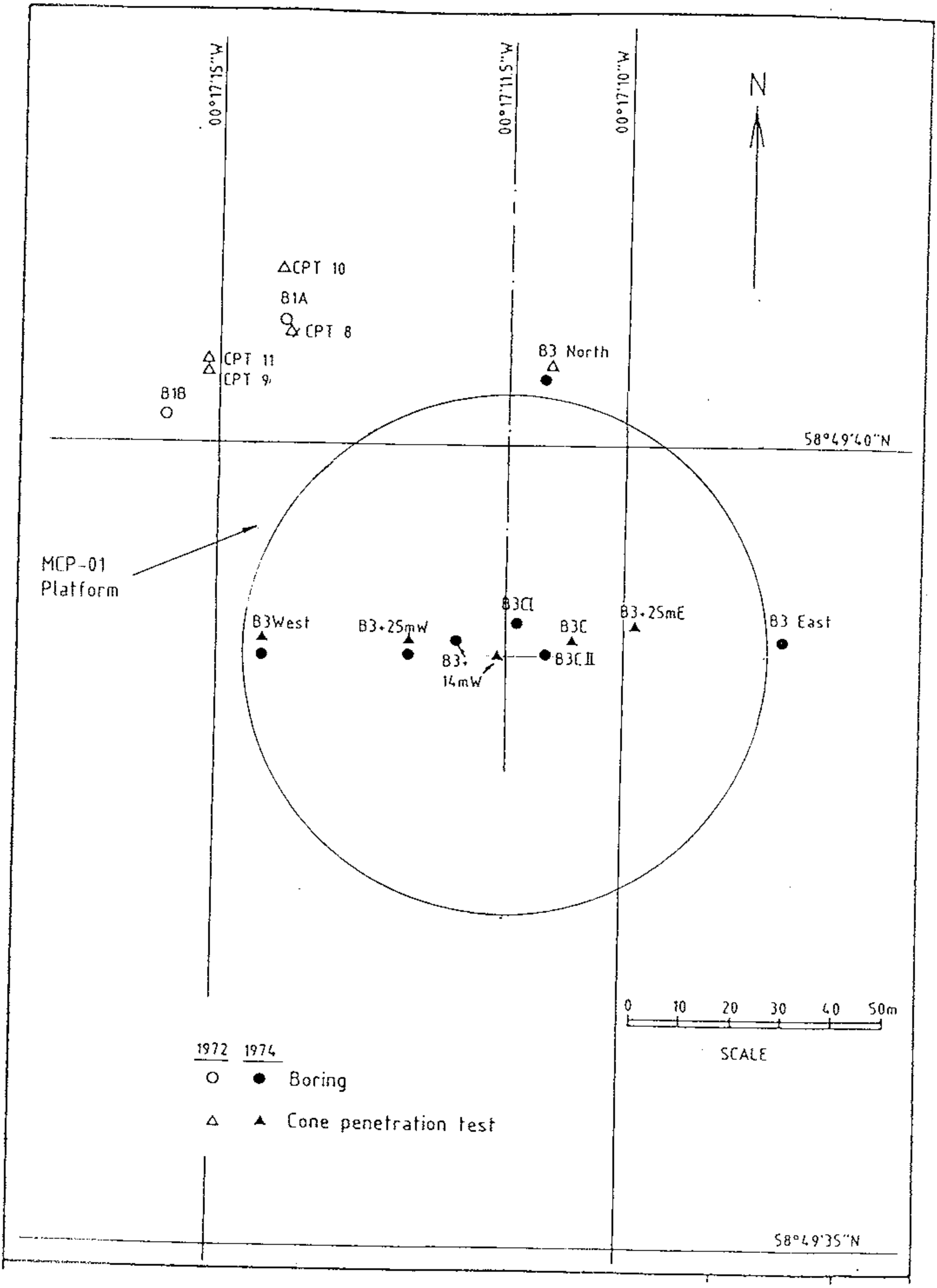
Two methods of modelling the soil support conditions are suitable:

i. spring supports at base nodes of structure with stiffness based on calculated soil stiffness.

ii. Pressure loading on base of model with the pressure distribution required to balance the applied loads calculated for each load case based on calculation for each load cure based on calculation of soil behaviour.

The second method is considered a better approximation to real soil behaviour as it can take into account non linear behaviour of soil (i.e. limiting values of strength can be used in calculation), interaction between adjacent areas of soil, and uplift if it occurs (springs would have to be decoupled if pressure reduces below zero). However, it requires the calculation of balancing soil pressures for every load case which will be more time consuming. Residual reactions due to lack of complete balancing can be equilibriated by a rigid support for each degree of freedom but care must be taken to avoid artificial stress concentrations.

The pressure distribution on the base depends on the relative stiffness of the foundation. For a rigid base the pressure distribution would be uniform under vertical load whereas for a flexible base an elastic distribution resulting in higher edge pressure would be expected. It is suggested that both conditions are analysed in order to give bounds to soil/structure interaction.



BOREHOLE AND CPT LOCATIONS

FIG 3.1

4.0 ENVIRONMENTAL DATA

4.1 ORIGINAL DATA

The platform was designed originally to the following data. The confirmatory letter from Lloyds is given in Appendix 4A.

i. Storm wave: 100 year occurrence
X = 29 metres
T = 16 seconds

ii. Operating wave: One month occurrence, on which serviceability limit states were checked
H = 18 metres
T = 12.5 seconds

iii. Current: Surface 1.5m/s
Bottom 0.25m/s

The variation was assumed linear between the surface and the bottom.

iv. Wind: 36m/s (10m above surface)
One minutes gust 53m/s (10m above surface)

v. Fatigue: 20 year duration total no. of waves 0.8×10^8
Using a cumulative probability of a linear curve between wave height and log number of cycles.

vi. Temperature effects as per DTI rules previously issued for the steel platform.

4.2 DATA FOR RE-ANALYSIS

Operators are generally obliged by the DEn to record environmental data for their installation. However, for MCP-01 recordings have only recently been restarted and would therefore be insufficient to provide statistically correct information.

The alternatives are therefore to use:

- Original Data;
- Data from the Guidance Notes;
- Purchase up to date site specific Data.

The Original Data is expected to be conservative (wave heights have been shown to be lower) and although the Guidance Notes could be used, Lloyds prefer that site specific data is used. Total have decided to purchase data from MAREX Ltd based on records from either the Buchan, Brae or Forties fields and it is proposed to use this data.

The return period for extreme conditions should be 100 years. 50 year values could be used but these would require more detailed assessment by Lloyds to ensure adequate safety levels for a 20 year life.

MAREX Ltd will supply data for waves, wind and current. Temperature and snow/ice data would be taken from the Guidance Notes.

It has been recommended that Total buy all the data listed in the MAREX proposal No. P3883 dated 29 June 1992 (Appendix 4B). It was considered that although not all the data may be used it might be useful for future work and that the minimal additional cost was therefore worthwhile.

5.0 LOADING.

5.1 SUB STRUCTURE

5.1.1 Concrete Structure

Quantities and weights for the structure and sand ballast are given in various documents (Section 5.1.6). There appear to be discrepancies of a few thousand tonnes between the documents in the quoted or implied total dry weight of the structure. It is difficult to reconcile the various weights and quantities for the following reasons:

- comparison of dry weight and apparent weight requires calculation of submerged volume of concrete

- it is not clear whether quoted concrete volumes are gross or net (this could result in a dry weight difference of up to 3,500t)

- the division of quantities is different in different documents due to the sequence of construction.

The sequence of construction was as follows:

1. Construction in dry dock to +15m +18m
2. Construction afloat to +105m (top of breakwater wall) and shaft to +127m.
3. Erection of columns and precast deck beams.
4. Tow to field.
5. Erection of precast sections of shaft (+127m to +147m) and manifold deck.

The concrete volumes and steel weights given in D5237 are different from the original specification (D1022) and therefore it is concluded that the quantities as built differ from those originally specified.

The revised construction procedure (D1077) gives records of construction up to +30.2m including calculation of concrete density based on measurements of draught and water density. The results are given in Table 5.1.

TABLE 5.1 MEASURED CONCRETE DENSITY

Construction Level	Plain Concrete t/m^3	Reinforced Concrete t/m^3
+15m	2.445	2.594
+15m/+18m	2.450	2.609
+27.5m	2.455	2.604

These values agree well with values between 2.43 and 2.45 measured on three cores cut from the breakwater wall in 1988.

A concrete density of 2.45 t/m³ and density of reinforced concrete of 2.60 would appear to be reasonable values to assume. A check on weights at the end of stage 2 (see above) gives good agreement between D5237 and D1077 using this density (Appendix 5A). There is a discrepancy between the weight of concrete above this level as assumed for towing calculations in D1077 and as quoted in D5327 (Appendix 5A). This discrepancy appears to relate to concrete in the columns and the other concrete volumes. The assumed densities appear to be in agreement.

5.1.2 Sand Ballast

Some sand ballast was added before installation and capped with a concrete slab (0.5m thick). The rest of the sand ballast was added after installation. Quoted densities are:

- wet sand	1.60 t/m³	(D1077)
- saturated sand	1.90 t/m³	(D1077)
- submerged sand	0.88 t/m³	(D1161)
- concrete slab	2.38 t/m³	(D1077)

The unsubmerged weight of sand and concrete slab placed before installation is quoted as 53,623t (D1077) and the submerged weight as 30,000t (D5237). These figures do not tally with the densities above and it is recommended that a check on the volume is carried out by reference to the levels quoted in D1077.

The volume of sand placed offshore was 91,047m³. This has been established by measurement of levels on top of ballast and allows for 10% loss through breakwater wall from the amount pumped. This is a submerged weight of 80,120t (6,000t less than the figure quoted in D5237).

5.1.3 Total Initial Weight of Structure

The apparent weight of the structure on the seabed after installation but before sand ballasting was 118,692t (D1161). This agrees with 119,000t implied by D5327.

Additional weight added (excluding topsides) consists of sand ballast, top of shaft, manifold decks and external riser giving a total initial weight of 201,400t as shown in Table 5.2.

TABLE 5.2 INITIAL WEIGHT OF STRUCTURE

Item	Weight
Structure as placed	118,700
Sand Ballast	80,100
Top of Shaft	1,000
Manifold Deck	1,300
External Riser	300
TOTAL	**201,400**

5.1.4 Marine Growth and Debris

Inspection reports have recorded the presence of debris within the raft. This consists mainly of scaffolding and the weight is considered to be very small and of no significance in assessing structural performance. It is proposed to ignore debris in the analysis.

There has been considerable marine growth on the structure and this weight should be included. Growth of about 100mm and 100% coverage down to +68.0m level was recorded up to 1985. More recent information is not available yet.

Marine growth within holes in the Jarlan wall will also affect wave loading.

5.1.5 Weights for Analysis

The following data is proposed for analysis:

Densities	t/m^3
Concrete density	
- unreinforced	2.45
- reinforced	2.60
Sea water density	1.025
Sand Ballast submerged density	0.88
Concrete slabs over sand, density	2.38

Weights

External Riser submerged weight	260t
Sand Ballast placed after installation	80,100t
Total structure weight including sand ballast and external riser, but excluding marine growth	201,400t

Marine Growth

Extent and type to be taken from latest reports on marine growth and typical densities to be taken from published literature.

However, it is recommended that the volume of sand placed before installation is checked and that total computed weight is checked against the total of 201,400t and adjustments made if necessary.

5.1.6 References

D1022 Technical Specification for the Fabrication and Installation of A Concrete Manifold Platform, Section 4 List of Quantities, Doris.

D1077 Revised Construction Procedure, Doris, December 1975.

D1161 Sand-Ballasting Report of MCP-01 Platform, Doris, 26 August 1976.

D5237 Specification for Inspection of Frigg MCP-01 Structure, Section 1.2 Description of the Structure, Doris, March 1979.

Unnumbered Durability Assessment of Concrete Samples from MCP-01 Platform, McAlpine Sea Services, 5 December 1989.

5.2 TOPSIDES LOADING (WEIGHT)

5.2.1 General

The finite element model of the concrete deck is required to be of sufficient detail to allow the main deck beams and the columns to be checked for structural capacity and serviceability conditions. To this end, beam elements will be used to represent the columns and the main concrete deck beams. Additionally the concrete manifold support beams and columns will be modelled.

The load repartition structure will have to be studied to determine the distribution of loading between support points. An FE model of the LRS could either be included in the overall deck model or form a separate loading study.

The steel beams, deck plate and grating that span between the concrete beams should not be modelled. Where the deck plates form shear panels acting as horizontal bracing additional dummy members may be used to represent this effect.

The checking of the capacity of structural members will be carried out using limit state code requirements in BS 8110. Consideration of the load factors that are suitable leads to the following conclusion. The dead weight of the equipment is to be treated as dead load, as are the weights of the concrete and steel structures. The superimposed loads and the vessel contents loads are to be considered as live loads. Where a breakdown between dead and live loads is not given (vent stack and cranes) a single 'combined dead/live load' load factor will be used.

More data than is currently available at ODE will be required to adequately define and confirm the topsides loadings. If additional data is not available then an offshore survey will be necessary. Some of the required data is of a specific nature relating to single skids - the cranes, or the Occidental riser module. The other major piece of information that is missing is up to date equipment plot plans and elevations for the topsides. These are required to ensure that no major equipment items are included or omitted in error.

Four sources of data relating to the topsides loadings are available although none are independent or complete sets of data. However, comparison between them allows an assessment as to which areas have well defined loadings and which areas need more data.

The four available sources are:

- original design calculations
- original loading drawings AI.MP.2004 series.
- ODE loading drawings for compression project MP-5009-M4-15 series.
- Total Marine weight control report

Comparison of loadings from these sources (where available) leads to the following conclusion that the loading data shown on ODE drawings No. MP-5009-M4-15-01 to 04 are reasonably correct. Loading due to individual items is discussed below.

5.2.2 Manifold Loadings

The manifold loadings should be taken from MP-5009-M4-15-04 (+133.0m level). These loadings should be applied to the FE model of the concrete manifold support structure. The self weight and layout of the manifold support structure can be calculated from the original design drawings AI-MP-2OU-7001 to 7011. Details of the reinforcement and prestressing can be taken from AI-MP-2QU-7201 to 7214. Modelling of the concrete manifold support structure is required to take proper account of main deck beam flexibility and to calculate the loadings induced in the central tower.

5.2.3 Load Repartition Structure

The overall loadings given for the LRS at level 123m on MP-5009-M4-15-01 define loading at four support points, neglecting one of the supports on the central column. In order to properly calculate the couple in the central column and to determine the division of load between upper and lower connections a beam element model of the LRS will be required.

The beam element model of the load repartition structure could be treated as a small stand alone model with spring supports to model the deck beam flexibility.

Alternatively the LRS model could be incorporated with the overall deck model. The LRS model will be relatively small and will not add greatly to the overall finite element model size.

The compressor module loads (with the lifting frame weight and live load) shown on drawing MP-5009-M4-15-04 are similar to those in the weight control report and design calculations. Additional loadings for UDL live load, piping, escape tunnel, stairways etc. must be taken from the Structural Design Report MP-5009-M4-CL-31 Volume 2 Part 1 and the weight control list. The LRS geometry is defined on drawings MP-5009-M4-15-11 to 15, 40, 41, 60 and 70 and in the drawings MP-5035 series (which ODE do not have).

5.2.4 Occidental Riser Skid

There is a difference between the weight report and loading drawings MP-5009-M4-15-01. The weight report gives operating weight = 110t, the loading drawing gives dead load = 148t and live load 310t. No design data books are currently available. More data is required or an offshore assessment will have to be carried out. If no further data, the deck beams should be checked for both loadings. This difference is not likely to have a significant affect on other structural members.

5.2.5 Other Packages

The following packages on the main decks have similar loadings given in the weight control and loading drawings. Loads should be taken from the loading drawing as details of load distribution between packages is given. These packages are:

Quarters Package
MCC Generator Package
Utilities Package
Valve Manifold Skid
North and South Firewater Pump Houses
Fuel Gas Treatment Skid
Vent Stack
Cranes

The operating load reported in the weight control report is less than the drawing live load as it does not include blanket superimposed loads.

5.2.6 Underdeck Loadings

The underdeck loadings shown on drawing MP-5009-M4-15-03 should be used. The loads should be applied as uniform distributed loading along the main deck beams that support the secondary steelwork.

5.2.7 Blanket Loadings

The dead weight of the grating/stringer/secondary deck beams should be applied as a uniform distributed load on the appropriate concrete deck beams.

The live loadings shown on loading drawings should also be applied as uniform distributed loads on the appropriate concrete deck beams.

An additional blanket dead load is specified between gridlines D and E at level 123.0m to allow for the pilot gas skid, the utilities gas skid and the blowdown control skid.

5.2.8 Walkway, Muster Area and Laydown Area Loads

The designated walkways, lifeboat muster areas and laydown areas may be identified from platform plot plans and the finite element model loaded accordingly. The loadings from lifeboats and liferafts should be modelled.

5.2.9 Limitation of Total Applied Superimposed Load

The live load applied to the platform using the loading philosophy defined above will clearly be far greater than the actual load on the platform. However, the loading on each individual member is of the correct magnitude for design.

It is not proposed to place a global limit on the total applied live load on the platform for the following reasons:

- The deck is supported on 15 columns and it is reasonable to assume that any one of these might have to carry the maximum live load.

- The 15 columns are evenly distributed around the substructure perimeter and will not cause an unrealistic concentration of loading in the concrete substructure.

- The critical foundation condition is probably the minimum vertical loading which is covered by a standard load combination (load factor of zero on live load).

Therefore it is not unrealistically conservative or optimistic to use full superimposed loadings.

5.2.10 Further Work to Define Deckloads

The main uncertainty is whether or not each piece of equipment has been identified or if additional packages have been added to the topsides. A specific area that is poorly defined is between gridlines D and E at the main deck level.

The weight of the Occidental skid cannot be identified from the currently available documentation.

It is recommended that Total provide equipment layout drawings marked up with current status and the dry and operating weights for the Occidental skid.

5.3 WIND LOADING

The wind loadings shown on the loading drawings have been calculated using CP3 Chapter V Part 2 for the wind pressure and DnV Appendix B for the wind force. Design wind speed is shown as 53m/sec at 10m above sea level for a 3 second gust. However, the wind loadings have been calculated for the design of the individual modules. The wind loading for the overall re-analysis of the platform should be recalculated to reflect the following:

- New wind speed criteria from MAREX corrected for elevation.

- Gust duration that is applicable to the overall platform size.

- Shielding of some modules (e.g. utilities and quarters modules are so close that they will act as one).

To allow the calculation of the wind loading the equipment plot plans and elevations should be made available to the re-analysis contractor.

Wind loading will need to be assessed from four cardinal directions to ensure that maximum loadings are applied to the deck grillage.

5.4 WAVE/CURRENT LOADING

The environmental data to be used for the hydrodynamic loading regimes will be taken from the new data prepared by MAREX for the 100 year extreme and the 1 month operating condition environmental conditions.

The wave loading on the MCP-01 platform will be highly non-linear with wave height due to the complexity of the structure. The major non-linearity being when the wave passes over the top of the Jarlan breakwater wall and acts directly on the deck columns and the centre shaft. The hydrodynamics around the platform in a wave field are highly complex and a suite of wave loading programs will be needed that have the following main features:

- The platform may be considered as being axisymmetric. (Although this will complicate transferring the loading onto the FE model.)

- The wave field on the outside of the breakwater may be described by the superimposition of an incident potential and a partially diffracted potential. Airy wave theory will be used to describe the incident wave potential. The net potential can be written as a series expansion in a similar form to that developed by McCamy and Fuchs. Each term contains a reflection coefficient, the net potential field may be calculated by calculation of the coefficients and backsubstitution into the series expansion.

- The wave field inside the Jarlan breakwater wall is also described as a series expansion defining the potential field.

- The potentials on the inside and outside of the breakwater wall are related by a pressure loss equation which takes into account the Jarlan hole diameter, the wall thickness and the overall porosity of the wall and waves passing over the breakwater wall.

- The wave loading on the columns and central shaft above breakwater level are calculated using a more sophisticated theory (such as Stokes 5th order) to better represent the true wave surface profile.

- The hydrodynamic forces, or mean hydrodynamic pressure may then be calculated by applying the mass flow theorem to two control surfaces located in the position of the breakwater wall.

It is not expected that the wave loading program will provide details of the local pressure variations around the Jarlan holes. The program is, however, required to give a good estimate of the overall forces and moments applied to the concrete platform, and the mean local pressures that generate these global loadings.

The local pressure distribution will be used as input loading to the global finite element analysis.

Due to the complexity and non-linearity of the wave loading regime it is required that the computer program used for the calculations of wave loads will be calibrated to the original model tests of the MCP-01 structure.

As the structure is being considered as axisymmetric the wave loading need only be calculated from one direction. Full calculation of the operating and extreme wave conditions will be required. Each condition will have to be analysed for a series of time steps through the structure to calculate the maximum pressure loading and maximum overall loads.

Due to the size of the finite element model it will be necessary to develop an automatic loading routine for transferring calculated wave load pressures onto the finite element model. It may also be required to be able to preview the wave pressure distribution and overall wave loadings to allow selection of critical load cases.

Directionality of wave loading should also be considered.

5.5 ACCIDENT LOAD CASES

Accident load cases will be developed from the platform safety case. However, the MCP-01 platform will be operating as an unmanned facility and hence will not be subjected to the same accident scenarios as a fully manned platform. The main design requirements is that an accident does not lead to progressive collapse of the structure. Possible design accidents include:

- ship impact on Jarlan breakwater wall and the possible consequence to deck support column
- blast within compression area
- blast within central shaft/tunnels
- fire loading
- dropped object from crane operations

5.5.1 Ship Impact

It is suggested that the ship impact criteria given in the safety case documentation should be used. However, these have not been seen and should be reviewed and agreed with Lloyds. Damage to the platform and vessel are inevitable but the integrity of the platform must not be compromised. This could be a problem if a column was made ineffective as many of the deckbeams are not continuous and hence have little redundancy. The range of possible impact levels needs consideration but would be in the range LAT-10m to HAT+12m.

5.5.2 Blast Loading

Blast loading is unlikely to cause a major problem for the concrete structure. A major explosion is most likely to occur in the compression/separation modules. These are positioned on top of the LRS in an open exposed location. A blast in this area would not result in damage to either the substructure or main concrete deck beams.

An explosion within the central shaft or the radial tunnels could occur following hydrocarbon build up. The central shaft and the tunnels are heavily precompressed by hydrostatic pressure and damage is likely to result in the loss of watertightness and flooding of the tunnels and central shaft.

It is not proposed to include blast loading in the FE analysis. However, an assessment of the effects taking account of the stoichiometric ratio and degree of venting at possible blast locations may be necessary.

5.5.3 Fire Loading

As the hydrocarbon product is gaseous, only jet type fires may be expected and these are by their nature quite localised in effect. It is therefore not proposed to include fire loading in the FE analysis but, the safety case should be reviewed to confirm that there is no significant fire loading likely.

5.5.4 Dropped Objects

There is a possibility of items being dropped from the cranes . The size and nature of the dropped object should be taken from the safety case documents and agreed with Lloyds. Vulnerable areas appear to be the radial beams at +68m and +105m, the raft outside the lobed walls and the deck beams. Loading from dropped objects will be local in effect and FE analysis would only be required if overall stability is compromised by the damage, e.g. loss of radial beam. Possible scenarios from the safety case should be reviewed.

5.6 PRESTRESS

The prestressing loads on the concrete structure may be modelled as a series of point loads and UDLs. The loading should reflect the application of prestress force at the anchorage, the losses along the length of the tendon, the forces induced by the curvature of the tendon, and the anchorage force at the other end.

The force variation along the prestressing tendons given in the original design calculations will form the basis for this loading. Prestress losses will be reassessed taking account of measured elongation at stressing where appropriate.

5.7 FATIGUE

The fatigue limit state is a design condition that is related to the repeated loadings from wind, wave, current and dynamic equipment loads. The fatigue analysis should be carried out in accordance with the DEn Guidance notes but taking account of the 20 years of operation already completed.

The calculation of the fatigue life of the prestressed concrete structure will be based upon the results of the main static analysis. The fatigue calculations will be carried out with a load factor of unity. The modular ratio, $^{ES}/_{EC}$ used in the analysis of concrete sections should be taken as 10. This value is selected to reflect the degradation of the concrete stiffness under cyclic loading.

The fatigue loading should be based on the environmental conditions given in the MAREX report and an assessment of the most onerous combination of waves and wind.

5.8 SEISMIC LOADING

Lloyds have indicated that they do not expect a seismic analysis and it has therefore been omitted from the specification. However, the HSE may require a seismic analysis and it is thus discussed below.

The platform structure consists of fairly rigid walls with a more flexible central shaft. The deck columns are very flexible by comparison and support the mass of the topsides. The modelling used for a seismic analysis would need to identify several specific loadings.

- Foundation vertical overturning and sliding forces.
- Lateral pressures on walls required to accelerate added mass of water and walls own mass.
- The deck sway loads and their distribution between the steel columns and the central shaft.
- The interaction between the central shaft and the Jarlan breakwater wall where they are connected by the radial beams, at +68m and +105m level.

5.9 DYNAMIC LOADING

The natural period of the structure is about 1 second and global dynamic effects will be very small. However, there are significant changes of stiffness between different levels of the structure notably at the deck columns and consideration will need to be given to local dynamic loading.

The foundation level forces could be predicted with reasonable accuracy by considering the platform as rigid and carrying out a 2 degree of freedom analysis for rocking and sliding and a single degree of freedom analysis for vertical motion then combining the results.

However, as the structure has a radical change in stiffness at the wall to steel column interface a rather more sophisticated analysis may be required which could also be used for seismic analysis if required. A finite element model consisting of beam elements could be used with the concrete substructure modelled as a cantilever tube (the breakwater walls) enclosing a tube (the central shaft). The central shaft would start at +15m level and be radially restrained at +68m and +105m. The columns would be modelled running from the top of the breakwater wall to the deck level where they are connected to the central shaft, by the deck beams.

The foundation spring stiffnesses and participating masses should be calculated on the basis of the soil being an elastic half space. The mass of the structure would include the structure, ballast, trapped water and added mass of water. The stiffness of each element at each cross-section would be calculated based on the structural behaviour. For instance, from the seabed to +15m the cylinder would have the properties of all the circumferential walls and the radial walls acting as a single unit. Above this level the lobate walls and central shaft are separate apart from radial ties at +68m and +105m.

A simplified analysis like this would allow the sway forces on the deck to be apportioned between the central shaft and the steel columns, the bending in the central shaft, axial forces in the radial beams, the foundation forces and displacements and acceleration of the deck to be predicted.

5.10 LOAD COMBINATIONS

5.10.1 Modelling of Loads

Assuming that the foundation loads are modelled using pressure loading, the loading will be performed in three stages: elementary, equilibrium and combination. If the foundation is modelled using springs the loading would be in two stages, elementary and combination.

The elementary loads are the permanent, functional, environmental and deformation loads described previously in Section 5.0. If a linear foundation model is used to model the foundation soil pressures then a soil pressure distribution that is in equilibrium with each of the elementary loadcases will be calculated and added to the elementary loadcase. The combined elementary load and soil pressure load cases will then be factored and combined to give the design load cases.

If a non-linear foundation model is used, it is necessary to combine the factored elementary loadcases and to then calculate an equilibrium foundation pressure for each design loadcase.

5.10.2 Load Cases

The basic load cases and expected number of loadings under each case are shown in Table 5.3. The wave and wind loadings will be for different directions and wave phases. There are a large number of possible directions and phases and some pre-selection will be necessary to reduce the number analysed.

TABLE 5.3 LOAD CASES

NO.	LOAD CASE	NUMBER OF LOADS
1	STRUCTURE WEIGHT	
1.1	Concrete	1
1.2	Equipment	1
1.3	Solid Ballast	1
1.4	Marine Growth	1
2	TOPSIDES	
2.1	Dead Load	1
2.2	Live Load	1
3	HYDROSTATIC PRESSURE	
3.1	HAT	1
3.2	LAT	1
4	PRESTRESS	1
5	DEFORMATION	2
6	WAVE AND CURRENT	
6.1	Extreme	8
6.2	Operating	4
7	WIND	
7.1	Extreme	2
7.2	Operating	2

Additional analyses will be required to identify the critical loadings on the deck beams and columns. These elements will be sensitive to the direction of the environmental loads.

5.9.3 Load Conditions and Combinations

There are six load conditions which require examination and there will be a number of combinations of the basic load cases for each condition.

- Load Condition I: Extreme environmental loads plus the associated maximum functional loads.

- Load Condition II: Extreme environmental loads plus the associated minimum functional loads.

- Load Condition III: Operational environmental loads plus the maximum associated functional loads.

- Load Condition IV: Long term wave loads both past and future for fatigue life evaluation.

- Load Condition V: Accidental loads plus post accident load conditions, in combination with the appropriate environmental conditions.

- Load Condition VI: Load combinations for the verification for serviceability criteria.

Likely load combinations are given in Tables 5.4 and 5.5 for Load Conditions I, II, III, and VI.

	Load Condition	VI	VI	VI	VI
Load Case No.	**Load Combination**	1	2	3	4
1	Structure self weight	1.0	1.0	1.0	1.0
2.1	Deck Dead	1.0	1.0	1.0	1.0
2.2	Deck Live	1.0	1.0	1.0	1.0
3.1	Hydrostatic LAT	1.0	-	1.0	-
3.2	Hydrostatic HAT	-	1.0	-	1.0
4 & 5	Prestress and Deformation	1.0	1.0	1.0	1.0
6 & 7	Wave/Wind operating	1.0	1.0	-	-
6 & 7	Wave/Wind extreme			1.0	1.0
	No. of Cases	4	4	8	8

TABLE 5.4 SERVICEABILITY LOAD COMBINATIONS (ASSUMING ENVIRONMENTAL FORCES FROM 1 DIRECTION)

	Load Condition	III	III	III	III	I	II	I	II	I	II	I	II
Load Case No.	**Load Combination**	1	3	5	7	9	10	11	12	13	14	15	16
1	Structure self weight	1.2	1.2	1.2	1.2	1.2	0.9	1.2	0.9	1.2	0.9	1.2	0.9
2.1	Deck Dead	1.2	1.2	1.2	1.2	1.2	1.0	1.2	1.0	1.2	1.0	1.2	1.0
2.2	Deck Live	1.6	1.6	1.6	1.6	1.2	-	1.2	-	1.2	-	1.2	-
3.1	Hydrostatic LAT	0.9	-	0.9	-	0.9	0.9	-	-	0.9	0.9	-	-
3.2	Hydrostatic HAT	-	1.2	-	1.2	-	-	1.2	1.2	-	-	1.2	1.2
4,5	Prestress & Deformation	1.2	1.2	0.9	0.9	1.2	1.2	1.2	1.2	0.9	0.9	0.9	0.9
6 & 7	Wave/Wind operating	1.4	1.4	1.4	1.4	-	-	-	-	-	-	-	-
6 & 7	Wave/Wind extreme	-	-	-	-	1.2	1.4	1.2	1.4	1.2	1.4	1.2	1.4
	No of cases	4	4	4	4	8	8	8	8	8	8	8	8

TABLE 5.5 ULTIMATE LOAD COMBINATIONS (ASSUMING ENVIRONMENTAL FORCES FROM 1 DIRECTION)

6.0 MATERIALS AND STRUCTURAL CONDITION

6.1 PROPERTIES OF MATERIALS

6.1.1 Concrete

The concrete specification for MCP-01 (document no. MP2D 1013) was for a concrete with a 28 day compressive strength measured on 150mm x 300mm cylinders of 400 kg/cm² (39.2 N/mm²). The concrete produced (see document D5237, Section 5.7) has the following properties:

- made with OPC with C_3A content between 5 and 6 percent (specification called for less than 8%).
- sand 0-8mm from natural deposit in Sweden.
- aggregate 8-16mm and 16-27mm from natural deposit in Norway.
- admixtures to improve workability and control setting time.
- cement content varying between 410 and 450 kg/m³.
- water/cement ratio varying between 0.36 and 0.43.
- in the splash zone air content 3±0.75% and cement content of 430 kg/m³.

Mean 28 day cylinder strength of all concrete (measured on 484 samples) was 536 kg/m² (52.6 N/mm²) with a coefficient of variation of 6.3%. In the splash zone mean strength was 519 kg/cm² (50.9 N/mm²). Tests on some cylinders (number not known) at 90 days showed an average increase in strength between 28 and 90 days of 1.16.

Strengths of three cores taken from the Breakwater Wall below sea level in 1988 averaged 95 N/mm². These cores also exhibited tears caused during construction which penetrate to the reinforcement. These tears have become encrusted with calcites. There are still some cores held by McAlpine Sea Services which are available for testing.

Mix proportions for the three most commonly used mixes are given in D5237.

6.1.2 Reinforcing Steel

All reinforcement was specified to conform to ASTM 615 grade 60 (D1013).

Characteristic strengths of 380 N/mm² (for diameters of 25mm and above) and 400 N/mm² (for diameters of 20mm and below) have been used in design.

6.1.3 Prestressing

Cables consisting of 12 or 24 no. 15.2mm strands (12T15 or 24T15) with minimum breaking strength of 25.4t per strand (nominal area 143mm²) were specified. This is equivalent to a stress of 178 kg/mm² (1742 N/mm²). An ultimate strength of 183.2 kg/mm² (1797 N/mm²) has been used in design.

Specified jack pressures corresponded to cable forces of 249t (12T15) or 484t (24T15), the equivalent stress in the steel is 145 kg/mm² (1422 N/mm²).

Specified pressures and elongations were allowed to vary by 5% (D1013). Measurements of elongations were compared with calculated values and variations in prestressing force calculated. Changes between actual forces and expected forces were recorded in 19 members and vary from +4.4% to -13.7% (D5237).

6.2 CRITICAL ZONES OF THE STRUCTURE

6.2.1 Introduction

Critical zones of the structure were identified in document D5237 in order to plan the inspection programme. These zones are summarised below and may be divided into four groups, zones subject to high stress during construction and/or installation, zones subject to high stresses during operation, zones subject to concentrated loading or with abrupt changes of section, zones where construction was difficult.

6.2.2 Highly Stressed Zones during Temporary Phases (Figure 6.1)

6.2.2.1 Tunnel Walls

During installation the tunnel walls were subject to bending due to external hydrostatic pressure leading to cracking and some leakage.

During operation the tunnel walls are subject to bending in the other direction due to internal pressure from sand ballast. This tended to close cracks but some leakage persisted.

6.2.2.2 Lobed Walls

During installation the lobed walls were subject to external pressure. When combined with prestress this led to high compressive forces and high local bending moments at the nodes. During operation the loading reversed and the walls are subject to internal pressure from sand ballast.

6.2.2.3 Central Shaft at +68.0m

During erection of radial and strut beams at +105.0m the central shaft acted as a cantilever supported at +68.0m subject to horizontal loads from the derrick mast used for erection.

6.2.2.4 Top of Breakwater Wall

The staylegs for the derrick mast used for beam erection (See 6.2.2.3) induced bending moment at the top of the breakwater wall.

6.2.3 Highly Stressed Zones during Operation (Figure 6.2)

6.2.3.1 Antiscour Wall

This acts as a deep beam spanning between the exterior diaphragms and foundation forces are transmitted to these supports by compression at 45°. Tensile forces are induced close to the Jarlan holes and some local cracking may occur.

6.2.3.2 Exterior Diaphragms

These transmit loads into the foundation (formed by the annulus between edge of raft and lobed wall) and the upper zone is therefore acting as a strut carrying high compressive forces.

6.2.3.3 Breakwater Wall

High concentrated forces are applied at column pedestals and the zones below the pedestals may be subject to high stress concentrations around Jarlan holes.

6.2.3.4 Radial Beams at +105.0m

Additional prestressing was applied to these beams to cater for possible additional central loading. In the absence of this loading some tension occurs in the lower fibre at the connection with the breakwater wall. This is exacerbated by differential vertical movement between the central shaft and breakwater wall caused by temperature differences.

6.2.3.5 Deck Columns

These are critical to the stability of the deck and are subject to high compression and also to bending from horizontal deck loads and wave action.

6.2.3.6 Deck Beams

Some of these beams are highly loaded and some were cast prior to final design. Although the design was approved, the Certifying Authority was of the opinion that some beams were acting to the limit of their capacity in shear and torsion.

6.2.4 Concentrated Loads and Changes of Section (Figure 6.3)

The following zones are subject to concentrated loads:

- pipe anchorages in tunnels
- riser anchorages in central shaft at +119.0m
- connection of radial beams with central shaft at +105.0m
- connection of deck beams with central shaft at +123.0m
- fixation block of radio mast at top of central shaft (+147.5m)

The following zone has an abrupt change of section:

- lobed wall below +65.0m

6.2.5 Zones of Difficult Construction (Figure 6.4)

The following construction joints were cast under difficult conditions and/or are subject to differential hydrostatic pressure:

a. 6m Joint

The horizontal joint at +6.0m in the central shaft, just above tunnel entrances. The tunnel entrances were cast after the shaft above had been slipformed. This joint and the vertical joints in the tunnel walls are subject to differential hydrostatic pressure. A number of horizontal and vertical cables pass through this zone.

b. Lobed Wall, Ring Belt at +68.0

The transition between the 550mm thick wall up to +65.0 and the 1200mm wall above +68.5 was cast after slipforming of the rest of the wall.

c. Strut/Node Joint at +65/+68m

Couplers were embedded in the node during slipforming and bars threaded into them before casting the struts. Due to the high level of reinforcement and prestress some couplers were omitted.

d. Connection between the radial beams and central shaft at +105.0m.

This connection was cast in situ between the precast beams and the shaft in which openings had been left while slipforming.

e. Joint in Shaft at 127.6m

An in situ joint was cast between the previously slipformed shaft and prefabricated units of shaft above.

f. Main Deck Joints

Large number of couplers were cast into these joints making construction difficult particularly E6, F6, D4 North, D7 North, G4, G7, I4, I7.

g. Seal Caissons in Tunnels

These are of particular importance because of the riser pipes passing through them and the construction joint between the caissons and the tunnels are subject to differential pressure.

h. Joint in Central Slab

The joint surrounding the central hexagonal pour of the slab is 15.6m from the axis of the shaft and is subject to differential pressures.

i. Joints in Central Shaft

Joints were made at +18.0, +27.5, +68.5, +79.45.

j Prestressing Cables subject to pressure difference between Anchorages

The following cables have one anchorage subject to water pressure and the other in atmospheric conditions.

- horizontal tunnel cables
- lower horizontal cables of interior walls
- interior diaphragm lower horizontal cables
- central shaft vertical cables

k. Prestressing cables anchored in Splash Zone

- horizontal cables in breakwater wall, +80m to +105m
- horizontal cables in radial beams anchored in outer face of node (+101/+105)
- horizontal cables in strut beams anchored in sides of nodes (+105)
- vertical cables anchored in top of breakwater wall

l. Deck Beam Cables

All anchorages are located in a salty atmosphere.

6.3 AREAS OF DETERIORATION OF STRUCTURE

6.3.1 Inspection of Structure

Annual inspections of the structure have been carried out since 1977. These have consisted of an atmospheric inspection and underwater inspections by both divers and ROVs. Only part of the structure is inspected each year, but the parts varied under a 4 year programme to give coverage of all critical zones. The inspections have concentrated on critical zones of the structure and those where defects have been found. The standards of inspection varies between Class 3 and Class 1 with the more detailed Class 1 being concentrated on areas with known or suspected defects. Many of the defects reported are of no structural significance. Those of significance for the strength or durability or the structure have been highlighted in appraisal reports.

Apart from the tunnels (see below), the appraisal of the 1978-1981 inspection programme (D2109) concluded that there were no significant defects in zones which had been subject to high stresses during construction and installation (i.e. lobed walls, central shaft at +68m, top of breakwater wall below derrick staylegs). These areas were therefore not subject to further particular inspection.

Potential mapping of the structure was carried out in 1986 but was not repeated other than locally. It is difficult to estimate the likelihood of corrosion having taken place on the basis of one year's mapping.

There is no positive evidence of corrosion except at the tunnel entrances. However, it is possible that some corrosion of prestressing cables may have taken place where cachet-ages are known to have leaked and it is possible that some corrosion may take place in the future.

Areas of deterioration are discussed below and the structure has been divided into three sections for this purpose. The foundations have been discussed in Section 3.0 and are not considered further here.

6.3.2 Structure up to +68.0

There are four areas where defects of significance have been reported, the exterior diaphragm walls, tunnel caisson walls, tunnel walls and bottom of central shaft.

6.3.2.1 Exterior Diaphragm Walls

Diagonal cracking has been identified at the top of most walls mainly between +18 and +23.5m. These are believed to be due to the distribution of prestressing forces and stresses arising from structure settlement. No significant deterioration has been observed.

6.3.2.2 Tunnel Caisson Walls

Leakage of the construction joint has been reported and some cracking. Repairs have been carried out and have generally been effective in controlling leakage. No growth of cracking has been reported recently and the cracks are believed to date from construction.

6.3.2.3 Tunnel Walls

Cracks have been identified following the lines of prestress and have developed, and leakage of cracks has been reported. Cachetages to cables in the tunnel walls and interior walls have also shown leakage and incrustation. Repairs to cracks have been carried out and cachetages have been injected. The situation is not considered to be of great structural concern but slow deterioration has been observed. These are concerns for long term durability but no signs of steel corrosion have been observed.

6.3.2.4 Bottom of Central Shaft

There has been leakage through the 6m joint above tunnel entrances and leakage through longitudinal cracks at the tunnel entrance. Water penetration into the 6m joint has been confirmed and injection to seal the 6m joint has been carried out and this joint is now drier that the tunnel entrances. Some slight leakage appears to be occurring at the 18m joint.

There has also been radial cracking with leakage of the central shaft floor, and leakage around the central pulley block.

6.3.3 Structure from +68.0 to +105.0

No significant defects have been reported here.

6.3.4 Structure above +105.0m

Two areas of concern have been noted, column bases and deck beams.

6.3.4.1 Column Bases

Some cracking of the grout annuli has been reported which has raised concern about possible corrosion of the J-bolts. An examination of one J-bolt showed no corrosion had taken place.

6.3.4.2 Deck Beams

Extensive cracking of beams 4 G-I, 4 A-D, 7 A-D and 7 G-H have been reported with through cracking of 4 G-H and 7 G-H. Cracking of other beams has also been noted. The cracking has been attributed to prestressing forces rather than operational stresses in most cases, although some cracking has been attributed to addition/ removal of topside loads. The main concern is of possible steel corrosion. Repairs have apparently been carried out although documentation is not available. No changes to crack patterns have been observed in recent years. Some cachetages have been resealed.

6.4 PROPERTIES TO BE USED FOR ANALYSIS

6.4.1 Material Properties

6.4.1.1 Concrete

No values of modulus of elasticity or Poisson's ratio for the concrete used in construction are available. Therefore values appropriate to the mixes used (D5237) and the concrete strength will have to be derived. For this purpose it is proposed that a cylinder strength of 643 kg/cm² (equivalent cube strength 74 N/mm²) is assumed. This corresponds to the average measure 28 day cylinder strength with an age factor of 1.2. For strength calculations a conservative value of characteristic cylinder strength of 480 kg/cm² (cube strength 58 N/mm²) is proposed. This corresponds to the design strength times an age factor of 1.2. For the FE model a modulus for the composite material (concrete and rebar) will have to be derived.

6.4.1.2 Reinforcement

A modulus of elasticity of 2.1×10^5 N/mm² may be used for reinforcing steel.

A characteristic strength corresponding to ASTM 615 grade 60 is proposed.

6.4.1.3 Prestressing Steel

The following values were used for original design and are assumed to be appropriate for re-analysis:

- Modulus of elasticity — 1.91×10^5 N/mm²
- Breaking stress — 1790 N/mm²
- Relaxation — 7%
- Friction — 0.18 (horizontal cables)
0.14 (vertical cables)
- Wobble factor — 0.002 (horizontal 12T15 cables)
0.001 (horizontal 25T15 cables)
0 (vertical cables)

6.4.2 Modelling of Prestressing

Prestressing cables are scheduled on the drawings. A jacking force of 249t (12T15 cables) or 484t (24T15 cables) was applied. This corresponds to a stress of 1420 N/mm². The actual prestress applied to the member in the long term will be reduced by losses due to anchorage slip, creep, shrinkage and relaxation and will vary along the length of the cable due to frictional losses.

Friction losses will be least for straight cables and greatest for profiled cables. Calculated losses in the original design were about 20-25%.

Members with straight cables may be modelled by applying forces at each end and profiled cables may be modelled by applying UDLs at suitable points along the length of the cable to represent the varying effect of the prestress.

The forces used should take account of prestress losses and of the known variations between calculated and actual elongations (D5237 Section 5.3). Losses can be established either by reference to the original calculations or by recalculation of losses. In either case consideration of every cable or cable group will be very time consuming. It is proposed that the original calculations for typical cables are examined and that average values for losses are then assumed taking account of the recorded differences at stressing.

The effects of the sequence of prestressing should be considered since stressing of the higher levels of the structure could have modified the prestressing force in the lower parts of the structure which had already been stressed.

6.4.3 Modelling of Areas of Deterioration

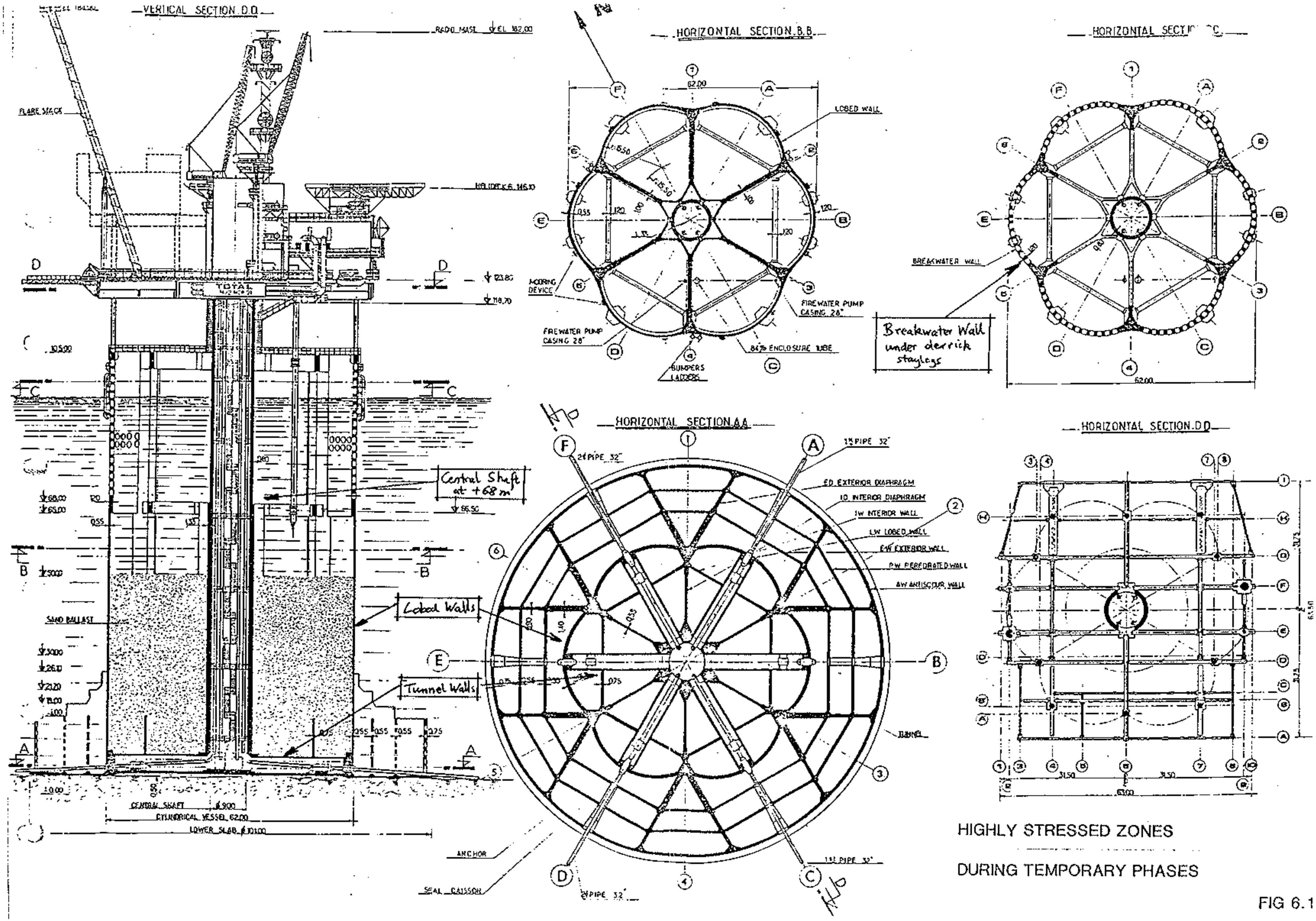
The areas of significant deterioration are:

- tunnel caisson walls
- tunnel walls, roofs and floors
- tunnel entrances
- central shaft base slab
- exterior diaphragms
- deck beams

Concrete is weak in tension and will crack under direct tension or bending tension. Provided that the concrete is adequately reinforced and that the reinforcement has not corroded structural behaviour is not impaired by cracking. Crack width is controlled mainly to provide good protection against corrosion although deep cracking could affect performance in shear.

In areas of extensive cracking or where cracking is known to be deep a reduced stiffness appropriate to a cracked section may be calculated and used in the Finite Element model. Alternatively and more accurately a reinforced concrete element could be used in the local model which takes account of cracking. Areas proposed for this approach are the most severely cracked areas of the exterior diaphragms and the most severely crack deck beams. In other areas of deterioration it is proposed that normal stiffness is used but that properties appropriate to cracked sections are used when assessing shear capacity.

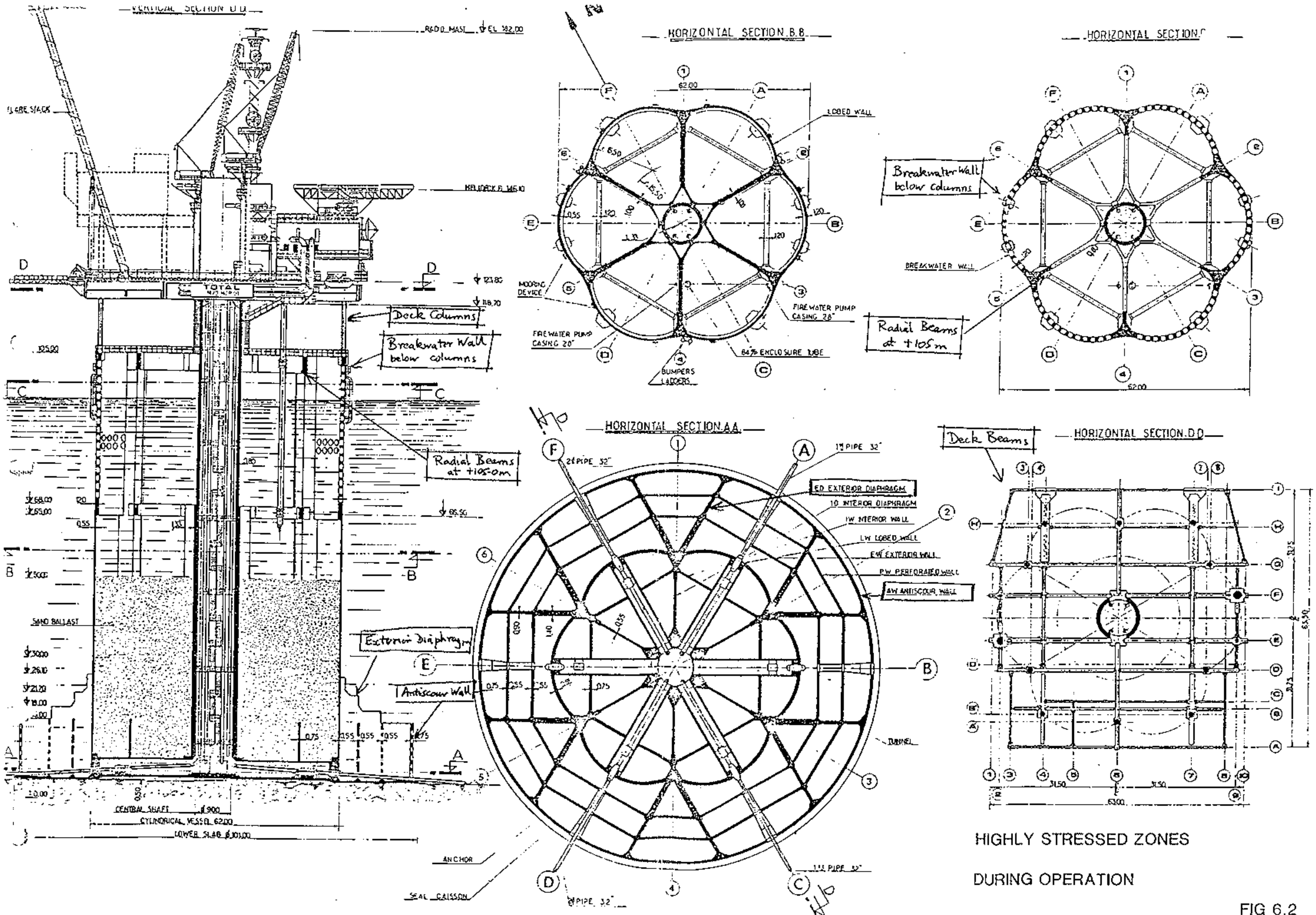
The leakage observed from some cachetages in the tunnel area raises the possibility of some corrosion of prestressing cables. There is no overt evidence of this and the affected cachetages have been repaired apparently successfully. However, it is proposed that as part of a sensitivity analysis reduction in prestress force in these areas is assumed (lower interior walls, tunnel walls, interior diaphragms). A reduction in reinforcement area in zones of deterioration is also proposed as part of a sensitivity study. Zones proposed are tunnel entrances, tunnel walls, central shaft floor and deck beams. The reduction in reinforcement and prestress should be based on observed deterioration and leakage.



HIGHLY STRESSED ZONES

DURING TEMPORARY PHASES

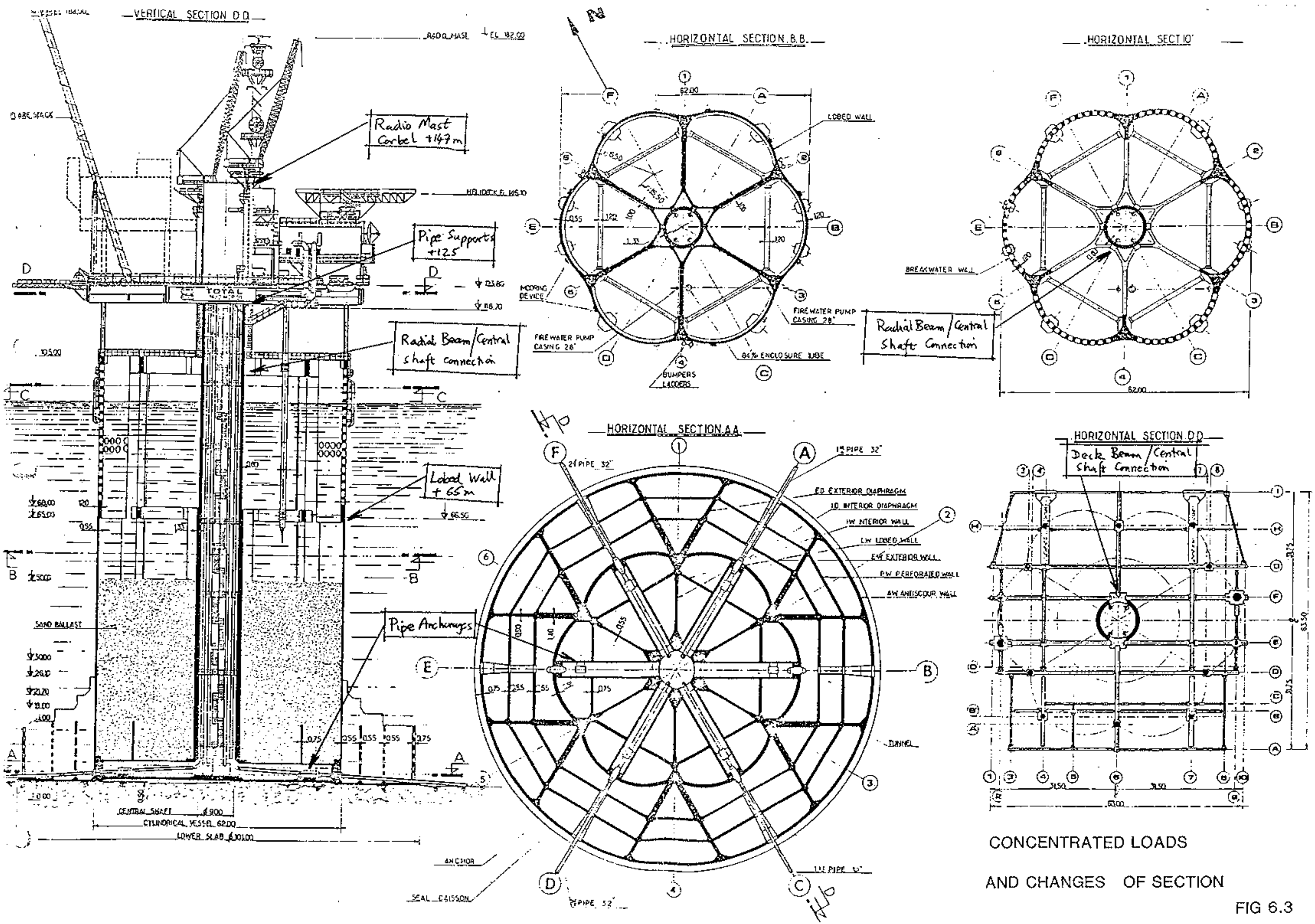
FIG 6.1



HIGHLY STRESSED ZONES

DURING OPERATION

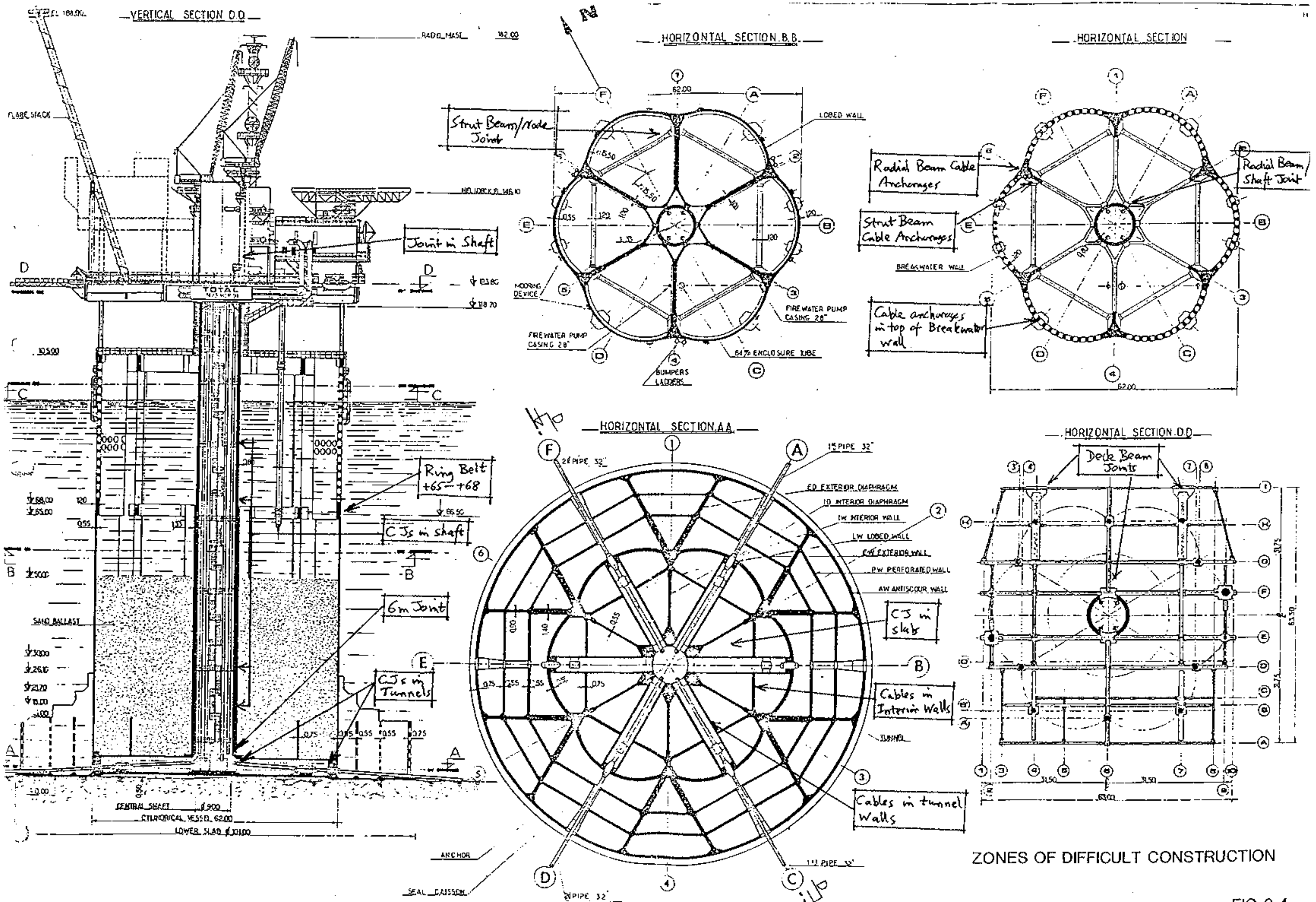
FIG 6.2



CONCENTRATED LOADS

AND CHANGES OF SECTION

FIG 6.3



ZONES OF DIFFICULT CONSTRUCTION

FIG 6.4

7.0 FINITE ELEMENT MODEL

7.1 STRUCTURE

7.1.1 General

The original design and analysis of MCP-01 relied heavily on hand calculations. The flat slabs and walls were designed using yield line theory and deep beam/corbel analogies. The lobate walls were analysed using a curved grillage of straight beam elements. Design of the thickness, reinforcement and prestress levels was carried out by hand.

The re-analysis will require the development of a full 3D FE Model that represents the whole of the super and substructure. Modelling half the structure will probably not be adequate due to the non-symmetric nature of the deck loads and the positions of the defects. Additionally, experience suggests that interpretation of results is facilitated by a model of the whole structure. It is proposed that a global model of sufficient detail to verify overall structural behaviour is used and that detailed models for critical areas are prepared separately. This approach is particularly well suited to providing an FE model mesh that can be used for wave and current load generation and conforms with Total's requirement that they can retain the model to study the effects of future loadings or structural deterioration.

7.1.2 Structural Elements

Consideration must be given to the different structural forms present in the deck and substructure which include:

- Jarlan holes
- beams/columns
- node sections
- thick slabs

Jarlan holes would be modelled using elements with revised properties, with local stress concentrations being calculated from a detailed local model.

Beam elements can be modelled as beams or plates with modified widths/thicknesses. However, attention must be paid to ensure that the connection to the walls is accurately modelled.

Possible solutions for modelling the node sections are:

i. use solid elements throughout model.

ii. Torsional stiffness of the node is modelled by a vertical bar of suitable inertia characteristics.

   Additional stiff or rigid bars join this bar to the plate which represents the thinner portion of the wall.

iii. Use solid elements to model the node. Additional plate elements of suitable characteristics would be introduced at the connection between the solid elements and the plate elements modelling the thinner part of the walls, to maintain appropriate continuity of rotation at the connection. Thick slabs can be modelled with plate or shell elements.

7.1.3 Element Types

Various types of elements could be used. These include shell, plate, beam or solid elements. The mixing of different of element types should ideally be kept to a minimum although in certain areas this will not be possible. In particular shell elements should not be mixed with plate elements. Curved elements will be needed in many areas.

Solid elements will increase the overall number of DOF, although they will allow nodes etc. to be more accurately modelled. However, output will be in the form of stresses which would need to be converted into equivalent forces/moments per unit width to allow the concrete code checks to be performed.

Other factors to be considered in the choice of elements include:

- constant or variable thickness
- curvature
- number of nodes per element
- number of Gaussian points
- position of Gaussian point within the element (fixed or adjustable)
- integration law for element (linear, quadratic or else)
- element mesh size needed to give the required accuracy

Cracked zones could be modelled by a Reduced Stiffness Analysis (RSA) using either orthropic or anisotropic elements. It is proposed that this should be limited to areas of structure that are subject to detailed modelling.

Elements should be generally rectangular, although in transfer zones triangular or wedge shaped elements would be used.

7.1.4 Model Size

The MCP-01 sub-structure incorporates many different structural elements, e.g. breakwater walls (lobed and perforated), radial walls, tunnel sections, beams etc. Although axisymmetric to an extent horizontally, in the vertical direction there are numerous cross-sectional changes which would prevent the use of symmetry or large elements.

A shell element model is therefore likely to have up to 60,000 nodes with 300,000 DOF. For a solid model this number is likely to double.

Sub-structuring or superelement techniques can be used to reduce the size of the main analysis by breaking the structure down into various components and sub-components. However, this increases the total number of analyses as substructure stiffness and loading matrices are set up and then the boundary displacements back-substituted.

7.1.5 Critical Zones

Areas which are known to be critical could either be represented by a finer mesh in the global model or be subject to a more detailed separate local analysis. The following are the critical areas where more detailed finite element modelling is required, including some solid modelling.

i. antiscour wall
ii. top of exterior diaphragms
iii. tunnels
iv. breakwater walls beneath column bases
v. radio mast corbel
vi. nodes between walls and slabs
vii. nodes of lobate walls
viii. circumferential walls at connection with radial beams
ix. central shaft at connection with deck/manifold support beams
x. central shaft at connection with load repartition structure

Some additional highly stressed areas may be identified in the course of the re-analysis.

Where there are several identical members (e.g. antiscour wall) a finer mesh could only be used in two or three cases where loading is expected to be highest or where deterioration is worst. If this approach is taken, which is not recommended, care is needed to ensure that section strength checks are based on the correct zones.

Review of the results from the global analysis will allow identification of areas that will be remodelled in greater detail. The main criteria for selection of zones for remodelling will be areas of high stress, areas where the modelling has made simplifying geometric assumptions (such as around Jarlan holes) and areas where the reported stress gradients across elements are high or discontinuous (such as in zones of high restraint).

Clearly the number of areas that will require remodelling will depend on the skill and amount of detail that was used in the first global model. There will be a trade off of complexity in the global model against fewer additional areas to study in more detail.

Experience indicates that the problem is better resolved by using a global model with a relatively constant mesh size. Then producing a series of local models in which boundary conditions are equated to those of the main model. Such local models should be clearly identified in the initial meshing so that nodes will correspond.

7.1.6 FE Programs

There are a number of FE programs available that should be capable of modelling the structure, these include:

- PAFEC
- ANSYS
- STRUDL

- ASAS
- NASTRAN
- SESAM
- LUSAS
- ADINA

Associated element libraries would include plate, shell, beam or solid elements.

Pre- and post-processing options are important as they will substantially reduce the amount of inputting etc. It would be useful to be able to generate and modify the meshes via a screen interface. It is required that plots of the generated model, displacement and stress results can be viewed via a screen interface and hard copy of selected views made.

The post-processing of the analysis results will include the printed and graphical output required in Section 7.1.8. In addition to the standard stress/force output, the sorting and checking of the serviceability and ultimate limit states of each concrete section will be required, as outlined in Section 7.3.

Attention would also need to be given to the back-up service provided by the software supplier and any limitations on overall model size. The contractor will be required to provide documentation showing that the FE package developers and the contractors installation of the package have effective quality assurance procedures in place.

Hardware availability is important too with respect to overall model size and analysis time.

7.1.7 Verification Procedures

Any model will need suitable verification procedures to ensure that it is performing as expected. These should include model and deflected shape plots etc. In all areas the contractor must be able to demonstrate that his model accurately represents the behaviour of the structure.

This will include finite element solution of simple problems similar to sections of the concrete gravity base. The FE solutions will be repeated with finer meshes to show how well key results converge with the theoretical solution. This work will form the basis on which the mesh size is selected. If mesh density is to be varied across large panels, representative test models will be used to verify the mesh algorithm used.

7.1.8 Documentation

As a minimum the documentation should give the following information:

Software Documentation:

User and theoretical manuals;
Status lists of software;
Quality Assurance Documentation.

Geometry:

Sketches of mesh divisions with dimensions/elevations and boundary/nodes with other superelements;
Nodal co-ordinates, Gauss point co-ordinates and Result point co-ordinates;
Plots of nodes/elements/supernodes (connecting nodes);
Plots of shrunk elements;
Plots of node/element numbers;
Plots of element thicknesses;
Thicknesses of nodal, Gauss and results points.

Loading:

Plots of loaded surfaces;
Plots of nodal loads;
Loadsums for each local loadcase.

Results:

Reactions at external node supports for each load combination.
Nodal displacements and rotations.

In plane forces (Nx, Ny, Nxy), moments (Mx, My, Mxy), transverse shear forces (Vxz, Vyz), principal in plane forces/moments. This information should be given per unit width (kN/m and kNm/m).

Principal skin stresses as well as transverse shear stresses.

For validation purposes scans on stresses shall be performed to identify any highly stressed areas due to modelling deficiencies.

Plots of Deformed Geometry:

Deformed geometry of the structure given on vertical, and horizontal plan sections.

Plots of principal force/moment (magnitude and vectors):

Plots of principal force/moment should be produced. Dependent on the type of element, information is required at the nodes, at the centre and at the Gaussian points.

7.2 LOADS

The loading would be performed in three different stages: elementary, equilibrium and combination.

In the elementary stage the basic load cases for each type of loading (permanent, functional etc.) are applied to the structure. In the equilibrium stages these loads are equated with the equivalent soil pressure distributions if this method of modelling the soil is used. Finally in the combination stage the equilibrium loads are combined together to give the specified load combinations.

The following sections describe the elementary load cases that would be input as appropriate pressure or point loads.

7.2.1 Permanent Loads

The concrete structure weight will be generated by the program. Topsides weight will be input as reactions on the concrete beams. Ballast and hydrostatic loads will be input as pressure loads.

7.2.2 Functional Loads

Functional loads from operations on the topsides will input as loads on the deck beams.

7.2.3 Environmental Loads

7.2.3.1 Waves

Hydrodynamic loads from waves can be input directly from the diffraction analysis program provided that a similar mesh has been utilised for modelling the structure.

7.2.3.2 Current

Current loads are input in a similar way to wave loads.

7.2.3.3 Wind

Wind load from the topsides would be input as point loads at the module supports. Wind load on the substructure and deck beams would be input as UDLs or pressure loads as appropriate.

7.2.4 Deformation Loads

Deformation loads are due to prestressing cables. Prestressing would be entered as a basic load case and would take into account both forces at cable anchorages and those due to cable curvature, friction and offset from the centre of the section.

7.2.5 Soil Pressure Loads

Soils can be modelled either as springs or as an equilibrating pressure on the base of the substructure. For the latter soil loads are first broken down into pressure distributions representing the six DOFs at the soil/structure interface. Equilibrium soil pressure loads are then calculated for the individual elementary load cases.

Two types of soil pressure distribution (elastic and inelastic) should be taken into account, giving 12 (2 x 6) basic soil pressure distributions.

If the soil is modelled as a series of springs the spring stiffnesses and their variation over the foundation area will be calculated and the springs decoupled from the base if tensile stresses exist.

7.2.6 Number of Load Cases

It is estimated that the number of load cases required for the analysis will be approximately 30 and the final number of load combinations is expected to be approximately 100 (see Section 5.3).

7.3 FE POST-PROCESSING

Output will be large and will require considerable post-processing. It is suggested that the structure is broken down into groups of elements (called design groups) with similar thicknesses and reinforcement. The critical section forces in these groups are then determined from all the load combinations and a code check performed. Concrete sections can be dimensioned by one or more of the following 26 criteria:

Nxx,	and the associated force components	:	+/-	)	
Nyy,	and the associated force components	:	+/-	)	
Nxy,	and the associated force components	:	+/-	)	12
Mxx,	and the associated force components	:	+/-	)	
Myy,	and the associated force components	:	+/-	)	
Mxy,	and the associated force components	:	+/-	)	
N1	(principal force)			)	
N2	(principal force)			)	4
M1	(principal moment)			)	
M2	(principal moment)			)	
Vxz	transverse shear		+/-	)	4
Vyz	transverse shear		+/-	)	
σ_1	Principal skin stress + face	:	+/-	)	4
σ_2	Principal skin stress - face	:	+/-	)	
V	Principal transverse shear	:	+/-	)	2
					26

Having selected the critical loadcases for each design group (maximum 26) the section forces can then be checked against the code requirements for the respective ULS and SLS cases.

The code check program should be able to calculate the relevant stress and strain levels in the concrete, reinforcement and prestressing cables and check against allowables. This process should take full amount of any divergence of the principal stress direction from that of the reinforcement.

The program should also be able to accommodate shear checks for ULS and crack width calculations for SLS. Section data for each design group (i.e. thickness, reinforcement etc.) should be read in automatically from a database as the code check is performed.

Final output should present results in a tabular format with pass/failure recorded against each criteria. More detailed back-up data should be made available if required for further verification.

APPENDIX 1A

SCOPE OF WORK
FOR PREPARATION OF SPECIFICATION

SECTION IV - SCOPE OF WORK/SERVICES

1.0 INTRODUCTION

MCP-01 has a design life of twenty (20) years which is due to expire in 1995. In preparation for extending its design life to the year 2015 COMPANY requires CONTRACTOR to prepare a detailed specification for a re-analysis to be carried out to establish the platform's structural integrity.

2.0 SCOPE OF WORK/SERVICES

CONTRACTOR is required to prepare a detailed specification for the structural re-analysis of MCP-01 based on the following activities. CONTRACTOR may with COMPANY approval alter or amend the activities to be contained in the specification.

1.0 **Objectives and Requirements**

- 1.1 Agree COMPANY Objectives
- 1.2 Determine Lloyds Requirements and Codes to be used

2.0 **Documents**

- 2.1 Assemble set of current Topside drawings
- 2.2 Assemble set of current Sub-structure drawings
- 2.3 Assemble relevant design reports
- 2.4 Assemble inspection reports

3.0 **Soils Data**

- 3.1 Assemble original data and any new data available
- 3.2 Determine procedure for deciding soils parameters

4.0 **Environmental Data**

- 4.1 Assemble original design data
- 4.2 Examine report from Marex
- 4.3 Confirm data to be used in analysis

5.0 **Loading**

- 5.1 Sub-structure: Decide basis of calculation (concrete density, ballast density/properties, marine growth, debris etc.)
- 5.2 Topside: Assemble loading data from both Doris records and COMPANY records
- 5.3 Wind: Decide basis of calculations

5.4 Wave/Current: Decide load cases to be examined, wave theory to be used

5.0 **Loading (Continued)**

5.5 Accident: Agree with COMPANY accident cases to be considered and then determine loads to be used
5.6 Load Combinations: Decide combination and load factors to be considered in analysis

6.0 **Materials and Structural Condition**

6.1 Assess design/inspection reports to determine critical zones of structure
6.2 Decide Materials properties to be used in analysis

7.0 **Finite Element Model**

7.1 Structure: Decide approximate mesh density acceptable element types and acceptable computer programmes. Decide zones of structure likely to need fine mesh analysis
7.2 Soils: Decide method of modelling of soil (2 or 3-D FE model or simple spring?)
7.3 Output: Decide form of output, post-processing and structural assessment of results, required.

8.0 **Specification for Re-Analysis**

8.1 Assemble results of the foregoing Articles 2.0 - 7.0 inclusive into specification for re-analysis

9.0 **Management**

9.1 Reporting of progress
9.2 Progress Meetings

3.0 DELIVERABLES

CONTRACTOR will initially produce a draft detailed specification for comment by Lloyds and COMPANY approval along with a report on the quality of information available, assumptions made etc. Thereafter CONTRACTOR will incorporate all comments received and re-issue the final detailed specification.

APPENDIX 1B .COMPARISON BETWEEN ACI 318-71 AND BS 8110 1985

The differences in the capacity of a concrete section when designed to the requirements of ACI 318-71 and BS 8110-85 are tabulated below.

Section Description	Capacity Comparison
700 x 1200 deep 11760mm² reinforcement 2500kNm live load	ACI 318 has 5.1% excess capacity BS 8110 has 5.2% excess capacity
As above with 275kN shear live load	Both codes require nominal links BS 8110 requires 25% more than ACI 318
As above with 550kN shear live load	Both codes require that shear links be provided. BS 8110 requires 47% more shear steel than ACI 318
700 x 1200 deep 6616kN prestress dp = 762mm 2500kNm live load 11760mm² reinforcement	Underserviceability BS 8110 fails by 10%. ACI 318 passes by 6%. BS 8110 predicts 5% less ultimate capacity than ACI 318

APPENDIX 3A . SOIL REPORTS AVAILABLE

SITE INVESTIGATION REPORTS

72 007-4	Soil Investigation performed at the Proposed Booster Station, Block 14/9, Norwegian Geotechnical Institute, December 1972.
74 015-3	1974 Soil Investigation at the Manifold, Norwegian Geotechnical Institute, November 1974.

FOUNDATION DESIGN AND STABILITY ANALYSES

MP2 D1055	Foundation Design, CG Doris, July 1975
D1588	Behaviour of the Concrete Structure and its Foundation Soil taking into account Marine Growth, CG Doris
MPI/TIR/86/03	Review of MCP-01 Foundation Status, Total Oil Marine, October 1986
87-281	MCP-01 Foundation Study, Study No. 86/8, Offshore Design Engineering, April 1987
87 314-1	MCP-01 Evaluation of Foundation Stability, Norwegian Geotechnical Institute, May 1988

APPENDIX 4A

ORIGINAL ENVIRONMENTAL DATA

Lloyd's Register of Shipping
71 Fenchurch Street, London, EC3M 4BS

Telephone 01-709 9166 Telex 888379 Cables Committee, London EC3

Compagnie Francaise des Petroles,
5, Rue Michel-Ange,
75781 Paris Cedex 16,
Paris,
France.

Please address further communications to The Secretary, and quote

Our Ref Ocean Engineering
Your Ref TEP/DP/GPM/EP
Date 22nd January 1974

For the attention of Mr.J.C.Hamard
and Mr.T. Fossena.

cc. Paris
cc. LRIS.

Dear Sirs,

Frigg Field to Scotland Pipeline-
Intermediate Manifold Platform
Design Criteria

This is to confirm that the following design criteria are acceptable for the platform installed in Block 15.

1. Storm wave: 100 Year occurrence
H = 29 meters
T = 16 seconds

2. Operating Wave: One-month occurrence, on which serviceability limit state will be checked
H = 18 meters
T = 12.5. seconds

3. Current: Surface 1.5 m/s
Bottom 0.25 m/s

The variation will be assumed linear between the surface and the bottom.

4. Wind: 36 m/s (10m above surface)
One minute gust 52 m/s (10m above surface)

5. Fatigue: 20 year duration total no. of waves 0.8×10^8
Using a cumulative probability of a linear curve between wave height and log number of cycles

cont....

Reçu le 31.01.74

NB. MCP-01 Coordinates 58° 49' 38.717 N
00° 17' 11.488 W
after installation data

Hammer.

6. Temperature effects as per P.T.I. rules previously issued for the steel platform.

The above were agreed during meeting No. E 005 Q-9.

Yours faithfully,

V. Alfred (Mrs.)
For the Secretary.

APPENDIX 4B

MAREX PROPOSAL

PROPOSAL

Environmental Design Criteria
for MCP 01
UK Blocks 14/4 and 14/9

Marex Proposal No. P3883
Issued 29 June 1992

Marex

MAREX TECHNOLOGY LIMITED
COWES ISLE OF WIGHT ENGLAND PO31 7AW
TELEPHONE 0983 296011 • FACSIMILE 0983 291776

CONTENTS

INTRODUCTION

This proposal has been prepared in response to an enquiry from Total Oil Marine plc. The document describes the data sources and analytical procedures recommended by Marex for the derivation of environmental design criteria for the Gas Compression Platform MCP 01 which is situated at approximately 58° 50' North, 00° 18' West. The water depth in this area is in excess of 100m.

Calculations are performed using metric parameters and standard oceanographic convention is observed.

The main body of the final report will contain full explanations of the various data sets, analytical techniques and procedures employed to derive the design criteria. An abstract in which the final values are summarised in a concise and simple format will be included.

It is understood that the objective of this study is to establish design criteria suitable for presentation to a certifying authority. A 'mobilisation' charge is included to cover likely costs with respect to preliminary meetings with Total and/or the certifying authority to establish the scope of work and one further review meeting towards the end of the study.

Note on Confidentiality of Data

It is recommended that at least one of the following measured data sets is acquired for use in this study:

Buchan Field	-	1984 to Present
Brae Field	-	1984 to Present

The Buchan field data are confidential to BP Exploration Ltd. Whilst data have been recorded since 1984, the data recorded during the last two years have not been read from the field tapes, quality controlled or processed in any way. The field tapes are held by Marex pending a requirement which will justify processing them. In view of this it is conceivable that BP may consider an exchange of data in return for bringing the database up to date, analysing the data and providing a copy of the final report (which will include the results of the analysis of the Buchan data). Whilst we cannot speak for BP, it may be worth seeking their reaction in this matter to which end I recommend you contact Graham Sharpe of BP Dyce.

The Brae Field data are confidential to Marathon Oil UK Limited. The data are fully up to date and ready for analysis. Should Total wish to seek access to these data we recommend you contact Nick Cresswell of Marathon London to establish Marathon's willingness, or otherwise, to release the data to Total, and any associated exchange of data, or fee.

The Forties data measured between 1974 and 1980 are available to Total through a UKOOA Oceanographic Committee agreement which was in operation at that time. The Forties database does continue to the present, but subsequent data remain confidential to BP. Access could be negotiated, but given that Total might wish to approach BP, then the Buchan data set would be preferable in this instance. We include the Forties data only because we know, at the time of writing, that this is the nearest location to which Total already have access.

Ships' Observations	-	Public Domain
Kirkwall Coastal station	-	Public Domain
Current data	-	Public Domain

1. DATA SOURCES

1.1 Wind Data

Source	Location	Period
Buchan Platform	53° 03' N, 02° 14' E	1984 - 1992
Brae Alpha Platform	58°42'N, 01°17'E	1984 - 1992
Forties Platform	52° 42' N, 02° 18' E	1974 - 1980
Ships' Observations	58.2° - 60.5°N, 0.8°E - 1.4°W	1949 - 1991
Kirkwall Coastal Station	58° 59'N, 02°59'W	1970 - 1985

Note: Ships' Observations will be extracted from the data base with the following selections:

i) Stationary Vessels EXCLUDED
ii) Normalisation ON
iii) Presentation in 8 Directional Sectors

1.2 Wave Data

Source	Location	Period
Buchan Platform	53° 03' N, 02° 14' E	1984 - 1992
Brae Alpha Platform	58°42'N, 01°17'E	1984 - 1992
Forties Platform	52° 42' N, 02° 18' E	1974 - 1980
Ships' Observations	58.2° - 60.5°N, 0.8°E - 1.4°W	1949 - 1991

Note: Ships' Observations will be extracted from the data base with the following selections

i) Wave Steepness Check ON at steepness 1:9
ii) Stationary Vessels EXCLUDED
iii) Normalisation ON
iv) Presentation in 8 Directional Sectors

The following wind-wave prediction technique will be used to verify results obtained from the measured data:

A depth dependent form of the JONSWAP spectrum, the TMA spectrum, developed by Bouws et al.

1.3 Currents and Water Levels

Output from the 'Continental Shelf Model' (CSM) run by the Institute of Oceanographic Sciences for the following area:

58°40' to 59°00'N by 00°00' to 00°30'W - Grid Square 25,12

Note: Information is supplied by IOS (BODC) in the form of a short report. This document will be attached to the Marex report as an appendix.

Information provided by admiralty charts and tide tables and by selected Tidal Atlases.

2.4 Additional Information

In addition to the information specified in Item 2.2 above, the following factors are provided:

i) Factors for the calculation of extreme wind speeds with durations other than 1 hour, including:

Seconds	Minutes	Hours
3	1	3
5	10	6
10		12
15		18
20		24
30		36
		48
		72

ii) Factors for the calculation of wind speeds at heights other than 10 metres above mean sea level, namely 25, 50, 75 and 100 metres above sea level.

3. EXTREME WAVE CONDITIONS

3.1 General

The measured data sets will be analysed separately to provide estimates of the 50 and 100 year extreme significant wave height at the study area. Extreme values given by the various analyses will be adjusted, if necessary, for relative severity with respect to both time and location and the resulting values reviewed with full explanations for reasons of preference or doubt. Comparison of the results will be made with the current DoE Guidance Notes. Appropriate design criteria will be recommended.

3.2 Summary of Predictions

i) Extreme omnidirectional wave heights and periods for return periods. 1, 5, 10, 50 and 100 years, including:
   - Significant Wave Height (Hs)
   - Maximum Wave Height (Hmax 3 hr)
   - Mean zero-crossing period (Tz)
   - Zero-crossing period for Hmax (Tmax)
   - Range of likely zero-crossing periods for Hmax

ii) Extreme wave heights and periods in 8 x 45 degree sectors including all periods and parameters discussed in 3.2 (i) above.

iii) Individual wave height distributions, omnidirectional and by 8 directions (using 9 wave scatter diagrams), including numbers of waves in height class intervals as required, nominally 1 metre. The associated mean and range of periods for each height class is provided.

iv) Mean JONSWAP, TMA wave spectra by direction for the 50 or 100 year extreme sea state.

3.3 Analytical Procedures

3.3.1 Cumulative Frequency Extrapolation

Data are processed to provide a cumulative frequency distribution which is then extrapolated to the low probability levels of exceedence using Weibull and Fisher-Tippett 1, 2 & 3 functions.

3.3.2 Analysis of Maxima

Monthly maximum values are extracted from each measured data set and tabulated. The annual maxima are analysed to predict extreme values corresponding to the selected return periods. Observed maxima from this analysis are subsequently compared with the final results in order to compare the predicted values with the observed data.

3.3.3 Peaks over Threshold Analysis

The data are processed to produce peak values of storm events which exceed a selected threshold. These values are then fitted to Weibull, Fisher-Tippett 1, Log-Extremal and Exponential distributions using both Least Squares and Method of Moments fitting techniques to produce estimates corresponding to 1, 5, 10, 50 and 100 year return periods.

3.3.4 Fatigue Studies

The most appropriate wave scatter diagram (significant wave heights against mean zero-crossing periods) is analysed to provide an individual wave height/period distribution. The mean and range of individual wave periods in each height class will be provided. Note that, in the absence of directional wave data, it will, for the purpose of this analysis, be necessary to assume that wave directions correspond to the simultaneous wind direction.

4. EXTREME WATER LEVELS

4.1 Summary of Predictions

4.1.1 Tidal Parameters

The following tidal parameters are estimated:

i) Lowest Astronomical Tide (LAT)
ii) Mean Low Water of Spring Tides (MLWS)
iii) Mean Low Water of Neap Tides (MLWN)
iv) Mean Water Level (ML)
v) Mean High Water of Neap Tides (MHWN)
vi) Mean High Water of Spring Tides (MHWS)
vii) Highest Astronomical Tide (HAT)
viii) Time difference from nearest standard reference port
ix) Height ratio against nearest standard reference port

Note: In addition estimates of extreme levels for 1, 5, 10, 50 and 100 year return periods are provided (see Section 4.1.3)

4.1.2 Surge Levels

Estimates of extreme positive and negative surge levels are provided for return periods of 50 and 100 years.

4.1.3 Extreme Water Levels

The estimates from 4.1.1 and 4.1.2 above are combined and added to the water depth at the proposed platform location in order to produce estimates of extreme water depth for return periods of 50 and 100 years including:

i) Depth at LAT
ii) Design Still Water level
iii) Design Crest Elevation above maximum still water level
iv) Design Extreme High Water level above sea bed

4.2 Analytical Procedures

Tidal information is provided by the IOS Continental Shelf Model, various tidal atlases and Admiralty publications. Extreme tide and storm surge estimates are obtained from the Continental Shelf Model run by the Institute of Oceanographic Sciences. The output from this model is supplied to Marex by the IOS in the form of a short report. This report will be appended to the final report produced by Marex. Extreme still water level calculations include appropriate factors to account for the probability of tide and surge occurring simultaneously.

4.3 Additional Information

A summary description of the meteorological effects on water level, extracted from the Admiralty Tide Tables, is included.

5. EXTREME CURRENT CONDITIONS

5.1 Summary of Predictions

5.1.1 Tidal Current

Tidal current ellipse parameters (semi-major axis, semi-minor axis and orientation of the semi-major axis) are calculated for the following depth-mean current conditions:

i) $M_2 + S_2$ Tide

ii) Mean Spring Tide

iii) Maximum Tide

5.1.2 Surge Currents

Surge currents for the 50 & 100 year return period are provided as ellipse parameters or in 24 x 15 degree directional sectors.

5.1.3 Extreme Depth-mean Currents

Estimates of extreme depth-mean currents with 1, 5, 10, 50 and 100 year return periods are provided in 24 x 15 degree sectors and 8 x 45 degree sectors.

5.1.4 Current Profiles

Current profiles in 8 x 45 degree sectors are presented for return periods of 50 and 100 years. The following current speeds are provided.

i) Surface, 0.5, 2, 5 and 10 metres and thereafter at 10 metre intervals until the current remains constant with depth.

ii) 0.5, 2, 5 and 10 metres above the bottom and thereafter at 10 metre intervals until the current remains constant with depth.

Fine detail of the estimated current profile and/or depth integrated values in selected zones can be provided on request.

5.2 Analytical Procedures

5.2.1 Tidal Currents

Comparison is made between measured current data, modelled tidal currents and tidal diamonds taken from Admiralty Charts and the DHI tidal atlas. Appropriate values are then selected.

5.2.2 Surge Currents

Surge currents are modelled from 16 of the most severe events in recent years. Estimates of extreme surge currents are enveloped to provide a smoothed distribution suitable for engineering studies.

5.2.3 Extreme Depth-mean Currents

Extreme currents, calculated from the tide and surge components are factored to account for the probability of simultaneous occurrence of extreme tide and surge.

5.2.4 Current Profiles

Current profiles are calculated on the basis of a logarithmic decay of wind induced current with depth, and a power law relationship (0.1) extending over the lower 25% of the water column.

5.3 Additional Information

All procedures are fully supported with appropriate graphs and diagrams. All current profiles are tabulated and plotted. A table of factors relating 50 year return surge to other return periods is provided.

6.2.3 Current

Current profiles are calculated for 3 conditions as follows:

i) Mean Neap Condition

Mean neap tidal currents are assessed with a wind speed of 10 m/s (Beaufort 5) in each direction.

ii) Mean Spring Condition

Mean spring tidal currents are assessed with wind speeds of 20 m/s (Beaufort 8) in each direction. Alternative wind speeds can be used on request.

iii) Maximum Tide Condition

Maximum tidal currents are assessed with wind speeds corresponding to the 1 year return extreme wind speed in each direction.

7. REPORTING

Reports are prepared using a modern word processor and laser printer. Tables are blocked and charts, diagrams, etc. produced using computer graphics or a professional graphics artist. Reproduction generally corresponds to the form of this proposal. If necessary colour photocopies are used for added clarity.

Reports are comb-bound in 300 gram/m^2 card covers. The cover can, on request, be printed using a photograph supplied by the client. In the absence of any special request the cover is printed with a map of the North Sea with relevant data sources and areas of interest shown. This map is also reproduced within the body of the report.

APPENDIX 5

CHECK ON STRUCTURE WEIGHT