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Title of Report

FRIGG FIELD

CONDEEP TCP2 - STEEL SUPPORT FRAME DESIGN, FABRICATION AND INSTALLATION (DFI) RESUMÈ

Client/Sponsor of project

Norwegian Petroleum Directorate

Work carried out by

Det norske Veritas

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TECHNICAL REPORT

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Reporters sign.	[signature]

The present report deals with the design, fabrication and installation of the Steel Support Frame on the Fixed Offshore Structure. Treatment and Compressor Platform No. 2 on the Frigg Field (TCP-2).
The related design control and inspection activities carried out by det norske Veritas on behalf of the Norwegian Petroleum Directorate are described.
The permanent Modules and Pancakes which are located on the platform will be dealt with in a separate report.

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1.0 INTRODUCTION

This report deals with the design, fabrication and installation of the Steel Support Fram (SSF) on the Fixed Offshore Structure; Treatment and Compressor Platform No. 2 (TCP-2). The related control and inspection activities carried out by Det norske Veritas, on behalf of Norwegian Petroleum Directorate are explained.

"Avtale mellom Statens Oljedirektorat (OD) og Det norske Veritas (DnV)" of October 1974 forms the basis for the control and inspection activities performed by DnV. The scope of these activites are further laid down in "Scope of Work for Control and Inspection of Fixed Offshore Platform TCP2 - Frigg Field on behalf of Norwegian Petroleum Directorate" dated January 14th, 1977.

Both the above referenced documents are attached in Appendix 1 to this report.

This report which, as mentioned above, comprices of a resymé of the design, fabrication and installation of the Steel Support Frame on the Frigg Field Treatment Compressor Platform No. 2 TCP2, covers only the structural parts of the platform. The Modules will be dealt with in a separate report.

With reference to the above "Scope of work" only one recommendation remains to be given by DnV for NPD-approval for TCP 2, namely:

7. Approval for platform to be taken into use of hydrocarbon production.

Based on results from DnV inspection of offshore works presently going on, such recommendation and approval will be given by DnV/NPD in 1978.

The present report contains the most essential information with respect to the design, fabrication and installation of the platform as well as the premises upon which these activities are based. For further details reference is made to the appropriate documents referred to in the following.

The aim of the DFI-resumé is also to establish a basis for planning inservice inspection programs in order to:

- maintain the NPD-approval in the operation phase of the platform
- provide assurance to Elf that platform will perform safely in the operation phase

DFI-resumés for TCP 2 Concrete Structure as well as Gas Riser Pipes will be prepared separately by DnV.

2.0 GENERAL DESCRIPTION

The Steel Support Frame being a module deck which consist of four longitudinal girders and six transverse girders as shown on K.E. drawings Fig. 2.0a-c.

The main dimensions: ⌈LxBxH 73mx51mx9.6m⌈ capacity: approx.

22.500 tons on two decks.

The SSF is supported on three condeep shafts which forms the corner of an equilateral triangle.

The structure receives live load from pancakes on the lower chords and live load from the Modules on the upper chords. The loading on the structure will be explained in more detail in section 3.4.

The main members of the support frame consist of box sections which are fabricated by welding. The section properties of the members are shown on drawings Fig. 2.0d-h.

The deck was fixed to the concrete columns by means of 427 posttentioned anchor bolts (Dyvidag).

Construction periode: November 1975-June 1977.
Towing to Frigg: June 1977.

3.0 DESIGN RESUMÉ

3.1 Specification

The following main specification is the basis for the design and construction of the TCP2 - Steel Support Frame.

TECHNICAL SPECIFICATION
FOR
A STEEL SUPPORT FRAME DESIGN TCP-2
ELF - NORGE A/S
TREATMENT-COMPRESSION PLATFORM NO. 2 FRIGG FIELD
February 1975 1st Revision

A list of different specifications applied on TCP2 is included in the list of 'References' in Appendix 8.

Comments and the application of these specs. will be given in the relevant chapters/subsections.

3.2 Organization

NORWEGIAN CONTRACTORS has been Main Contractor for the TCP-2 structure for the Frigg Field, in the design and construction of both base- and deck structure.

The client, ELF NORGE A/S, is operator for the Frigg Field.

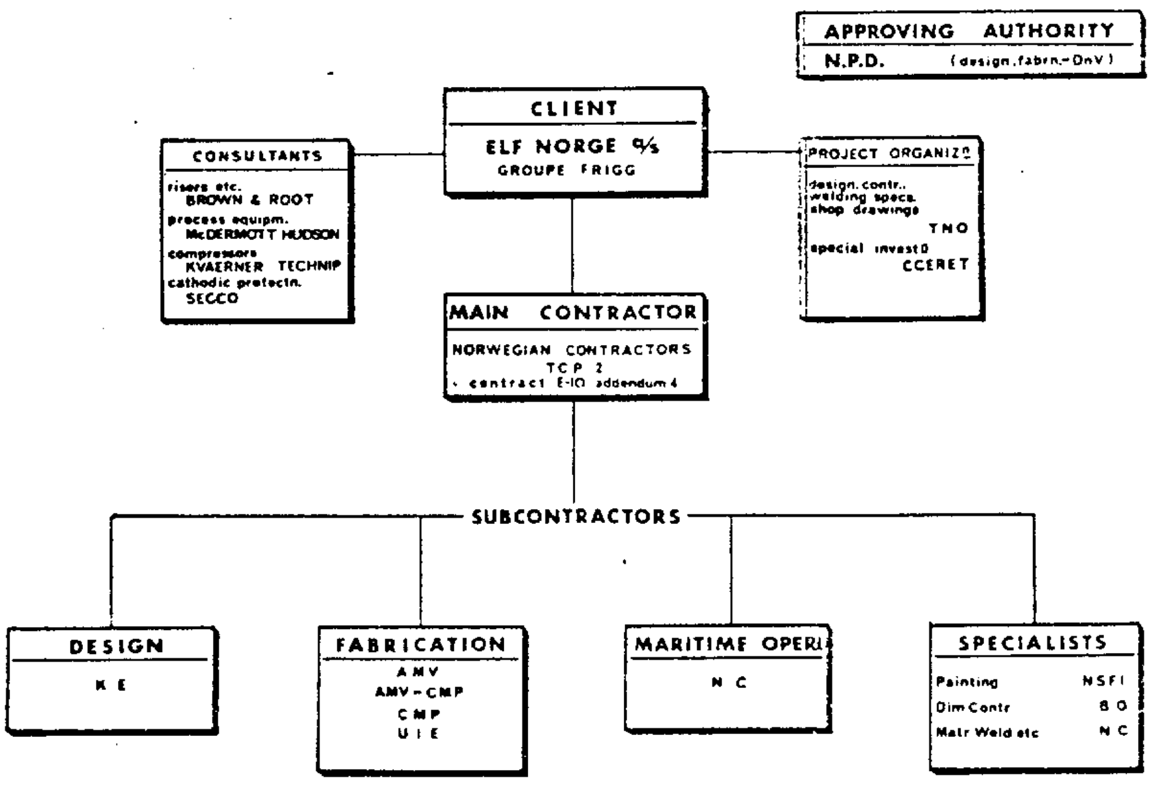
GENERAL

Norwegian Contractors Scope of work under Addendum 4 to Contract E-10 can generally be described as

> Design and fabrication of the TCP-2 Steel Support Frame, handling and transportation until delivery on two barges, safely moored at quay in Åndalsnes.

Also included, is the project management for this work.

ORGANIZATION PLAN



3.3 Steel Material

3.3.1 Material specification

Selection of materials has been based on

- Elf Norge's specification "1052 No. 5-302/Rev. 0/JPS of May 1975

and applied for delivery of rolled plates, shapes, sections and profiles. These plates have been produced from high strength steel and mild steels. The designations used in the specifications and on drawing as applicable have been as listed below reflecting the mechanical characteristics and typical applications:

Table 1: Designations of structural streels in TPC-2 material specifications.

Designation	Steel grade acc. to DIN 17100	Thickness (mm)	Classification acc. to DnV Rules	Typical use
SHS 20	St. 52-3N (modified)	12-15-20 -25	Special	Nodes, girders, trusses
SHS 40	St. 52-3N (modified)	30-40-50 -60-80- -100	Special	Heavy wall nodes, girders, trusses
ML 0	St. 37-3U	various	Secondary	Plating, sections
ML 20	St. 37-3N	various	Primary	Plating, sections, profiles

For ST 52-3N steel, a minimum yield strength of 3400 kp/m^2 are used for all plate thickness.

All structural steel have been specified to be made acc. to DIN 17100 and modified as considered appropriate for the grade and application. The following main modifications/supplementary requirements have been specified for high strength steel, St. 52-3N:

Designation	Chemical composition	Mechanical properties	Soundness of steel plates
SHS 20	As for HS 20 except: Max 0.18 C Max 0.015 S CE max 0.43 for t < 30 mm CE max 0.45 for t > 30 mm Vacuum degassed	As for HS 20 plus Z-direction tensile tests with RA_Z > 30% (Note 2)	Level 2 of S.E. 062-69 at plate edges and plate body
SHS 40	As for SHS 20 except impact testing at -40°C		

Notes (1) CE = C+Mn/6 + Si/24 + (Cr+Mo+V)/5 + (Ni+Cu)/15

(2) RA_Z = Reduction of area measured acc. DnV recommendations (1973)

(3) Soundness: Stahl-Eisen Lieferbedingungen 072-69 "Ultraschall geprüftes Grobblech".

General delivery conditions has been based on ASTM A6 "Standard specification for delivery of rolled steel plates, shapes, sheet piling and bars for structural use" as regards dimensions and straightness.

Non-structural materials

Non-structural materials used for walkways, ladders, brackets etc. have been selected from one of the following grades:

- Plates/sections shapes: DIN 17100/St. 37-2U, St. 42-2, St. 52-3 or equivalent grades.

Origin of structural steels

The major part of structural steels have been delivered by Suomitomo Metal Industries, Japan. Certificates have been issued by the steel mill, but endorsed by DnV.

Additional deliveries have been made by Oxeløsund Jernverk, Sweden.

Purchasing of structural steels has mainly been handled by the Owner.

Total steel weight was appr. 3443 tons.

3.3.2 Material Application

The structure is classified into the following group:
(Ref. Fig. 3.3 a) b)).

Special structural parts (SSP)

Support ring D3, B6 and F6.

Node between crossing of girders:

Row 2-C, 2-E
" 5-A, 5-C, 5-E, 5-G
" 7-A, 7-C, 7-E, 7-G

i.e. mode no.:
206, 256, 207, 257,
406, 456, 407, 457,
500, 550, 501, 551, 506, 556, 507, 557,
700, 750, 701, 751, 706, 756, 707, 757.

Node above support ring:

Node no.: 21, 41, 505, 555, 705, 755.

Primary structure (PS)

All other structural parts of the steel support frame, including the main members in the structural deck.

Secondary structure (SS)

Structural deck and stringers except as mentioned above.

3.3.3 Corrotion Protection

The coating of the steel support frame are carried out according to the Elf Specification, see /5/, /6/ and /7/.

3.4 Design premises

3.4.1 Design codes

The steel support frame are designed in accordance with the AISC Specification /8/ and the DnV Rules for Fixed Offshore Structures 1974 /9/. Reference are also given to the Norwegian Code for Steel Construction /10/.

For storm conditions the AISC basic allowable stresses are increased by one third as stated in the DnV Rules /9/.

Deck members supporting modules/pancakes subjected to large hydrostatic test loads, members subjected to heave and roll acceleration forces during transportation or similar excessive short turn loads designed by considering a one third increase in the basic AISC allowable stresses.

However, stresses produced by normal operating loads shall not exceed the AISC basic allowable stresses without the one third increase.

3.4.2 Environmental condition

a) Storm conditions

Wave height: 29 m
Wave period: 14 s

Current velocity	at surface:	1.35 m/s
" "	30 m above sea bottom:	0.70 m/s
" "	at sea bottom:	0.30 m/s

Between these levels the current velocity varies linearely.

A maximum 1 minute sustained wind velocity of 55 m/s used constant from the sea level to + 10m. Above 10 m the velocity is given by:

$$V_z = 55\sqrt{C_h}$$

where

$$C_h = 2.5 \frac{z + 18}{z + 60} \quad \text{(z in meters)}$$

b) Operating Conditions

Wave height: 18.7 m
Wave period: 11.2 s

Current velocity	at surface:	1.00 m/s
" "	30 m above sea bottom:	0.58 m/s
" "	at sea bottom:	0.30 m/s

Between these levels the current velocity varies linearely.

A maximum 1 minute sustained wind velocity of 35 m/s is used constant from the sea level to + 10 m. Above 10 m the velocity is given by:

$$V_z = 35\sqrt{C_h}$$

where

$$C_h = 2.5\,\frac{z + 18}{z + 60} \quad \text{(z in meters)}$$

c) Transportation and Towing

i) Transport of part of SSF from Dunkirk to Cherbourg.
Max. wave height 6.0 m to calculate deflection of barge in waves. Wave length equal to length of barge.
Interaction forces due to barge motions acting in c.g. of deck structure.

- Vertical 0.5 g
- horizontal/transverse 0.5 g
- horizontal/longitudinal 0.25 g

ii) Transport of deck from Cherborug to Åndalsnes.

Wave height $H_{1/3}$ = 6.0 m.
Period T = 8.5 sec.

This result in a max. roll angle of 23°.

iii) Towing (from Åndalsnes to Frigg Field).
According to ref. /1/, the platform were designed to withstand operating wave and wind forces i.e.

Wave height: 18.7 m
Wave period: 11.2 S.

And a maximum 1 min sustained wind of 35 m/sec.

The TCP 2 response in floating conditions is evaluated in Høyer Ellefsen Report No. H-E 7504 /22/.

Høyer Ellefsen Report No. H-E 7509 /23/ gives a summary of the conclusions from the above mentioned report.

A spectral analysis of the responce during tow-out to the installation field is given in Høyer Ellefsen Report No. 76028 /24/.

However, the most severe load condition during towing, will occure if water is coming into the structure due to damage. This situation is treated in Høyer Ellefsen Report No. 7525 /25/.

d) Temperature, snow and ice

Temperatures

The effect of differences in temperature between the concrete structure and the steel support frame are calculated.

It is assumed that no differences in temperature occur between the steel truss members. The temperature of the concrete structure and the steel support rings is assumed to be the same as the water temperature.

For the location considered the temperature falls between the following limits /1/:

Air temperature:	$-15^{o}C$ and $+32^{o}C$
Water " :	$+14^{o}C$ and $+17^{o}C$

When assuming a temperature range of $2^{o}C$ - $15^{o}C$ during deck erection in Åndalsnes, this leads to a temperature deviation of plus minus $30^{o}C$.

Snow and Ice Accumulation

Snow and ice accumulation are not considered in the overall calculation.

3.5 Design Approach

3.5.1 Structural Model for the Overall Analysis

a) Operational Plase

The steel support frame and the Condeep shafts form a space structure.

When analysing the support frame the structure is considered as a space frame, consisting of the three vertical concrete shafts and the steel deck.

The connection between the shafts and the deck structure is assumed fixed.

The shafts are taken as clamped at the caisson of the Condeep platform.

In order to set up the mathematical model:

- The steel deck is considered as an assemblage of individual beam elements. The beam elements are assumed to be straight bars having their axes at the senterlines of the steel sections.

- The top of the concrete shafts are subdevided into shell elements. Beneath the shell sections beam elements are used.

- The centreline of the shaft beams and the shell structures are connected by a set of infinitely riged beams.

- The centreline of the support ring beams and the shell structures are connected by a set of fictive finite elements.

- The rigidity of the shafts are based on the entire concrete cross sections ignoring the reinforcement.

- The shafts are assumed clamped at the top of the Condeep caisson.

The overall analysis of the structure has been carried out by KE by application of STARDYNE 3 /18/ which is a static and dynamic structural analysis system of computer programs for linear elastic structural models.

The computer model and the node numbering first used by KE, are shown on the following drawings: (enclosed, appendix 5).

UC 3337 - 0440 - 20 - 1001
UC 3337 - 0449 - 20 - 1002
UC 3337 - 0449 - 20 - 1003
UC 3337 - 0449 - 20 - 1004
UC 3337 - 0449 - 20 - 1005
UC 3337 - 0449 - 20 - 1006

As shown on the three last drawings the rings are represented by an assemblage of 12 straight beam elements. The upper parts of the concrete shafts are modelled by planar quadrilateral shell elements. The ring beam elements and the shaft elements are connected by a number of extra beam and shell elements.

These connections will be explained in more detail.

The following assumptions are made:

- Beam theory is valid for the ring beams.
- Twisting of the beams are taken as pure torsion, i.e., warping is neglected. (This is also a STARDYNE assumption.)

- The bottom flange of the rings are rigidly connected to the top of the shafts.

The finite element modelling of ring - top of shaft is shoen on Fig. 3.5 a).

The elements which connect the ring and the shaft are given stiffness ridgities which yield realistic deflection of the beam resting on the shaft.

When a point load is acting on the ring beding moments and shear forces will be introduced. The distribution of this forces depends on the axial and shear deformation of the connection elements, see Fig. 3.5 b). An assemblage of shell elements having a thickness of $2t_w$ and the module of elasticity of steel, will support the ring element in a rather realistic way.

The introduction of such elements in addition to the beam elements means:

- Only a small increase in the bending stiffness of the section above the concrete part. This is due to the fact that this shell element do not have a rotational degree of freedom perpendicular to its own plane.

- The axial stiffness is increased somewhat, but since axial deflections do not mean much for the stress resultants this may be tolerated.

So far only the bending behaviour has been considered. Torsional moments must also be taken into account. Twisting of the ring may introduce local bending moments in top of the shaft wall.

In the mathematical model vertical beam elements have been introduced between nodes in the ring and nodes in the shaft.

The bending ridgities about the weak axis are neglected.

The model mentioned has a coarse mesh of the rings and the upper part of the shafts in the circumfirential direction. When using simple straight beam elements and planar shell elements this may lead to inaccurate for stresses in the steel ring areas.

We have therefore put up a refined model for this part of the structure. The model and the node numbering are shown on the drawings:

UC - 3337 - 0449 - 20 - 1009
UC - 3337 - 0449 - 20 - 1010
UC - 3337 - 0449 - 20 - 1011

This model should give acceptable results for the overall stresses in both the steel and the concrete structure.

Rigid offsets at nodes have been used for two reasons:

1. To represent the excentricities of the diagonals and verticals.
2. To give the upper and lower chords an increased stiffness at the nodes.

At an early stage a model without any stiff offset at all was used.
Later on, stiff offsets were specified on the verticals and the diagonals to represent the excentricities in the nodes.

A model specifying 2/3-l where l is the distance from the centreline of the node to the end of the gusset plate, has been suggested, see MODEL B.

The finite element analysis of node 410 indicates that the offset beam should have a length of 1/2·l. The model based on this assumption was called MODEL C.

b) Transportation

During transportation on barges and deck installation, a computer model neglecting the concrete shafts is used. The same node numbering system as shown in drawings. VC 3337 0449 20 1001 A is used for the deck structure.

c) Towing Phase

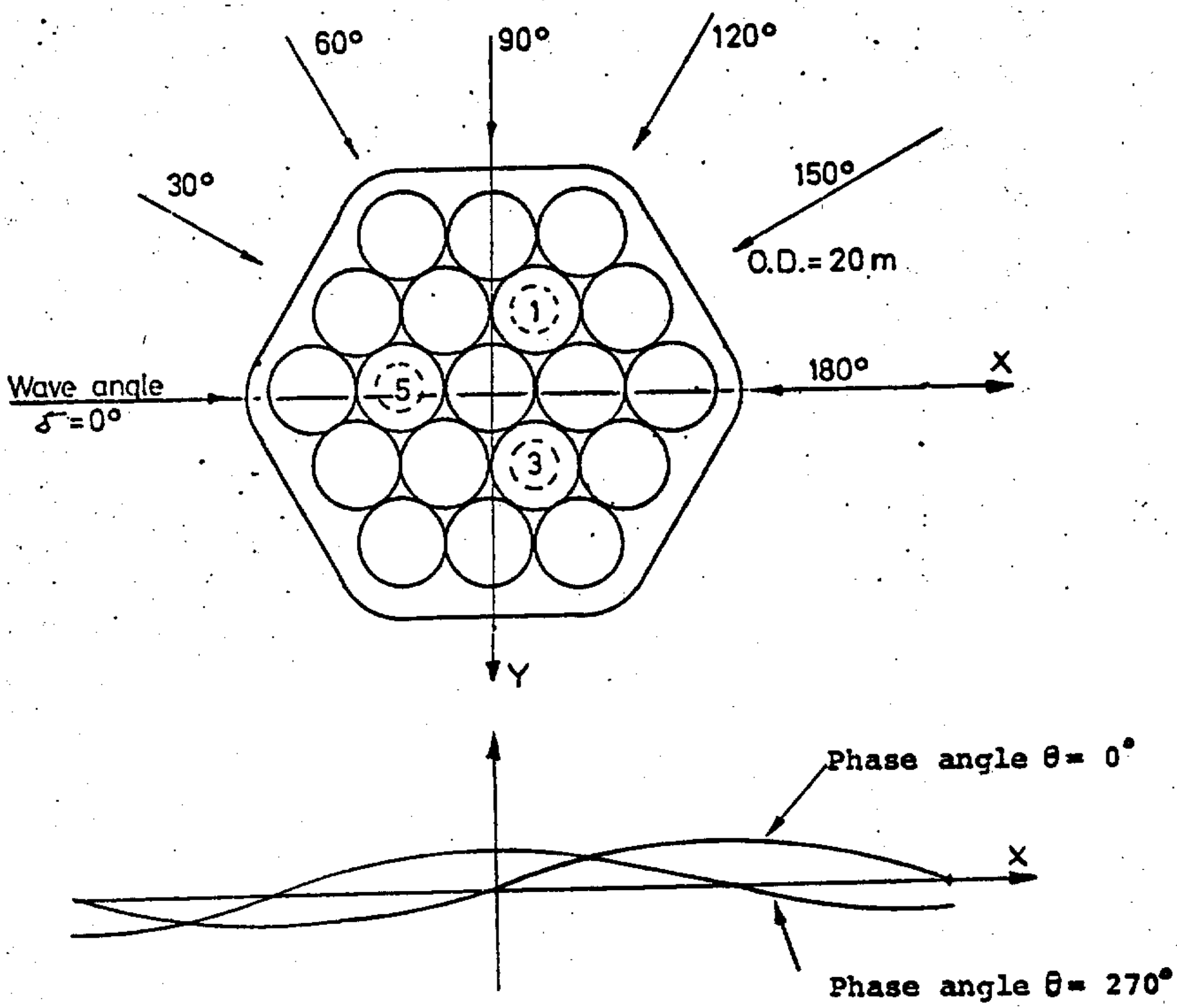
During towing the same model as for Operational Phase is valid.

3.5.2 Loadings

a) Operational Phase

i) Wave and Current Forces

The forces on the Condeep shafts due to waves and current is calculated by Høyer Ellefsen /11/ for different wave angles and different phase angles, see Fig. below.



Wave angles and phase angles

The horizontal forces and overturning moments for operational waves are given in table below for different wave angles and phase angles. The variation are also shown on figure 3.5 c) d). It may be seen that a phase angle of 300° results in the largest horizontal forces and overturning moments for almost every wave directions. Therefore this phase angle are used in all calculations concerning operational waves.

OPERATIONAL WAVE FORCES ACTING ON THE SHAFTS

Wave angle	phase angle / Force	285°	300°	315°	330°
0°	Force	5941	5918	5492	
	Moment	204690	215680	211170	
30°	Force	5970	5962	5553	
	Moment	206360	218830	216230	
60°	Force		5999	5595	4689
	Moment		221360	219910	195240
90°	Force		5946	5538	
	Moment		217680	215160	
120°	Force	5932	5898	5472	
	Moment	204080	214310	209770	
180°	Force		5999	5595	4689
	Moment		221360	219910	195240

NOTE:

Force = Total horizontal force on shafts.

Moment = Overturning moment at shaft base.

For the storm wave forces Table below has been prepared. In this case the horizontal force has its maximum for a phase angle of 315^{o} and the overturning moment has its maximum for a phase angle of 330^{o}.

As the maximum horizontal force and the maximum overturning moment do not take place for the same phase angle, a separate calculation has been performed to find the governing phase angle.

A computer run for storm wave $\delta = 0$, phase angle $\theta = 285^{o}$, 300^{o}, 315^{o}, 330^{o}, 345^{o} has been made, and axial forces and bending moments has been compared. It may be concluded from this computer run that a phase angle of 300^{o} is governing.

The operational waves govern the design for the steel deck structure (execpt 2 members). The reason for that is the one third increase of the allowable stresses for the storm condition.

Wave angle	Phase angle / Force	300^o	315^o	330^o
0^o	Force	9593	9633	8932
	Moment	326500	365400	375100
30^o	Force	9612	9678	8994
	Moment	327360	368500	380400
60^o	Force	9615	9705	9053
	Moment	327100	370300	385400
90^o	Force	9563	9617	8970
	Moment	324260	363900	378500
120^o	Force	9537	9541	8877
	Moment	322600	358600	370800
130^o	Force	9592	9636	8962
	Moment	325490	365400	378600

Force = Total horizontal force on shafts.
Moment = Overturning moment at shaft base.

Force (max.) => phase angle = 315^o
Moment (max.) => phase angle = 330^o.

ii) Wind Forces

Wind Forces on Deck and Modules

The calculations of the wind forces are based on the DnV-rules /9/ and "Technical Specification for a Steel Support Frame Design TCP-2" /1/.

The wind forces are calculated using the formula:

$$F = C_s \cdot \frac{Vz^2}{16} \cdot \sin\alpha \cdot A$$

C_s - shape coefficient
V_z - wind velocity in m/s
α - angle between wind direction and the surface
A - projected area in square metres

The wind forces are calculated for operational and storm condition using the following parameters.

Operational condition:
$V_{10} = 35$ m/s
Storm condition:
$V_{10} = 55$ m/s

The values for V_{10} are the maximum 1 minute sustained wind velocity at level 0 to + 10 m.
Above + 10 m the velocity is given by the following formula:

$$V_z^2 = 2,5 \frac{z + 18}{z + 60} \cdot V_{10}^2$$

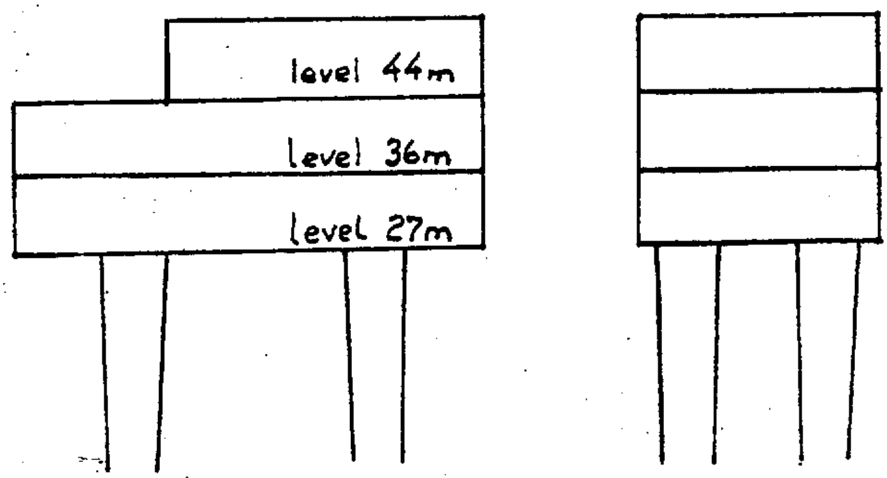
(Z in m = height above sea level).

In the computer calculations the operational wind forces are calculated by multiplication of the storm wind force with a factor = 0,405.
This factor is equal ratio

$$(V_{op.cond})^2/(V_{storm\ cond})^2 = \frac{35^2}{55^2} = 0,405$$

The storm wind forces are calculated for 3 levels.

Shape of platform deck.



level 27 m: Steel Support Frame
level 36 m: Treatment Modules and Compressormodules
level 44 m: Tanks on top of Modules etc.

LEVEL	V_z^2	$\frac{V_z^2}{16}$	C_s
m	m^2/s^2	kp/m^2	
27	3902	243,9	1,8
36	4265	266,6	1,8
44	4507	281,7	0,6

iii) Dead Load and Live Load

The dead load of the structure are calculated using the following specific weights:

Steel: $\gamma s = 8{,}0\ t/m^3$
Reinforced concrete: $\gamma c = 2{,}5\ t/m^3$

Drawings (included in Appendix 3)

UC 3337 - 0449 - 20 - 1021 Rev. J
UC 3337 - 0449 - 20 - 1022 Rev. H

show the total live load (LIVE LOAD 1) on cellar deck and main deck respectivily. This loadings exist when the platform is fully equipped.

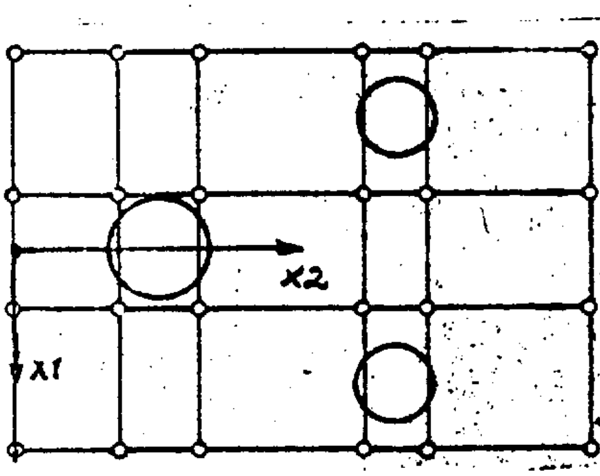
The first years after platform installation the compressor modules are not installed. The structure is therefore designed for an alternative live load (LIVE LOAD 2) shown on drawings:

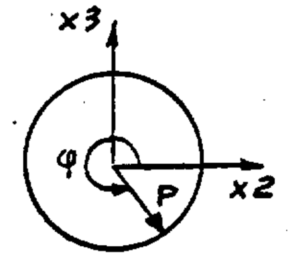
UC 3337 - 0449 - 20 - 1023 Rev. E
UC 3337 - 0449 - 20 - 1024 Rev. D.

The dynamic factors used, account for the dynamic forces caused by the rotating machinery inside the modules and pancakes on the compressor side. The dynamic factors are as follows:

6% for module 30, 31, 33, 34, 35 and 36
5% for pancake 42 and 45
10% for pancake 41 and 44

It is assumed that the dynamic forces are acting in the vertical plane through the X2-axis, and that the direction of the force vectors may rotate as shown on Fig.





Rotating force P
Direction φ

The loading shown on the above mentioned drawings, are the one used in the final computer runs.

However, during the design period the loading has been changed many times. Some of the design calculations are based on earlier revisions of drawing 1021 to 1024.

However, all members are checked for the final loading recieved in November 1976.

In the chapters where a change of loading could influence the chosen dimentions, separate calculations accounting for the new loading have been carried out.

b) Transportation and Installation Phase

The equipment loaded on the Support Frame when transported on one barge from Cherbourg to Åndalsnes is shown on drawings:

UC 3337 0449 20 1061 Rev. C
UC 3337 0449 20 1062 Rev. D

The max. loading when the SSF is supported on two barges in Åndalsnes, is shown on drawings:

UC 3338 0449 20 1063 Rev. D
UC 3337 0449 20 1064 Rev. C.

Check of stresses in the SSF during transportation is performed.

c) Towing Phase

The FRIGG TCP 2 response in floating conditions is evaluated in Høyer Ellefsen Report No. H-E 7504 /22/.

Høyer Ellefsen Report No. H-E 7509 /23/ gives a summary of the conclusions from the above mentioned report.

A spectral analysis of the responce during two-out to the installation field is given in Høyer Ellefsen Report No. 76028 /24/.

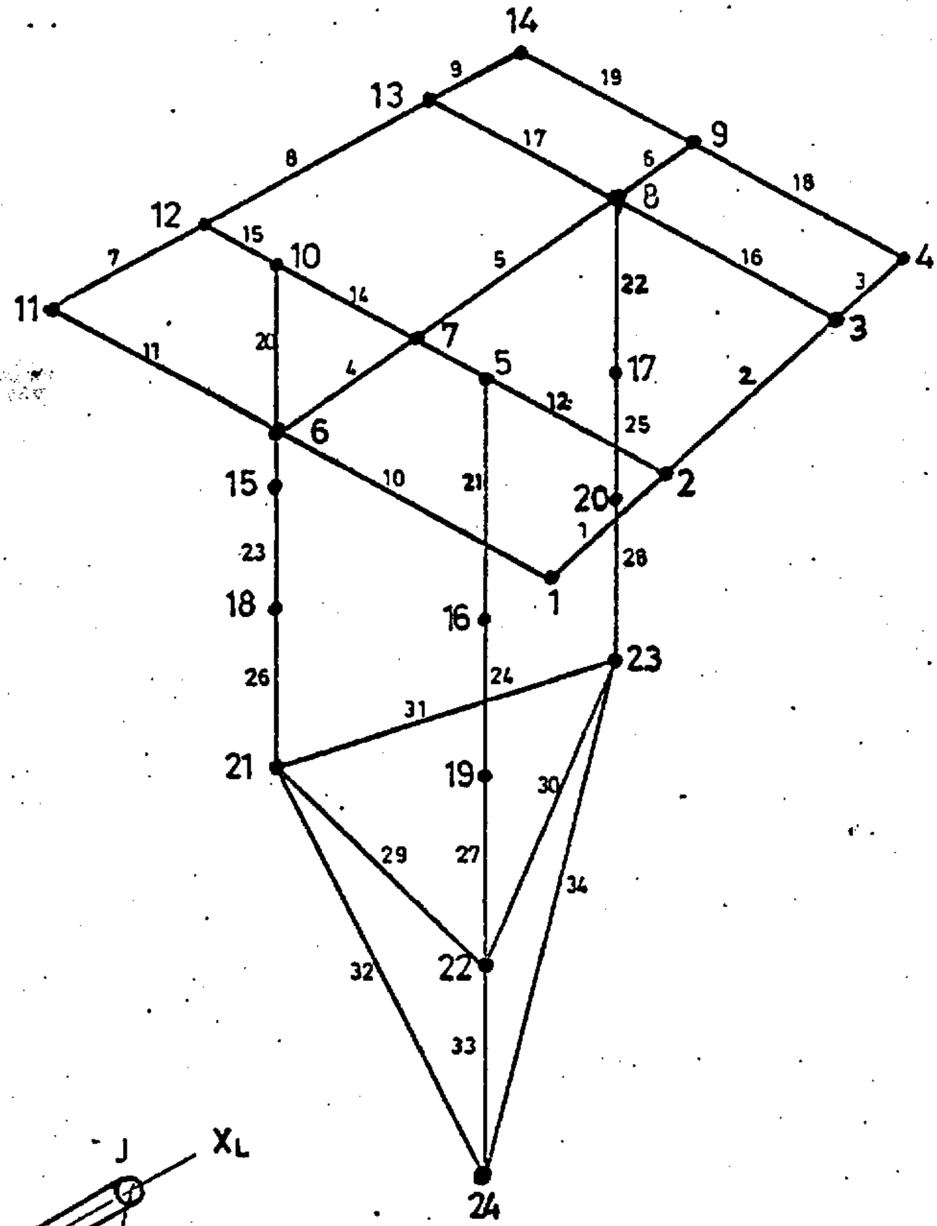
However, the most severe load condition during towing, will occure if water is coming into the structure due to damage. This situation is treated in Høyer Ellefsen Report No. 7525 /25/.

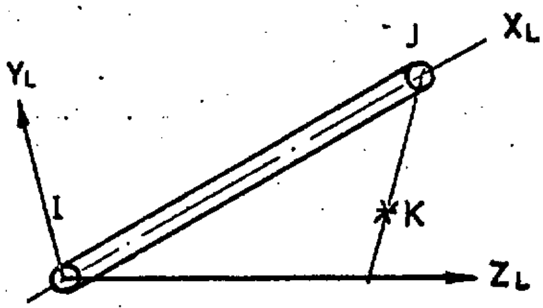
3.5.3 Dynamic Analysis

The dynamic aspects of the Structure are presented in report /19/, /20/ and /21/.

The first report /19/ uses the deck representation shown on Fig. This report presents dynamic amplification factors for bending moment at the base and top of the shafts. The calculation are performed for different wave periods and wave heights. A list of the model data are listed in the report.

Report /20/ compute dynamic amplification factors using the same model with the exception of the soil data. The geometrical soil damping is redused, and for the shorter wave periods the soil shear module is assumed to have a lower and upper bond.

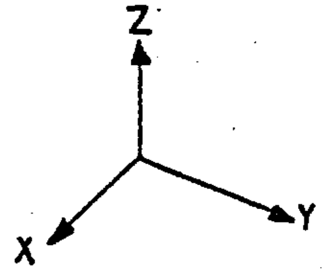




Local coordinate system.

I_y — Cross-sectional moment of inertia about the local Z_L axis.

I_z — Cross sectional m of i about the local Y_L axis.



Global coordinate system

Finite Element Model

Evaluated the above reports in a letter dated 7th October 1975 and arrived at the following values for the dynamic load factors:

	Wave periods T (sec.)					
	2,16	4	6	8	11,2	14
Shaft top moments	7,89	1,89	1,12	1,12	1,17	1,23 (1,13)

For the fatigue calculations the number in bracket for T = 14 sec. will be used.

For the strenght calculations a dynamic load factor of 1,23 is used for both T = 11,2 sec. and 14 sec.

The acceleration of the deck for various wave periods are listed in /21/.

3.5.4 Fatigue

a) General approach

The fatigue life of the steel support frame are calculated using:

1) the stress range theory

2) the relationship between stress range and allowable number of cycles as given in the paper: "A reanalysis of fatigue data for welded joints in steel", Welding Research International, vol. 3 1973, No. 4. (proposal to new fatigue strength calculations in B.S. 153-3B).

3) planned fatigue life = 20 years.

4) the cumulative damage ratio theory (Miners rule).

b) Procedure for the fatigue strength calculations

1. The waves concidered:

The only loads which are assumed to contribute to the stress range are the waveloads.

The stresses are calculated through the whole structure for

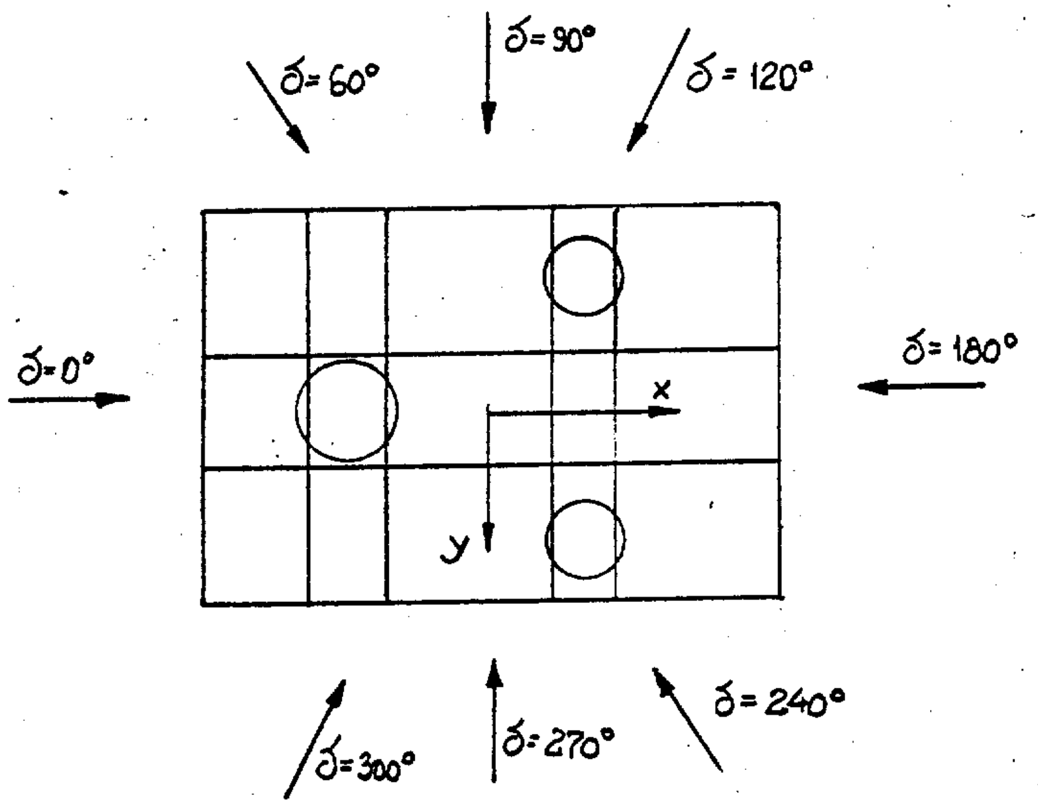
a) the storm wave
(T = 14.0 sec. H = 29.0 m)

b) the operational wave
(T = 11.2 sec. H = 18.7 m)

c) a wave with T = 10.0 sec., H = 12.0 m

d) the wave with period equal the first period of reasonance (T = 2.92 sec., H = 0.65 m).[x]

For wave a) & b) stresses have been calculated for cfr. Fig. B.4.1: $\delta = 0^{\circ}, 60^{\circ}, 90^{\circ}, 120^{\circ}$ and 180°.
For the other wave - conditions mentioned, stresses have been calculated for $\delta = 0^{\circ}$ & $\delta = 90^{\circ}$.



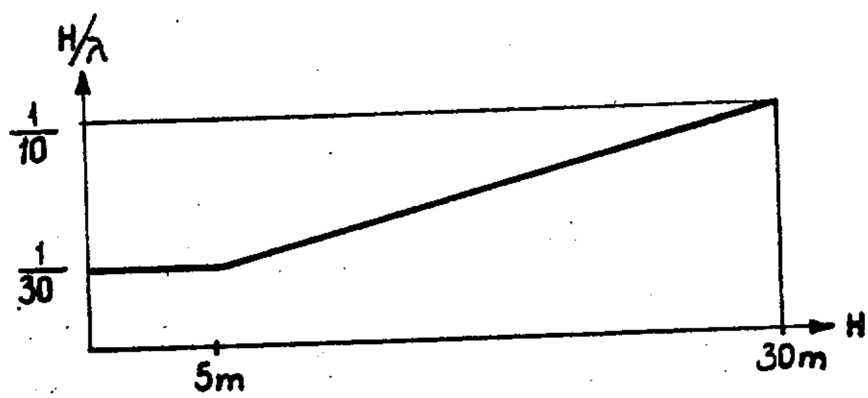
[x] This wave has later been changed to:
T = 2.16 sec., H = 0.36 m.

We assume that the mirror - image of the wave attack directions about x-axis, will give equal stresses in elements laying symetrical about this axis.

With basis in the storm wave the following 8 directions will thus be considered:
$\delta = 0^{\circ}, 60^{\circ}, 90^{\circ}, 120^{\circ}, 180^{\circ}, 240^{\circ}, 270^{\circ}, 300^{\circ}$.

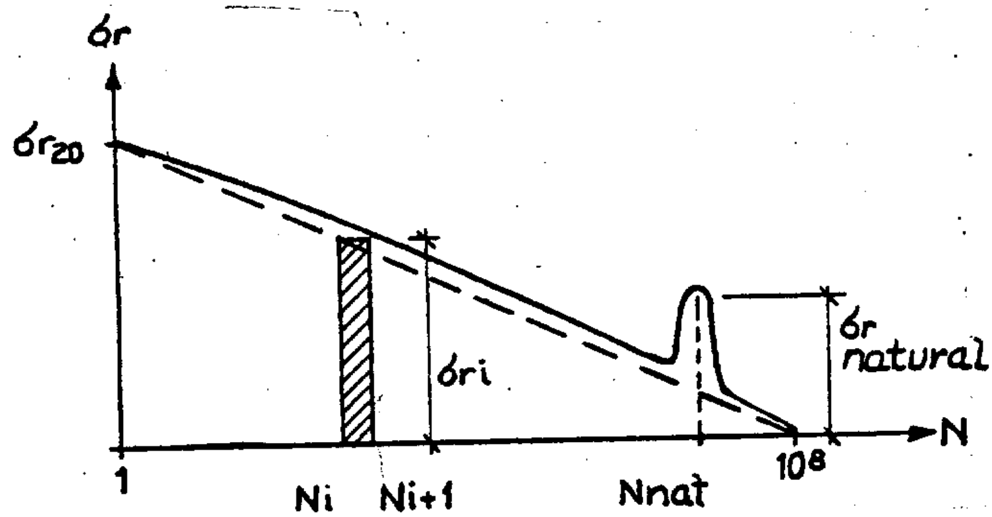
In the present calculation we use maximum stress multiplied by two as the actual stress-range.

All stress-ranges will be increased by the amplification-factor corresponding to the actual wave-height (or period). The relation-ship between wave height and period can be found from Fig. and the formulae below.



λ = wave length
We have from linear wave theory
$\lambda = 1.56\ T^2$.

A typical δ_r/logN-wave will be as shown on Fig.



Resulting long term stress range distribution.

The δ_r/log N-wave will be divided in stress blocks with constant stress range.
Block No. i will contain a number of waves equal to $N_{i+1} - N_1$. For waves with periods equal to or close to the period corresponding to the natural frequency the constant-stressblocks must be small enough to simulate the peak on the δ_r/log N-curve.

The number of waves entering in each direction will be found from the long term wave height distribution and from the metrological report giving the probaility of occurance in each direction.

Allowable number of cycles

Damage ratios

The S-N curves in the paper:

"A re-analysis of fatigue data for welded joints in steel" (Welding Research International vol. 3, 1973, No. 4) give the ultimate number of cycles at a certain stress range. The actual structural detail is to be placed in one of the classes A-G in B.S. 153-3B. depending on the structural configuration, type of stress and type of weld. The definitions of classes A-G and the relevant S-N curves are enclosed.

The X-curve in A.W.S D1-1-72 can be used when stresses are found from a detail analysis (finite element method).

The actual number of cycles in one direction divided on the ultimate number or cycles (n/N) at the actual stress-range, gives the damage ratio. Firstly the ratio n/N must be summed up for all stress blocks (all wave-heights) and secondly for the 8 wave directions considered.

3.5.5 Local Structural Analysis

The global analysis will not give actual stresses in gusset plates, bulkheads etc. For parts of the structure finite element calculations are performed to find the actual stress distribution. In these analyses the forces from the global analyses will be used as the loading input.

Node 410 has been choosen as a typical node. A finite element calculation of this node has been performed in order to establish the general stress level and peak stress for use in the fatigue analysis and facture mechanic evaluations.

3.6 Design Calculations

3.6.1 Introduction

The selected framing and member sizes is based on a lot of preliminary handcalculations and introductory computer runs. The final design cales is based on 3-dimensional structural computer analysis. Detail and local analysis is performed and based on resulting forces and moments from the 3-D analysis.
For storm condition, the AISC allowable stresses is increased by one third. This means that operating condition will govern the design of the members if the stresses due to storm wave and wind forces are less than $1/3 \cdot 0.6\ \sigma y = 0.2\ \sigma y$. Detailed cales has shown that operational condition is govering.

3.6.2 Loads and Load Combinations

The following unit load cases are considered:

a0	=	Dead weight
a1	=	Live Load 1
a1v	=	Live Load 1 incl. vertical comp. forces
a2	=	Live Load 2

b1	=	Operational wave	$\delta = 0^{o}$
b2	=	" "	$\delta = 60^{o}$
b3	=	" "	$\delta = 90^{o}$
b4	=	" "	$\delta = 120^{o}$
b5	=	" "	$\delta = 180^{o}$
b6	=	" "	$\delta = 240^{o}$
b7	=	" "	$\delta = 270^{o}$
b8	=	" "	$\delta = 300^{o}$

d1	=	Storm Wind	$\delta = 0^{o}$
d2	=	" "	$\delta = 60^{o}$
d3	=	" "	$\delta = 90^{o}$
d4	=	" "	$\delta = 120^{o}$
d5	=	" "	$\delta = 280^{o}$
d6	=	" "	$\delta = 240^{o}$
d7	=	" "	$\delta = 270^{o}$
d8	=	" "	$\delta = 300^{o}$

e1	=	Temperature + $30^{o}C$
e2	=	Horizontal compressor forces $\delta = 0^{o}$

Note that:

- The operating wind forces are obtained by multiplying the storm wind forces by $v^{o}_{oper.}/v^{2}_{storm} = 35^{2}/55^{2} =$ 0,405.

- Load case e1 is obtained by assuming a temperature deviation of + 30^{O} between the steel and the concrete structur.

- In load case atv the dynamic forces on the compressor side are acting downward.

Based on the above unit load cases the following load combinations have been generated:

A11+ = a0 + a1v + $1,23^{x)}$ b1 + 0,405 d1 + e1
A11- = a0 + a1v + 1,23 b1 + 0,405 d1 - e1
A12+ = a0 + a1v + 1,23 b2 + 0,405 d2 + e1
A12- = a0 + a1v + 1,23 b2 + 0,405 d2 - e1
A13+ = a0 + a1v + 1,23 b3 + 0,405 d3 + e1
A13- = a0 + a1v + 1,23 b3 + 0,405 d3 - e1
A14+ = a0 + a1v + 1,23 b4 + 0,405 d4 + e1
A14- = a0 + a1v + 1,23 b4 + 0,405 d4 - e1
A15+ = a0 + a1v + 1,23 b5 + 0,405 d5 + e1
A15- = a0 + a1v + 1,23 b5 + 0,405 d5 - e1
A16+ = a0 + a1v + 1,23 b6 + 0,405 d6 + e1
A16- = a0 + a1v + 1,23 b6 + 0,405 d6 - e1
A17+ = a0 + a1v + 1,23 b7 + 0,405 d7 + e1
A17- = a0 + a1v + 1,23 b7 + 0,405 d7 - e1
A18+ = a0 + a1v + 1,23 b8 + 0,405 d8 + e1
A18- = a0 + a1v + 1,23 b8 + 0,405 d8 - e1
A21+ = a0 + a2 + $1,23^{x)}$ b1 + 0,405 d1 + e1
A21- = a0 + a2 + 1,23 b1 + 0,405 d1 - e1
A22+ = a0 + a2 + 1,23 b2 + 0,405 d2 + e1
A22- = a0 + a2 + 1,23 b2 + 0,405 d2 - e1
A23+ = a0 + a2 + 1,23 b3 + 0,405 d3 + e1
A23- = a0 + a2 + 1,23 b3 + 0,405 d3 - e1

x) Dynamic load factor used

A24+ = a0 + a2 + 1,23 b4 + 0,405 d4 + el

A24- = a0 + a2 + 1,23 b4 + 0,405 d4 - el

A25+ = a0 + a2 + 1,23 b5 + 0,405 d5 + el

A25- = a0 + a2 + 1,23 b5 + 0,405 d5 - el

A26+ = a0 + a2 + 1,23 b6 + 0,405 d6 + el

A26- = a0 + a2 + 1,23 b6 + 0,405 d6 - el

A27+ = a0 + a2 + 1,23 b7 + 0,405 d7 + el

A27- = a0 + a2 + 1,23 b7 + 0,405 d7 - el

A28+ = a0 + a2 + 1,23 b8 + 0,405 d8 + el

A28- = a0 + a2 + 1,23 b8 + 0,405 d8 - el

H11+ = a0 + a1 + 1,23 b1 + 0,405 d1 + e2 + el

H11- = a0 + a1 + 1,23 b1 + 0,405 d1 + e2 - el

H15+ = a0 + a1 + 1,23 b5 + 0,405 d5 - e2 + el

H15- = a0 + a1 + 1,23 b5 + 0,405 d5 - e2 - el

LOAD COMBINATION

No	Comb.	No.	Comb.	No.	Comb.	No.	Comb.
1	A11+	11	A16+	21	A23+	31	A28+
2	A11-	12	A16-	22	A23-	32	A28-
3	A12+	13	A17+	23	A24+	33	H11+
4	A12-	14	A17-	24	A24-	34	H11-
5	A13+	15	A18+	25	A25+	35	H15+
6	A13-	16	A18-	26	A25-	36	H15-
7	A14+	17	A21+	27	A26+		
8	A14-	18	A21-	28	A26-		
9	A15+	19	A22+	29	A27+		
10	A15-	20	A22-	30	A27-		

3.6.3 Summary of Load Effects

Beam element forces and stresses for all load combinations are given in Part III K.E. design report.

The operating conditions will govern the design as the stresses due to storm forces are less than 0,2σ.

The program prints the element forces and the stresses for the ends of every beam.

The highly stressed elements are checked manually.

Buckling are investigated seperately for members having high compression stresses.

Drawing UC 3337 0449 20 1053 shows the stress levels for the different members. The high stresses in the members adjacent to shaft B6 should be noted.

3.6.4 Check of stresses

a) Truss Members.

Main Members and Wind Bracing.

Members subjected to both axial compression and bending are proportioned to satisfy Section 1.6.1 of the AISC Specification, /8/:

$$\frac{f_a}{F_a} + \frac{C_{mx}\ f_{bx}}{(1-\frac{f_a}{F'_{ey}})F_{by}} \leq 1.0 \qquad \text{(Formula 1.6-1a)}$$

$$\frac{f_a}{0{,}60\ F_y} + \frac{f_{by}}{0{,}66\ F_y} \leq 1.0 \qquad \text{(Formula 1.6-1b)}$$

Buckling is govern by 1.6 - 1a. In this formula the bending stresses f_b are amplified by the factor

$$\frac{1}{1-\frac{f_a}{fe'}}$$

Depending upon the shape of the moment diagram this factor may overestimate the extend of the moment. To take care of this condition the amplification factor is modified by a reduction factor C_m.

As will be seen later, combined axial and bending stresses will usually be limited by general yielding rather than stability. This means that Formula 1.6-1b would govern the design for most of the members.

For webs and flanges in compression usage factors will be determined according to the DnV-rules /9/.

It is convenient in the calculation to refer to the beam numbers and the node numbers in the computer output. For Details, see K.E. design report, chapter C4.

The following calculation contains a stress control of the beams in the steel structure for latest known loads on the deck. (per 15/8-77).
The loads used in the computer run are given on KE-dwgs. 1021J and 1022H.
The stresses has been listed out for 36 load-combinations. If the beam stress exceeds 2200 kp/cm^2 in the most severe load case, the beam is picked out and a more close investigation is made. (See page).

List of members with stresses > 2200 kp/cm^2 in operational condition

Beam no.	Load comb.	σaxial kp/cm^2	σ' max kp/cm^2	η
115	34	1285	2159	
117	33	1268	2195	
119	12	1217	2111	0.99
121	3	1165	2272	
202	7	509	2281	
209	10	449	2279	
253	14	1650	2359	
255	7	506	2422	
319	7	951	2250	
321	8	945	2522	
322	12	779	2258	
353	2	437	2275	

Beam no.	Load comb.	σ<axial kp/cm^2	σ max kp/cm^2	η
355	3	886	2375	
359	36	953	2444	
361	5	1021	2355	
439	6	1051	2484	
440	12	1033	2212	
605	4	758	2762	
653	1	1625	2269	
705	36	727	2339	
805	33	104	2370	
875	34	1454	2256	
906	8	304	3072	
909	6	644	2795	
910	14	749	2790	
352	12	230	2372	
974	33	350	2531	
976	36	1683	2306	
979	8	1624	2200	
950	10	1215	2274	
981	8	1445	2245	
982	12	1660	2495	
975	36	1517	2170	

The following members is exceeding the $0.66\sigma_F$ criteria in the operation phase, (including high stressed members).

Beam	Node	η
115	500	1.00
117	500	1.02
121	700	1.04
253	730	1.06
255	800	1.00
321	706	1.02
355	207	1.01
359	407	1.00
653	21	1.00
875	501	1.06
909	704	1.00
975	701	1.11
979	705	1.01
981	706	1.10
982	756	1.06

As may be seen from the previous pages, some of the member stresses are exceeding $1,66\sigma_F$ in the operation phase. For two of the members, i.e. beam 975 and 981, the usage factors are 1,11 and 1,10 respectively.

It should be noted that the load combinations yielding these usage factors, includes operational waves, dynamic compressor forces and temperature forces, i.e. a rare event.

The same member have in the storm phase the following stresses:

Beam 975 $\sigma_{max} = 2383$ kp/cm^2

Beam 981 $\sigma_{max} = 2500$ kp/cm^2

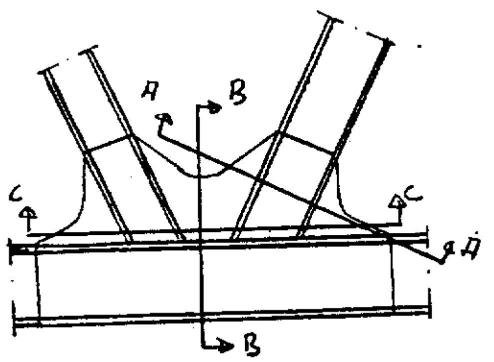
These stresses are far below the permissible stresses for the storm condition ($0,8\sigma_y$ = 2720 kp/cm^2).

The structure are therefore considered safe from a strength point of view.

b) Nodes

The design of the node may be subdivided into the following

i) Gusset plates



The control of the capasity of the gusset plates are devided in five sections:

SECTION 1:
Axial force capasity of the gusset plates at cross section A-A, see Figure, compared with the axial force capasity of the diagonals and verticals. Here is also calculated the necessary height of the gusset plates.

SECTION 2:
Axial force capasity and bending moment capasity at cross section B-B compared with the axial force and bending moment capasity of the diagonal and the chord.

SECTION 3:

Control of shear force capasity at cross section C-C.

SECTION 4:

Data from a finite element analysis of node 410 in Truss Row A.

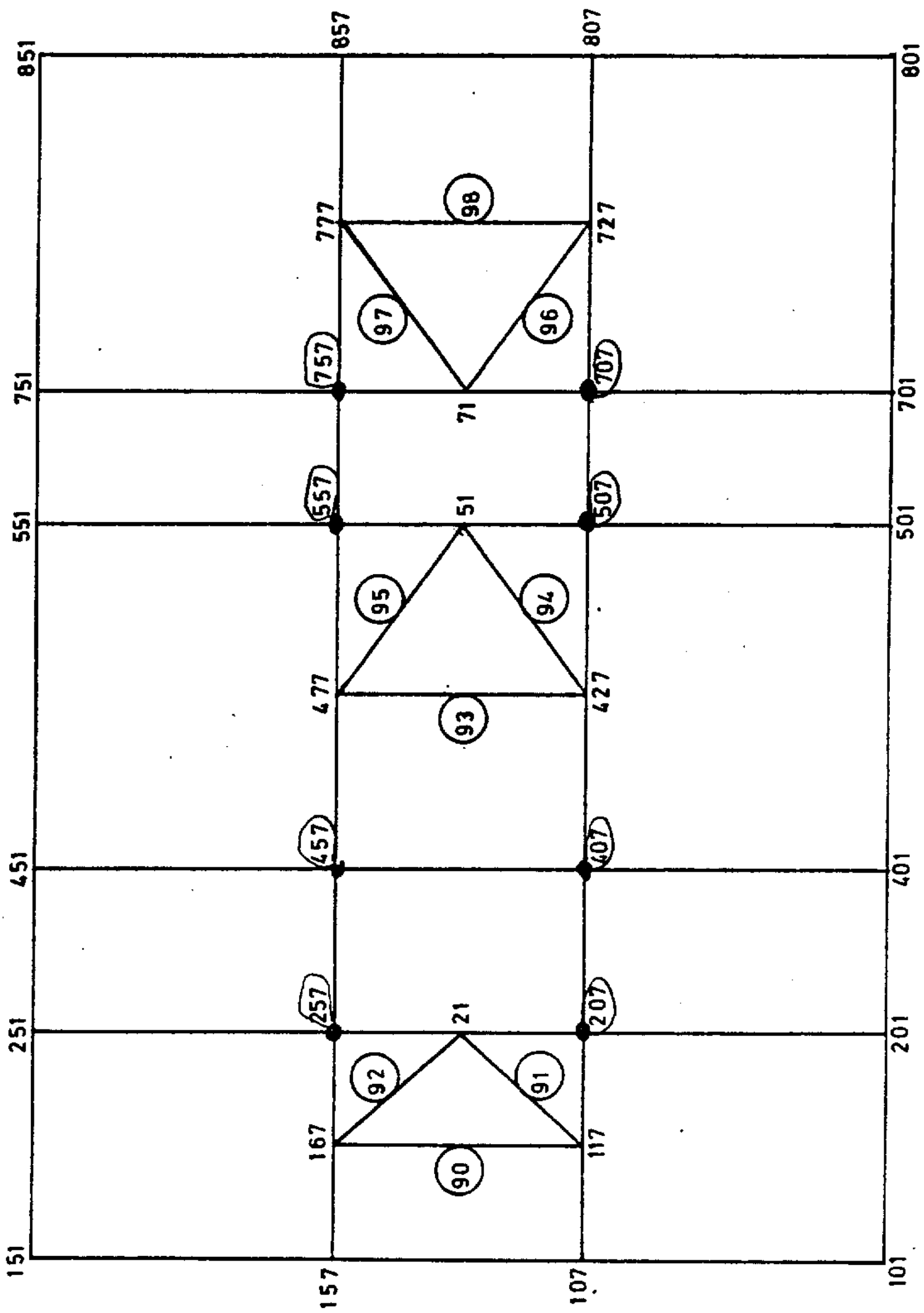
SECTION 5:

A refined capasity check for some typical nodes, i.e. node no. 410, 430, 710 and 600.

All the above evaluation have been carried out with resulting stresses well below the set criteria.
For detail calculation. See K-E, chapter C 4.2.1.

ii) Crossing Flanges

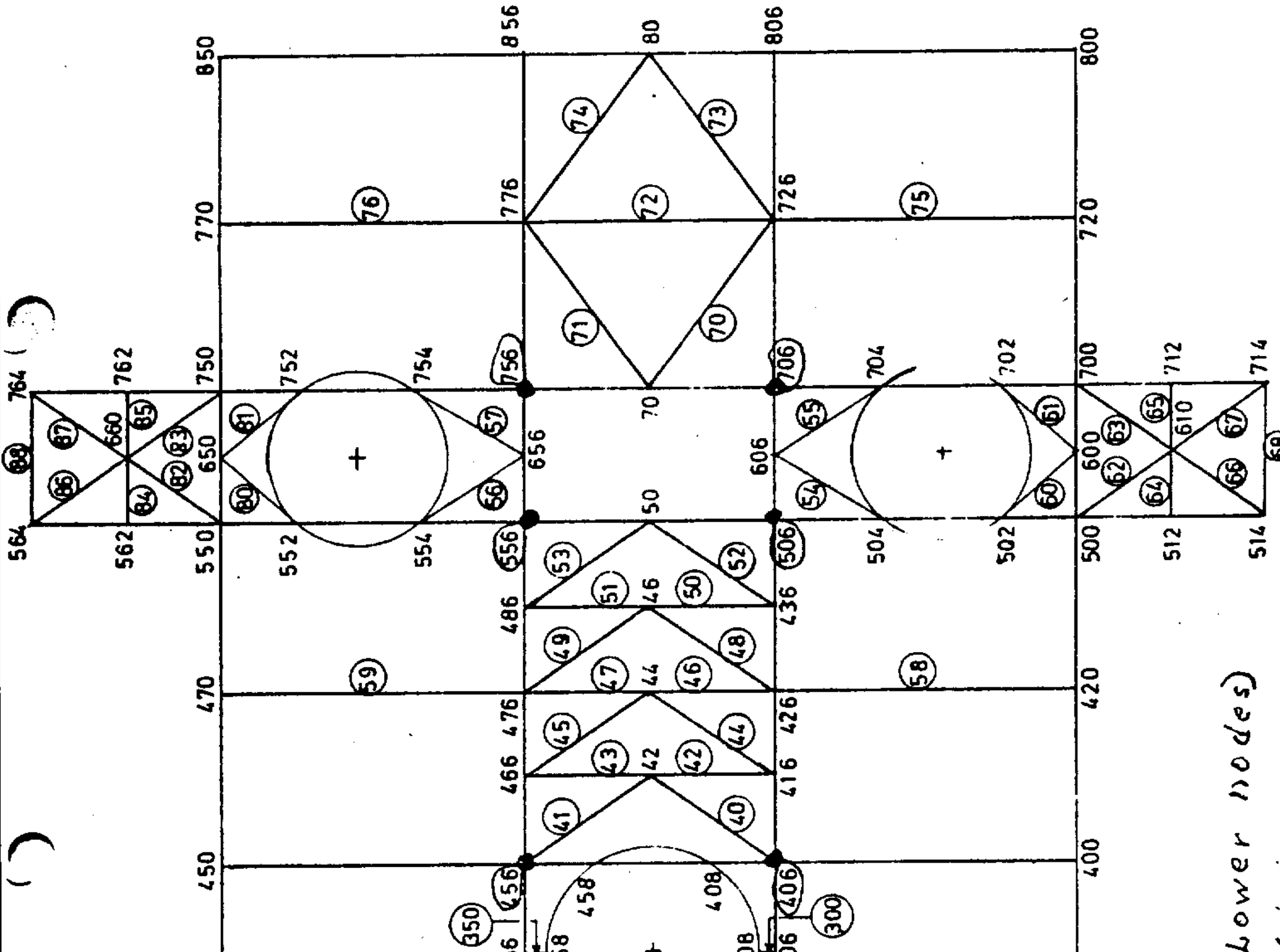
The following corssing flange have been investigated.



SWAY BRACING

MAIN DECK (upper nodes)

● actual calculated nodes



SWAY BRACING

CELLAR DECK (Lower nodes)

● actual calculated nodes.

3.6.5 Design of Rings

General.

The design of the rings, connecting the steel deck to the top of concrete shafts, is mainly based on the output of forces and stresses from the global overall analysis of the steel deck and concrete shafts. The verification of stresses in welds and basis materials is subdevided in 3 parts; the stresses in boxsections, bulkheads and bottomplates. Due to the symmetrical properties of the structure, only ring F6 and D3 are investigated. A typical ring configuration is shown on K.F. drawings in appendix 3:

VC 3337 0449 20 0702
VC 3337 0449 20 0704
VC 3337 0449 20 0708

Stresses in boxsection.

At the points B and E the K-welds ref. fig. 3.6 a) and b), are generally not fully penetrated, and here the welds are seperately checked. For the rest there is made no distinction between fully penetration K-welds and basis materials. The shear stresses (σew) in the K-welds and the von-mises stresses σe in basis material are calculated in accordance with well recognized formulas:

$$\sigma ew = \frac{t}{a} \cdot \sqrt{32}^2$$

t and a is thickness of web and weld τ = shear stress from output of global analysis.

$$\sigma e = \sqrt{\sigma x^2 + \sigma y^2 - \sigma x \cdot \sigma y + 3\tau^2}$$

σx = horisontal normal stress at B, C, D, E.

σy^2 = vertical normal stress at B, C, D, E.

The stresses σx, σy and τ is calculated from the output of global analysis, and $\sigma y \neq 0$ only at nodes where vertical forces are introduced in the rings by beams from the deck. Load cased A11, A12 and A13 all governing, and a rather comprehensive documentation has been given of the stresses.

The shear stresses of the K-welds are not to exceed

$$\tau = \frac{0.66}{2\sqrt{3}} \cdot \sigma_F = 648 \text{ kp/cm}^2.$$

At the nodes 752 and 1512 of ring F6 and the nodes 1302 and 1328 of ring D3 the K-welds are made fully penetrated at locations E and B in order not to exceed the above limit. As regards the von-mises stresses σe in basis material, the webplates of node 752 and 552 is increased from 40 to 50 mm in order to keep the σe lower than 0,66 σ_F.

The von-mises stress of one half or each of the rings F6 and D3 are shown in Fig. 3.6 a) and Fig. 3.6 b).

Stresses in bulkheads.

In the calculation of the vertical bulkheads of the ring, the conservative assumption is made that the bulkheads are to transfer the total shearforce at the sections where the bulkheads are situated. As the bulkheads are designed with a manhole in the middle, a separat finite element analysis of the bulkheads has been performed in order to determine the stress consentrations. In this way the largest von-mises stress in base material found is 1909 kp/cm^2 < 0,66 σ_F and the largest stress in weld at bulkhead is 706 kp/cm^2 < 0,66 σ_F.

Stresses in bottomplates.

The calculation os stresses in bottomplates is devided in two cases, stresses due to maximum compressive and due to maximum tensile forces in the webplates of the ring.

The stresses in the bottomplate due to maximum compressive force in the webplate are calculated by a separate finite element analysis which takes into account the webforce, the prestressed force of the Dywidag bar anchored at the top of the bottomplate, and the concrete shaft on which the bottomplate is situated. The stresses obtained by this analysis are then added to the global stresses due to bending moments and axial forces of the ring. DnV made more detailed finite element analysis of a radial section of bottomplate and top of concrete shaft and from the DnV-analysis of bottomplate and top of concrete shaft, and from the DnV-analysis somewhat lower stresses were obtained.

The results of the K.E. detail calculation of bottomplate are given in Figures 3.6 c) d), which indicate that there are moderate stresses in the bottomplate due to maximum compressive force in the webplate.

The calculation of stresses in bottomplate due to maximum tensile force in webplate has been separated in two different conditions - stiffened and unstiffened bottomplate. The maximums tensile force pr. unit length of the webs has been found from the output of global stress analysis. Combining this for tensileforce with the axial force and bending moments of the ring the maximums stresses are found.

The most severe stressed area is located as shown on Fig. 3.6 d) and the maximum stresses are found to be 2919 kp/cm^2 in storm condition and 2120 in operational condition for ring D3. the corresponding maximum stresses of ring F6, B6 are 2911 kp/cm^2 and 2087 kp/cm^2 respectively.

3.6.6 Buckling control of Plates/Webs

The general control is based on DnV-Rules /9/ section 4, C300 and appendix 2A, B and C.

Klüppel/Scheer: Beulwerte ausgesteifter Rechteckplatten /14/ and P.S. Bulson: The stability of flat plates /13/ are used to complement the above mentioned Rule.

Member forces and stresses are taken from the computer outputs.

The buckling control for operational condition is based on K.E.-drawing.
UC 3337 - 0449 - 20 - 1021F
and
UC 3337 - 0449 - 20 - 1022E.

Consentrated loads from modules/pancakes seating have been taken into account in the evaluation.

Longitudinally buckling stiffeners UNP300 has been applied to the following members (web) in order to meet the set criteria:
members: 103 - 104 - 105 - 106 - 107 - 108 - 109 - 110 -
111 - 112 - 123 - 124 - 125 - 126 - 311 - 312 -
313 - 314 - 323 - 324 - 325 - 326.

The resulting buckling stresses are within the set criteria - (DnV-Rules -74).

3.6.7 Design of Welded connection

Calculation of necessary weld for beam elements, nodes and wip are based on the computer outputs - element forces and moments. A septematic node of the member forces has been carried out, the actual equivalent stresses in the weld calculated and compared with the criteria set in DnV -74 Rules. The stresses is howhere exceeded.
For further details, see K.E. design report C.6.1, C.6.2, C.6.3.

3.6.8 Fatigue Investigation

The procedure for fatigue calculations is outlined in chapter 3.5.4 above, and the following may be added on the general wave statistic used.
It is assumed a semi-log linear relation between the wave heights and the number of times of exceedance.

The dynamic analysis has been performed for six different waveperiods, the following T = 2.16 S, 4.0, 6.0, 8.0, 11.3, 14.80.

In table 1 is shown the relation-ship between waveperiods and waveheights.

The 100 year wave, for which the majority of the static computer analysis is performed, as the basis for further fatigue calculations:

Table 1:

Tp	H	λ	λ/H	$\log_{10} N_0^H$	$\log_{10} N^{100}$	$\log_{10} N^{20}$	T_R	DAF_c	$HDAF_c$
2.16	0.36	7.3	20.3	0.108	8.592	7.892	8.1 sek	7.89	2.89
4.0	1.25	25.0	20.0	0.375	8.325	7.625	14.9 sek	1.89	2.36
6.0	2.81	56.0	19.9	0.843	7.857	7.157	43.8 s	1.23	3.15
8.0	5.0	100.0	20.0	1.500	7.200	6.500	3m 19s	1.12	5.60
11.2	18.7	210.9	11.3	5.610	3.090	2.320	29.7 day	1.17	21.9
14.0	29.0	321.4	11.1	8.700	0.000	0.700	100 yr.	1.3	32.8

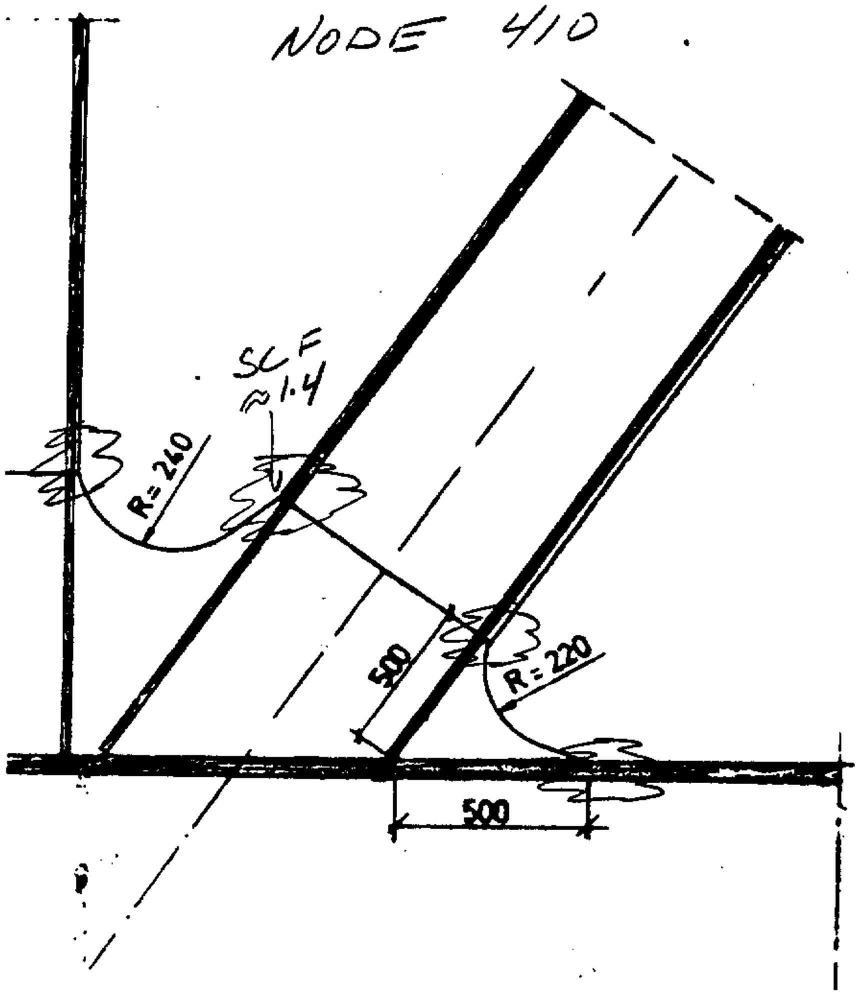
Taking into account, dynamic amplification factor, probability of occurance of waves from different directions, and based on the general conservative design assumption, the following relationship resumé may be presented for the usage factors for different stress ranges (corresponding to 100 ya. prob. level) and fatigue curve G.

σr	η
400	0.004
600	0.05
800	0.28
1000	0.52
1040	0.59
1200	1.25
1300	1.62
1400	2.42
1500	3.23

σr	η
1600	4.01
1700	4.93
1800	5.78
1900	6.78
2000	7.85
2100	9.03
2200	10.29
2400	13.21
2600	16.64

i) Fatigue cale and evaluation of the Truss Members.

The more critical fatique area for the members are at the ends of the gurset plate.
A F.E. calculation of a typical node i.e. node 410 has been performed.
A stress consentration factor (ScF) at the end of gusset plate of order 1.3-1.4 has been evaluated.



The following elements have η ≥ 1.0 when using fatigue curve G and SCF ≥ 1.30.

Elno	η_G	Elno	η_G	Elno	η_G
54	0.23	423	1.89	879	1.73
55	0.31	424	1.89	880	1.47
56	0.20	775	1.30	881	0.74
57	0.25	776	1.31	882	0.76
307	0.88	809	3.86	909	2.06
308	0.92	810	3.59	910	1.82
309	1.02	855	0.52	977	0.96
411	0.97	875	1.18	979	1.55
412	0.98	876	1.08	980	1.64

The structural detail considered on the elements - end of gusset plate - may be classified according to SN-curve F.
I.e. elements do have design life > 20 years when $\eta_G \leq 2.50$.

By a detail investigation into the members 809/810 and 909/910, taking into account the actual bending moment distribution at the gusset end, the following usage factors may be derived at

member	η_G
809	1.62
810	1.40
909	0.70
910	0.63

i.e. They are all satisfied $\eta_G \leq 2.50$.

The SN-curve used theory:

SN-curve F: $\log N_F = 14.82 - 4.0 \log \sigma_R$ (kp/cm^2)
" " G: $\log N_G = 14.42 - 3.0 \log \sigma_R$ (" ")
" " W: $\log N_W = 14.22 - 3.0 \log \sigma_R$ (" ")

The more critical welds in the elements are checked using SN-curve W.
No critical weld has been found.

ii) Fatigue Analysis of steel Support Rings.

A similar procedure as for the mode and members is adapted also for the analyse of the ringnodes.
The fatigue strength is well below the set criteria, ref. 3.5.4.

3.6.9 Design of Attachments

This include design of

a) Pancake Supports and External Contition Beams.
b) Bridge landing.
c) Structural Deck.
d) Bulkheads.

Re. a) Pancake Supports and External Condition Beams.
This has been designed, based on K.E. drawings of load data.

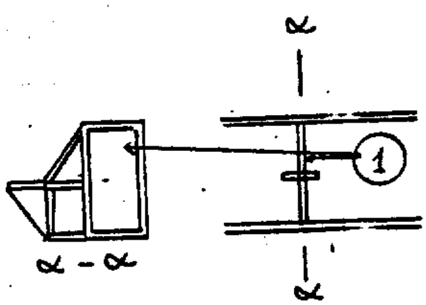
Where impact factor of 2 have been included in the "setting down" condition.

b) Bridge landing.
A separate computer analysis has been performed. Reaction forces and moments is given by the Dermott Hudson, a detail study is presented in K.E. Design report chapter C 10. The stresses are found well below the set design criteria.

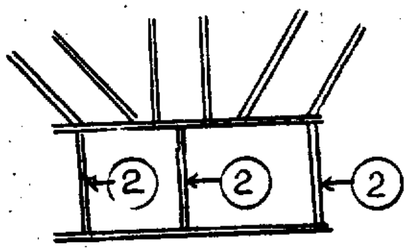
c) Design of Structural Deck.
The design of deck has been performed based on a consentrated load from truck = 2.5 T and a uniform load of 1.8 T/m^2. Reference are also made to Conference Note from McDH 29th April 1975. The design criteria is not violated in the design of structural deck.

d) Bulkheads.

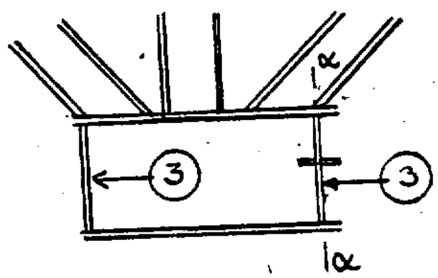
The bulkheads are devided into follows types:



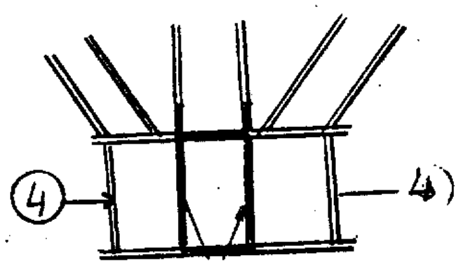
1 The bulkhead of contiliver - see Design of pancake support.



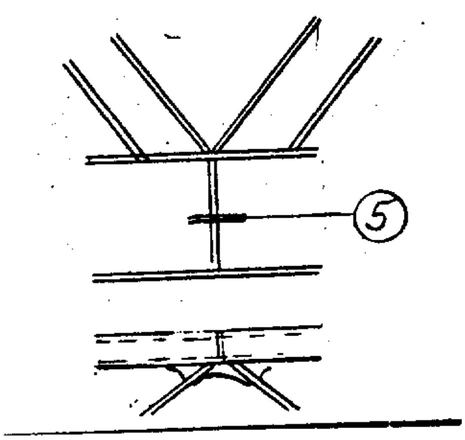
2 The external bulkhead of node.



3 The external bulkhead of node with contieiver.



4 The external bulkhead of cross node.



5 The external bulkhead connected to lateral bracing.

A systematic study of the different bulkheads has been performed. Unit respons of bulkheads have been evaluated based on a finite element analysis. Forces and moments from the Stardyne global analysis have been extracted and stresses in bulkheads derived from unit loads scaled according to actual forces/moments.

The stresses derived at were within the set criteria.

3.6.10 Transportation and Towing

This may be subdivided into the following phases.

a) Deck supported on one Barge.

This include two separate transport.

i) Transport from Dunkrik to Cherbourg of part of the structure (approx 3000 tons).
This transport took place on "Vikbarge".
Interaction between barge and deck when evaluated and taken into acount with the design of sea-fastening.

ii) Transport from Cherbourg to Åndalsnes.
This transport was performed by "Seapontoon IV".
Approximate weight deck during this transport = 4800 tons.
The loading on top of supports during sea transport is shown on K.E. drawing.
UC 3337 0449 20 SK-069 B.

The load combinations
1) sagging /hogging and pitch
2) heave and roll.

The interaction and hence reaction forces during transport were derived at by a motion respons analysis and with the criteria as given in 3.4.2 c).
The design revealed no overstressing during this transport.

b) Deck supported on two barges.

The deck were transported from Seapontoon IV to the two barges Norbarge I & II at Åndalsnes before deck mating. The strength of the deck structure were checked when supported on two barges. Total weight of deck at deck mating were of the order 8600 tons.

The seafastening were checked to with stand a horizontal acceleration of 0.1 g when deck supported on two barges at Åndalsnes.

4.0 FABRICATION RESUMÉ

4.1 General

4.1.1 Specification

Shop fabrication, assembleying and construction of the various parts of the support frame have been based on:

a) Elf Norge's Fabrication Specification 1052-No. 3/155 Rev. 2/JPS of Febr. 1974.
b) Minutes of meeting Elf, DnV and Norwegian Contractors at Elf-Norge, Paris office 4.11.75.

These documents list accepted reference codes/standards, rules and technical notes and gives detailed guidance and requirements to preparation of structural members, materials, fabrication, tolerances, welding, inspection etc.

Welding has in general been based on AWS D.1.1-1972 "Structural welding code" with following supplementary requirements:

- Qualification of welding procedures in accordance with DnV's Technical Note Cl/2 (1976) with additional impact testing of root region of heavy section joints. Butt-welds and T-joints were required qualified separately. In PWHT condition the test temperature for impact testing were redused to -20°C based on COD evaluation.
- Qualification of welders in accordance with DnV's Technical Note Cl/3 (1976) with additional performance plate test in 2G-position.
- Fracture mechanics testing of welded joints in accordance with British Standard DD19-1972 "Crack opening displacement tests" using a testing temperature of -10°C.

4.1.2 Method of fabrication

The fabrication of the SSF can be split up as follows:

1. Prefabrication of nodes and members.
2. Assembly of nodes and members into sections.
3. Assembly of sections into parts.
4. Assembly of parts into SSF.

The fabrication of nodes, elements and sections was separated into 3 major portions of the deck with the following designation and fabrication responsibility.

WHITE PART	AMV	Joint Venture
RED PART	CMP	AMV/CMP
GREEN PART	CMP	

The Red part, including Central Square of White part, was assembled by CMP at Mardyck and transported on one barge to the UIE yard at Cherbourg.

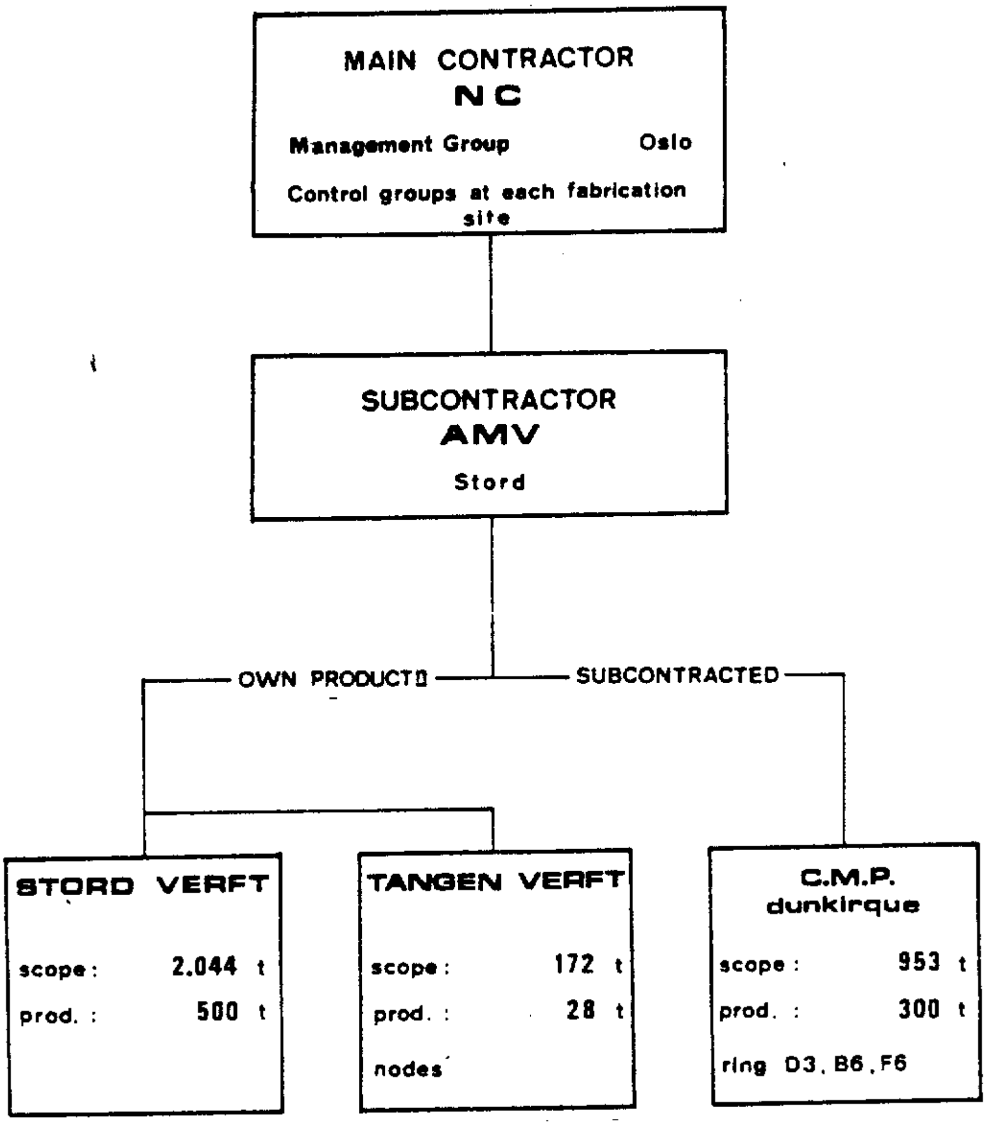
Remainding White part, together with Green part, was shipped as nodes/elements and sections from fabrication sites in Norway and France respectively to Cherbourg for the final assembly.

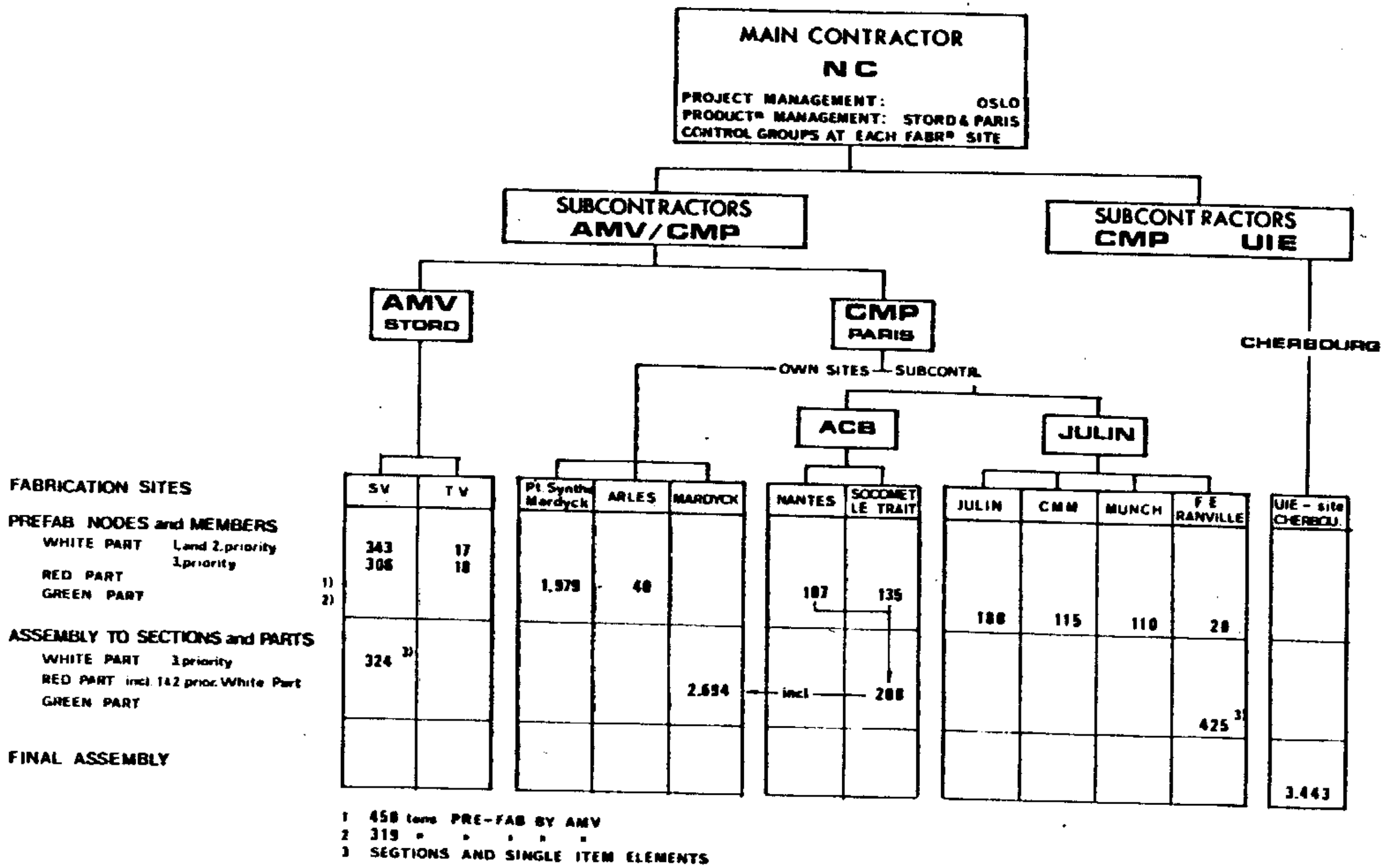
The responsibility for these transportations has rested with the Joint Venture AMV/CMP.

Final assembly at UIE's yard in Cherbourg has been performed under the responsibility of CMP whilst UIE had as its responsibility all loading, positioning and transportation within the yard area in Cherbourg, including the load-out of the completed support frame into the transportation barge. The different parts are shown in Appendix 4, Fig. 4.1 a) b) and c).

4.1.3 Organization

Phase I Nov. 1975 - June 1st., 1976.



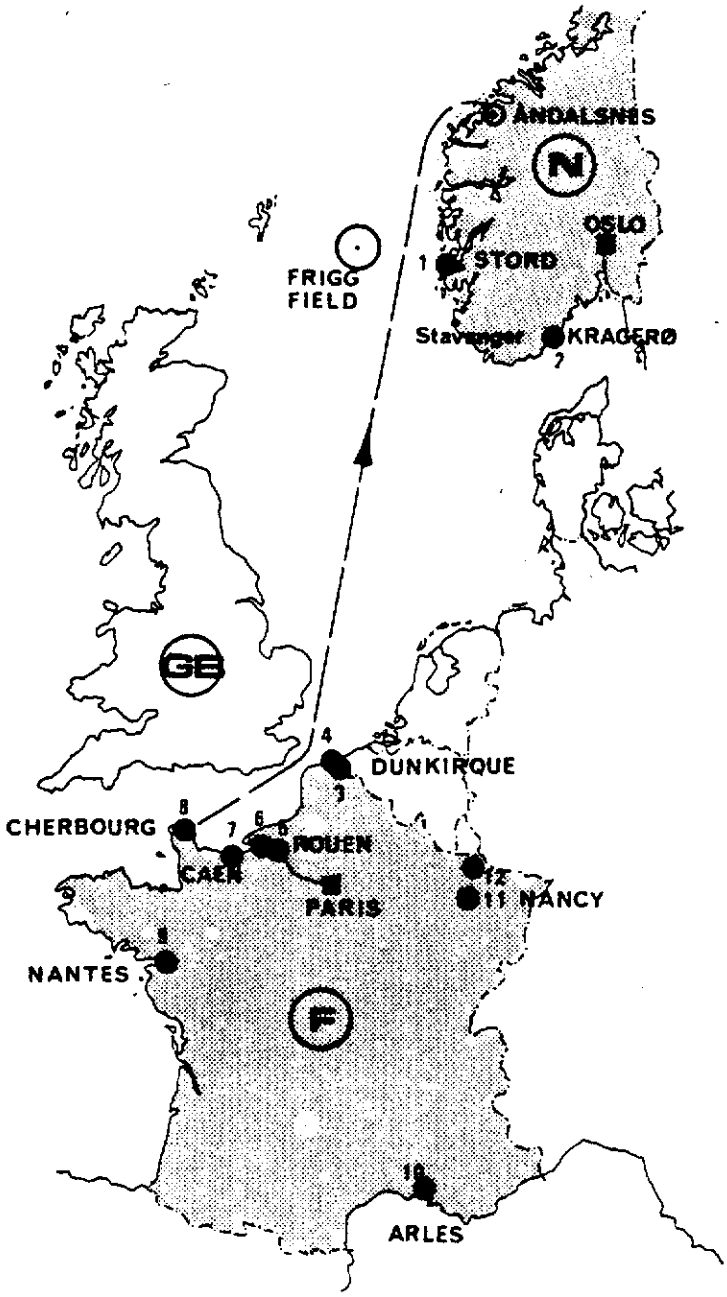


1) 450 tons pre-fabricated by AMV

2) 319 tons " " "

3) Sections and single item elements

4.1.4 Production Sites



PRODUCTION SITES

1 AMV STORD VERFT (SV)
2 AMV TANGEN VERFT (TV)
3 CMP, Petit Synthe
4 CMP, Mardyck
5 JULIN
6 SOCOMET Le Trait
7 FE Ranville
8 UIE
9 ACB
10 CMP
11 MUNCH
12 CMM
13 Åndalsnes

4.1.5 Main Transport Phases

The following main transports were carried out successfully.

Transport From	To	Of	t Weight	Type of vessel Name/Dimension/Cap	Period	Resp.
STORD	MARDYCK	PREFAB NODES AND ELEMENTS SURPLYS MATERIAL FOR RED & GREEN PART	2900	SUPPLY SHIP	JAN & JUNE 76	AMV
STORD	MARDYCK	1 & 2 PRIORITY OF WHITE PART	360	SUPPLY SHIP	OCT 76	AMV
SOCOMET LE TRAIT	MARDYCK	PART OF RED PART	208	BARGE: 'Abeille 601' 60 mx19mx4.5m 3300 TONS TUG: Abeille 15 3500 IHP	OCT 76	CMP
RANVILLE	CHERBOURG	GREEN PART SECTION & ELEMENTS	425	CARGO SHIP	NOV/DEC 76	JULIN
MARDYCK	CHERBOURG	RED PART INCL 1 & 2 PRIORITY OF WHITE PART	2694	BARGE: "Vikbarge" 914mx27.4mx 6.1 m 9400 TONS TUG: 'Abeille 30' 8000 IHP	JAN 77	CMP
CHERBOURG	ANDALSNES	COMPLETED SSP INCL. 1500 T of EQUIPMENT	5000	BARGE: "Seepontoon 4" 114mx30mx6.1m TUG: "Musketer Fury" "Musketer Fighter" 2 x 8000 IHP	MARCH 77	NC

4.2 Inspection/Quality controll.

The basis for DnV's inspection at the 13 different production yards for the Steel Support Frame, has been the Yard, the Oil company and Contractors own Quality controll Organization.

A contious effort in assuring that the yards, oilcompany and contractors QCO has been working satisfactory, were emphezised throughout the production.

All the fabrication and inspection performed were essentially based on the criteria laid down in the material and fabrication specifications pertinent to the project - see chapter 4.1.1 above.

DnV surveyors attended the fabrication at all the yards involved. On the main assembly yards - in Stord and Dunkirk - as well as at U.I.E. Cherbourgh and during deck mounting at Åndalsnes. DnV followed the work continuously by surveyors specifically assigned to the project. On the other yards performing prefabrication of structural components, DnV's surveyors attended the fabrication to such an extent as found necessary in each specific case.

Welding has been performed with the manual metal arc and the submerged arc welding processes only. Welding consumables have been of low hydrogen type.

Welding procedure tests were performed at each fabricator in question. Fracture mechanics testing of procedure welds were restricted to the main contractors, Stord Verft and CMP, while subcontractors were required to perform conventional procedure tests using welding consumables previously selected based on the fracture mechanics tests.

Design of welded connections, joints and splices and fabrication tolerances: AWS D1,1-72 and AISC "Steel construction manuals".

Post weld heat treatment was carried out in general for all joint thicknesses 50 mm and above with excemption for splice welds of 50 mm thickness when the maximum tensile stress was less than 1000 kg/cm^2. Heat treatment was carried out at temperature 550-600^{O}C/1 hr per 25 mm with controlled heating and cooling as per ASME VIII for furnace treatment. Local pwht of field and splice welds was performed by electric heating for a width min 2-3 times thickness either side of the weld with temperature difference at line of symmetries kept within appr. 30^{O}C during the heat treatment cyclus.

Non-destructive testing consisted of visual examination and either radiographic, ultrasonic or magnetic particle testing depending on the joint in question and the most suitable method to detect, locate and assess probable weld defects. The initial inspection was 100% (full length) and then relaxed to the following level when quality level found satisfactory: See Appendix 4.

Non-destructive examination was generally based on the fabrication code AWS D.1.-1-72. except that ultrasonic testing was changed to follow ASME pressure and Vessel Code Sec. VIII since former code is not suited to inspection of T-joints and heavy section welds.

Each finished section and complete assembly was regularly dimensional checked.

Production test welds were not performed.

It was generally concluded from the surveys carried out that the quality control systems worked satisfactory and that fabrication and assembly work was carried out under proper supervision and in accordance with relevant specifications, procedures and drawings. The quality of the work was found to be satisfactory and the extent of NDT inspection was found to be in compliance with that set forth DnV carried out spot checks of the NDT inspection performed by others. These spot checks were carried out by DnV's own NDT personell.

Some of the defects discovered were found in complicated structural components involving great thicknesses, high heat input and large residual stresses.

The relevant members and joints were, as mentioned above, subjected to rigorous NDT inspection and grinding as well as furnace post weld heat treatment where possible, otherwise pwht only local. In addition to the testing specified in the fabrication specification, also COD testing was adopted for approval of the welding procedures pertinent to these heavy thicknesses.

Geometrical measurements were also conducted on the structure overall as well as on individual components and members.

Material marking and logistics routines were also found to be satisfactory.
ELF in cooperation with the main contractor and yards kept control of the materials until finally placed in the structure.

From the records taken it is possible to trace the material certificates for each primary structural component in the Support Frame.

A copy of the documents produced for this purpose was handed over to the DnV surveyors in Cherbourg, Julin and Stord upon completion of the work a complete file of all records, drawings, certificates etc. pertinent to the platform is kept as indicated in Chapter 6.0. This file shall be available to the Authorities involved.

The load-out operations and installation of tie-downs for the Deck Support Frame were attended and surveyed by DnV and found to be in compliance with appropriate and accepted procedures and drawings.

DnV head office personell involved in project coordination and design review visited the main fabrication yards at several occations in order to ensure a proper flow of information between the parties involved and in order to ascertain that the design intentions were properly taken into account during fabrication and inspection. Drawings showing the most critical areas and joints were supplied to the local DnV surveyors in order that they could consentrate on the most important areas and joints and thereby ascertain that they were adequately taken care of during fabrication and inspection.

The final reports from DnV's inspection at the main yards - in Stord, Cherbourg/Dunkirk, Julin and Åndalsnes - are attached in Appendix 7 to this report.

4.3 Brittle Fracture and postweld heat treatment

The support frame is a welded truss structure with plate thicknesses up to 100 mm. In order to assess the risk of brittle fracture in the heavy section welds, it was at an early stage decided to perform fracture mechanics tests using the COD-technique (Crack Opening Displacement tests). Such tests have been carried for joint thicknesses 100 mm - 80 mm - 60 mm - 50 mm the smaler joint to determine whether adequate toughness could be achieved without post weld heat treatment.

Sample joints for COD-testing were produced at both Stord Verft and CMP following the welding procedure specifications established for production welding.

The following main parameters were evaluated:

a) Welding consumables
b) Effect of post weld heat treatment
c) Position of notch.

The COD-tests were performed at -10^{o}C which was considered to be the lowest probable air temperature at the Frigg field. The tests were carried out at the Laboratories of DnV in Oslo and at Institute de La Soudure, Paris in the presence of DnV, NC and Elf represntatives. For each serie and condition at least 3, test sperimens were performed, and the lowest COD-value, δ_c, was used to calculate the defect tolerance parameter, $\bar{a}_{max}$.

Table 1 (Appendix 4) summarizes the results otbained for the various conditions tested: All together 25 test series were run.

The defect tolerance parameter have been calculated as follows:

$$\bar{a}_{max} = \frac{\delta_c}{2\ \pi\varepsilon_{yw}(\frac{e}{e_{yw}} - 0.25)}$$

where ε = total applied strain = $e_{residual} + e_{external}$

ε_{yw} = local yield strain at region of notch.

The external applied strain were evaluated from finite element analysis resulting in a local stress of max. 1.2 on the yield stress.

Selection of welding procedures aimed at conditions at which a defect toleranse parameter of 12 mm or more would be obtained. This value was specified to give confidence that brittle failure would not initiate from weld defects missed during standard techniques of non-destructive testing or which could potentially grow to critical size because of fatigue. A smaller defect tolerance than 12 mm was only accepted subject to improved non-destructive testing technique and evaluation of stress levels for the joints in question.

Based on these tests the following conclusion was made:

- For full penetration welded joints of thickness 60 - 80 10 mm post weld heat treatment was required to achieve acceptable resistance to initiation of brittle fracture.
- Full penetration weld joint of 50 mm. Provided the welds were ground smoth to facilitate a more detailed N.D.T. testing, the nodal joints having a nominal stress level of less than 1000 kp/cm^2, or joints displaced 1 m from critical node regions, with nominal stress level less than 1500 kp/cm^2, did not require post weld heat treatment in order to achieve acceptable resistance to brittle fracture.

- For welded joints of 50 mm thickness but stressed in compression, pwht would be waived.

The following welding consumables were selected for welding of the heavy section joints:

Condition	Manual metal arc welding	Submerged are welding
PWHT	Fortrex 8018 Cl OK 73.68	Commercy SP 1500/880 SD3/OPTT 41
As welded (50 mm fiel joints)	Fortrex 8018-Cl	-

The resulting nodes and field joints which where stress relieved are shown on the following drawings, enclosed in Appendix 4.:

UC 3337 0449 20 1071 Rev. A
UC 3337 0449 20 1072 Rev. A.

Table 1: Fracture mechanics tests - TCP-2

Welding consumable Brand	Joint details Groove Pos.	Specimen Size Steel	Testing Cond. Temp. °C	As welded /PWHT	Position of notch	COD δc (Wells) (Pop-in[x])	Fabricator Date	Report ref.
OK 73.68	K-groove 36	100 x 100 SHSS 40	-10°C	As welded	Weld metal	0.043[x] - 0.104 0.055[x] - 0.048[x] 0.23[x] - 1.39	Stord Verft	DnV
					HAZ	1.47 -> 1.54 - 1.60	January 1976	571674/1
			-10°C	PWHT	Weld metal	0.14 - 0.31 - 0.57		
					HAZ	0.72 - 1.98 - > 2.2		
OK 48.15	K-groove 36	100 x 100 SHSS 40	-10°C	PWHT	Weld metal	0.064 - 0.241 -0.383	CMP-Arles	
					HAZ	0.377 - 0.365 -0.707		
Fortrex 8018-Cl	K-groove 3G	50 x 50 SHSS 40	-10°C	As welded	Weld metal	1.03-0.046-0.35	CMP-Dunkirk	Institute de la
				PWHT	Weld metal	0.32-1.50-0.56	May 1976	Saldene no. 5896
Phillips	K-groove 3G	100 x 100 SHSS 40	-10°C	PWHT	Weld metal	0.435 - 0.466 -0.367	CMP-Dunkirk	I.S. 6111
					HAZ	2.05-0.64-1.72		
Commercy SP 1500/	K-groove 1G	100 x 100 SHSS 40	-10°C	PWHT	Weld metal	2.14 - 2.25 -2.29	CMP Dunkirk	1
Flux Linoln 880					HAZ	0.76 - 0.94	Okt. 1976	

Welding consumable Brand	Joint details Groove Pos.	Specimen Size Steel	Testing Cond. Temp. °C	As welded /PWHT	Position of notch	COD δc (Wells) (Pop-in^x)	Fabricator Date	Report ref.
Fortrex 8018C1	K-groove 3G	60 x 60 SHSS 40	-10°C	As welded	Weld metal	0.54 0.20 1.12 0.21	Stord Verft Febr. 1976	DnV 571674/2
Fortrex 8018C1	X-groove 3G	60 x 60	-10°C	As welded	Weld metal	0.88 0.29 0.16 0.087	Stord Verft Febr. 1976	DnV 571674/2
Fortrex 8018C1	K-groove 3G	50 x 50 SHSS 40	-10°C	As welded	Weld metal	0.04^x - 0.15	Stord Verft	DnV 571674/3
				PWHT	Weld metal	0.84-0.87-1.20	Febr. 1976	
Fortrex 8018C1	K-groove 3G	60 x 60 SHSS 40	-10°C	As welded	Weld metal	0.17-0.196-0.51	Stord Verft	DnV 572040/1
				PWHT	Weld metal	0.33-1.44-1.45	April 1976	
Fortrex 8018C1	K-groove 3G	80 x 80 SHSS 40	-10°C	As welded	Weld metal	0.098 - 0.079^x -0.14	Stord Verft	DnV 572040/1
				PWHT	Weld metal	0.05->2.05-1.28	April 1976	
Fortrex Murex	K-groove 3G	100 x 200 SHSS 40	-10°C	As welded	Weld metal	0.376-0.056 0.164	Institute de la Soudure Jan. 1976	I.s 5908
Oerlikon Tenacito 70B	K-groove 3G	100 x 200 SHSS 40	-10°C	As welded	Weld metal	0.035 - 0.10 -0.074	Institute de la Soudure Jan. 1976	I.S. 5908
Esab OK FU	K-groove 3G	80 x 160 SHSS 40	10°C	As welded	Weld metal	0.118 - 0.064 -<<6.06^x	Institute de la Soudure	I.S. 5908
SD3 wire/TT41	X-groove 1G	80 x 80 SHSS 40	-10°C	As welded	Weld metal	>1.8 - 0.88- 0.35	Stord Verft	DnV report 76-313
				PWHT	Weld metal	>1.8->1.8->1.8	July 1976	

4.4 Repairs

a) Ring nodes 208/258.

Repeated cracking in the weld connections of node 208/258 (produced at CMP, Petite Synthe) were observed after stress relieving. The crack were located in the weld metal, below the surface, and with length up to 800 mm. A detail discription of the cracking is given in telex d.d. 23/6/76 no. 4692. (Appendix 4).
The major repairs in nodes 208 and 258 were welded with Fortrex electrode 8018-C1 without further PWHT.

The ultrasonic control was carried out using 4 mHz 45°, 60° and 70° probes type MWB and a 4 mHz normal probe type MB4S.

The examination was carried out from both the outside and inside faces of the plate material and the repairs were found to be acceptable, only a few widely scattered minor inclusions being found.

b) Ring nodes 408/458.

Cracking in the weld connections of node 408/458 (produced at CMP, Mordyck) were discovered after PWHT. Crack lengths 30 mm, and depth 20 mm to 30 mm from outside also defect 700 mm at a max. depth of 17 mm were recorded. The repairs were welded with Fortrex electrode 8018-C1 without further PWHT.

The ultrasonic control was carried out using 4 mHz 45°, 60° and 70° probes type MWB and a 4 mHz normal probe type MB4S.

The examination was carried out from both the outside and inside faces of the plate material and the repairs were found to be acceptable, only a few widely scattered minor inclusions being found.

c) Deflection after jacking at node 156 and 106.

After jacking at Cherbourg yard, several indents at lower flange truss row C and E at node area 106 and 156 were revealed. Depths up to 29 mm were recorded. (See UIE sketch Appendix 4).
The indents where compensated by doubling plates.
Official drawings of doubling plates are not yet issued.

d) Overheating of node 755.

During straigtening operations, the node 755 were recorded overheated.
A metallographic examination of heat-affected material of the node were carried out, concluding that surface material 2 - 2.5 mm depth were to be ground.
For detail - see DnV-report no. 15 1292/76.

e) Deformed upper chord.

Deformation of web plate of upper chard where recorded after module 01 placing.

Weld between member 160 and node 451, both sides in the box are deformed up to 25 mm laterally.
At the presant state, the indents are judged to have little influence on the capacity.

f) Deformed lower chord.

Some buckling of webs in the bottom chord was observed in areas a + 3 m and g - 3 m in line 5 and 7.
The buckling were reinforced by extra longitudinal stiffeners as shown on K.E. drawing
UC 3337 0449 0990 0.

5.0 INSTALLATION

5.1 Steel support frame (S.S.F.)

The steel support frame was succesfully installed from the two barges (Norbarge I & IV) onto the concrete shafts in the fjord of Åndalsnes in spring 1977. The deck was fixed to the concrete colums by means of 427 posttentioned anchor/bolts (Dyvidag). The average bolt hole deviation at mating was 1.8 cm with a max. of 3 cm, well below tolerances.
The deck mating was carried out according to predetermined procedure issued by NC/H.E.

Deck weight at arrival Åndalsnes	4812 tons
Of this steel weight S.S.F	3443 tons.
Deck weight before deck mating:	8643 tons.

The net weight of module/equipment at arrival Åndalsnes and at deck mating is given on K.E. drawings:

UC 3337 0449 20 1061 C
UC 3337 0449 20 1062 D
UC 3337 0449 20 1063 D
UC 3337 0449 20 1064 C.

5.2 Modules

The following modules/pancakes and equipment were installed on the S.S.F at Åndalsnes under the surveilance of DnV and according to predetermined procedures.

pancake 62 + 10 cabins	102 (in tons)
1st rb 150 (winch)	60
manitowoc	190
mod 42	300
mod 43	80
mod 67	270
mod 64	204
mod 68 (without helideck)	550
psf 1	310
pancake 05	290
psf 2	110
pancake 08	370
pancake 09	350
pancake 13	130
pancake 12	30
pancake 11	37
mod 41	100
2nd rb 150 (on 41)	80
helideck	155
captain bridge	70
module 02	1150
module 01	670
module 03	840
metering module 02	120
metering module 03	50
module 74	290

miscellaneous equipments:

mooring winches	30 t
mooring platforms	100 t
syminex platform	20 t
norcon lower pack on 41	10 t
3 emergency shut down valves	35 t
5 life boats	20 t

Module 04 were installed offshore, summer 1977.

5.3 Bridge between TCP2 - TP1

The bridge was succesfully installed at location, Frigg Field. Total weight at installation - 577 tons.

6.0 DESIGN DOCUMENTATION

A comprehensive design report for the steel support frame has been issued by the main contractor - Norwegian Contractors. This report contain all relevant information conserning the design of the steel support frame, and is divided into the following main part.

Part I - Containing design premises and design approach. (One Volume)

Part II - Design calculations. (Eight Volumes)

Part III - Computer outputs. (Six Volumes)

In additional, a Summary Report is issued by NC. The full design documentation has been sent directly to N.P.D. A detailed list of contact of the contractors design report is enclosed in the Appendix 6.

A list of different documentation issued and filed in connection with S.S.F. is given below.

DOCUMENTATION LISTING

Document	Pre. by	Dated	Filed
SPECIFICATIONS			
- ELF DEP 1052-No. 5-720 "Minutes of meeting, NORCO-DnV-ELF"	ELF	241075	ELF/NC
- ELF DEP 1052 No. 3-155, Rev. 2 "Fabrication Specification"	ELF	FEB 74	ELF/NC
- ELF DEP 1052 No. 5-302, Rev. 0 "Plate Material Specific.n"	ELF	MAY 75	ELF/NC

Document	Prep. by	Dated	Filed
- ELF DEP 1052 No. 5-498, Rev. 1 "Coating System for Steel Structure of Phase 2"	ELF	OCT 75	ELF/NC
- ELF DEP 1052 No. 3-169, Rev. 1 "Painting Specification for Steel Structures"	ELF	MAR 74	ELF/NC
JOB COMPLETION FILE, FABRICATION	AKER CMP UIE	MAY 77	CMP
DESIGN REPORT Detailed calculations and Construction drawings	KE	AUG 77	KE
SUMMARY REPORT - SSF	NC	AUG 77	NC
FINALIZATION OF CONTRACTS	NC	SEP 77	NC

7.0 FABRICATION DOCUMENTATION
(See also chapter 6.0)

The fabrication-documentation from the different yards involved, in mainly contained in "Data Books".
A data book is work out for the different beams/nodes or subassembly of these and comprises:

- Material certificate copies
- Dimensional controll reports
- COD-reports
- Stress relieving curves
- Welding procedure
- Namelist of welders.

Separate documentation are worked and submitted from the following three main yards:
(Summary enclosed in Appendix 7)

CMP/UIE	enclosure I	
Julin	enclosure II	(d.d. 5/1-77)
Stord	enclosureIII	(d.d. 11/1-77)

It should be noted that as of d.d. 23/12-77, certain material mill certificates are missing as listed in enclosure Appendix 4, enclosure IV.

A list of drawings - verified as "carried out" is enclosed in Appendix 4, enclosure V.

8.0 DRAWING INDEX

A drawing index of latest issued drawings is enclosed in Appendix 8.

9.0 REFERENCES

A list of references is enclosed in Appendix 9.

APPENDIX NO. 1.

AGREEMENT NPD/DnV AND SCOPE OF WORK

A V T A L E

MELLOM STATENS OLJEDIREKTORAT (HERETTER KALT OD)

OG

DET NORSKE VERITAS (HERETTER KALT DnV)

er inngått følgende avtale:

1 OPPDRAGETS OMFANG

1.1 DnV påtar seg å være OD's hovedkonsulent under kontroll med beregninger, materialer og den praktiske utførelse under bygging og installasjon av stålplattform DP 2 og betongplattform TCP 2 med tilhørende bore-, produksjons- og hjelpeutstyr samt moduler og eventuelt rørsystem for bøyelasting som blir plassert på Frigg-feltet (felt 25, blokk 1).

1.2 Kontrollen i henhold til pkt 1.1 vil omfatte konstruksjon og bygging av plattformene. Videre vil kontrollen omfatte kjeler, trykkbeholdere, varmevekslere, trykkrørsystemer, elektriske anlegg, samt kraner med opplagring og helikopterdekk for så vidt angår den styrkemessige konstruksjon.

1.3 DnV skal også bistå ved kontroll med plattformenes sikkerhetsutstyr, innkludert systemer for deteksjon av gass og brann, nødstopp, nødkraft, alarm og intern kommunikasjon samt brannslukningsutstyr og brannsikring.

1.4 Kontrollen omfatter følgende faser:

1.4.1 Vurdering av designkriterier.

1.4.2 Designkontroll av plattformenes utstyr og modulenes konstruksjon.

1.4.3 Kontroll med materialer, sveise-prosedyrer og utførelse.

1.4.4 Byggeplasskontroll ved de steder hvor de enkelte enheter bygges.

1.4.5 Vurdering av belastningspåkjenninger av enhetene fra byggesteder til endelig plassering, samt kontroll med at det ikke har oppstått skader ved slik forflytting.

1.4.6 Kontroll av stabilitetsberegninger og vurdering av belastningspåkjenninger som kan oppstå ved forflytting av plattformene fra byggeplass til Frigg-feltet og setting på feltet.

1.4.7 Kontroll i henhold til pkt 1.4.6 under uttauing og setting.

1.4.8 Kontroll ved legging av interne rørledninger.

1.5 Metode for kontrollberegninger (dataprogrammer) skal godkjennes av OD.

1.6 Arbeidet skal videre omfatte kontroll av alle nødvendige geotekniske beregninger og vurderinger i forbindelse med fundamentering for plassering av plattformen.

1.7 DnV vil utarbeide forslag til arbeidsplaner (scope of work) for den praktiske gjennomføring av kontrollen. Disse planer skal godkjennes av OD.

1.8 I tillegg til offentlige sikkerhetsforskrifter som eksisterer eller som måtte bli fastsatt, kan OD til enhver tid fastlegge nærmere retningslinjer for kontrollen.

1.9 DnV vil legge frem eksisterende regler og bestemmelser eller forslag til slike som skal legges til grunn for kontrollen i tillegg til offentlige sikkerhetsforskrifter og retningslinjer som nevnt i pkt 1.8. Slike regler og bestemmelser skal godkjennes av OD.

2 INFORMASJON M V

2.1 DnV vil sende OD alle relevante spesifikasjoner, instrukser, prosedyrer etc, samt liste over alle aktuelle standarder og relevante tekniske faglige kompendier som DnV legger til grunn ved kontrollarbeidet.

2.2 DnV påtar seg å informere OD på forhånd om møter av prinsipiell natur som DnV har med byggherren, entreprenører, konsulenter etc i forbindelse med kontrollen, slik at OD eventuelt kan være representert. DnV vil omgående sende OD kopier av referater og rapporter fra disse møter.

2.3 Saker av prinsipiell art skal forelegges OD som fatter avgjørelse og utsteder eventuelle pålegg til rettighetshaveren.

2.4 DnV vil sende OD anbefalinger om godkjennelse av de enkelte enheter. OD utsteder de endelige godkjennelser til rettighetshaveren.

2.5 DnV vil under gjennomføring av kontrollen i de faser som er nevnt under pkt 1 holde OD underrettet ved månedlige skriftlige rapporter om de områder kontrollen har dekket. Av rapportene skal fremgå spesielle vurderinger DnV har lagt til grunn ved kontrollen. Fortrinnsvis skal disse rapporter fremlegges på månedlige kontaktmøter.

2.6 DnV vil sende OD kopier av all skriftlig korrespondanse, herunder telexkorrespondanse mellom DnV og rettighetshaver/konstruktør m v i forbindelse med kontrollen.

3 BEGRENSNINGER

3.1 OD står fritt til å anvende andre konsulenter i kontrollarbeidet. Dersom DnV ønsker å engasjere konsulenter, skal godkjennelse av slike konsulenter og de nærmere betingelser innhentes fra OD.

3.2 OD forutsetter at DnV ikke har påtatt seg eller påtar seg å utføre konstruksjonsarbeid av de konstruksjoner som omfattes av oppdraget. Firmaet forplikter seg til ikke å ta oppdrag for andre enn OD vedrørende det kontrollobjekt som omfattes av denne avtale, med mindre samtykke er innhentet fra OD.

4 PERSONELL, ØKONOMISKE FORHOLD, HONORAR M V

4.1 DnV vil til enhver tid gjennomføre kontrollen med nødvendig og kvalifisert personell. OD skal til enhver tid være underrettet om hvilke personer som er ansvarlig for de enkelte fagområder, samt hvem som står som hovedansvarshavende.

4.2 DnV skal beregne sin godtgjørelse i henhold til spesifiserte regler som er gitt i vedlegg 1 til denne avtale. Spesifisert regning sendes OD hvert kvartal. Eventuell justering av de økonomiske vilkår skal godkjennes av OD.

4.3 OD forbeholder seg retten til å gjennomgå DnV's prosjekt/kostnadsregnskaper for å lette kontroll og kostnadsoppfølging.

5 KONFIDENSIALITET

5.1 DnV er forpliktet til å behandle konfidensielt alle prosjektdata og opplysninger som fremkommer i forbindelse med behandling av slike data.

5.2 Bare de av DnV's personale som er nødvendig for en tilfredsstillende gjennomføring av denne avtale skal informeres om eller få adgang til de data og opplysninger som er nevnt under pkt 5.1 ovenfor. DnV skal påse at ingen tredjemann utenom rettighetshaver får adgang til disse data og opplysninger uten skriftlig samtykke fra OD.

6 ANSVARSFORHOLD

6.1 OD har intet ansvar for tap og skade som DnV under utførelse av dette oppdrag påfører:

6.1.1 DnV's personale eller eiendom

6.1.2 tredjemanns personale eller eiendom.

7 OPPSIGELSE

7.1 Avtalen kan av OD sies opp med en måneds skriftlig varsel.

7.2 Dersom avtalen sies opp, plikter DnV å overlate til OD alt materiale som er samlet og alle vurderinger og beregninger som er foretatt pr oppsigelsesdato.

8 AVSLUTNING

8.1 Nærværende avtale skal gjelde inntil installasjonene er montert og godkjent til bruk.

8.2 Etter oppdragets fullførelse skal DnV oversende til OD en sluttrapport for det utførte arbeid. Materiale som er mottatt av DnV i forbindelse med oppdraget skal oversendes OD etter nærmere retningslinjer som utarbeides av OD.

9 VOLDGIFT

9.1 Dersom det oppstår tvist om forståelsen av denne avtale, skal tvisten løses ved voldgift i henhold til tvistemålslovens kapitel 32.

Denne avtale avløser avtale datert 2.4.73 mellom det Kgl Departement for Industri og Håndverk og DnV angående kontroll av anlegg for petroleumsproduksjon på Frigg-feltet.

Denne avtale trer i kraft straks.

Denne avtalen er utstedt i 2 eksemplarer, ett for hver av partene. Hvert av disse eksemplarene er å oppfatte som en original for alle formål.

Stavanger, 23/10 1974	Oslo, 31/10 1974
For Statens oljedirektorat	For Det norske Veritas
F. Hagemann	[signature]

SCOPE OF WORK

FOR

control and inspection

of fixed offshore platform

TCP 2 - FRIGG FIELD

on behalf of

Norwegian Petroleum Directorate

1. INTRODUCTION

This paper describes the general procedure adopted by Det norske Veritas for control and inspection of all stages for the fixed offshore installation TCP-2, Frigg Field on behalf of Norwegian Petroleum Directorate.

The inspection by DnV during all stages of the project is meant to be additional to and not a replacement of the control activities of the owner, designer or contractors, to ensure that the fabrication and installation is carried out according to design and specifications under proper supervision. The organisation of owner's/contractor's inspection, reporting of results etc. for each construction site is to accepted by DnV.

DnV will evaluate and issue their letter of acceptance for the following main stages of the platform:

1. Floating of bottom section and tow out from dry dock.

2. Any major transportation phase of deck or deck components including assembly of deck components.

3. Immersion and installation of deck on main structur

4. Towing operation from the construction site to the Frigg field.

5. Setting at location on the field.

6. Before any platform living accomodation is taken into use.

7. Before the platform is taken into use for hydrocarbon production.

2. DESCRIPTION

2.1 Evaluation of Design Premises

- Environmental conditions including water depth, soil and seabed conditions.

- Codes, standards and specifications used for main concrete substructure, main deck structure and primary and secondary structures inside and outside main structure, pipelines and risers, lifting appliances, equipment, machinery and systems for materials, welding, fabrication, inspection and corrosion protection.

- Design criteria, including applicable regulations, codes and standards, in accordance with accepted practice for design of offshore structures to ensure an acceptable level of safety and serviceability.

2.2 Design review and evaluation

- General arrangement, configuration and system drawings for final structure and for structure during all relevant temporary stages.

- Environmental design loads for all relevant stages.

- Foundations design, including soil stability analysis

- Structural analysis and design calculations for final structure and for structure during all temporary stages.

- Detailed drawings, reinforcement drawings etc. including welding description for steel parts.

- Application of materials, welding and inspection methods.

- Water tightness during floating stages.

- All water, air, grouting and hydraulic systems affecting the integrety of the platform during all relevant temporary stages. This includes evaluation of feasibility of these systems for failsafe operation.

- Equipment and systems with internal pressure (for operation phase) including pipeline and riser systems.

- Design of structure with regard to forces from mooring and towing systems according to principle of weak link outside structure

- Floating stability and motion characteritics during all relevant temporary stages.

- Instrumentation systems to ensure safe operations during construction and installation such as:

    - earth pressure gauges on domes
    - strain gauges on dowels and in concr. structure
    - skirt compartment pressure gauges
    - inclinometer
    - water level in cells
    - skirt penetration
    - echosounders for keel clearance
    - drought

- corrosion protection systems affecting the primary structure including pipelines and riser systems. Evaluation of corrosion protection of secondary structures and process systems in relation to access for in-service inspection.

- For electrical systems: one-line diagrams, methods of protection in hazardous areas, short circuit and fault protection.
  (Operation phase of platform only)

To assist as required by NPD to review platform safety including (operation phase only):

- Platform arrangement

- Hazardous areas, including ventilation/pressurisation system.

- Escape plan

- Safety systems including systems for detection of gas and fire, process and emergency shut down, alarm and public adress, fire-fighting and fire-protection.

- Emergency power

- Relief, flare and vent systems

2.3 Inspection and certification of construction materials

- Component material quality and testing.

- Survey of material fabrication and witnessing of material certification tests (in special cases).

- Review of certificates and material marking.

2.4 Inspection during construction

- Review of construction standards and methods

- Survey of concrete quality, material testing, placing of steel and concrete.

- Survey of tensioning and grouting prestressing tendons.

- Approval of welding procedures, welding equipment and welders.

- Survey of fabrication of steel deck components and assembly of deck frame.

- Survey of fabrication of modules and secondary and temporary structures.

- Survey of fabrication of risers and pipeline parts.

- Survey for code compliance for process and utility equipment and piping.

- Survey of construction tolerances

- Survey of quality of repairs

2.5 Evaluation and surveillance of as-launched structure

- Launching operation method

- Measures to ensure the structure to free itself from bottom.

- Air cushion pressure system under structure.

- Calibration and monitoring instrumentation to ensure operation safety.

2.6 Evaluation and surveillance of structure during construction afloat including deck installation

- Mooring forces on structure with view to environmental conditions, taking into account the possibility for long periodic waves in the water construction basin.

- Safety against entry of sea.

- Rate of concreting, ballasting and submergence with view to possibility for overstressing uncured concrete.

- Air pressure system inside structure.

- Operations to submerge structure and marine operations for installation of deck including surveillance of commissioning of the temporary systems.

- Calibration and monitoring instrumentation to ensure operation safety. Surveillance of commissioning of instruments prior to operations.

- Surveillance of operations.

2.7 Evaluation and surveillance of final tow-out phase and all intermediate towing phases

- Evaluation of conditions along towing route taking into account seasonal limitations.

- Towing forces.

- Safety against entry of sea.

- Temporary systems.

- Survey of instrumentation installed for operation safety e.g. water level in cells, draught, keel clearance, inclinometers.

- Surveillance of operations.

2.8 Evaluation of installation phase operations and inspection during installation

- Installation procedure for the structure and surveillance of installation operation for compliance with planned procedure.

- Evaluation of data from installation instrumentatio

- Monitoring of initial settlements

- Grouting

- Scour protection

- Tightness of oil storage

- Possible localized damage

- Transportation method for larger items, (modules etc.) and reinspection after arrival on site

- Module installation

- Survey during pipelaying

- Final assembly for the structure and the process and utility systems also including completion of riser and pipeline systems.

- Witnessing of load tests of lifting appliaces.

- Electrical installation.

Fje/Røl/AHE

14.1.1977

APPENDIX NO. 2

FIGURES - GENERAL

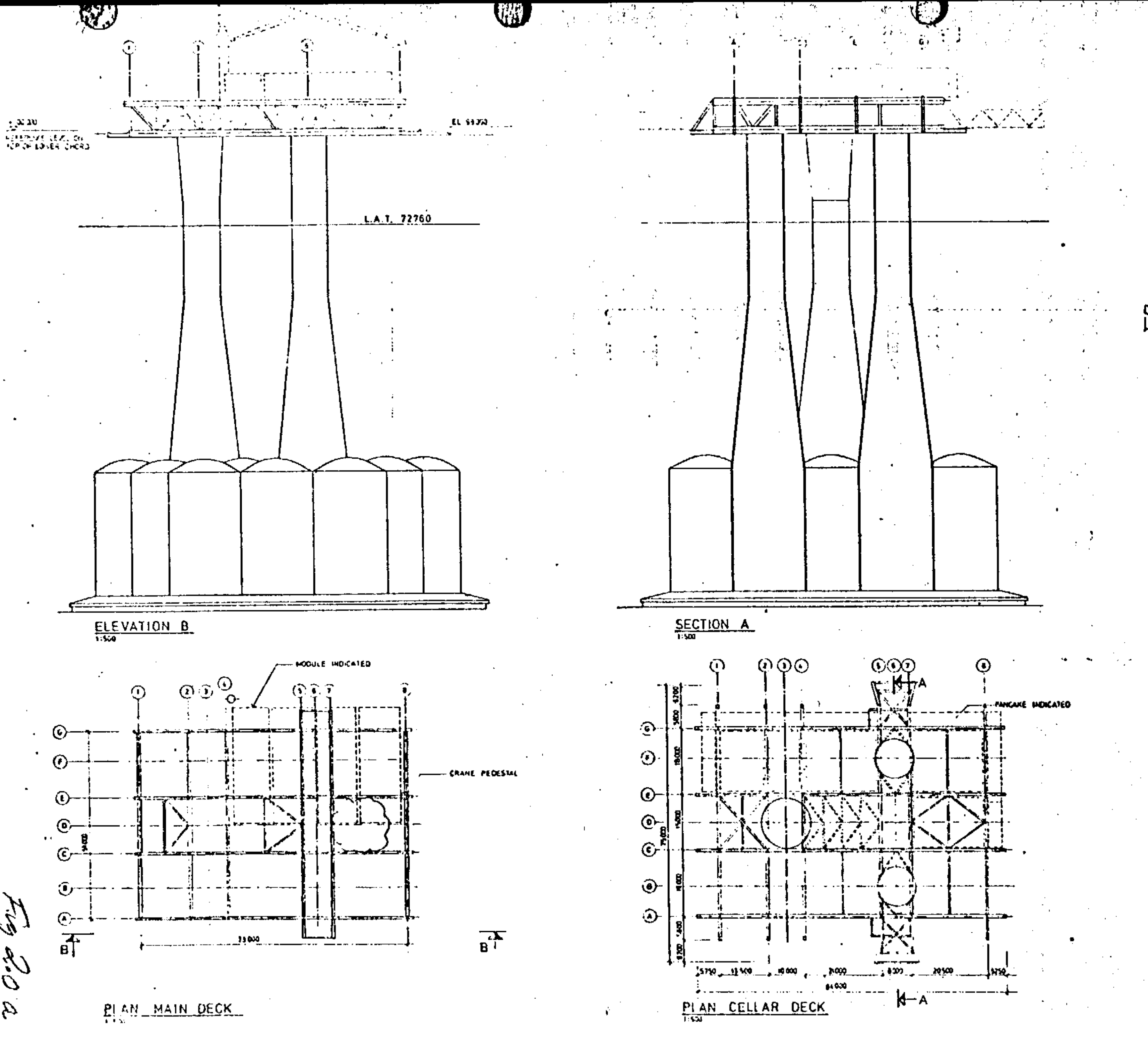


Fig 2.0 a

SPECIFICATION

MATERIAL, FABRICATION AND PAINTING TO BE IN ACCORDANCE WITH ELF - NORGE SPECIFICATIONS.

MATERIAL SPEC 1052 Nº 3-145 REV 1
FABRICATION SPEC 1052 Nº 3-155 REV 2
PAINTING SPEC 1052 Nº 3-165 REV 1

MATERIALS: SHSS-20 FROM 12mm TO 25mm PLATES
SHSS-40 FROM 30mm TO 100mm PLATES
UNLESS OTHERWISE NOTED

WELDING SYMBOLES NS 1421 REV 1
THE DRAWINGS SPECIFY THE REQUIRED EFFECTIVE THROAT OF PARTIAL JOINS PENETRATIONS WELDS.

FABRICATION TOLERANCES

1A TOLERANCES FOR DIMENSIONS (MEMBERS, NODES AND SECTIONS.

LENGTH	TOLERANCE
0 TO 500	MAX ± 2mm
501 TO 1000	" ± 3mm
1001 TO 4000	" ± 4mm
OVER 4001	" ± 5mm

1B TOLERANCES FOR DIMESNIONS (OVERALL LENGTH AND WIDTH) +/- 20mm

1C TOLERANCE FOR DIMESNIONS (C/C RINGS) +/- 15mm

2A STRAIGHTNESS (MEMBERS SECTION AND TRUSS ROWS)

l_x TOLERANCE ALONG x = $\frac{L}{1000}$
l_y TOLERANCE ALONG y = $\frac{L}{1000}$
MAKS l_x & l_y 12mm

2B STRAIGHTNESS (PLATES IN PROFILES)
TOLERANCE ALONG x OR y $\leq \frac{L}{200}$

3 ANGULAR DISPLACEMENT.

ON DIRECTLY LOADED SECTIONS $l \leq \frac{L}{500}$
OTHERWISE $l \leq \frac{L}{200}$

4 DEVIATIONS FROM ELEVATIONS.

BRACES AND MEMBERS	± 6 mm
LANDING AND STAIRWAY	± 12 mm
DIFFERENCE IN DECK PLATE OR GRATING AT JOINT	1,5 mm

5 DEVIATIONS OF CENTER LINES.
TRUSS MEMBERS AND GIRDERS AT CONNECTIONS ± 8mm
THE ANGLE BETWEEN TRUSS ROWS SHALL BE WITHIN ONE MINUTE OF 90 DEGREES.

6 ABUTTING PARTS TO BE JOINED BY BUTT WELDS SHALL BE CAREFULLY ALIGNED WHERE THE PARTS ARE EFFECTIVELY RESTRAINED AGAINST BENDING DUE TO ECCENTRICITY IN ALIGNMENT, AN OFFSET NOT EXCEEDING 10 % OF THE THICKNESS OF THE THINNER PART JOINED, BUT IN NO CASE MORE THAN 3mm, MAY BE PERMITTED AS A DEPARTURE FROM THE THEORETICAL ALIGNMENT.
TAPER WELD TO MAINTAIN NET SECTION TO BE GRIND IN SLOPE 1:4

CORNER CUT OUT FOR WELDS.
t ≤ 30 , R=50
t = 40 , R=60
t > 40 , R=80

DIMENSIONS:

ALL DIMENSIONS ARE GIVEN IN MM

2	15776	BRACING REMOVED
1	8 476	GENERAL NOTES
0	122.76	ISSUED FOR CONSTRUCTION
D	24.11 75	NOTES
C	290 75	ISSUED FOR APPROVAL
B	17 6 75	BRACING AND ELEVATION
A	4 7 75	ISSUED FOR INFORMATION

ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

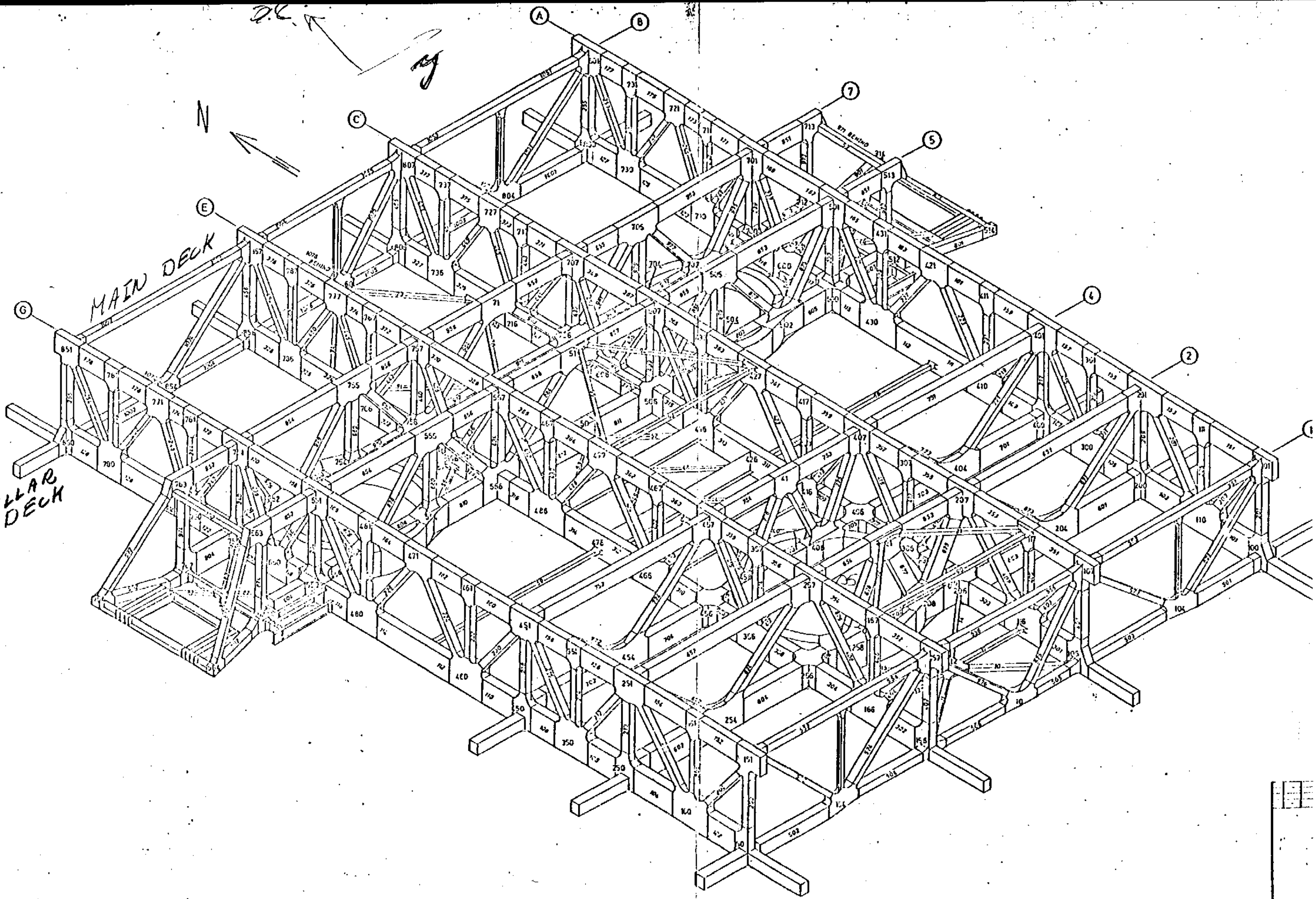
CONDEEP

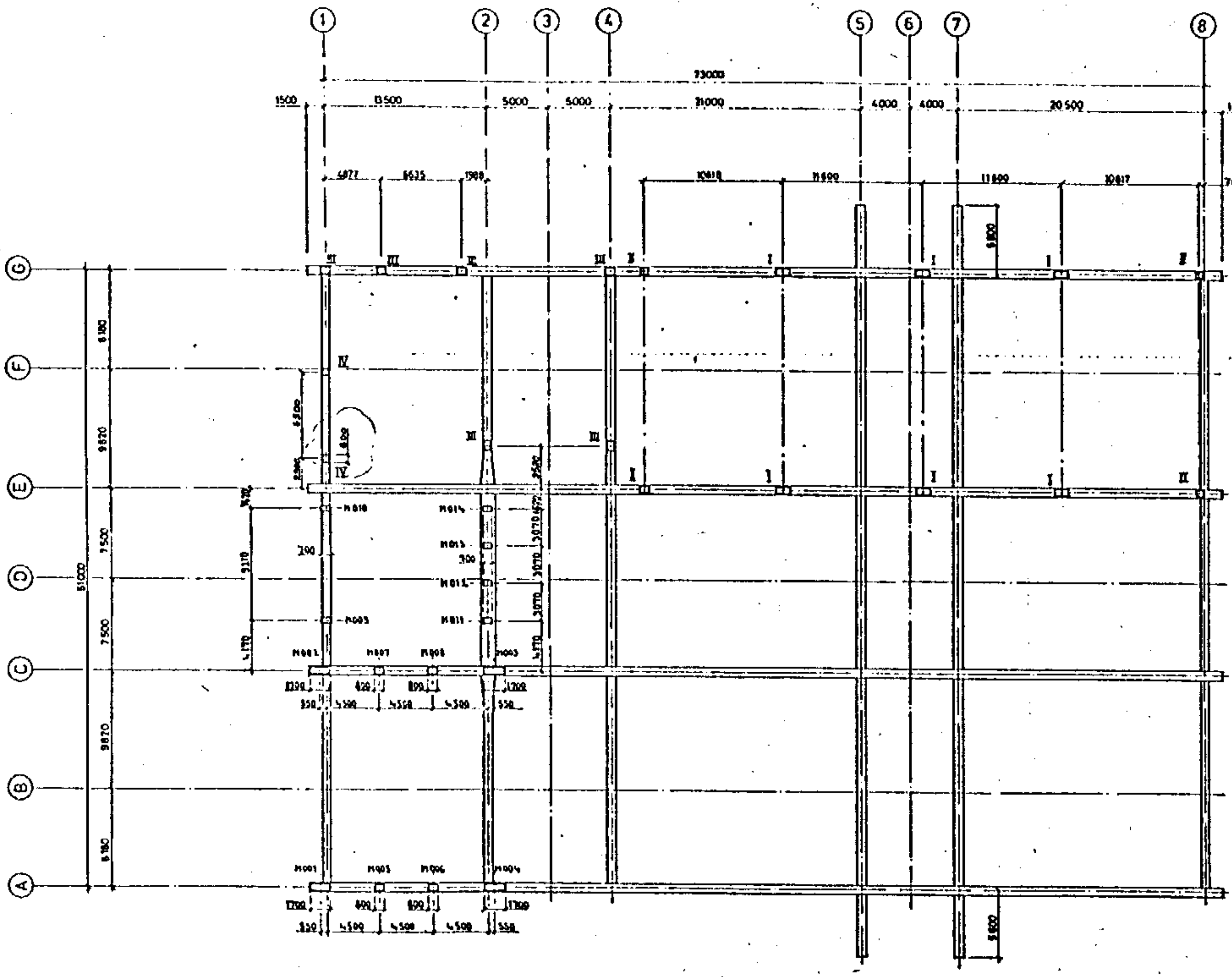
OUTLINE

1:500

HOYER ELLEFSEN

UC 3337 0449 20 0103

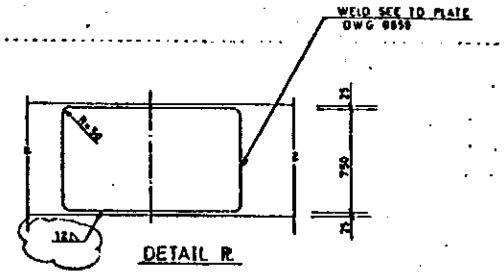

N
MAIN DECK
CELLAR DECK
A
B
C
E
G
7
5
4
2



DIMENSION OF PLATES

TYPE	DIMENSION
M001·M002·M003·M004	10×750×1700
M005·M006·M007·M008	10×700×800
M009·M010·M011·M012·M013·M014	20×500×700

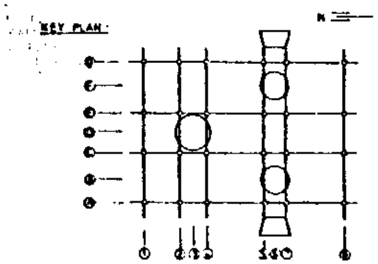
TYPE I	20×750×3130
TYPE II	20×750×1640
TYPE III	20×750×750
TYPE IV	20×600×650



NOTES:

ALL PLATE CORNERS TO BE ROUNDED R=50
TYPICAL WELD SEE DWG 0950, TO PLATE
MATERIAL TO BE SHSS-20

KEY PLAN:



10	24 7 77	R ADDED, HOLE REMOVED
9	5 1 77	MISC
8	11 2 76	DIM OF R CHANGED

5	21 10 76	PL M011+M014 ADDED SKID REMOVED
4	2 7 76	ALL GUIDES TYPE I
3	16 6 76	DETAIL A & B
2	23 4 76	GUIDE III
1	27 2 76	GUIDES ADDED
0	12 2 76	ISSUED FOR CONSTRUCTION
C	30 1 76	SKID PLATES
B	29 8 75	ISSUED FOR APPROVAL
A	11 2 75	ISSUED FOR INFORMATION

ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

CONDEEP
production platform

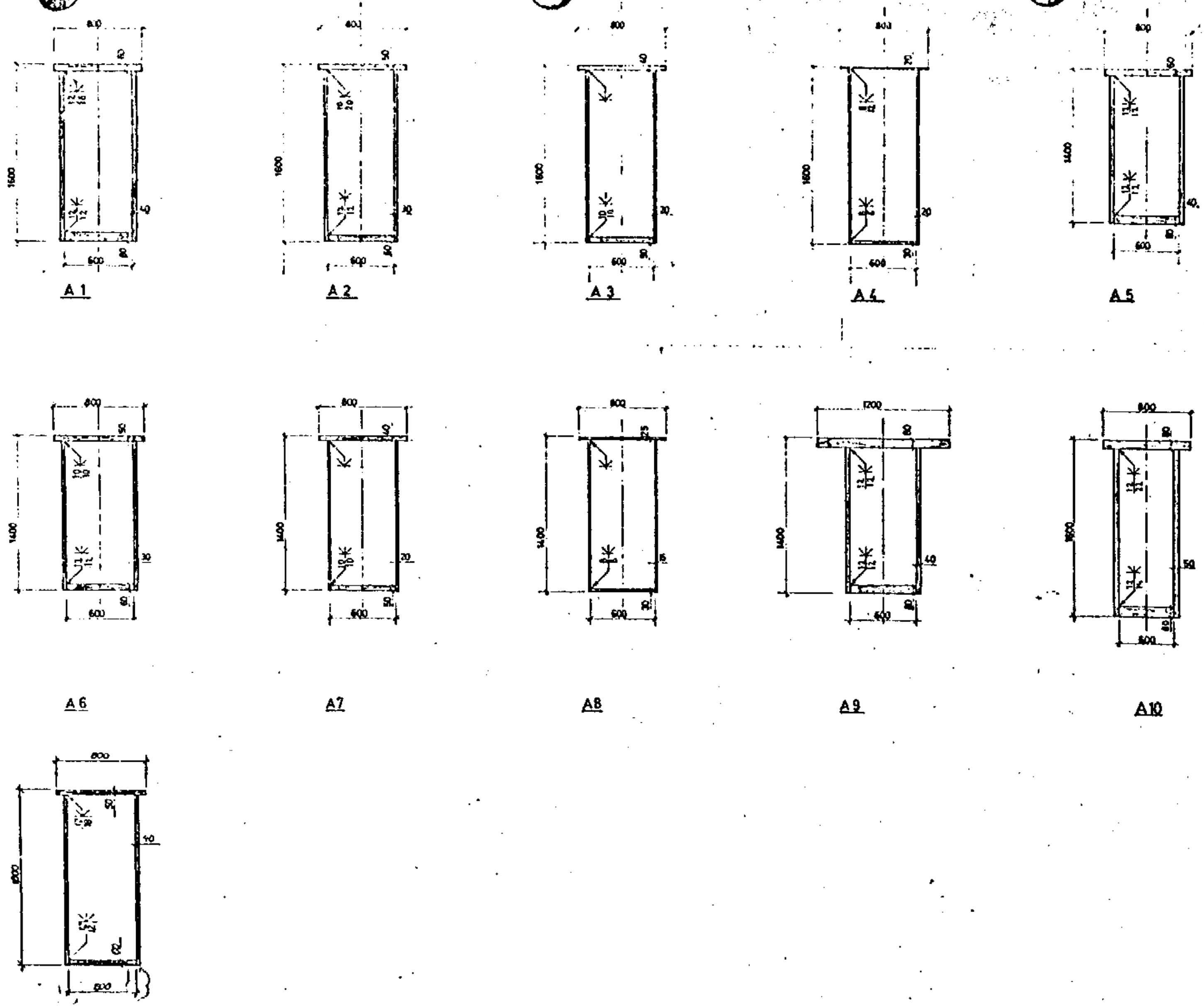
PLAN MAIN DECK

1:200

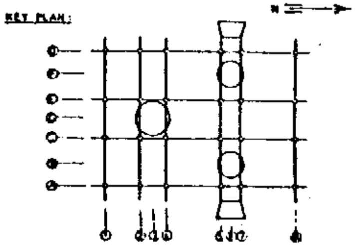
HOYER
OSLO 2 - NORWAY

UC 3337 0449 20 0104

Fig. 2.0 c)



NOTES:
GENERAL NOTES SEE DWG 0103



2	9.4.76	A18
1	19.2.76	A28
0	11.2.76	ISSUED FOR CONSTRUCTION
E	30.11.75	WELD DIM.
D	24.9.75	GENERAL REVISION
C	29.8.75	ISSUED FOR APPROVAL
B	11.7.75	WEB CHANGED
A	4.7.75	ISSUED FOR INFORMATION

CONDEEP

HØYER

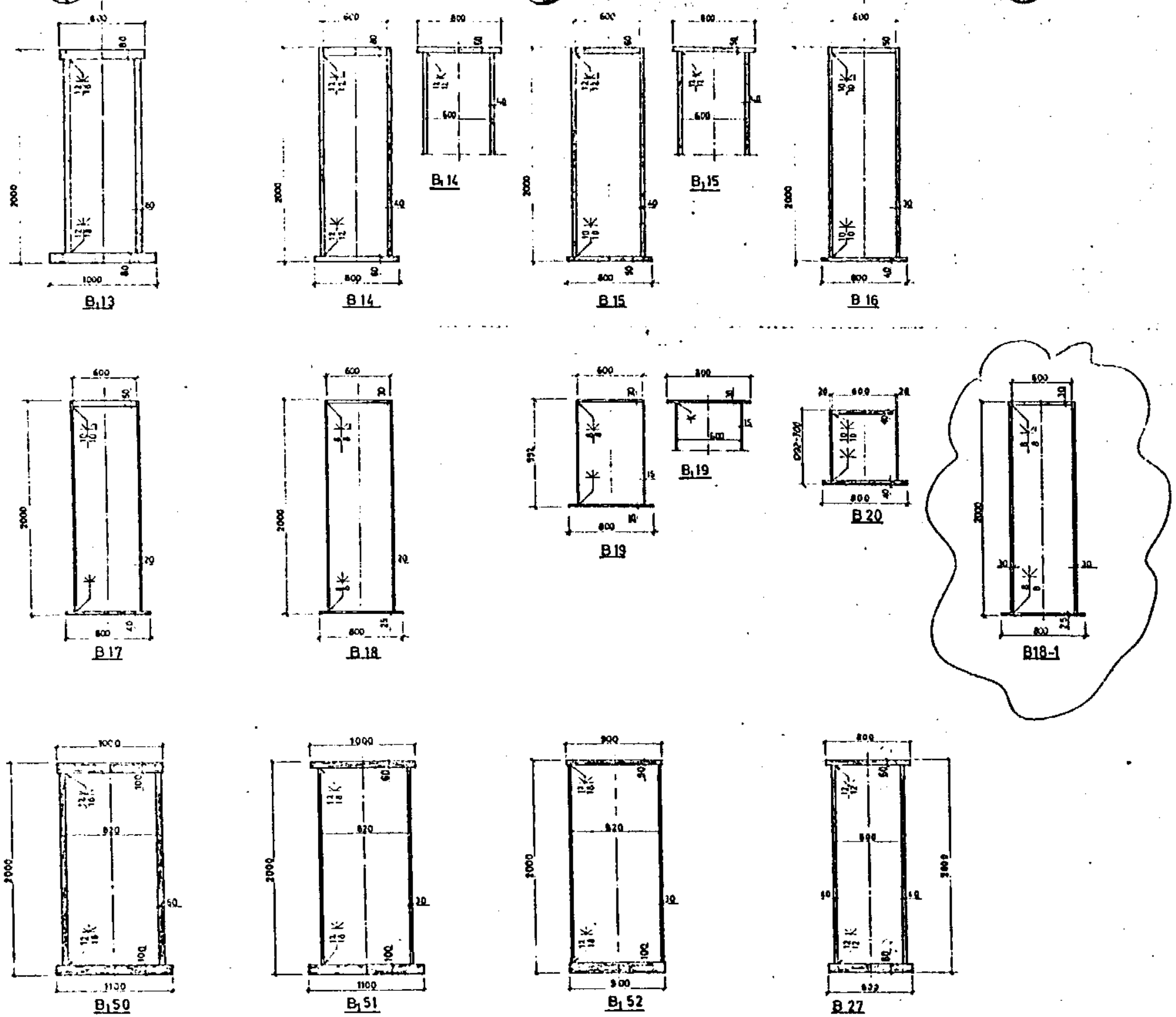
ELF – NORGE A/S – E 10
FRIGG – TCP 2 – STEEL FRAME

SECTIONS OF PROFILES A

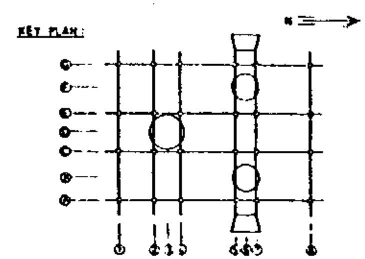
1:20

UC 3337 0449 20 0106 2

Fig 2.0 d)



NOTES:
GENERAL NOTES SEE DWG 0103
USE SAME WELDS ON PROFILS B & B₁



3	11 3 76 PL 40 AND 65 PROFILE B27
2	27276 PROFILES
1	19 2 76 D21 REMOVED
0	11 1 76 ISSUED FOR CONSTRUCTION
E	30 11 75 GRINDING ON TOP FLANGE
D	26 11 75 GENERAL REVISION
C	23 6 75 ISSUED FOR APPROVAL
B	11 7 75 WEB CHANGED
A	4 7 75 ISSUED FOR INFORMATION

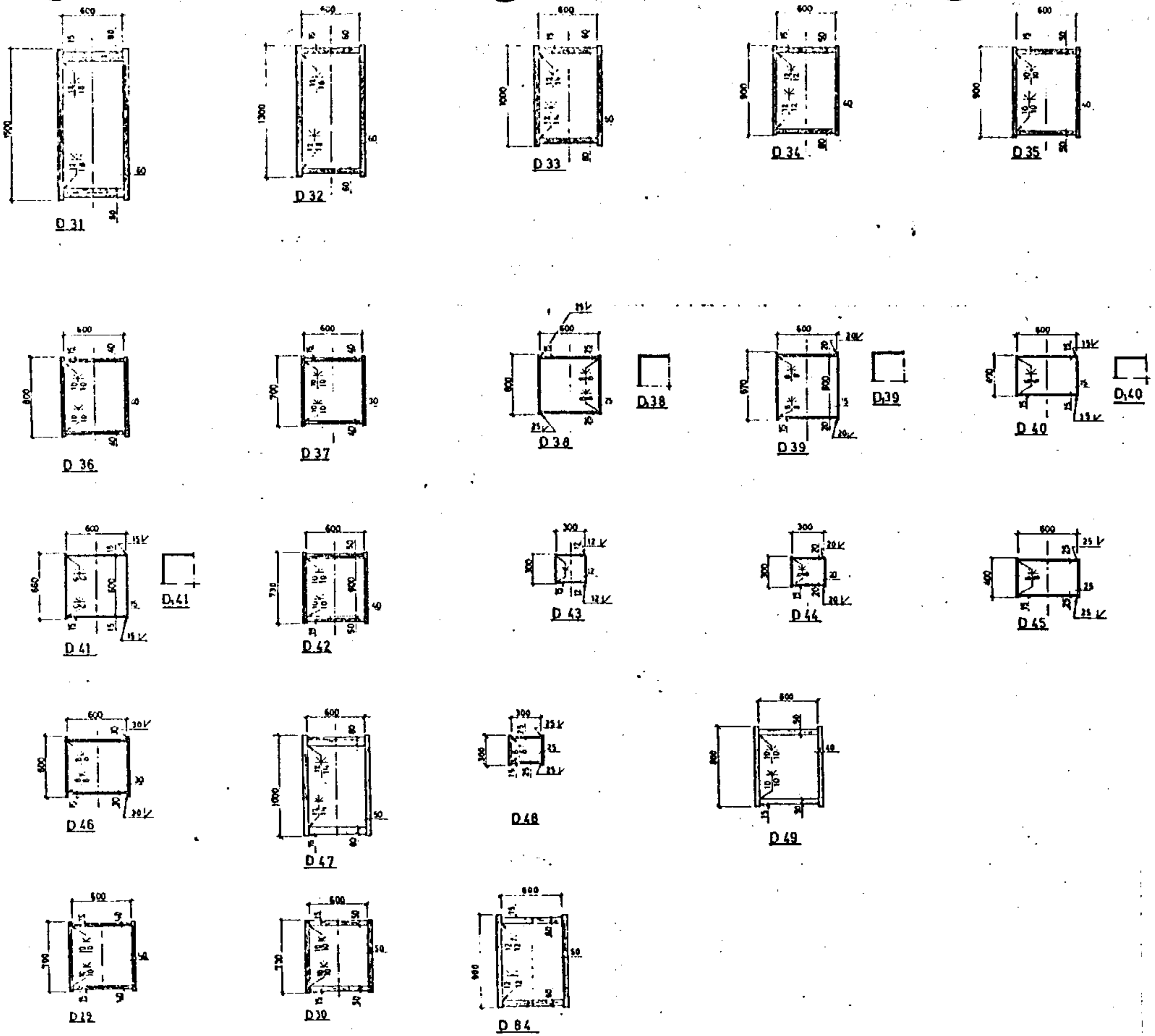
CONDEEP

ELF – NORGE A/S – E 10
FRIGG – TCP 2 – STEEL FRAME

SECTIONS OF PROFILES B

1:20

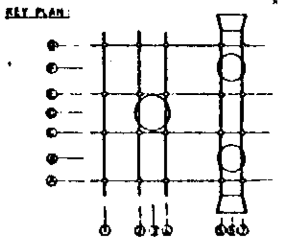
UC 3337 0449 20 0107 .4



NOTES:

GENERAL NOTES SEE DWG 0103
USE SAME WELDS ON PROFILES D & D_1

KEY PLAN:



Rev.	Date	Revision
1	21.2.76	DEL.
0	12.2.76	ISSUED FOR CONSTRUCTION
D	24.11.75	GENERAL REVISION
C	29.8.75	ISSUED FOR APPROVAL
B	11.7.75	GENERAL REVISION
A	16.7.75	ISSUED FOR INFORMATION

CONDEEP production platform

HØYER ELLEFSEN

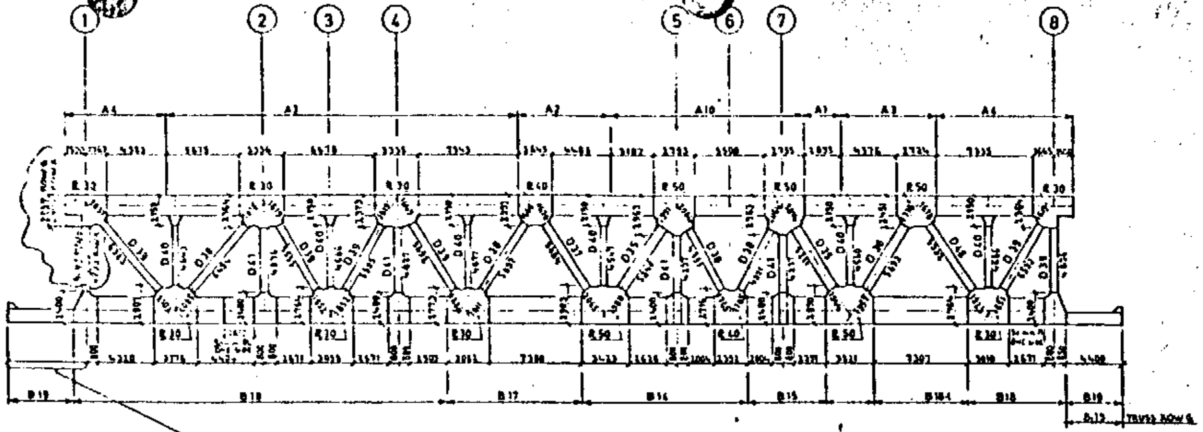
OSLO

ELF – NORGE A/S – E 10

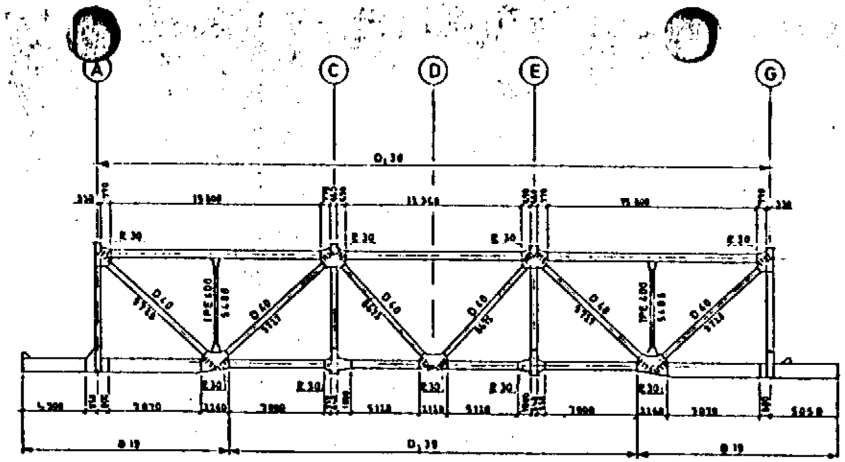
FRIGG – TCP 2 – STEEL FR

SECTIONS OF PROFILES D

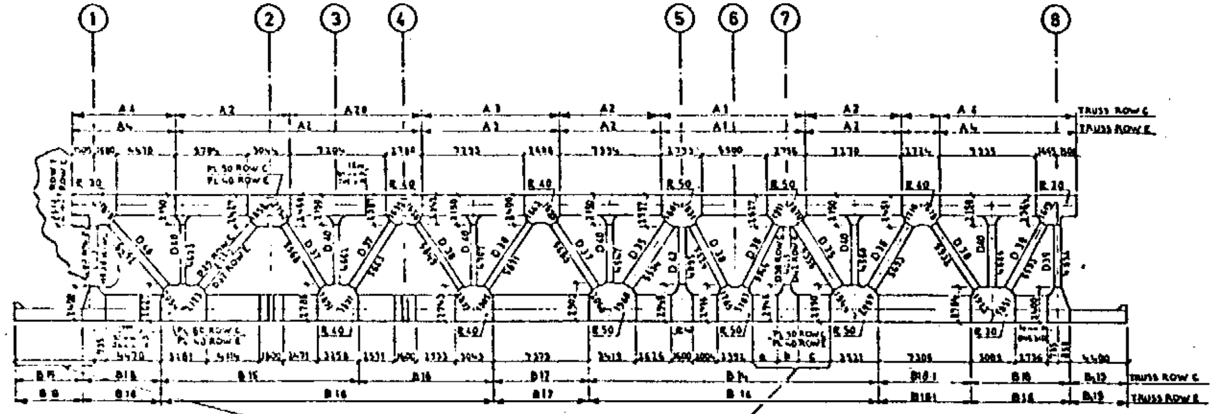
UC 3337 0449 20 0



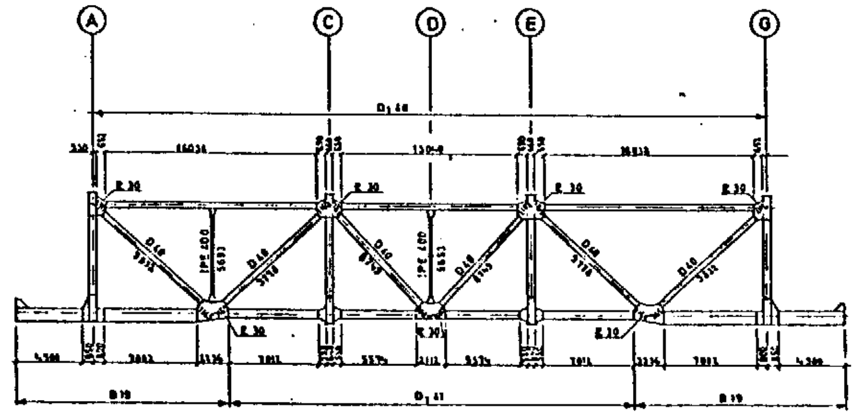
TRUSS ROW A-G



TRUSS ROW 1



TRUSS ROW C-E



TRUSS ROW B

4	31.7.76	D39 ROW 1-A,C,E,G
3		B 35
2	15.3.76	D38 ROW C AND E 6-7
1	19.2.76	CANT. CHORD, IPE 433
0	13.2.76	ISSUED FOR CONSTRUCTION
C	31.1.75	30 mm PL. MOVED
B	22.12.75	PLATES FOR MANHOLE
A	16.12.75	ISSUED FOR INFORMATION

CONDEEP

HØYER ELLEFSEN A/S

ELF - NORGE A/S - E10

FRIGG - TCP 2 - STEEL FRAME

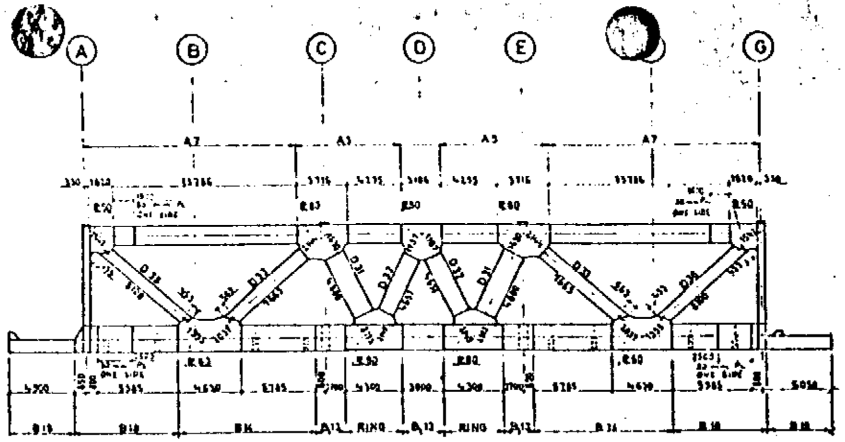
DIMENSIONS

TRUSS ROW A,C,E,G, 1 AND B

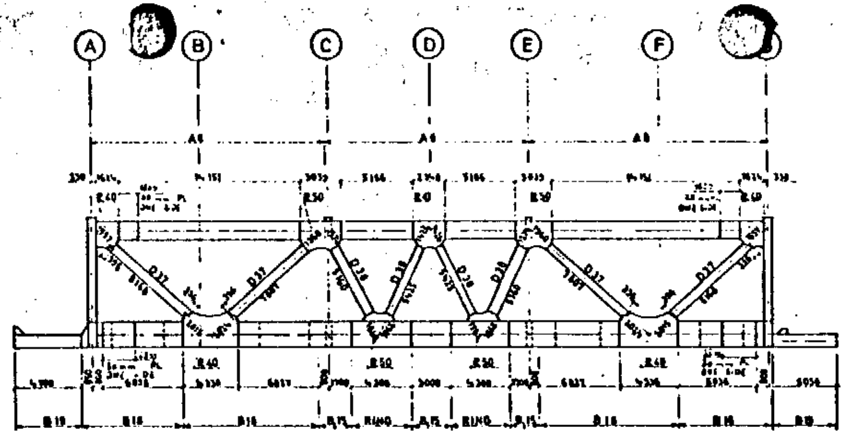
1:200

UC3337 0449 20 0110 4

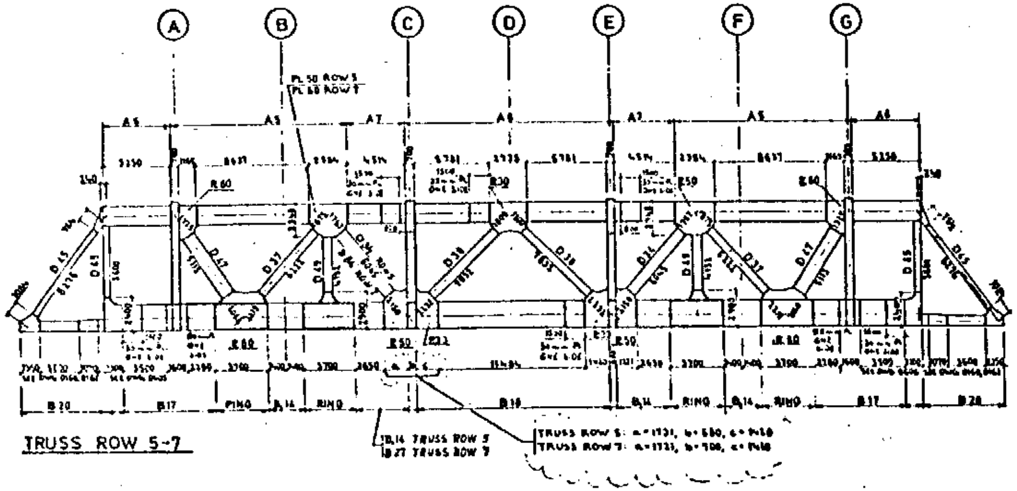
(Fig 20.9)



TRUSS ROW 2



TRUSS ROW 4



TRUSS ROW 5-7

H	8.5.76	LOWER CHORD ROW 5C
G	27.7.76	TRUSS ROW 5 AND 7
F	25.7.76	PROFILES
E	19.7.76	CANTILEVERS
D	12.7.76	ISSUED FOR CONSTRUCTION
C	9.7.76	PLATE THICKNESS
B	22.12.75	PLATES FOR MANHOLE
A	18.12.75	ISSUED FOR INFORMATION

ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

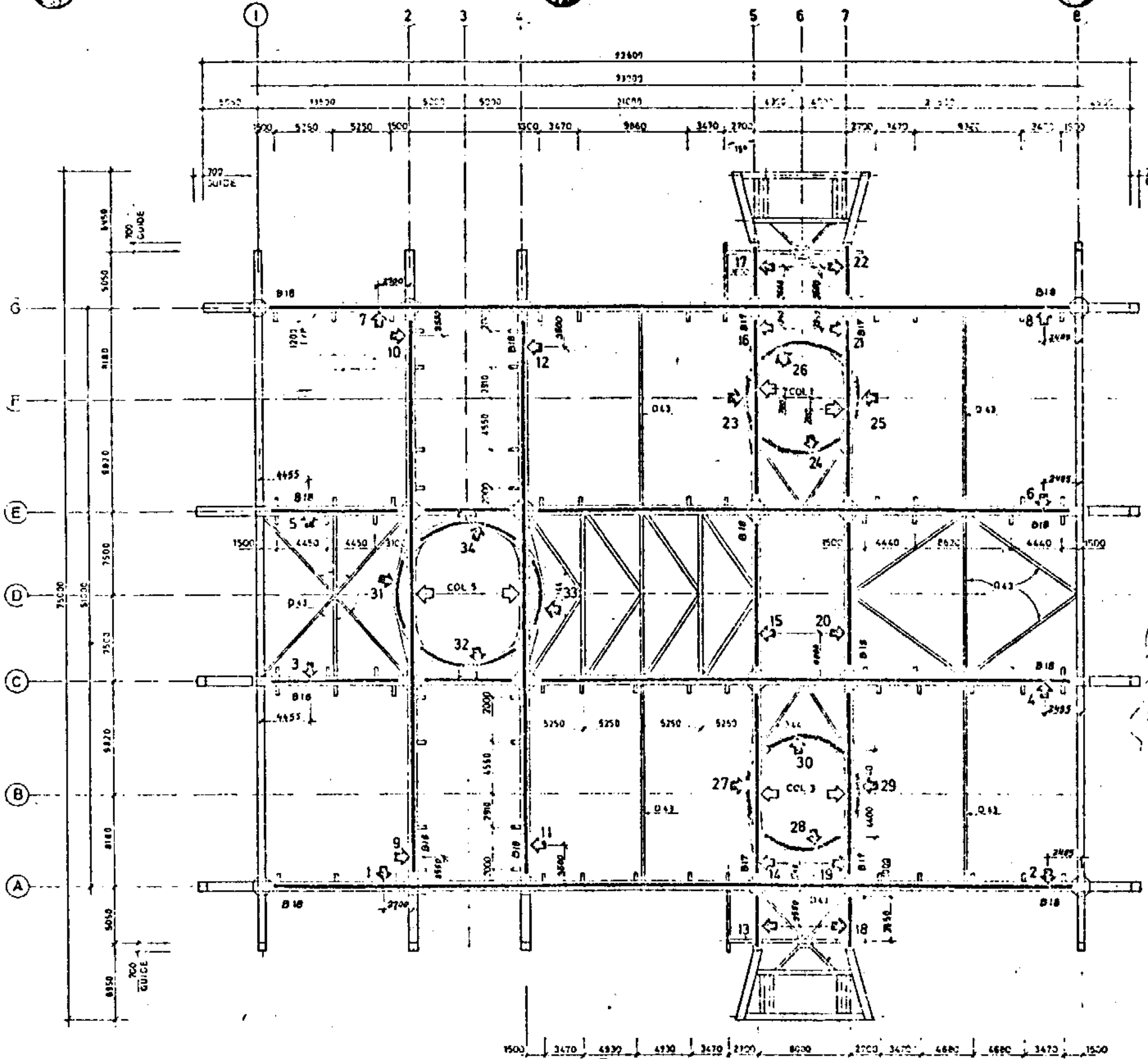
DIMENSJONS
TRUSS ROW 2,4,5 AND 7

CONDEEP production platform

HOYER ELLEFSEN
OSLO - NORWAY

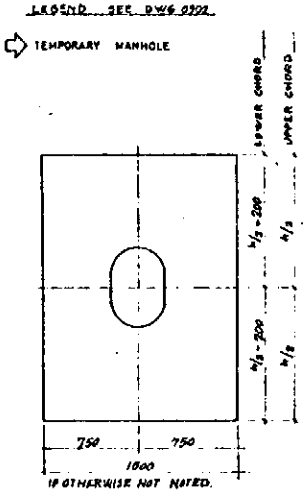
UC 3337 0449 20 0111

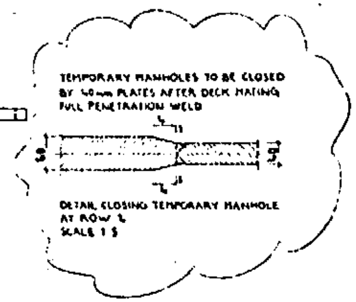
Fig. 20 (A)



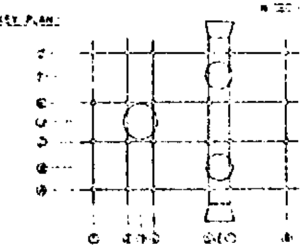
NOTES:
LEGEND SEE DWG 0902
⇨ TEMPORARY MANHOLE

REPLACEMENT OF WEB PLATE AT INSPECTION MANHOLE			
SECTION	WEB WITHOUT MANHOLE	WEB WITH MANHOLE	HIGHT OF WEB
A 2	R 30	R 40	1550
A 7	R 20	R 30	1360
A 8	R 15	R 25	1375
B 17	R 20	R 30	1930
B 18	R 20	R 30	1975
A 51	R 30	R 40	1840
A 52	R 30	R 40	1860





KEY PLAN:



INSP HOLES, ACCESS FROM PAINTING TROLLEY:	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 15, 20, 23, 25, 27, 29, 33
INSP HOLES, ACCESS FROM GANGWAYS:	14, 16, 19, 21, 25
INSP HOLES, ACCESS FROM EXTRA GANGWAY IN CON. WITH PAINTING TR.:	31, 33
INSP HOLES, ACCESS FROM DECK INSIDE RINGS:	24, 26, 28, 30, 32, 34

E	15.10.76	ACCESS HOP HOLES CLOSING TOP MANH
F	31.8.76	TEMP. MANHOLE
D		ISSUED FOR CONSTRUCTION
D		DIMENSION CHANGED
C		MANHOLES MOVED
B		ISSUED FOR APPROVAL
A		ISSUED FOR INFORMATION

ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

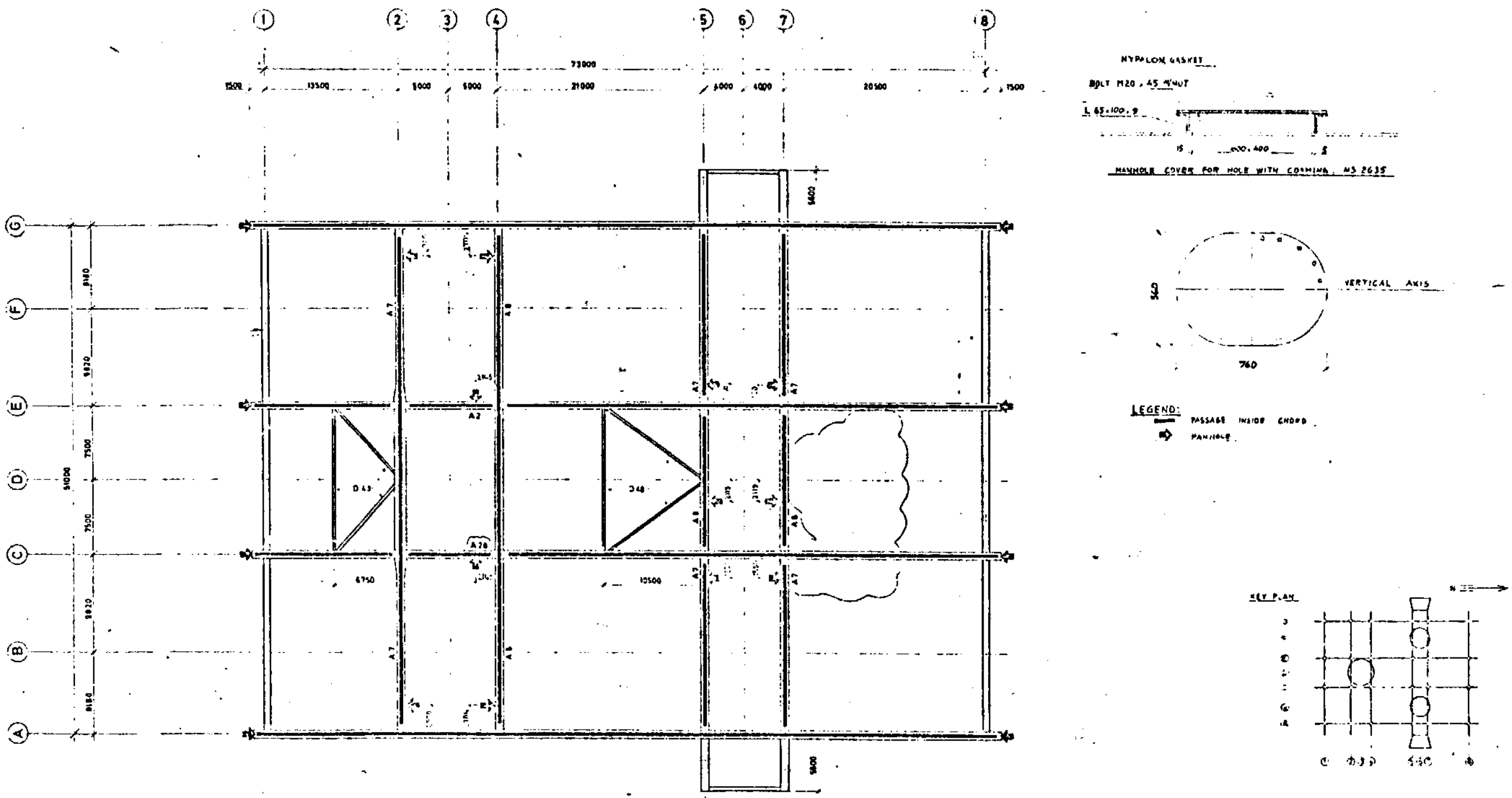
COMPTEP

INSPECTION WAYS CELLAR DECK

1:200

903 2

2.4(a)



PLAN MAIN DECK

Rev	Date	Description
2	15.2.76	BRACING REMOVED
1	3.2.76	PROFILE NF
0	12.1.76	ISSUED FOR CONSTRUCTION
C	[illegible]	MANHOLES MOVED
B	24.9.75	ISSUED FOR APPROVAL
A	[illegible]	ISSUED FOR INFORMATION

CONDEEP

ELF – NORGE A/S – E 10
FRIGG – TCP 2 – STEEL FRAME

INSPECTION WAYS MAIN DECK

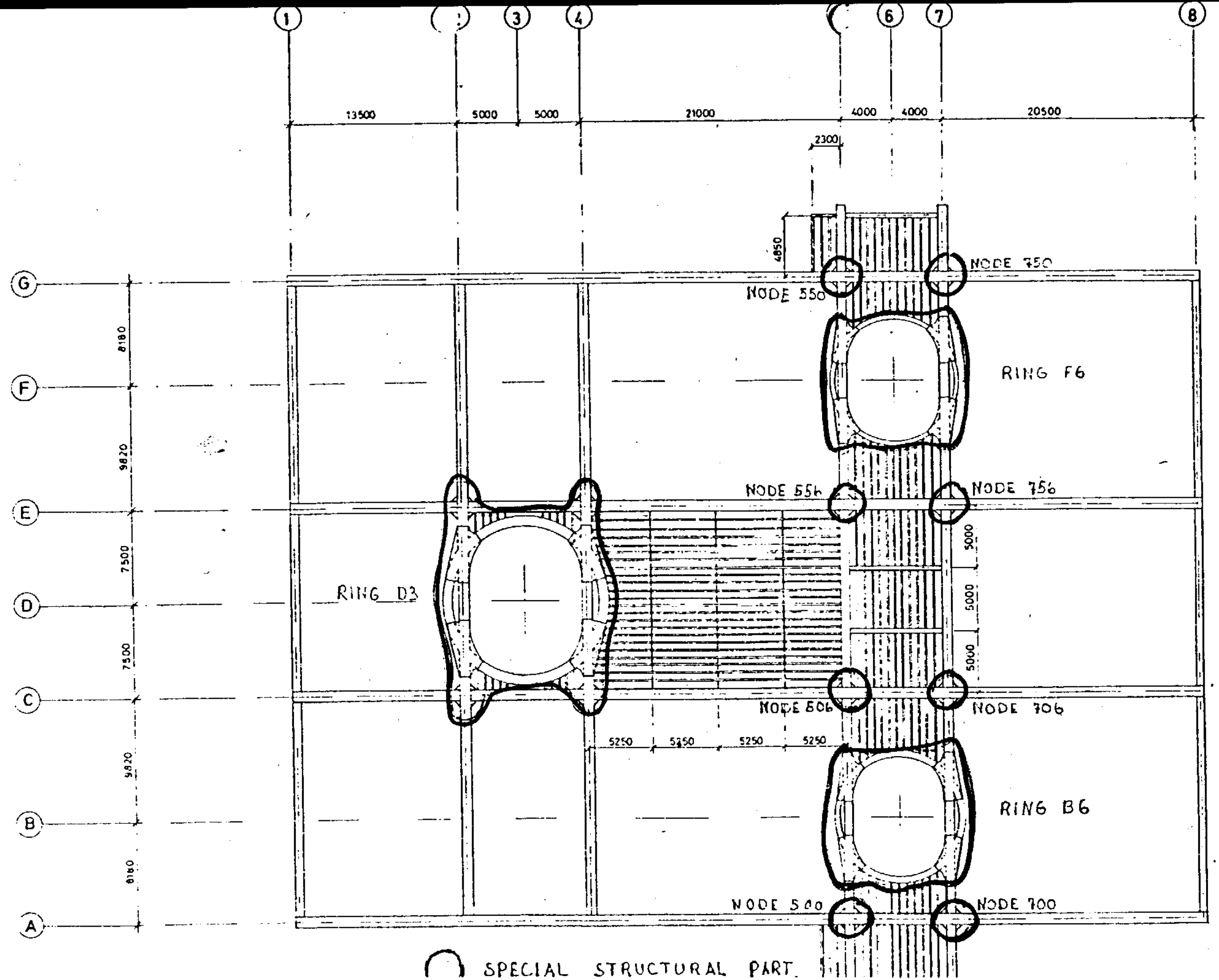
1:200

HØYER

UC 3337 0449 20 0902 2

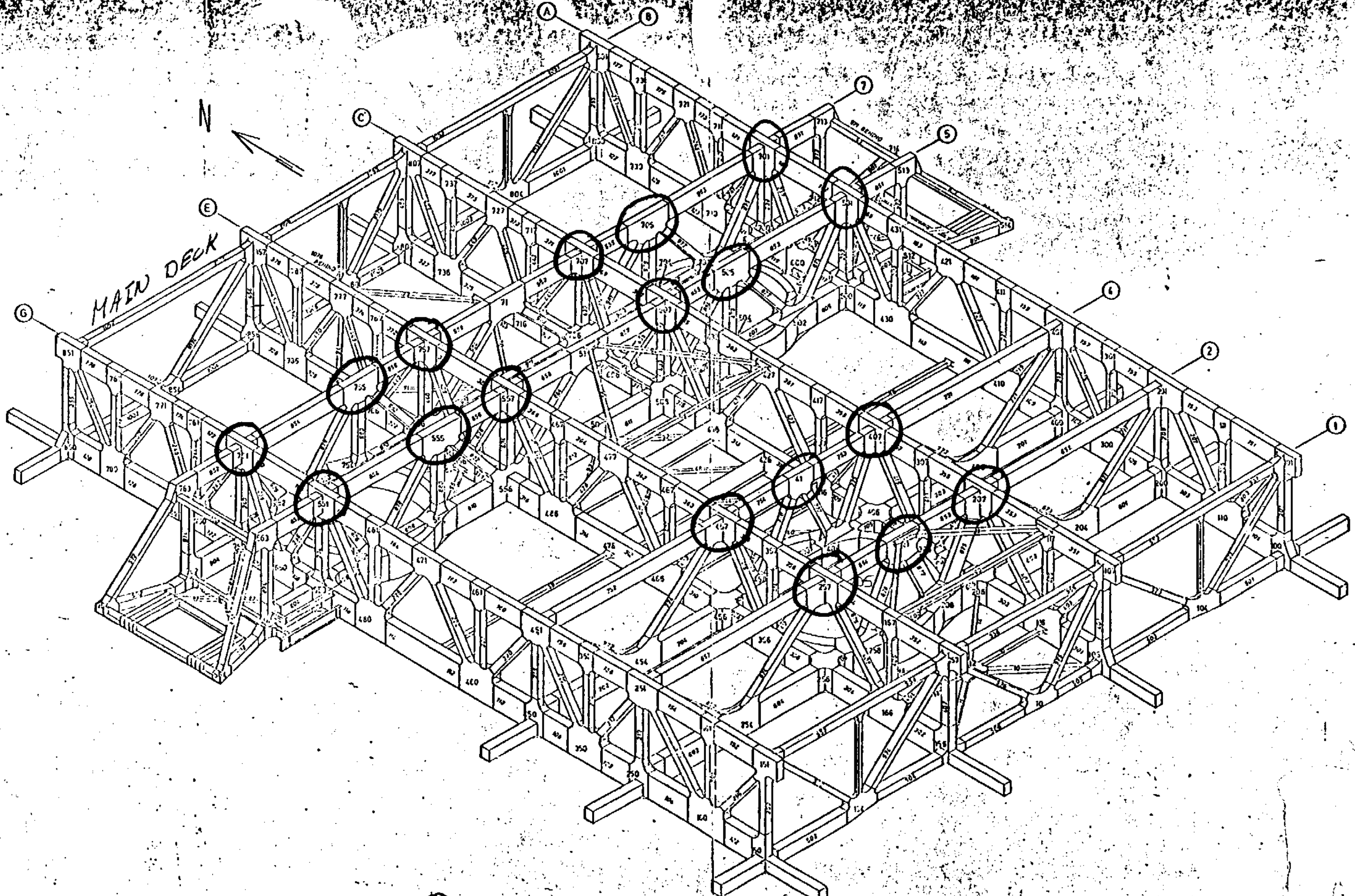
APPENDIX NO. 3.

FIGURES/DRAWINGS - DESIGN



1
3
4
6
7
8
13500
5000
5000
21000
4000
4000
20500
2300
4850
G
F
E
D
C
B
A
8180
9820
7500
7500
9820
8180
NODE 750
NODE 550
RING F6
NODE 556
NODE 756
RING D3
5000
5000
5000
NODE 506
NODE 706
5250
5250
5250
5250
RING B6
NODE 500
NODE 700
SPECIAL STRUCTURAL PART.

Fig. 3.3 (i)



O SPECIAL STRUCTURAL PART

Fig 33(b)

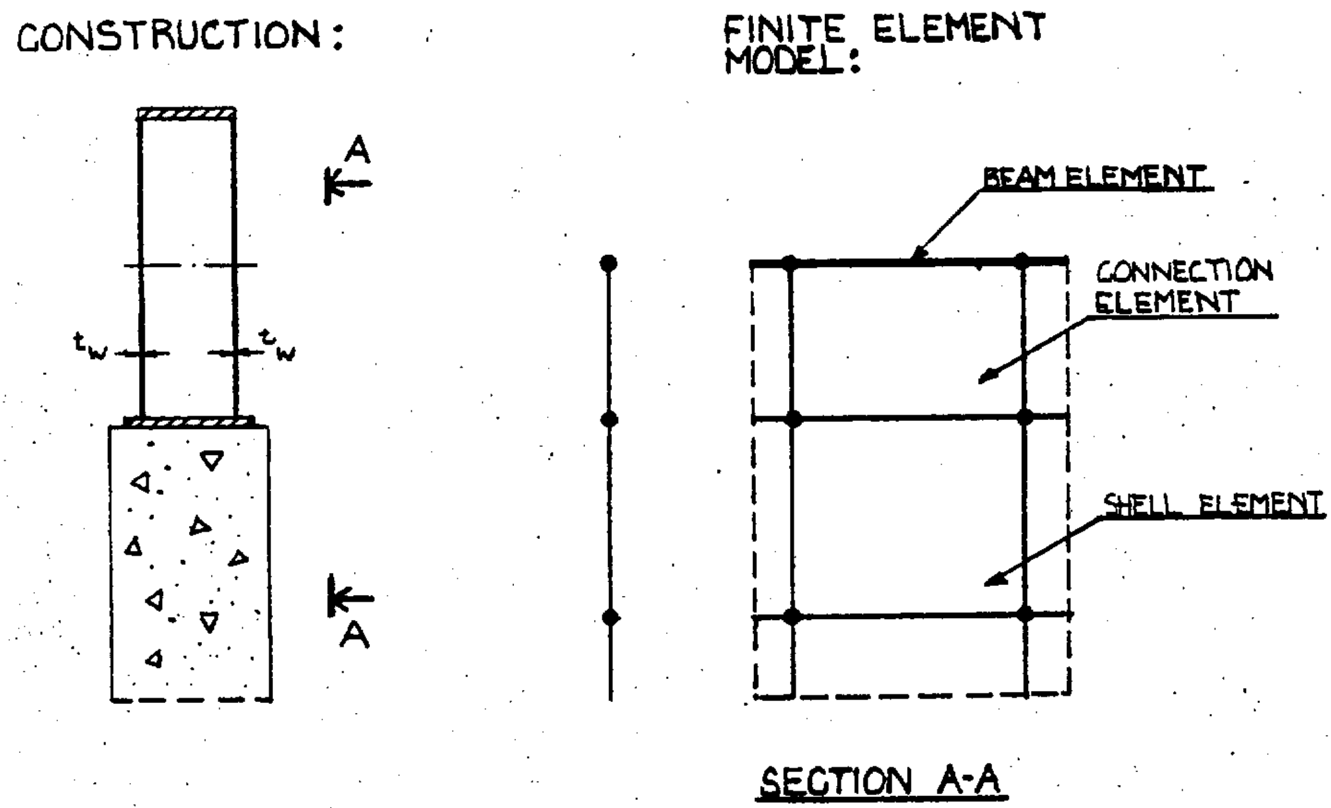


Fig 3.5a) Ring-top of shaft

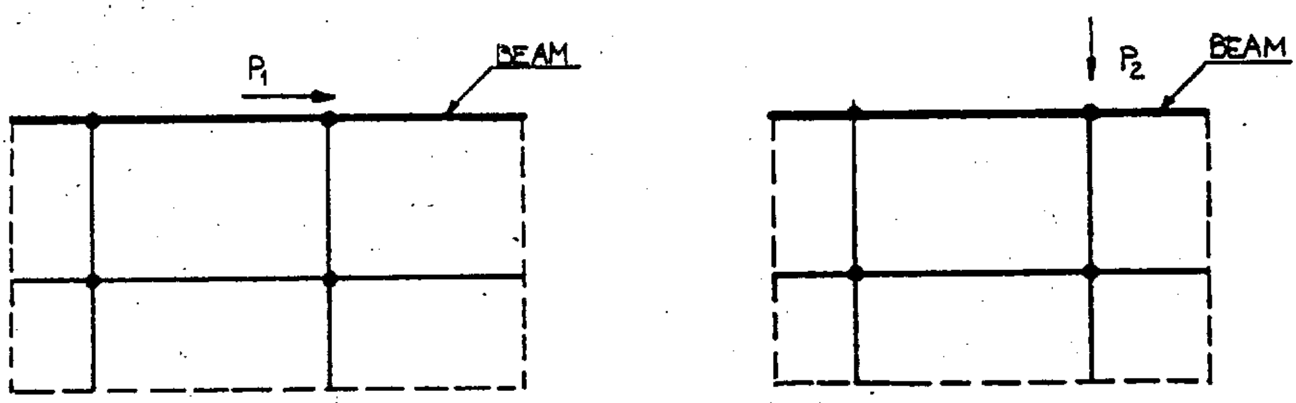
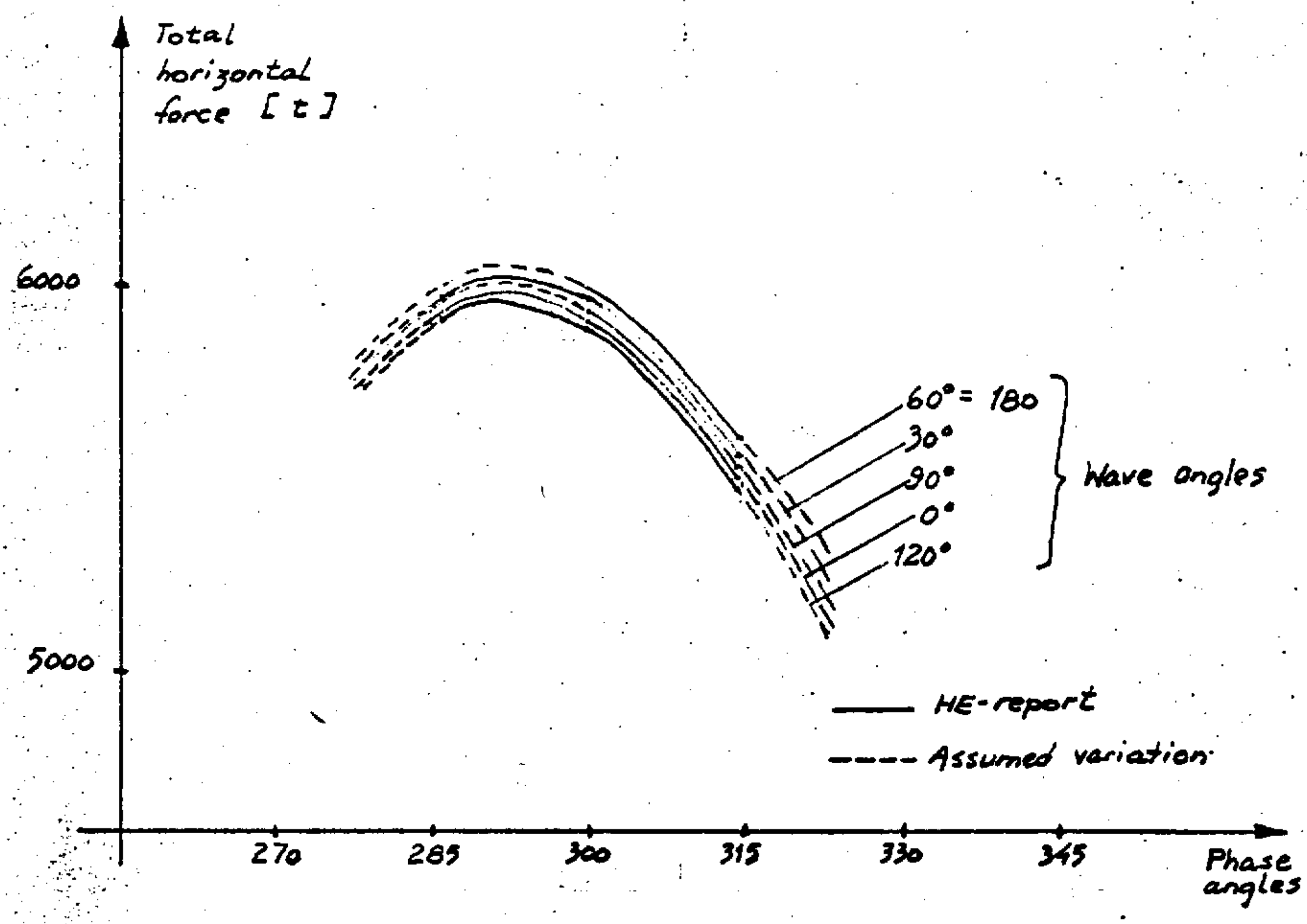
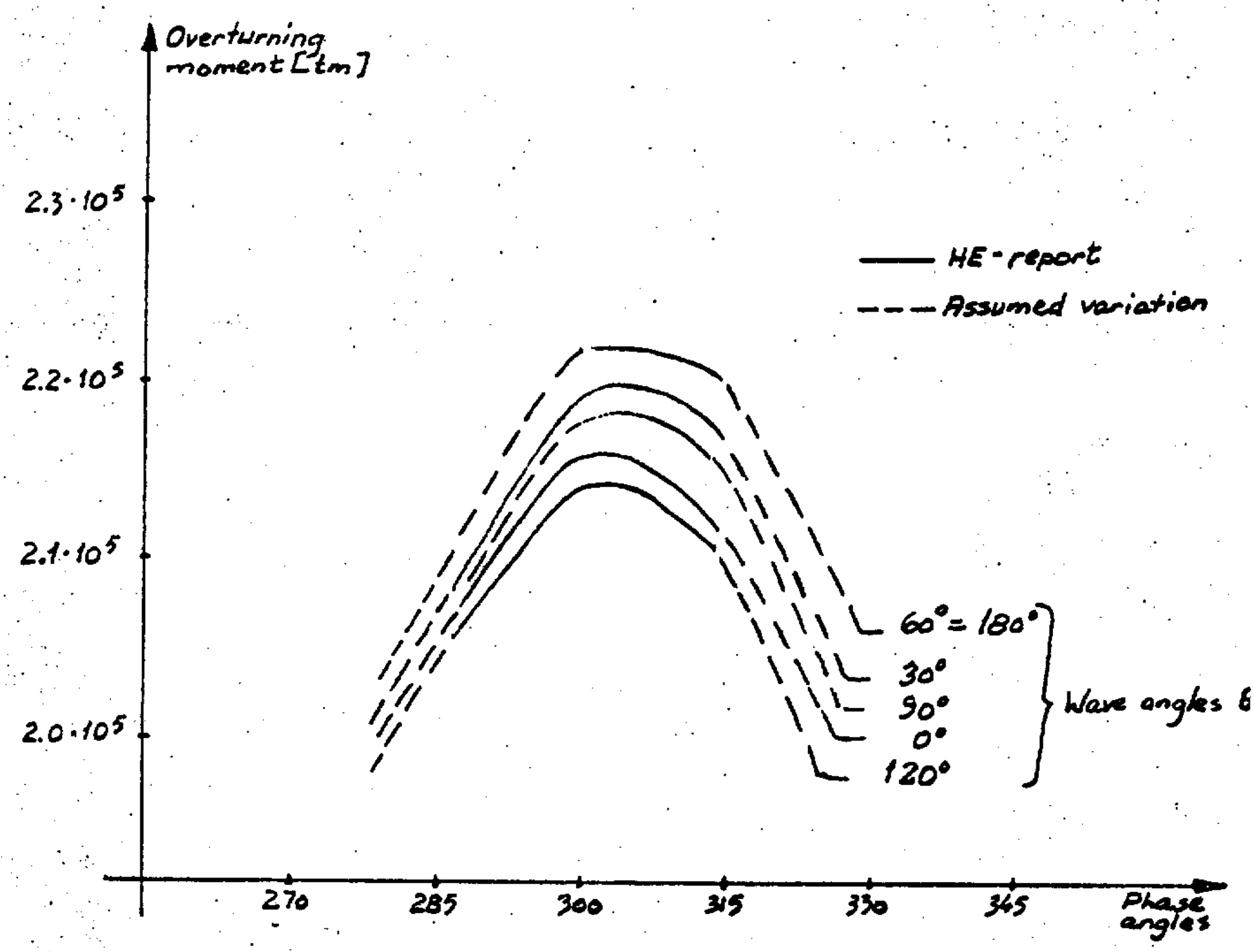


Fig 3.5b) Point loads acting on the ring



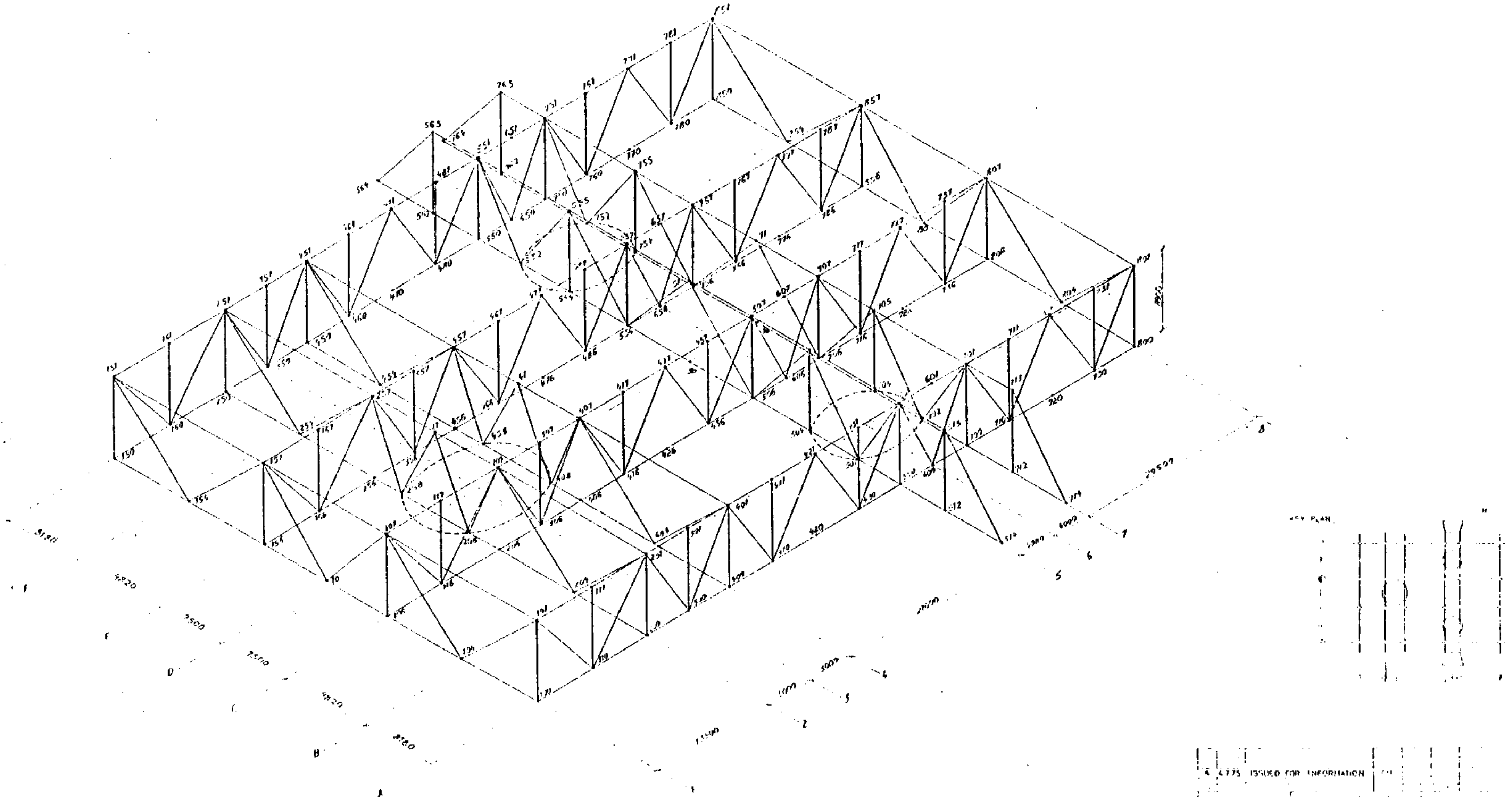
OVERTURNING MOMENT FOR
DIFFERENT WAVE ANGLES AND
PHASE ANGLES - OPERATIONAL WAVES

Fig 3.5 C



OVERTURNING MOMENT FOR DIFFERENT WAVE ANGLES AND PHASE ANGLES - OPERATIONAL WAVES

Fig 3.5



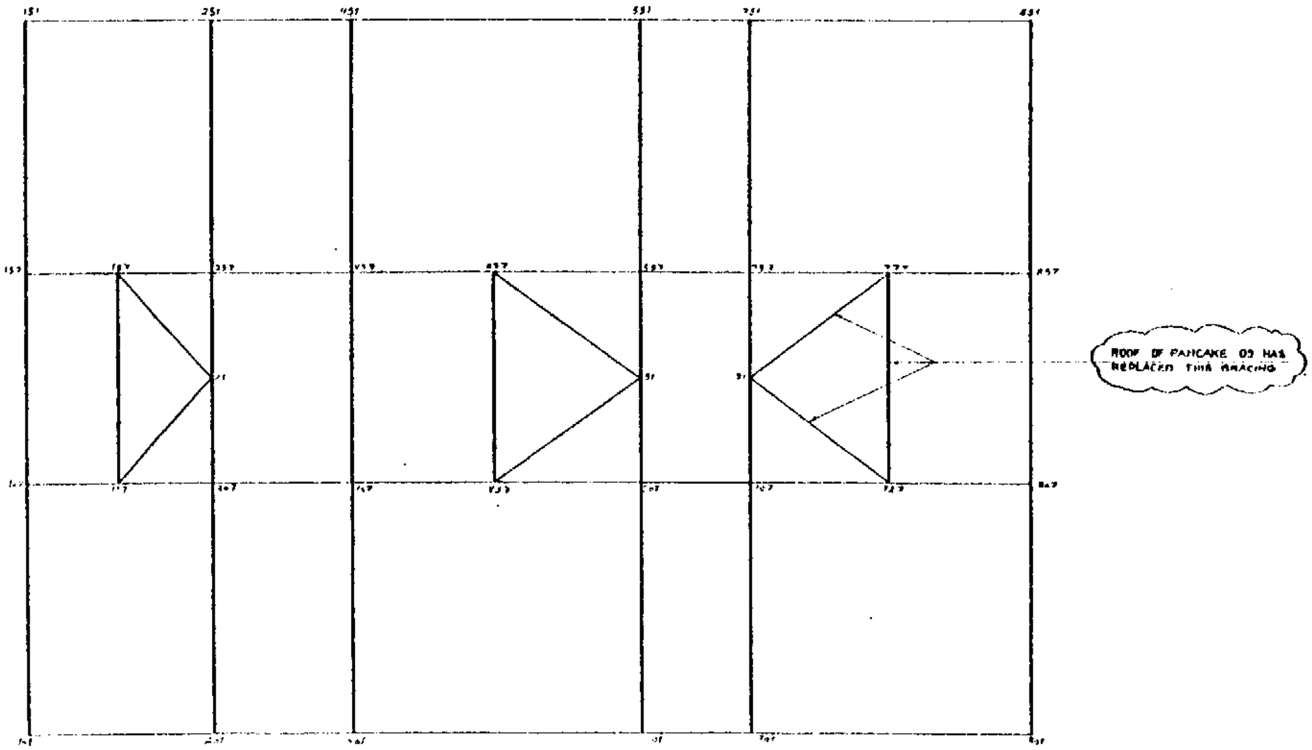
A 4 7 75	ISSUED FOR INFORMATION

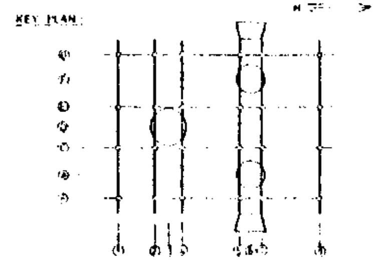
ELF – NORGE A/S – E 10
FRIGG TCP 2 – STEEL FRAME

STRUCTURAL ANALYSIS
MATHEMATICAL MODEL
STEEL FRAME MEMBERS

12 6 7

1001





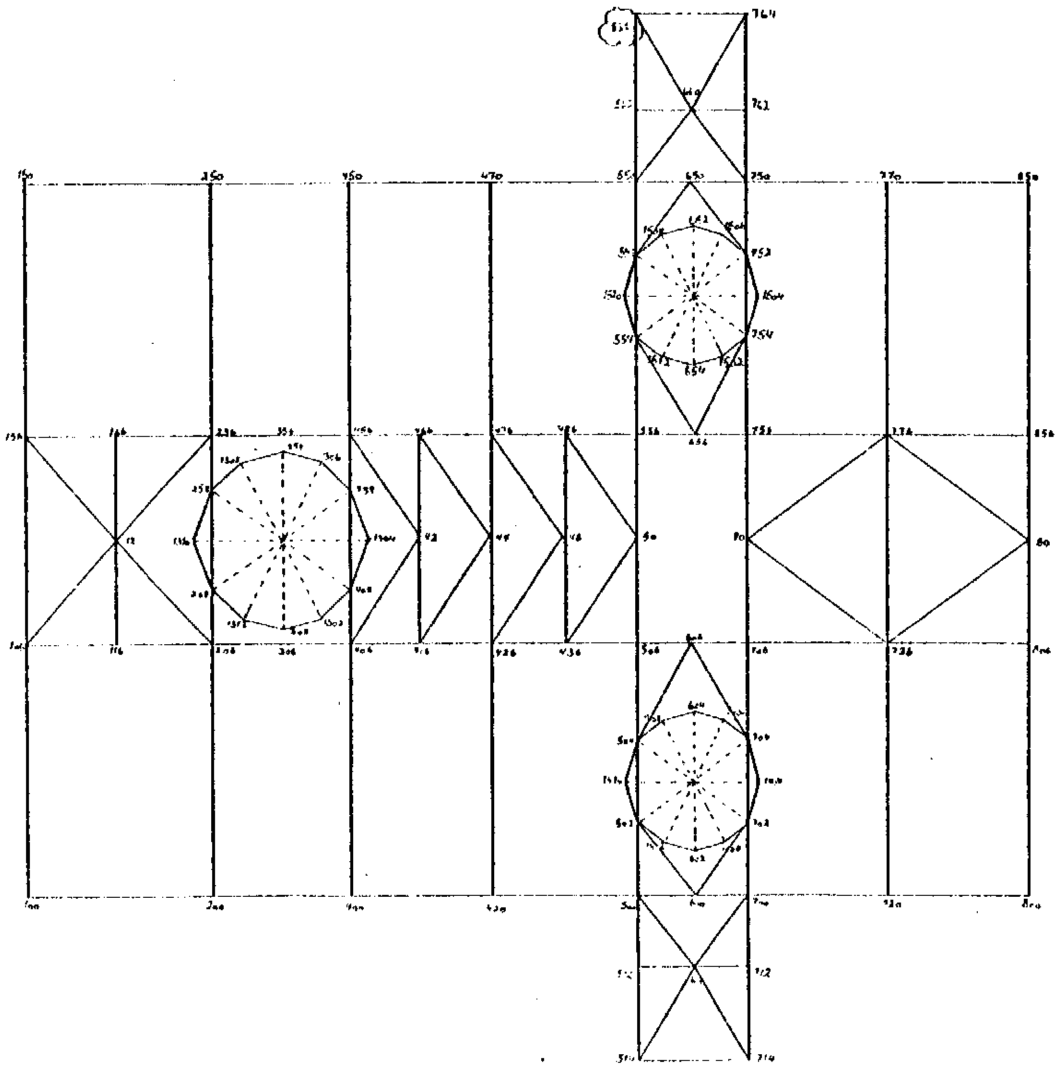
C	5.5.77	NOTE ADDED
B	2.8.75	REVISION OF BRACING
A	4.7.75	ISSUED FOR INFORMATION

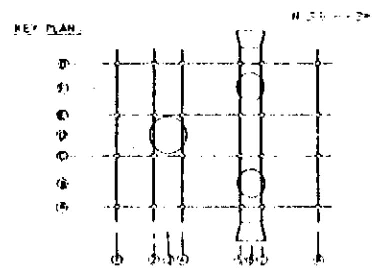
ELF - NORGE A/S - E 10

FRIGG - TCP 2 - STEEL FRAME

STRUCTURAL ANALYSIS
MATHEMATICAL MODEL
Y-BAY BRACING
MAIN DECK

OC 5347 0459 20 1002 C



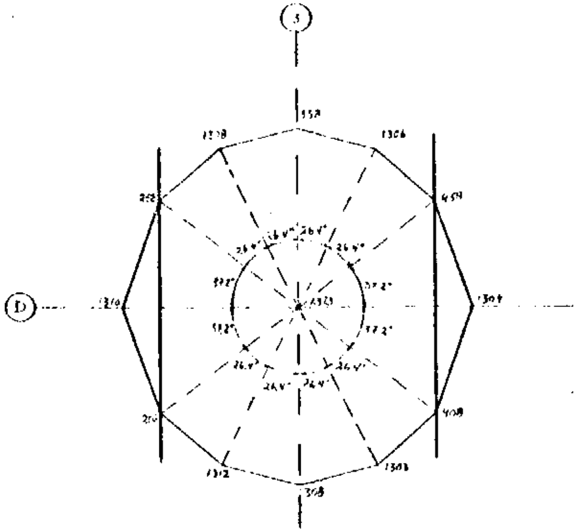


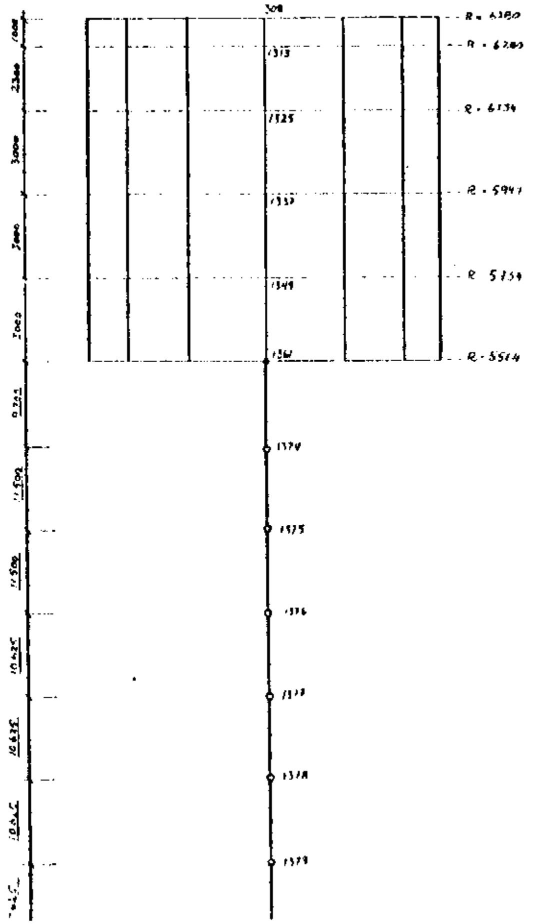
E 5877 NUMBER CHANGED
A 4775 ISSUED FOR INFORMATION

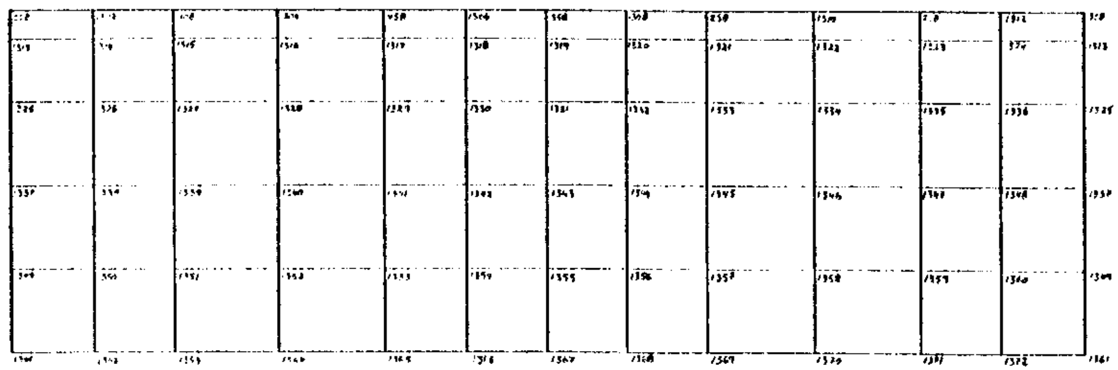
ELF – NORGE A/S – E 10
FRIGG – TCP 2 STEEL FRAME

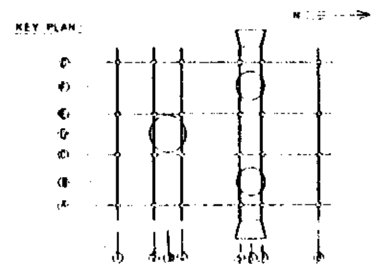
STRUCTURAL ANALYSIS
MATHEMATICAL MODEL
SKEW BRACING
CELLAR DECK

UC 3337 0449 20 1003 6









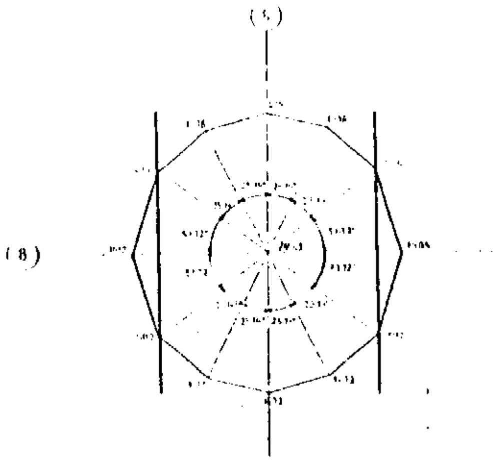
B	3677 DWG. TITLE CORRECTED
A	4775 ISSUED FOR INFORMATION

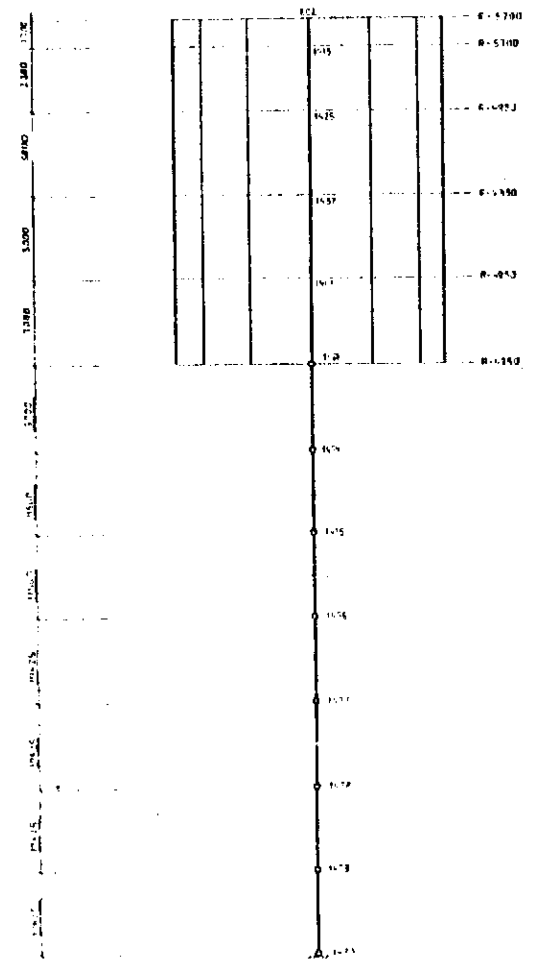
ELF – NORGE A/S – E 10

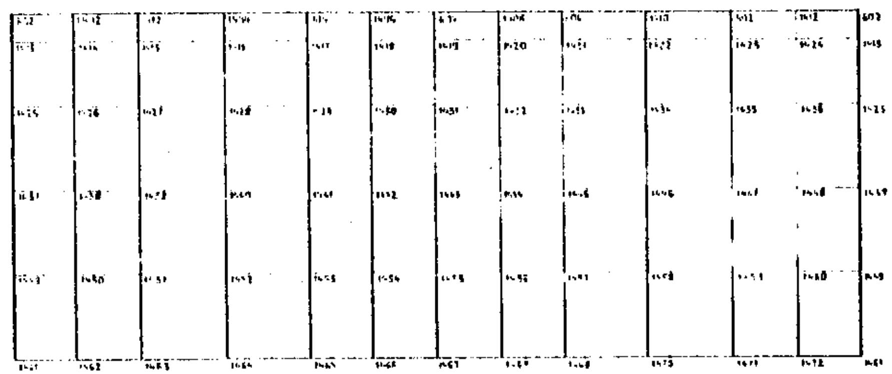
FRIGG – TCP 2 – STEEL FRAME

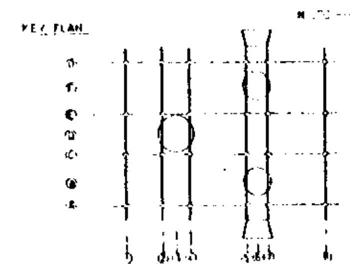
STRUCTURAL ANALYSIS
MATHEMATICAL MODEL
CONCR. ELEMENT MODEL
SUPPORT RING OF SHAFT

UG 3337 0449 20 1004 B









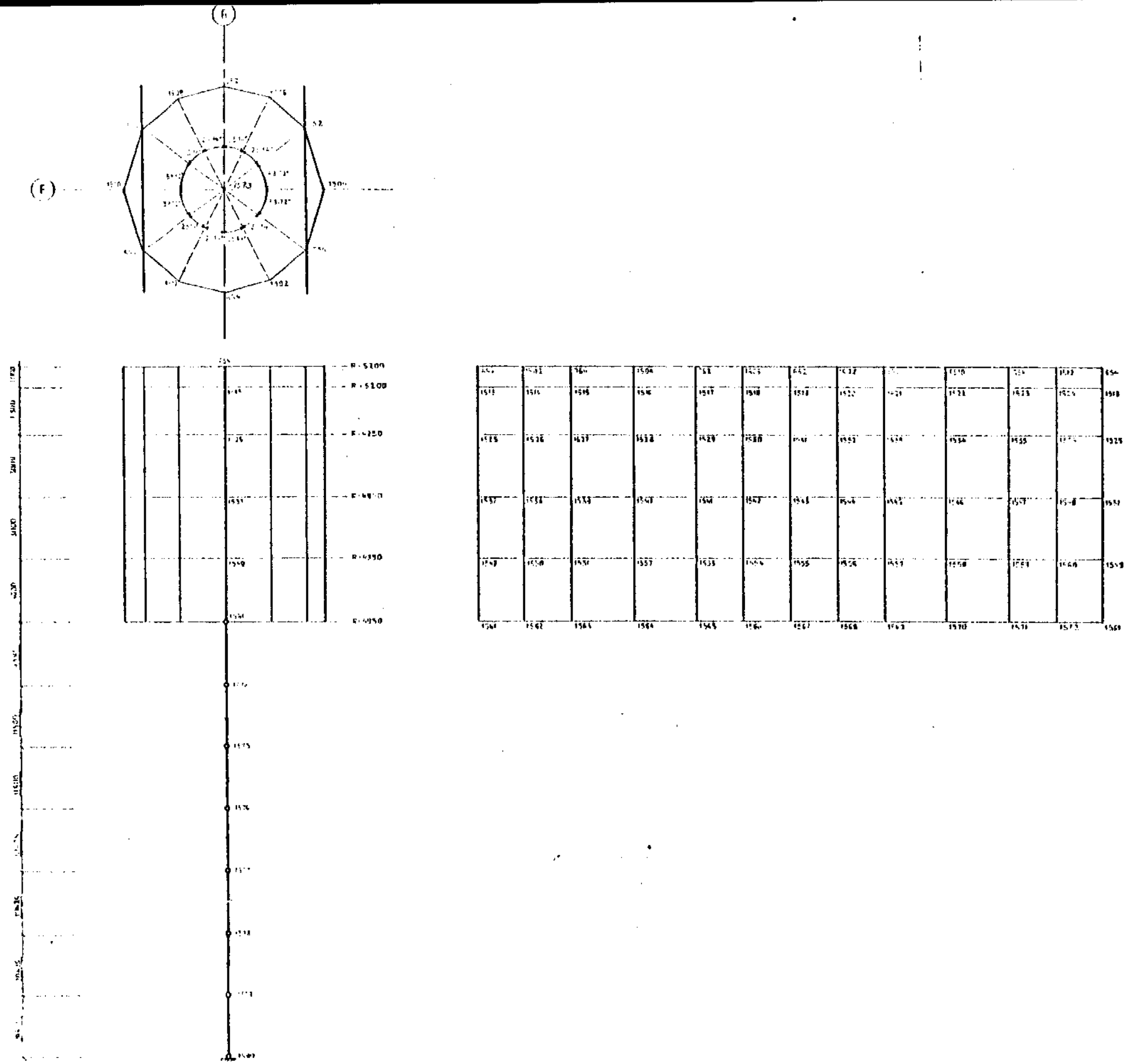
B 5877 DWG TITLE CORRECTED
A 4775 ISSUED FOR INFORMATION

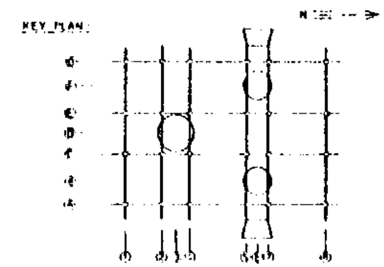
COMSEP

ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

STRUCTURAL ANALYSIS
MATHEMATICAL MODEL
COARSE ELEMENT MODEL
SUPPORT RING B6-SHAFT

HC 3337 0449 20 1005



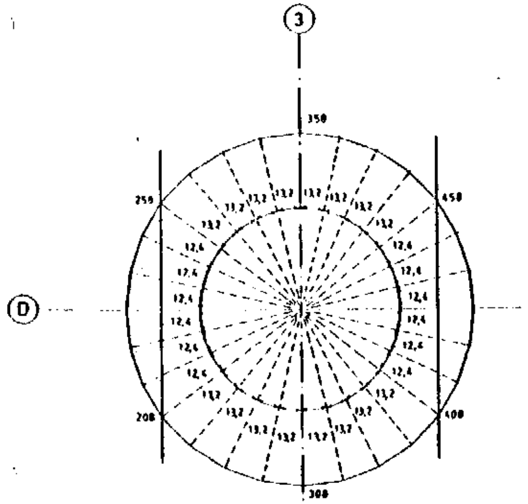


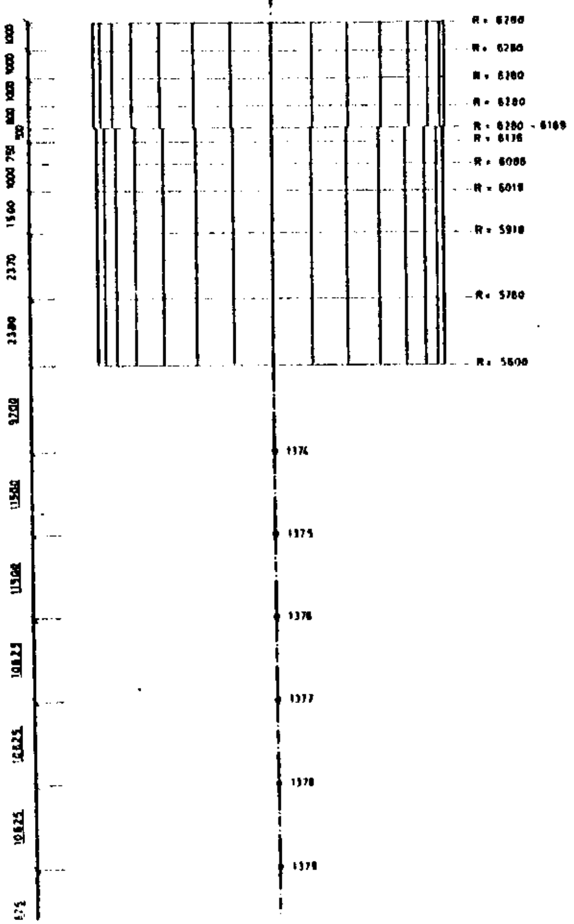
B 5877 DWG TITLE CORRECTED
A 4775 ISSUED FOR INFORMATION

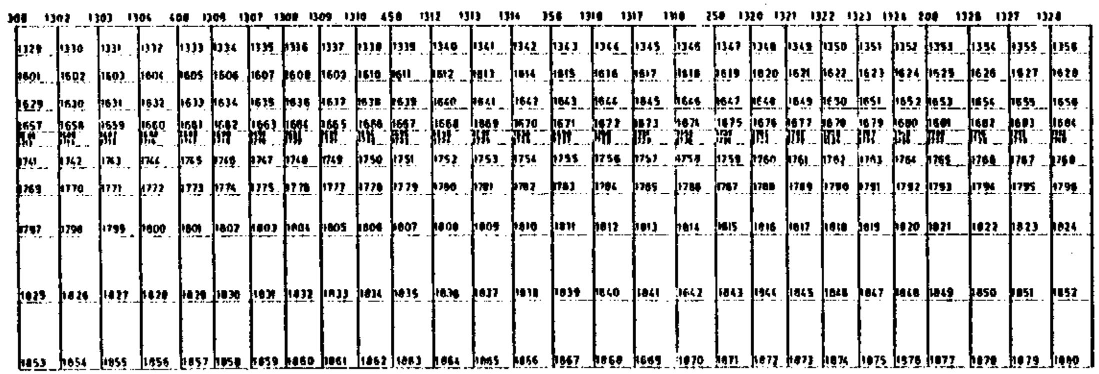
ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

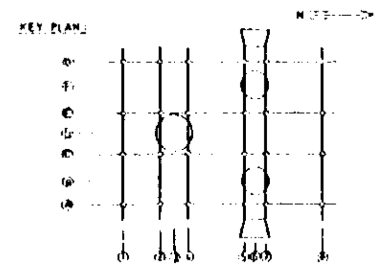
STRUCTURAL ANALYSIS
MATHEMATICAL MODEL
COARSE ELEMENT MODEL
SUPPORT RING F6-SHAFT

UC 1337 0449 20 1066 B









B	5837	DWG TITLE CORRECTED
A		ISSUED FOR INFORMATION

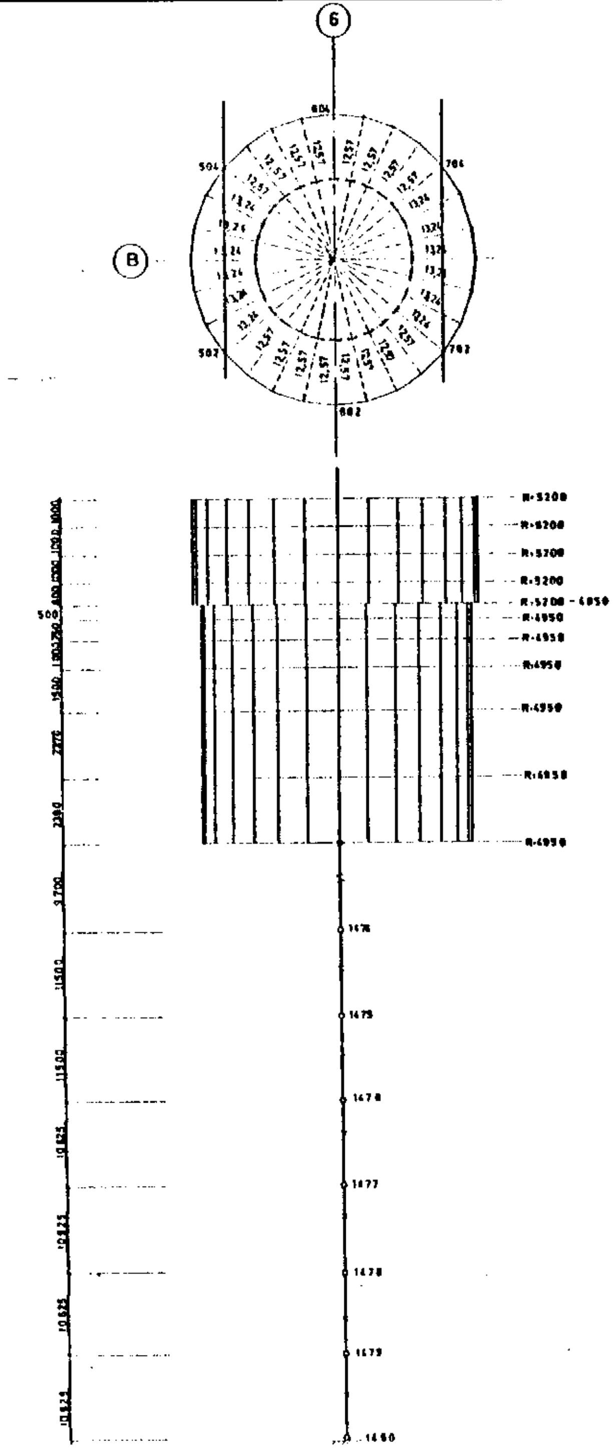
CONTECP

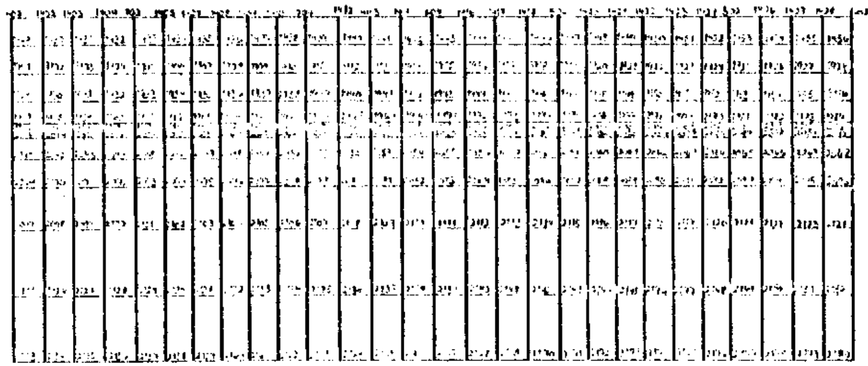
ELF – NORGE A/S – E 10

FRIGG TCP 2 – STEEL FRAME

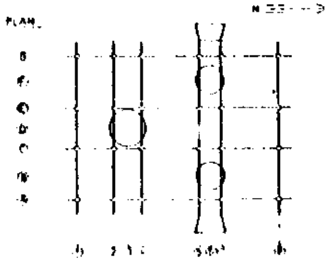
STRUCTURAL ANALYSIS
MATHEMATICAL MODEL –
FINITE ELEMENT MODEL
SUPPORT RING OF SHAFT

UC 3337 0449 20 1009 B





KEY PLAN



B 38.77 DWG TITLE CORRECTED
A ISSUED FOR INFORMATION

COMDEEP

HOVER

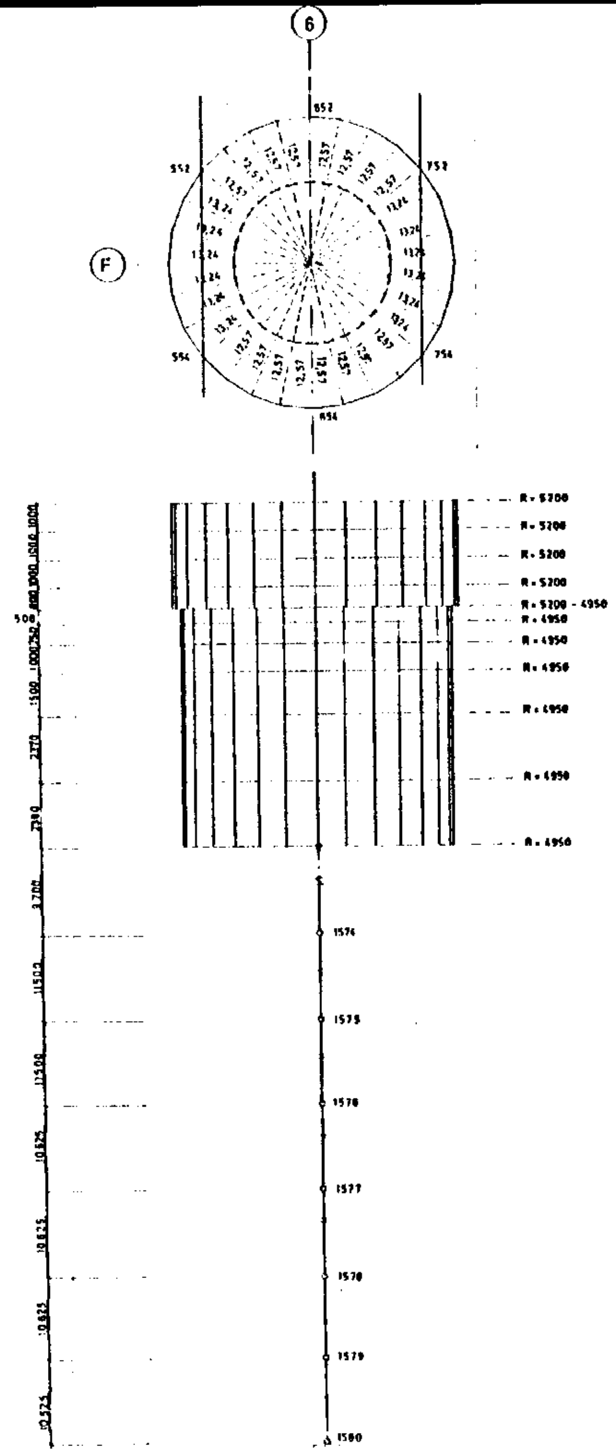
OSLO

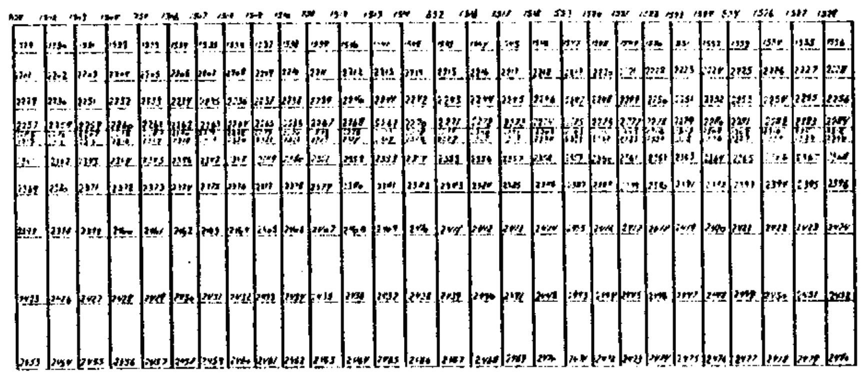
ELF - NORGE A/S - E 10
FRIGG - TCP 2 STEEL FRAME

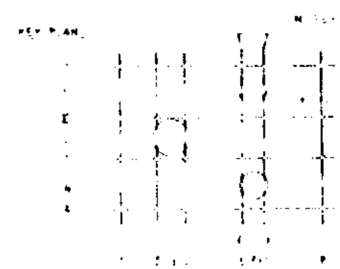
STRUCTURAL ANALYSIS
MATHEMATICAL MODEL
REFINED ELEMENT MODEL
SUPPORT RING B6-SHAFT

01

UC 3337 0449 20 1010







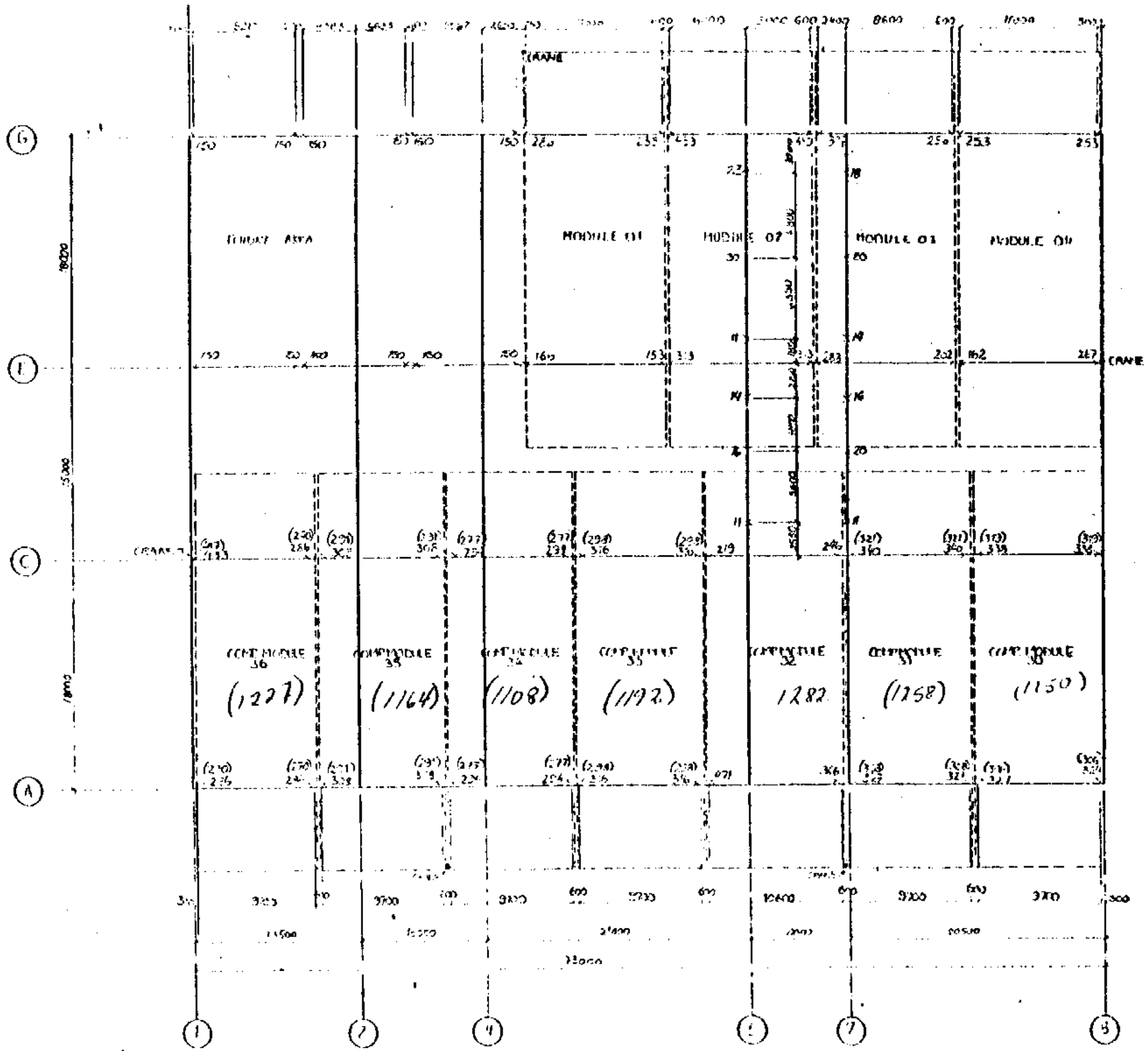
B 5.8.77 DWG TITLE CORRECTED
A ISSUED FOR INFORMATION

ELF - NORGE A/S - E 10
FRIGG TCP 2 STEEL FRAME

STRUCTURAL ANALYSIS
MATHEMATICAL MODELS
REFINED ELEMENT MODEL
SUPPORT RING F 6 - SHAFT

CONDEEP

UC 3337 0449 20 1011



REFERENCES

ELF/TELEPRINT 26.1.77, LOADCASE II

McDH DWG ELN 2117 SK-5-101 Rev 10

ELN 2117 SK-5-137 Rev 1

McDH EQUIPMENT WEIGHT SHEDULE Rev. 24

NOTES:

THE COMPRESSOR MODULE LOADS ARE INCREASED BY SIX PERCENT (DYNAMIC FACTOR 1.06)

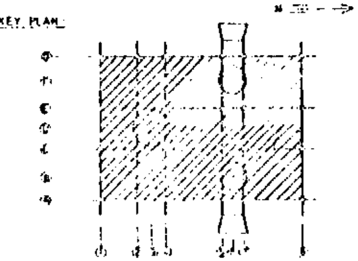
THE COMPRESSOR MODULE LOADS IN PARENTHESIS ARE WITHOUT DYNAMIC FACTOR

UNITS:

DIMENSIONS IN mm

LOADS IN METRIC TONS

KEY PLAN:



ELF - NORGE A/S - E 10

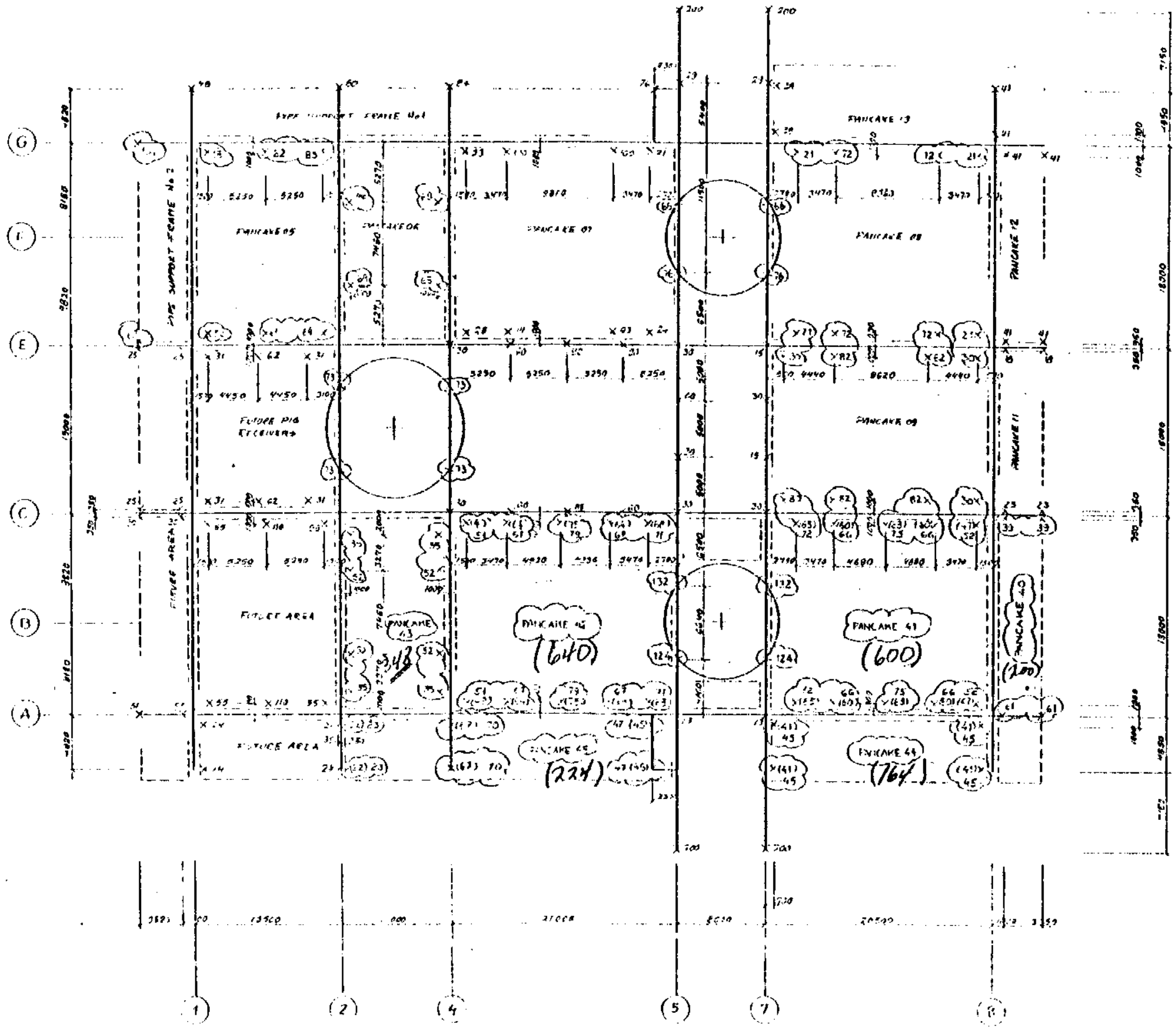
FRIGG - TCP 2 - STEEL FRAME

STRUCTURAL ANALYSIS

LIVE LOAD MAIN DECK

LOAD CASE LIVE LOAD I

1:200



REFERENCES

ELF TELEPRINT 26.1.77, LOADCASE II
[illegible] DWG. [illegible] 2177 SK S III Rev 7
MODI EQUIPMENT WEIGHT SCHEDULE Rev 24

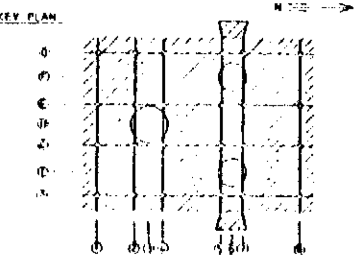
NOTES:

THE COMPRESSOR PANCAKE LOADS ARE INCREASED BY TEN PERCENT (DYNAMIC FACTOR 1.10) ON PANCAKE 41 AND 44, AND BY FIVE PERCENT (DYNAMIC FACTOR 1.05) ON PANCAKE 42 AND 45.
THE COMPRESSOR PANCAKE LOADS IN PARANTHESIS ARE WITHOUT DYNAMIC FACTOR

UNITS:

DIMENSIONS IN mm
LOADS IN METRIC TONS

KEY PLAN



Rev	Date	Revision
H		LOADS UPDATED
G	16.11.76	LOADS UPDATED
F	15.3.76	LOADS UPDATED
E	22.1.76	GENERAL REVISION
D	12.12.75	AFTER ELN 2177 SK S III REV 7
C	18.11.75	GENERAL REVISION
B	6.8.75	LOADS CHANGED
A	4.7.75	ISSUED FOR INFORMATION

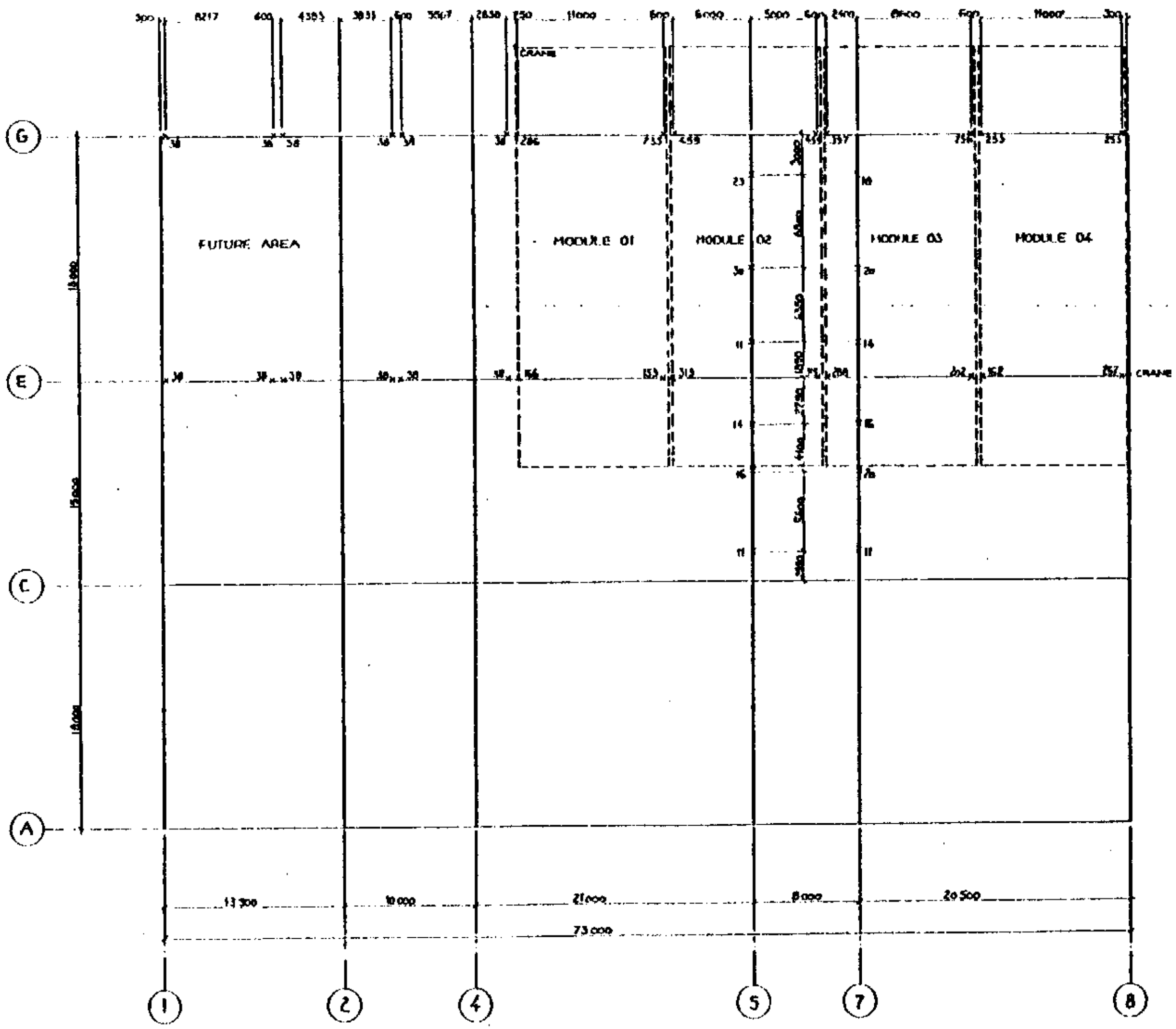
CONTECP

ELF - NORGE A/S - E 10
FRIGO - TCP 2 - STEEL FRAME

STRUCTURAL ANALYSIS
LIVE LOAD CELLAR DECK
LOAD CASE LIVE LOAD 1

Scale 1:200

UC 3337 0449 20 1022 H



REFERENCES:

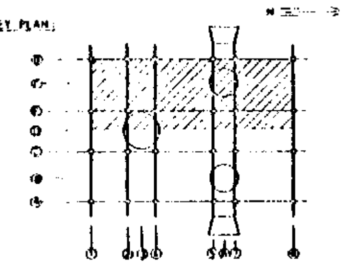
FROM DWG ELN 2177 SK-S-111 Rev 10
ELN 2177 SK-S-157 Rev 1
FROM EQUIPMENT WEIGHT SCHEDULE Rev 24

NOTES:

UNITS:

DIMENSIONS IN mm
LOADS IN METRIC TONS

KEY PLAN:



Rev	Date	Revision
E	15.8.77	GEN. REVISION
D	15.3.76	LOADS UPDATED
C	28.1.76	GENERAL REVISION
B	1.12.75	REVISED
A	20.11.75	ISSUED FOR INFORMATION

CONDTEP

AKER HOYER

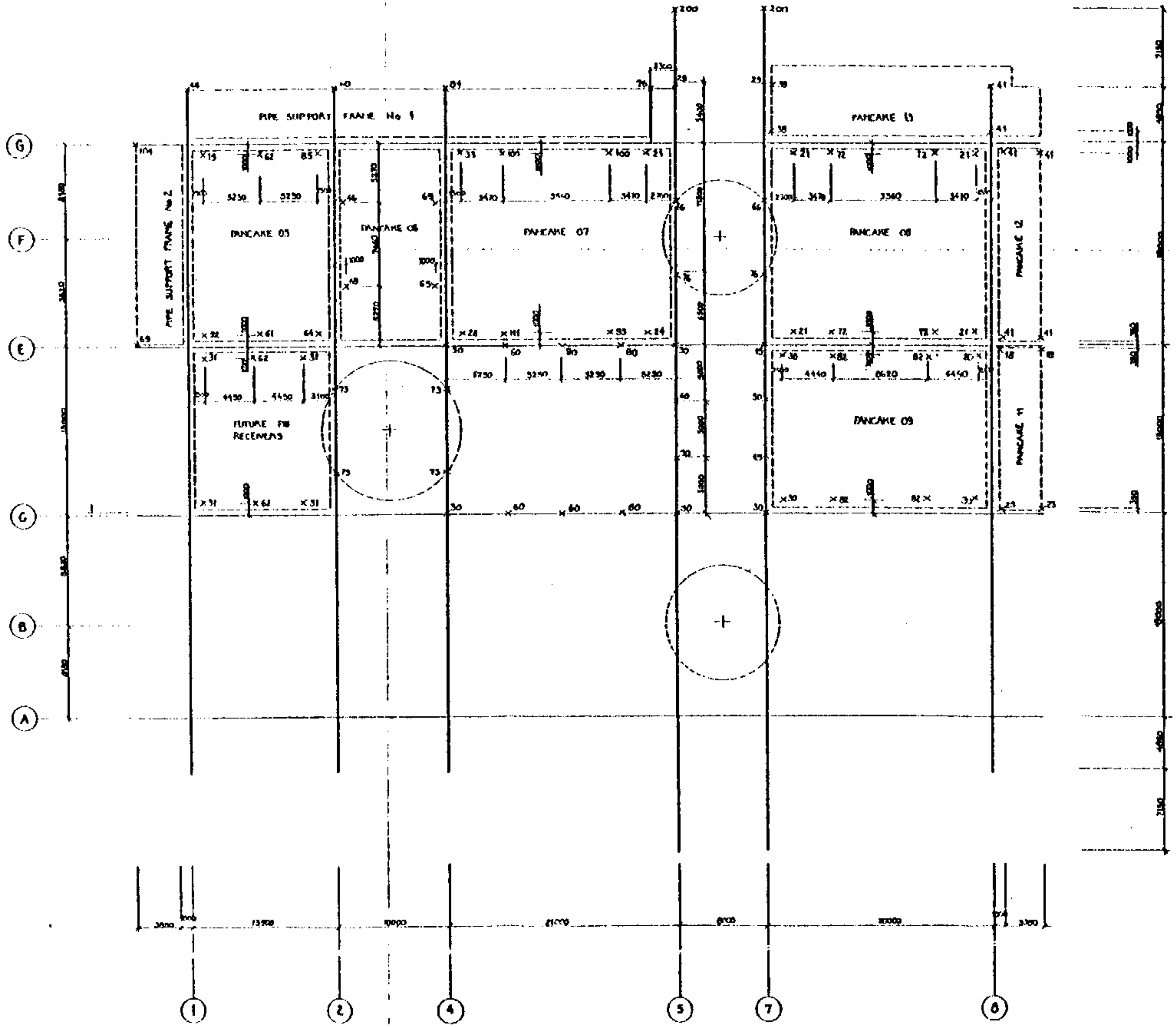
OSLO NORWAY

ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

STRUCTURAL ANALYSIS
LIVE LOAD MAIN DECK
LOAD CASE: LIVE LOAD 2

Scale 1:200

UC 3337 0449 20 1023



REFERENCES:

McDH DWS ELN 2111 SK-S-111 Rev 10

—— —— ELN 2111 SK-S-151 Rev 1

McDH EQUIPMENT WEIGHT SCHEDULE Rev 24

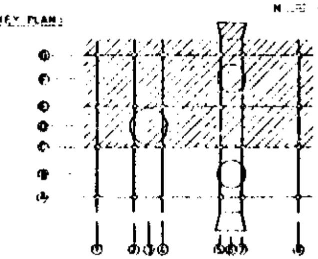
NOTES:

UNITS:

DIMENSIONS IN mm

LOADS IN METRIC TONS

KEY PLAN:



Rev.	Date	Revision
D	15.8.77	GEN. REVISION
C	15.5.76	LOADS UPDATED
B	28.1.76	GENERAL REVISION
A	20.11.75	ISSUED FOR INFORMATION

CONDEEP

A.S HOYER ELLEFSEN A/S

OSLO

ELF - NORGE A/S - E 10

FRIGG - TCP 2 - STEEL FRAME

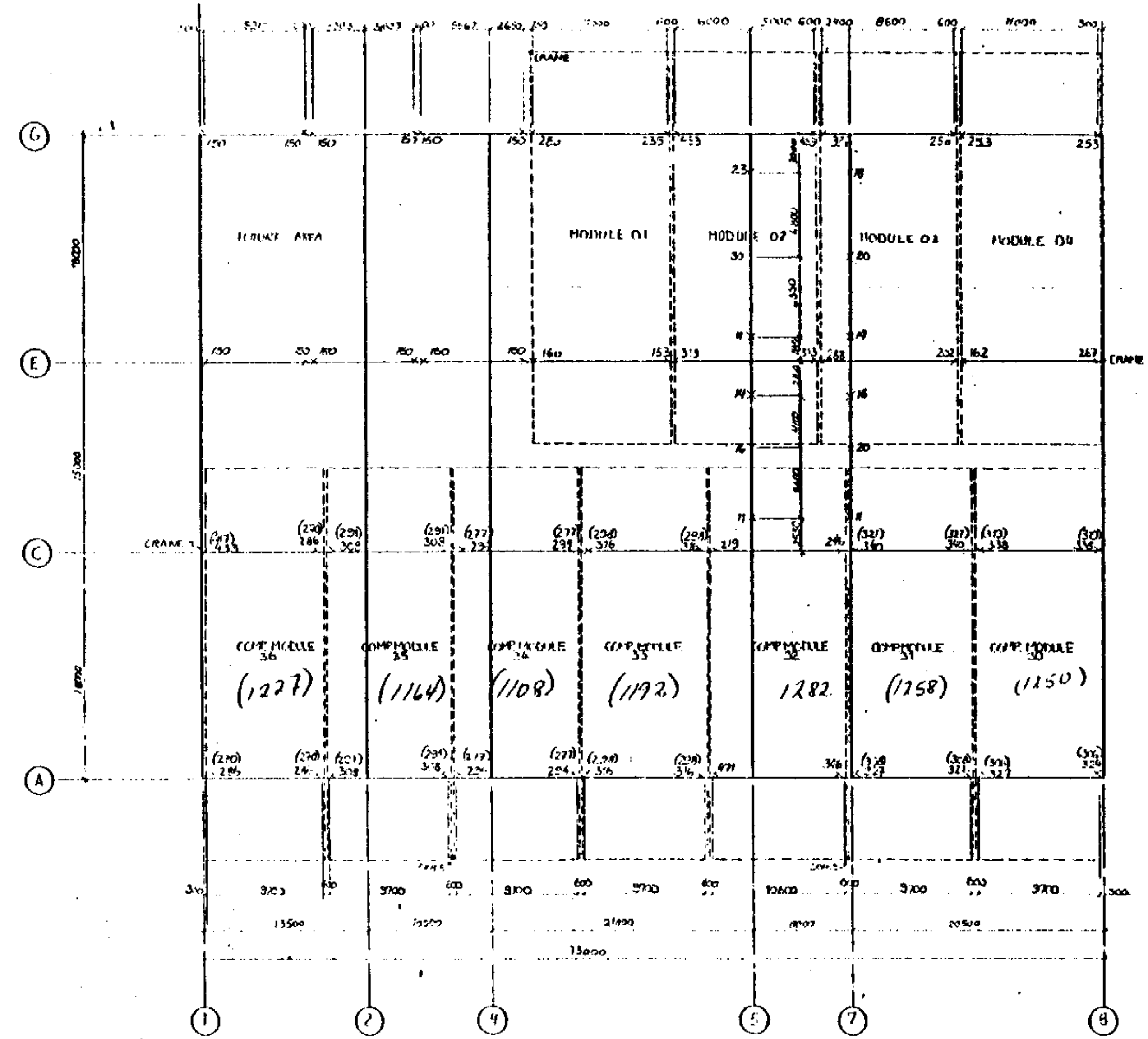
STRUCTURAL ANALYSIS

LIVE LOAD CELLAR DECK

LOAD CASE: LIVE LOAD 2

Scale 1:200

UC 3337 0449 20 1024



REFERENCES

ELF/TELEFRINT 26.1.77, LOADCASE 11

McDH DWG ELN 2177 SK-5-111 Rev 10

ELN 2177 SK-5-137 Rev 1

McDH EQUIPMENT WEIGHT SCHEDULE Rev. 24

NOTES:

THE COMPRESSOR MODULE LOADS ARE INCREASED BY SIX PERCENT (DYNAMIC FACTOR 1.06)

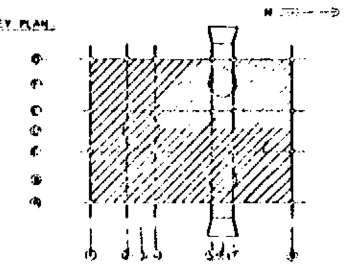
THE COMPRESSOR MODULE LOADS IN PARENTHESIS ARE WITHOUT DYNAMIC FACTOR

UNITS:

DIMENSIONS IN mm

LOADS IN METRIC TONS

KEY PLAN:



E ... AFTER ELN 2177 SK 5-111 REV ...
D ... GENERAL REVISION
C 27.11.75 LOADS IN AXIS C
B 6.8.75 CRANE LOAD CHANGED
A ... ISSUED FOR INFORMATION

ELF - NORGE A/S E 10

FRIGG - TCP 2 - STEEL FRAME

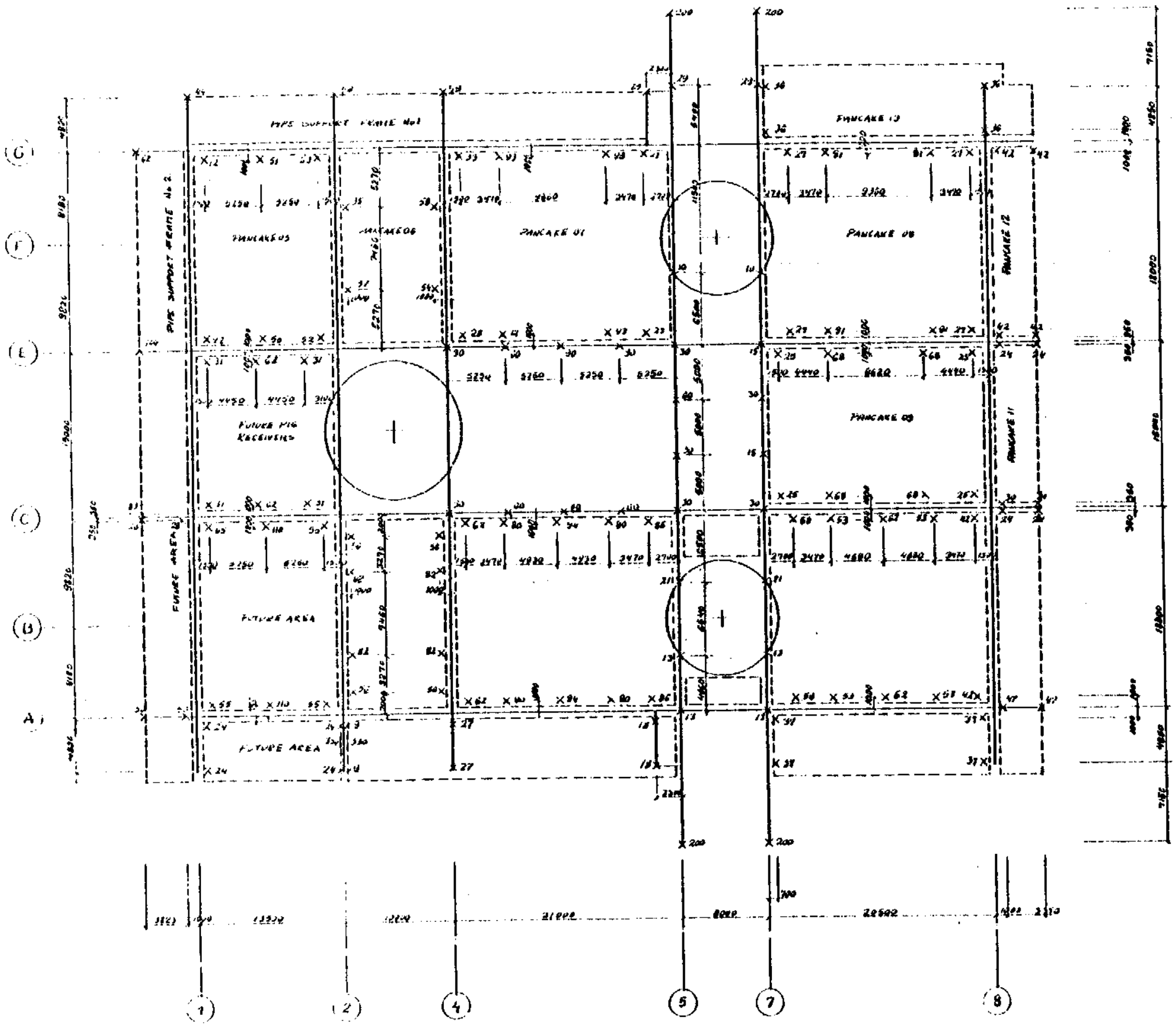
STRUCTURAL ANALYSIS

LIVE LOAD MAIN DECK

LOAD CASE LIVE LOAD I

1:200

PC 3337 0449 29 1021



REFERENCES:

ELF DWG 140 E 2 0047 Rev 0
------------ 140 E 2 0048 Rev 0
McDH DWG ELN 2177 SK-S-111 Rev 7
McDH EQUIPMENT WEIGHT SCHEDULE Rev 11

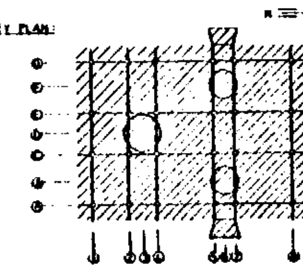
NOTES.

INTERNAL LOADS ON STEEL SUPPORT RING: SEE DWG 1025

UNITS:

DIMENSIONS IN mm
LOADS IN METRIC TONS

KEY PLAN:



Rev	Date	Revision
E	22.1.76	GENERAL REVISION
D	17.12.75	AFTER ELN 2177 SK-S-111 REV 7
C	18.11.75	GENERAL REVISION
B	6.8.75	LOADS CHANGED
A	4.7.75	ISSUED FOR INFORMATION

CONDEEP

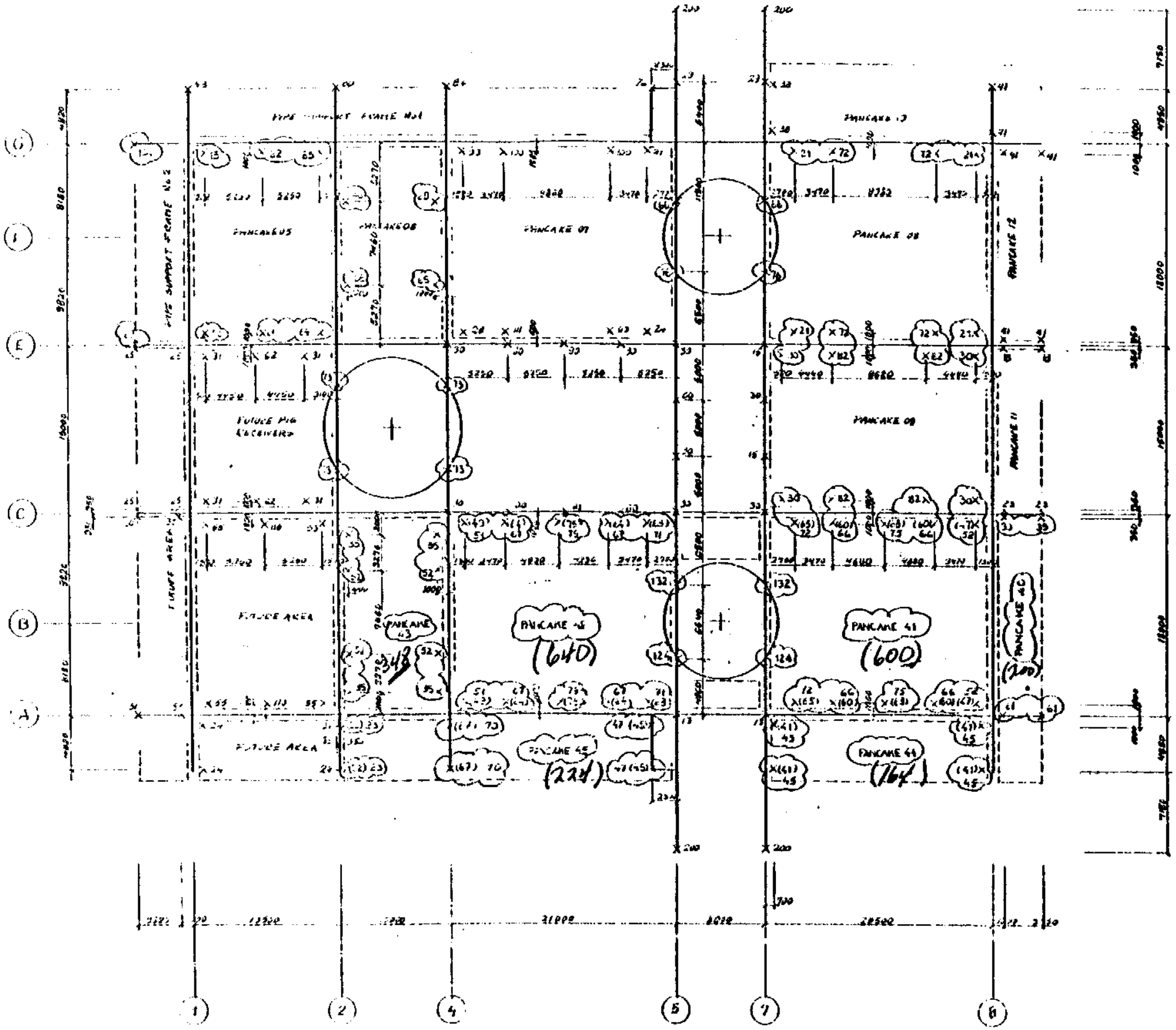
A/S HØYER ELLEFSEN OSLO

ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

STRUCTURAL ANALYSIS
LIVE LOAD CELLAR DECK
LOAD CASE: LIVE LOAD 1

Scale 1:200

UC 3337 0449 20 1022



REFERENCES.

ELF TELEPRINT 26.1.77, LOADCASE II
MCDH DWG ELN 2177, SK-S-117 Rev 15
MCDH EQUIPMENT WEIGHT SCHEDULE Rev 24

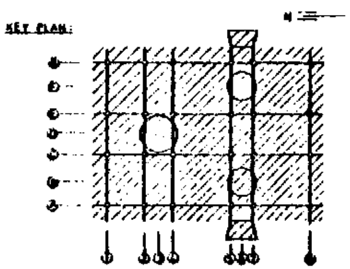
NOTES.

THE COMPRESSOR PANCAKE LOADS ARE INCREASED BY TEN PERCENT (DYNAMIC FACTOR 1.10) ON PANCAKE 41 AND 44, AND BY FIVE PERCENT (DYNAMIC FACTOR 1.05) ON PANCAKE 42 AND 45
THE COMPRESSOR PANCAKE LOADS IN PARANTHESIS ARE WITHOUT DYNAMIC FACTOR.

UNITS.

DIMENSIONS IN mm
LOADS IN METRIC TONS

KEY PLAN:



Rev.	Date	Revision
H		LOADS UPDATED
G	16.11.76	LOADS UPDATED
F	15.1.76	LOADS UPDATED
E	22.1.76	GENERAL REVISION
D	17.12.75	AFTER ELN 2177 SK-S-111 REV 2
C	18.11.75	GENERAL REVISION
B	6.8.75	LOADS CHANGED
A	7.7.75	ISSUED FOR INFORMATION

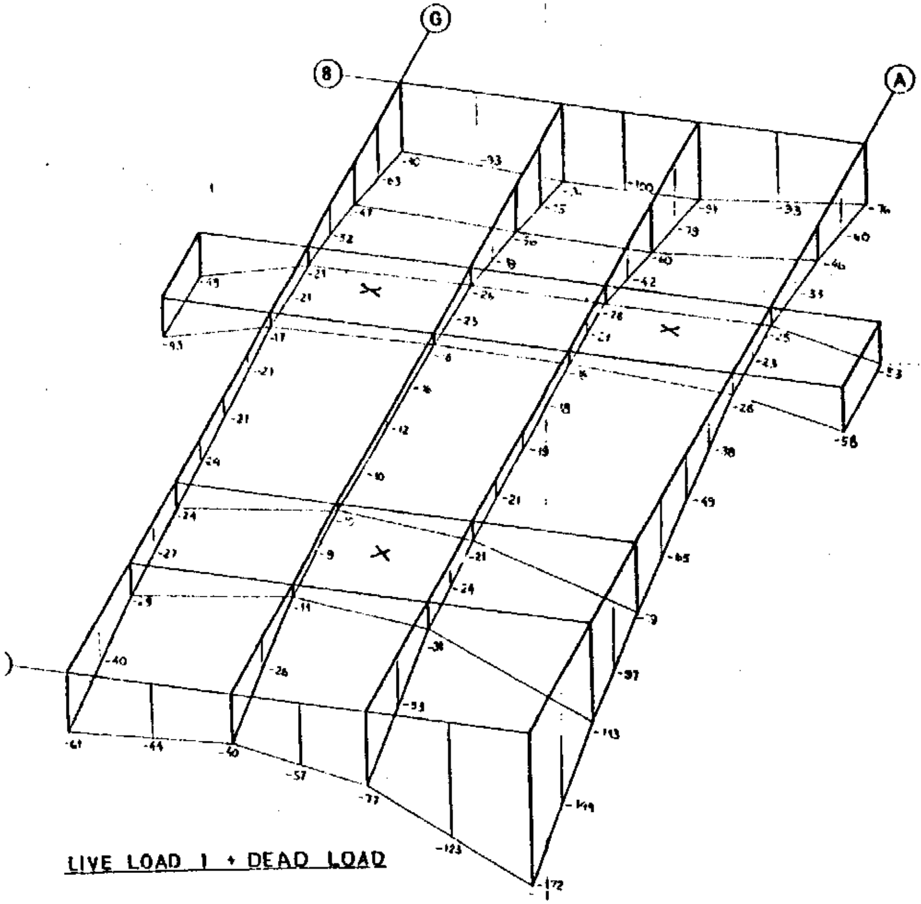
CONDEEP

ELF - NORGE A/S - E 10
FRIGG-TCP 2 - STEEL FRAME

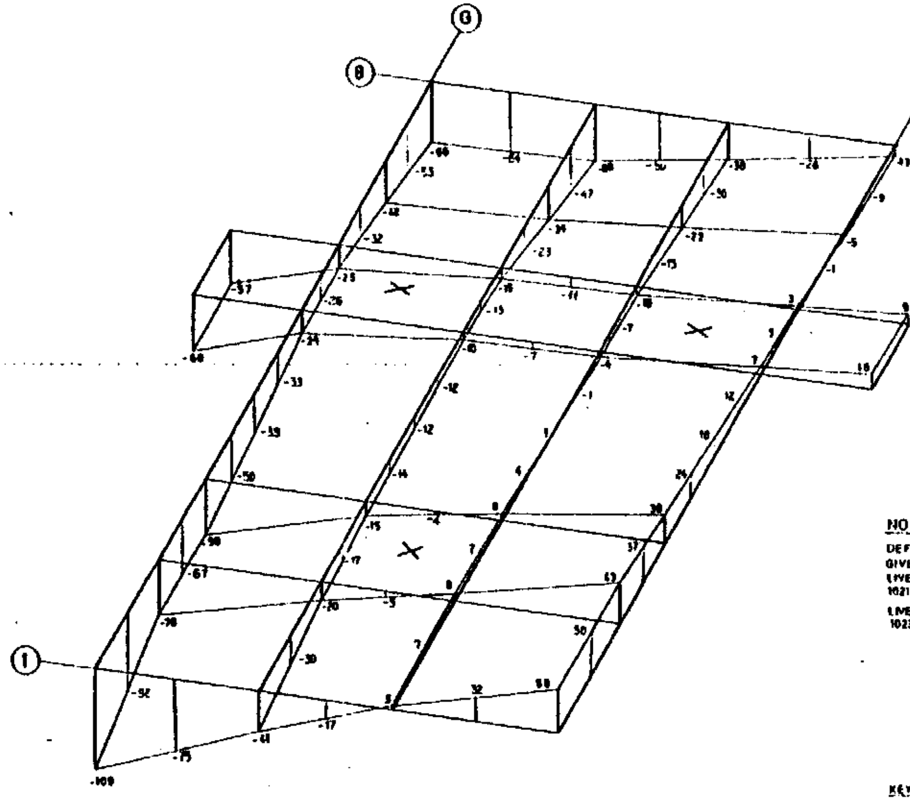
STRUCTURAL ANALYSIS
LIVE LOAD CELLAR DECK
LOAD CASE. LIVE LOAD 1

Scale 1:200

A.F. HØYER ELLEFSEN
OSLO



LIVE LOAD 1 + DEAD LOAD



LIVE LOAD 2 + DEAD LOAD

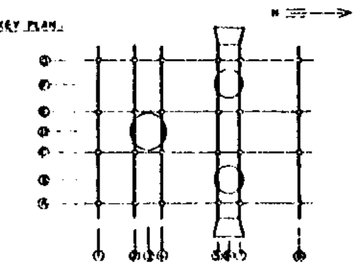
NOTES:

DEFLECTION ARE GIVEN IN mm.

LIVE LOAD 1 ARE GIVEN ON KE-DWG: 1021 J AND 1022 H

LIVE LOAD 2 ARE GIVEN ON KE-DWG: 1023 C, 1024 B AND 1025 A.

KEY PLAN:



C	[illegible]	LIVE LOAD 1 REVISED
B	[illegible]	INTERMEDIATE POINTS ADDED
A	15 7 76	ISSUED FOR INFORMATION

CONDEEP

A. J. HOYER

OSLO

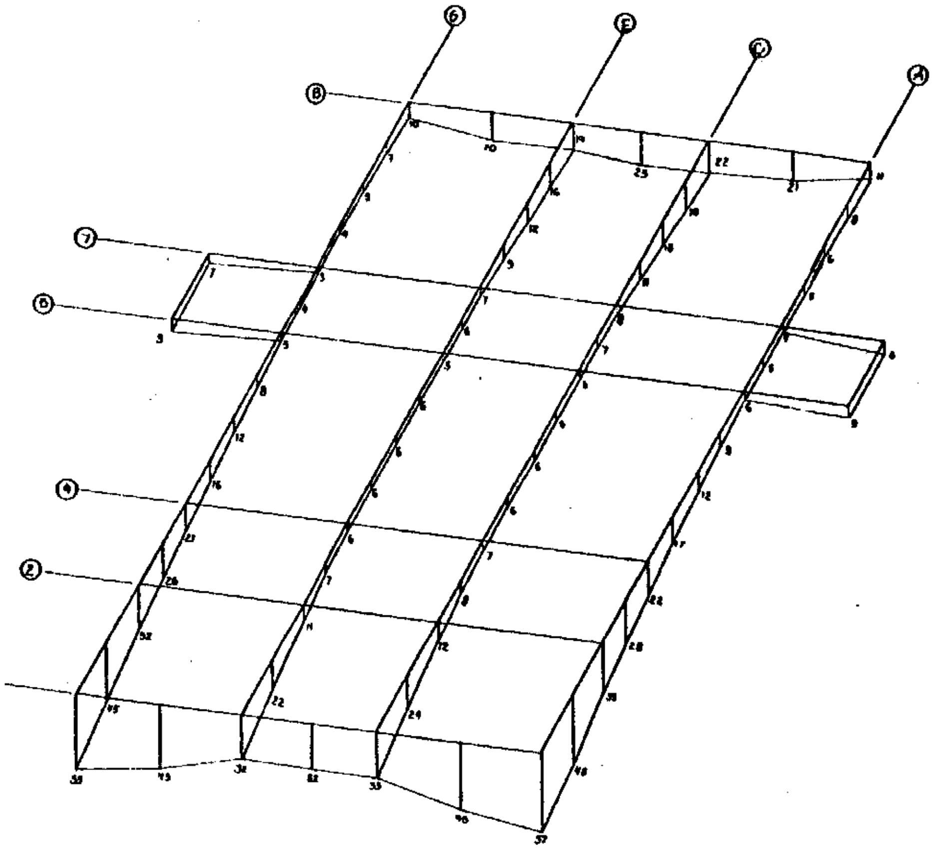
ELF – NORGE A/S – E 10

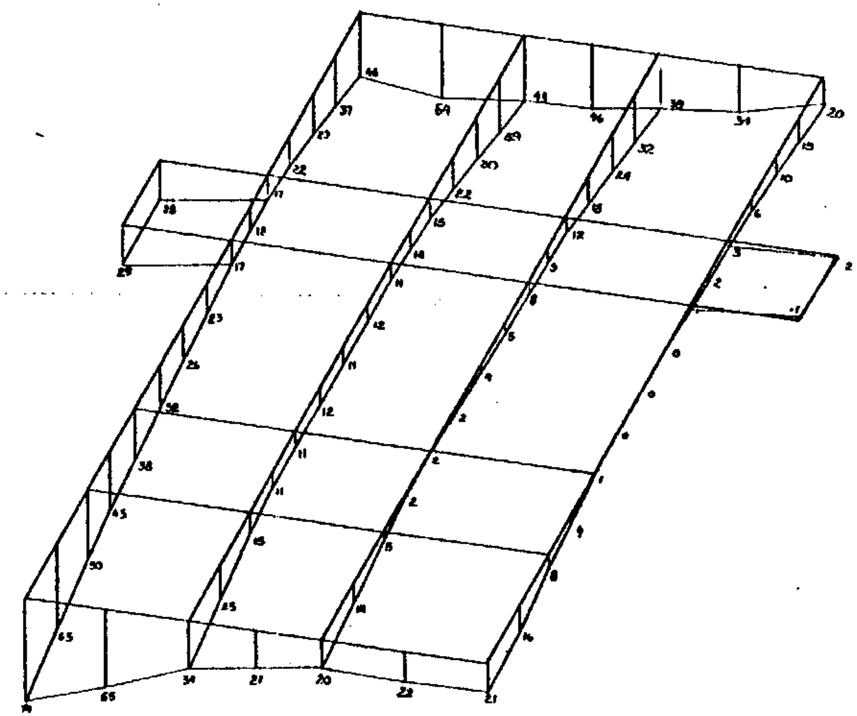
FRIGG – TCP 2 – STEEL FRAME

STRUCTURAL ANALYSIS
VERTICAL DEFLECTION
REVISED LIVE LOAD

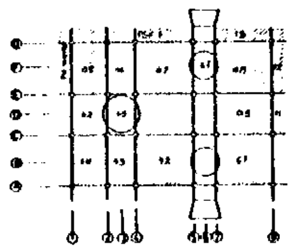
1:300

UC 3337 0449 20 1044 C



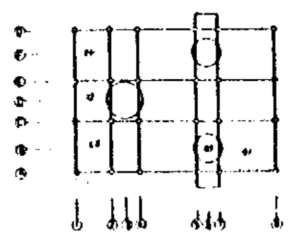


LOAD CASE 1



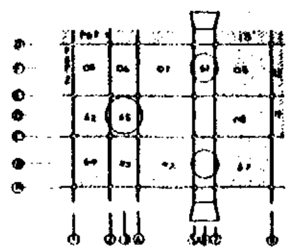
CELLAR DECK

NOTE:
42 AND 43
ARE GROUTING
PANCAKES



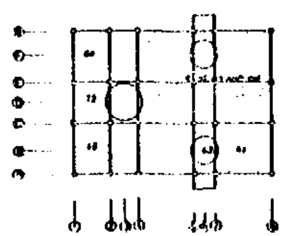
MAIN DECK

LOAD CASE 2



CELLAR DECK

NOTE ONLY DEAD WEIGHT OF PANCAKES 42 AND 43



MAIN DECK

NOTE DEFLECTIONS IN MM

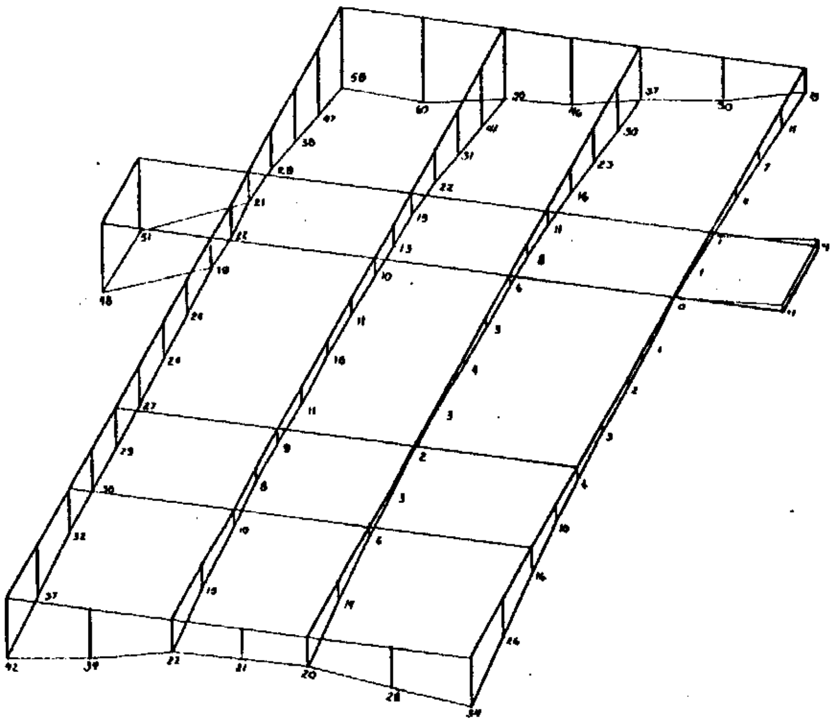
ALL DEFLECTIONS ARE DOWNWARDS
EXCEPT THOSE MARKED +
DEAD WEIGHT OF FRAME IS INCLUDED

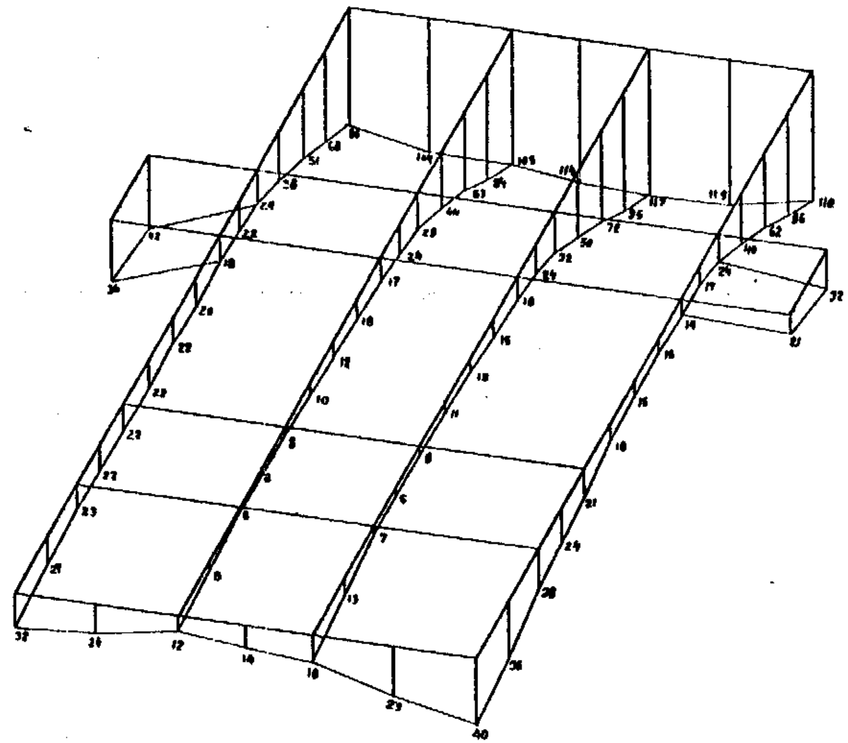
B	08.11.76	NOTE ADDED
A	25.9.76	ISSUED FOR INFORMATION

DEFLECTION ANALYSIS
VERTICAL TRANSLATION
LOAD CASE 1-2

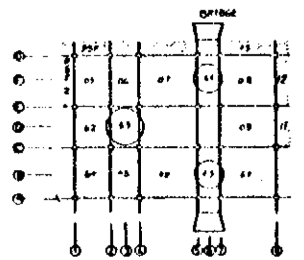
1:300

OC 3337 0410 20 1016 B

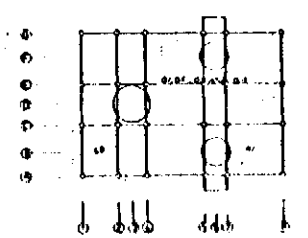




LOAD CASE 3

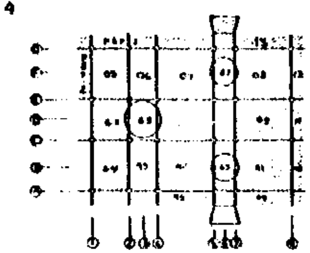


CELLAR DECK



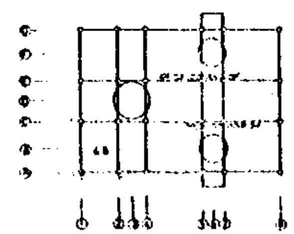
MAIN DECK

LOAD CASE 4



CELLAR DECK

NOTE: 40, 41, 42, 43, 44 AND 45 ARE COMPRESSION PANCAKES



MAIN DECK

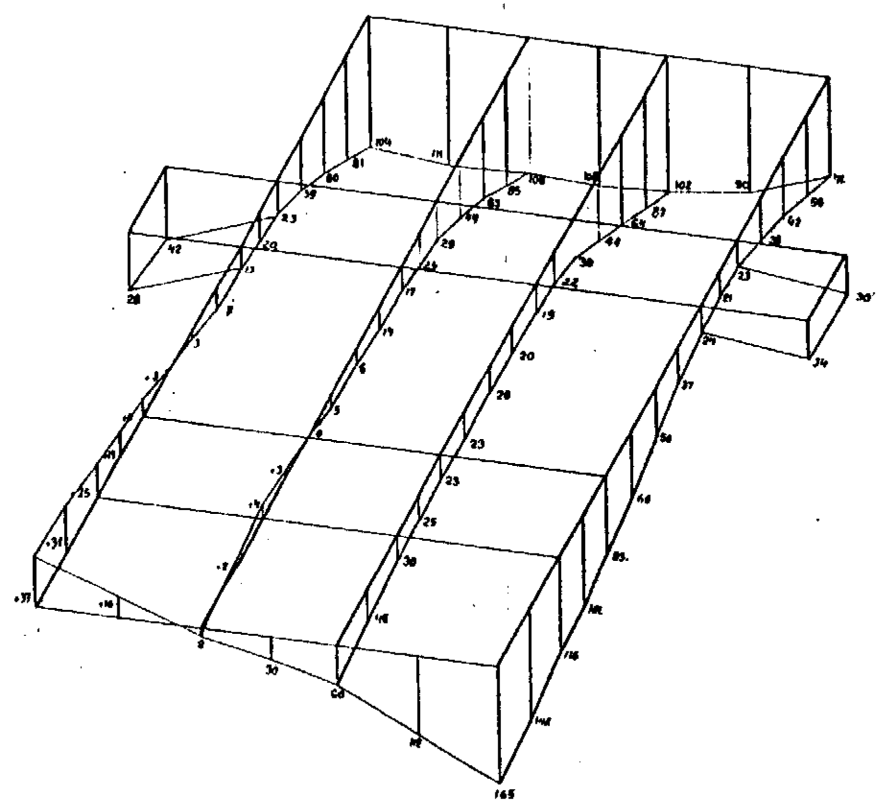
NOTE: DEFLECTIONS IN MM
ALL DEFLECTIONS ARE DOWNWARDS EXCEPT THOSE MARKED +
DEAD WEIGHT OF FRAME IS INCLUDED

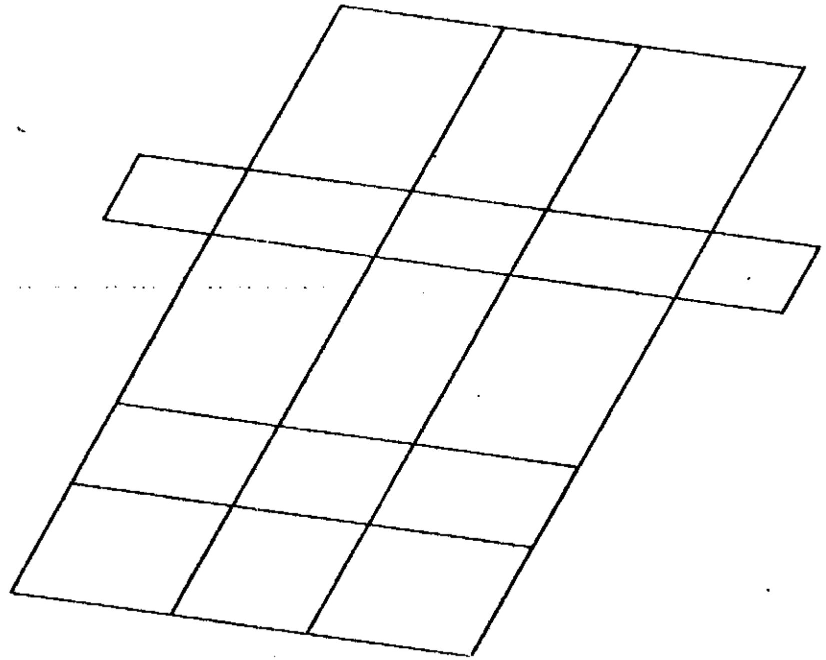
Rev.	Date	Revision
C	[illegible]	LOAD CASE 4 UPDATED
B	[illegible]	NOTE ADDED
A	[illegible]	ISSUED FOR APPROVAL

DEFLECTION ANALYSIS
VERTICAL TRANSLATION
LOAD CASE 3-4

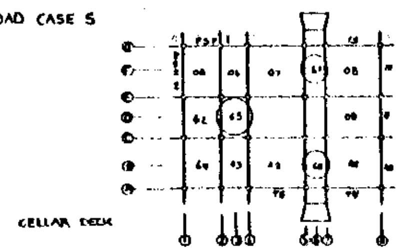
Scale 1:800

UC 3357 0418 20 1047 C

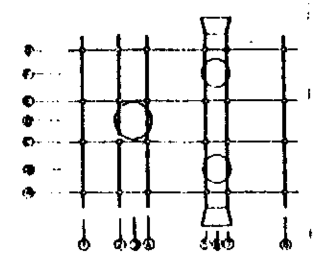


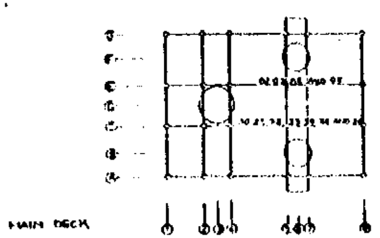


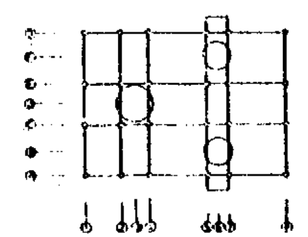
LOAD CASE 5



NOTE:
40, 41, 42, 95, 97 AND 98
ARE COMPRESSION
PANCAKES







NOTE: DEFLECTIONS IN MM

ALL DEFLECTIONS ARE DOWNWARDS
EXCEPT THOSE MARKED +
DEAD WEIGHT OF FRAME IS INCLUDED

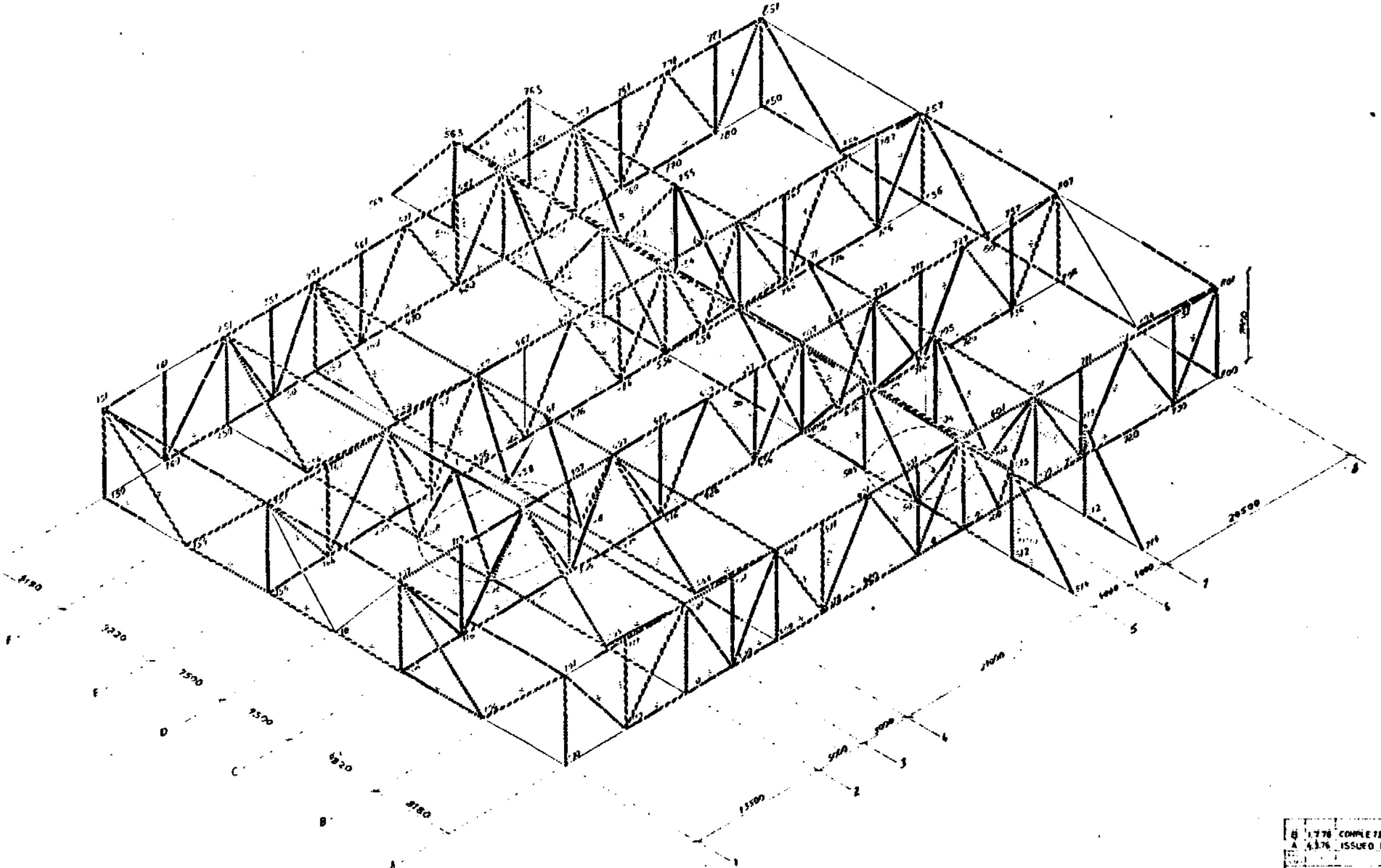
Rev.	Date	Revision
C		LOAD CASE 5 UPDATED
B	06.10.76	NOTE ADDED
A	20.9.76	ISSUED FOR INFORMATION

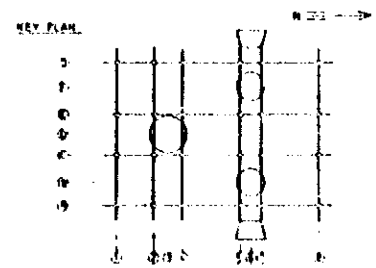
DEFLECTION ANALYSIS
VERTICAL DEFLECTIONS

LOAD CASE 5

Scale 1 300

UC 3557 | 0449 | 20 | 1048 | C





Operational condition
Approximate stress levels

B	1.7.76	COMPLETED
A	4.3.76	ISSUED FOR INFORMATION

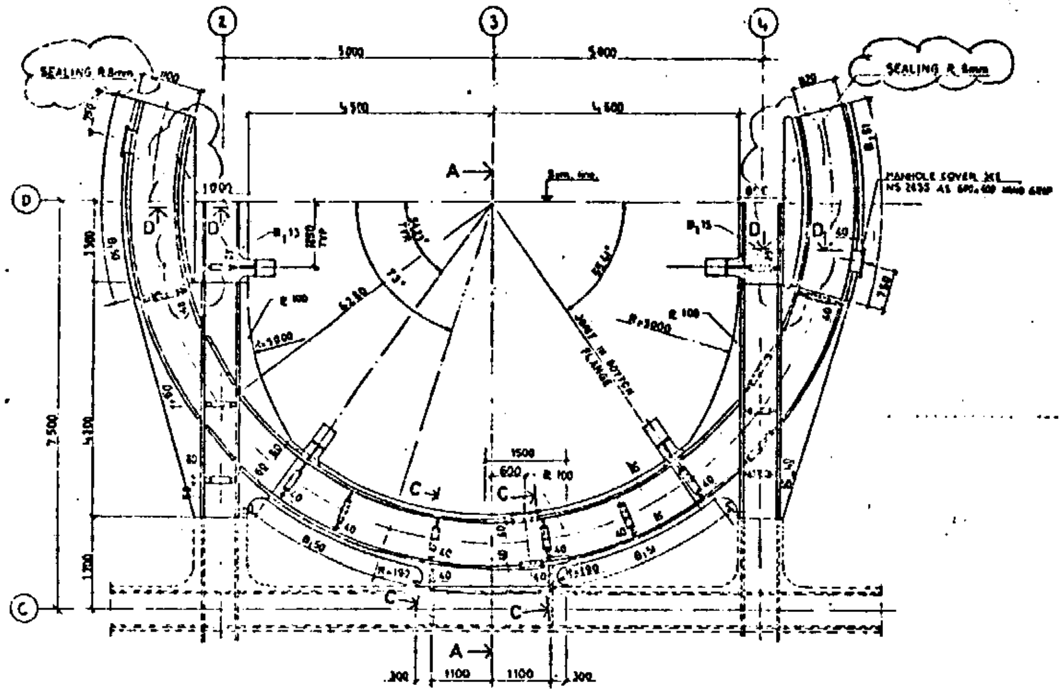
ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

CONDEEP

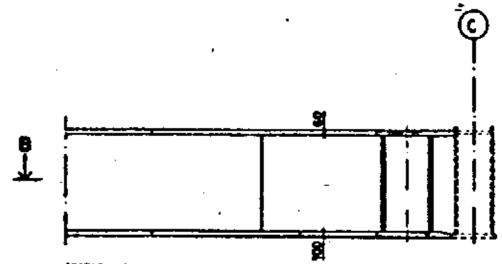
STRUCTURAL ANALYSIS
STRESS LEVELS

HØYER

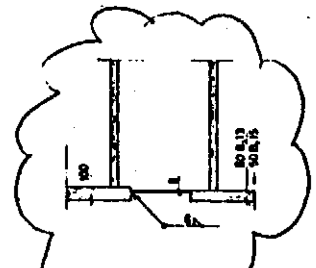
UC 3337 0449 20 1053 B



SECTION B-B



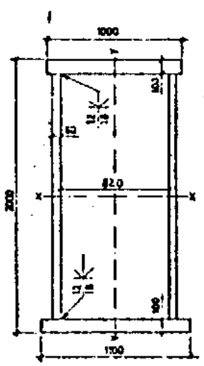
SECTION A-A



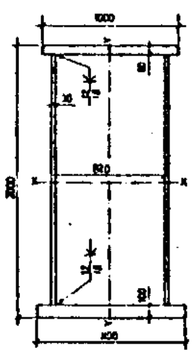
SECTION D-D

NOTES:

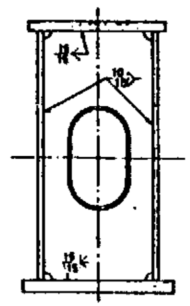
GENERAL NOTES SEE DWG 0103
DETAILS CANTILEVERS SEE DWG 0904



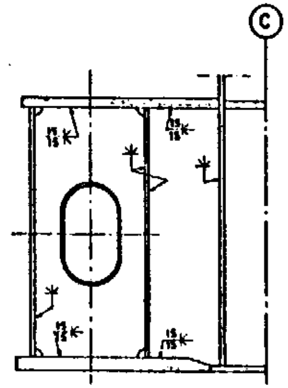
B_1 50



B_1 51

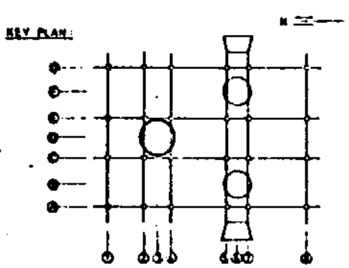


* WELDS BULK HEAD B_1 50 AND B_1 51
WELDS BULK HEAD AT CANTILEVER SEE DWG 0904



SECTION C-C

KEY PLAN:



F	28.5.76	SEALING PLATES ADDED
G	8.3.76	ISSUED FOR CONSTRUCTION
F	30.1.76	CANTILEVERS & W/P HOLE
E	30.11.75	MANHOLE
D	24.11.75	BULK HEAD CHANGED
C	9.10.75	GENERAL REVISION
B	29.8.75	ISSUED FOR APPROVAL
A	1.8.75	ISSUED FOR INFORMATION

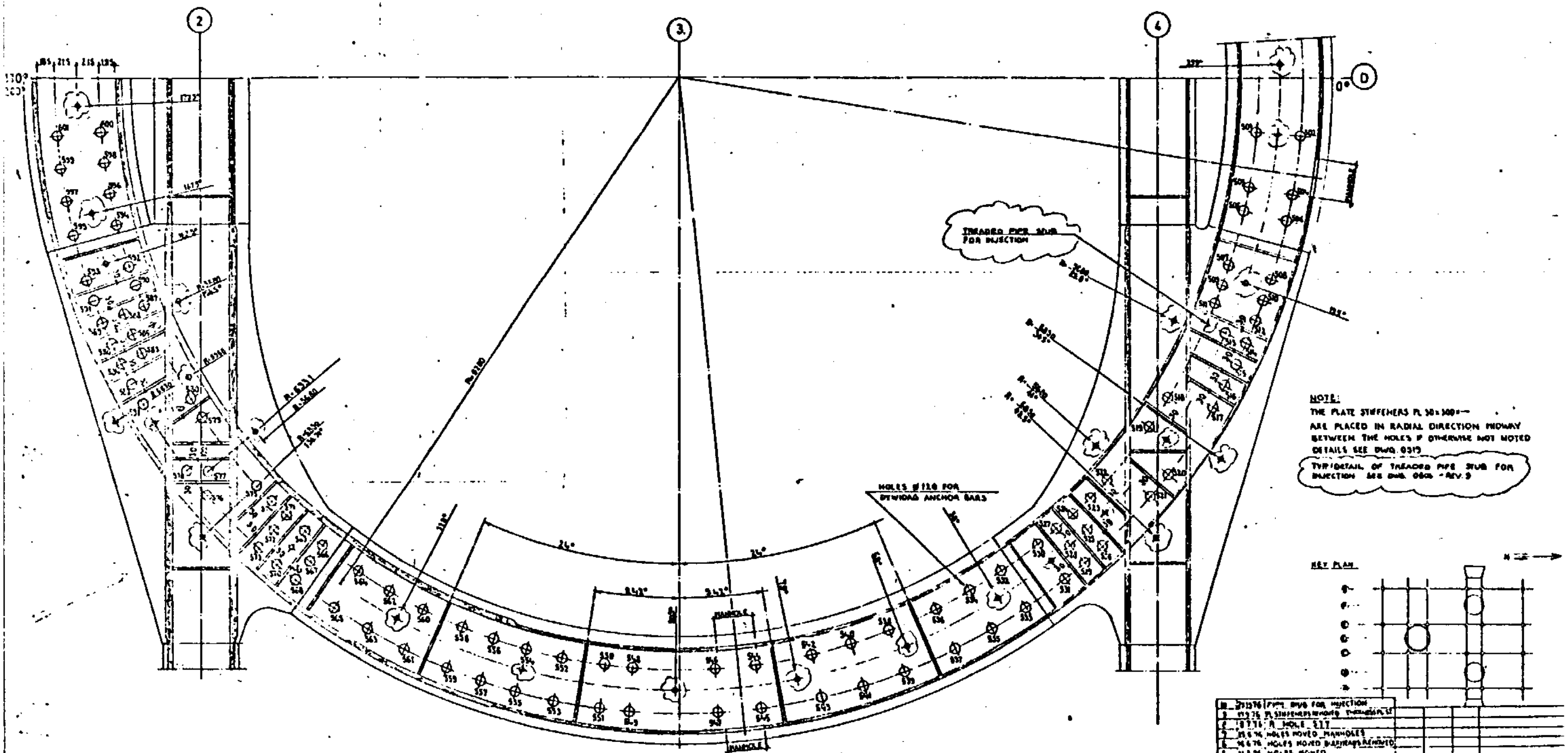
CONDEEP
HØYER ELLEFSEN
OSLO NORWAY

ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

SUPPORT RING 03
PLAN SECTION

1:50
1:20

UC 3337 0449 20 0702



HOLE NO	LOCATION		
501-502	5,20°	5,78°	
503-504	10,60°	11,78°	
505-506	13,20°	14,67°	
507-508	18,40°	20,44°	
509-510	20,50°	22,75°	
511-512	22,60°	25,11°	
513-514	24,70°	27,44°	
515	26,60°	29,76°	
516	28,90°	32,11°	
517	31,00°	34,44°	
518	32,77°	36,41°	
519	36,11°	40,12°	

HOLE NO	LOCATION		
520	38,26°	42,51°	
521	41,02°	45,58°	
522	42,80°	47,56°	
523	45,10°	50,11°	
524-525-526	47,40°	52,67°	
527-528-529	49,70°	55,22°	
530-531	52,62°	58,78°	
532-533	56,30°	62,56°	
534-535	60,10°	66,78°	
536-537	63,90°	71,00°	
538-539	69,00°	76,67°	
540-541	73,00°	81,11°	

HOLE NO	LOCATION		
542-543	77,00°	85,56°	
544-545	82,70°	91,89°	
546-547	86,50°	96,11°	
548-549	94,70°	105,22°	
550-551	97,30°	108,11°	
552-553	101,10°	113,00°	
554-555	105,10°	116,78°	
556-557	108,50°	120,56°	
558-559	111,90°	124,33°	
560-561	116,10°	129,00°	
562-563	119,90°	133,22°	
564-565	123,70°	137,44°	

HOLE NO	LOCATION		
566-567-568	127,90°	142,11°	
569-570	130,10°	144,56°	
571-572-573	132,30°	147,00°	
574	134,50°	149,44°	
575	136,70°	151,89°	
576	139,11°	154,56°	
577	140,64°	156,26°	
578	141,81°	157,57°	
579	145,14°	161,27°	
580	147,24°	163,60°	
581	149,20°	165,78°	
582	151,30°	168,11°	
583-584	153,40°	170,44°	

HOLE NO	LOCATION		
585-586	155,50°	172,78°	
587-588-589	157,60°	175,11°	
590-591	159,70°	177,44°	
592-593	161,60°	179,56°	
594-595	165,80°	184,22°	
596-597	168,80°	187,56°	
598-599	171,83°	190,89°	
600-601	174,80°	194,22°	

10	27.10.76	PIPE STUB FOR INJECTION
9	11.9.76	PL STIFFENERS MOVED THICKNESS
8	18.7.76	R HOLE 517
7	15.6.76	HOLES MOVED MANHOLES
6	16.6.76	HOLES MOVED DIAPHRAGMS REMOVED
5	31.5.76	HOLES MOVED
4	15.5.76	SUPPORTS HOLES MOVED
3	14.5.76	[illegible]
2	[illegible]	HOLES MOVED
1	[illegible]	HOLES MOVED
0	[illegible]	ISSUED FOR CONSTRUCTION
B	[illegible]	ISSUED FOR APPROVAL
A	[illegible]	ISSUED FOR INFORMATION

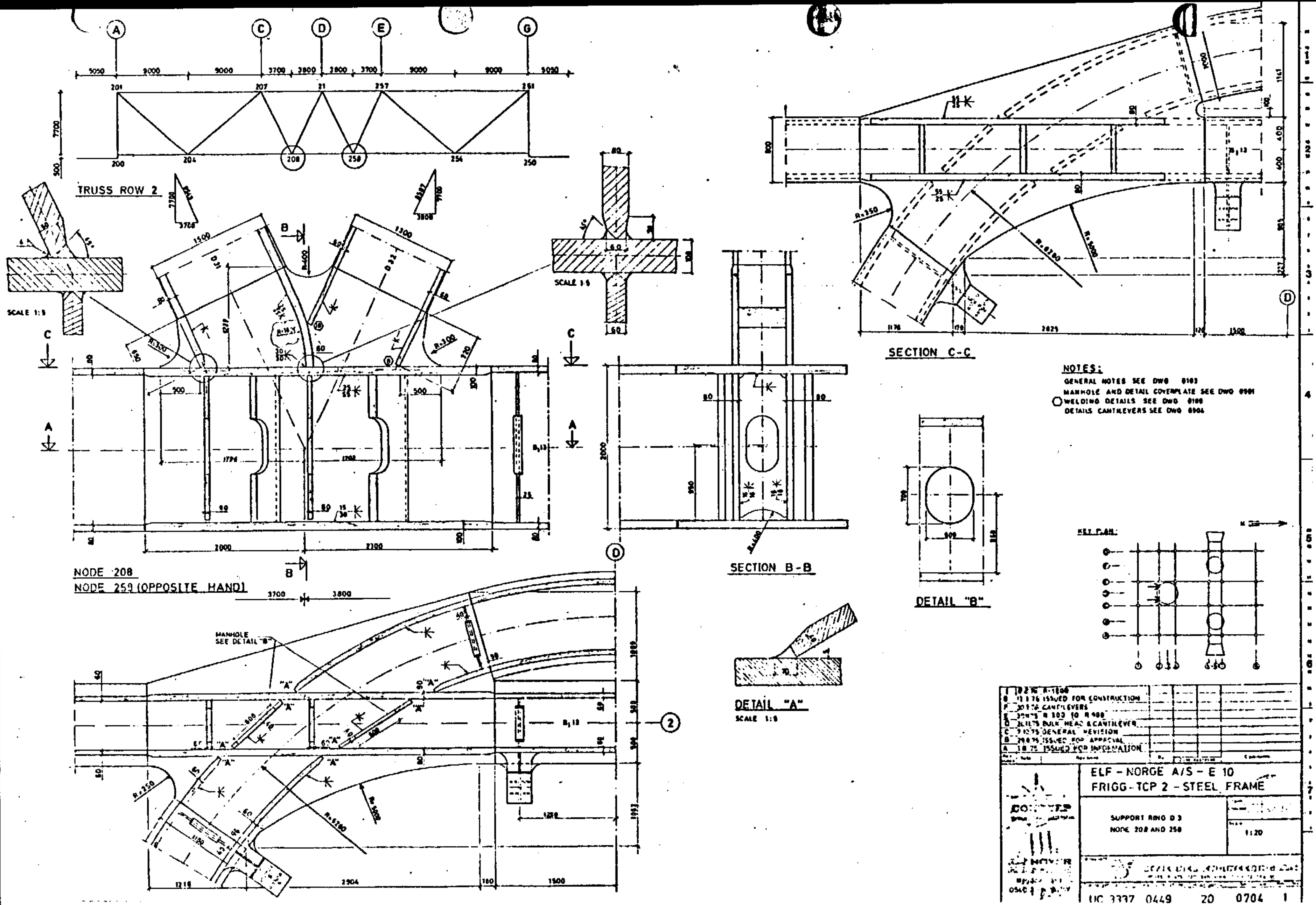
COMPREP

ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

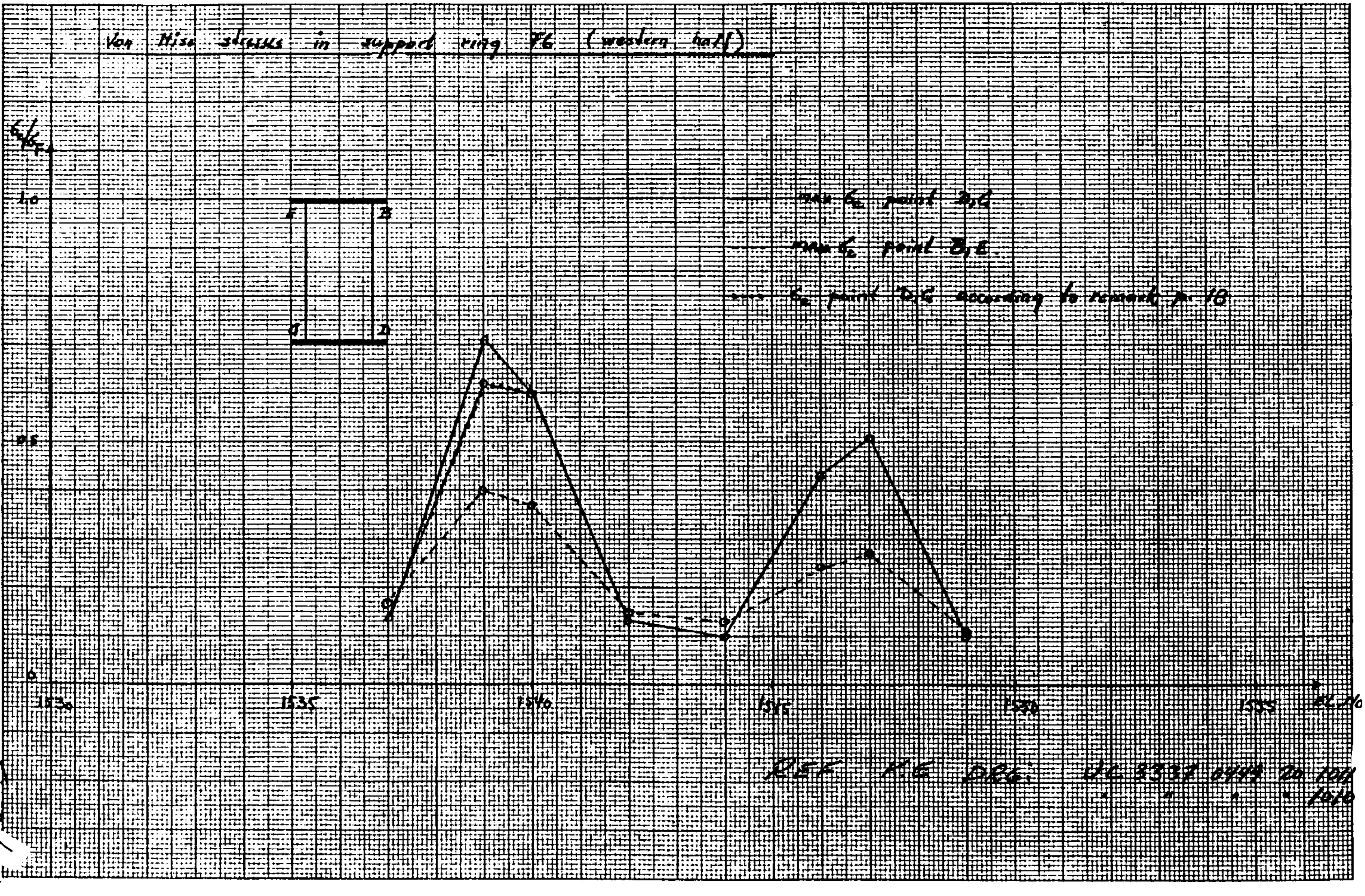
SUPPORT RING 03
ARRANGEMENT OF ANCHOR BARS
EASTERN HALF

1 : 20

UC 3337 0449 20 0708 10



TRUSS ROW 2
SCALE 1:5
NODE 208
NODE 259 (OPPOSITE HAND)
SECTION B-B
SECTION C-C
DETAIL "A"
SCALE 1:5
DETAIL "B"
MANHOLE SEE DETAIL "B"
KEY PLAN:
NOTES:
GENERAL NOTES SEE DWG 0103
MANHOLE AND DETAIL COVERPLATE SEE DWG 0901
WELDING DETAILS SEE DWG 0106
DETAILS CANTILEVERS SEE DWG 0906
ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME
SUPPORT RING D 3
NODE 208 AND 258
1:20
UC 3337 0449 20 0704 1


Von Mise stresses in support ring F6 (western half)
1.0
0.5
0
1530
1535
1540
1545
1550
1555

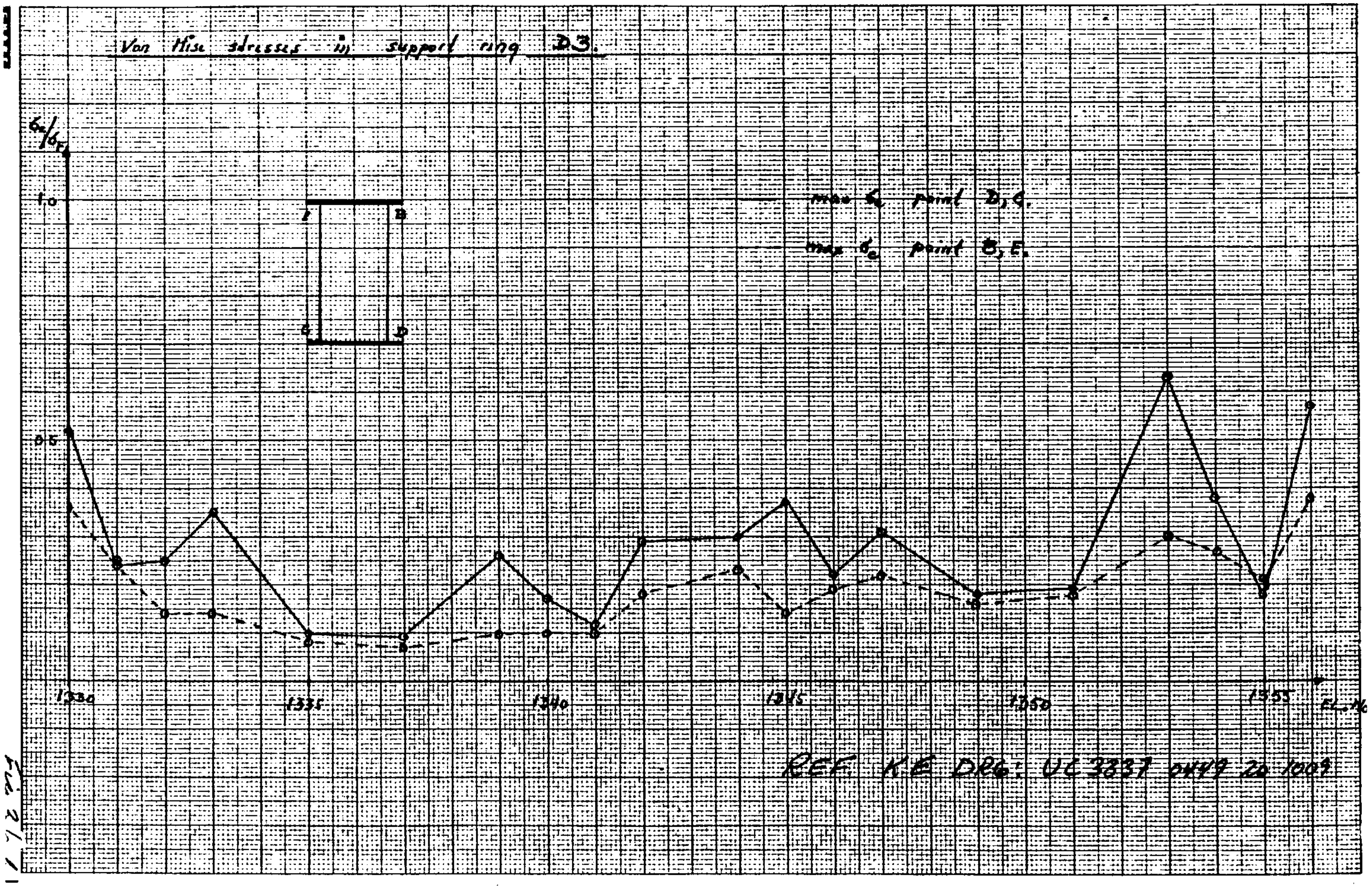
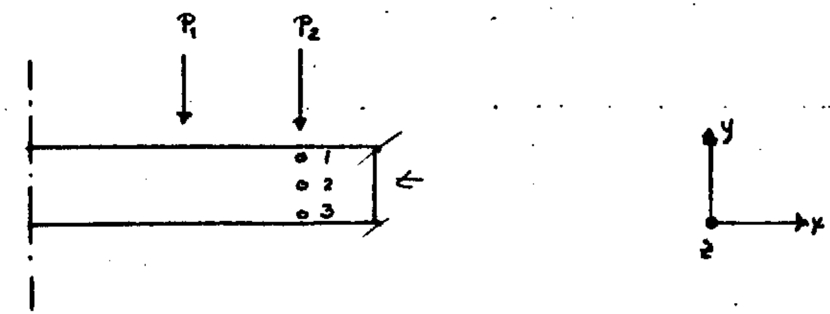

Van Mise stresses in support ring D3.
1.0
0.5
max σ point D, C.
max σ point B, E.
1330
1335
1340
1345
1350
1355
EL. No
REF. K.E. DRG: UC 3837 0449 20 1009

Fig 2.1

Ring D3

Max stresses:



Position	σ_x	σ_y	σ_z	τ_{xy}	τ_{yz}	τ_{xz}	σ_e	σ_e/σ_F
1 Storm	-2056	-1825	600	~0	~0	300	2601	0.77
1 Operation	-1910	-1697	500	~0	~0	200	2337	0.69
2 Storm	-404	-1160	600	1153	~0	300	2568	0.76
2 Oper.	-375	-1080	500	1072	~0	200	2334	0.69
3 Storm	1239	-495	600	~0	~0	300	1605	0.47
3 Opera	1160	-462	500	~0	~0	200	1455	0.43

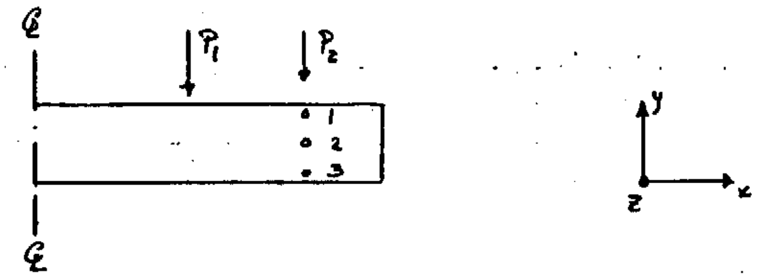
Storm: $P_2^{max} = 10.95$ Mp/cm $\Rightarrow \frac{10950}{6} = 1825$ kp/cm² (pos 1) $\qquad P_1^{max} \sim 2.9$ Mp/cm

Operation: $P_2^{max} = 10.18$ Mp/cm $\Rightarrow \frac{10180}{6} = 1697$ kp/cm² (–) $\qquad P_1^{min} \sim 0.9$ Mp/cm

Fig 36 (c)

Ring B6, F6.

Max stresses:

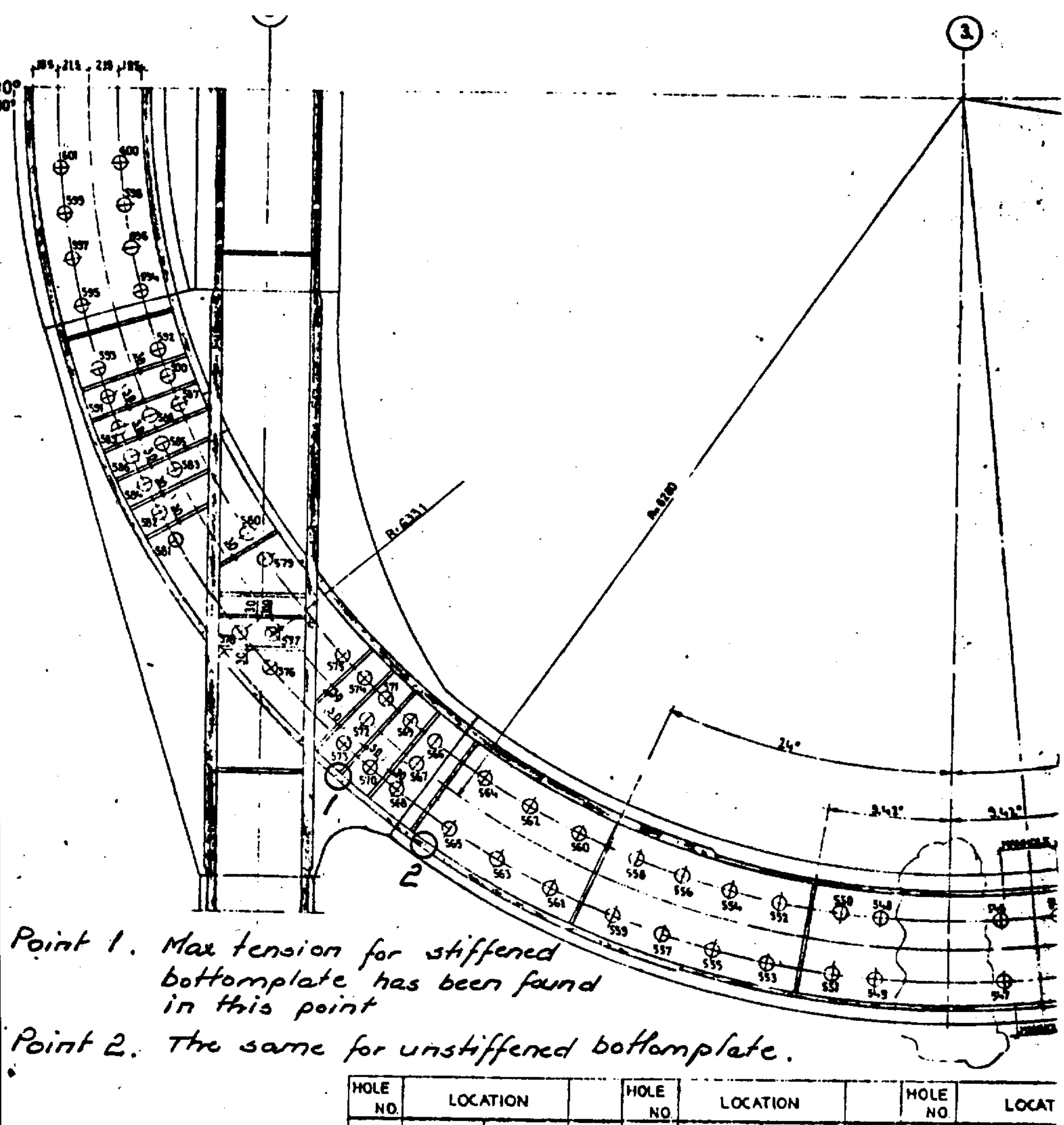


Position	σ_x	σ_y	σ_z	τ_{xy}	τ_{yz}	τ_{xz}	σ_e	σ_e/σ_F
1 Storm	-1735	-2220	500	~0	~0	300	2566	0.75
1 Oper.	-1469	-1880	400	~0	~0	200	2133	0.63
2 Storm	-402	-1301	500	798	~0	300	2148	0.63
2 Oper.	-340	-1105	400	675	~0	200	1883	0.55
3 Storm	932	-381	500	~0	~0	300	1270	0.37
3 Oper.	789	-327	400	~0	~0	200	1041	0.31

Storm: $P_2^{max} = 8.88$ Mp/cm $\Rightarrow \frac{8.88 \cdot 10^3}{4} = 2220$ kp/cm² $\qquad P_1^{max} \sim 29$ Mp/cm.

Operation: $P_2^{max} = 7.52$ Mp/cm $\Rightarrow \frac{7.52 \cdot 10^3}{4} = 1880$ kp/cm² $\qquad P_1^{min} \sim 0$

(Fig. 26)



Point 1. Max tension for stiffened bottomplate has been found in this point

Point 2. The same for unstiffened bottomplate.

HOLE NO.	LOCATION			HOLE NO.	LOCATION			HOLE NO.	LOCAT
501-502	5,20°	5,78°		520	38,26°	42,51°		542-543	77,00°
503-504	10,60°	11,78°		521	41,02°	45,58°		544-545	81,70°
505-506	13,20°	14,67°		522	42,80°	47,56°		546-547	86,50°
507-508	18,40°	20,44°		523	45,10°	50,11°		548-549	94,70°
509-510	20,50°	22,78°		524-525-526	47,40°	52,67°		550-551	97,30°
511-512	22,60°	25,11°		527-528-529	49,70°	55,22°		552-553	101,70°
513-514	24,70°	27,44°		530-531	52,00°	57,78°		554-555	105,10°
515	26,60°	29,78°		532-533	56,30°	62,56°		556-557	108,50°
516	28,90°	34,11°		534-535	60,10°	66,78°		558-559	111,90°
517	31,00°	34,44°		536-537	63,90°	71,00°		560-561	116,10°
518	32,77°	36,41°		538-539	69,00°	76,67°		562-563	119,90°
519	36,11°	40,12°		540-541	73,00°	81,11°		564-565	123,70°

Fig 36 e)

APPENDIX NO. 4.

FIGURES - FABRICATION

D.E.P.

1052 N° 5- 720
RM/md

COURBEVOIE, October 24th, 1975

2 3 NOV. 1975

MINUTES OF MEETING NORCO/DNV/ELF

HELD ON OCTOBER 22nd 1975 IN ELF PARIS OFFICES

SUBJECT - FRIGG Field - TCP2 support-frame welding and inspection of welds

In attendance :

Messrs IRGENS KVAM	}	NORCO
EIDE MISUND	}	DNV
SAMARAN (partial time) PUIDEBAT PLOUVIER ROZEN TRICAN METZ	}	ELF

I - MEETING PURPOSE

Restatement of what shall be planned concerning welding procedure qualification tests, welders qualification and weld inspection.

II - BASE STEEL SPECIFICATION

ELF precises that the plates have been ordered according ELF specification 1052 N° 5-302 - Rev. 0 issued on May 1975.

Distribution of plates

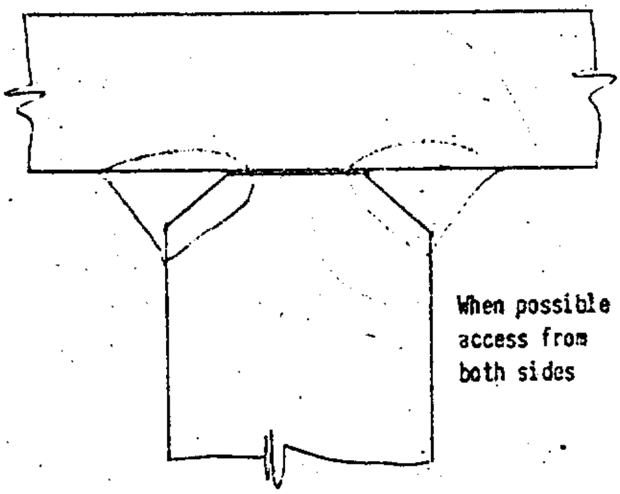
SHS - 20 for thicknesses ⩽ 25 mm

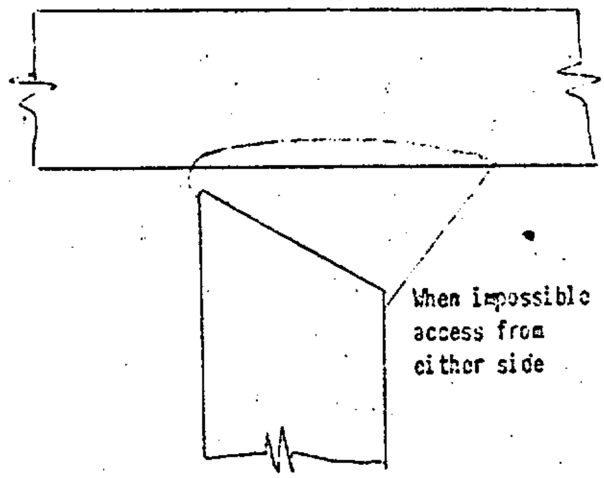
SHS - 40 " " > 25 mm

III - TYPES OF WELDING PROCESSES AND OPERATING CONDITIONS

According to Mr. KVAM, arc handwelding with coated electrodes will be used for every weld except for partial penetrated web on flange welds of current boxes, which will be made by automatic welding.

Principle design of web on flange welds :





In principle, webs and flanges of boxes will not have butt joints. The only ones to be considered will be the further mounting butt welds of current boxes with prefabricated nodes.

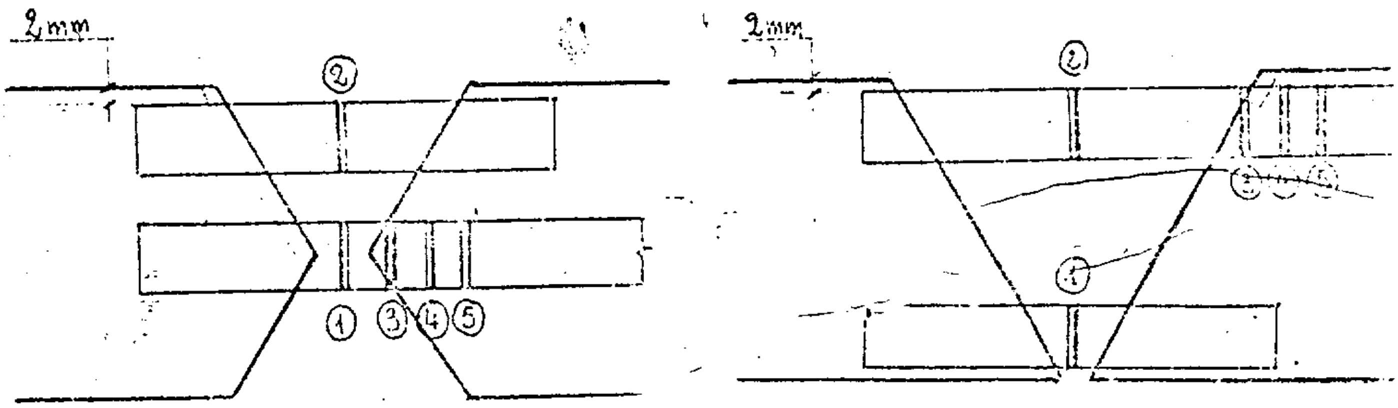
IV - NOTCH IMPACT TESTING - METHOD OF CUTTING THE SPECIMENS

For thicknesses $\geqslant$ 50 mm, DNV asks to cut one set of 3 impact specimens in weld metal at the root, in addition to the 4 sets whi are usually required.

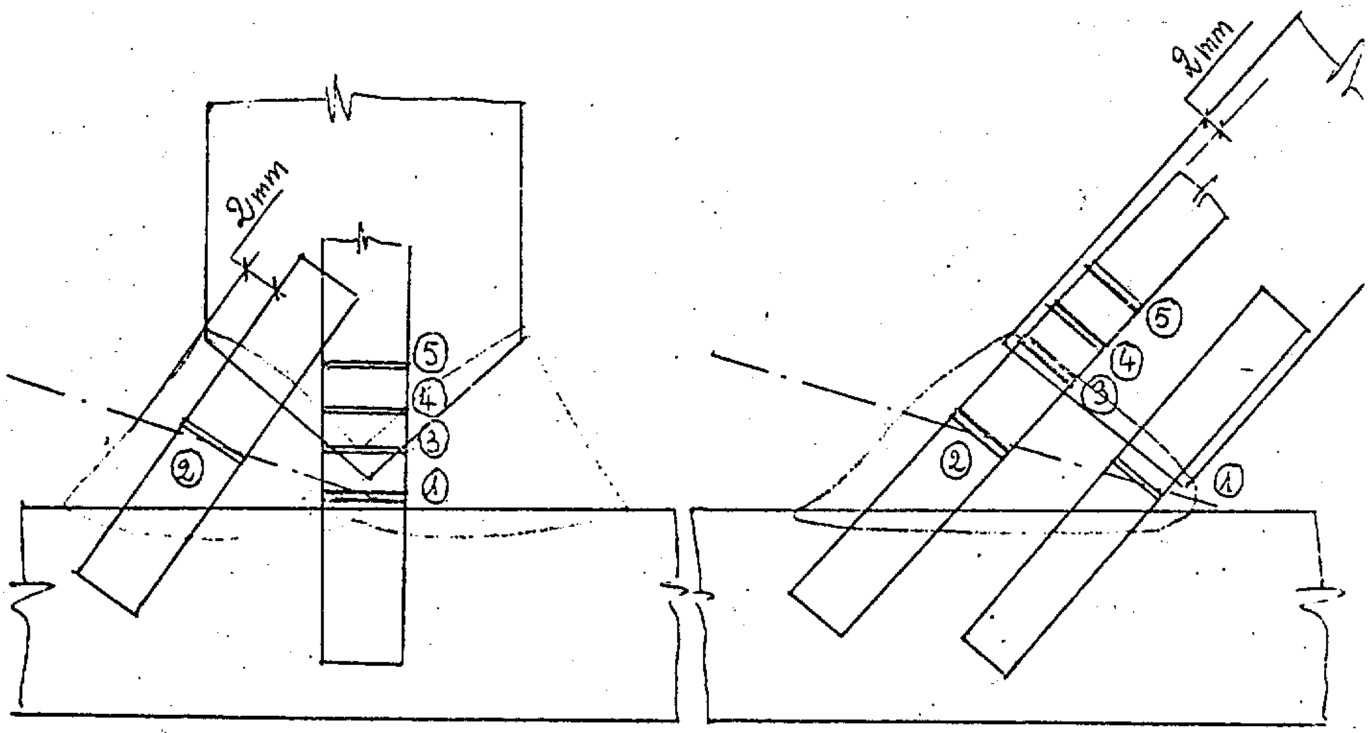
The following provisions are adopted.

a) Method of cutting specimens for t $\geqslant$ 50 mm

Butt welds



Groove fillet welds



① - Center of the weld (at the root)

② - Center of the weld (below surface with capping runs)

③ - Fusion line

④ - 2 mm from fusion line

⑤ - 5 mm from fusion line

b) <u>Method of cutting specimens for t < 50 mm</u>

Set ① will be suppressed and all other sets (②, ③, ④ and ⑤) will be cut 2 mm below the surface with capping runs (see above figures).

c) <u>Impact levels required</u> : KV - 40° C

- in weld metal at root (set ①): 3,6 kgm average (according DNV rules)

- at all other locations (sets ②, ③, ④, ⑤): 4,1 kgm average (according ELF fabrication - Specification)

../..

V - TYPES AND NUMBER OF SPECIMENS FOR MECHANICAL TESTS OTHER THAN TOUGHNESS TESTS

(butt welds and full penetration fillet welds)

The types and number of specimens for mechanical tests other than specimens planned for impact toughness measurement, will be those specified in DNV Technical Note N° C 1/2 of June 1st 1975.

Standards of acceptance of these tests will be the most severe ones of ELF Specification and DNV Rules.

Notice - Mechanical test specimens shall be cut only after favorable non-destructive inspection of the test weld.

VI - THICKNESS RANGES FOR WELDING PROCEDURE QUALIFICATION TESTS

Grades and thicknesses met in the construction of the support-frame :

- SHS - 20 : 12 - 15 - 20 - 25 mm
- SHS - 40 : 30 - 40 - 50 - 60 - 80-100 mm

The following provisions are decided in agreement with DNV :

Thickness ranges for qualification (mm)	Thickness for qualification test (mm)
80 ⩽ t ⩽ 100 (SHS - 40)	100 (SHS - 40)
50 ⩽ t < 80 (SHS - 40)	60 (SHS - 40)
25 < t < 50 (SHS - 40)	40 (SHS - 40)
12 ⩽ t ⩽ 25 (SHS - 20)	20 (SHS - 20)

VII - COD MEASUREMENT

According to the Technical Note N° C 1/2, DNV asks to perform a COD measurement for thicknesses over than 50 mm.

a) Testing temperature : - 10° C

However, DNV asks ELF to investigate upon the origin and validity of the minimum temperature of - 15° C reported in a technical document relating to this matter.

../..

b) Thickness, type of bevel and welding position

AKKER welding procedure has not yet been communicated ; therefo it is not possible to state the welding conditions for the qualification test.

DNV says that a welding test made in a 100 mm sample with symetrical K bevel and upwards welding would be valid for all positions and thicknesses met in the structure construction (ha welding only).

DNV asks to make sure that no automatic submerged are welding will be made on thicknesses over than 50 mm.

c) Number of COD specimens

3 specimens in weld metal and 3 specimens in HAZ, on test weld in as-weld condition and PWHT condition.

d) Method of calculation of acceptable defects

ELF asks DNV to give them the following informations :

- the formula and stresses to be adopted for the calculation of the defect parameter $\bar{a}_{max}$ (applied stresses, local stress concentration),
- minimum value $\bar{a}_{max}$ acceptable by DNV (as a rule),
- the calculation formulas allowing to go from $\bar{a}_{max}$ to a_{max} or $2a_{max}$ for surface or burried defects.

e) COD tests performance

DNV is ready to carry out COD tests relating to this matter. It would take about three weeks.

Besides, DNV is not opposed to the fact that these tests be carried out by another recognized organism, on condition that a DNV's representative should attend the tests execution.

VIII - WELDERS QUALIFICATION

- Qualification on plates

ELF agrees to apply provisions of DNV Technical Note C 1/3 of 28.05.75 (table 2) except that to be qualified for "all positions the welders must comply with positions 2G + 3G + 4G (instead of 1G + 3G + 4G).

- Qualified welders on pipes

These welders can work on plates according provisions of DNV Technical Note C 1/3.

IX - NDT OF WELDS

The NDT on complete welds planned by ELF specification are as follows :

. All full penetration welds	:	100 %
. All other welds	:	10 %

DNV proposes following provisions :

Classification of members	PRODUCTION *	REPAIR *
Special	100 %	100 %
Primary	mini 20 %	100 %
Secondary	mini 5 %	20 %
Non structural	SPOT	5 %

* Referring to total length of welds in each classification

NOTE - Initial inspection should be 100 % until a satisfactory level of quality has been verified

This DNV proposal involves the previous statement of the distribution of support-frame members in orders of importance : ELF will give an answer after investigation.

X - POST-WELD HEAT TREATMENT

In particular case of this job, DNV think it advisable to search for a welding metal and operating conditions sufficient to avoid PWHT.

XI - ELF SPECIFICATIONS REVISION

ELF specifications will be revised especially as regards the welding of heavy thicknesses. But for the sake of the contract, the present revision will remain valid. The modifications requested by DNV for this matter will be, however, applied in full in fabrication and inspection.

Red part

(including Central Square of White Part)

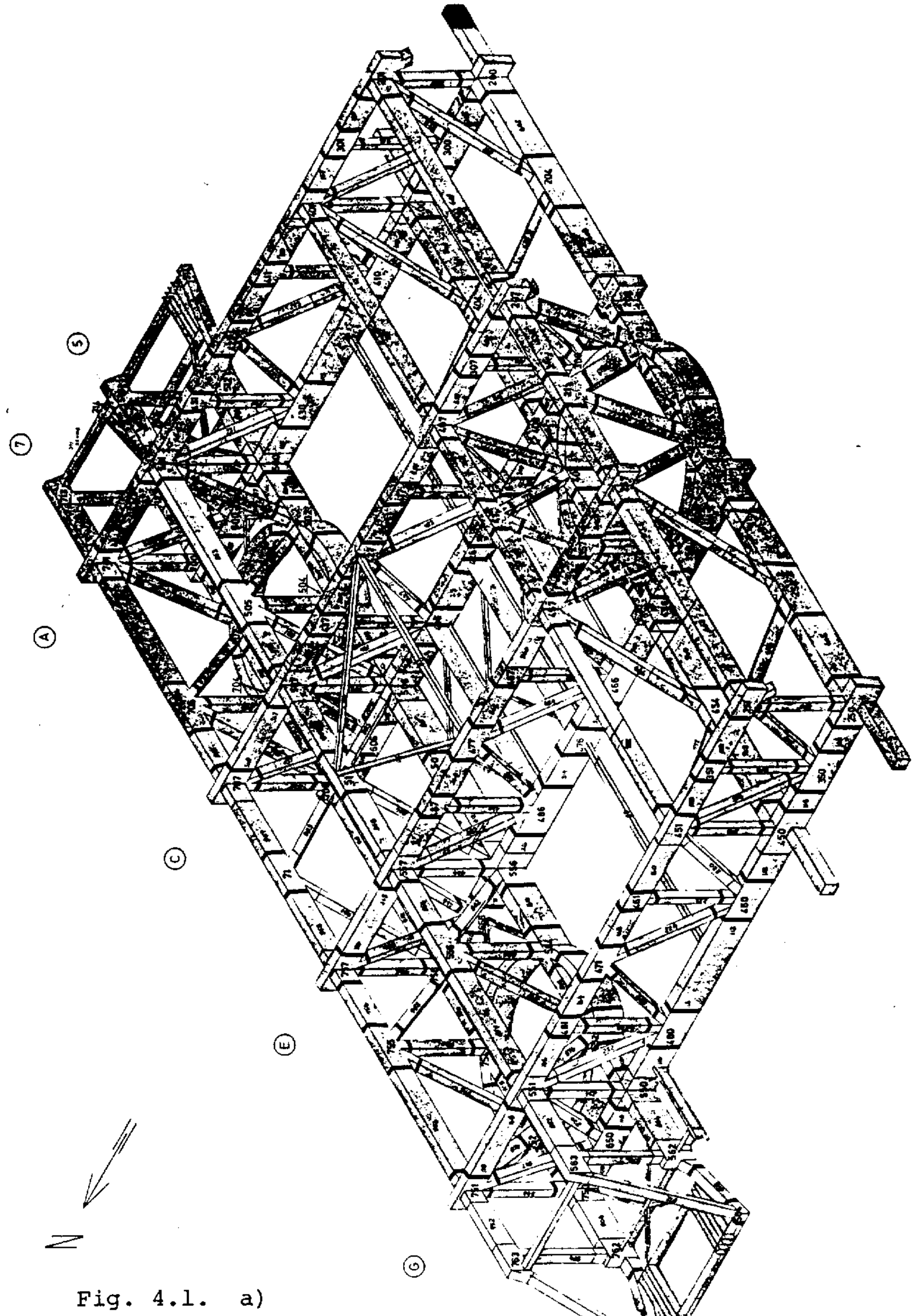


Fig. 4.1. a)

Green part

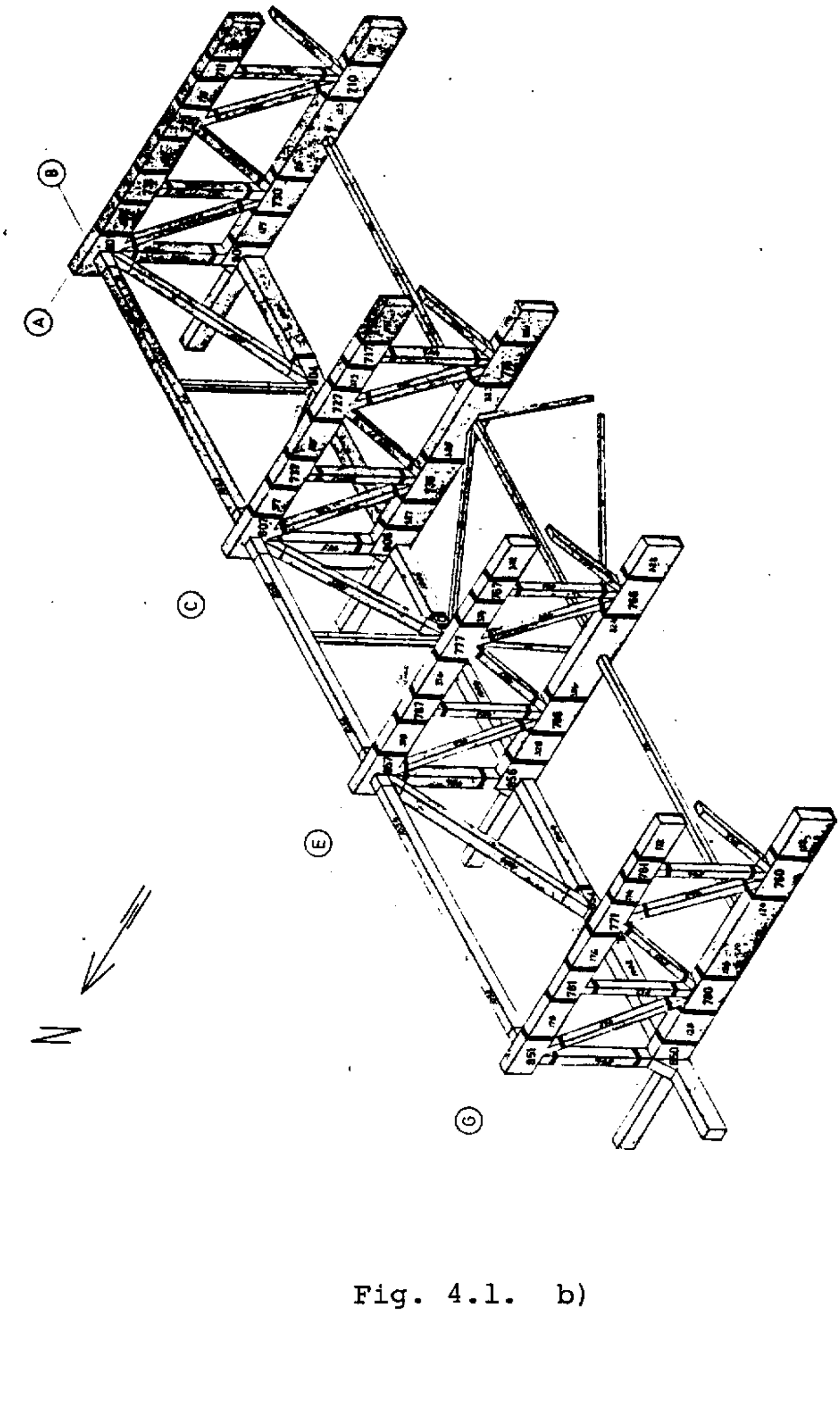


Fig. 4.1. b)

White part - Remanding Part

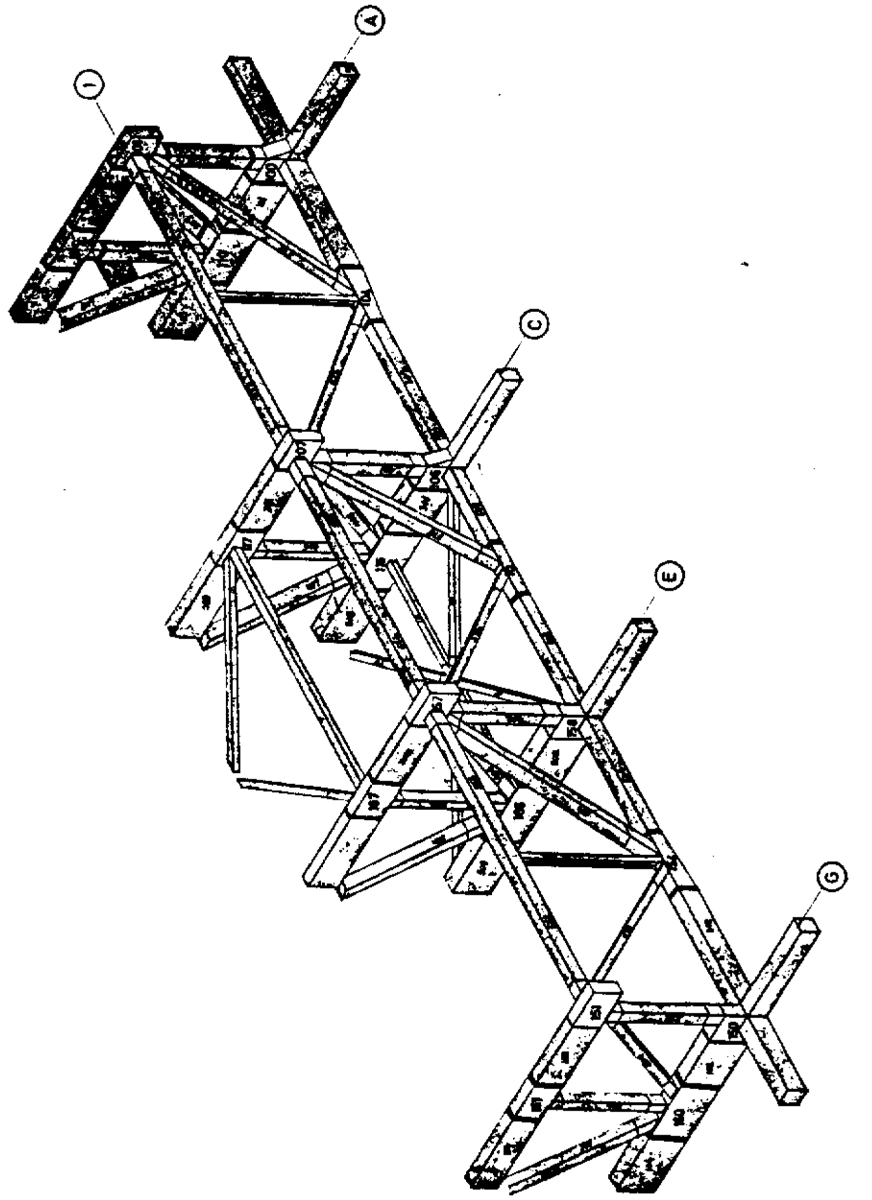


Fig. 4.1. c)

Form No. 40.91a

Project : ELF NORGE - CONDEEP PLATFORM T.C.P.2. FRIGG FIELD.

V.N.I. Project n° : 601623.

Contractors : C.M.P. Dunkerque - C.M.P. Hardyck.
C.M.P. Arles.
Ets. Julin - Rouen.
Ateliers et Chantiers de Bretagne - Nantes.
Socomet - Le Trait.
Ranville.

Ultrasonic Control.

Ultrasonic control to be carried out according to this modification to C.MP. "Programme for Non-Destructive Testing" of 2.2.76.

ITEM	Full Penetration Welds		Partial Penetration Welds	
	with P.W.H.T.	without PWHT	with P.W.H.T	without PWHT
Special Structural Parts	100 % after P.W.H.T.	100 %	100 % after P.W.H.T.	50 % *
Primary Structural Parts	100 %	100 %	100 %	25 % *
Secondary Structural Parts		10 % *		5 % *
Non-Structural Parts		Spot *		Spot *

* Refers to length of each weld in each classification after a satisfactory quality level has been achieved.

Notes : 1.- All repair areas to be examined 100 %.

2.- Any weld examined by percentage method and found to be defective shall be examined 100 %.

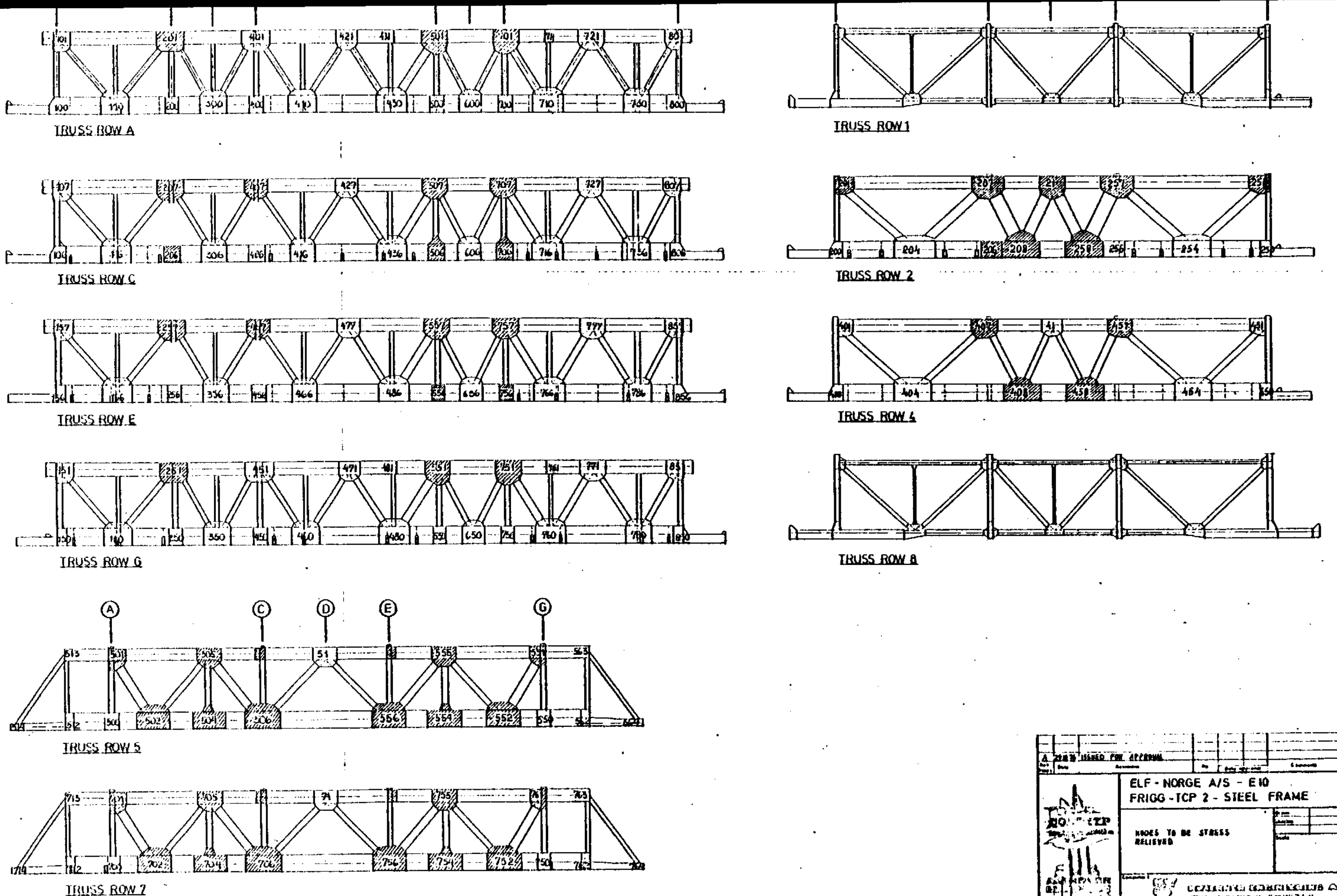
Dunkerque,
30.7.76

C. Cuvelier

Table 1: Fracture mechanics tests - TCP-2

Welding consumable Brand	Joint details Groove Pos.	Specimen Size Steel	Testing Cond. Temp. °C	Testing Cond. As welded /PWHT	Position of notch	COD δc (Wells) (Pop-in^x)	Fabricator Date	Report ref.
OK 73.68	K-groove 36	100 x 100 SHSS 40	-10°C	As welded	Weld metal	0.043^x - 0.104 0.055^x - 0.048^x 0.23^x - 1.39	Stord Verft January 1976	DnV 571674/1
					HAZ	1.47 -> 1.54 - 1.60		
			-10°C	PWHT	Weld metal	0.14 - 0.31 - 0.57		
					HAZ	0.72 - 1.98 - > 2.2		
OK 48.15	K-groove 36	100 x 100 SHSS 40	-10°C	PWHT	Weld metal	0.064 - 0.241 -0.383	CMP-Arles	
					HAZ	0.377 - 0.365 -0.707		
Fortrex 8018-C1	K-groove 3G	50 x 50 SHSS 40	-10°C	As welded	Weld metal	1.03-0.046-0.35	CMP-Dunkirk May 1976	Institute de la Saldene no. 5896
				PWHT	Weld metal	0.32-1.50-0.56		
Phillips	K-groove 3G	100 x 100 SHSS 40	-10°C	PWHT	Weld metal	0.435 - 0.466 -0.367	CMP-Dunkirk	I.S. 6111
					HAZ	2.05-0.64-1.72		
Commercy SP 1500/	K-groove 1G	100 x 100 SHSS 40	-10°C	PWHT	Weld metal	2.14 - 2.25 -2.29	CMP Dunkirk	1
Flux Linoln 880					HAZ	0.76 - 0.94	Okt. 1976	

Welding consumable Brand	Joint details Groove Pos.	Specimen Size Steel	Testing Temp. °C	Cond. As welded /PWHT	Position of notch	COD δc (Wells) (Pop-in^x)	Fabricator Date	Report ref.
Fortrex 8018Cl	K-groove 3G	60 x 60 SHSS 40	-10°C	As welded	Weld metal	0.54 0.20 1.12 0.21	Stord Verft Febr. 1976	DnV 571674/2
Fortrex 8018Cl	X-groove 3G	60 x 60	-10°C	As welded	Weld metal	0.88 0.29 0.16 0.087	Stord Verft Febr. 1976	DnV 571674/2
Fortrex 8018Cl	K-groove 3G	50 x 50 SHSS 40	-10°C	As welded	Weld metal	0.04^x - 0.15	Stord Verft	DnV 571674/3
				PWHT	Weld metal	0.84-0.87-1.20	Febr. 1976	
Fortrex 8018Cl	K-groove 3G	60 x 60 SHSS 40	-10°C	As welded	Weld metal	0.17-0.196-0.51	Stord Verft	DnV 572040/1
				PWHT	Weld metal	0.33-1.44-1.45	April 1976	
Fortrex 8018Cl	K-groove 3G	80 x 80 SHSS 40	-10°C	As welded	Weld metal	0.098 - 0.079^x -0.14	Stord Verft	DnV 572040/1
				PWHT	Weld metal	0.05->2.05-1.28	April 1976	
Fortrex Murex	K-groove 3G	100 x 200 SHSS 40	-10°C	As welded	Weld metal	0.376-0.056 0.164	Institute de la Soudure Jan. 1976	I.s 5908
Oerlikon Tenacito 70B	K-groove 3G	100 x 200 SHSS 40	-10°C	As welded	Weld metal	0.035 - 0.10 -0.074	Institute de la Soudure Jan. 1976	I.S. 5908
Esab ØK FU	K-groove 3G	80 x 160 SHSS 40	10°C	As welded	Weld metal	0.118 - 0.064 -<<6.06^x	Institute de la Soudure	I.S. 5908
SD3 wire/TT41	X-groove 1G	80 x 80 SHSS 40	-10°C	As welded	Weld metal	>1.8 - 0.88-0.35	Stord Verft	DnV report 76-313
				PWHT	Weld metal	>1.8->1.8->1.8	July 1976	



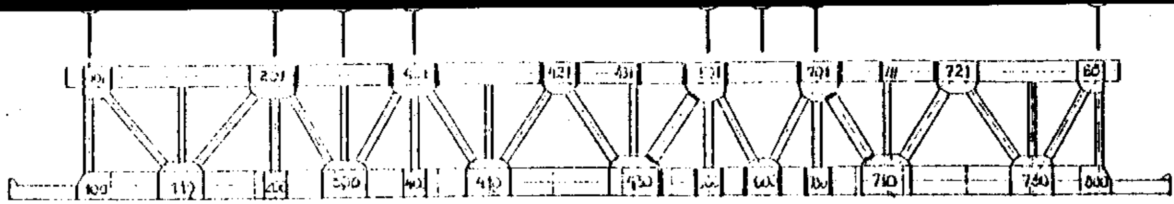
A ISSUED FOR APPROVAL

ELF - NORGE A/S - E 10

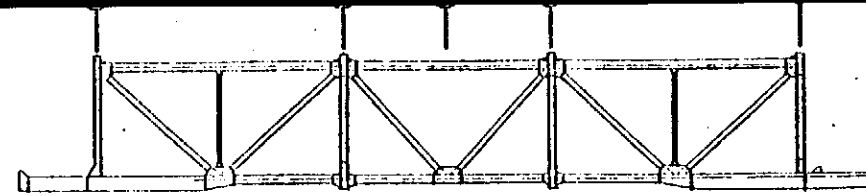
FRIGG - TCP 2 - STEEL FRAME

NODES TO BE STRESS RELIEVED

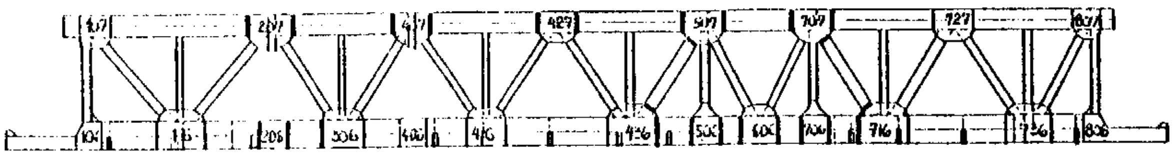
UC 3337 0449 20 1071 A



TRUSS ROW A



TRUSS ROW 1



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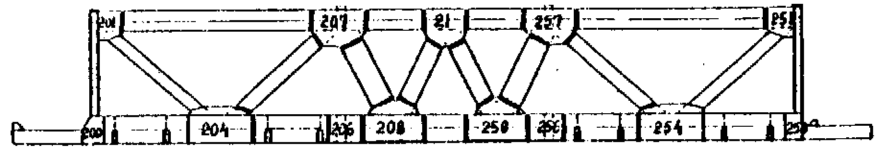
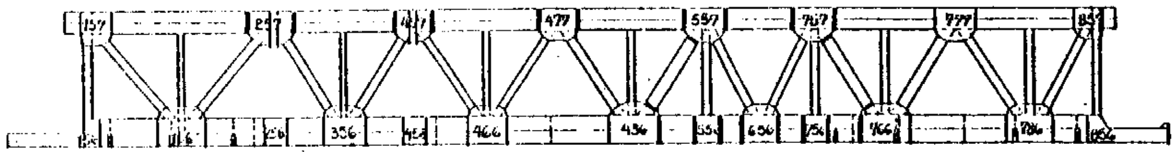
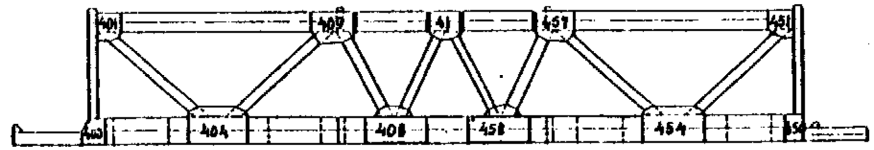


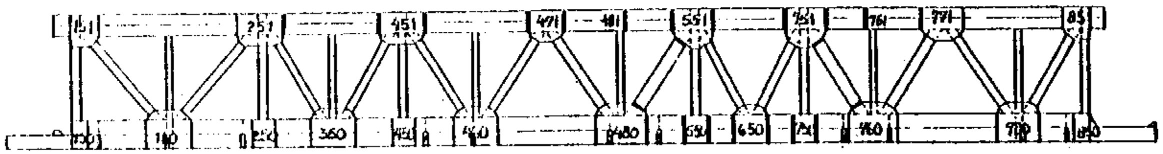
TRUSS ROW 2



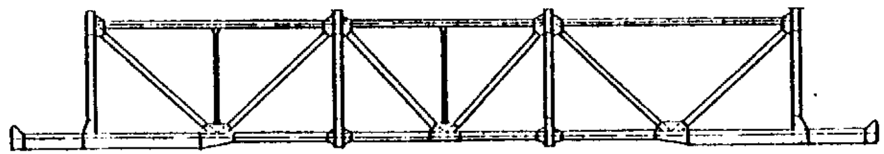
TRUSS ROW E



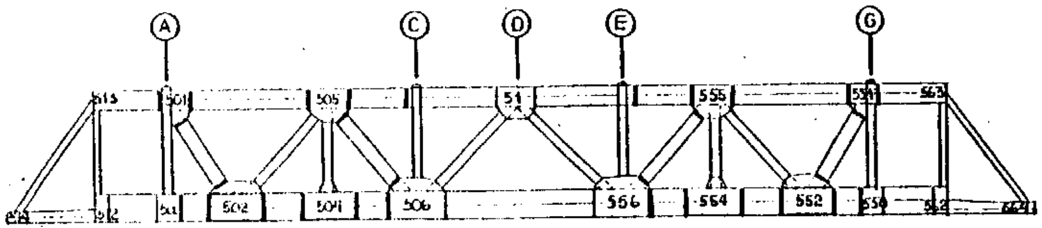
TRUSS ROW 4



TRUSS ROW G



TRUSS ROW 8



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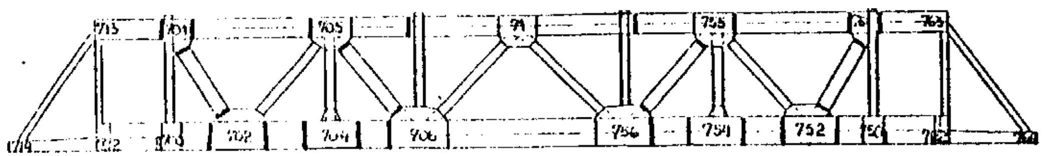
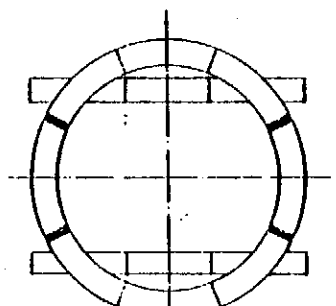


TRUSS ROW 7



THE FOUR (4) JOINTS SHOWN HERE ABOVE HAVE TO BE STRESS-RELIEVED ON THE THREE (3) MASS

A	ISSUED FOR APPROVAL

ELF-NORGE A/S - E10

FRIGG-TCP 2 - STEEL FRAME

JOINTS TO BE STRESS RELIEVED

UC 3337 0449 20 1072

subject : tcp2 nodes 208/ 258 at petite synthe.

after final stress relieving at 580 deg. c + or - 15 deg. c
2ø30 control reveals cracks in five welds.

1- weld ref. 1a/7 node 258
crack 800mm long 17-35mm deep from inside in weld metal top flange

2- weld ref 1b/ 7 node 258
crack 600mm long in top flange in flate material

3- weld ref 11/6 node 208
cracks 800mm long 25-37 mm deep from inside in weld metal bottom flange.

4- weld ref liaison 27/258 bottom flange (butt weld) length 800mm
depth 37mm
- 47 mm from inside in haz

5- weld ref 2a/6 node 258
(partial penetration weld)
cracking in 2 zones.

first zone : 500 mm long depth 10mm outside to 32mm (root of outside weld)

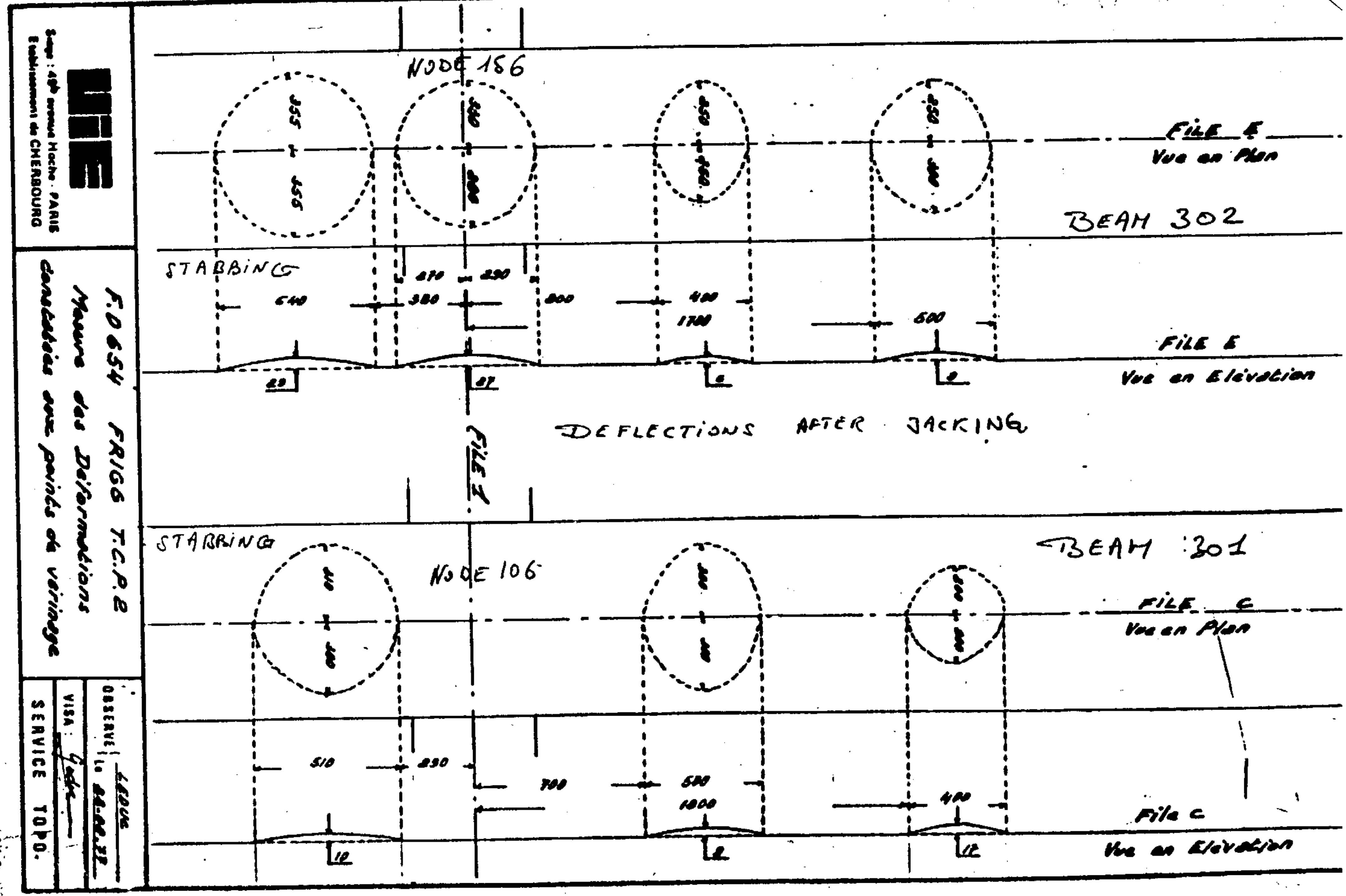
second zone : 200mm long depth 10mm - 10 mm outside to 32mm(root of outside weld).

250mm between these defects.

cmp propose to carry out the repairs without stress relieving
the maximum depth of repair withll be less tahn 50mm
cmp also propose to peen each layer after the first three runs have been laid.

please advise if further stress relieving is required and if so are further mechanical tests required.

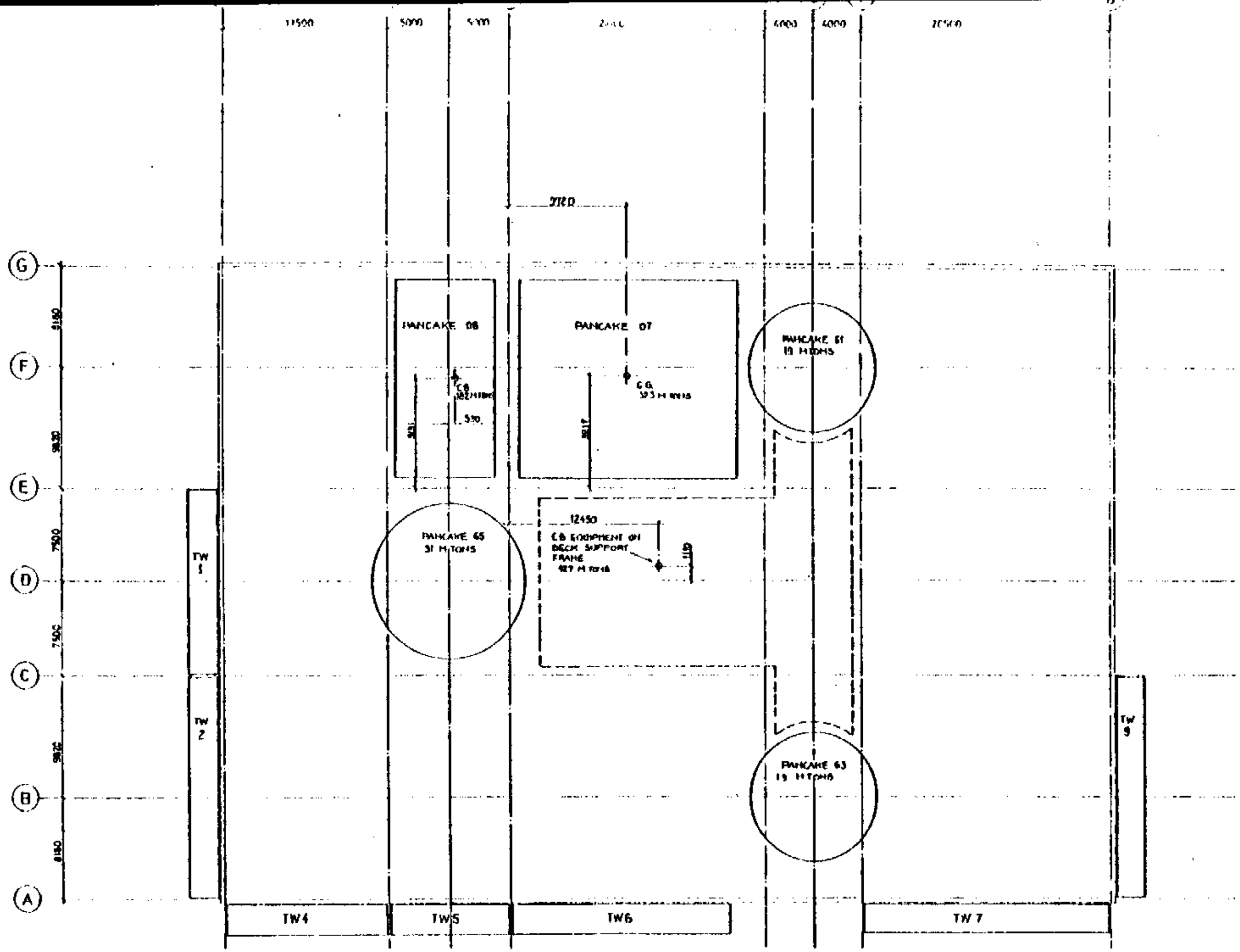
please phone to mr. dursky at cmp this afternoon.

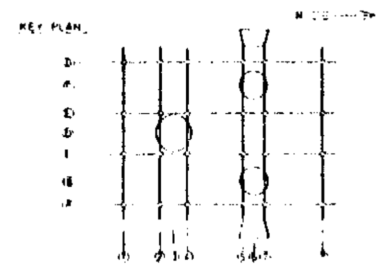


Siège : 48 avenue Hoche - PARIS
Etablissement de CHERBOURG
F.D 654 FRIGG T.C.P.2
Mesures des Déformations constatées aux points de vérinage
OBSERVE
VISA :
SERVICE TOPO.
NODE 186
FILE E
Vue en Plan
BEAM 302
STABBING
640
380
270
230
800
400
1700
500
FILE E
Vue en Elevation
DEFLECTIONS AFTER JACKING
FILE 1
STABBING
NODE 106
BEAM 301
FILE C
Vue en Plan
510
230
700
500
1000
400
File C
Vue en Elevation

APPENDIX NO. 5.

INSTALLATION





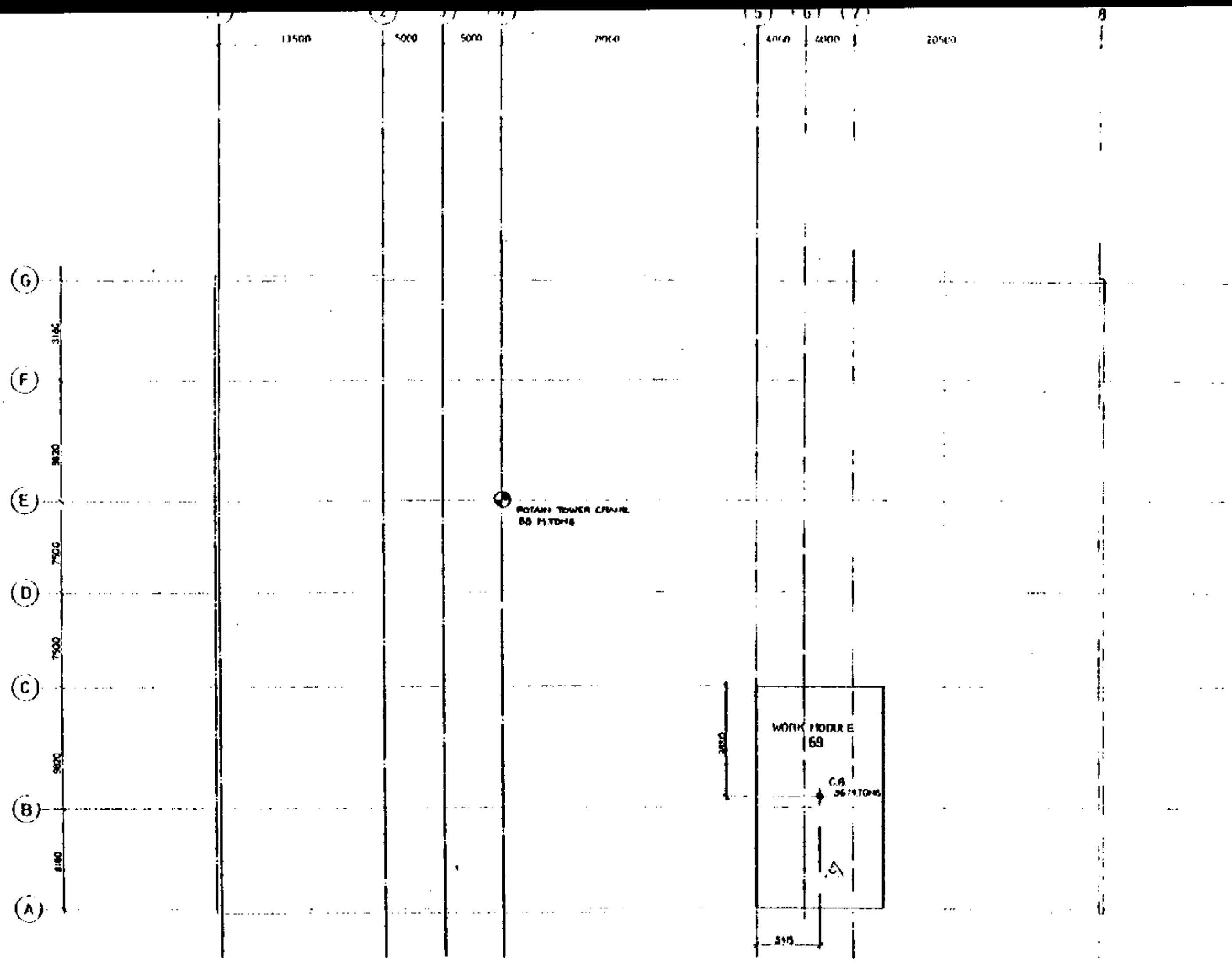
G 18.3.77 ISSUED FOR APPROVAL
B GENERAL REVISION
A 6.8.76 ISSUED FOR INFORMATION

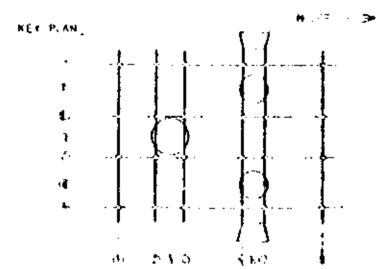
ELF – NORGE A/S – E 10
FRIGG – TCP 2 – STEEL FRAME

C.G. CELLAR DECK PACKAGES
DURING TRANSPORTATION
FROM CHERBOURG TO
ANTONIS

1:200

LC 3337 0449 20 1061 G





D	31.2.77	ISSUED FOR APPROVAL
C	6.12.76	MOBIL CRANE REMOVED
B		GENERAL REVISION
A		ISSUED FOR INFORMATION

CONTEP

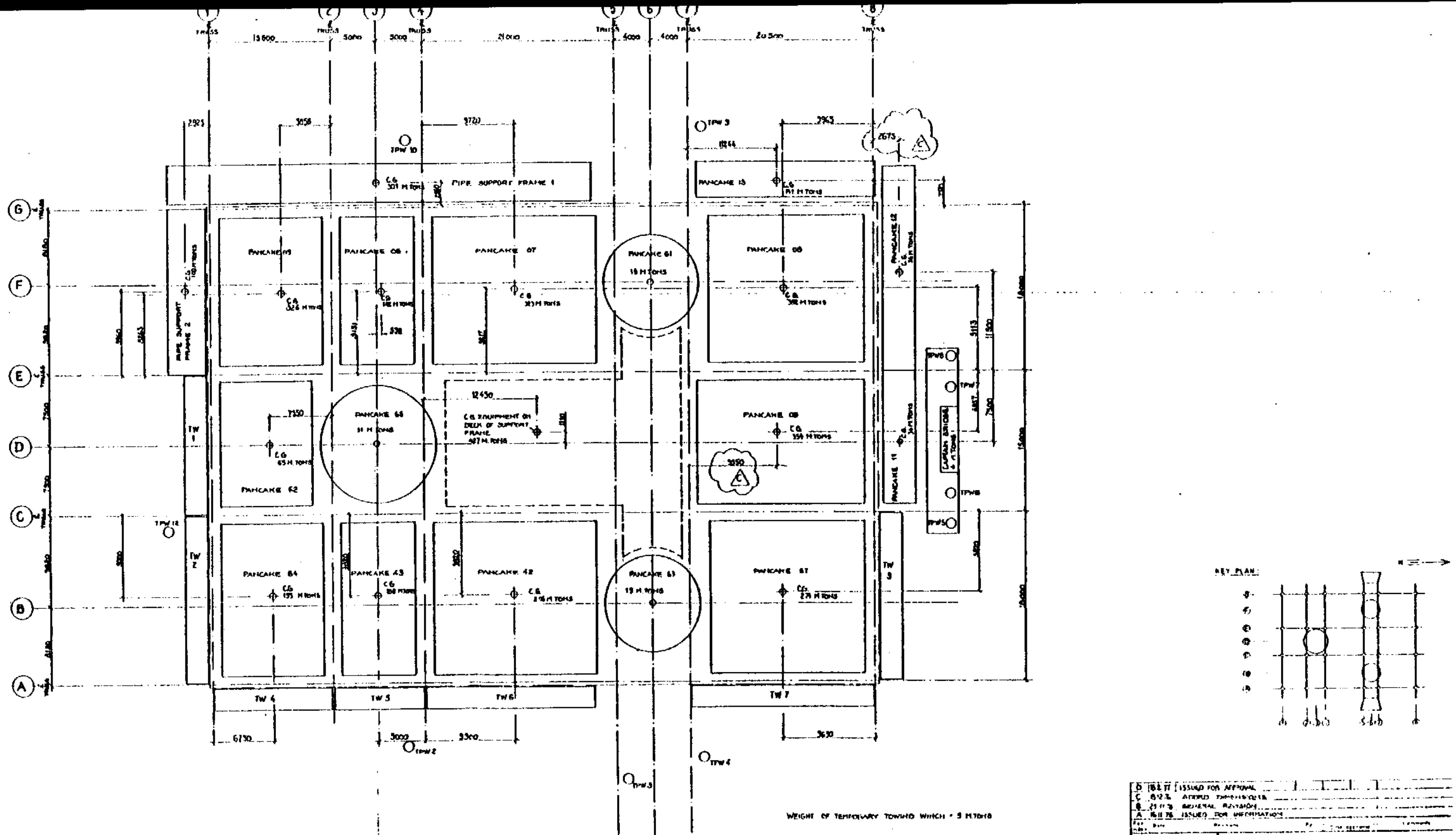
ELF - NORGE A/S - E 10

FRIGG ICP 2 STEEL FRAME

CG MAIN DECK PACKAGES DURING TRANSPORTATION FROM CHERBOURG TO ANDALSNAES

1:200

HC 3337 0449 20 1062 D



D	18.2.77	ISSUED FOR APPROVAL
C	6.12.76	ADDED DIMENSIONS
B	2.11.76	GENERAL REVISION
A	16.11.76	ISSUED FOR INFORMATION

CONDEEP

A.F. HØYER-ELLEFSEN A/S

OSLO NORWAY

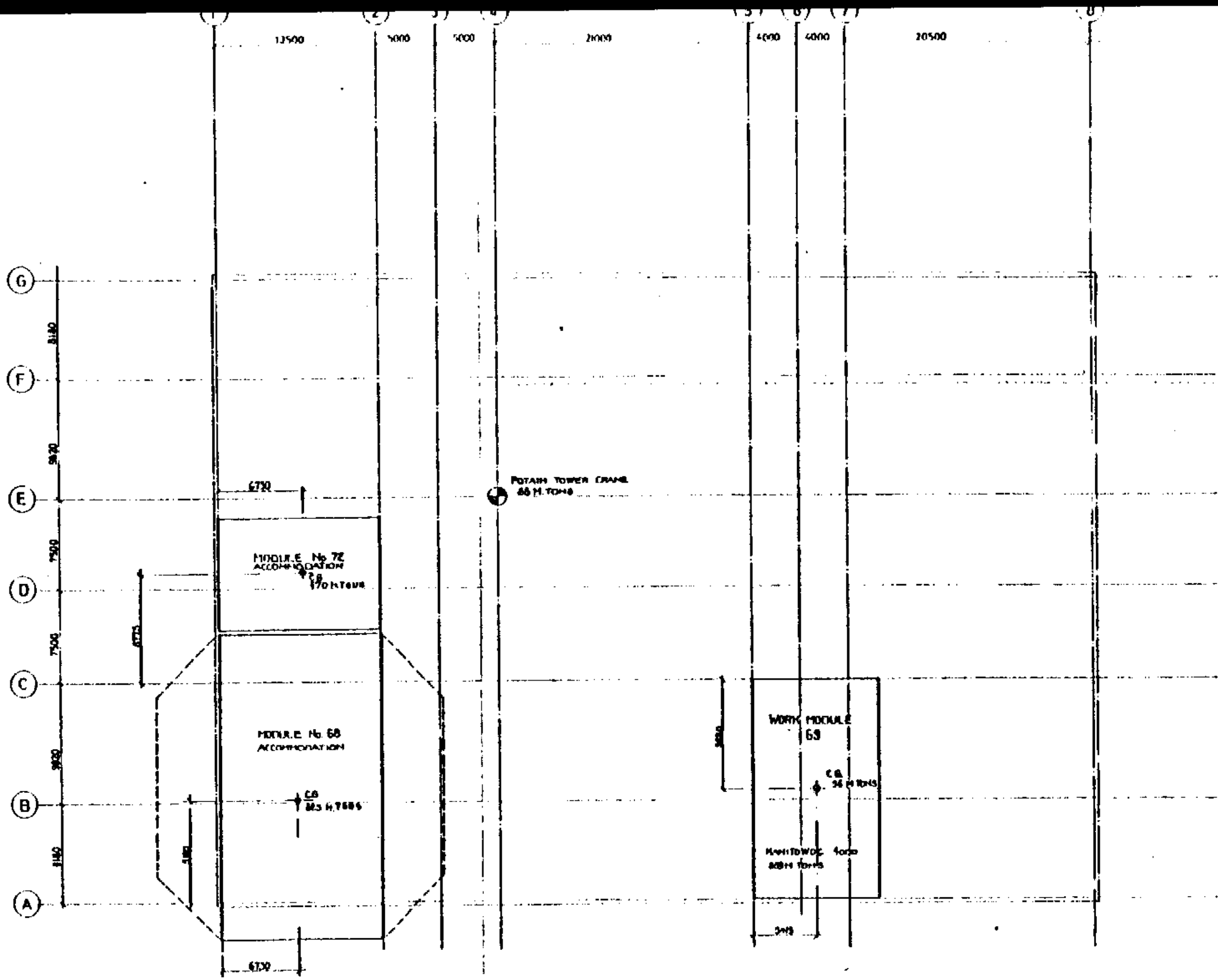
ELF – NORGE A/S – E 10

FRIGG - TCP 2 – STEEL FRAME

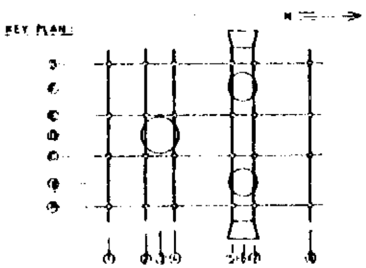
CG. CELLAR DECK PACKAGES DURING DECK MATING

1:200

UC 3337 0449 20 1063 D



KEY PLAN



C	18.1.77	ISSUED FOR APPROVAL
B	27.11.76	GENERAL REVISIONS
A	16.11.76	ISSUED FOR INFORMATION

ELF - NORGE A/S - E 10
FRIGG - TCP 2 - STEEL FRAME

CG MAIN DECK PACKAGES
DURING DECK MATING

1:200

CONDEEP

HØYER ELLEFSEN

UC 3337 0449 20 1064 C

APPENDIX NO. 6.

DESIGN DOCUMENTATION

elf - norge a/s

FRIGG TCP 2

STEEL SUPPORT FRAME

STRUCTURAL DESIGN REPORT

PART I

KVAERNER ENGINEERING A.S

Norwegian Contractors

PART I

References
Drawings for the Structural Analysis

A. DESIGN PREMISES

A.1 General Considerations

A.2 Configuration

A.3 Environmental Conditions

A.3.1 Storm Conditions
A.3.2 Operating Conditions
A.3.3 Conditions during Towing and Immersion
A.3.4 Temperatures
A.3.5 Snow and Ice Accumulation

A.4 Design Codes

A.5 Materials

A.6 Corrosion Protection

B. DESIGN APPROACH

B.1 Structural Model for the Overall Analysis

B.1.1 Operation Phase
B.1.2 Transportation Phase
B.1.3 Towing Phase

B.2 Loadings

B.2.1 Operation Phase

B.2.1.1 Wave and Current Forces
B.2.1.2 Wind Forces
B.2.1.3 Dead Load and Live Load

B.2.2 Transportation Phase
B.2.3 Towing Phase

B.3 Dynamic Analysis

B.4 Fatigue

B.4.1 General Approach

B.4.2 Procedure for the Fatigue Strength Calculations

B.5 Local Structural Analysis

PART II

C. DESIGN CALCULATIONS - OPERATION PHASE

C.1 Introduction

C.2 Loads and Load Combinations

C.3 Summary and Load Effects and Stresses

C.4 Check of Stresses

C.4.1 Truss Members

C.4.1.1 Main Members and Wind Bracing

C.4.1.2 Welds between Web and Upper Flange

C.4.2 Nodes

C.4.2.1 Gusset Plates

C.4.2.2 Crossing Flanges

C.4.2.3 Internal Bulkheads

C.5 Buckling Control of Plates

C.6 Design of Welded Connections

C.6.1 Calculations of Necessary Welds for Beam Elements

C.6.2 Calculation of Longitudinal Welds between Web and Flange of Chords at the Nodes

C.6.3 Design of Connections between Lateral Bracing and Main Trusses

C.7 Design of Prestressing Bars

C.8 Design Calculation of Rings

C.8.1 Stresses in Box-Sections

B.8.2 Stresses in Bottom Plates

C.9 Design of Pancake Supports and External Cantilever Beams

C.9.1 Pancake Supports on the Frame

C.9.2 Pancake Supports on the Rings

C.9.3 External Cantilever Beams and Adjacent Nodes

C.10 Design of Bridge Landing

C.11 Design of Structural Deck

C.12 Design of Bulkheads

C.12.1 Design of External Bulkheads

C.12.2 Design of Plates connecting the Flanges of the Upper Chords

C.13 Fatigue Investigations

C.13.1 Fatigue Calculation and Evaluation of the Truss Members

C.13.2 Fatigue Calculation of the Support Rings (Old Model)

C.13.3 Fatigue Analysis of the Steel Support Rings (Refined Model)

C.13.4 Additional Fatigue Analysis of the Truss Members

D. DESIGN CALCULATIONS - TRANSPORTATION AND TOWING PHASES

D.1 Deck Supported on one Barge

D.1.1 Control of Stresses in Support Frame when transported on Vikbarge

D.1.2 Supports on Seapontoon IV

D.2 Deck Supported on two Barges

D.2.1 Main Supports on Norbarge I and II

D.2.2 Struts between Deck of Norbarge I and II and the underside of the Steel Support Frame.

D.2.3 Check of Stresses in Support Frame

D.3 Stresses during Towing

D.4 Winch Platforms

D.5 Painting Trolley

APPENDIX E :

E1. Calculation of Section Properties of the Truss Members

E2. Calculations of Section Properties of the Nodes

APPENDIX F :

F1. Computer Plots for the Overall Structural Model

F2. Storm Condition versus Operationsl Condition

F3. Comparison of Stresses in the Truss Members due to Operational Waves in different Directions

APPENDIX H :

H1. Extract from the Computer Analysis of Node 410

H2. Stress Analysis of a typical Bulkhead

H3. The effect of prevented Warping in Closed Box-Sections

H4. Wind excited Oscillations of the Members of the TCP 2 Deck Construction

H5. Difference in Stresses for a Deck fixed to the Shafts and a simply Supported Deck

H6. Interaction between Module 01, 02, 03 and 04 and the Support Frame

PART III

K. COMPUTER OUTPUT

K.1 Geometry Check

K.2 Print of Unit Load Cases

K.3 Load Combinations

K.4 Element Loads
- K.4.1 Wind bracing
- K.4.2 Truss row A - G
- K.4.3 Truss row C - E
- K.4.4 Truss row 1 - 2 - 4 - 5 - 7 - 8
- K.4.5 Rings

K.5 Element Stresses
- K.5.1 Wind bracing
- K.5.2 Truss row A - G
- K.5.3 Truss row C - E
- K.5.4 Truss row 1 - 2 - 4 - 5 - 7 - 8
- K.5.5 Rings

K.6 Influence of corrected bridge reaction

APPENDIX NO. 7.

FABRICATION DOCUMENTION

Form No. 40.91a

Enclosure 1

ELF NORGE

T.C.P.2. - STEEL FRAME CHP/UIE

STRUCTURE.

LIST OF DATA BOOKS

1/6 - Welding Procedure CHP/AKER

2/6 - Welding Procedure CHP/AKER

3/6 - C.O.D. test

4/6 - Control Report

5/6 - Electrical stress relieving curves

6/6 - Provisional acceptance - Control reports.

- Beams 434 - 436 - 438 - 238 - 234 - 240
- Beams 676 - 777 - 778 - 675 - 980 - 978 - 979 - 880
  876 - 875 - 878.
- Beams 879 - 877 - 976 - 977 - 975
- Sub assembly 407 - 753 - 41 - 754 - 457
- Sub assembly 207 - 653 - 21 - 654 - 257
- Bracings 413 - 414.
- Beams 256 - 356 - 456
- Bracing 984 Row 7
- Bracing 883 Row 5
- Beams and nodes 858/857 - 51 - Row 5
- Beam 422
- Beam 419
- Beam 431
- Beam 423
- Beam 53
- Beam 93.

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TCP2 Steel frame - Data Books (continued)

- Beam 320
- Beam 424
- Beam 52
- Beams 43 - 42

Beam 421

- Nodes 502 - 504 - 702 - 704
- Nodes 556 - 656 - 756
- Beams 550 - 650 - 750
- Beams 206 - 306 - 406
- Nodes 208 - 408 - 258 - 458
- Beams D 45
  (513 - 45 - 713) (563- 45 - 763)
  (763 - 45 - 764) (563 - 45 - 564)
  (513 - 45 - 514)
- Beams 355/357 and 356/358
- Bracings 677 and 678
- Bracings 60 and 61
- Bracing 237
- Beam 951 + node 713 - Beam 851 + node 513
- Beams 440 - 882 - 982
  (776 - 440 - 757) (555 - 882 - 556)
  (755 - 982 - 756)
- Beams 958/957 and node 51 - Row 7
- Bracing 884 Row 5
- Beam 430
- Beam 428
- Beam 427
- Beam 420
- Bracings 411 - 412 - 415 - 416
- Bracings 433 - 437 - 439
- Bracings 775 - 776
- Bracings 881 - 981
- Beams 911/912 Row 7
- Beams 811/812 Row 5
- Bracing 983 Row 7

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- Beams 56 - 57 - 80 - 81
  (554 - 56 - 656) (656 - 57 - 754)
  (650 - 80 - 552) (752 - 81 - 650)
- Beams 803 - 903 - 804 - 904
  (500 - 803 - 512) (712 - 903 - 700)
  (562 - 804 - 550) (750 - 904 - 762)
- Beams 220 - 232 - 236
  (451 - 220 - 460) (480 - 232 - 551)
  (650 - 236 - 551)
- Beams 120/118 and node 650
- Beam 952 and node 763
- Beam 852 and node 563
- Beams 219 - 231 - 233 - 239
  (410 - 219 - 401) (501 - 231 - 430)
  (501 - 233 - 500) (700 - 239 - 701)
- Upper chord B6
- Bracings 54 and 55
- Nodes 500 - 600 - 700
- Bracings 235/435
- Nodes 552 - 554 - 752 - 754
- Sub assembly : (166 - 481 - 164 - 471 - 162 - 461 - 160)
  (116 - 480 - 114 - 112- 460 - 110) (481 - 230 - 480)
  (480 - 228 - 471) (471 - 224 - 460) (460 - 222 - 461)
- Bridge landing Ring F6
  Sub assembly (562 - 564) (762 - 764)
- Bridge landing Ring B6
  Sub assembly (512 - 514) (712 - 714)
- Beam D 43
- Beams 874 - 974 - 873 - 973
  (562 - 874 - 563) (762 - 974 - 763)
  (513 - 873 - 512) (713 - 973 - 712)
- Node 750 and Beam 906
- Node 550 and Beam 806
- Sub assembly : (751 - 954 - 755 - 956 - 757)
  (551 - 854 - 555 - 856 - 557)
  (751 - 170 - 168 - 551) (757- 370 - 368 - 557)
- Sub assembly : (165 - 431 - 163 - 421 - 161 - 411 - 159)
  (115 - 430 - 113 - 111 - 410 - 109 -) (421 - 223 - 410)
  (431 - 229 - 430) (430 - 227 - 421) (410 - 221 - 411).

...

T.C.P.2. Steel frame - Data books

- Beam 809
- Beam 319
- Beam 318
- Beam 317
- Beam 909

FRIGG PROJECT

TCP 2 steel frame

Production at JULIN, Grand Quevilly and France Entreprises, Ranville

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Final report

(to be read in conjunction with progress reports)

JULIN, Grand Quevilly as subcontractor to CMP, Dunkerque, has in cooperation with MUNCH, Frouard and CMM, Yutz, produced the norther most part of the steel frame between truss row 7 and 8.

France Entreprises, Ranville has, as a member of the same group of fabricators done the assembly of prefabricated parts from Julin, CMM and Munch.

The following documentation is submitted with this report :

- stamped K.E. drawings, see list below.
- Julin shop drawings, see list below.
- report files containing the following documentation :
  - Material certificate copies
  - Dimensional-control reports
  - Diagrams for PWHT (if any)
- Julin welding procedures
- production weld test at F.E.
- Julin working and control procedures
- namelist of approved welders for the job at Julin and F.E.
- Organisation scheme for Julin and F.E.

Le Havre, 5th January, 1977.

for Arild Bachke.

05798 17.01.77

DET NORSKE VERITAS
KLASSIFIKASJON OG REGISTRERING AV SKIP
GRUNNLAGT 1864
HOVEDKONTOR: OSLO

enclosure III

Det norske Veritas
Industrial and Offshore Div.
P.box 300

1322 HØVIK

DISTRIKT: STORD, BØMLO og HALSNØY
POSTADRESSE: POSTBOKS 235, 5401 STORD
TELEGRAMADR.: "VERITAS", STORD
TELEFON: (054) 10 893
BANKGIRO: 8420.07.02.238
BANK: SUNNHORDLANDSBANKEN, STORD

Att: A. Misund

DERES REF.: VÅR REF.: PBre/MSC 5400 STORD, DEN 11 January 1977

FRIGG FIELD II - SUPPORT FRAME TCP 2
FINAL DOCUMENTATION

As separate mail we have today submitted:

- Signed construction drawings in accordance with separate list.
- 2 off draw. No. 2130 "Extension of steel built at Stord Verft A/S", identificating the different sections by their respective numbers.
- 2 off draw. No 1071/1072 indicating which nodes and joints which have been stress-relieved at Stord Verft A/S.
- One booklet with names of certified NDT-operators, who have been working on the project.
- One booklet of applied ultrasonic testing procedures.
- One booklet of welding procedures, production tests, special tests (i.e. for peening, repair and some experiments which Stord Verft carried out with different preheating temperatures), COD-tests and lists of welding operators (their certificates are enclosed).
- Specification of all Reports for single parts, sections etc. built at Stord Verft, and the reports listed there.

The abovementioned should be all available documentation for the steel parts built at Stord Verft A/S.

Enclosed in this letter please find some photographs taken during the construction.

Yours faithfully
for DET NORSKE VERITAS

Magne Olsen
Ass. principal surveyor

Form No. 40.91a

Enclosure IV
VII

T.C.P.2. - STEEL FRAME

MILL CERTIFICATES MISSING

	Item	t	Heat number
Node 755	11B	20 mm	724.300.422
Node 550	25 and 26	20 mm	725.010.143
Beam 806	92	30 mm	724.170.201
Node 750	26	20 mm	725.010.143
Beam 109	51	30 mm	472.002.702
	165	20 mm	795.010.143
Node 410	14	20 mm	725.010.143
	16	20 mm	971.304.702
Node 430	13	20 mm	871.304.702
	14	20 mm	725.010.143
Node 431	38	30 mm	724.170.201
Beam 874	128/130	25 mm	449.568.601
Beam D43-550	185	12 mm	891.068.808
D43-750	186	12 mm	891.068.808
Beam 228	9	30 mm	724.170.201
	10	30 mm	472.003.102
	103	15 mm	724.960.451
Node 460	1	30 mm	472.002.702
	13	30 mm	472.002.702
	14 and 16	20 mm	725.010.143
Node 480	13A	20 mm	725.010.143
	13B	20 mm	871.304.702
Beam 160/162	160	12 mm	725.010.442
	160	12 mm	724.120.341
Beam 164	37	30 mm	472.003.102
	161	12 mm	725.010.442
Ring F6		40 mm	870.104.813.

.../

Steel frame - TCP2
<u>Mill Certificates missing</u>

	t	Heat number
Bracing 435	25 mm	725.310.021
Nodes 500.600.700	20 mm	725.270.522
	20 mm	871.304.702
Bracing 54 - 55	20 mm	719.270.211
Upperchord of B6	20 mm	724.300.442
	60 mm	719.460.601
	50 mm	718.360.701
	80 mm	773.008.901
	20 mm	724.190.101
	50 mm	701.460.201
	20 mm	719.970.441
Nodes 256-356-456	80 mm	725.000.622
Bracing 413	15 mm	724.960.621.

Form No. 40.91a

Enclosure V

FRIGG T. C. P. 2.

STEEL FRAME

LIST OF DRAWINGS.

N°	TITLE	Revision
20-0103	Outline	2
20.0104	Plan main deck	10
20.0105	Plan cellar deck	0
20.0106	Sections of profiles A	2
20.0107	Sections of profiles B	4
20.0108	Sections of profiles D	1
20.0109	Welding details.	1
20.0110	Dimensions, truss row A,C,E,G, 1 and 8	4
20.0111	Dimensions, truss row 2,4,5 and 7	4
20.0151	Plan main deck bracing	1
20.0152	Main deck bracing details	0
20.0153	Plan cellar deck bracing	1
20.0154	Cellar deck bracing - details 1	1
20.0155	Cellar deck bracing - details 2	2

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Steel frame - T.C.P.2.

N°	TITLE	Revision
20.156	Plan structural deck	7
20.0157	Structural deck details 1	1
20.0158	Structural deck details 2	1
20.0159	Structural deck details 3	3
20.0160	Bridge landing	2
20.0161	Details, bridge landing	3
20.0162	Burner boom supports	0
20.0167	Stabbing guides	2
20.0201	Truss row A with details of upper chord	2
20.0202	Truss row A with details of lower chord"	1
20.0203	Truss row A node 100	1
20.0204	Truss row A node 101	0
20.0205	Truss row A node 110	0
20.0206	Truss row A node 200	0
20.0207	Truss row A node 201	0
20.0208	Truss row A node 300	0
20.0209	Truss row A node 400	0
20.0210	Truss row A node 401	1
20.0211	Truss row A node 410	1
20.0212	Truss row A node 421	0
20.0213	Truss row A node 430	0
20.0214	Truss row A node 500	0
20.0215	Truss row A node 501	3
20.0216	Truss row A node 600	0
20.0217	Truss row A node 700	0
20.0218	Truss row A node 701	0
20.0219	Truss row A node 710	2
20.0220	Truss row A node 721	0

...

Steel frame - T.C..P2.

N°	TITLE	Revision
20.0221	Truss row A node 730	1
20.0222	Truss row A node 800	1
20.0223	Truss row A node 801	1
20.0251	Truss row G with details of upper chord	3
20.0252	Truss row G with details of lower chord	1
20.0253	Truss row G node 150	2
20.0254	Truss row G node 151	0
20.0255	Truss row G node 160	0
20.0256	Truss row G node 250	1
20.0257	Truss row G node 251	0
20.0258	Truss row G node 350	0
20.0259	Truss row G node 450	1
20.0260	Truss row G node 451	1
20.0261	Truss row G node 460	1
20.0262	Truss row G node 471	0
20.0263	Truss row G node 480	0
20.0264	Truss row G node 550	0
20.0265	Truss row G node 551	1
20.0266	Truss row G node 650	0
20.0267	Truss row G node 750	0
20.0268	Truss row G node 751	2
20.0269	Truss row node 760	2
20.0270	Truss row G node 771	0
20.0271	Truss row G node 780	1
20.0272	Truss row G node 850	1
20.0273	Truss row G node 851	0

Steel frame - T.C.P.2.

N°	TITLE	Revision
20.0301	Truss row C with details of upper chord	1
20.0302	Truss row C with details of lower chord.	1
20.0303	Truss row C node 106	3
20.0304	Truss row C node 107	1
20.0305	Truss row C node 116	2
20.0306	Truss row C node 207	5
20.0307	Truss row C node 407	3
20.0308	Truss row C node 416	1
20.0309	Truss row C node 427	0
20.0310	Truss row node 436	1
20.0311	Truss row C node 506	1
20.0312	Truss row C node 507	0
20.0313	Truss row C node 606	0
20.0314	Truss row C node 706	1
20.0315	Truss row C node 707	0
20.0316	Truss row C node 716	1
20.0317	Truss row C node 727	1
20.0318	Truss row C node 736	2
20.0319	Truss row C node 806	1
20.0320	Truss row node 807	1
20.0351	Truss row E with details of upper chord	1
20.0352	Truss row E with details of lower chord	1
20.0353	Truss row E node 156	2
20.0354	Truss row E node 157	0
20.0355	Truss row E node 166	0
20.0356	Truss row E node 257	4
20.0357	Truss row E node 457	1
20.0358	Truss row E node 466	1

Steel frame - T.C.P.2.

N°	TITLE	Revision
20.0359	Truss row E node 477	0
20.0360	Truss row E node 486	0
20.0361	Truss row E node 556	0
20.0362	Truss row E node 557	0
20.0363	Truss row E node 656	0
20.0364	Truss row E node 765	1
20.0365	Truss row E node 757	0
20.0366	Truss row E node 766	1
20.0367	Truss row E node 777	2
20.0368	Truss row E node 786	1
20.0369	Truss row E node 856	1
20.0370	Truss row E node 857	0
20.0401	Truss row 1 with details of upper chord	2
20.0402	Truss row 1 with details of lower chord	0
20.403	Truss row 1 node 10, 104 and 154	1
20.0451	Truss row 8 with details of upper chord	1
20.0452	Truss row 8 with details of lower chord	0
20.0453	Truss row 8 node 80, 804 and 854	1
20.0501	Truss row 2 with details of upper chord	1
20.0502	Truss row 2 with details of lower chord	1
20.0503	Truss row 2 node 21	1
20.0504	Truss row 2 node 204 and 254	0
20.0551	Truss row 4 with details of upper chord	1
20.0552	Truss row 4 with details of lower chord.	1
20.0553	Truss row 4 node 41	0
20.0554	Truss row 4 node 404 and 454	0

Steel frame - T.C.P.2.

N°	TITLE	REVISION
20.0601	Truss row 5 with details of upper chord	0
20.0602	Truss row 5 with details of lower chord	0
20.0603	Truss row 5 node 51, 513 and 563	1
20.0604	Truss row 5 node 505 and 555	1
20.0605	Truss row 5 node 514 and 512	1
20.0606	Truss row 5 node 564 and 562	1
20.0651	Truss row 7 with details of upper chord	0
20.0652	Truss row 7 with details of lower chord	0
20.0653	Truss row 7 node 71, 713 and 763	3
20.0654	Truss row 7 node 705 and 755	1
20.0655	Truss row 7 node 714 and 712	0
20.0656	Truss row 7 node 764 and 762.	0
20.0701	Supporting ring D3 plan	1
20.0702	Supporting ring D3 plan - section	1
20.0703	Supporting ring D3 node 206 and 256	2
20.0704	Supporting ring D3 node 208 and 258	1
20.0705	Support ring D3 node 306 and 356	1
20.0706	Support ring D3 node 406 and 456	2
20.0707	Support ring D3 node 408 and 458	1
20.0708	Support ring D3 arrangement of anchor bars eastern half	10.
20.0709	Support ring D3 arrangement of anchor bars western half	9
20.0710	Support ring D3 node 256.	3
20.0801	Support ring B6 plan	1
20.0802	Support ring B6 plan - section	2
20.0803	Support ring B6 node 502 and 702	1
20.0804	Support ring B6 node 504 and 704	1
20.0805	Support ring B6 - arrangements of anchor bars eastern half	7
20.0806	Support ring B6 - arrangements of anchor bars western half.	9

Steel frame - T.C.P.2.

N°	TITLE	Revision
20.0851	Support ring F6 plan -	2
20.0852	Support ring F6 plan - section	2
20.0853	Support ring F6 node 552 and 752	0
20.0854	Support ring F6 node 554 and 754	1
20.0856	Support ring F6 - arrangement of anchor bars eastern half	7
20.0855	Support ring F6 - arrangements of anchor bars western half.	7
20.0901	Typical details	3
20.0902	Inspection ways-main deck	2
20.0903	Inspection ways - cellar deck	2
20.0904	Cantilevers for ring pancakes	3
20.0905	Plating and support of deck between pancakes	1
20.0906	Corner guides for pancakes	C
20.0907	Templets for anchor bars - ring D3 - eastern half	3
20.0908	Templets for anchor bars - ring D3 - eastern half	2
20.0909	Templets for anchor bars - ring B6 eastern half	1
20.0910	Templets for anchor bars - ring B6 western half	2
20.0911	Templets for anchor bars - ring F6 eastern half	1
20.0912	Templets for anchor bars - ring F6 western half	1
20.0913	Gantry girders for painting trolley	4
20.0914	12 mm steel plate at top of lower chord	0
20.0915	Temporary sidings	3
20.0917	Guides for pancakes	B
20.0918	Gantry girders details	2
20.0919	Details plate stiffeners and anchor plates in rings.	1
20.0920	Details plating row 1 and 8	0

Steel frame - T.C.P.2.

N°	TITLE	Revision
20.0921	Details plating part of row 1 E-G	0
20.0922	Plan and details templets ring F6, B6 and D3	0
20.0923	Stiffeners at lower chord row A,C, E and G	0
20.0924	Angles and pads truss row A,C,E and G	4
20.0925	Angles and pad truss row 1,2 and 4	3
20.0926	Angles and pads truss row 5,7 and 8	3
20.0927	Angles and pads details	3
20.0928	Angles and pads details	4
20.0929	Plan cellar deck, location of towing winches, captains bridge and walkways	1
20.0930	Winch platform n° 2	2
20.0931	Winch platform n° 10	2
20.0932	Winch platform n° 2 - grating	0
20.0933	Winch platform n° 10 - grating	1
20.0934	Pipe support on bridge landing west side	2
20.0935	Drainage piping penetration cellar deck.	2
20.936	Plate support on truss row G 1.7 lower chord	1
20.0937	Winch platform 5,6 and 8 - dwgI	2
20.0938	Winch platform 5,6 and 8 - dwg II	2
20.0939	Winch platform 5,6 and 8 - grating	0
20.0940	Winch platform n° 9 and 4	2
20.0941	Winch platform n° 3	0
20.0942	Temporary sidings - general view	1
20.0943	Temporary sidings - details II	3
20.0944	Temporary platforms	4
20.0945	Location supports longitudinal and transverse girders.	2
20.0946	Baseplates of tower crane	0

Steel frame - T.C.P.2.

N°	TITLE	Revision
20.0947	Attachment plates on cantilevers	2
20.0948	Support for 50 tons jacks	1
20.0949	Base plate requirement for mooring system shaft 1	5
20.0950	Base plate requirement for mooring system shaft 3	3
20.0951	Base plate requirement for mooring system shaft 5	2
20.0952	Shock absorber attachment	2
20.0953	Tensioner attachment plate	1
20.0954	Cable tray supports truss row 1,8,G	B
20.0956	Brackets for steel frame mooring padeys	0
20.0957	Winch platform n° 4 and 9 - grating	0
20.0958	Winch platform n° 3 - grating	0
20.0959	Welding plates for Brown & Root	2
20.0960	Base plate requirements for mooring system, truss row 4	1
20.0961	Cable tray supports - sectional views	0
20.0962	Temporary handrails truss row C between row 1 and 2	0
20.0963	Location of pancakes guides	3
20.0964	Temporary platform 33	0
20.0965	Winch platform 5,6,7 and 8 - Dwg III	0
20.0966	Guides for pancakes - Type I and II	A
20.0967	Temporary platform 23A and 25A	0
20.0968	Temporary platform 31A	0
20.0970	Welding plates on truss row A	0
20.0971	Welding plates on truss row C	0
20.0972	Welding plates on truss row E	0
20.0973	Welding plates on truss row 5	0
20.0974	Welding plates on truss row 1	1
20.0975	Welding plates on truss row 2	2
20.0976	Welding plates on truss row 4	1
20.0977	Access mooring system in structural deck	0

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N°	TITLE	Revision
20.0978	Rail for access to winch support platforms	0
20.0979	Welding plates on truss row 7	0
20.0980	Extension of temporary Siding Truss row 1	1
20.0981	Plates on truss row A,C,E and G for stability arrangements on norbarge I and II in andalsnes.	0
20.1101	Temporary platforms, plan shaft 1	0
20.1102	Temporary platforms, plan shaft 3	0
20.1103	Temporary platforms, plan shaft 5	0
20.1201	Temporary platforms and pancakes, plan shaft 1, F6	A
20.1203	Temporary platforms and pancakes, plan shaft 5, D3	A
20.1202	Temporary platforms and pancakes plan shaft 3, B6	A
20.2001	Location of support points and support frames on deck - Sea Pontoon 4	0
20.2002	Grid system under line A-C-E-G Sea Pontoon 4	0
20.2005	Stiffening and cantilevers for jacks on skid beams - Sea Pontoon 4	0
202006	Vertical supports transverse seafaste-ning - Sea Pontoon 4	0
20.2007	Support girders Line A and G alternativ I - Sea Pontoon 4	0
20.2008	Support girders line C and E alterna-tiv I - Sea Pontoon 4	0
20.2009	Transversal seafastening Sea Pontoon 4	0
20.2010	Longitudinal seafastening Sea Pontoon 4	0
20.2011	Lengthening of skid beams - Seapontoon	0
20.2012	End support of skidbeams Row A and G Sea Pontoon 4	0
20.2013	End support of skidbeams Row C and E	0
20.2021	Jacking and seafastening operations on the Barge/	A

ET NORSKE VERITAS
O. BOX 190
01 ÅNDALSNES

TCP-2 Steel Frame

List of 'AS CARRIED OUT DRAWINGS'

- STIFFENERS AT LOWER CHORD ROW 5 and 7
  Dwg. KE 0990 Rev. 0
- BASE PLATE REQUIREMENT FOR MOORING SYSTEM SHAFT 3
  Dwg. KE 0950 Rev. 4
- BRACKETS FOR STEEL FRAME MOORING PADEYES
  Dgw. KE 0956 Rev. 1
- BASE PLATE REQUIREMENT FOR MOORING SYSTEM SHAFT 1
  Dwg. KE 0949 Rev. 6
- ACCESS MOORING SYSTEM IN STRUCTURAL DECK
  Dwg. KE 0977 Rev. 0
- WELDING PLATES ON TRUSS ROW C
  Dwg. KE 0971 Rev. 0
- WELDING PLATES ON TRUSS ROW A
  Dwg. KE 0970 Rev. 0
- ANGLES AND PADS TRUSS ROW A - C - E and G.
  Dwg. KE 0924 Rev. 4
- PLAN STRUCTURAL DECK
  Dwg. KE 0156 Rev. 7
- PLAN MAIN DECK BRACING
  Dwg. KE 0151 Rev. 1
- PLAN MAIN DECK
  Dwg. KE 0104 Rev. 10

Åndalsnes, 7.6.77

W. Boulogne

W. Boulogne

APPENDIX NO. 8.

DRAWING INDEX

DRAWING INDEX

FRIGG TCP 2

STEEL SUPPORT FRAME

KVÆRNER ENGINEERING A.S

20-0103	Rev.	2	Outline
20-0104	"	10	Plan main deck
20-0105	"	0	Plan cellar deck
20-0106	"	2	Sections of profiles A
20-0107	"	4	Sections of profiles B
20-0108	"	1	Sections of profiles D
20-0109	"	1	Welding details
20-0110	"	4	Dimentions, truss row A,C,E,G,1 and 8
20-0111	"	4	Dimentions, truss row 2,4,5 and 7
20-0151	"	1	Plan main deck bracing
20-0152	"	0	Main deck bracing details
20-0153	"	2	Plan cellar deck bracing
20-0154	"	1	Cellar deck bracing details 1
20-0155	"	2	Cellar deck bracing details 2
20-0156	"	8	Plan structural deck
20-0157	"	1	Structural deck details 1
20-0158	"	1	Structural deck details 2
20-0159	"	3	Structural deck details 3
20-0160	"	2	Bridge landing
20-0161	"	3	Details, bridge landing
20-0162	"	0	Burner boom supports, upper and lower chord
20-0167	"	2	Cantilevers with stabbing guides at end of trusses
20-0201	"	2	Truss row A with details of upper chord
20-0202	"	1	Truss row A with details of lower chord
20-0203	"	1	Truss row A node 100
20-0204	"	0	Truss row A node 101
20-0205	"	0	Truss row A node 110

20-0206	Rev.	0	Truss row A node 200
20-0207	"	0	Truss row A node 201
20-0208	"	0	Truss row A node 300
20-0209	"	0	Truss row A node 400
20-0210	"	1	Truss row A node 401
20-0211	"	1	Truss row A node 410
20-0212	"	0	Truss row A node 420
20-0213	"	0	Truss row A node 430
20-0214	"	0	Truss row A node 500
20-0215	"	3	Truss row A node 501
20-0216	"	0	Truss row A node 600
20-0217	"	0	Truss row A node 700
20-0218	"	0	Truss row A node 701
20-0219	"	2	Truss row A node 710
20-0220	"	0	Truss row A node 721
20-0221	"	1	Truss row A node 730
20-0222	"	1	Truss row A node 800
20-0223	"	1	Truss row A node 801
20-0251	"	3	Truss row G with details of upper chord
20-0252	"	1	Truss row G with details of lower chord
20-0253	"	2	Truss row G node 150
20-0254	"	0	Truss row G node 151
20-0255	"	0	Truss row G node 160
20-0256	"	1	Truss row G node 250
20-0257	"	0	Truss row G node 251
20-0258	"	0	Truss row G node 350
20-0259	"	1	Truss row G node 450
20-0260	"	1	Truss row G node 451

20-0261	Rev.	1	Truss row G node 460
20-0262	"	0	Truss row G node 471
20-0263	"	0	Truss row G node 480
20-0264	"	0	Truss row G node 550
20-0265	"	1	Truss row G node 551
20-0266	"	0	Truss row G node 650
20-0267	"	0	Truss row G node 750
20-0268	"	2	Truss row G node 751
20-0269	"	2	Truss row G node 760
20-0270	"	0	Truss row G node 771
20-0271	"	1	Truss row G node 780
20-0272	"	1	Truss row G node 850
20-0273	"	0	Truss row G node 851
20-0301	"	1	Truss row C with details of upper chord
20-0302	"	1	Truss row C with details of lower chord
20-0303	"	3	Truss row C node 107
20-0304	"	1	Truss row C node 107
20-0305	"	2	Truss row C node 116
20-0306	"	5	Truss row C node 207
20-0307	"	3	Truss row C node 407
20-0308	"	1	Truss row C node 416
20-0309	"	0	Truss row C node 427
20-0310	"	1	Truss row C node 436
20-0311	"	1	Truss row C node 506
20-0312	"	0	Truss row C node 507
20-0313	"	0	Truss row C node 606
20-0314	"	1	Truss row C node 706
20-0315	"	0	Truss row C node 707

20-0316	Rev.	1	Truss row C node 716
20-0317	"	1	Truss row C node 727
20-0318	"	2	Truss row C node 736
20-0319	"	1	Truss row C node 806
20-0320	"	1	Truss row C node 807
20-0351	"	1	Truss row E with details of upper chord
20-0352	"	1	Truss row E with details of lower chord
20-0353	"	2	Truss row E node 156
20-0354	"	0	Truss row E node 157
20-0355	"	0	Truss row E node 166
20-0356	"	4	Truss row E node 257
20-0357	"	1	Truss row E node 457
20-0358	"	1	Truss row E node 466
20-0359	"	0	Truss row E node 477
20-0360	"	0	Truss row E node 486
20-0361	"	0	Truss row E node 556
20-0362	"	0	Truss row E node 557
20-0363	"	0	Truss row E node 656
20-0364	"	1	Truss row E node 765
20-0365	"	0	Truss row E node 757
20-0366	"	1	Truss row E node 766
20-0367	"	2	Truss row E node 777
20-0368	"	1	Truss row E node 786
20-0369	"	1	Truss row E node 856
20-0370	"	0	Truss row E node 857
20-0401	"	2	Truss row 1 with details of upper chord
20-0402	"	0	Truss row 1 with details of lower chord
20-0403	"	1	Truss row 1 node 10, 104 and 154

20-0451	Rev.	1	Truss row 8 with details of upper chord
20-0452	"	0	Truss row 8 with details of lower chord
20-0453	"	1	Truss row 8 node 80, 804 and 854
20-0501	"	1	Truss row 2 with details of upper chord
20-0502	"	1	Truss row 2 with details of lower chord
20-0503	"	1	Truss row 2 node 21
20-0504	"	0	Truss row 2 node 204 and 254
20-0551	"	1	Truss row 4 with details of upper chord
20-0552	"	1	Truss row 4 with details of lower chord
20-0553	"	0	Truss row 4 node 41
20-0554	"	0	Truss row 4 node 404 and 454
20-0601	"	0	Truss row 5 with details of upper chord
20-0602	"	0	Truss row 5 with details of lower chord
20-0603	"	1	Truss row 5 node 51, 513 and 563
20-0604	"	1	Truss row 5 node 505 and 555
20-0605	"	1	Truss row 5 node 514 and 512
20-0606	"	1	Truss row 5 node 564 and 562
20-0651	"	0	Truss row 7 with details of upper chord
20-0652	"	0	Truss row 7 with details of lower chord
20-0653	"	3	Truss row 7 node 71, 713 and 763
20-0654	"	1	Truss row 7 node 705 and 755
20-0655	"	0	Truss row 7 node 714 and 712
20-0656	"	0	Truss row 7 node 764 and 762
20-0701	"	1	Supporting ring D3 plan
20-0702	"	1	Supporting ring D3 plan section
20-0703	"	2	Supporting ring D3 node 206 and 256
20-0704	"	1	Supporting ring D3 node 208 and 258
20-0705	"	1	Support ring D3 node 306 and 356

20-0706	Rev.	2	Support ring D3 node 406 and 456
20-0707	"	1	Support ring D3 node 408 and 458
20-0708	"	10	Support ring D3 arrangement of anchor bars eastern half
20-0709	"	9	Support ring D3 arrangement of anchor bars western half
20-0710	"	3	Support ring D3 node 256
20-0801	"	1	Support ring B6 plan
20-0802	"	2	Support ring B6 plan selection
20-0803	"	1	Support ring B6 node 502 and 702
20-0804	"	1	Support ring B6 node 504 and 704
20-0805	"	7	Support ring B6 arrangements of anchor bars eastern half
20-0806	"	9	Support ring B6 arrangement of anchor bars western half
20-0851	"	2	Support ring F6 plan
20-0852	"	2	Support ring F6 plan section
20-0853	"	0	Support ring F6 node 552 and 752
20-0854	"	1	Support ring F6 node 554 and 754
20-0855	"	7	Support ring F6 arrangement of anchor bars eastern half
20-0856	"	7	Support ring F6 arrangement of anchor bars western half
20-0901	"	3	Typical details
20-0902	"	2	Inspection ways main deck
20-0903	"	2	Inspection ways cellar deck
20-0904	"	3	Cantilevers for ring pancakes
20-0905	"	1	Plating and support of deck between pancakes
20-0907	"	3	Templets for anchor bars ring D3 eastern half
20-0908	"	2	Templets for anchor bars ring D3 western half
20-0909	"	1	Templets for anchor bars ring B6 eastern half
20-0910	"	2	Templets for anchor bars ring B6 western half

20-0911	Rev.	1	Templets for anchor bars ring F6 eastern half
20-0912	"	1	Templets for anchor bars ring F6 western half
20-0913	"	4	Gantry girders for painting trolley
20-0914	"	0	12mm steel plate at top of lower chord
20-0915	"	3	Temporary sidings
20-0917	"	0	Guides for pancakes type III and IV
20-0918	"	2	Gantry girders details
20-0919	"	1	Details plate stiffeners & anchor plates in ring
20-0920	"	0	Details plating row 1 and 8
20-0921	"	0	Details plating part of row 1
20-0922	"	0	Plan and details templets ring F6, B6 and D3
20-0923	"	0	Stiffeners at lower chord row A,C,E and G
20-0924	"	4	Angles and pads truss row A,C,E and G ✓
20-0925	"	3	Angles and pads truss row 1,2 and 4 ✓
20-0926	"	3	Angles and pads truss row 5,7 and 8 ✓
20-0927	"	3	Angles and pads details
20-0928	"	4	Angles and pads details
20-0929	"	1	Plan cellar deck, location of towing winches, captains bridge and walkways ✓
20-0930	"	2	Winch platform no 2 ✓
20-0931	"	2	Winch platform no 10
20-0932	"	0	Winch platform no 2 grating
20-0933	"	1	Winch platform no 10 grating
20-0934	"	2	Pipe support on bridge landing west side
20-0935	"	2	Drainage piping penetrations cellar deck
20-0936	"	1	Plate support on truss row G 1-7 lower chord
20-0937	"	2	Winch platform 5,6 and 8 dwg I
20-0938	"	3	Winch platform 5,6 and 8 dwg II

20-0939	Rev.	0	Winch platform 5,6 and 8 grating
20-0940	"	2	Winch platform no 9 and 4
20-0941	"	0	Winch platform no 3
20-0942	"	2	Temporary sidings general view
20-0943	"	3	Temporary sidings details II
20-0944	"	4	Temporary platforms
20-0945	"	2	Location support longitudinal girders, details end bracing
20-0946	"	0	Baseplates of tower crane
20-0947	"	2	Attachment plates on cantilevers
20-0948	"	1	Support for 50 tonns jacks
20-0949	"	6	Base plate requirement for mooring system shaft 1
20-0950	"	4	Base plate requirement for mooring system shaft 3
20-0951	"	2	Base plate requirement for mooring system shaft 5
20-0952	"	2	Shock absorber attachement
20-0953	"	1	Tensioner attachement plate
20-0956	"	1	Brackets for steel frame mooring padeyes
20-0957	"	0	Winch platform no 4 and 9 grating
20-0958	"	0	Winch platform no 3 grating
20-0959	"	2	Welding plates for Brown & Root
20-0960	"	1	Base plate requirements for mooring system, truss row 4
20-0961	"	2	Cable tray supports location
20-0962	"	0	Temporary handrails truss row C between row 1 and 2
20-0963	"	0	Location of pancake guides
20-0964	"	0	Temporary platform 33
20-0965	"	0	Winch platform 5,6,7 and 8 dwgIII

20-0966	Rev.	0	Guides for pancakes Type I and II
20-0967	"	0	Temporary platform 23A and 25A
20-0968	"	0	Temporary platform 31A
20-0970	"	0	Welding plates on truss row A
20-0971	"	0	Welding plates on truss row C
20-0972	"	0	Welding plates on truss row E
20-0973	"	0	Welding plates on truss row
20-0974	"	1	Welding plates on truss row 1
20-0975	"	2	Welding plates on truss row 2
20-0976	"	1	Welding plates on truss row 4
20-0977	"	0	Access mooring system in structural deck
20-0978	"	1	Rail for access to winch support platforms
20-0979	"	0	Welding plates on truss row 7
20-0980	"	1	Extension of temporary siding truss row 1
20-0981	"	0	Plates on truss row A,C,E and G for stability arrangements on Norbarge I and II in Åndalsnes
20-0982	"	1	Painting trolley with details
20-0983	"	0	Painting trolley driving machinery
20-0984	"	0	Painting trolley driving machinery, details
20-0985	"	0	Padeye painting trolley, detail
20-0986	"	0	Detail for painting trolley
20-0987	"	0	Securing arrangement for wheel sets
20-0989	"	0	Welding plates on bridge landing, west side
20-0990	"	0	Stiffeners at lower chord, row 5 and 7
20-0991	"	0	Fire wall on truss row A and truss row 8
20-1101	"	0	Temporary platforms, plan shaft 1
20-1102	"	0	Temporary platforms, plan shaft 3
20-1103	"	0	Temporary platforms, plan shaft 5

20-2001	Rev. 0	Location of support points and support frames on deck, Seapontoon 4
20-2002	" 0	Grid system under line A-C-E-G, Seapontoon 4
20-2005	" 1	Stiffening and cantilevers for jacks on skid beams, Seapontoon 4
20-2006	" 1	Vertical supports transverse seafastening
20-2007	" 0	Support girders Line A and G Alternative I
20-2008	" 0	Support girders Line C and E Alternative I
20-2009	" 1	Transversal seafastening, Seapontoon 4
20-2010	" 1	Longitudinal seafastening, Seapontoon 4
20-2011	" 0	Lengthening of skidbeams, Seapontoon 4
20-2012	" 0	End support of skidbeams row A and G, Seapontoon 4
20-2013	" 0	End support of skidbeams row C and E, Seapontoon 4
20-2021	" A	Jacking and seafastening operations on the barge
20-2101	" 0	Location of grid system on barge
20-2102	" 0	Location of support frames
20-2103	" 0	Grid system on barge
20-2104	" 0	Bracing system, Norbarge I and II
20-2105	" 0	Support frames, Norbarge I and II
20-2106	" 0	Stability arrangements, Norbarge I and II
20-2107	" 1	General view stability arrangement under line C and E
20-2108	" 0	General view stability arrangement under line A and G
20-2109	" 0	Stability arrangements, detail 1 and 2
20-2110	" 1	Stability arrangements, detail 3
20-2111	" 0	Stability arrangements, details 4,5,6, and 7

APPENDIX NO. 9.

REFERENCES

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