VERITAS Report No.
601636/77

Subject Group

Title of Report

FRIGG FIELD - DRILLING PLATFORM NO. 2
STRUCTURAL PARTS
DESIGN, FABRICATION AND INSTALLATION (DFI) RESUMÉ

Client/Sponsor of project

NORWEGIAN PETROLEUM DIRECTORATE

DET NORSKE VERITAS ★ ESTABLISHED 1864 ★

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TECHNICAL REPORT

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SUMMARY

This report deals with the design, fabrication and installation of the fixed offshore structure Frigg Field Drilling Platform No. 2. The related control and inspection activities carried out by Det norske Veritas on behalf of the Norwegian Petroleum Directorate are described.

After a general description of the platform, an outline of the basis for the design, fabrication and installation of the structure follows. Thereafter, the design calculations received and evaluated are explained. Also, the fabrication and installation of the structure is briefly reviewed.

In Appendix 5 those areas which, based on design calculations and inspection during fabrication and installation, are considered to be most critical, are pointed out, i.e. Design/Fabrication/Inspection (DFI) resumé.

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1. INTRODUCTION

This report deals with the design, fabrication and installation of the fixed offshore structure Frigg Field Drilling Platform No. 2. The related control and inspection activities carried out by Det norske Veritas, on behalf of Norwegian Petroleum Directorate is also 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 activities are further laid down in "Scope of Work for Control and Inspection of Fixed Offshore Platform DP2 - Frigg Field on behalf of Norwegian Petroleum Directorate" dated October 28 , 1975.

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

This report which as mentioned above comprices of a resyme of the design, fabrication and installation of the Frigg Field Drilling Platform No. 2 (DP2), covers only the structural parts of the platform.

The report contains initially general information of the platform - including Jacket with Piles, Deck Support Frame and the Production Modules A, B, C and D. The Forex Nepture Drilling Modules and Derrick are covered by a separate Report. Same also applies to risers.

Chapter 3 outlines the basic premises with respect to design, fabrication and installation of the structure.

Chapter 4 reviews the design of the platform and the chapter 5 and 6 respectively, briefly describes the fabrication and installation of the structure.

A number of appendices are attached to this report. These appendices describe various aspects related to the design, fabrication and installation of the structure in more depth than that given in the report, itself. Appendix no. 5 points out those joints and members of the structure which-based on our review of the design as well as our inspection during fabrication and installation of the platform - are considered most critical, relatively spoken. This appendix is ment to serve as a guide when the inservice inspection program is to be worked out.

Aspects which are considered important for the inservice inspection of the platform is also underlined in the following text.

This 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.

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2. GENERAL DESCRIPTION

2.1 General

Frigg Field Drilling Platform No. 2 is a 24-well drilling and production platform installed in the Frigg Field, North Sea, during the summer of 1976. The platform which is a steel template structure is secured to the sea bed by driven and insert piles in a water depth of 97,4 m MLW. A plate girder Deck Support Frame is installed on the top of the Jacket to support four Production Modules. Drilling contractor's (Forex Neptune) equipment is located on top of the production modules. The drilling contractor's modules are self-contained and of modular construction. The structural design of the platform has been undertaken by McDermott-Hudson Engineering, London.

2.2 Description of Jacket and Piles (Dwg ELN 2144 Shts 001, 002, 003)

The steel Jacket is essentially an eight legged structure rectangular in plan, 348 ft in height, divided vertically into 6 elevations with base dimensions of 202' - 6" by 142' - 5" at EL(-)332' - 7" and 157' - 8" by 82' - 4" at the top EL(+)20'. The Jacket is secured to the sea bed by 16 piles located in groups of 4 equally spaced on a 16 ft diameter circle around each corner leg. Four erection piles were used in the initial stages on the centre legs to assist in securing the Jacket to the sea bed. For the permanent inplace condition, these piles are assumed to take shear load only, and no axial load. The long axis of the Jacket has been designated column line A and B and the minor axis have been subdivided into column lines 1, 2, 3 and 4, each containing two main legs of the Jacket. The horizontal frames are designated by their distance in meters from the Mean Low Water line (Design water depth). The corner legs are of varying diameter from 62"Ø at the surface to 120"Ø at mudline. Four pile sleeves are attached to the 120"Ø leg by vertical shear plates and are the main load carrying members of the structure transferring shear and axial load to the foundation. Twenty-four wells will be used during production from the gas field and these are located in two groups of 12 between column lines 1 and 2 of the Jacket and are restrained by conductor framing at each horizontal elevation. The design calculations assume that the conductors do not participate in the platform foundation.

Column lines 2 and 3 have been designed to allow for skidding of the structure in the fabrication yard and on the launch barge. A launch cradle complete with timbers has been located along legs A2 and A3.

Additional buoyancy during the flotation sequence was provided by sixteen 62"Ø, two 100"Ø, and two 144"Ø buoyancy tanks. The sixteen 62"Ø were located within the pile guides at each corner and the 100"Ø on the inside of legs A2 and A3, and the 144"Ø on the inside of legs B2 and B3.

During launch the Jacket floated in the water at an angle from the horizontal on side A and it was rotated or "upended" by controlled flooding of the corner legs.

This operation was conducted from a control panel mounted between legs B2 and B3 at jacket EL(+)20'.

Provision was made for a 26"Ø production riser and an $8\frac{5}{8}$"Ø kill line on leg B2 with a 26"Ø production riser and $4\frac{1}{2}$"Ø mudline on leg B3. These risers have been attached by adjustable clamps at intervals down each leg. One $8\frac{5}{8}$"Ø electrical riser is provided on the inboard side of legs B2 and B3, each equipped with a bellmouth at mudline.

Each leg is protected at water line by a barge bumper while a boat landing is provided on column line A between legs A1 and A2. Riser protectors are provided on legs B3 and B4. (from 1978).

A walkway is provided on the inboard side of the top horizontals at EL(+)20' on each column line.

A number of casings are located between Deck Support Frame and Jacket EL(-)37' - 6". Two 20"Ø sea water casings are positioned on column line 1, a 28"Ø excavator line on column line 2 and two 24"Ø and one 16"Ø sea water casing on column line 4.

The sixteen corner piles consist of a 56"Ø primary pile driven to a design penetration of 60 ft into the sea bed and grouted to sleeves attached to the bottle legs. The annulus between pile and sleeve was closed by an inflatable seal to prevent loss of grout. 42"Ø insert piles, approximately 450 ft in length were placed in a 48"Ø predrilled hole of penetration 385 ft below sea bed level and grouted. Grouting generally occured in two stages to prevent hydraulic fracture of the soil. A 4.5"Ø grout line was prefabricated within the lower sections of each insert pile. The upper section of grout stinger was field installed. The first stage runs from the tip of the insert to the two stage grout tool located just within and protected by the tip of the primary pile. The grouting of the section within the primary pile followed after activation of the two stage grout tool. The centre piles were driven to a design penetration approximately 70 ft, and grouted to the centre legs. The corner piles were supported on each horizontal elevation by a series of pile guides. Each primary pile terminates above the pile sleeves on the corner legs and driving was accomplished using followers projecting above water level. The piles required for the jacket were subdivided for ease of handling into sections which could be welded together offshore to make up the complete length. The corner piles were designed as a form of friction pile with end bearing while the centre piles have been assumed to take shear load only.

Tubular members have been used throughout the design in order to reduce the wave forces acting on the structure and to provide buoyancy during installation.

The jacket has been designed as a launch type structure to be fabricated horizontally on the A side and transported by a barge equipped with launch facilities.

The loadout weight of the jacket complete with buoyancy tanks was approximately 10,000 Short Tons with the centre of gravity located 165 ft above the bottom horizontals, 1.38 ft towards column line 2 and 0.33 ft towards column line A. The latter two measurements taken from the geometric vertical axis of the jacket.

2.3 Description of Deck Support Frame

A Deck Support Frame has been designed in accordance with the Elf specification so that the Production Modules could be positioned above the crest level of the maximum predicted storm. The storm crest is predicted to be at EL(+)18,5 m giving an approximate air gap of 5,40 m between design wave crest and bottom flange of the Deck Support Frame.

The Deck Support Frame is essentially an extension of the Jacket, being rectangular in plan and having eight legs. The four outer legs are located directly onto the top of the Jacket while the centre legs are extensions of the centre piles which were cut to the required elevation after installation.

The top chord of the Deck Support Frame consists of a 9'-6" deep plate girder on column lines A and B with an 8' deep plate girder on column lines 1, 2, 3 and 4. Conductor framing is located between column lines 1 and 2 with a diamond brace between column lines 2 and 3 and a 'K' brace between column lines 3 and 4.

Access from the walkway at Jacket EL(+)20' is by stairway up legs A2 and B2. A walkway is provided around three outer sides of the Deck Support Frame on column lines A and B and column line 1. Two storage tanks are located between column lines 3 and 4 with the long axis running parallel to column lines A and B. A pump platform is positioned between the tanks with access into module D and along column line 3 to the support frame walkway.

Lifting eyes were provided on the Deck Support Frame so that it could be lifted into position as one unit or divided into two parts with a field splice between column lines 2 and 3. The support frame weighs approximately 1020 ST as a complete unit or two sections of 518.0 ST and 506.0 ST for the parts between column lines 1 and 2 and between column lines 3 and 4 respectively.

Pile guides were provided at each corner to assist during the erector installation phase in which drilling and placing of the insert piles occured from the top of the support frame.

2.4 Description of Production Modules

Four Production Modules were positioned on top of the Deck Support Frame after completion of the insert piles. The modules are numbered consecutively A, B, C and D. Module A is located above the conductor framing adjacent to column line 1 of the Jacket, Modules B and C are located between Jacket column line 2 and 3 while Module D is situated above Jacket column line 4. Each module has two levels, an upper or drilling deck and a cellar or production deck.

The decks are supported by two trusses running the length of each module and extending beyond the Jacket column line A and Jacket column line B. The trusses are fabricated from structural shapes. Secondary deck beams transmit load to deck girders which in turn transmit load to the nodes of the trusses.

Module A located above the conductor framing is essentially a well head module. The actual drilling contractor's drill rig is located on skid beams at drilling deck level.

Module A is separated from Module B by a dividing wall through which production piping is routed.

Modules B and C are essentially production modules. Module B will house the scrubbers to partially knock out liquids in the gas while module C is connected to the production risers which transmit the gas to the treament facilities in the Frigg Complex.

Module D is a quarters module. The drilling deck of Modules B, C and D are used to support the drilling contractor's drilling modules, equipment, pipe racks and quarters.

All production equipment is installed in the production deck of each module in the fabrication yard before loadout. Module D is fully equipped with galley, power supplies, toilets, recreation facilities, and sleeping accomodation for permanent staff.

Lifeboats are provided at the ends of Module D at cellar deck level. The ends of the cellar deck on Module C can be used for storage. Equipment can be transferred within Module C by a travelling crane mounted beneath the drilling deck.

Access to the Deck Support Frame can be made from Module C down Jacket column lines A and B and from Module A via stairways down Jacket column line 1. A stairway links the pump platform in the Deck Support Frame to the cellar deck of Module D.

Each module is designed to be supported at 4 points on the main trusses and were lifted into position utilising lifting eyes located between drilling deck girder and top chord of each module truss.

Field installed sections of deck plate connect each module at drilling and cellar deck level after positioning on the Deck Support Frame. A series of stabbing guides were utilized to help locate each module as close to its correct position as possible. Thereafter, each package was jacked into its final position.

2.5 Description of location

The Frigg Field is approximately midway between a line joining Stavanger in Norway and Lerwick in the Shetlands and spans both the British and Norwegian sectors.

The platform have been located appoximately 150 metres from the boundary in the Norwegian zone at the approximate geographical coordinates N 59° 53' 10", E 02° 04' 23" in a water depth of approximately 97.4 metres. The long axis of the platform was located N 40° W true with the conductors on the south east side of the platform.

A bathymetric chart of the area shows that DP2 is located on a 100-metre ridge running almost North and South through the Frigg Field. A 98-metre hill occurs to the North near the Norwegian/UK boundary and it is here that the Jacket has been located. The Jacket was launched in the Norwegian zone due East of location in a water depth of 104 metres.

Soil surveys indicate that the sea bed consists of fairly compact silty sand overlying postglacial clay strata throughout the Frigg Field. The clay in turn rests on deep silty sand layers extending to over 400 ft below sea bed level.

2.6 Fabrication yards

Much of the fabrication of the platform was subcontracted by the principal contractor, Union Industrielle et d'Entreprise, but assembley took place at two locations prior to loadout for Frigg. Both fabrication yards are located in the North of France.
Cherbourg on the Contentin peninsula bordering the English Channel (the Manche) and St. Wandrille near Rouen on the river Seine.

The Jacket complete with risers and buoyancy tanks was loaded out at Cherbourg along with the boat landing and barge bumpers and sea water casings. The barge bumpers and boat landings were, however, shipped loose for field installation.

The piles, Deck Support Frame and Production Modules were fabricated in St. Wandrille.

To assist in the setting of the primary piles, a Temporary Work Deck was located on the 4 centre piles. The Temporary Work Deck was fabricated by Monberg & Thorsen at Aalborg in Denmark.

3. BASIS FOR DESIGN, FABRICATION AND INSTALLATION

This chapter outlines the basic premises adopted for the design, inspection, fabrication, installation and transportation of the platform. These premises have essentially been utilized for the permanent structure (Jacket, Piles, Deck, Support Frame and Production Modules) as well as temporary structures (Temporary Work Deck and Modules).

3.1 Design premises

In the following are described the basic reports used for determining the design premises for the structure. The most essential parameters are listed in the following of this chapter. Reference is also made to the following specifications:

- ELF NORGE - FRIGG FIELD 4-061-4-70
  Technical Specification for Design of Drilling Platform No. 2 March 13, 1974.

This specification is attached in appendix 2 to this report.

3.1.1 Environmental conditions

The following reports were utilized for determining the environmental conditions adopted for the design of the platform:

a) IMCOS MARINE LIMITED
ELF-NORGE
Design Environmental Conditions - Frigg Field
December, 1971.

b) IMCOS MARINE LIMITED
ELF-NORGE
Design Environmental Conditions - Frigg Field - Part 1
February, 1972.

c) IMCOS MARINE LIMITED
ELF-NORGE
Design Environmental Conditions - Frigg Field - Part 2
February, 1972.

d) A.H. GLENN and ASSOCIATES
Meteorological - Oceanographic Conditions affecting Offshore Petroleum Operations at the Frigg Site and along the Pipeline Route from Frigg Site to Peterhead, Scotland.
December 24th, 1971.

e) B.C.E.M.
Meteo and Oceangraphic Study - Frigg Site
May 14th, 1972.

f) Department of Energy.
Guidance on the Design and Construction of Offshore Installations.
March 1974.

g) Det norske Veritas
Report no. 75-301; Long Term Distributions of Wind and Waves in the North and Norwegian Sea
October 12, 1975.

h) Vassdrags og Havnelaboratoriet ved Norges Tekniske Høgskole.
Current Conditions at Frigg Field.
March 12th, 1976.

In service condition.

Based on the above reports and on relevant Litterature and standard practice the following specific parameters were established for the Inservice Design Condition:

Environmental criteria for Inservice Design Condition.	Extreme	Operating
Return period	100 years	
Design waterdepth (MLW)	100,0 m	100,0 m
Tide	1,6 m	1,6 m
Design waveheight : 1)	29,0 m	14,2 m
Design wave period	15,0 sec.	12,0 sec.
Design wind speed up to + 10 m: 1)		
1 min. sustained	50,0 m/sec.	34,4 m/sec.
3 sec. gust	62,5 m/sec.	
Design current: at surface	1,35 m/sec.	1,00 m/sec.
30 m above seabed	0,70 m/sec.	0,58 m/sec.
at seabed	0,30 m/sec.	0,30 m/sec.
Drag coefficient, C_D: Diam. ≤ 60 inch	0,70	
: Diam. > 60 inch	0,75	
Inertia coefficient, C_M: Diam. ≤ 60 inch	1,70	
Diam. > 60 inch	2,00	
Marine growth elevation: + 3,5 m to -1,5 m (on radius)	6,5 cm	
- 1,5 m to -8,5 m	10,0 cm	
- 8,5 m to - 15 m	6,5 cm	
Temperatures; Air maximum	+ 32°C	
Air minimum	- 15°C	
Water maximum	+ 17°C	
Water minimum	+ 1°C	

1) See also next page.

The design wave period for the extreme conditions was originally varied, and based on an evaluation of total base shear and overturning moments versus wave period, the period of 15 sec. was finally selected.

The wind speed variation with height above sea surface was calculated according to the following formula:

$$V_Z = V_{10} \cdot \sqrt{\left(\frac{Z + 18}{Z + 60}\right) \cdot 2,5}$$

where; V_Z : Wind speed at a height Z above sea surface

V_{10} : Wind speed at a height 10 m above sea surface

Z : Height in metres above sea surface (Z to be larger than 10)

The above referenced criteria were assumed to act from any direction.

It should also be noted that measurements conducted after the installation of the platform revealed that the actual water depth, MLW, is 97,4 m.

Transportation.

The criteria adopted by the designer for transportation of the Jacket, Deck Support Frame and Production Modules as well as the temporary Work Deck and Living Quarters were as follows:

Transportation criteria		
Roll; Angle (amplitude)	:	Barge deck into water or barge bottom out of water (Still water level used as reference).
Period	:	Calculated as per classical naval architecture
Heave; Amplitude	:	60 % of barge draught
Period	:	Calculated as per classical naval achictecture.
Pitch; Angle (amplitude)	:	Barge deck into water or barge bottom out of water (Still water level used as reference)
Period	:	Calculated as per classical naval architecture.

For determining forces in the structures and tie-downs the above motions were combined and an appropriate wind force added.

The above criteria have probably been obtained from U.S. Salvage. For the transportation of these structures, however, Noble Denton and Associates were acting on behalf of the "Underwriters".

For the transportation of the Jacket and Deck Support Frame, DnV calculated the motion response of the barge/ structure assembly in order to confirm the criteria adopted by the designer.

Meterological data for the towing route based on a 10 year return period storm indicated a maximum wave height of 16,5 m and a wave period of 14 seconds. Based on this data the following criteria were adopted for the transporation of the Jacket and the Deck Support Frame:

Criteria adopted by DnV for Transportation	
- Return period	: 10 years
- Significant wave height	: 8,85 m
- Mean period	: 12 sec.

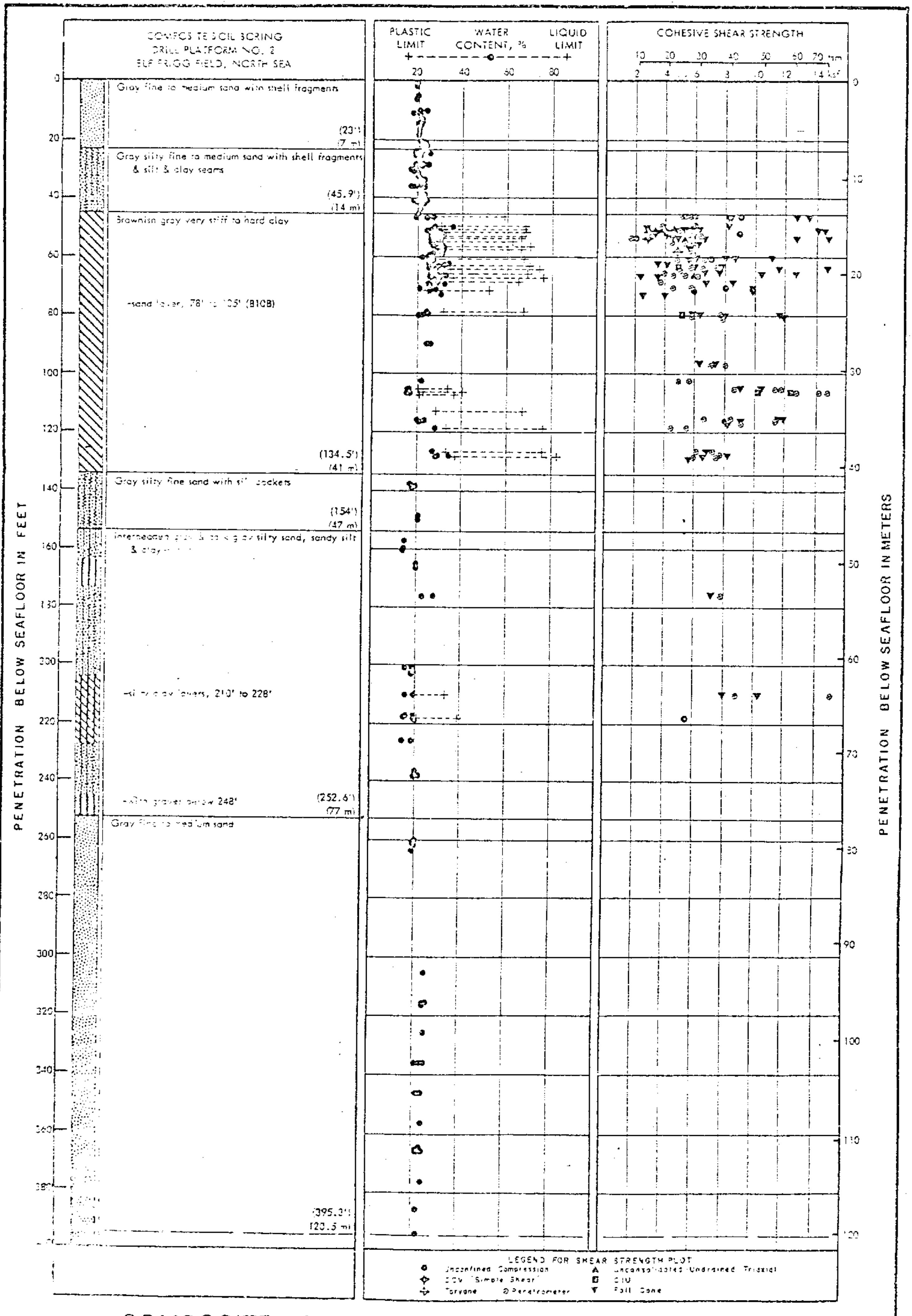
3.1.2 Soil Conditions

A great number of soil investigations have been carried out at the Frigg Field. However, only one investigation can be said to be directly applicable for the foundation of the DP2 structure.
This investigation was reported in:

- Norwegian Geotechnical Institute
  Report no. 73048-15
  Laboratory Test results and Soil Profile,
  DP2 location Frigg Field, North Sea.
  July 31, 1975.

- Fugro Cesco
  Report No. N1007/III
  Soil Investigation. Sampling and Well Logging
  Frigg Field
  May 16, 1975

The investigation consisted of one deep boring with soil sampling and well logging to a depth of 120.5m below the seabed.
The samples were examined and tested in order to classify the soil and measure strength parameters.
The investigation revealed a profile consisting of sand and clay in layers partly mixed with silt for the upper 75 m. Below this depth a uniform to medium sand layer was found. The figure on the next page defines the soil profile.



COMPOSITE LOG OF BORINGS AND TEST RESULTS

3.1.3 Design codes

The following codes were used as a basis for the design of the platform:

a) Det norske Veritas
Rules for the Design, Construction and Inspection of Fixed Offshore Structures, 1974.

b) American Petroleum Institute - API RP2A
API Recommended Practise for Planning. Designing and Constructing Fixed Offshore Platforms. Sixth edition 1975 and seventh edition 1976.

c) American Institute of Steel Construction
Manual of Steel Construction
Seventh edition, June 1973.

d) American Welding Society (AWS D1.1-72)
Structural Welding Code
September 15, 1972 with revision 1-73 and 2-74.

e) Det norske Veritas
Technical Notes for Fixed Offshore Structures.

It should be noted that the 1/3 increase in allowable stresses allowed by the above documents was not adopted for the design of this platform. The structural design was thus based on basic allowable stresses.

3.2. Materials and Fabrication Specifications

The following specifications were worked out for materials and fabrication of the structure.

- Elf Norge - Frigg Field - 1052 No. 3-145
  Fixed Offshore Structures
  Materials Specification
  Rev. 4 - December 1973.

- Elf Norge - Frigg Field - 1052 No. 3-155
  Fixed Offshore Structures
  Fabrication Specification
  Rev. 2 - February 1974

The following of this chapter briefly describes the criteria set forth in the above specifications.

3.2.1 Materials

Selection of high strength - and mild steels for plates, shapes and welded tubulars were based on the above referenced material specification.

The designations used on drawings and typical applications are listed in the following table:

(* Would satisfy primary structure steel.)

DESIGNATION	STEEL GRADE ACC. DIN 17 100	CLASSIFICATION ACC. TO DNV RULES	TYPICAL APPLICATION
HS 10	St 52-3N	Primary str.steel	Jacket braces and piles below el.(-)3m
HS 20	St 52-3N	Primary str.steel	Jacket and deck members and main module beams above el. (-)3m
SHS 10	St 52-3N	Special str.steel	Can sections and over-lapped and heavy wall stubs below el.(-)3m
SHS 20	St 52-3N	Special str.steel	Can sections and over-lapped and heavy wall stubs above el.(-)3m
ML 0	St 37-3U	Secondary str.steel	Conduc.frame members be-low el(-)3m&module beams
ML 20	St 37-3N	* Secondary str.steel	Con.frm.members above(-)3m

The following describes the most important criteria. All structural steel to be made according to DIN 17100 and modified as considered appropriate for the grade and application. The following main modifications and supplementary requirements to be applied for high strength steel, St 52-3N:

DESIGNATION	CHEMICAL COMPOSITION	MECHANICAL PROPERTIES	PLATE SOUNDNESS
HS 10	Al-killed, max 0.07 Al_{tot} max 1.60 Mn, max 0.35 Si, max 0.035 P & S CE max(1) 0.44 for t ≤ 30mm CE max 0.46 for t > 30mm	-σ0.2 ≤ 34 for t > 50mm -CV transverse: Min average 41 J Min single 34 J at (-)10°C. - Bend tests with former 3.5t for t > 50mm - Each mother plate/min. every 40 tons tested.	Level 3(3)
HS 20	As for HS 10 except that impact tests at -20°C.		
SHS 10	Max 0.18C, max 0.55 Si Max 1.60 Mn Max 0.020 P Max 0.015 S Max 0.03 Al_{tot} CE max: 0.43 for t≤30mm CE max: 0.45 for t>30mm	- σ0.2 ≤ 34 kp/mm^2 for t > 50mm - $CV_{transverse}$: Min average 41 J Min single 34 J at (-)10°C. - Shear area: Average 50% - Bend test with former 3.5t for t > 50mm - RA_z: Average (2) 30% or reduction of width in Z-direction min 35% of property in rolling direction	Level 2(3)
SHS 20	As for SHS 10 except that impact tests at -20°C.		

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

(2) RA_z = Reduction of area measured according to DnV recommendations, alternatively according to IIS/IIW doc: IXF - 74 - 18.

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

General delivery conditions to be 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 to be as follows:

- Plates/shapes: DIN 17100/St 37-2U, ASTM A 285 Gr. C or ASTM A 36.
- Tubes: DIN 1629/BL.3 - St 35, API 5L Gr B or ASTM A 53 Type E or S, grade B.

All bolts and nuts ½" and smaller to be of stainless type 316 while greater to conform to ASTM A-325 and to be hot dipped galvanized.

All shop rolled tubulars to be manufactured in accordance with API-2B: "Fabricated Structural steel pipe".

Tubulars to be made by cold rolling for $t \leq 50$mm. For greater thickness hot forming or hot rolling to be applied. Normalizing to be compulsory for hot rolling, and if micro structural modifications do occur during hot forming, mechanical properties to be retested in the final condition.

Welding of tubulars to be based on AWS D1.1-72 "Structural welding code" with following supplementary requirements:

- Charpy V-notch testing of base metal, weld metal and HAZ (O, f.l. 2&5 mm positions) to meet plate specification.
- Hardness/macro testing to meet max 300 HV5.
- Restricted qualified range to approximate ½" above and below test thickness.

Welding to be by the submerged arc. and/or the manual metal arc. processes. For the latter only low hydrogen electrodes to be selected.

Longitudinal welds to be visually examined and ultrasonic tested full length and interpreted according to ASME VIII/Div. 1. 200 mm of each end to be x-rayed. All x-rays to be interpreted to AWS D1.1-72
Procedures for NDT to be according to AWS D1.1-72
All tubulars made from St. 52-3N steels to be checked for hard spots according to API 5LX, i.e. max hardness in body of tube is not to exceed 327 HB.

Each finished tube to be dimensionally checked.

Production tests to be asked for at start-up and during regular production taking two tests during first 25 lengths and once every 50 lengths respectively.
The production tests consisted of tensile, bend, hardness and impact tests.

3.2.2 Fabrication

Assembleying and construction of the different structures were conducted in accordance with the above referenced fabrication specification which lists accepted reference codes/standards and gives detailed requirements to preparation of structural members, materials, fabrication tolerances, welding, inspection etc.

The following outlines the most important criteria:

Design of welded connections, joints and splices and fabrication tolerances: API-RP 2A, AWS D1, 1-72, AISC "Manual of Steel Construction".

The same conditions and requirements as specified for shop welding to be applied for assembleying and construction welding.
For welding of tubular joints special procedure tests required and these to be subject to visual inspection, and ultrasonic examination full length. Macrosections as well as Charpy-V notch tests (of weld Metal and HAZ) and hardness tests to be taken at four specified positions (12, 3, 6 and 9 o'clock). These sample joints had to meet the project material specification.

Welders generally to be qualified as per AWS D1.1-72 except that backing plates not permitted. For welding of tubular joints (nodes) the welders also to make a prototype joint in actual positions, and this prototype to be examined visually and by macrosections.

Post weld heat treatment only to be carried when instructed.

All full penetration welds to be examined 100% by ultrasonic or x-ray. Additionally other welds to be inspected 10% by one of these methods. Procedures and interpretation of NDT-tests to be according to standards as specified for shop-welding. Magnetic particle testing to be carried out to the discretion of the fabricator's and owner's representatives.

The above listed and described specifications which are reviced versions of equivalent specifications developped for earlier Frigg platforms were reviewed by DnV and found acceptable.

3.3 Specifications for Grouting- and Flooding Systems.

The following specifications, pertinent to the design, fabrication and testing of grouting and flooding systems, were worked out for this platform:

a) ELF NORGE - FRIGG FIELD
Fixed offshore structures
Specification for the Flooding and Grouting Lines
(Not dated).

b) Specification No. 2144-701
Jacket Ballasting Valves and Actuators
Rev.1, April 30, 1975

c) Specification No. 2144-702
Jacket Vent Valves
Rev. 1, May 5, 1975.

d) Specification No. 2144-703/1
Hydraulic Hose Bundles & Fittings
Rev. 1, March 5, 1975.

e) Specification NO. 2144-704.
Cleaning, Filling and Air Venting Nylon Hydraulic Lines
Rev.0, April 30, 1975

f) Specification No. 2144-705/0
Inflating Hose for Pile Seals
Rev. 0, May 30, 1975.

g) Specification No. 2144-706
Pressure Testing swage Couples Multitube Bundles
Rev. 1, February 1976.

h) Specification No. 2144-707
Flooding-Piping Specification EL.
Rev. 1, February 1976.

i) Specification No. 2144-708
Grouting-Piping Specification Class ED
Rev. 1, February, 1976.

j) Specification NO. 2144-709
loadout of Jacket
Rev. 0, March 2, 1976.

The above specifications which all pertains to the transportation and/or installation of the structure were received and reviewed by DnV. Some comments were given. These mainly affected the testing of the equipment or the criteria adopted relative to the design of the structure.

3.4 Corrosion Protection Specifications.

The following specifications pertaining to coating and painting were used:

a) Elf RE. Equipment
Standard Specification SG. P07
Coating for Marine Structures
Rev.0, September 1972

b) Elf Norge - Frigg Field - 1052 No3-169
Painting for steel structures
Rev. 1, March 1974.

c) Elf Norge - 1052 No. 5-498.
Coating Systems for Steel Structures of Phase II
Rev. 1 - October 1975.

The design criteria for the cathodic protection system (sacrificial anodes used) have been given in Elf letter No. 611 of August 4, 1975.

The criteria are given in the following table

Corrosion Protection Criteria	
- Design life	: 20 years
- Current density for bare steel in sea water	:130 MA/m^2
- Current density for bare steel in wind	: 20 MA/m^2
- Current density for coated steel in splash zone	: 50 MA/m^2
- Aluminium anodes of type ALAP of Nakagawa with material composition;	
- 3.25±0.3% Zn	
- 1.8 ±0.2% Mg	
-0.02 ±0.08% In	
-0.007±0.003%Sn	
- Al remainder	
Efficiency greater than 80% or greater than 2391 Ah/kg	

The above specifications and design criteria were evaluated by DnV and found satisfactory.

...

...

4.2. Design Conditions and Loads

4.2.1. General

The follwoing basic design conditions were investigated for the structure or parts thereof (as relevant):

- Load-out onto barge
- Transportation from construction yard to Frigg Field
- Launching and positioning (Jacket)
- Lifting from barge onto Jacket (Deck Support Frame and Modules)
- Stability of platform during various stages of pile installation.
- In service storm condition - drilling phase.

The loads imposed on the structure or parts thereof during these load conditions include (as relevant)

- Dead/gravity loads
- Live loads
- Buoyancy
- Wave-, current- and/or wind loads
- Inertia loads

The loads imposed during each of the above mentioned design conditions were determined by the designer. Methods of analyses and check calculations performed by DnV are explained in the following of this chapter (4,2).

4.2.2. Jacket Load-out

In the load out condition it was checked that the curvature and foundation of the skid-ways were such that excessive support loads (both with respect to the Jacket strength and skid-way support capability) would not occur.
This was essentially done by checking that the curvature of the skid-ways was within acceptable radius tolerances, and the skid-ways' supports' capability to withstand the imposed loads were checked by an evaluation of the bearing capacity of the soil and the supports themselves.
In order to arrive at the allowable radius/curvature of the skid-ways, the Jacket strength was checked for a range of skid-way curvatures by forcing it into these curvatures - i.e. apply the curvature of the skid-ways as boundary conditions in the form of node displacements.
Afterwards the actual curvature of the skid-ways was measured and found to be within acceptable tolerances.

It was also for the transfer from yard to barge checked that the ballasting procedure to be adopted was such that the barge progressively took its share of the Jacket weight as the Jacket was moved onto the barge. It was also checked that account had been taken for the local tide variations. Furthermore, it was checked that the equipment to be used for the skidding of the Jacket and ballasting of the barge had adequate capacity.

Also the Deck Support Frame and the Production Modules were skidded onto the transportation barges in a similar manner to that adopted for the Jacket. DnV did also briefly review the procedures for these operations. However, since the size and weight of these units were much smaller than those of the Jacket it was not found necessary to go into so much detail as that done for the Jacket load out.

4.2.3. Transportation

For the transportation condiiton the designer determined the loads imposed on the structures by applying the criteria outlined in chapter 3.1.1 above. Based on the natural periods for roll, heave and pitch and the associated barge motions the forces were determined and structural strength of the units and tie-downs was checked.
As explained in the foregoing DnV did not entirely agree to the calculation procedure adopted by the designer. Independent motion response analyses for the barge - Jacket and barge - Deck Support Frame assembleys were therefore conducted.

The results obtained from these independant calculations showed forces generally somewhat smaller than those determined by the designer. Based on this the calculations performed by the designer for the Production Modules and the Piles were reviewed and accepted.

4.2.4. Jacket Launching and Positioning.

To obtain the loads imposed on the Jacket during launching both theoretical methods and model tests were employed.
The theoretical method essentially compriced of a step-by-step procedure where the Jacket was moved along the barge skid-ways and rocker arms and the balancing forces (self weight verses buoyancy) were calculated by employing a solely static method i.e. the effects of inertia loads and wave drag forces were neglected.
The model tests were conducted at NSMB Wageningen the Netherlands.
Models of the Jacket as well as the barge were made and a series of launching tests were run both in calm water and also in irregular seas.
The results obtained from these model tests agreed very closely with those obtained from the theoretical method-despite the fact that the latter was a purely static method.
DnV reviewed the calculations performed by the designer as well as the reports from the model tests.
A DnV representative did also attend some of the model tests.

For the positioning of the Jacket detailed flotation studies were undertaken by the designer. The results were also confirmed by model tests carried out at NSMB at the same time as the launching tests.
From these tests the flotation characteristics of the Jacket were determined and a detailed up-end procedure established.
Separate strength calculations for the Jacket in the up-end conditions were not found necessary. The loads imposed were generally negligible and the Jacket members were for the in service condition checked for a hydrostatic pressure also covering this condition. Local strength of towing pad-eyes and their supports were, however, checked. Design criteria for these were generally based on the strength of the towing lines to be used.

4.2.5. Various Stages of Pile Driving

The loads on the Jacket were calculated for the various phases from an unpiled structure until the Jacket was firmly attached to the sea bottom. The results from these studies were essentially utilized to determine the most desirable installation sequence including pile installation sequence, i.e. when to install temporary and permanent Deck Support Frames and Modules, removal of buoyancy tanks etc. The main goal was to obtain as stable a platform as possible as soon as possible.

It was generally not found necessary to conduct any major strength analysis for these load conditions. The pile strength was, however, checked into.

4.2.6 Lifting

The production Modules and the Deck Support Frames (Permanent and Temporary) were lifted from the barge onto the Jacket.
For the lifting condition the structural strength of the units were checked for an impact factor of 2.0 after reasonable account had been taken for uncertainty in weight estimation.
The following weights were determined for the permanent units.

Unit	Calculated	Measured (dry)
Module A	524 t	420 t
Module B	536 t	640 t
Module C	794 t	520 t
Module D	787 t	690 t
Deck Support Frame	1020 t	

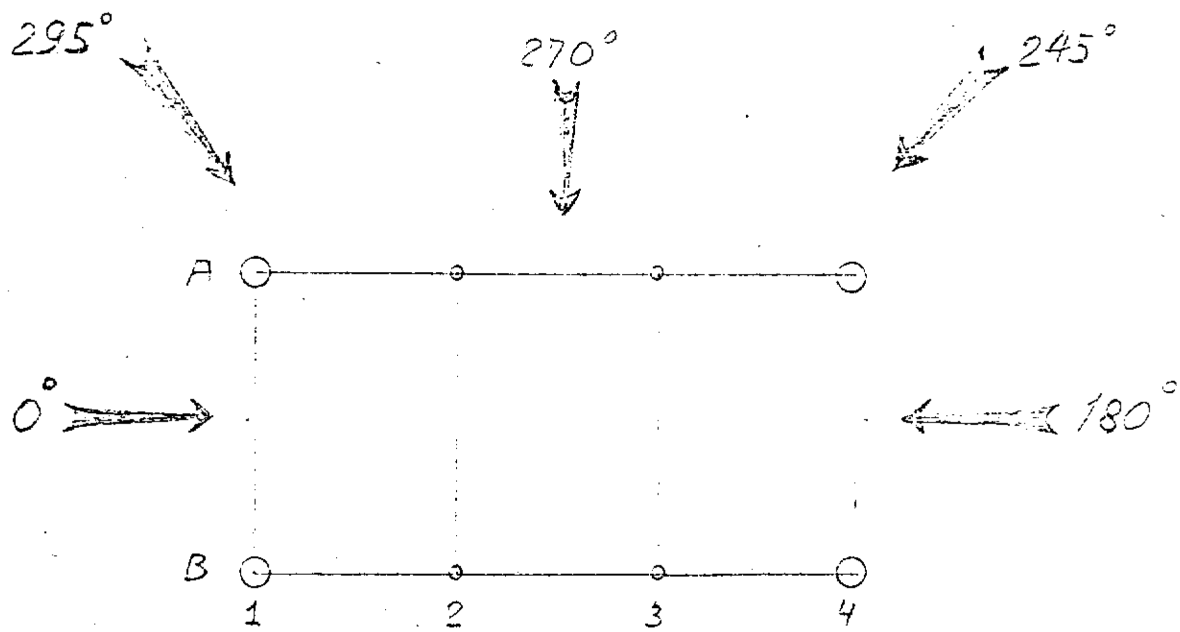
Weights were measured by load indicater on the derrick barge and these should not be considered as exact weights.

4.2.7 In service Condition

For the in service condition the structure and parts of it was designed for the 100 year environmental forces.

Generally only the drilling phase was investigated as this phase was most critical.
No reduction in live loads was considered in the extreme, 100 year, condition.

The Jacket with Deck Support Frame was checked statically for the wave directions given on the following Figure:



Combined waves, current and wind were assumed to act from each of these directions. There was not considered any directional distribution of wave heights, wind speed or current speed.

The design parameters referenced in chapter 3.1.1 were thus assumed to act simultaneously from all the above directions.
In the global analysis the deck modules' weights were input as point loads on the Deck Support Frame.
The combined wave- and current forces were calculated by vectorial summation of current- and water particle velocities. The forces on the individual members were obtained by applying the Morrison's formula:

$$\underline{F} = \tfrac{1}{2}\rho C_D D \dot{x}^2 \, d\xi + \rho c_m \frac{D^2}{4} \ddot{x} \, d\xi$$

where, $\dot{x}$: Combined water particle - and current velocity

$\ddot{x}$: Water particle accelleration

D : Member diameter

ρ : Mass density of water

$d\xi$: Differential member length projected in a plane normal to velocity/acceleration vector

The influence of appurtenences such as boat landings, barge bumpers, anodes, risers etc. was accounted for both with respect to gravity/buoyancy and environmental loads.
The dynamic response analysis revealed a dynamic amplification of approximately 10% in the extreme condition. However, since no increase in allowable stresses was allowed in this condition, it was not found necessary to perform an elaborate dynamic response analysis. For the same reason, it was not found necessary to investigate an operating condition with a reduced wave height.
In the in service condition also slamming loads and fatigue were investigated.

The slamming calculations generally followed the principles outlined in DnV's Technical Note A6/2.

Fatigue was checked by considering waves in two direction, transverse and longitudinal.
From the oceanographic data available it was assumed that 49 percent of the waves would come in the transverse direction and with 51 percent in the longitudinal direction.
A total number of $1.09 \cdot 10^8$ waves were taken into account in the fatigue analysis.

Also the hydrostatic pressure loads on the individual members of the structure were taken into account. This was essentially done by considering the members located at its normal position vis-avis the water line. It was however, also checked that the members could withstand a waterpressure equivalent to 125 percent of the water depth at the location.

For the design of riser protection the criteria set forth by NPD was utilized - i.e. an impact energy of 50 tm applied in any direction in the splash zone area.

The loads imposed on the structure in the in service condition were generally determined by the designer. It should be noted that prior to this project the designer had carried out wave load calculations on a test structure specified by DnV - Veritas Tower No. 1.
The results of these check calculations were that the designers wave load program was found acceptable.

Some spot checks were also conducted by DnV in order to ascertain that the loads determined on the DP2 platform were acceptable.

4.3. Foundation Design

Based upon the data obtained from the soil investigations McClelland Engineers Inc. developed engineering design parameters and data for the pile design. These data have been reported in.

-McClelland Engineers Inc. - Report No. 175-257-2
Foundation Analyses, ELF NORGE
Drill Platform No. 2 Frigg Field, North Sea
October 15 1976.

This report defines the ultimate pile capacity curves for axial loads and P-Y curves representing the soil resistance against lateral loads.

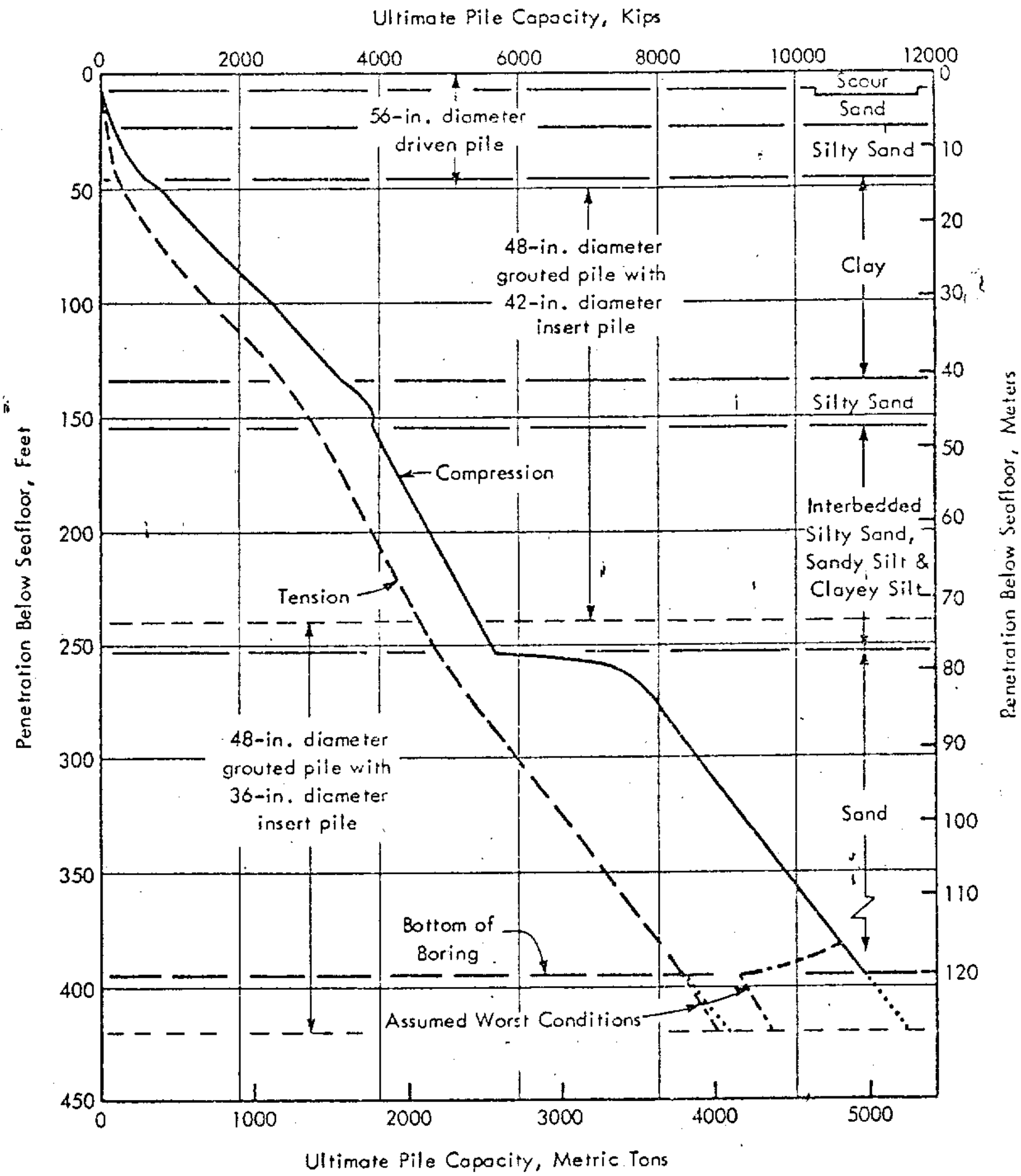
Figures No. 4.3.1. through 4.3.3. below gives the pile capacity curves for corner- and center piles.

Table no. 4.3.1. gives the P-Y curves for the corner piles.

Due to the group effect a "Y-factor" of 4.4 was used in calculating the lateral behaveour of the corner pile groups. (Y-factor to be multiplied with Y-values in P-Y diagrams to obtain P-Y curves for pile group.)

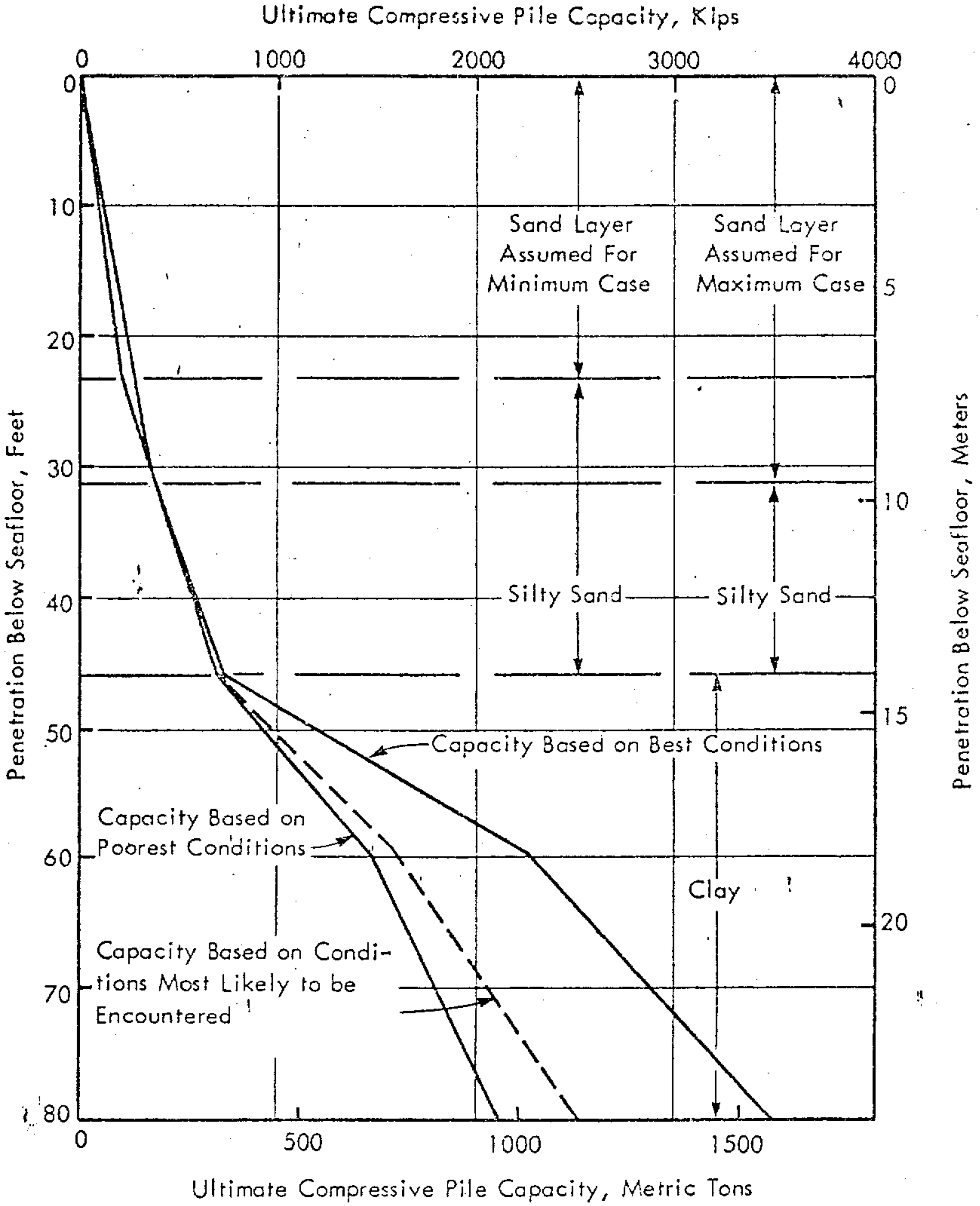
The Y-factor was obtained by an iteration process and McClelland Engineers Inc. provided the factor based on data furnished by the designer.

The axial capacity of the pile groups was set equal to the sum of the single pile capacities.



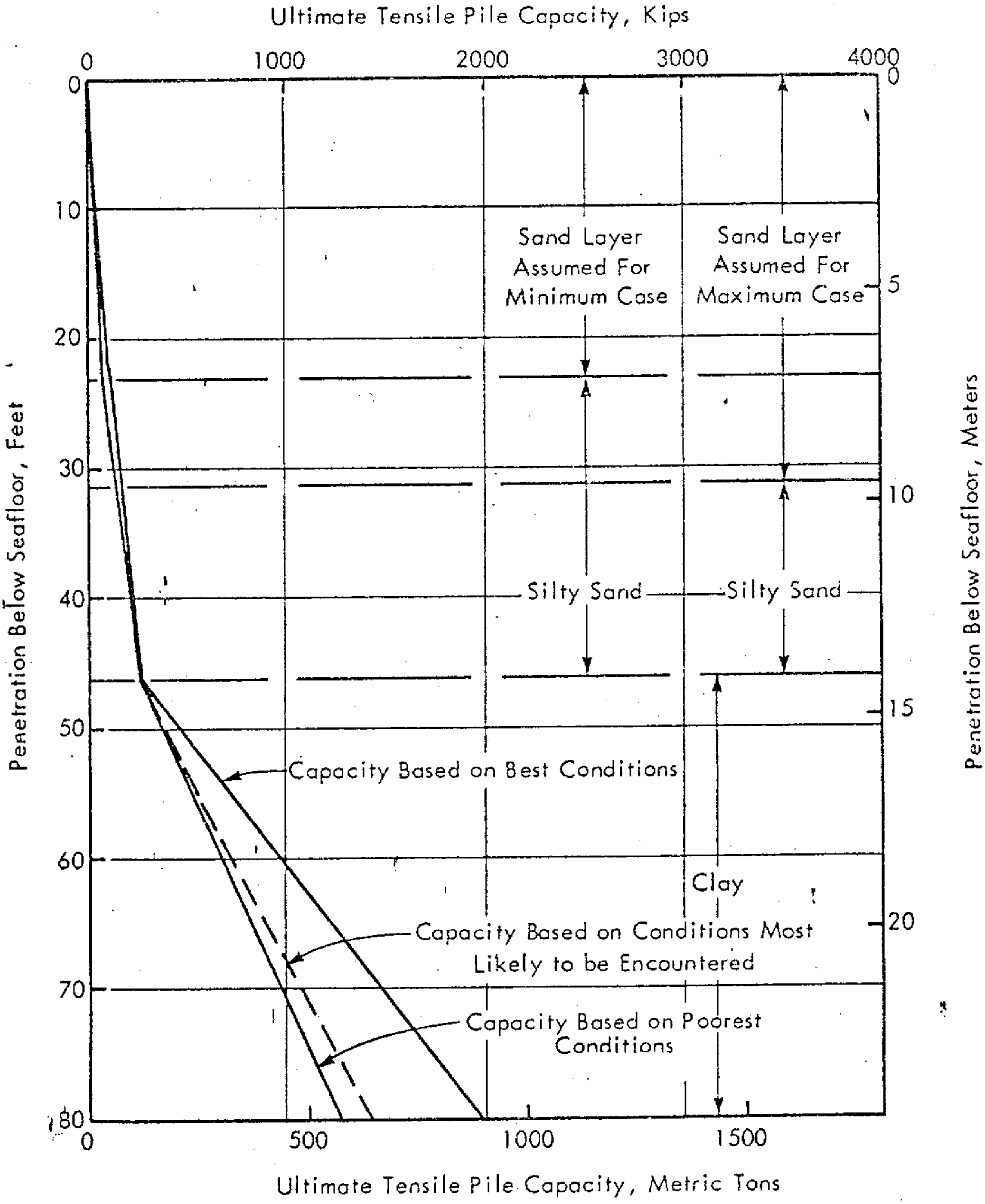
ULTIMATE PILE CAPACITY CURVES
Drill Platform No. 2
Frigg Field, North Sea

Fig. 4.3.1.



ULTIMATE COMPRESSIVE PILE CAPACITY CURVES
56-in. Diameter Driven Pipe Piles
Drill Platform No. 2
Frigg Field, North Sea

Fig.4.3.2

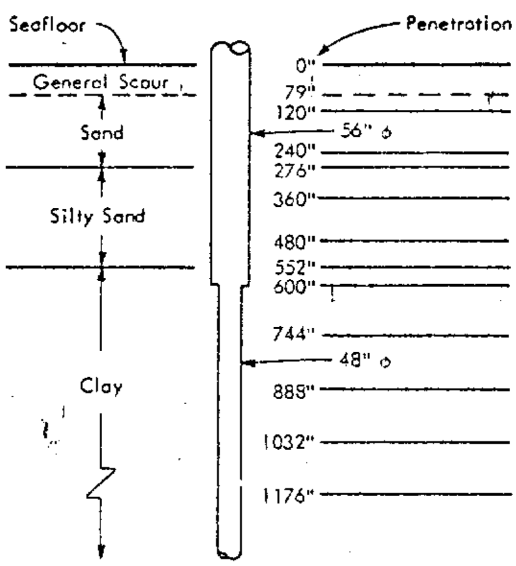


ULTIMATE TENSILE PILE CAPACITY CURVES
56-in. Diameter Driven Pipe Piles
Drill Platform No. 2
Frigg Field, North Sea

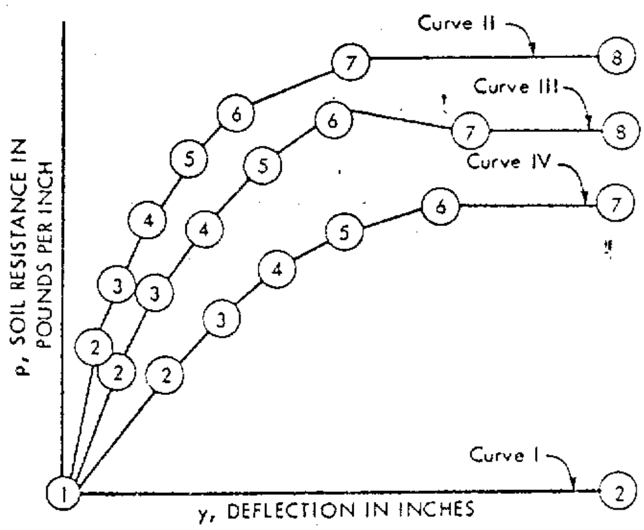
Fig. 4.3.3

48&56-IN. DIAMETER PILE

Penetration, Inches	Curve	Coordinates of Curve Points p_1	y_1	p_2	y_2	p_3	y_3	p_4	y_4	p_5	y_5	p_6	y_6	p_7	y_7	p_8	y_8	p_9	y_9
56-In.																			
0	I	0	0	0	20														
79	I	0	0	0	20														
120	II	0	0	244	0.15	273	0.24	310	0.41	340	0.60	377	0.93	488	2.1	488	20		
240	II	0	0	2289	0.36	2390	0.42	2561	0.55	2718	0.69	2941	0.93	3894	2.1	3894	20		
276	II	0	0	2260	0.29	2464	0.36	2807	0.50	3120	0.66	3568	0.93	5295	2.1	5295	20		
277	II	0	0	1210	0.15	1440	0.24	1778	0.41	2059	0.60	2437	0.93	3617	2.1	3617	20		
360	II	0	0	1636	0.15	2050	0.23	2683	0.41	3229	0.60	3991	0.93	6386	2.1	6386	20		
480	II	0	0	3307	0.16	4040	0.25	5186	0.42	6189	0.61	7599	0.93	12158	2.1	12158	20		
552	II	0	0	5106	0.22	5937	0.30	7303	0.46	8538	0.63	10310	0.93	16496	2.1	16496	20		
553	III	0	0	2104	0.02	2856	0.05	3876	0.11	5261	0.23	7141	0.70	6570	10.5	6570	21		
600	IV	0	0	2328	0.02	3160	0.05	4288	0.11	5821	0.28	7900	0.70	7900	21				
48-In.																			
601	IV	0	0	2043	0.02	2773	0.04	3763	0.10	5108	0.24	6933	0.60	6933	18				
744	IV	0	0	2461	0.02	3340	0.04	4532	0.10	6079	0.24	8350	0.60	8350	18				
888	IV	0	0	2702	0.02	3667	0.04	4976	0.10	6754	0.24	9167	0.60	9167	18				
1032	IV	0	0	3124	0.02	4240	0.04	5754	0.10	7810	0.24	10600	0.60	10600	18				
1176+	IV	0	0	3620	0.02	4914	0.04	6668	0.10	9051	0.24	12284	0.60	12284	18				



STRATIGRAPHY ASSUMED FOR P-Y DATA



TYPICAL CURVES

P-Y DATA

DRILL PLATFORM NO. 2
56-IN. DIAMETER DRIVEN PILE AND
48-IN. DIAMETER GROUTED PILE

Table 4.3.1

The pile design was made by McDermott Hudson based upon the soil data provided by McClelland.

The pile configuration consists of a four pile group in each corner and a single pile through each center leg.

The corner piles are a combination of a driven- and a drilled and grouted piles.
These piles are 117m long (penetration) and grouted to pile sleeves which are connected to the Jacket leg.

The centre piles are ordinary driven piles 21 m long (penetration) and grouted directly to the Jacket legs.

The pile make-up is shown on drawing nos. ELN 2144, sheet 127 through 130.

DnV evaluated the soil investigation and pile design.

The soil investigation was found to be minimal with respect to the type of platform involved. However, concervative design assumptions, good correlation between platform as set and boring location and a review of installation records revealed a sound design.

The designed pile penetrations were found sufficient to carry the axial loads with a f.o.s. of at least 1.5 for extreme conditions.
The piles were also found to have the strength to withstand the axial and horizontal forces.

The compability between rotations and deflections at top of pile, bottom of Jacket was also found acceptable. This deflection is calculated to be in the magnitude of 6 - 7 cm in extreme condition. A general scour of 2m was incorporated in the design.

4.4. Structural strength

4.4.1. General

The structural strength of the platform was for the above listed load conditions and loads (chapter 4.2), checked for the following modes of failure.

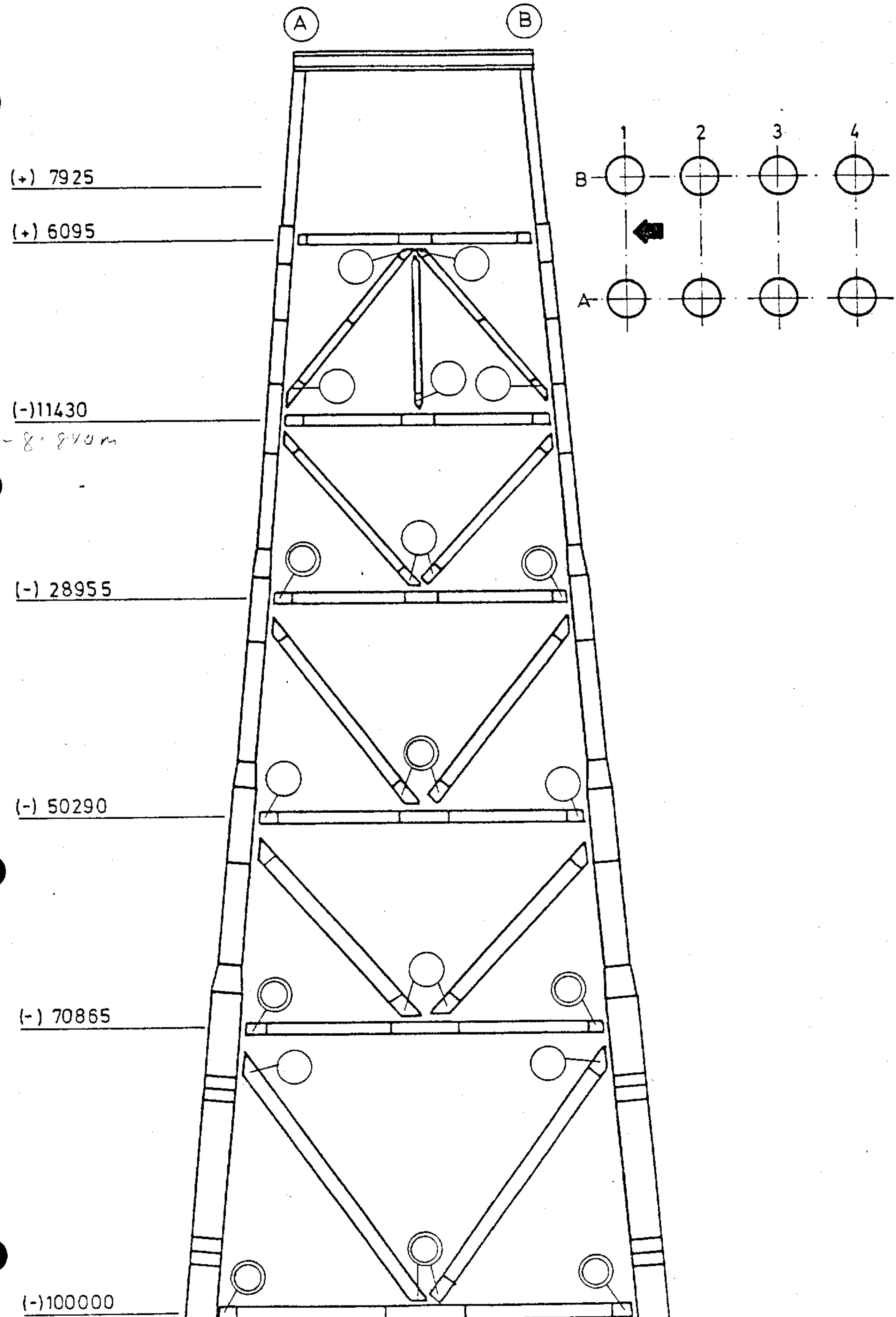
- Excessive yielding (stress check)
- Instability (Overall member buckling as well as buckling due to external water pressure)
- Fatigue
- Brittle fracture

The structural response of the Jacket with piles and Deck Support Frame was obtained by using the STRUDL computer program. The program used which is a special version devellopped for jacket type structures, has special preprosessors for generating the platform model and wave loads. Also a post prosessor for stress check, buckling check and joint check is connected to the program.
This program did not at the time the calculations were made have the possibility of performing fatigue analysis. This analysis was therefore run on another program by BCEM in Pau, France.
The BCEM program is such that it may accept input prepared for other programs - thus BCEM received prepared data input from McDermott Hudson Engineering

As mentioned in the foregoing the Jacket with piles and Deck Support Frame was analysed as one unit. Production and Drilling Modules' dead - and live load as well as wind loads were input to the global analysis as point loads at the appropriate support points.
The effects of overturning of modules due to wind were taken into account.

For all load conditions investigated with the exception of transportation and fatigue,only static methods were applied.
For the fatigue analysis the effects of dynamic respon-ces were accounted for. However, from this analysis it was concluded that the dynamic effects in the 100 year storm case was not significant - approximately 10 percent increase in horizontal deflections. Since the 1/3 increase in allowable stresses not was taken into account for this platform it was consluded that it was not necessary to perform any more elaborate dynamic analysis for the platform.
The natural period for the structure was determined to approximately 1.8 sec. in the weakest direction.

The modelling of the structure is shown is Appendix 4 to this report. This modelling was checked in detail by DnV and found adequate for the global ana-lyses. As can be seen from the figures in Appendix 4, the conductor frames were modelled as a simple double-x-brace. The most heavily loaded conductor frames (the upper 4) were, however, analysed separately as plane frames using a detailed model. Boundary conditions for these analyses were taken from the global calculations.



FRIGG DP2 COL. LINE 1

FIG. 4.3

The Production Modules were analysed in separate 3-dimensional computer analyses. Upon DnV's request also the effects of support deformations were investigated. This was done by taking deflections from the global analyses and input these to the strength calculations for one module. Module B was selected for this study. The results from this study revealed very small additional stresses in the module.

The following chapters (4.4.2. through 4.4.5) describes the calculations performed in more depth. Also the control work carried out by DnV is more fully described.

4.4.2. Excessive Yielding

The "allowable stress" method was used for designing the platform. Allowable stresses were taken in accordance with the design codes listed in chapter 3.1.3 above, i.e.:

- $\frac{\sigma a}{\sigma Aa} + \frac{\sigma b}{\sigma Ba} \leq 1$ and/or

- Von Mises equivalent stress $\leq$ 0,8 σyield

  where; σa : Actual axial stress
  σAa : Allowable axial stress = 0.6 σyield
  σb : Actual bending stress
  σBa : Allowable bending stress = 0.66 σyield

For the design of tubular joints both the criteria of API RP2A and DnV were adopted.

A number of the tubular joints in the Jacket are, however, stiffened by internal ring - and/or longitudinal (parallell to axis of the chord section) stiffeners.

Such joints were designed based on allowable stresses alone taking no account of any "punching shear". Roark's Formulae for Stress and Strain (Case with two point loads acting on a ring) were used. In the case of several brace, intersecting the same ring stiffener(s) the superposition principle was applied to determine forces and moments in the ring stiffeners and the chord plate.

For the case with combined axial- and bending stress in the intersecting brace, an equivalent axial force was calculated by taking the axial-plus bending stress multiplied with the cross sectional area of the brace.

In the design of stiffened tubular joints the stiffeners were designed to take all the loads from the braces - i.e. as mentioned above no effect from "punching shear" was assumed. When calculating the effective cross-section of the ring stiffener, however, an effective width of the chord section was accounted for as flange. For calculation of stresses in the ring stiffeners and also the flange, curved beam theory was applied. All joints stiffened by ring stiffeners were checked by DnV utilizing an inhouse computer program specially devellopped for analysing such joints.

A number of the stiffened joints have longitudinal stiffeners (parallell to the axis of the chord section) in addition to the ring stiffeners. These longitudinal stiffeners were designed by splitting the joints at convenient sections and assuming equilibrium of forces.

During the "load-out" and "launching" conditions, the transfer of forces in the tubular joints in the legs resting on the skid-ways or rocker arms will be different and more critical as compared to a traditional tubular joint. This is because the forces from the braces will be transferred through the chord sections and launch craddle into the skid-ways/ rocker arm and not as in the traditional tubular joint where the brace forces are taken up as shear in the chord and transferred to another brace.

The former case is generally considered more critical and therefore for two particular joints considered to be highly stressed in the launching case, a detailed computer analysis of the joints in question was carried out by DnV. The results of this study being that the joints were reinforced.

Appendix 5 defines the most heavily stressed members and joints for the inservice condition.
It should be noted that symmetry has been considered when working out the figures in Appendix 5.
Thus if one member is highly stressed also the symmetrical member(s) are marked on the drawings in Appendix 5.

4.4.3 Instability

For overall member buckling the criteria of AISC, Manual of Steel Construction was adopted, i.e.:

$$\frac{\sigma_a}{\sigma_{Acr}} + \frac{Cm \cdot \sigma_b}{(1-\frac{\sigma_a}{\sigma_e}) \cdot \sigma_{Ba}} \leq 1$$

where; σ_a : actual axial stress

σ_{Acr}: critical axial stress - see below

Cm : a coefficient ∿ 0.85

σ_e : Euler buckling stress = $\frac{12\Pi^2 \cdot E}{23 \cdot (kl/r)^2}$

σ_b : actual bending stress

σ_{Ba}: allowable bending stress = 0.66 σ_{yield}

The critical axial stress (σ_{Acr}) was also taken in accordance with the above referenced code:

$$\sigma_{Acr} = \frac{(1 - \frac{1}{2}(\frac{Kl}{rc})^2)\ \sigma_{yield}}{(\frac{5}{3} + \frac{3}{8}(\frac{Kl}{rc}) - \frac{1}{8} \cdot (\frac{Kl}{rc})^3)}$$

where; Kl/r : slenderness ratio

c : $\frac{2\Pi^2\ E}{\sigma_{yield}}$

Some minor variations to the above interaction formula were also considered in accordance with Section 1.5.1. of the above referenced code.

Member buckling or collaps due to external water pressure was checked in accordance with API RP2A as well as DnV rules.
These criteria were used for all Jacket members and buoyancy tanks for the launching/up-end cases assuming all members subjected to on external pressure equivalent to 125 percent o f the actual water depth.

The API RP2A criteria was also used for the Jacket members located at their correct position relative to the water line and combined with the stresses acting in the members in the extreme environmental condition.

4.4.4 Fatigue

Fatigue was generally checked according to the requirements of AWS D1.1.-72. The following S-N curves (stress versus number of cycles to failure) were used:

D': For non-overlapped and non-stiffened tubular joints (Brace axial-plus bending stress.)

E': For overlapped or stiffened tubular joints (Brace axial-plus bending stress)

K : For simple K-type tubular joints (Chord punching shear)

T : For simple T and Y-type tubular joints (Chord punching shear)

For the Jacket and Deck Support Frame, the fatigue analysis was carried out by BCEM, Pau, France. Geometry input to BCEM computer program was obtained from McDermott-Hudson Engineering.

Fatigue was calculated by considering two wave directions; longitudinal and transverse.

The long term distribution of stress range for each of these two directions was obtained by calculating the stress range for two different wave heighths.
H_1 = 29m and H_2 = 17.4m in each direction.

Stress range for each wave height was obtained by calculating the appropriate stress (axial-plus bending stress and punching shear in chord) for the two positions in the wave causing maximum and minimum wave forces. Stress range was then calculated by subtracting (with correct use of signs) these two stresses.

Having calculated the stress range for the above two wave heights in two directions the long term distribution of stress range was obtained through the following formulae:

$$\Delta\sigma = \Delta\sigma_1 \left(1 - \frac{L_g N}{L_g N_o}\right)^{\gamma} = \Delta\sigma_1 \left(\frac{H}{H_1}\right)^{\gamma}$$

$$\gamma = \frac{\ln\left(\frac{\Delta\sigma_1}{\Delta\sigma_2}\right)}{\ln\left(\frac{H_1}{H_2}\right)}$$

where

$\Delta\sigma_1$: Stress range for a wave height H_1

$\Delta\sigma_2$: Stress range for a wave height H_2

H : Any wave height

H_1 : Wave height equal to 17.4m

H_2 : Wave height equal to 29m

N : Number of times the wave height H is exceeded in wave direction considered.

N_o : Total number of waves in wave direction considered.

The total number of waves assumed were 1.09×10^8 with 49% in the transverse direction and 51% in the longitudinal direction of the platform.

The effects of dynamic amplification were also accounted for when determining the long term distribution of stress range. No current was considered.

To obtain the cumulative damage ratio, the Palmgren/Miner linear damage hypothesis was applied, i.e. damage ratio η, equals:

$$\eta = \sum_{i=1}^{K} \frac{n_{io}}{N_{io}} + \sum_{i=1}^{K} \frac{n_{i90}}{N_{i90}} \leq 0.5$$

where;

K : number of stress blocks

n_{io} and n_{i90}: number of stress cycles within block no. i for 0 and 90° direction waves respectively.

N_{io} and N_{i90}: number of cycles to failure for stress range $\Delta\sigma_{io}$ and $\Delta\sigma_{i90}$ respectively

$\Delta\sigma_{io}$ and $\Delta\sigma_{i90}$: constant stress range within block no. i for 0 and 90° direction waves respectively.

Generally, number of stress cycles were taken equal to number of wave cycles. However, for the horizontal member in the splash zone area also slamming effects were taken into account.

Generally, the damage ratio, η , was required to be below 0.5. For some joints, however, the calculated damage ratio turned out to be larger than 0.5 with a small margin. These joints were ground. DnV reviewed the design calculation provided by BCEM. Some check calculations were carried out and the fatigue calculations as well as the procedure adopted were found acceptable.

The highest stressed joints in fatigue are shown in Appendix 5 to this report.

For special investigations with respect to fatigue reference is also made to Chapter 4.4.5. below.

4.4.5 Brittle Fracture and Crack Propagation

The DP2 Jacket contains a number of members having very great wall thicknesses, up to 86mm.

During the design review and the fabrication of the structure it was realized that special investigation should be conducted with respect to the welding of these heavy thickness members. It was believed that such weldments might imply potential brittle fracture problems.

A number of welding procedure tests were therefore required and COD testing was conducted on the weldments produced. The COD tests were conducted by DnV in Oslo.

Based on the results of these tests, as well as on published data on the steel and deposited material in question, critical crack sizes and a_{max} values were calculated.

As the critical crack sizes turned out to be comparatively small, it was also decided to carry out crack propagation analyses. The crack propagation analyses were based on a series of assumed initial crack sizes.

For further details reference is made to separate DnV reports on this subject.

The conclusions drawn from these studies were (for highly stressed areas):

- careful grinding and post weld heat treatment (PWHT) of weldments involving plate thicknesses above 50mm.

- in cases where PWHT was impossible (due to very large assembleys) very careful grinding and rigorous NDT inspection by U.S. and M.P.

It was furthermore agreed and required that these joints be subject to close inspection in the future in order to detect and apply corrective action at a very early stage.

It should be mentioned that the reports issued in connection with this matter may be used as a guidance on determining the effects with respect to critical crack sizes and crack growth for any cracks found in the future.

This matter is considered essential since it applies to some of the most important members in the jacket. It is therefore imparative that these joints which are shown in Appendix 5 be subject to rigorous inspection in the future.

In connection with the investigations described previously in this chapter, as well as with general weld groove design in tubular joints having large thicknesses, DnV did also carry out a photoelastic analysis of a tubular joint. The aim of this study was to investigate the influence of weld groove and weld profile on the local stresses in a tubular joint.

The study therefore incorporated several weld groove/ profile alternatives.

One major concern was the problem of single sided welding of such joints. In most cases it was impossible to make a full penetration weld from the outside only. The concern thus was: Is it possible that a partial penetration welded tubular joint may have large stresses in the root region? Such a situation might then possibly lead to cracks starting from the root, i.e. cracks which could not be expected to be detected before having propagated through the weld.

Based on the studies above it was decided that the joints in question should have a full penetration weld, welded from both sides and with a weld contour merging smoothly into the parent metal both internally and externally. The required grinding of these joints therefore also applied to the internal weld surface.

...

4.5. Temporary Phases

4.5.1. General

This chapter conserns the procedures and investigations performed in connection with the temporary phases of the life of the platform. Temporary phases include load-out, transportation and installation.

The strength analysis for the platform and its individual parts in the temporary phases as well as the permanent in-service condition is covered by the preceeding chapters. This chapter thus reviews the operational aspects of the temporary phases.

4.5.2. Jacket load-out

As explained in the foregoing a number of investigations were conducted in connection with the load-out of the Jacket from yard to barge.

The procedure for this operation was worked out in detail, with definition of every step to be taken as a function of time. Data on tides in the Cherbourg harbour were obtained and data on the soil and foundation of the skid ways were investigated.
All essential equipment was tested on before hand and the lines of communication was established.
The most critical part of the work was the transfer of the Jacket from land onto the barge. The tidevariations at the location is relatively large and the barge did not have ballasting capability to let the load-out operation continue over one full cycle of tide variation. It was thus essential that the time shedule was maintained (with some safety margin).
The procedures worked out for the load out operation were evaluated in detail by DnV.

Also all the background data and investigations were reviewed. The latest revisions and updating of these procedures took place and were evaluated immediately prior to the actual operation in order to take into account the latest and best information with regard to ballasting capacities, tides etc.

4.5.3. Load-out of Deck Support Frame and Production Modules

The load-out of the Deck Support Frame and the Production Modules was performed essentially similar to the procedures adopted for the Jacket.

Skid-ways were laid from the construction site onto the barge.

Due to the comparatively (compared to the Jacket) smaller weights and masses involved and the much less tide variations, there were not carried out so extensive investigations for the load-out of these structures. However, in principle were similar types of analyses carried out.

These analyses along with the appropriate background data were received and reviewed by DnV and found in order.

4.5.4. Lifting operations.

The Production Modules as well as the temporary modules were lifted from the transportation barge onto the platform. Also the permanent as well as the temporary Deck Support Frames were lifted from the transportation barges onto the Jacket.

In addition to the strength analysis carried out for these structures for the lifting operations (described in the foregoing), detailed plans and procedures for the lifting operations were worked out by the designer. These plans and procedures described the equipment to be used such as barges, crane barges, slings, chackles etc.
Also details on mooring and locations of the barges during these operations were planned.

The documentation received in connection with these operations was evaluated by DnV and found in order.

4.5.5. Transportation

The following transportation barges or ships were used to transport the different main parts of the DP2 platform from the construction yard to the Frigg Site:

- INTERMAC 600 - Jacket
- REFANUT - Deck Support Frame
- REFANUT - Temporary Work Deck
- GRIEG 3 - Piles
- MÆRSK 7 - Production Modules A, B, C and D
- MØRELAND 6 - Temporary Quarters

For the Jacket and the Deck Support Frame, motion response analysis were carried out by DnV in order to properly verify the loads imposed on the structures during transportation.

Also, towing arrangements were evaluated in detail.
The towing route and points of shelter were also settled.
For the Jacket transportation, two tugs were attached to the barge to minimize the possibility of loosing control of the barge.

Also, intact as well as damaged stability were investigated.
The damaged stability essentially involved checking the stability of the barge for any watertight compartment flooded.

The stability calculations were checked by DnV and found acceptable.

4.5.6. Jacket launching

As explained in the foregoing model tests were performed to determine the Jacket behaviour during launching.
These tests were run in still water as well as in irregular seas. In addition to the Jacket behaviour also rocker arm reactions and slamming loads on the buoyancy tanks were recorded.

The Jacket launch site was determined at a site close to the intended installation site.
The water depth at the launch side was approximately 105m so as to have some safety margins with regard to bottom clearance for the subsequent up-ending of the Jacket.

Details on the towing and mooring arrangements as well as trimming of the barge were also worked out. The trim angle was determined so as to minimize the rocker arm reactions during launching.

The documentation received on the launching was evaluated and found satisfactory.
DnV also attended some of the model experiments carried out at MSMB in the Netherlands.

4.5.7. Jacket Up-End and Positioning

During the model tests at NSMB the controlled flooding sequence was determined, and the bottom clearance monitored for the critical phases.

Also mooring of the Jacket with a number of tugs was prescribed together with details on final positions of the unit.

The flooding of the relevant compartments were conducted from a control panel located on top of the Jacket.

The flooding lines as well as grout and vent lines were designed, fabricated and tested in accordance with ANSI B31.3.
An underwater survey of the launching and towing areas as well as the actual site was carried out by a submersible prior to the installation of the Jacket.

The documentation received in connection with the up-ending and positioning of the Jacket was reviewed by DnV and found to be in order.

4.6 Corrosion Protection Design

Elf performed the corrosion protection design calculations and issued drawings on the installation on anodes.

The design report and the drawings were received and evaluated by DnV and found to be in order. It was, however, required that potential measurements be carried out when the structure had been installed on the site for a short time.

The following table shows the distribution of the aluminium anodes and zink reference anodes throughout the Jacket structure.

Within the splash zone area, an extra thickness of ½inch (12.7mm) was incorporated in addition to that required for strength considerations.

The additional material was added as extra thickness and not in the form of wrap plates.

PART OF STRUCTURE	NUMBER OF ANODES			REFERENCE ELECTRODE
	A (385.1kg AL)	B (385.1kg AL)	C (265kg AL)	
Elevation (-)11.3m	100			E 1
Between elevation (-)11.3/(-)28.95	76			
Elevation (-)28.95	48			E 7
Between elevation (-)28.95/(-)50.29	105			E 4
Elevation (-)50.29	56			E 2
Between elevation (-)50.29/(-)70.86	106			
Elevation (-)70.86	91			E 5/E 8
Between elevation (-)70.86/(-)100	160		140	E 3
Elevation (-)100		87		E 6

(Piles and wells protecting by anodes on Jacket.)

5. FABRICATION

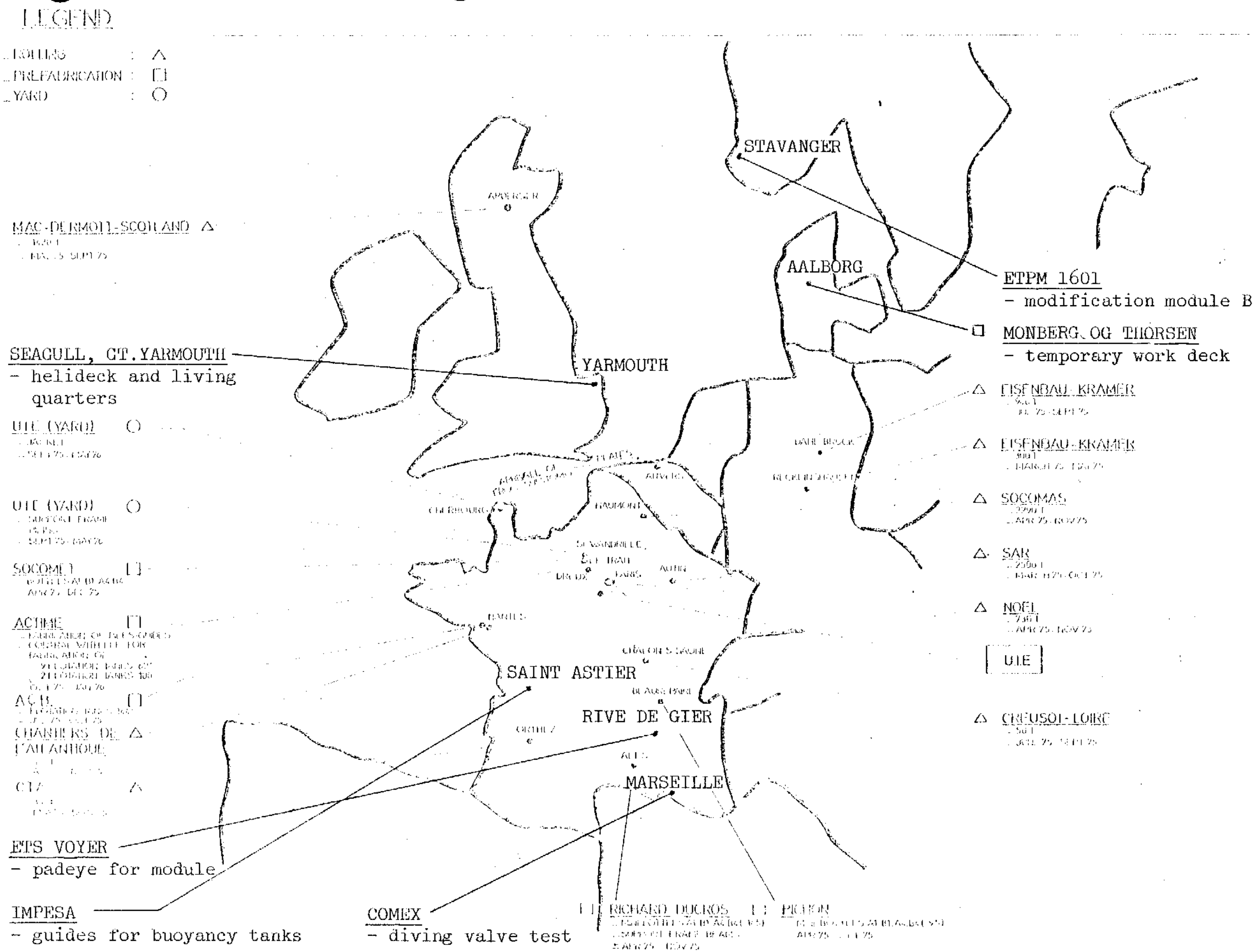
This chapter reviews the fabrication of the DP2 platform and parts thereof.
The associated control activities performed by DnV is also described.

The fabrication and the inspeciton are, in the following, only briefly reviewed since these aspects have been reported in separate progress- and inspection reports worked out by the local DnV Suveyors. For more detailed information on the control activities performed by DnV reference is therefore made to these progress- and inspection reports.

5.1. Fabrication Yards

Most of the structural materials used for the fabrication of the platform were produced by SUMITOMO, Japan.DnV surveyed the material fabrication and witnessed material certification tests.

The main fabrication yards involved in the project are shown on Fig. 5.1. on the next page. This figure shows the yards that performed rolling of tubulars, pre-fabrication of larger structural components as well as the assembly of the different main parts of the platform - i.e. Jacket, Deck Support Frame and Production Modules.
Also the approximate time at which fabrication took place at each individual yard is shown.


LEGEND
ROLLING : △
PREFABRICATION : □
YARD : ○
MAC-DERMOTT-SCOTLAND △
SEAGULL, GT.YARMOUTH
- helideck and living quarters
UIE (YARD) ○
UIE (YARD) ○
SOCOMET □
ACTIME □
ACB □
CHANTIERS DE L'ATLANTIQUE △
CTA △
ETS VOYER
- padeye for module
IMPESA
- guides for buoyancy tanks
COMEX
- diving valve test
RICHARD DUCROS
PICHON
STAVANGER
AALBORG
YARMOUTH
SAINT ASTIER
RIVE DE GIER
MARSEILLE
ETPM 1601
- modification module B
MONBERG OG THORSEN
- temporary work deck
EISENBAU-KRAMER
EISENBAU-KRAMER
SOCOMAS
SAR
NOEL
UIE
CREUSOT-LOIRE

5.2. Surveys and Inspection

Inspection including NDT at the different fabrication and assembly yards was carried out by the fabricators, ELF and/or the yard as well as the main contractor - UIE.

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 3.2 above.

DnV surveyors attended the fabrication at all the yards involved. On the main assembly yards - in Cherbourg and St. Wandrille - as well as at SOCOMET Le Trait, 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. All the surveys carried out in France were coordinated from the regional office in Marseille by one surveyor specifically assigned to coordinate all the activities related to inspection of offshore structures in France.

It was generally concluded from the surveys carried out that the 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 good and the extent of NDT inspection was found to be in compliance with that set forth in the fabrication specification DnV carried out spot checks of the NDT inspection performed by others. These spot checks were carried out by DnV's own NDT personell.

A majority of the defects discovered were found in complicated structural components involving high heat input and large residual stresses.

Reference is made to the chapters 4.1 and 4.4.5 conserning welding of the large thickness members.

The relevant members and joints were, as mentioned above, subjected to regorous NDT inspection and grinding as well as post weld heat treatment (at approximately 580 - 600 °C) where possible.
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.
DnV carried out survey of the fabrication and pressure testing of the grout - and vent lines to be used for grouting the piles. Thightness testing of the Jacket legs and buoyancy tanks was also surveyed by DnV.
The buoyancy tanks were generally surveyed as per primary structure, and with detailed geometrical measurements before and after installation in the Jacket.
Geometrical measurements were also conducted on the structure overall as well as on individual components and members, although not to the same degree as for the buoyancy tanks.

Material marking and logistics routines were also found to be satisfactory.
ELF in cooperation with the 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 Jacket and the Deck Support Frame.

A copy of the drawings and documents produced for this purpose was handed over to the DnV surveyors in Cherbourg and St. Wandrille upon completion of the work. For the part of the platform fabricated in Cherbourg, DnV also have the material certificates whereas for the Deck Support Frame and Piles DnV only received the tracing system and the material certificates were in accordance with agreement between NPD and DnV to be kept by ELF. This agreement implies that ELF is responsible for keeping a complete file of all records, drawings, certificates etc. pertinent to the platform. This file shall be available to the Authorities involved.

Also the coating and installation of the sacrificial anodes were found acceptable and in accordance with the accepted drawings and specifications.

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

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 inspeciton at the main yards - in Cherbourg and St. Wandrille - are attached in Appendix 6 to this report.

6. INSTALLATION

This chapter briefly review the installation of the structure and parts thereof.
The related control activities performed by DnV is also described. For further details on the control work carried out reference is also made to the progress- and inspection reports worked out by the DnV site surveyors.

6.1 Installation sequence

The DP2 Jacket was launched in the Norwegian part of the Frigg field during the night to May 11, 1976. After launching it was upended and towed to location and finally set down at 1400 hours the same day. The launching operation went exactly in accordance with the plans and procedures worked out. The up-end operation were performed by controlled flooding of the corner legs. The flooding was carried out from a control panel mounted on the top of the Jacket. Some problems were encountered when going to transfer personell to the Jacket. This operation proved to be difficult due to the wave action.
Some problems also arised during the actual up-end operation as the Jacket floated at an angle, i.e. the longitudinal axis of the jacket was not parallell to the water surface - resulting in a minimum of bottom clearance during the up-end - and subsequent towing operations. The Jacket might have touched bottom during these phases.
It was, however, successfully set down on location and an underwater survey carried out by a submersible immediately afterwards did not reveal any damages.

During the following two months all the centre-and corner primary piles were driven to grade. When eight of these piles had been installed from the barge DB22 the Temporary Work Deck was installed on the Jacket.

The installation of the centre- and corner primary piles was completed on July 14 1976.

Thereafter the Temporary Work Deck was removed and the Deck Support Frame installed.
Upon the installation of the Deck Support Frame a number of temporary work modules, two stiff-leg structures as well as a temporary quarter/helideck module were also lifted into the platform. The installation of all the corner insert piles were performed from these modules.

Upon completion of all pile installation the temporary modules and equipment were removed and the permanent Production Modules lifted on to the Deck Support Frame, skidded into position and welded.

The following table shows the dates when the different units were installed.

Installation dates	
Jacket	:May 11 1976
Temporary Work Deck	:June 24 1976
Deck Support Frame	:August 3 1976
Work Modules	:August10-20 1976
Temporary Quarter/Helideck Module	:November 2 1976
Production Modules A and B	:January17-18 1977
Production Modules C and D	:February 9 1977

6.2. Surveys and Inspection

The main contractor for the offshore work was Oceanic Contractors. However, also some other companies were involved in the offshore construction work.

The inspection was carried out by ELF as well as other companies hired for performing inspection including the NDT control.

DnV conducted continuous supervision and survey of the construction and erection work performed offshore. It was generally found that the work was done under proper supervision and in accordance with relevant and accepted drawings, procedures and specifications. Also the extent of NDT inspection was found adequate. The qualification of the NDT personell was checked and in one particular case, an NDT operator was found unreliable/ unqualified and was subsequently replaced.

DnV carried out spot checks of the NDT inspection by utilizing DnV's own NDT personell.

Relevant driving, drilling and grouting records were obtained from the pile installation. These records were sent to DnV headoffice for review by the local DnV site surveyors. See chapter 6.3 for further details on foundation.

During welding of the Deck Support Frame/Jacket field splice,repeated cracking and repair occured. Eventually, however, a proper repair procedure was established and the welds were successfully repaired.

Temporary structures and attachments were removed by cutting and grinding and the areas were subsequently subjected to magnetic particle inspection.

Also some remaining grinding of joint stiffeners on the +6.095 m conductor frame was carried out successfully.

Relevant welding procedures were conducted on the field and shipped on land for testing and inspection.

It was generally found that the workmanship and inspection carried out offshore complied with appropriate and accepted drawings, procedures and specifications.

6.3 Foundation Installation

6.3.1 Main pile installation

Pile driving

The piles were driven with a Vulcan 060 and 560 hammer. All 4 centre piles were driven to predetermined penetration (21m) without any problems while 3 out of the 16 corner piles failed to reach, the predetermined penetration (18m) This applied to B4.1, B4.2 and B4.4 which was left 1.5 - 3 m short of penetration due to high blowcount. The piles and the foundation were, however, designed to accomodate an under- and overdrive of 10 feet (3m)
DnV has received the piledriving records and found the pile driving to be according to accepted specifications. No jetting or predrilling was applied. The lack of penetration is within the driving tolerances dictated by the design.

Pile grouting

All main piles were grouted to the structure. Some problems with packers not working properly and plugging of groutlines did occur but adequate steps were taken and the grouting was successfully completed. Quality control of the grouting was carried out with respect to slurry weight and cube strength .
The results are evaluated and found acceptable.

6.3.2. Insert pile installation

Drilling and inserting.

The insert piles were installed in holes drilled with a 1.22 m diameter drill tool through the driven piles. All holes were driven to a depth equal to 117 m by using a reversed circulation drilling technique. A prepared mud was used as drilling fluid.

The holes were drilled without any major problems, and the piles inserted to correct depth. The drilling records were reviewed by DnV and found satisfactory. During the design phase the presence of a possible clay layer at the pile tip was discussed. The drilling records indicates that such a layer is not encountered.

Grouting of insert piles.

The insert piles were grouted to the soil and the main piles in a two stage grouting operation.
The grout was pumped through a line inside the insert piles ending at the pile tip. First stage extended from the bottom of the hole up to 7,6 m inside the driven pile. At this level a valve in the groutline, made it possible to grout the rest of the annulus up to the top of the pile in a second stage by activating the second stage grout tool.
The reason for the two stage grouting process was to avoid a too high grout pressures which was likely to fracture the soil. However, despite this precaution the grouting records indicated that hydraulic fracturing of the soil occured at the A4-3, A4-4, B1-1, B1-4 and B4-2 corner insert piles.

A review of the installation records however, indicates that correct measures were taken, and that the grout placement was adequate.
The procedure of tracing the groutlevel with a radioactive source was not entirely successful, but grout was always confirmed by a samler at top of both 1 and 2 stage.

The cement used for incert pile grouting was of the "Diacemoil type". This in order to get a leight weight slurry compared to what the standard portland cement would make possible. The cube strength of grout samples taken is low and the resulting bond strength must be studied further.

It was therefore agreed between the parties involved to obtain more information on the bond strength obtained with the Diacemoil type grout by conducting appropriate tests. The results obtained will be reported separately.

See also appendix 7 for more details on DnV's evaluation of the foundation.

SCOPE OF WORK

for

Control and Inspection

of

Fixed Offshore Platform

DP2 - 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 the fixed offshore installation DP2, Frigg Field on behalf of Norwegian Petroleum Directorate.

In general the inspection by DnV 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.

DnV will evaluate and issue their letter recommending approval of the following main stages:

1. Design and fabrication of jacket structure
2. Design, fabrication and installation of risers, riser supports, and riser protections
3. Foundation design and pile installation
4. Design and fabrication of deck support frame structure
5. Complete installation after mounting of deck support frame structure and installation of modules A, B, C, and D

DnV will also evaluate and issue a statement on:

6. Temporary stages; transportation methods, loads imposed on main items during transportation, stability calculations for the transportation barges and stability of the structure at different stages of pile installation

2. SCOPE OF WORK

2.1 Evaluation of Design Premises

- Environmental conditions including soil and seabed conditions
- Codes, standards and specification used for the design and fabrication of primary and secondary structure, risers, helicopter deck, lifting appliances, derrick, equipment, machinery and system, and in particular for steel materials; welding, fabrication, inspection, and corrosion protection

2.2. Review of Design

- General arrangement, configuration and system drawings for final structure and all relevant temporary stages
- Environmental design loads for all relevant stages
- Application of materials, welding and inspection methods
- Foundation design
- Structural analysis and design calculations (stresses, buckling, joint check, fatigue) of the jacket structure at final and launching stages
- Structural analysis and design calculations of risers, riser supports and riser protections
- Structural analysis and design caluculations of deck support frame and modules with helicopter deck and derrick
- For temporary stages; evaluate stability calculations for trans-transportation barges, loads imposed on main items during transportation and lifting operations, stability of the structure at different stages of pile installation and loads imposed on buoyancy tanks during launching and upending stages
- For electrical systems, one-line diagrams, methods of protection in hazardous areas, short circuit and fault protection
- Corrosion Protection Systems

To assist as required by NPD to review platform safety including:

- Hazardous areas
- Escape plan
- Safety systems including systems for detection of gas and fire, emergency shut down, alarm and public address, firefighting and fire-protection
- Emergency power
- Relief, flare and vent systems

2.3 Inspection and Certification of Construction Materials

- Survey of material fabrication and witnessing of material certification tests
- Review of certificates and material marking

2.4 Inspection during Construction

- Approval of welding procedures welding equipment and welders
- Approval of repair procedures
- Survey of fabrication of components and finished products
- Inspection for code compliance for process equipment
- Survey of construction tolerances
- Survey of quality of repairs
- Survey and identification of mechanical systems relevant to the safety of the structure during upending and installation

2.5 Evaluation of Transportation and Installation Methods

- Environmental conditions along towing route taking into account seasonal limitations
- Transportation methods for larger items
- Installation procedures for the jacket support frame and modules
- Loads imposed on larger items during transportation and installation
- Review model test report

2.6 Inspection during Installation

- Reinspection of larger items after arrival on site
- Inspection during launching, upending, and final positioning and levelling
- Inspection of piling and evaluation of pile driving records
- Final assembly of the structure and the process and utility systems per paragraph 2.4 above
- Module installation
- Witnessing of load and functional tests of lifting appliances
- Electrical installations

Olb/Alf/ESJ

28.10.75

APPENDIX NO. 4

to

DnV Report No. 601636/12

DESIGN, FABRICATION AND INSTALLATION

RESUME

[illegible]
[illegible] 27-10-77

INTRODUCTION

This Appendix (No. 4) is comprised of a resumé of the design, fabrication and installation of the Frigg Field D--illing Platform No. 2 (DP-2). That is to say, those areas, joints members, etc., which based on DnV's evaluation of the structural design as well as inspection during construction and installation is considered to be most critical, relatively speaking, are pointed out.

Said areas, joints, etc., are in the following described by convenient and simplified drawings with an appropriate and short accompanying text. For obtaining a more complete and general background for the selection of critical areas reference is made to the main report.

It should be noted that symmetry has been considered when designing the platform and therefore also when selecting the critical areas.

It should furthermore be noted that the 1/3 increase in allowable stresses and joint capacities (static strength) allowed by the design codes, standards and rules used, has not been considered for the design of this platform.

It is also pointed out that the following figures except for the fatigue and fabrication/installation refers to the extreme environmental condition; drilling phase.

The critical areas are in the following designated as follows:

◎ (tripple circle): Frequent inspection by best available methods is most important

◎ (double circle) : Regular inspection is important

○ (single circle) : Regular inspection is relatively important

1. Tubular Joints - Static Strength

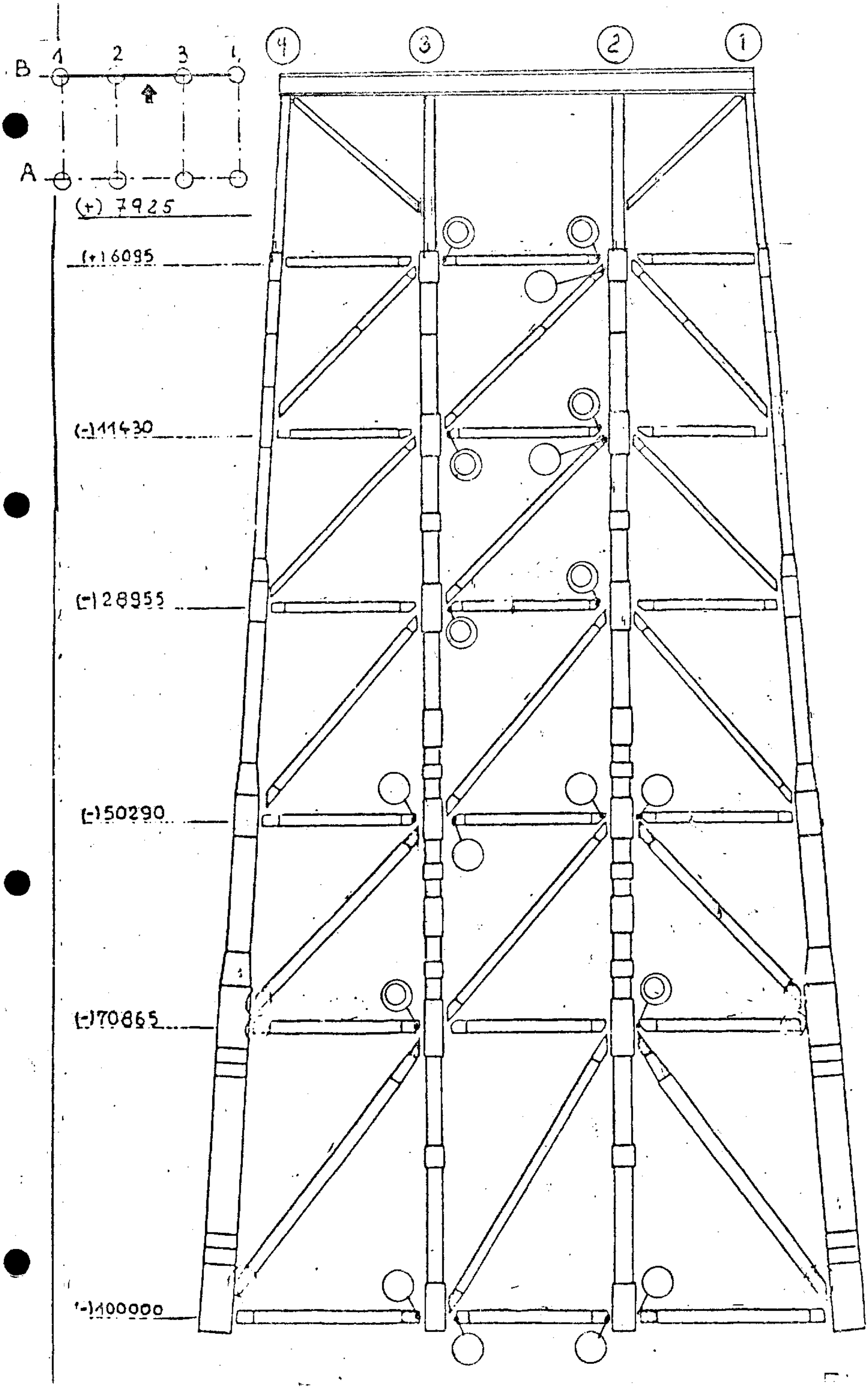
The following figures describe the most critical tubular joints in the jacket structure. The selection of the relevant joints is based on the requirements set forth in API RP 2A 7th edition.

It should be noted that all the can sections on the corner legs from elevation -28.955 m and downwards are stiffened internally by ring stiffeners as well as longitudinal stiffeners. These joints have not been designed in accordance with the above referenced criteria and they are therefore not included in the following figures.

Members marked with a double circle (◎) have usage factors equal to or greater than 0.9 in the extreme condition. Members marked with a single circle (○) have usage factors between 0.70 and 0.9, also in the extreme condition.

Usage factor is defined as the ratio: actual punching shear/allowable punching shear.

It should be noted that bracing member ends are marked with circle(s). The marking thus refers to the joint formed by this brace and the can to which it is welded. In the event of an overlapping joint it also refers to the weld between the brace and the member being overlapped.



B
A
1
2
3
4
(+) 7925
(+) 16095
(-) 11430
(-) 28955
(-) 50290
(-) 70865
(-) 100000

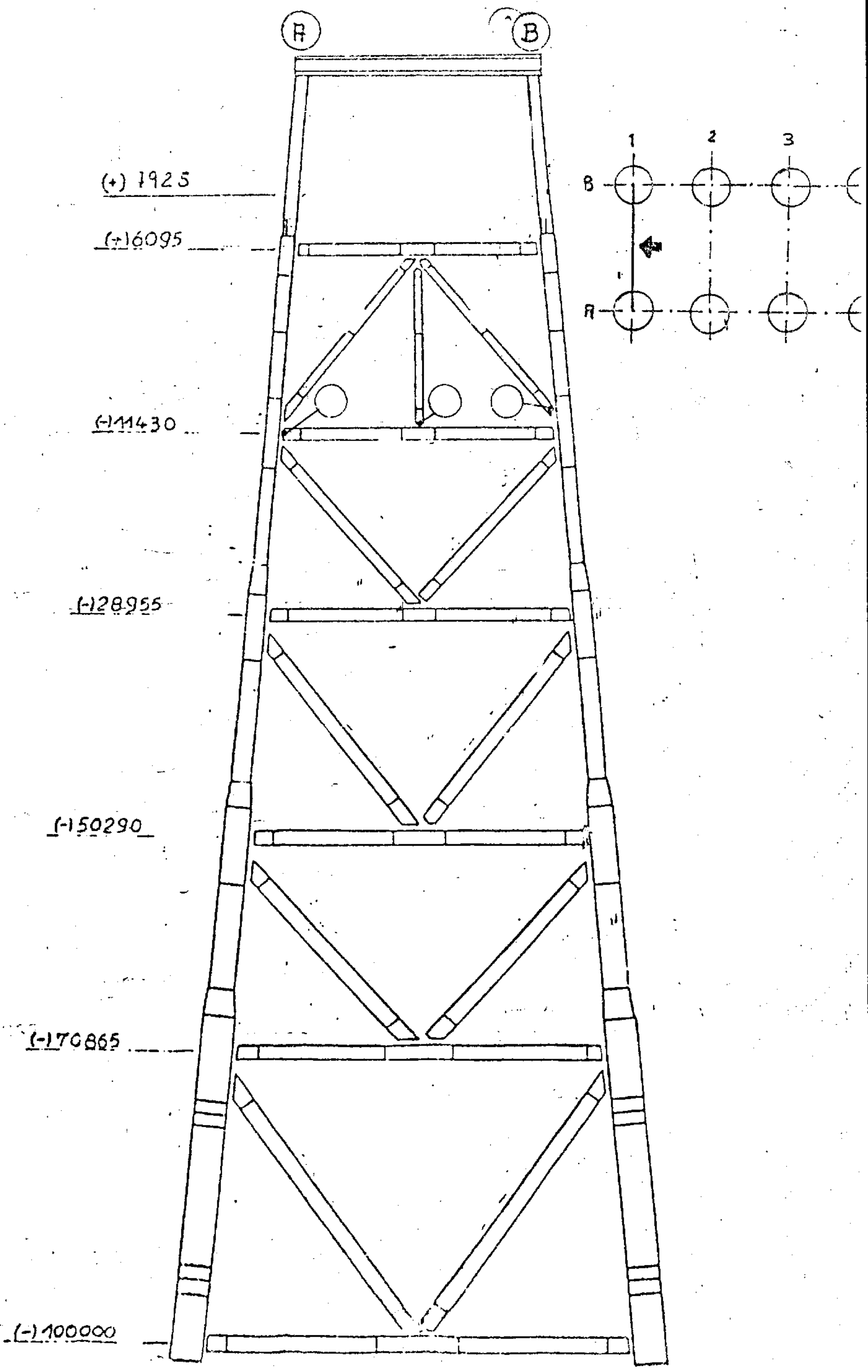
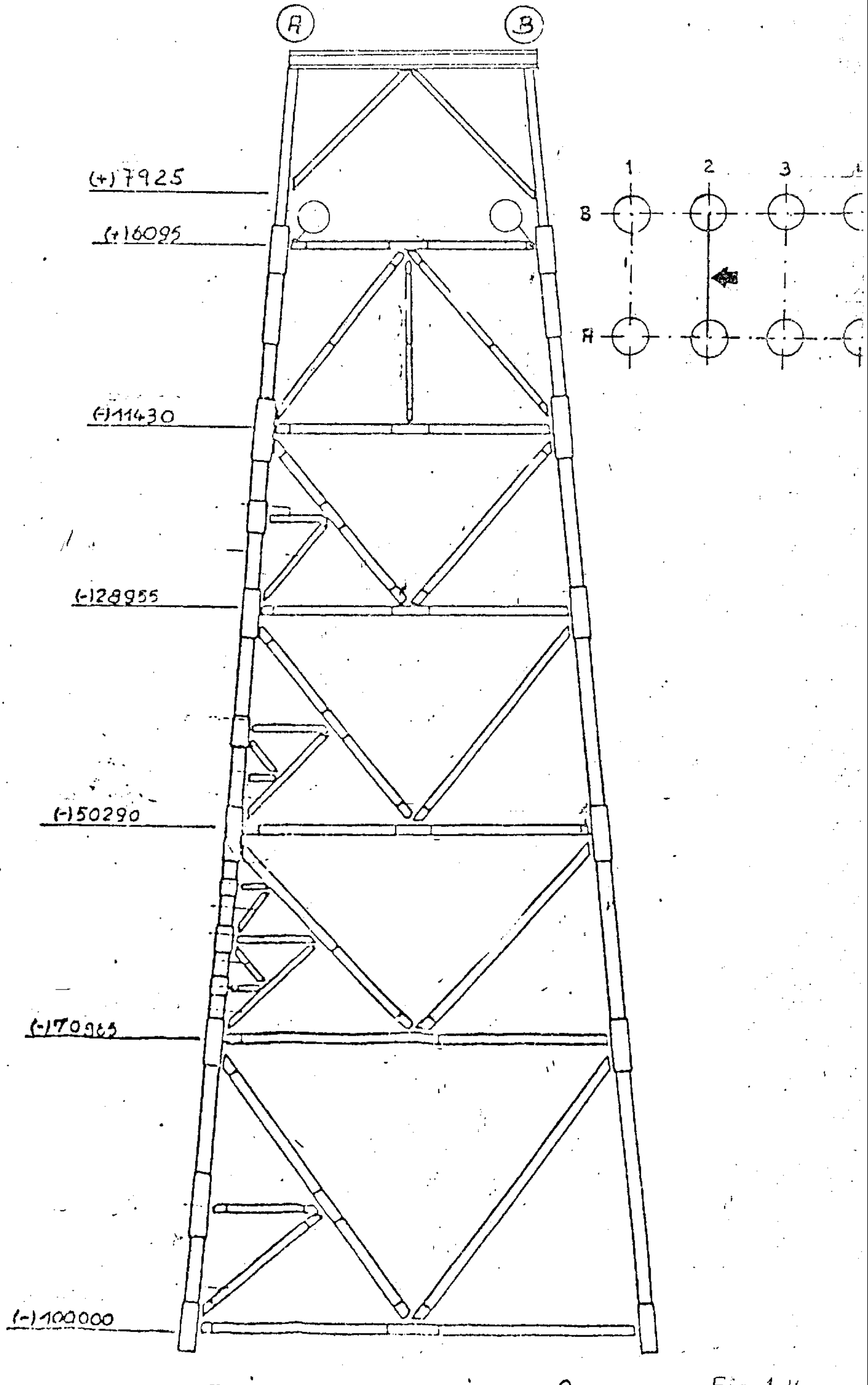


Fig. 1.3



A
B
(+)7925
(+)6095
(-)11430
(-)28955
(-)50290
(-)70965
(-)100000
1
2
3
B
A

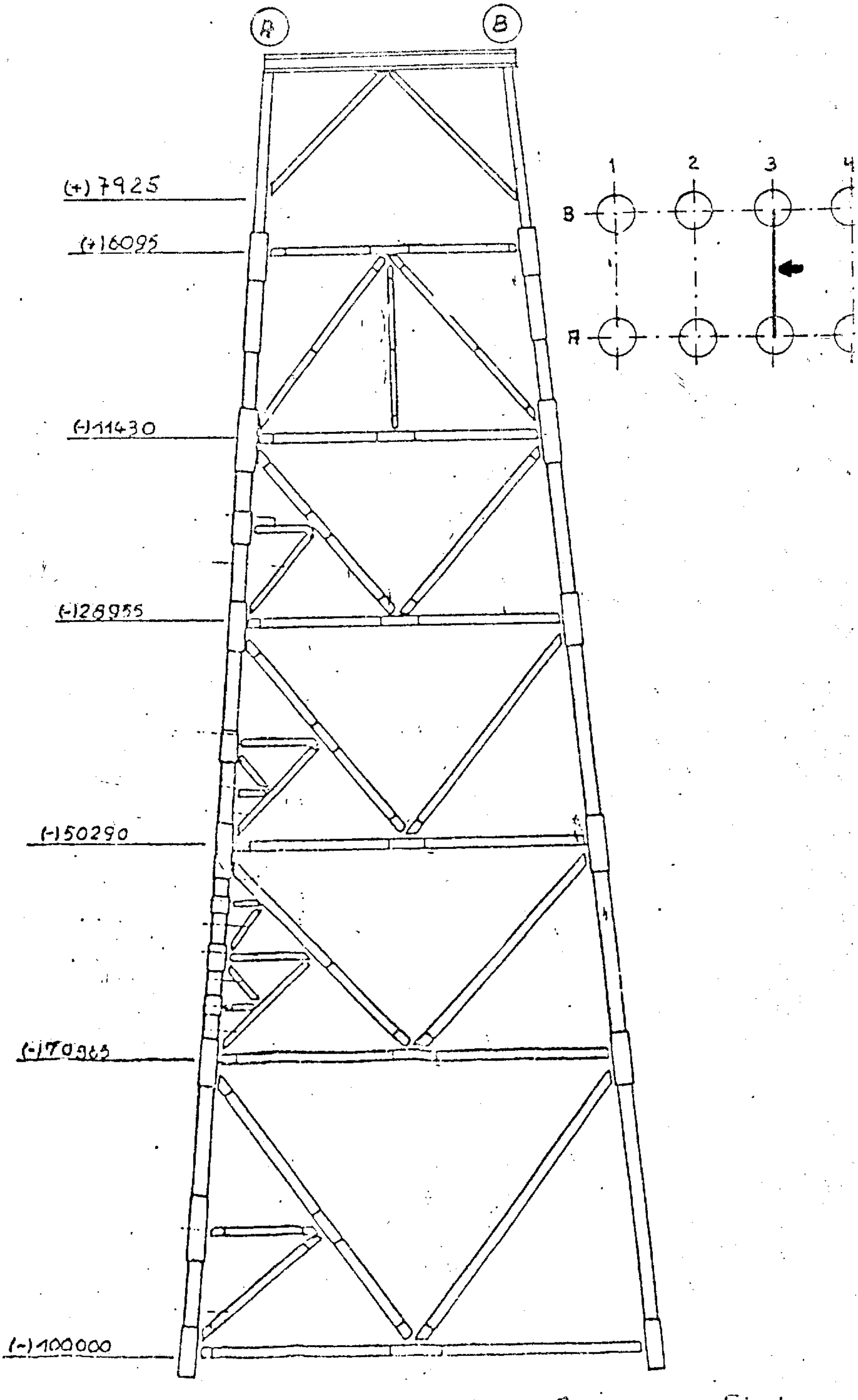
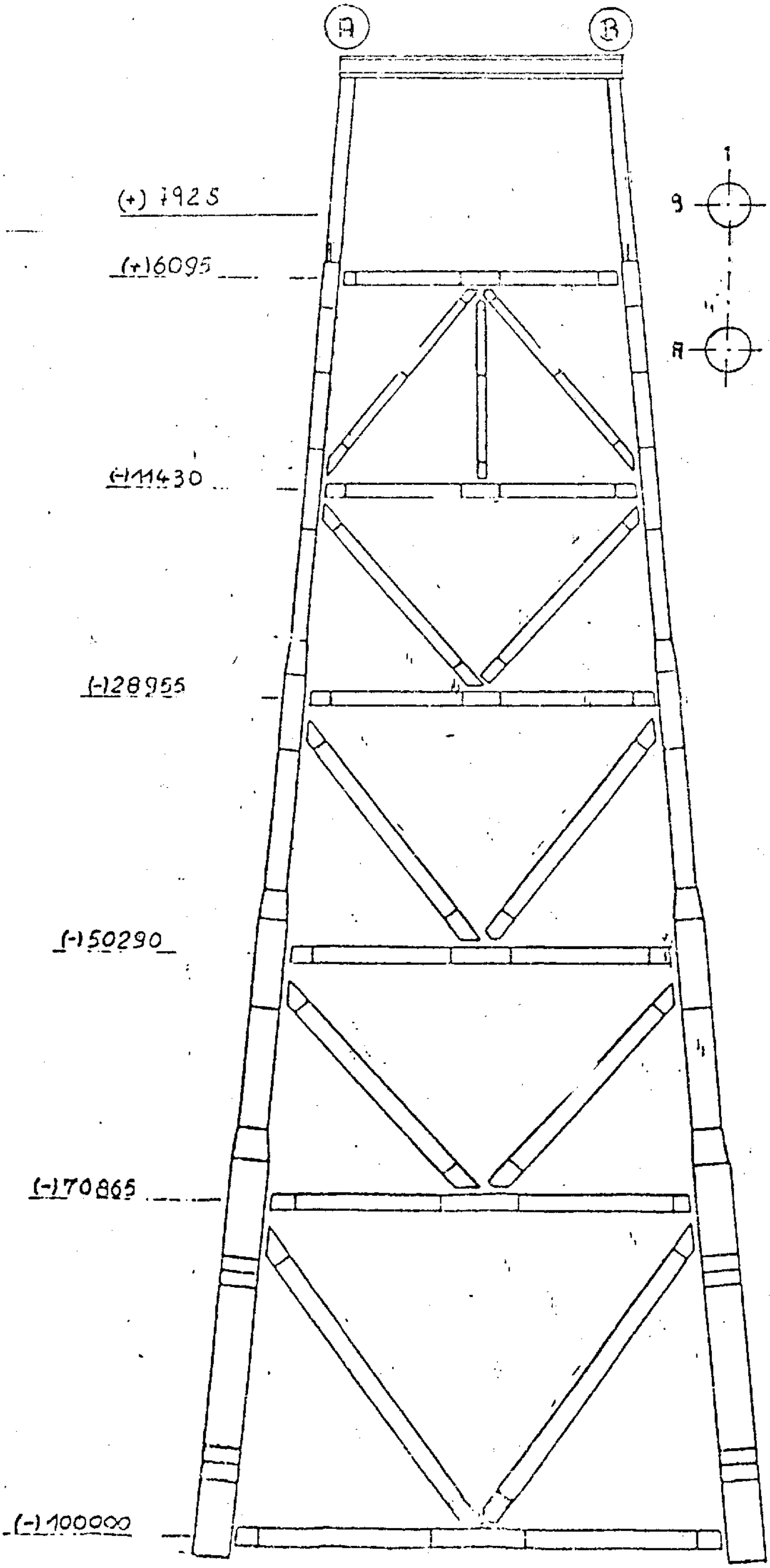


Fig 1.5



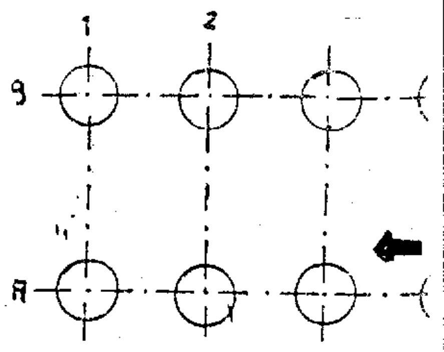


Fig 11

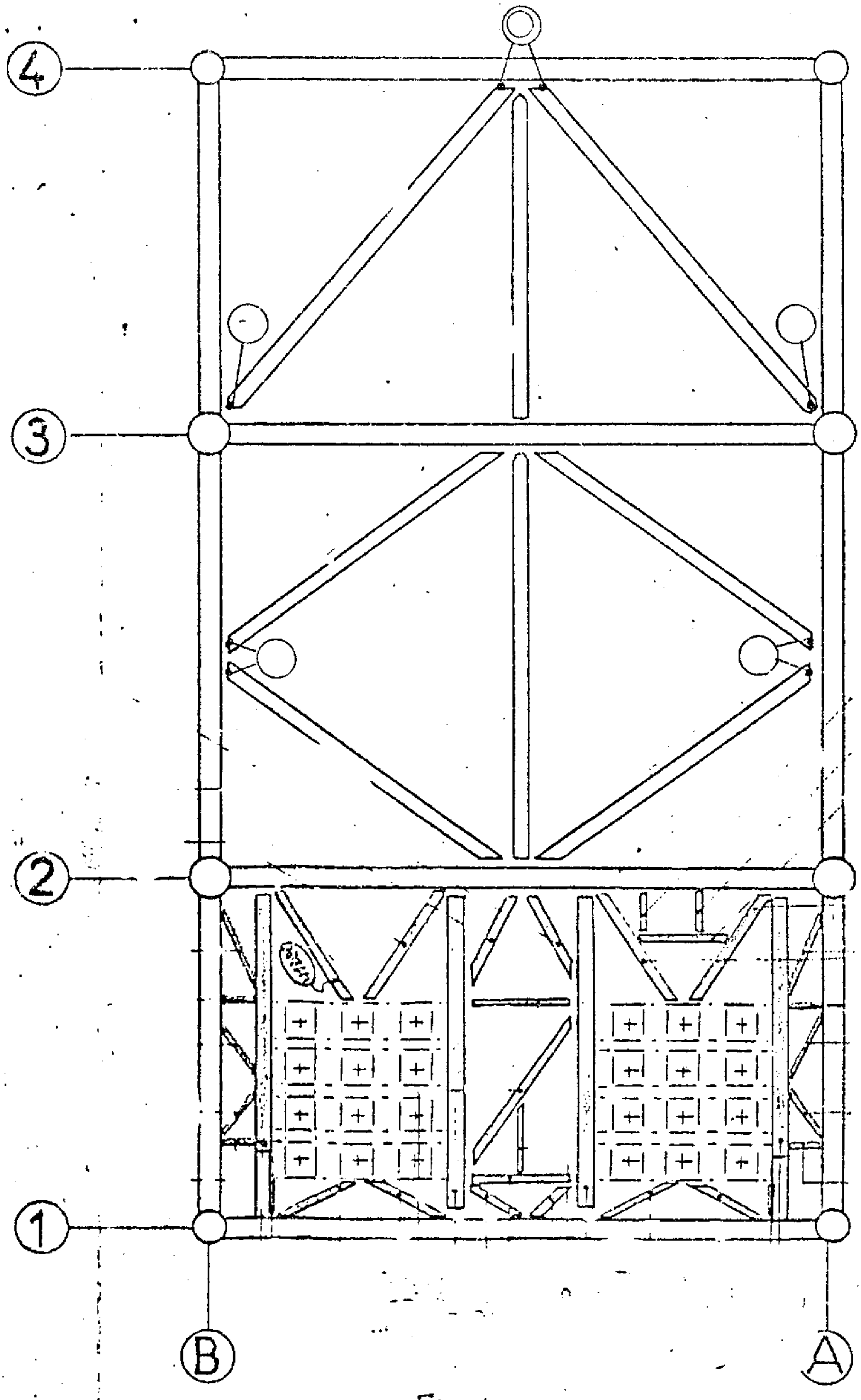


Fig 1.7

Siège : 4ᵇⁱˢ avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (+) 6095	Dessiné : Le : 19.08.75 Visa : Echelle : 1/250 N° Fo 457 695 1/13

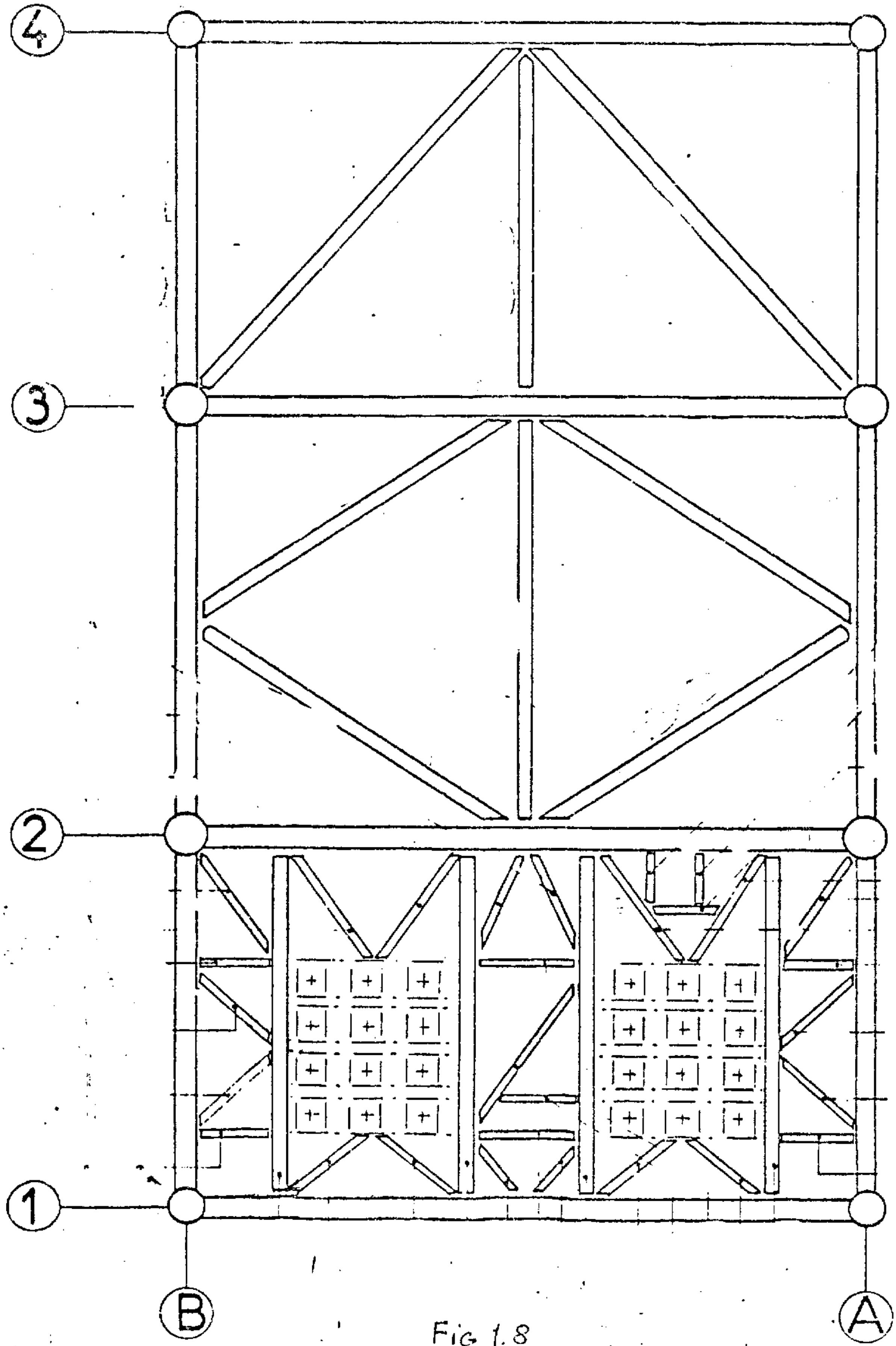


Fig 1.8

Siège : 49b avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (-)11430	Dessiné :	Le 19.08.75
		Visa :	Echelle 1/250
		N° Fo 454.699 6/43	

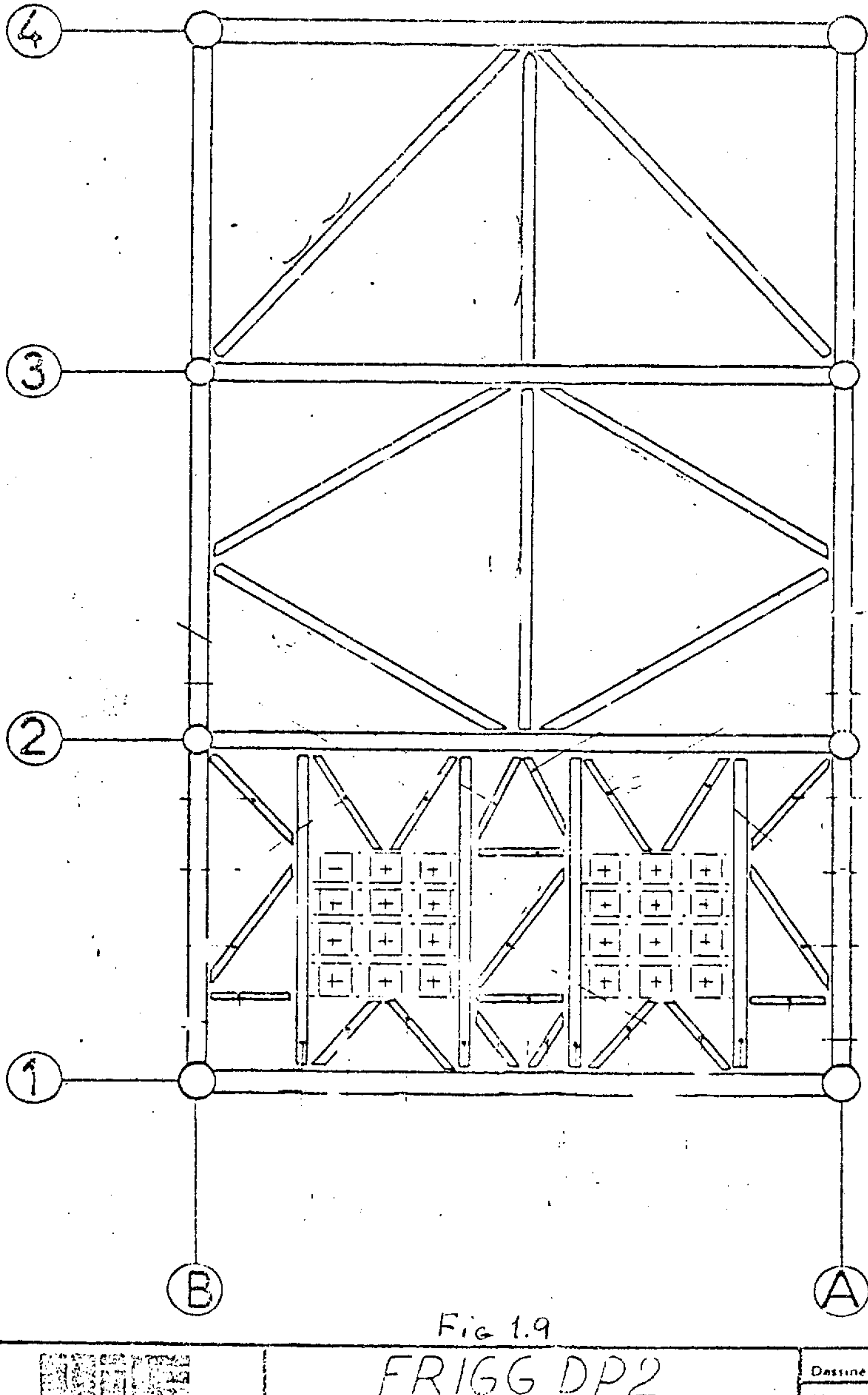


Fig 1.9

Siège : 49b avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (-)28955	Dessiné : / Le : 19.08.75 Visa : / Echelle : 1/300 N° Fo 454.699 5/13

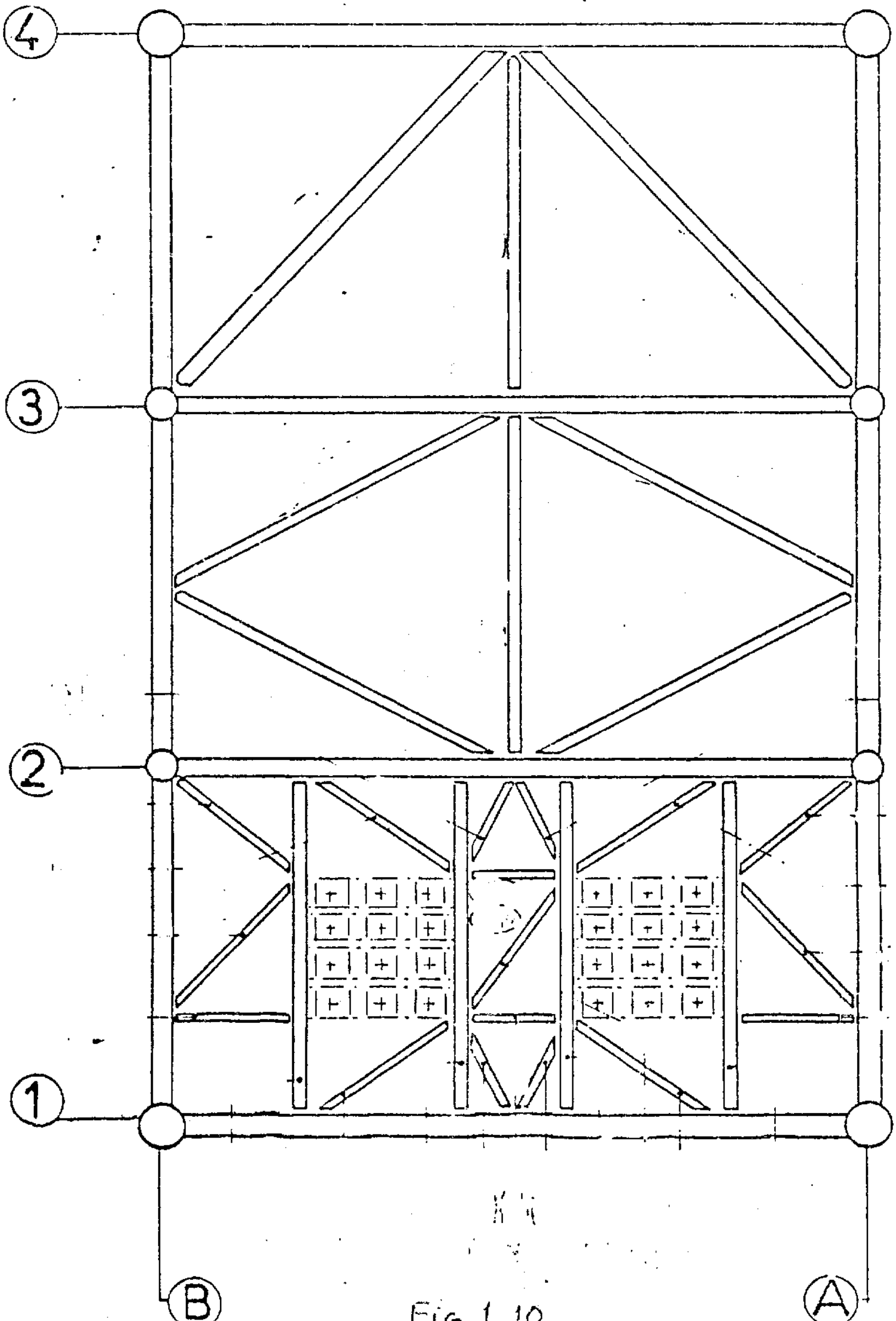


Fig 1.10

Siège : 40bis avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (-) 50290	Dessiné : Le 21.08.75 Visa : Echelle : 1/300 No Fo 454 693

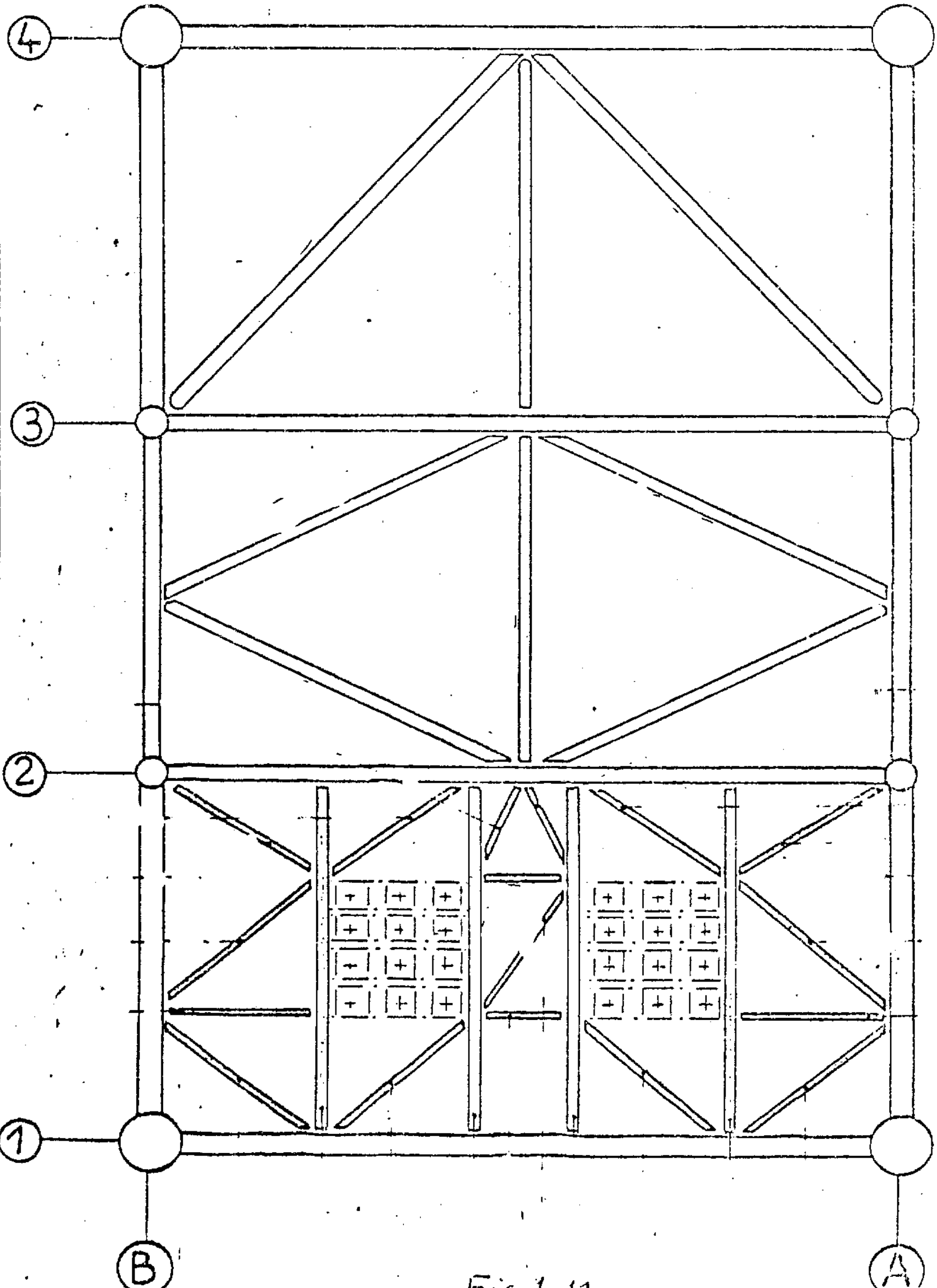


Fig 1.11

Siège : 49 avenue Hoche - PARIS Établissement de CHERBOURG	FRIGG DP2 Niveau (-)70865	Dessiné : Le [illegible] Visa : Échelle : [illegible] N° Fo [illegible]

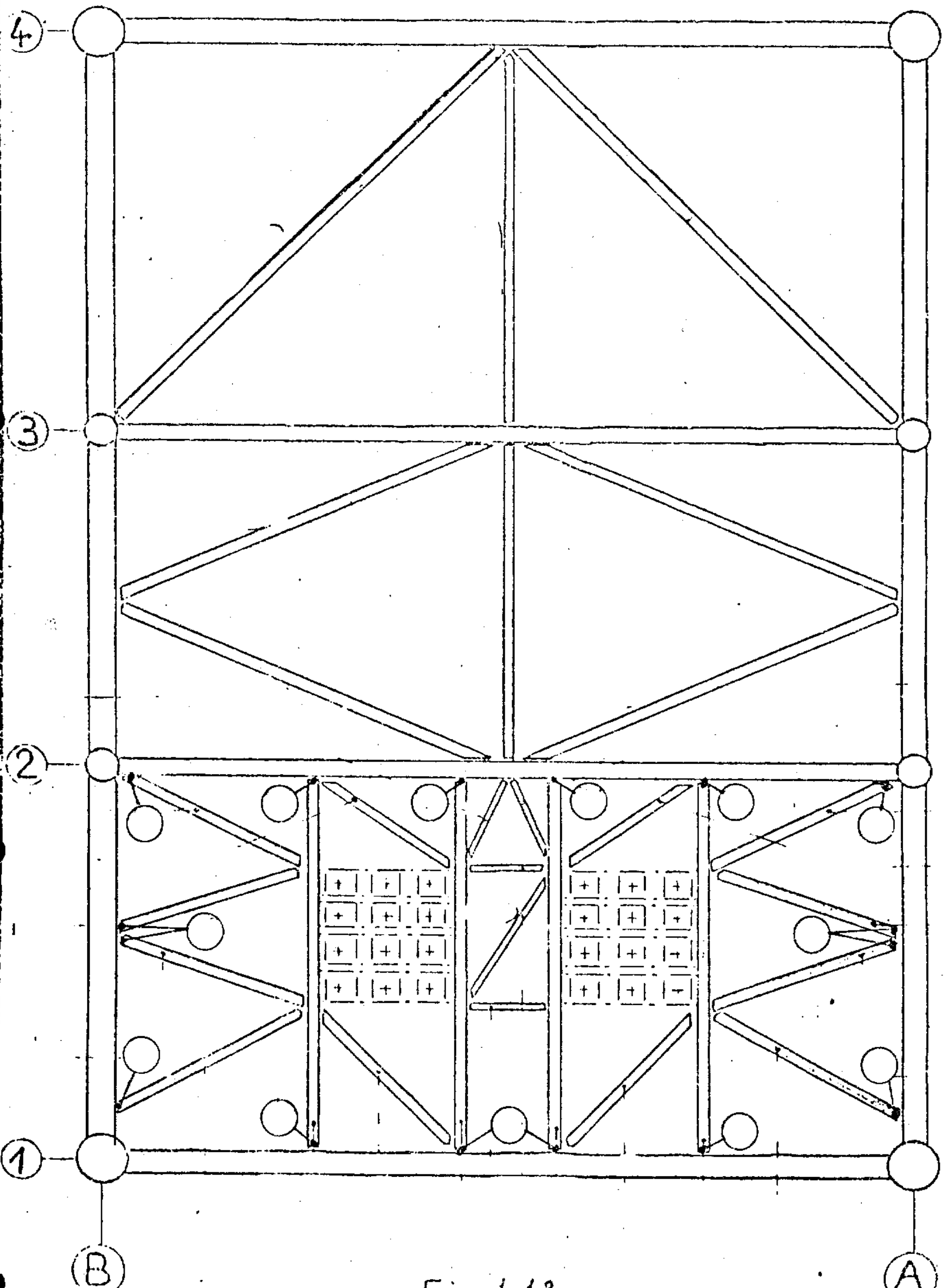


Fig. 1.12

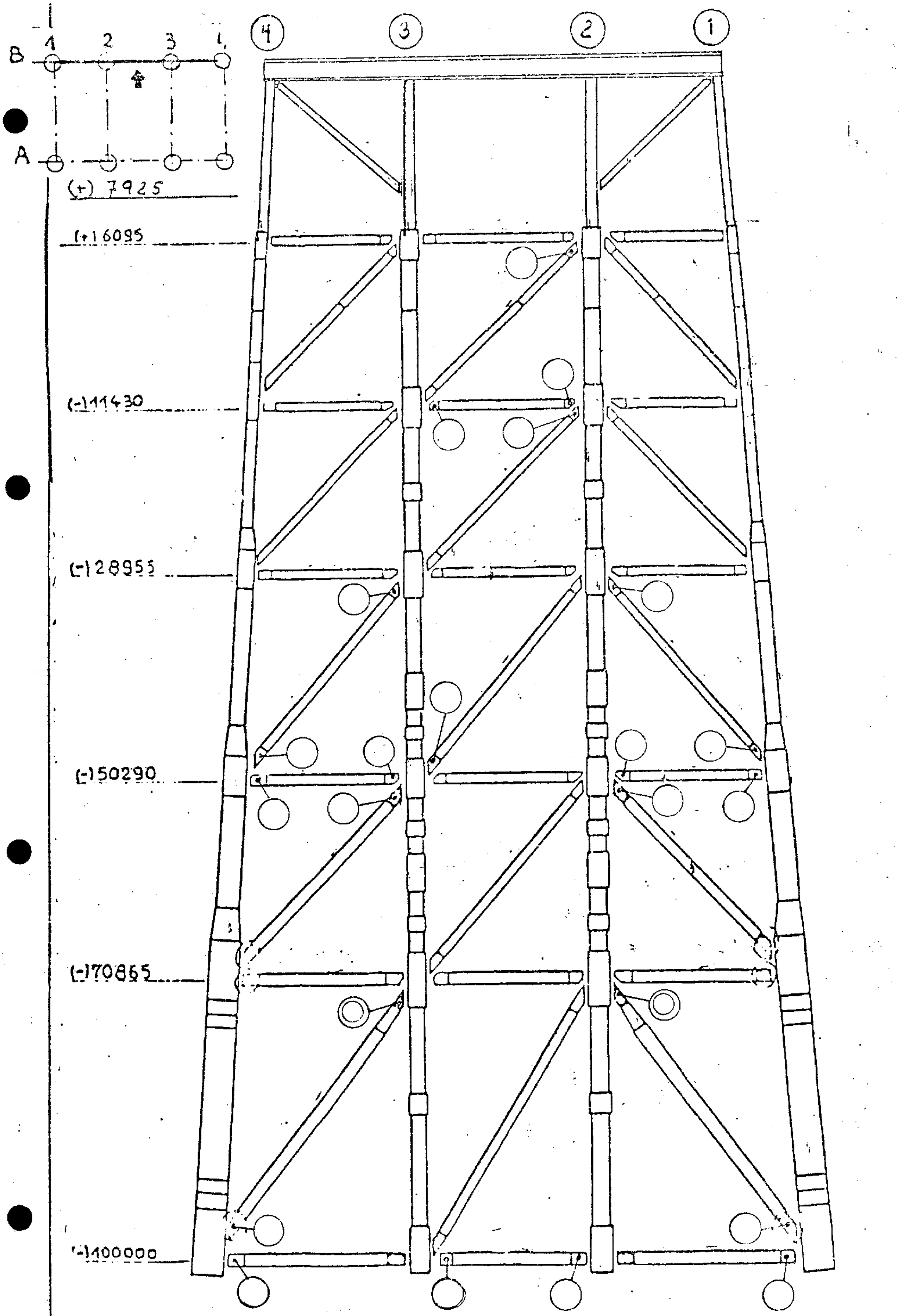
Siège : 49 avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (-) 100000	Dessiné	Le 21.08.75
		Visa	Echelle 1/300
		Nº Fo 454.699	12/12

2. Member End - Static Stress

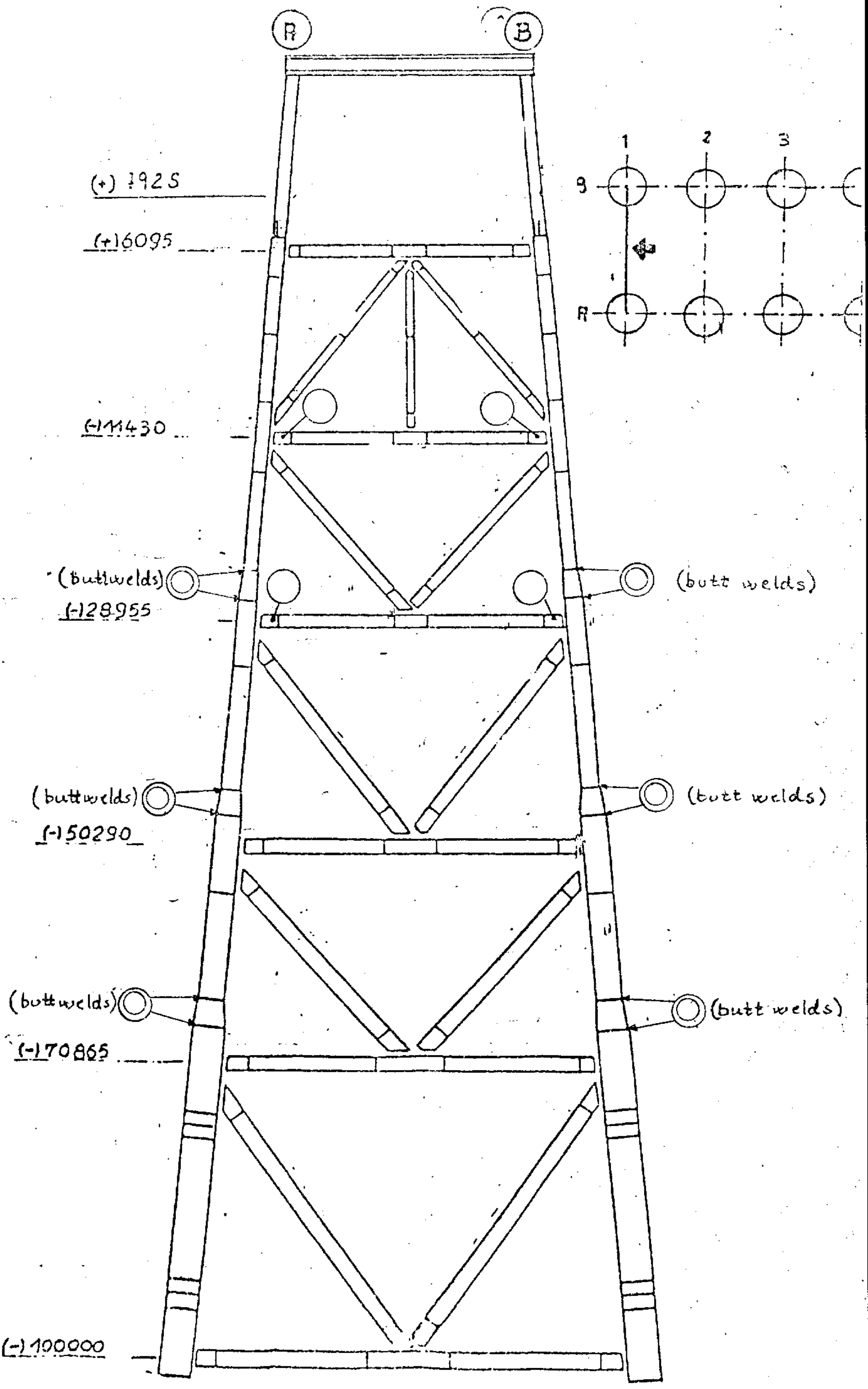
The following figures describe the most heavily stressed member ends and member sections in the structure. It must be noted this marking refers to nominal static axial-plus bending stress only. Thus no attempt has been made to show hot-spot stresses in the following figures. The corner legs are generally highly stressed, and particular attention should be paid to the transition welds between circular and cylindrical sections. These parts of the legs are generally stiffened by circular as well as longitudinal (parallel to the axis of the member) stiffeners.

The areas or member ends marked with a double circle (◎) have stresses equal to or above 20 KSI in the extreme condition.

The areas or member ends marked with a single circle (○) have stresses between 18 or 20 KSI in the extreme condition.



4
3
2
1
B
A
1
2
3
4
(+) 7925
(+)16095
(-)11430
(-)28955
(-)50290
(-)70865
(-)100000



R
B
1
2
3
9
R
(+) 1925
(+)6095
(-)11430
(buttwelds)
(butt welds)
(-)28955
(buttwelds)
(butt welds)
(-)50290
(buttwelds)
(butt welds)
(-)70865
(-)100000

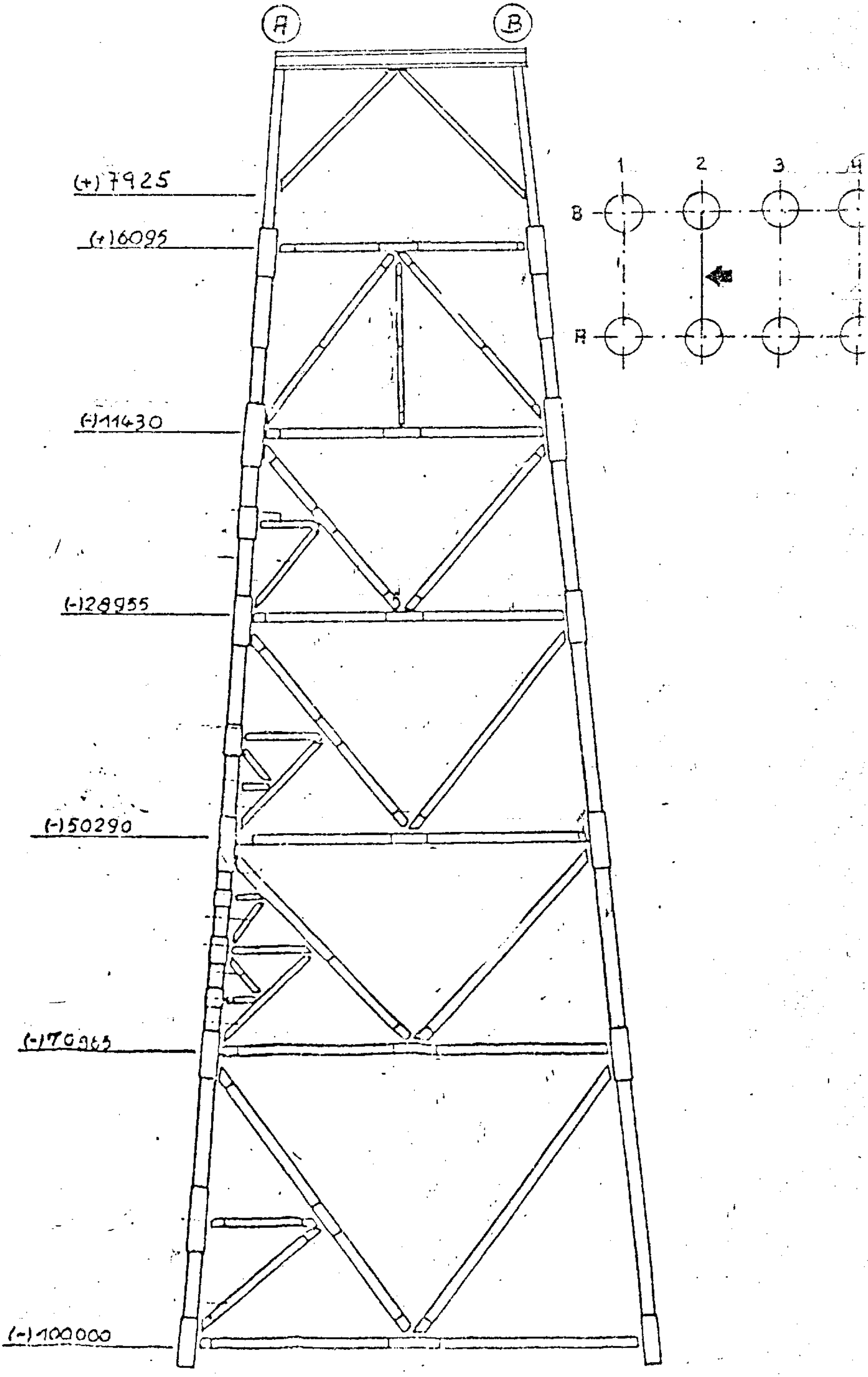
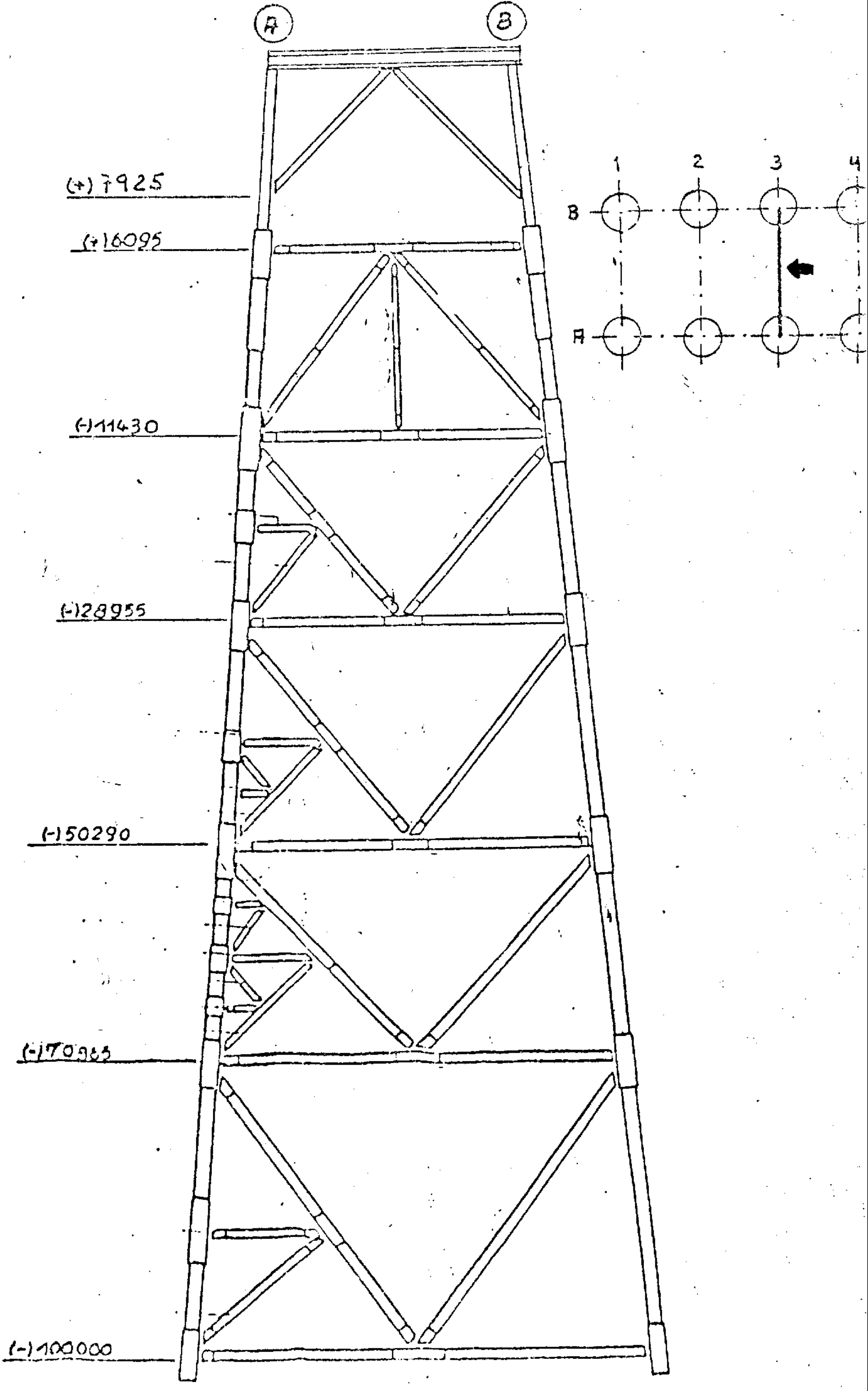
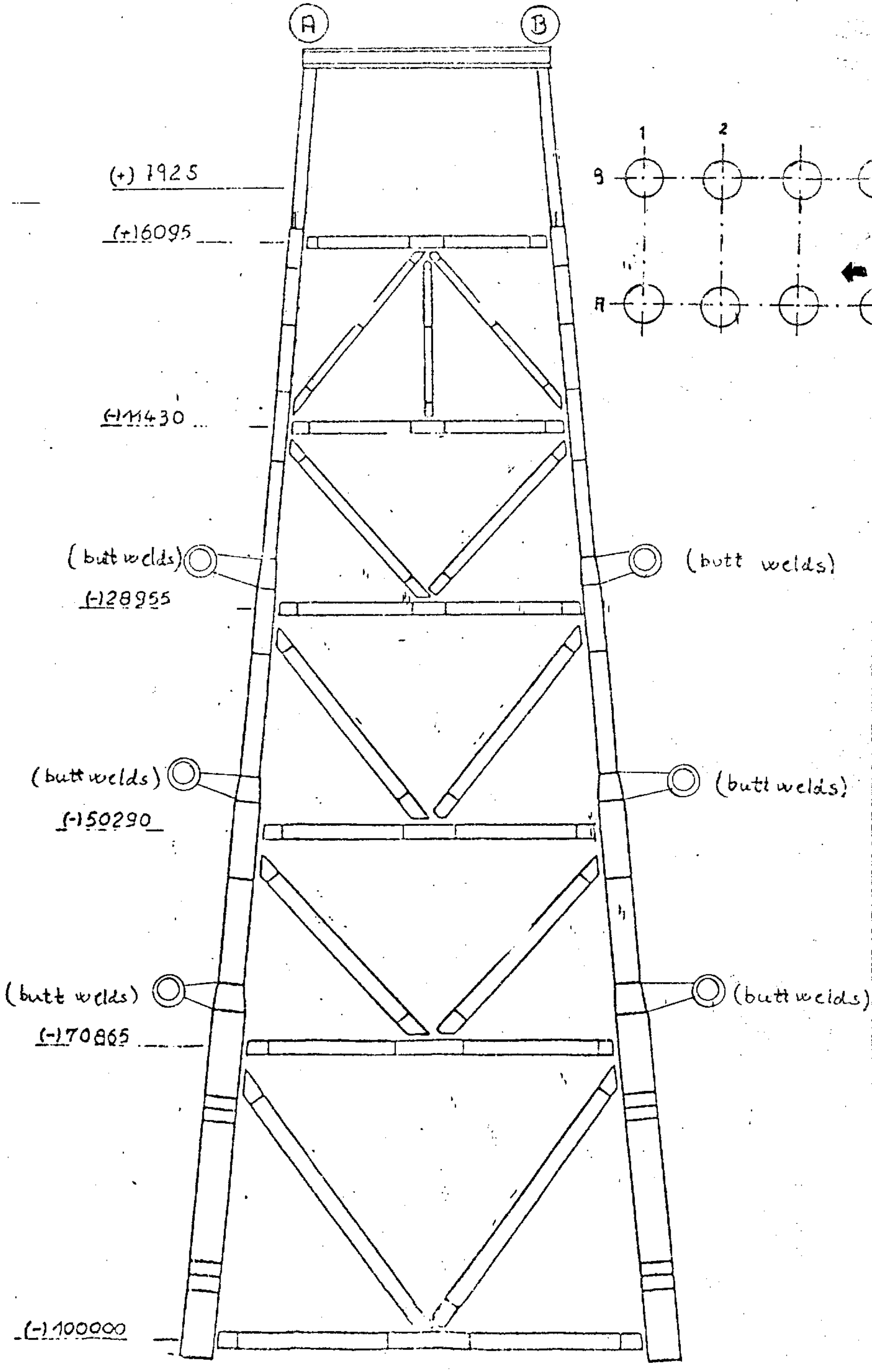


Fig 2.4


A
B
(+)7925
(+)16095
(-)11430
(-)28955
(-)50290
(-)70965
(-)100000
1
2
3
4
B
A



FRIGG DP2 FILE 4

Fig 2.6

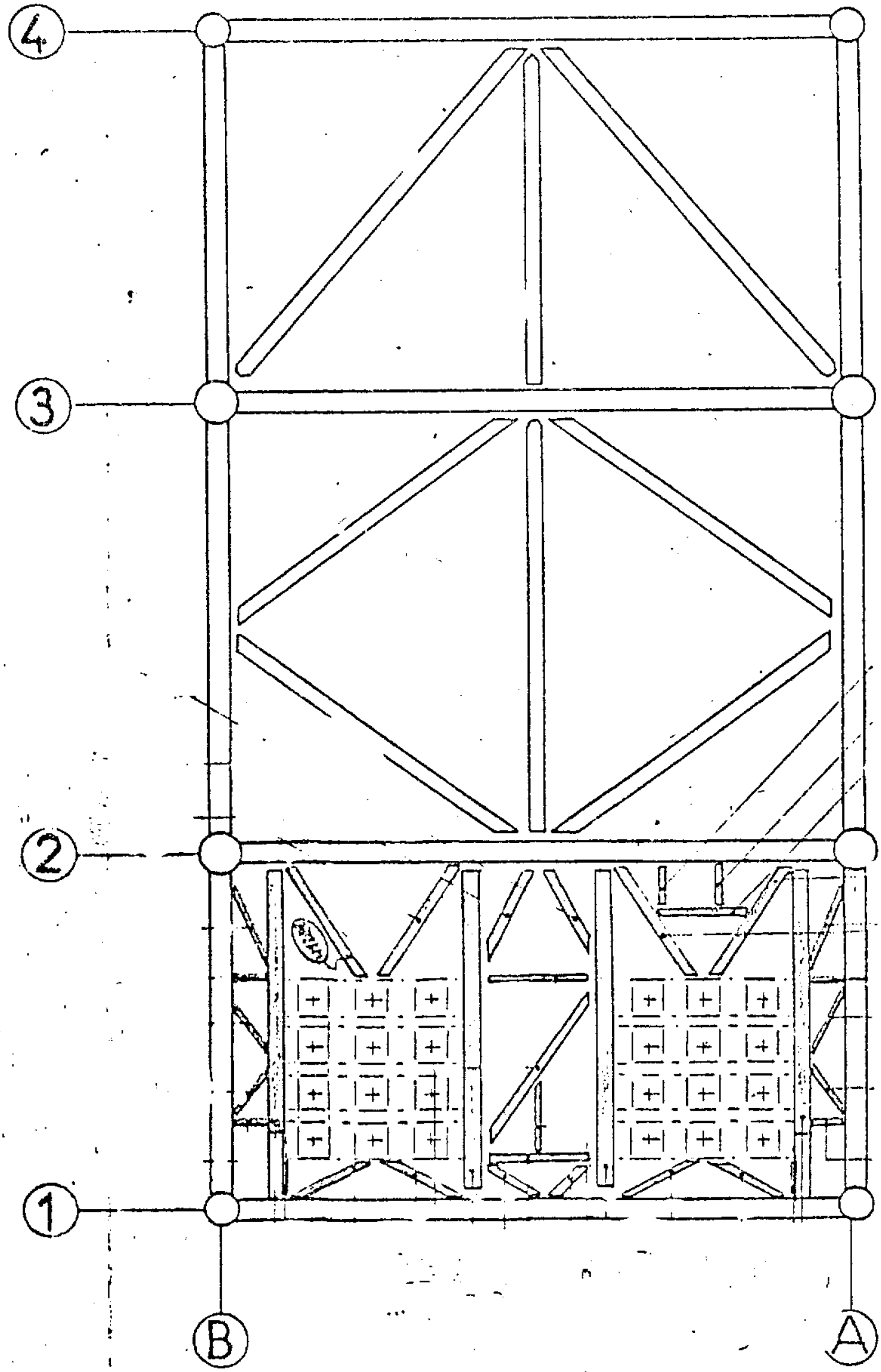


Fig 2.7

Siège : 4 bis avenue Hoche - PARIS Établissement de CHERBOURG	FRIGG DP2 Niveau (+) 6095	Dessiné : Le : 18.08.75 Visa : Échelle : 1/250 N° Fo 45+ 699 1/13

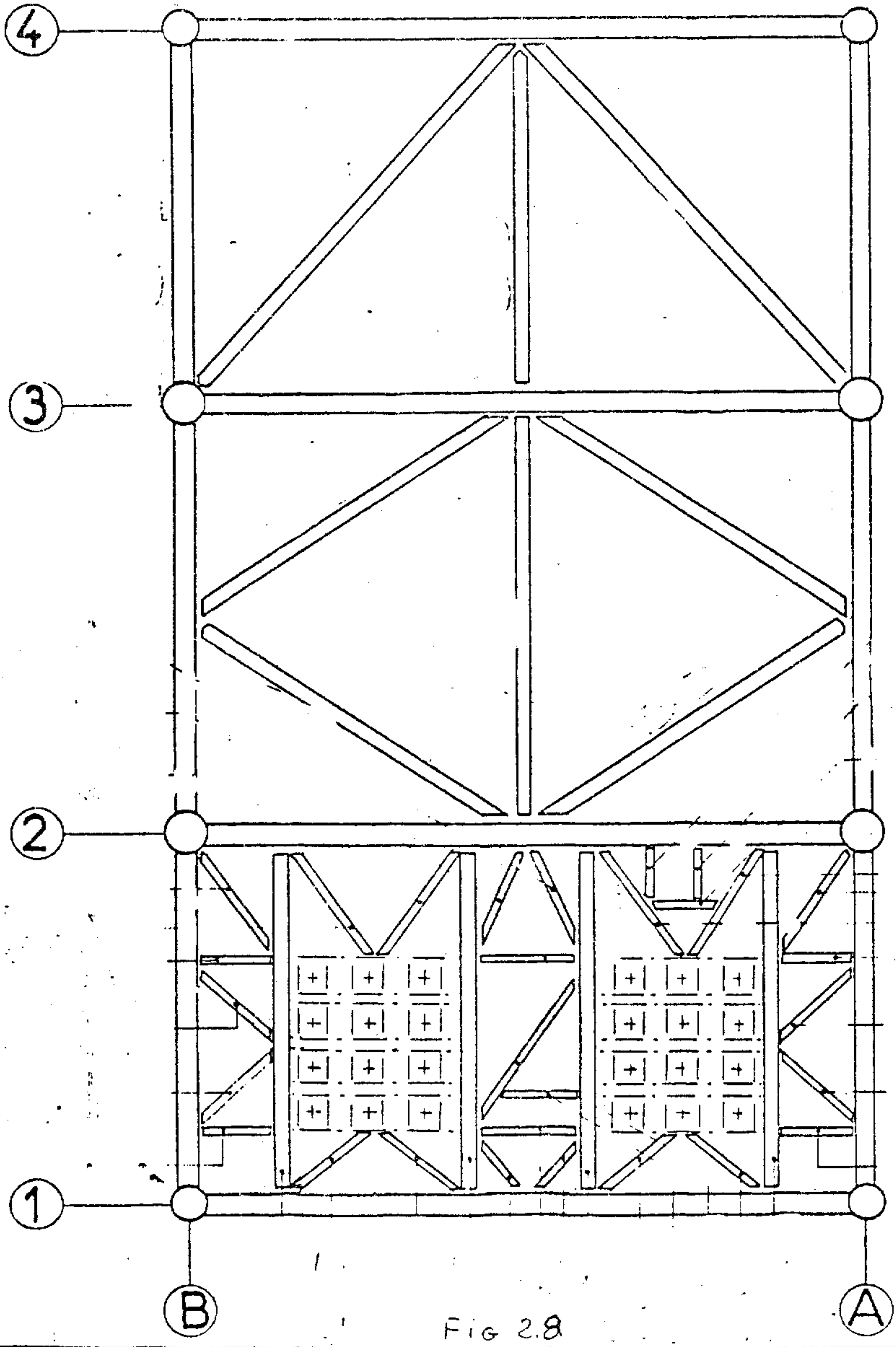


Fig 2.8

Sièg[e] : 49[b] avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (-) 11430	Dessiné :	Le 19.08.75
		Visa :	Echelle : 1/250
		Nº Fo 454.699 8/12	

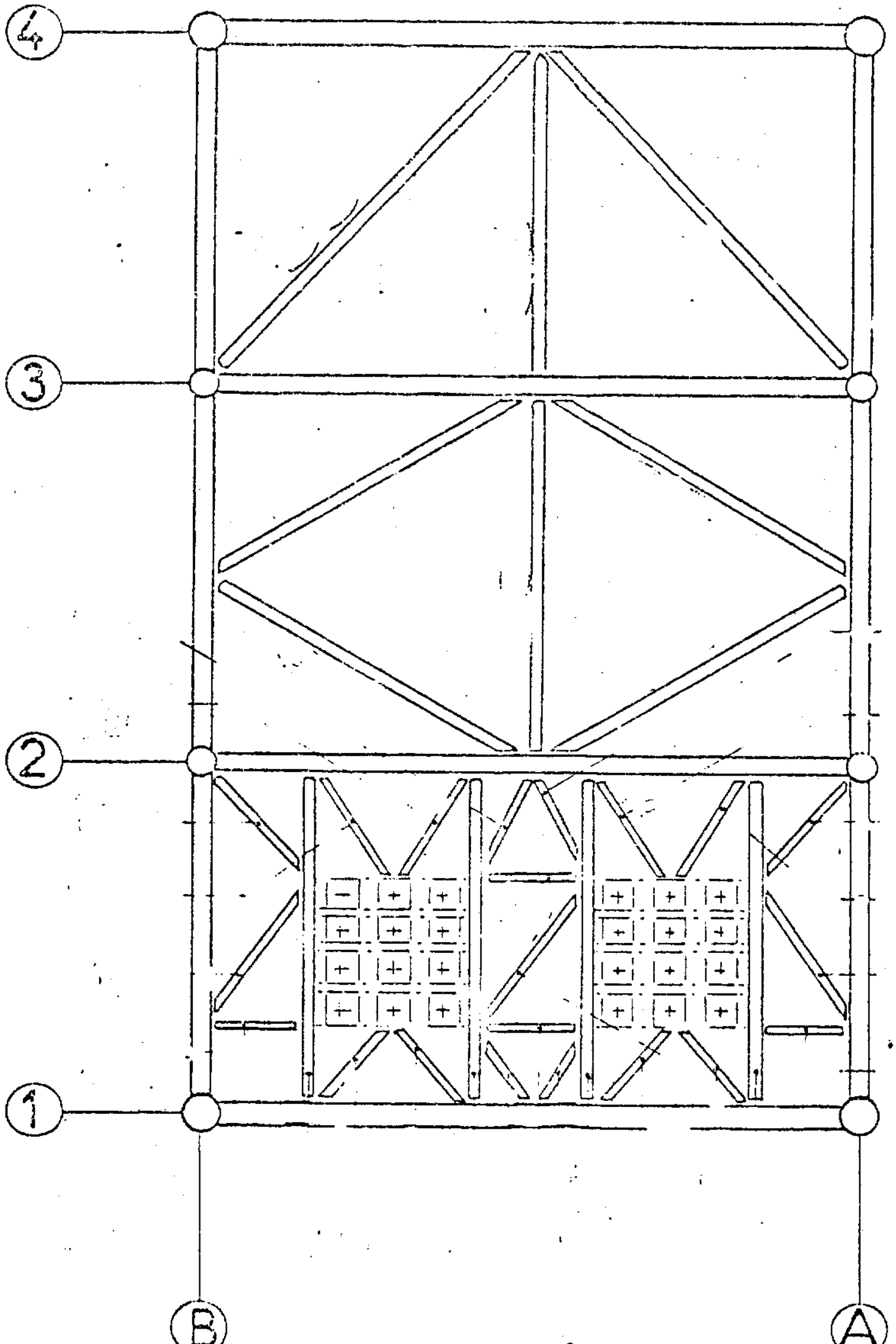


Fig 2.9

Siège : 49b avenue Hoche - PARIS
Etablissement de CHERBOURG

FRIGG DP2
Niveau (-)28955

Dessiné :	Le : 19.08.75
Visa :	Echelle : 1/300
N° Fo 454.699 9/12	

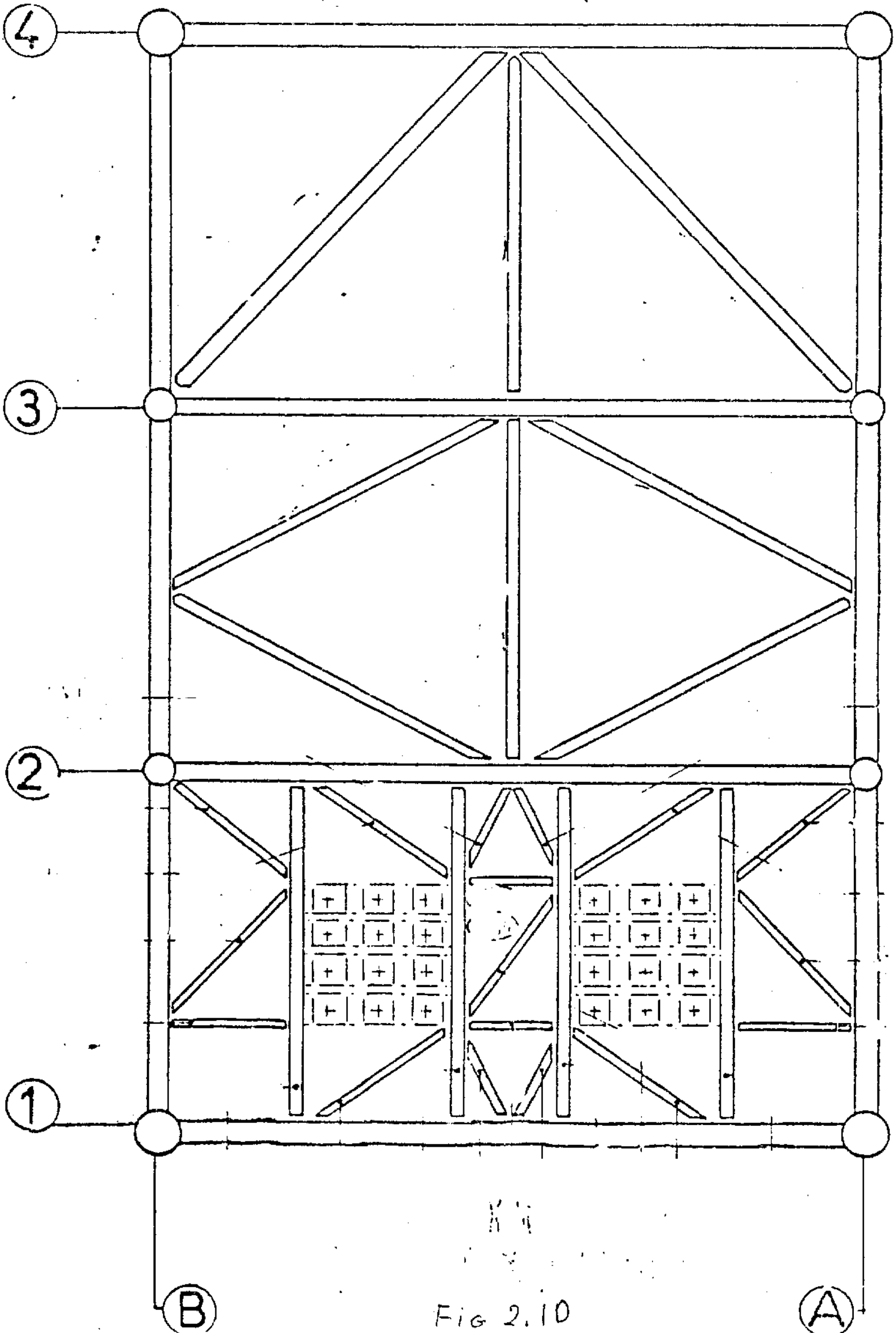


Fig 2.10

Siège : 49b avenue Hoche - PARIS
Etablissement de CHERBOURG

FRIGG DP2
Niveau (-) 50290

Dessiné :	Le 21.08.75
Visa :	Echelle : 1/300
N° Fo 454 699 10/1	

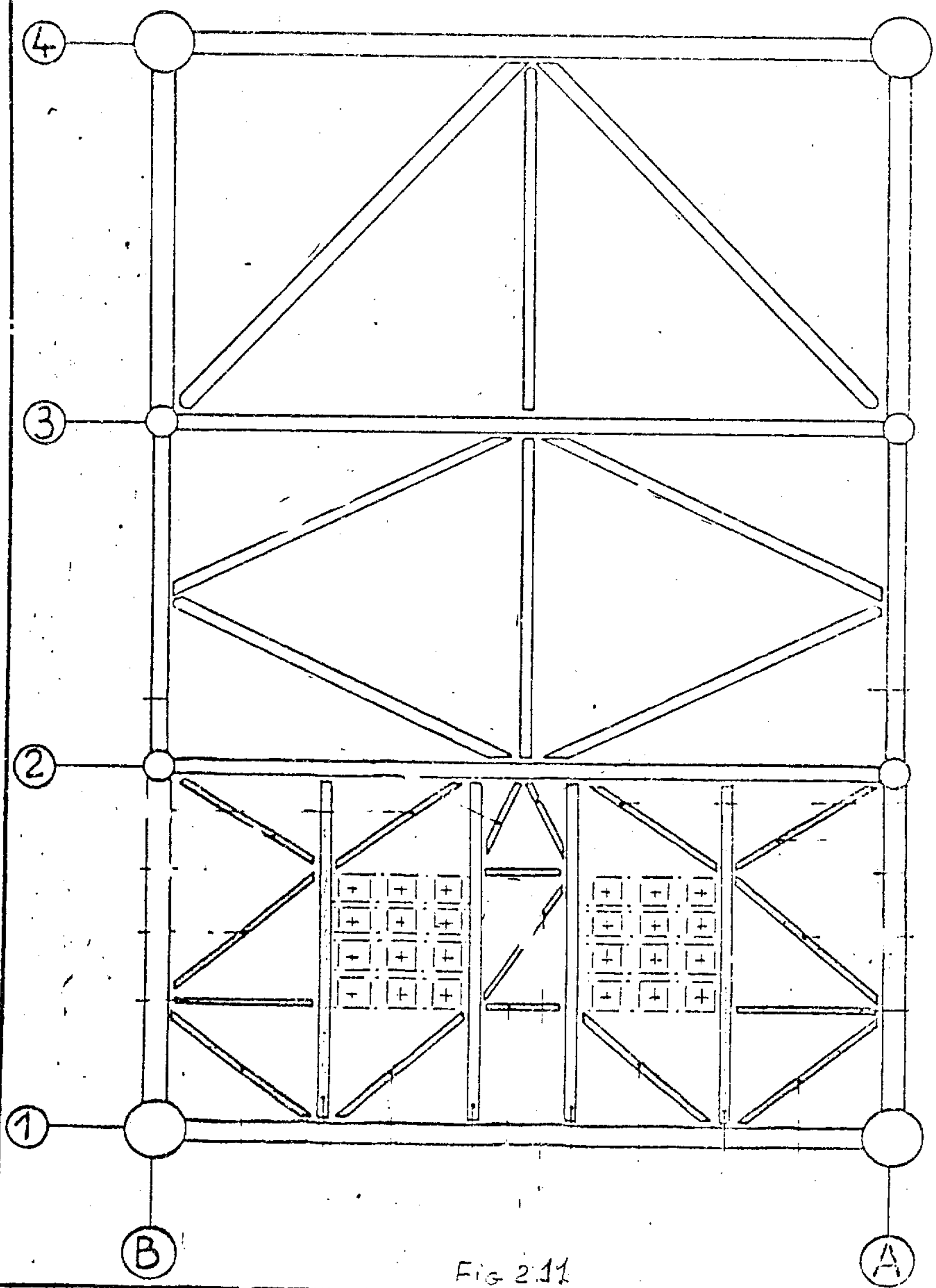


Fig 2.11

Siège : 49bis avenue Hoche - PARIS Établissement de CHERBOURG	FRIGG DP2 Niveau (-)70865	Dessiné — Le [illegible] Visa : — Echelle : 1/[illegible] No Fo [illegible]

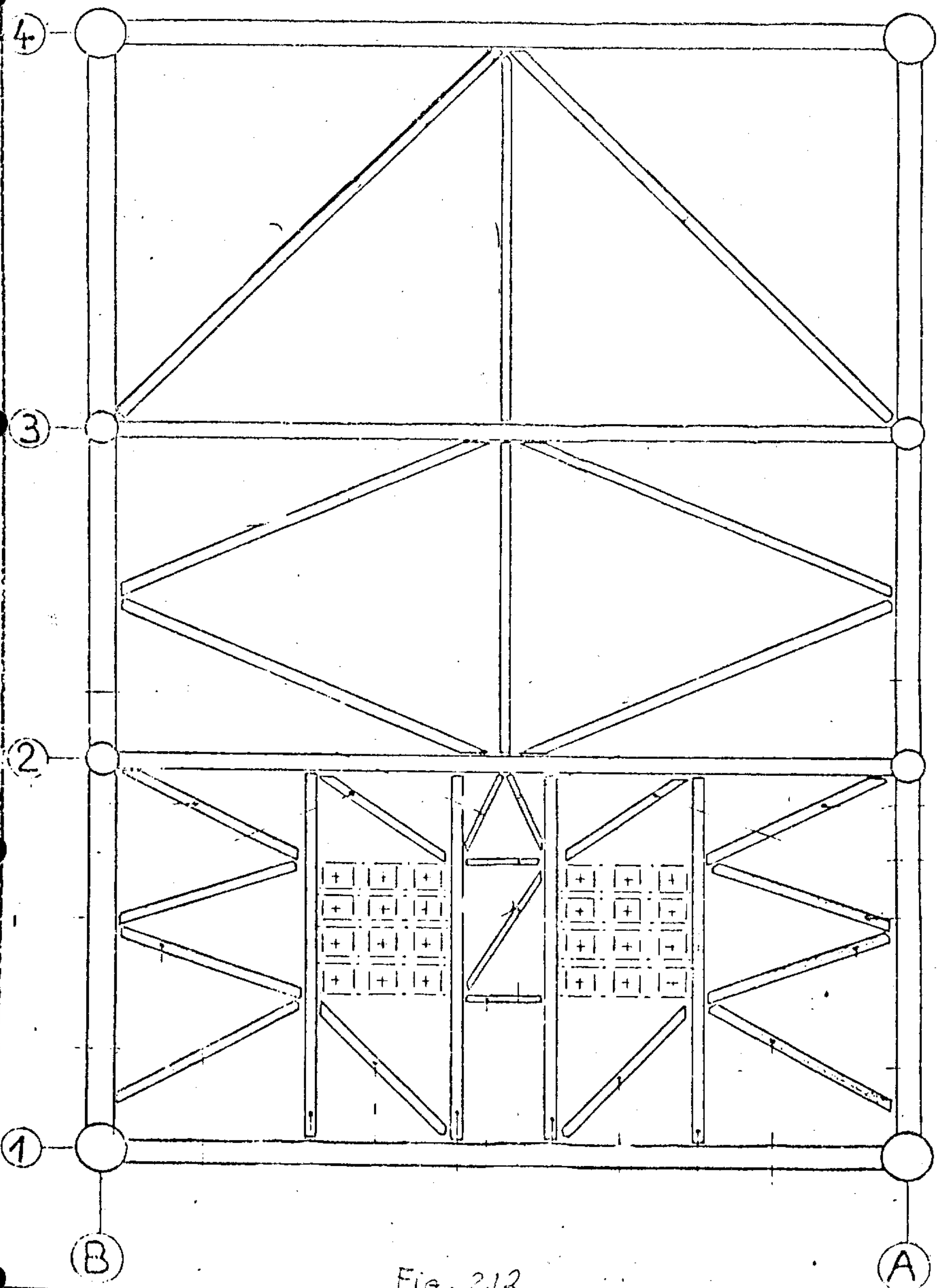


Fig. 212

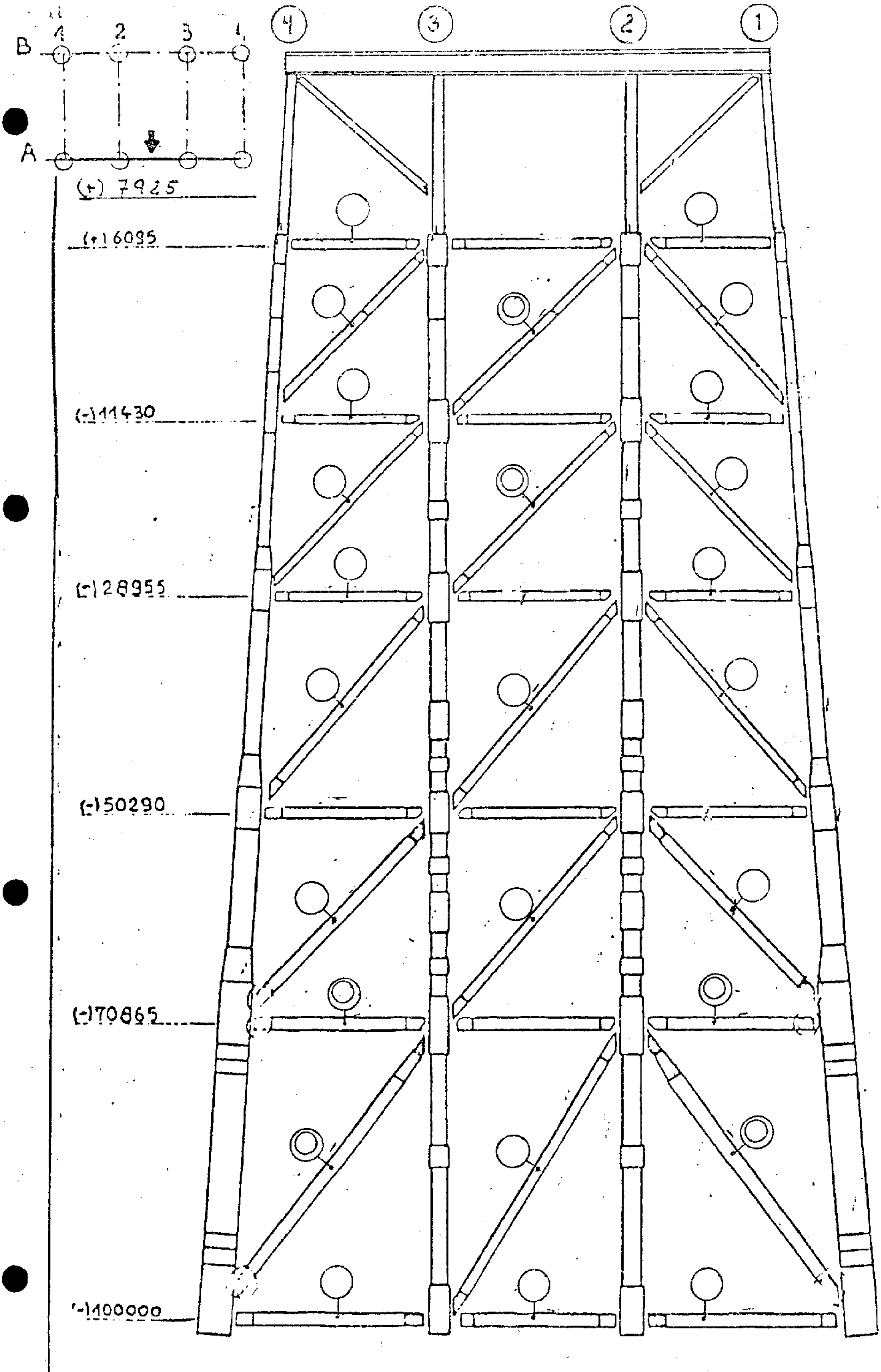
Siège : 40 avenue Hoche - PARIS	FRIGG DP2	Dessiné	Le 21.08.75
	Niveau (-) 100000	Visa	Echelle 1/300
		N° Fo 154.699 12/12	

3. Member Buckling

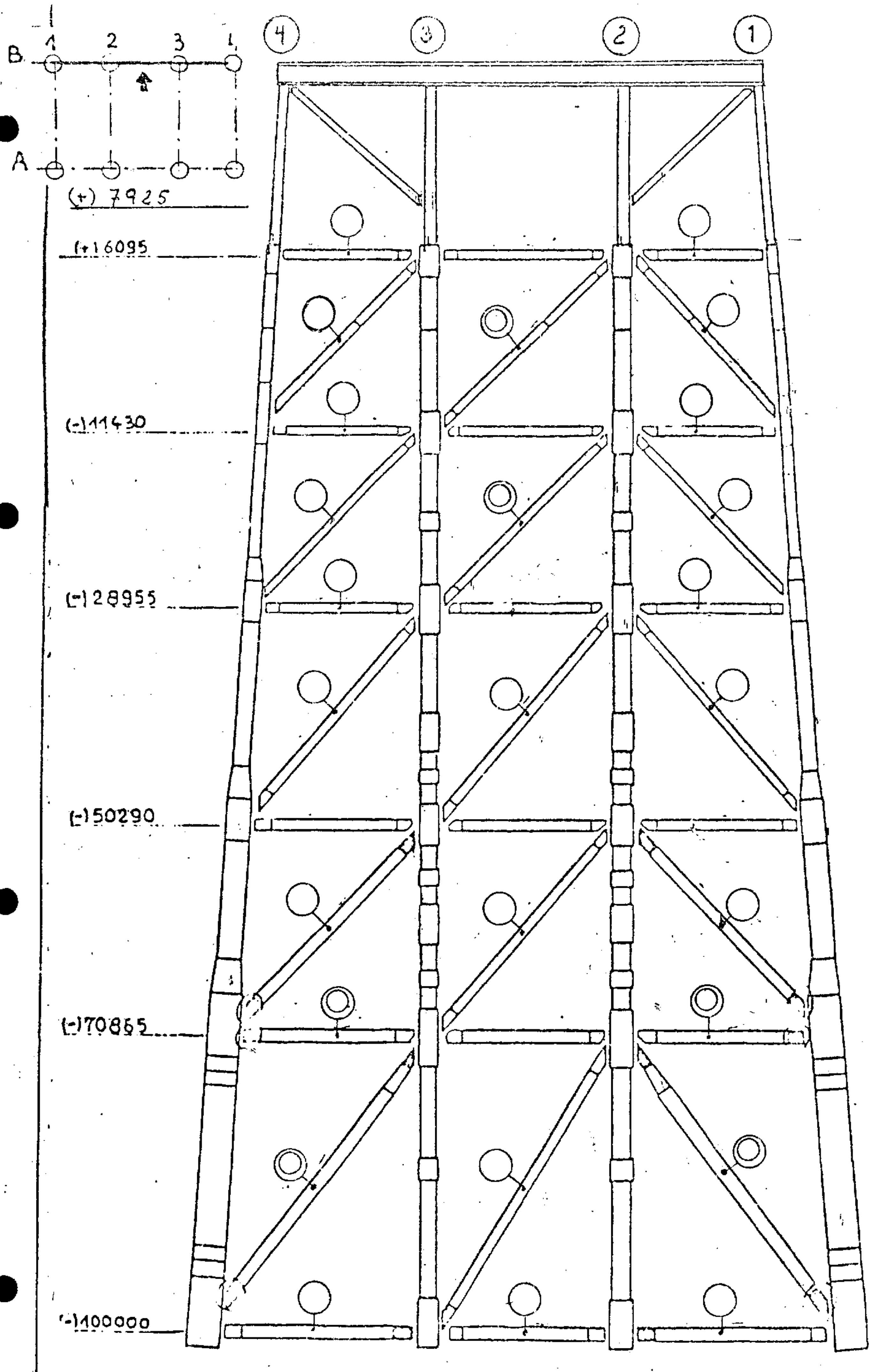
The following figures define the most critical members with respect to overall member buckling, i.e. not buckling or collapse due to external water pressure. The criteria of AISC "Manual of Steel Construction" has been used as a basis for the following figures; more specifically the formulae 1.6,1a has been used.

Members marked with a double circle (◎) have a buckling ratio equal to or greater than 0.9 in the extreme environmental condition.

Members marked with a single circle (○) have a buckling ratio between 0.7 and 0.9 in the extreme environmental condition.

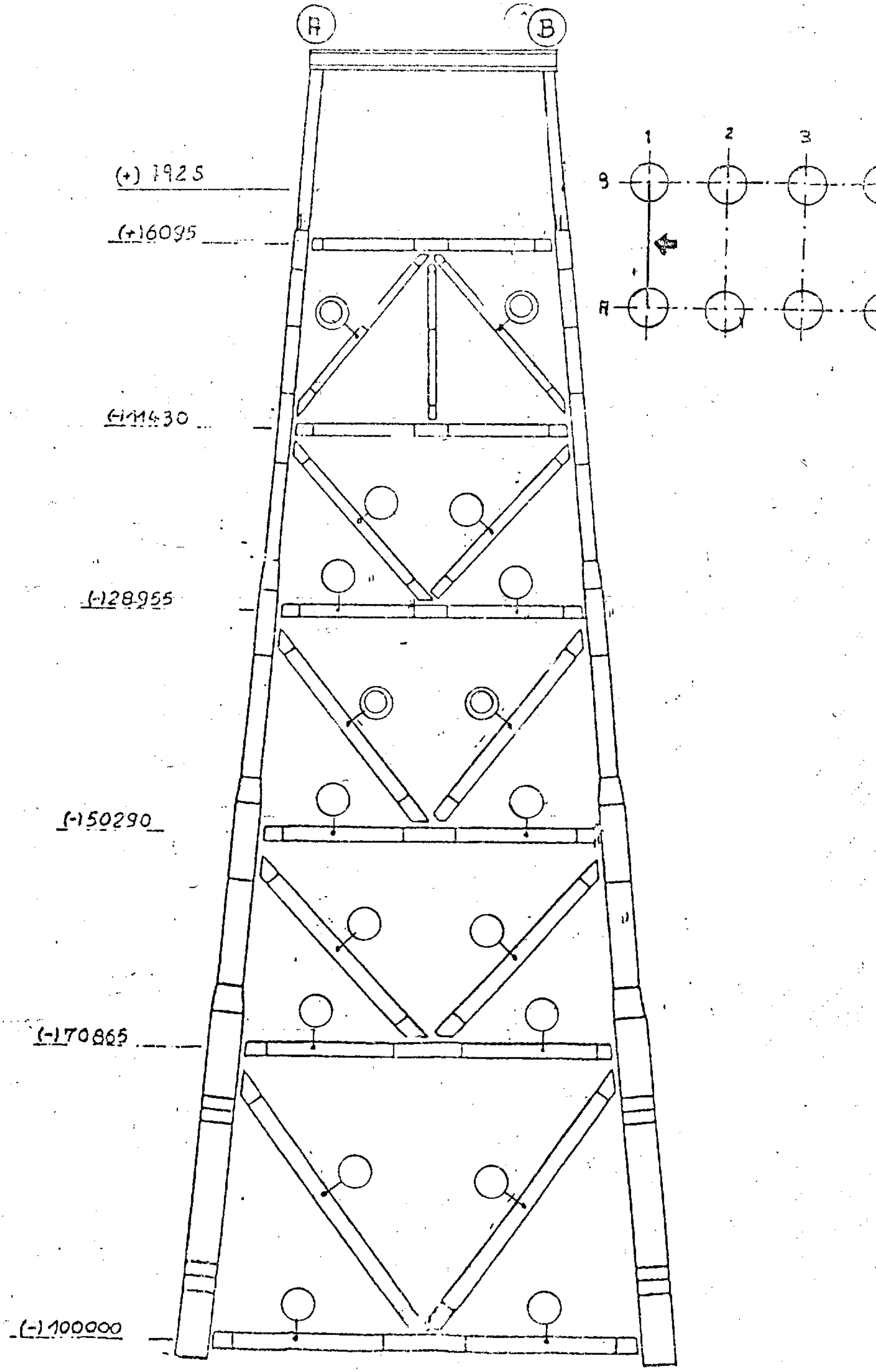


FRIGG DP2 FACE A



FRIGG DP2 FACE B

Fig 3.2



A
B
(+) 1925
(+)6095
(-)11430
(-)28955
(-)50290
(-)70865
(-)100000
1
2
3
B
A

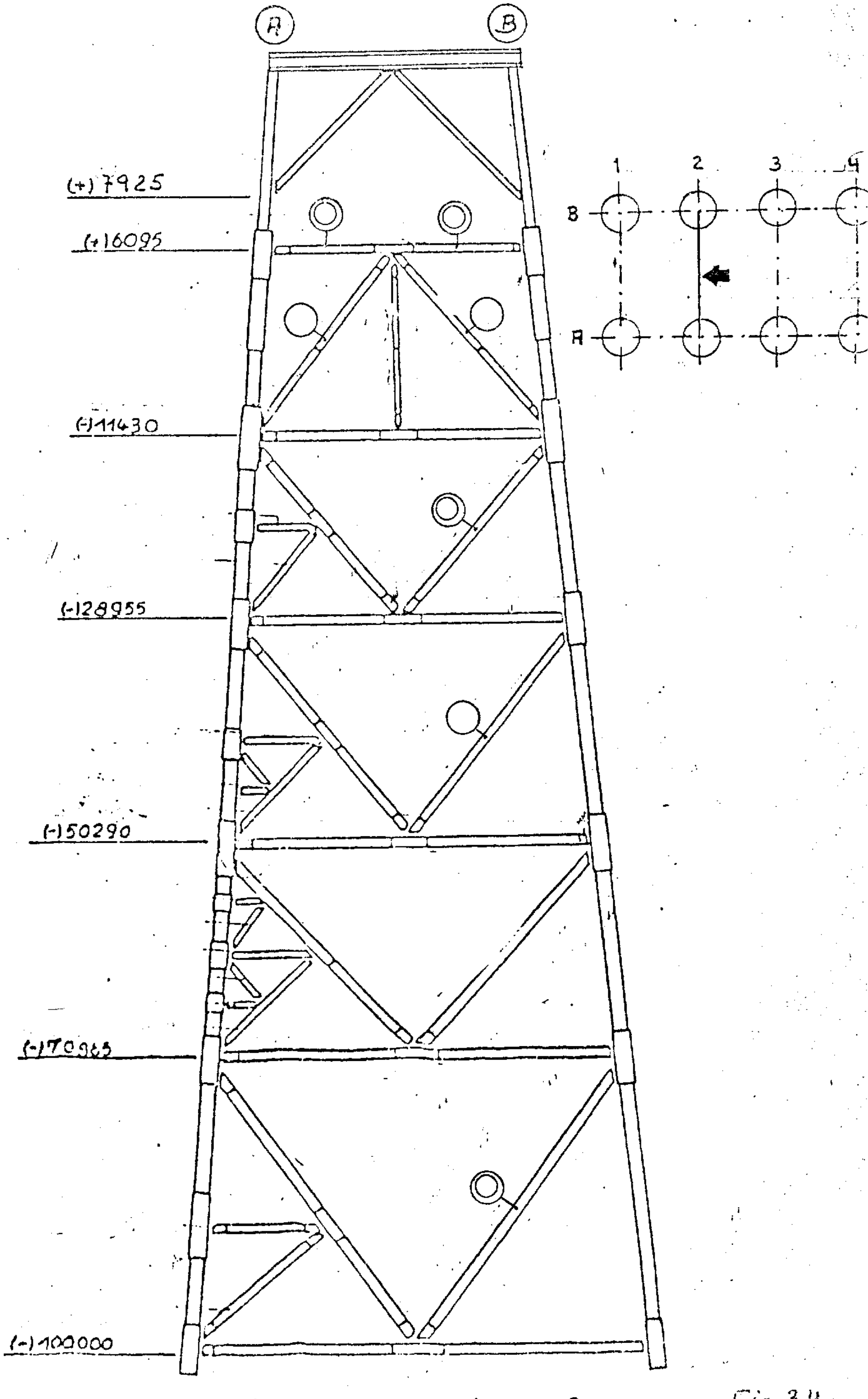


Fig. 3.4

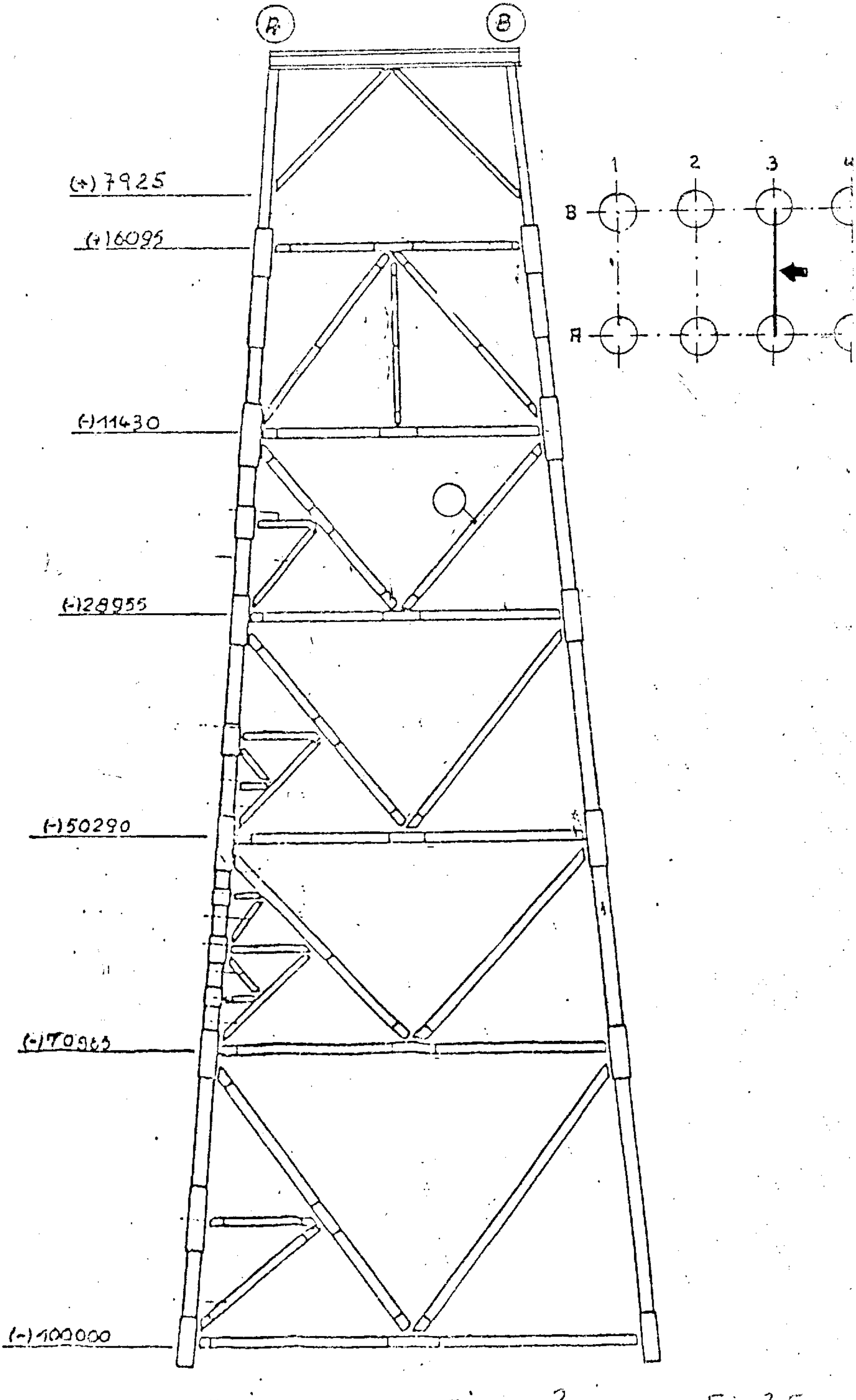
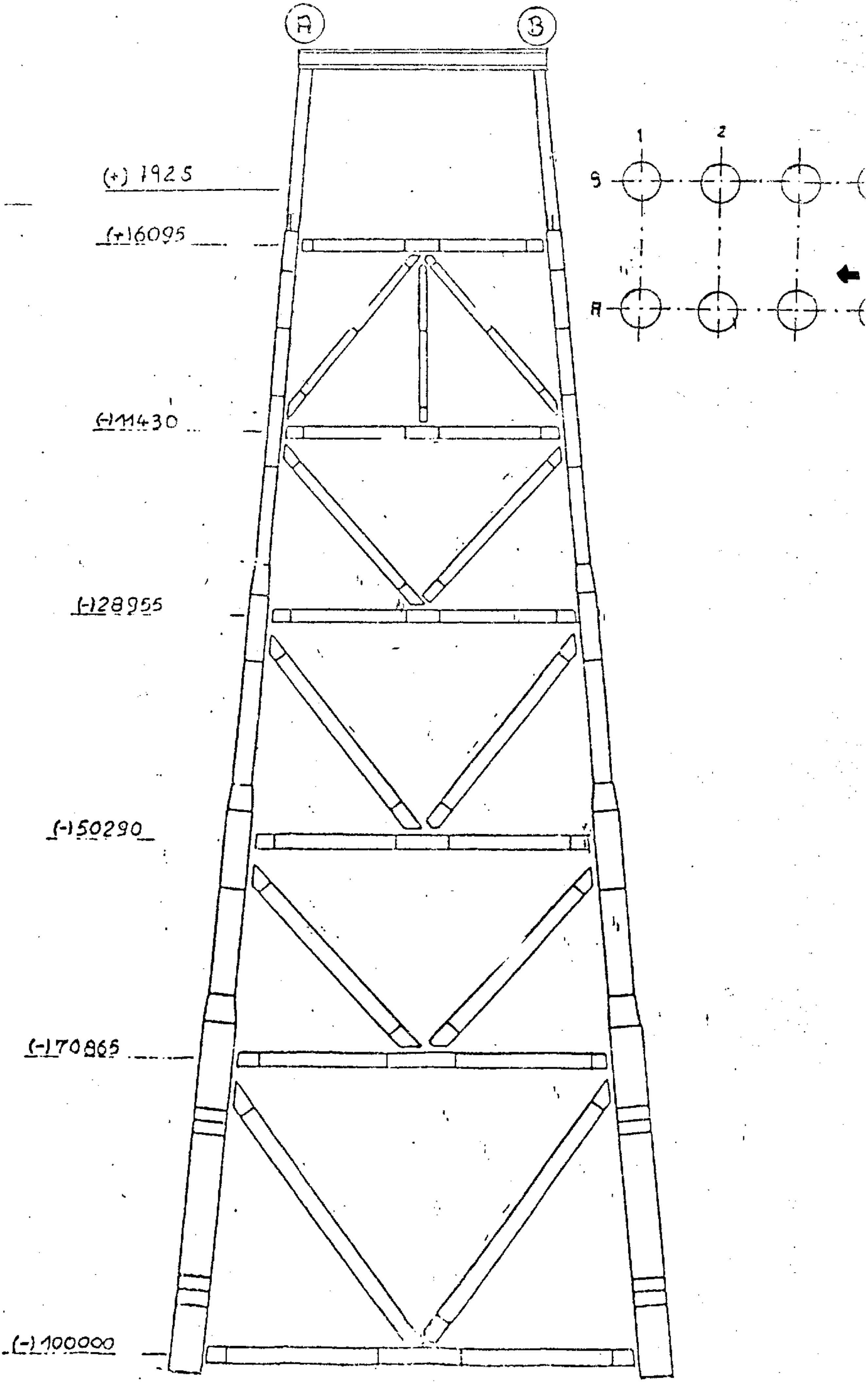


Fig 3.5



FRIGG DP2 FILE 4

Fig 3.1

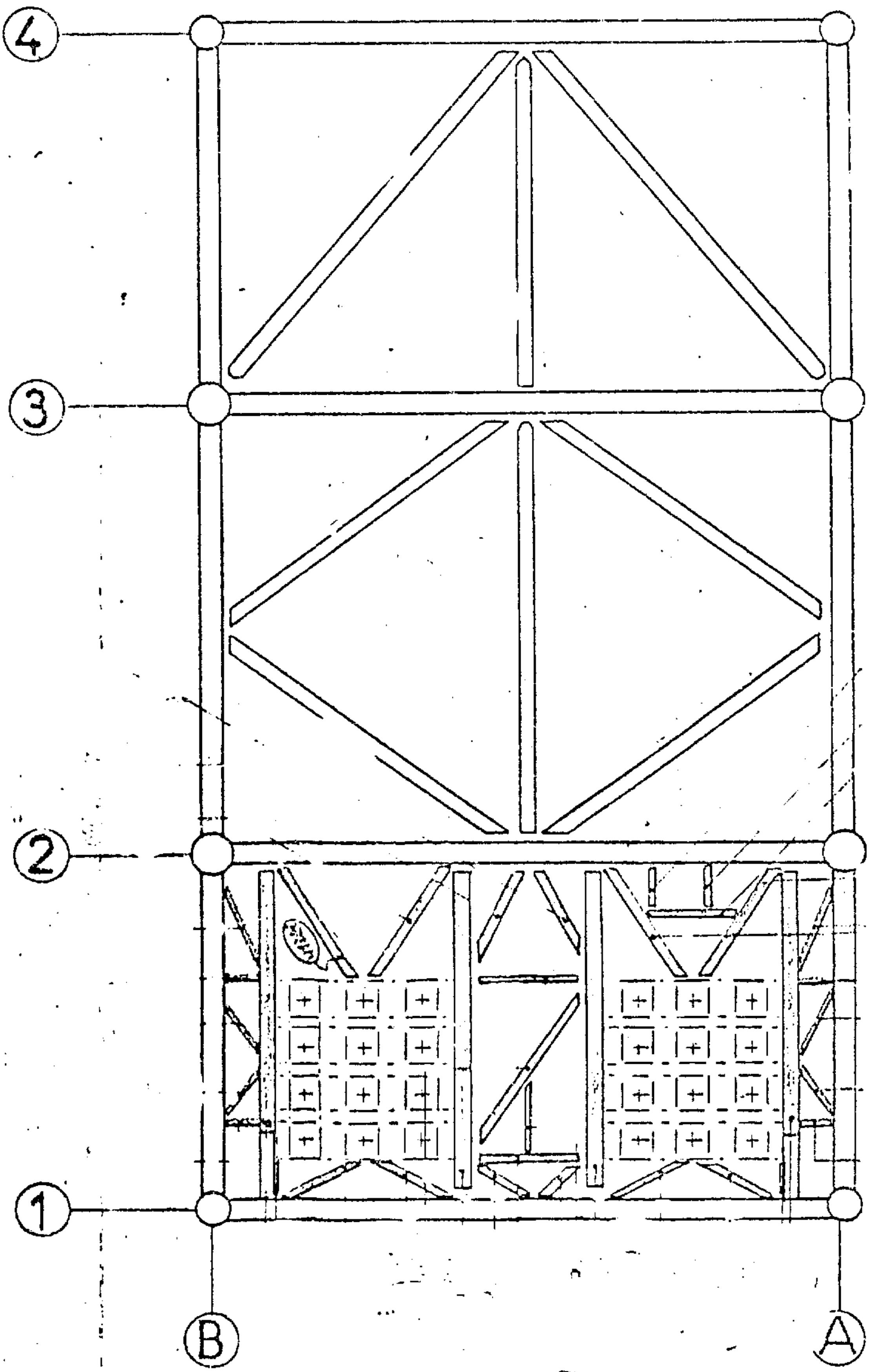


Fig. 3.7

Siège : 4bis avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (+) 6095	Dessiné : — Le : 19.08.75 Visa : — Echelle : 1/250 N° Fo 757 693 1/12

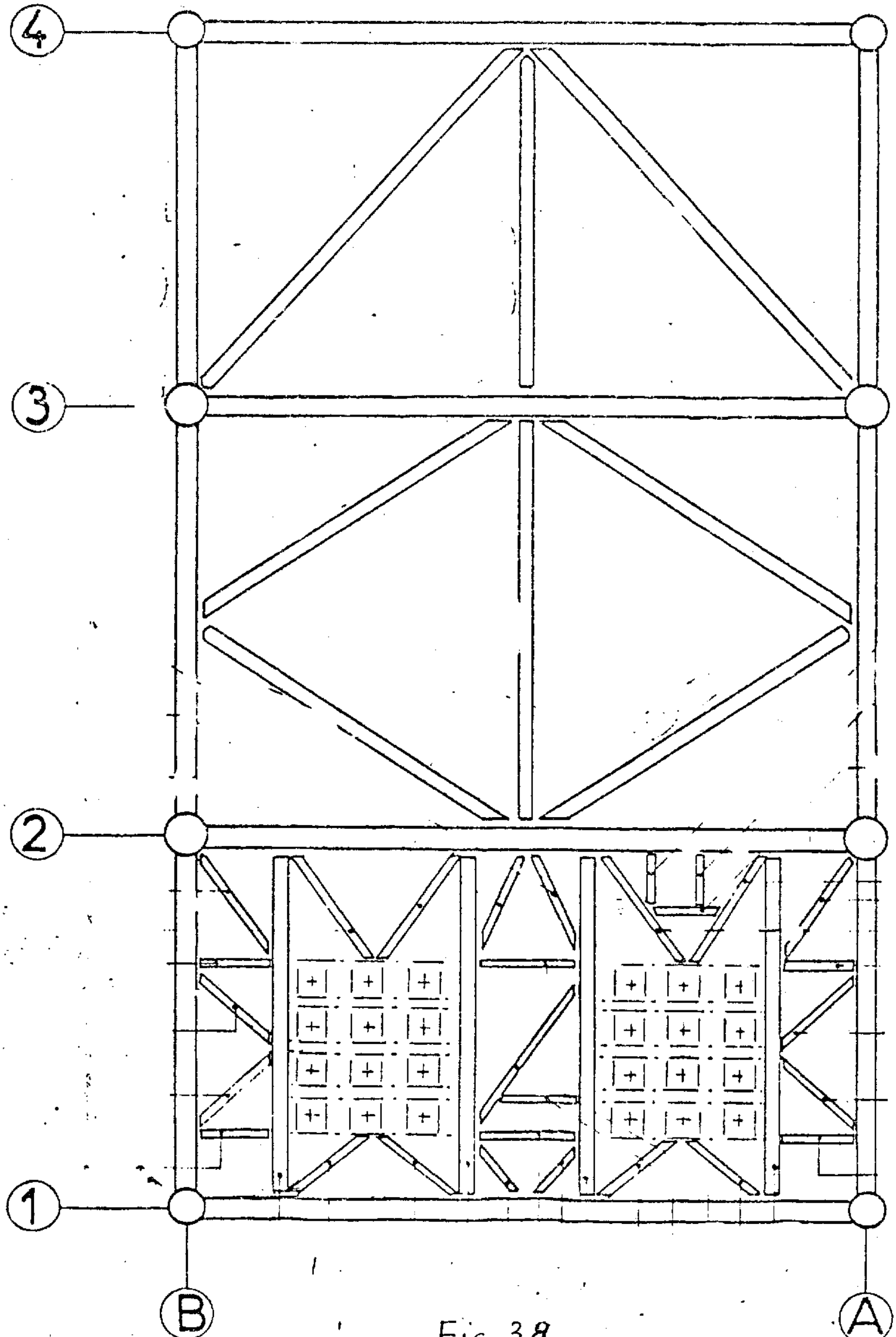


Fig 3.8

Siège : 49b avenue Hoche - PARIS
Établissement de CHERBOURG

FRIGG DP2
Niveau (-)11430

Dessiné : Le 19.08.75
Visa : Echelle : 1/250
N° Fo 454.699 8/12

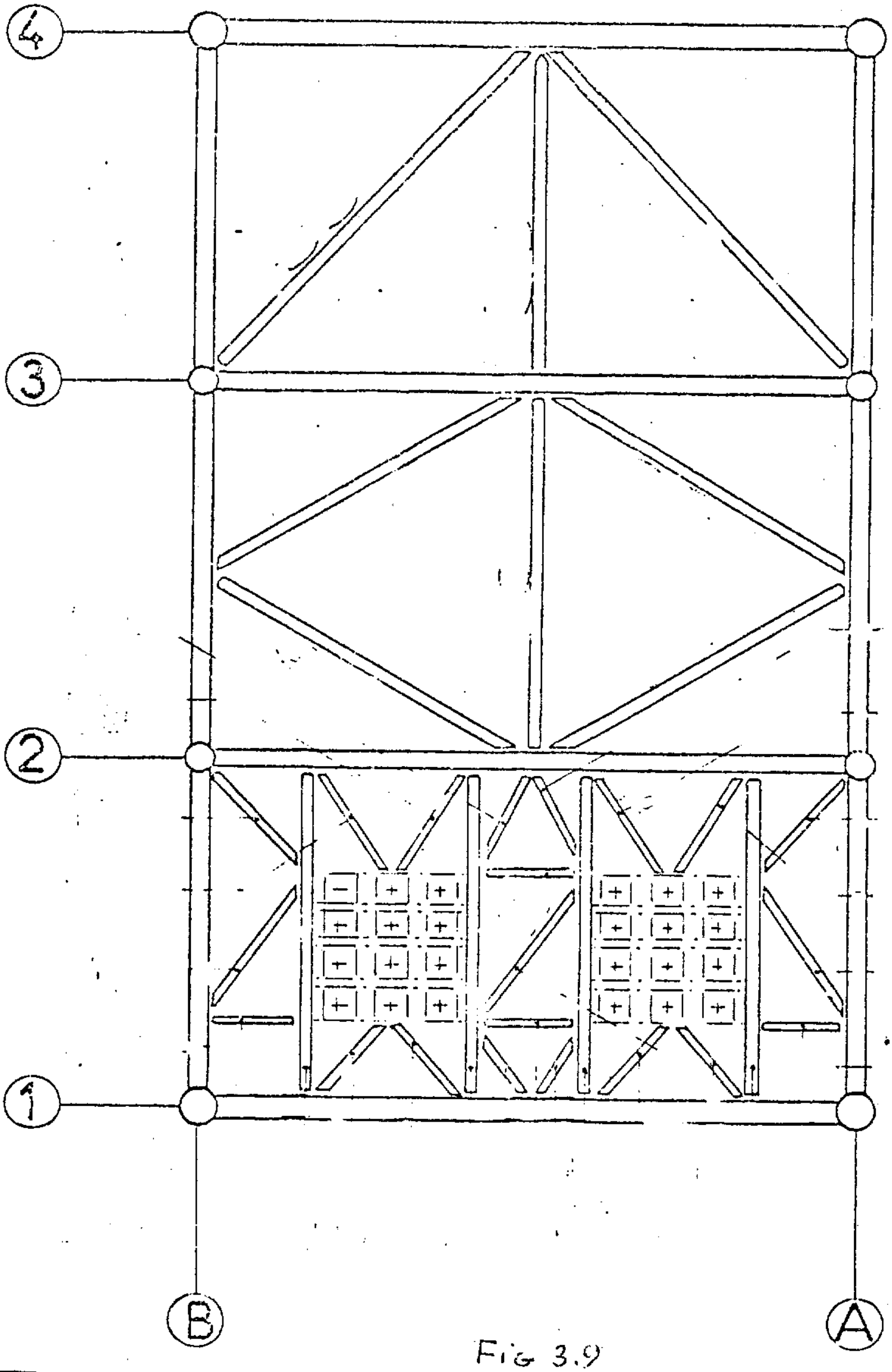


Fig 3.9

Siège : 49b avenue Hoche - PARIS
Établissement de CHERBOURG

FRIGG DP2
Niveau (-)28955

Dessiné :	Le : 18.08.75
Visa :	Echelle : 1/300
N° Fo 454.699 5/12	

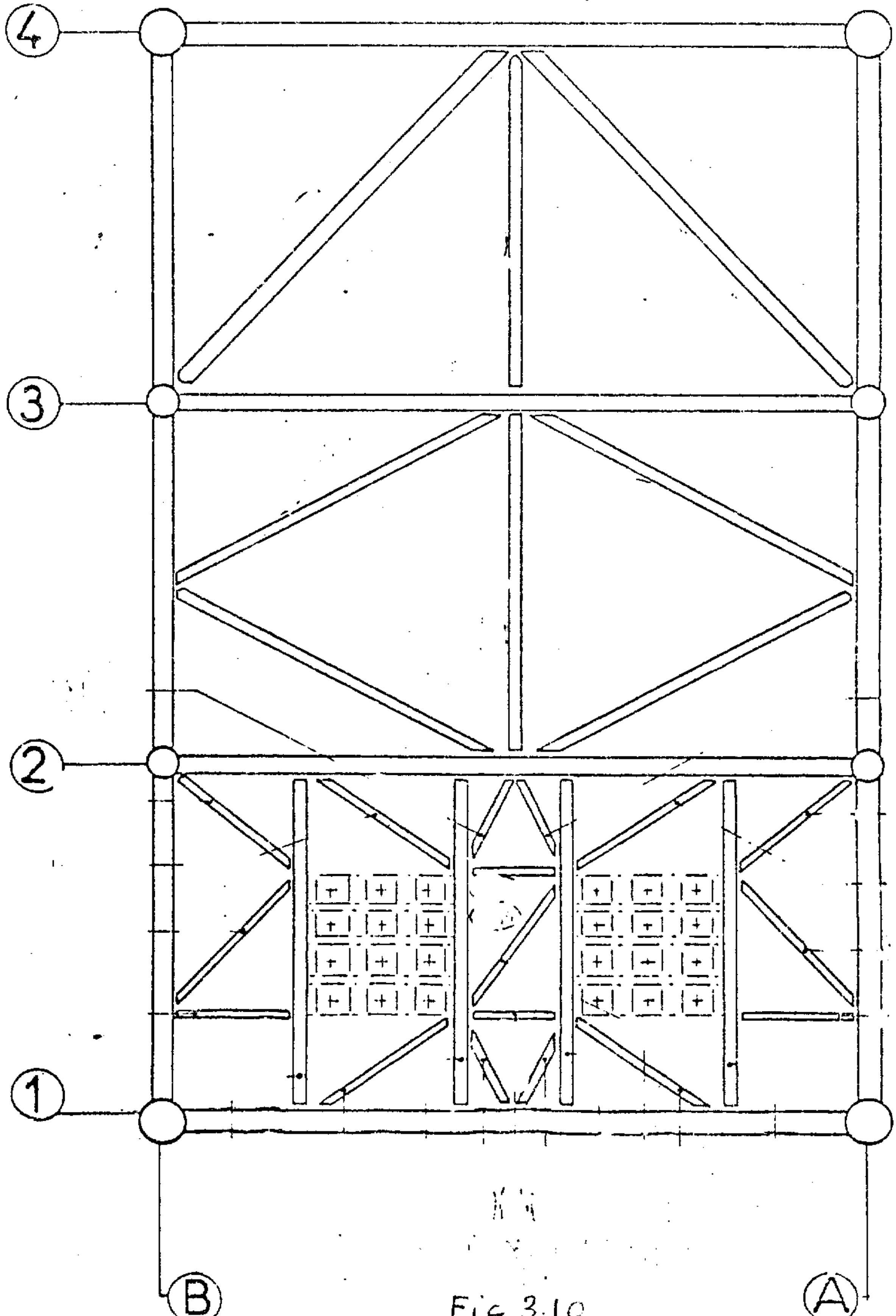


Fig 3.10

Siège : 40 bis avenue Hoche - PARIS
Établissement de CHERBOURG

FRIGG DP2
Niveau (-) 502 90

Dessiné :
Visa :
Le 21.08.7
Échelle : 1/300
N° Fo 454 699

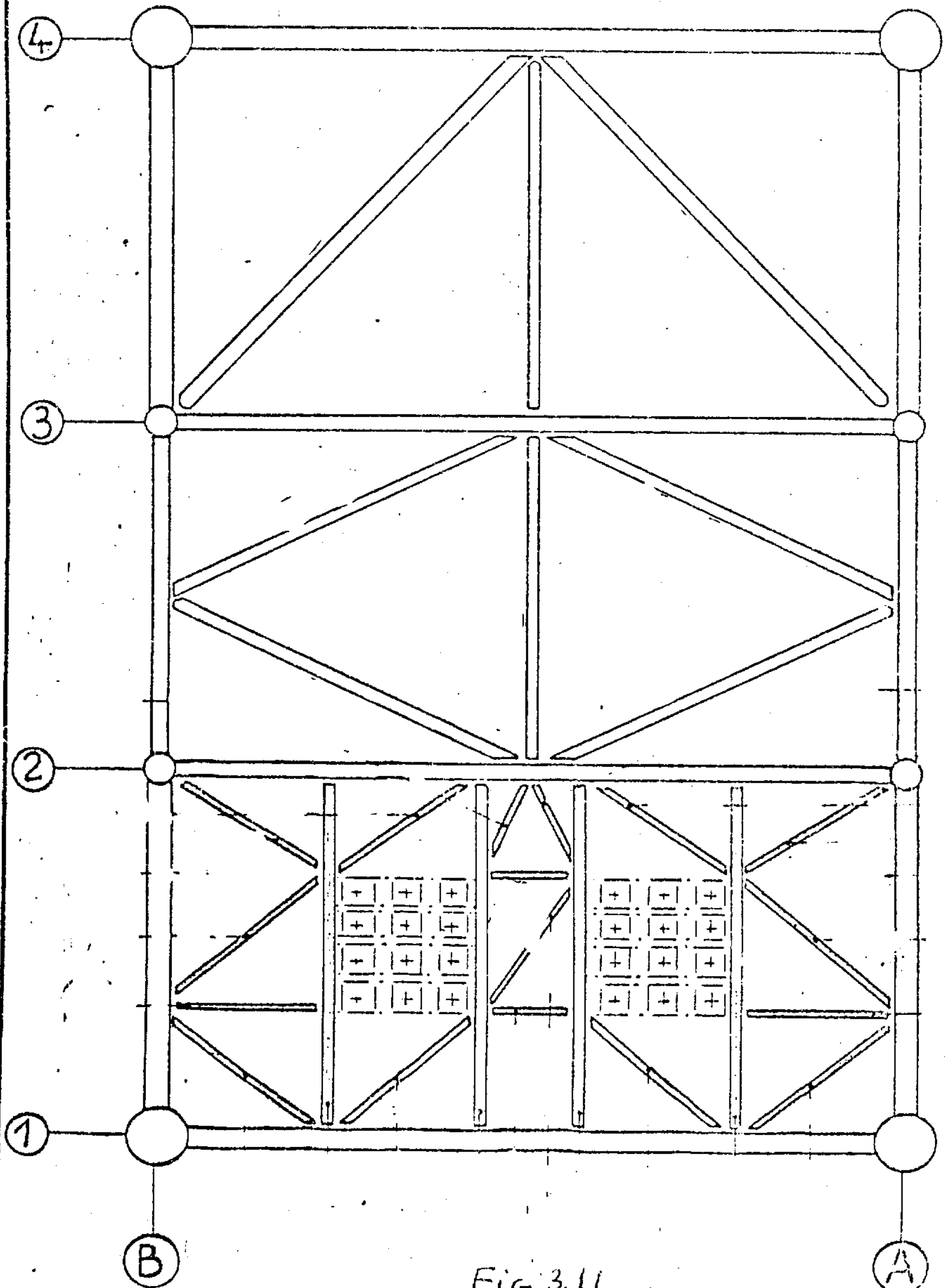


Fig. 3.11

Bieue 149bis avenue Hoche · PARIS
Etablissement de CHERBOURG

FRIGG DP2
Niveau (-) 70865

Dessiné :
Visa :
Echelle :
N° Fo

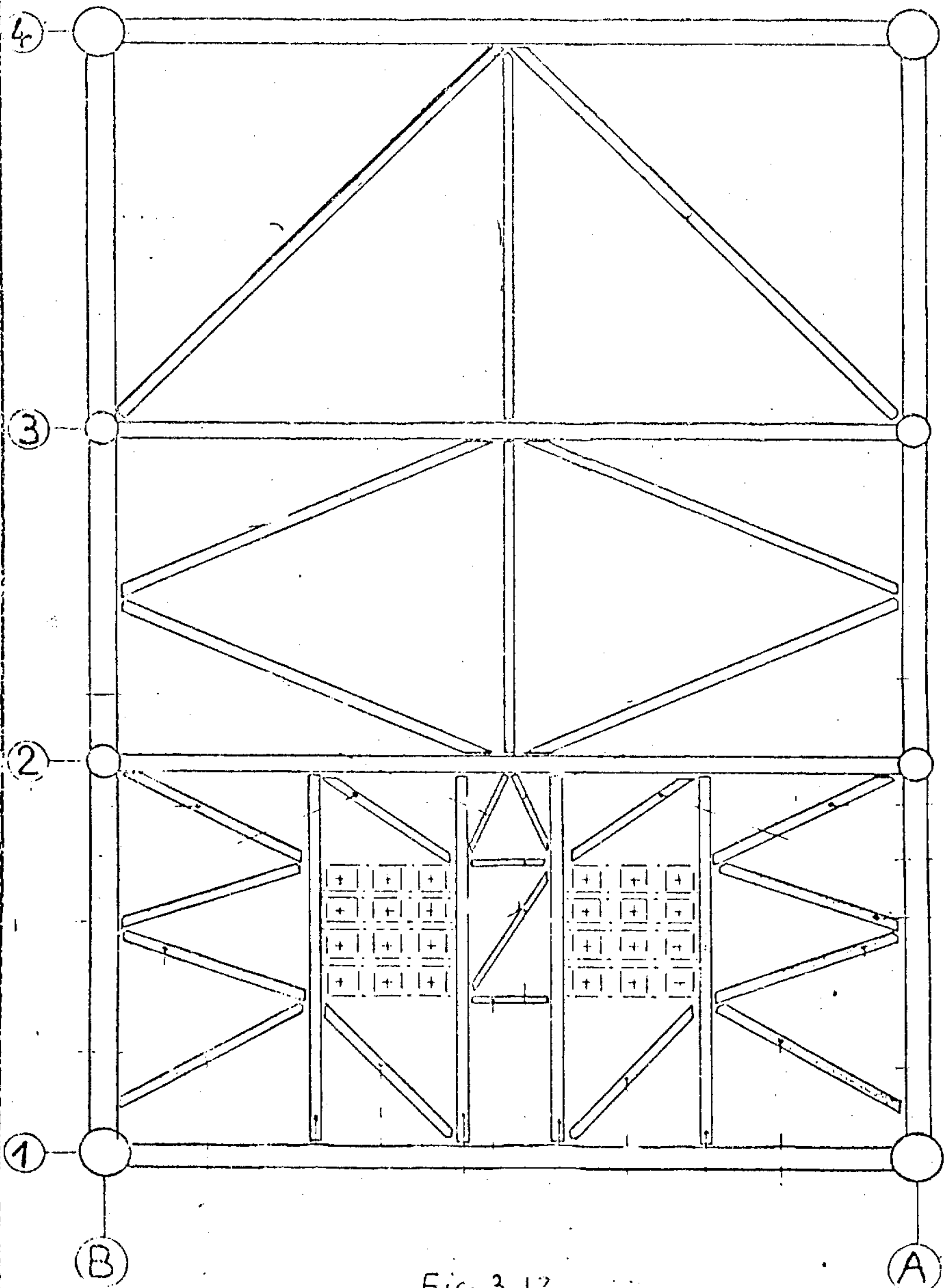


Fig 3.12

FRIGG DP2

Niveau (-) 100000

Siège : 49b avenue Hoche PARIS
Etablissement de CHERBOURG

Dessiné | Le 21.08.75
Visa : | Echelle 1/300
N° Fo 454.699 12/12

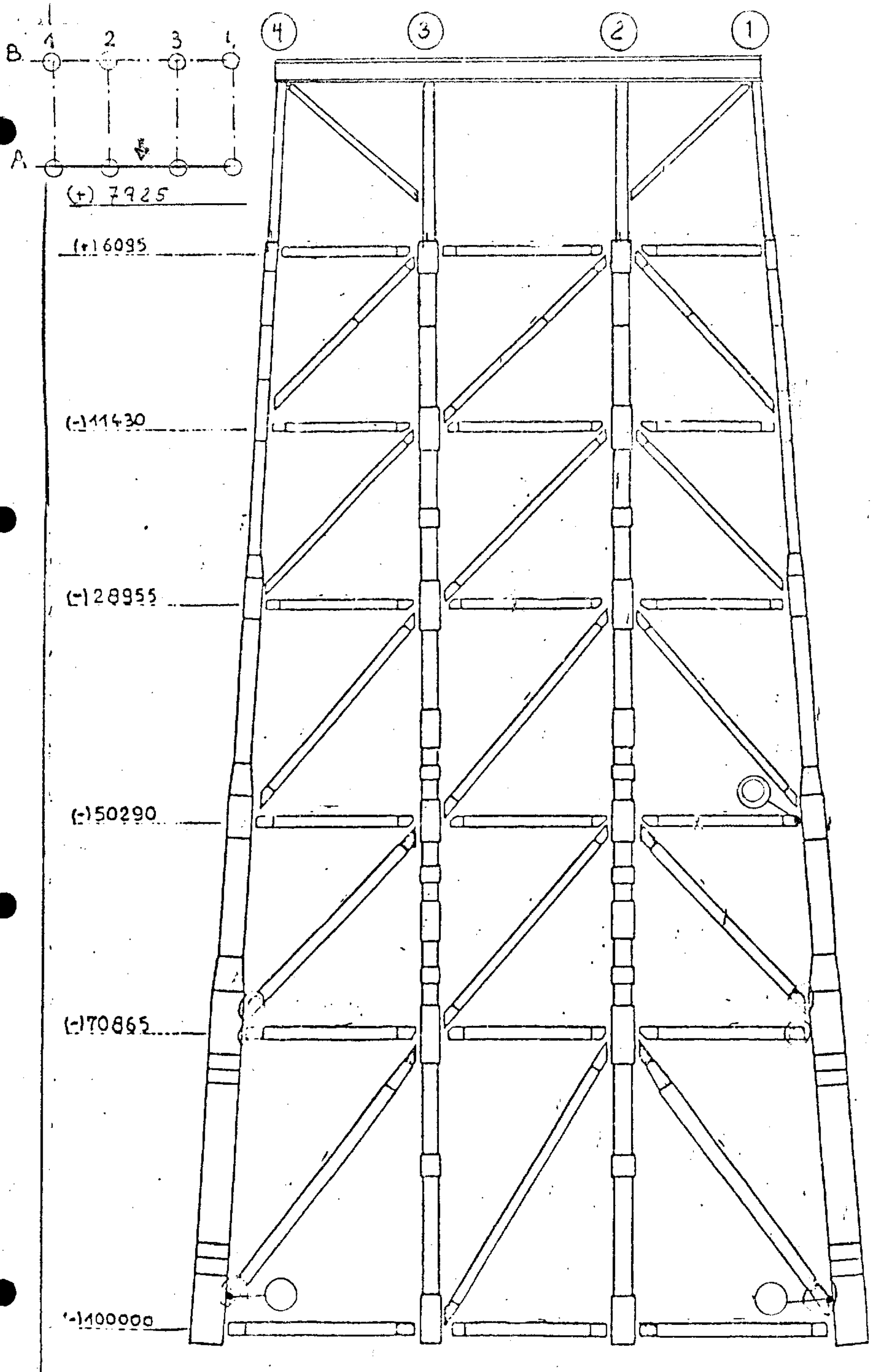
4. Fatigue

The following figures point out the most critical joints with respect to fatigue.

The SN-curves given in AWS D1.1-72 were used to determine number of cycles to failure whereas the Palmgren-Minal linear damage hypothesis was utilized to determine cumulative fatigue, i.e. usage factors.

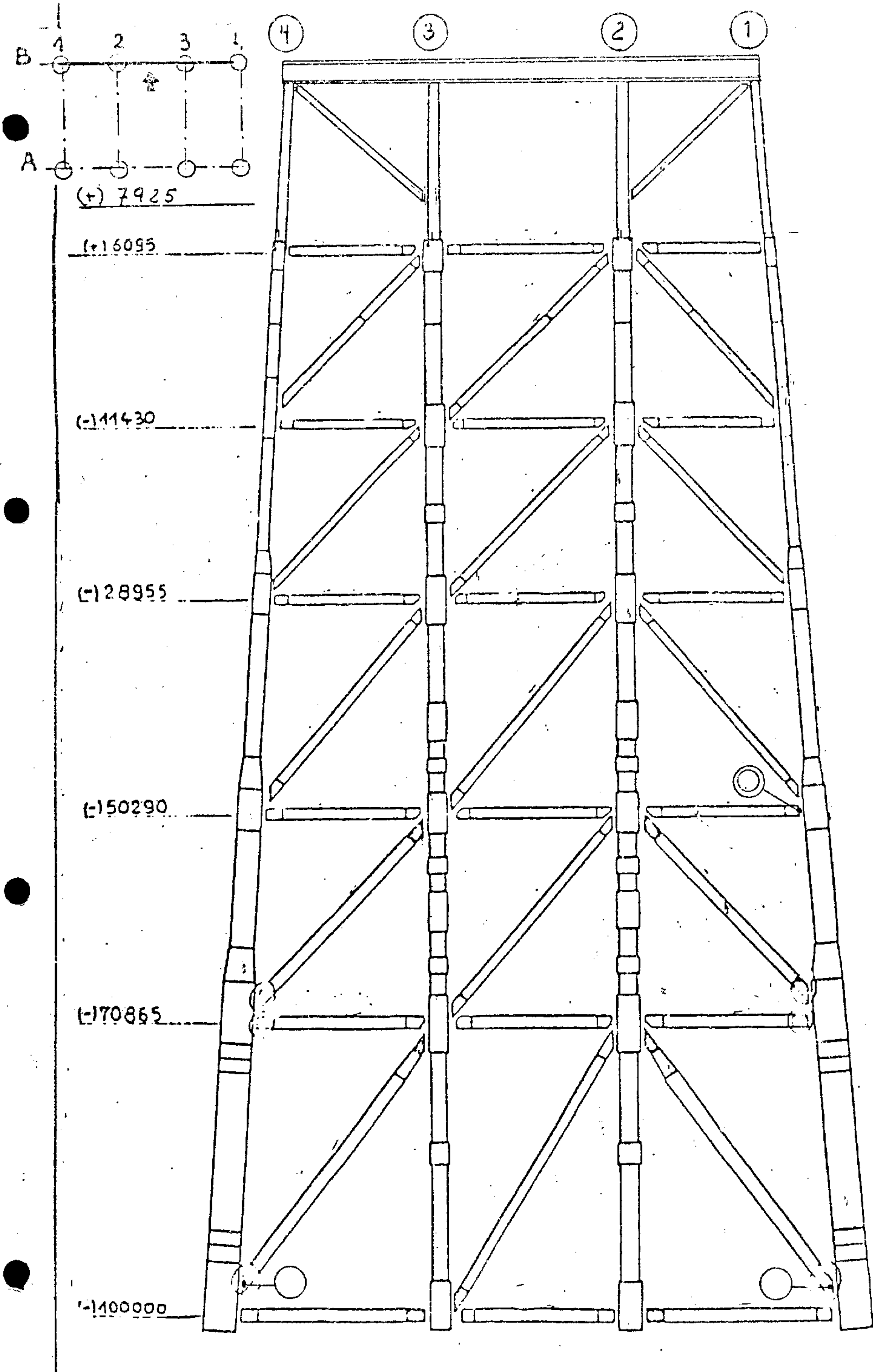
The member ends marked with a double circle (◎) have usage factors equal to or greater than 0.4. Member ends marked with a single circle (○) have usage factors between 0.1 and 0.4.

It should be noted that the members located on elevation (+)6.095m may be subjected to wave slamming loads possibly leading to fatigue failure. These members should therefore be subject to future inspection and the most critical joints have been marked with a double circle ( ), even if the calculated usage factor in all cases does not exceed 0.4.



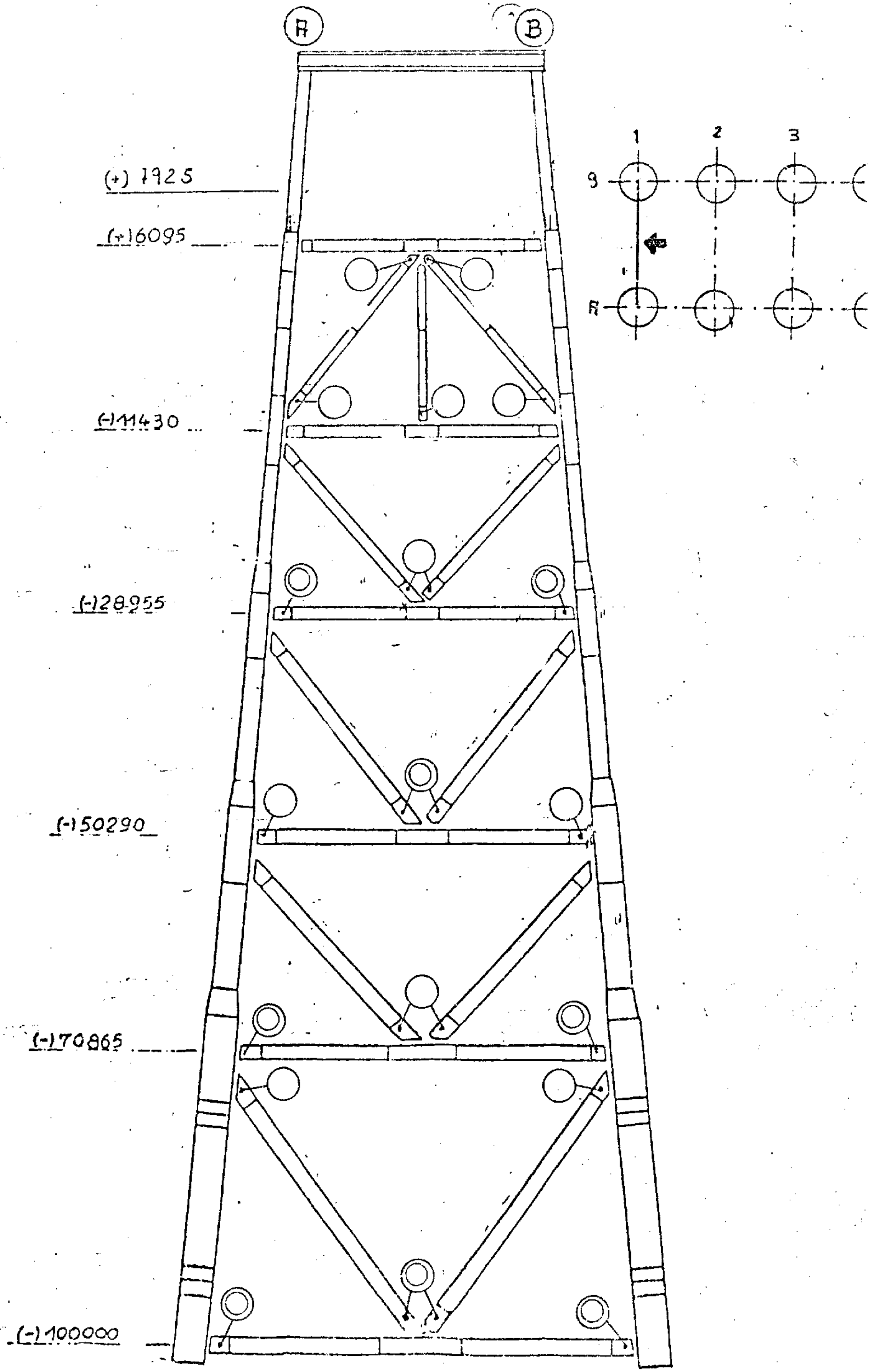
FRIGG DP2 FACE A

Fig. 4.1

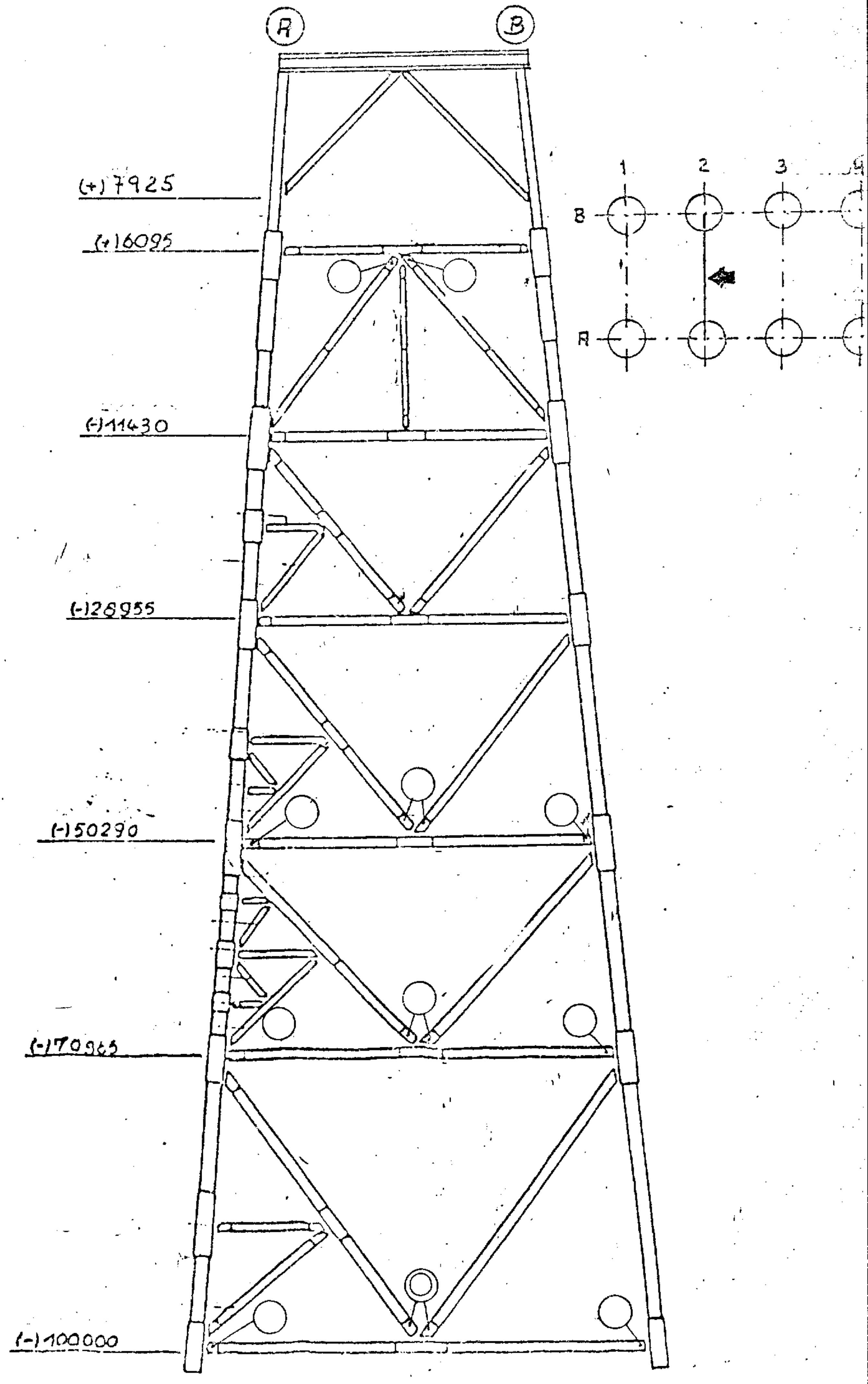


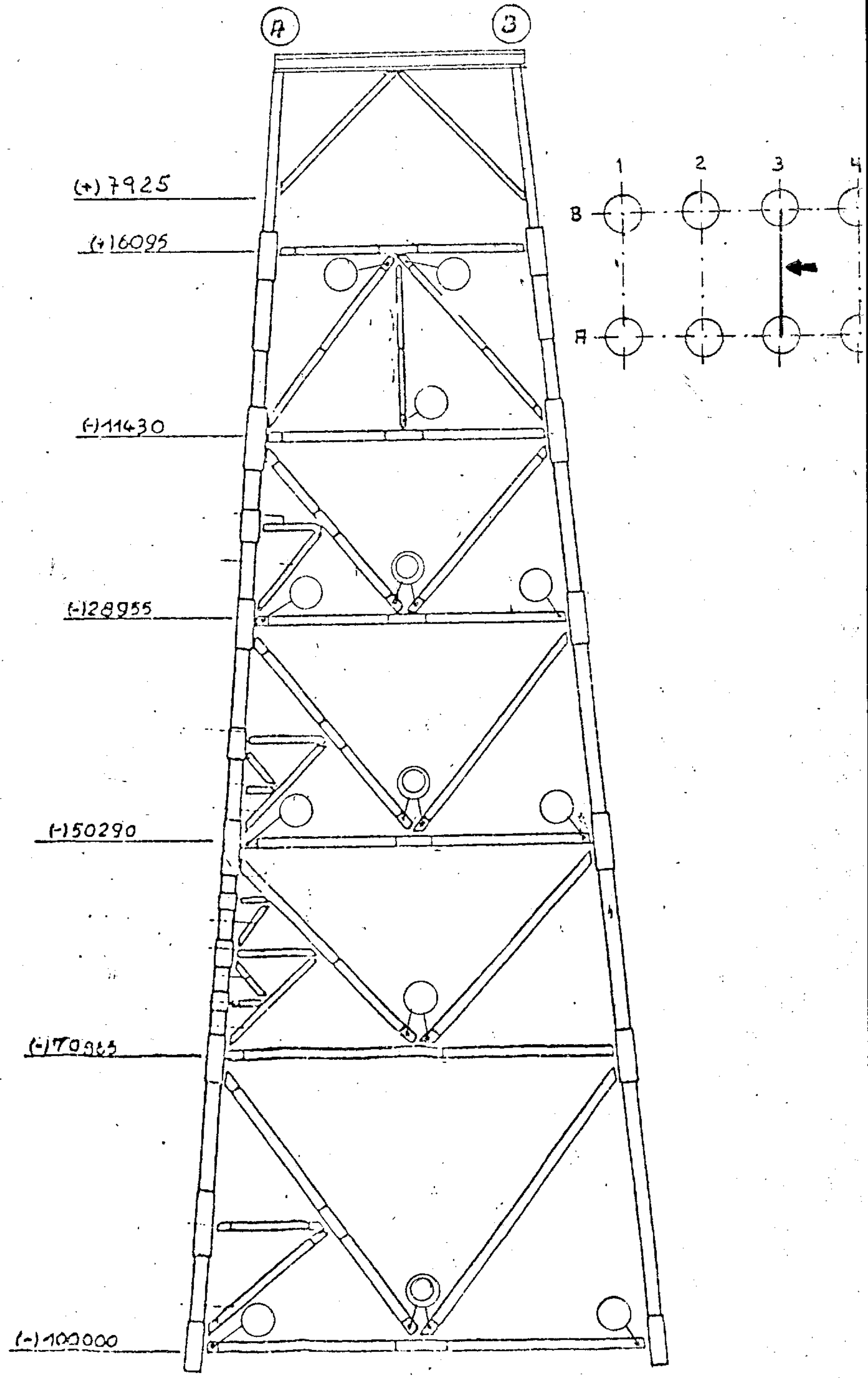
FRIGG DP2 FACE B

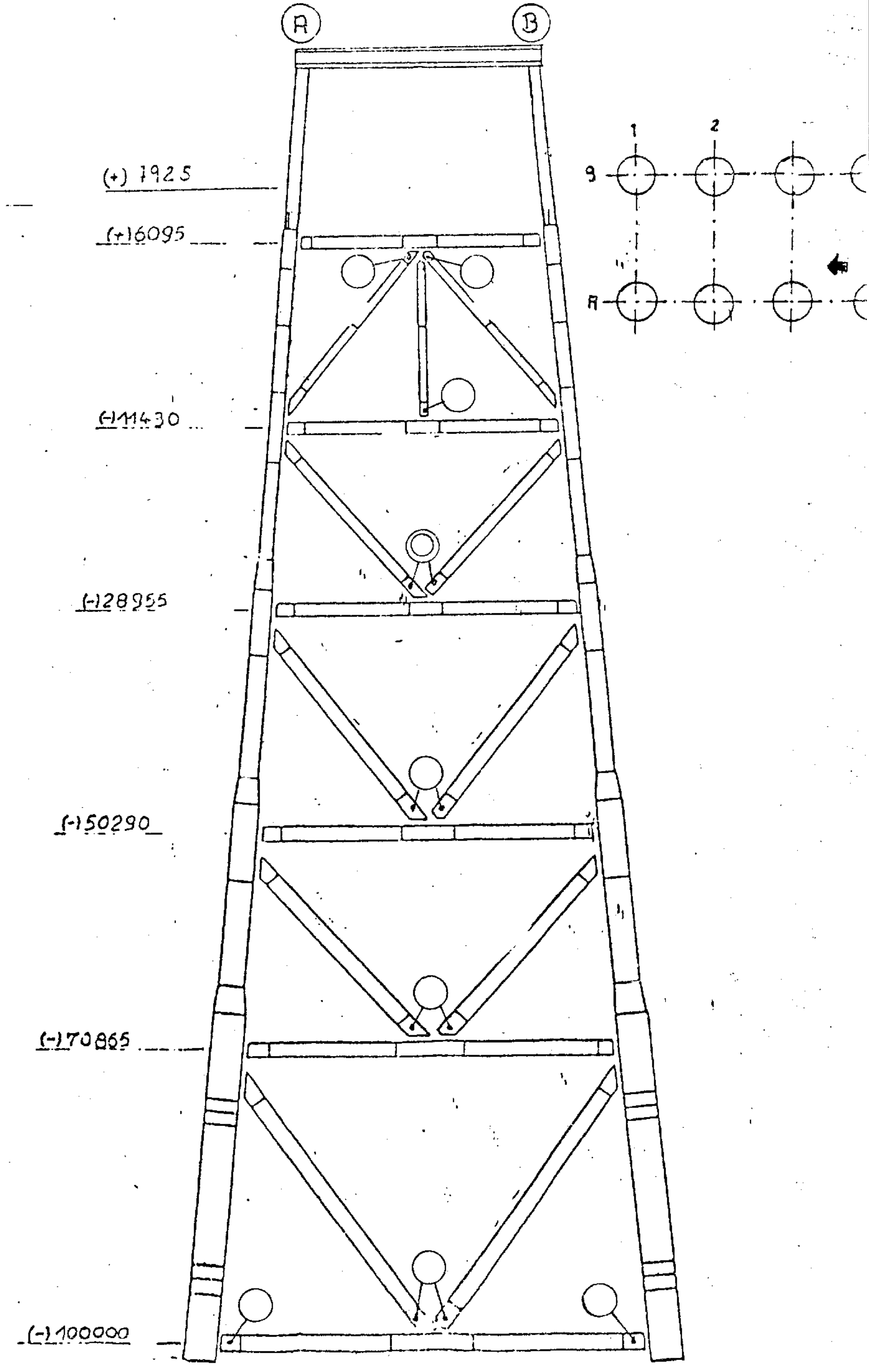
Fig 4.?



A
B
(+) 7925
(+)16095
(-)11430
(-)28955
(-)50290
(-)70865
(-)100000
1
2
3
9
A


A
B
(+)7925
(-)11430
(-)50290
(-)70965
(-)100000
1
2
3
B
A


A
B
(+)7925
(+)16095
(-)11430
(-)128955
(-)150290
(-)100000
1
2
3
4
B
A



A
B
(+) 1925
(+)16095
(-)11430
(-)28955
(-)50290
(-)70865
(-)100000
1
2
9
A

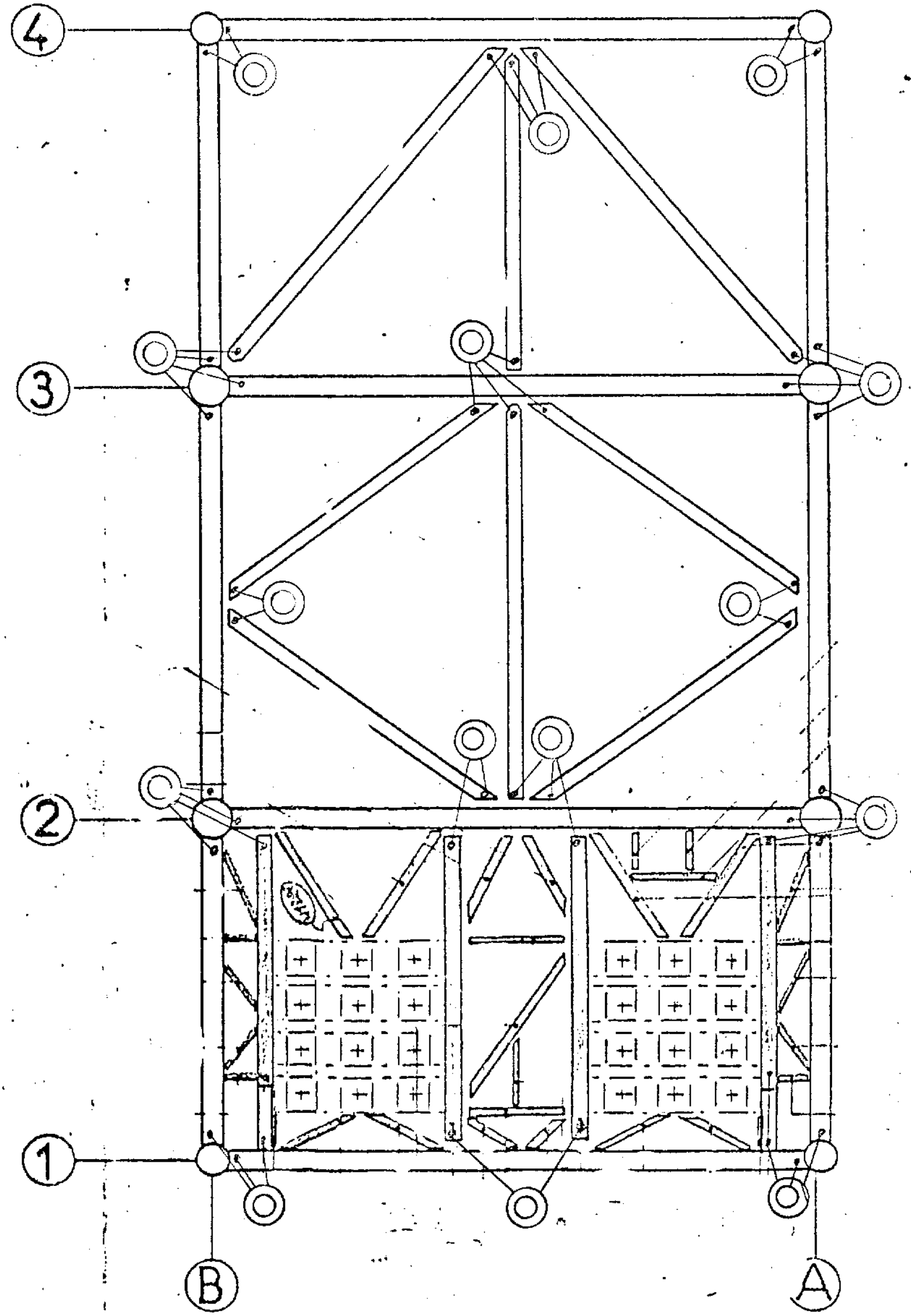


Fig. 4.7

Siège : 4bis avenue Hoche - PARIS
Établissement de CHERBOURG

FRIGG DP2
Niveau (+) 6095

Dessiné :
Le : 15.02.75
Visa :
Echelle : 1/250
N° Fo 457 699 11/13

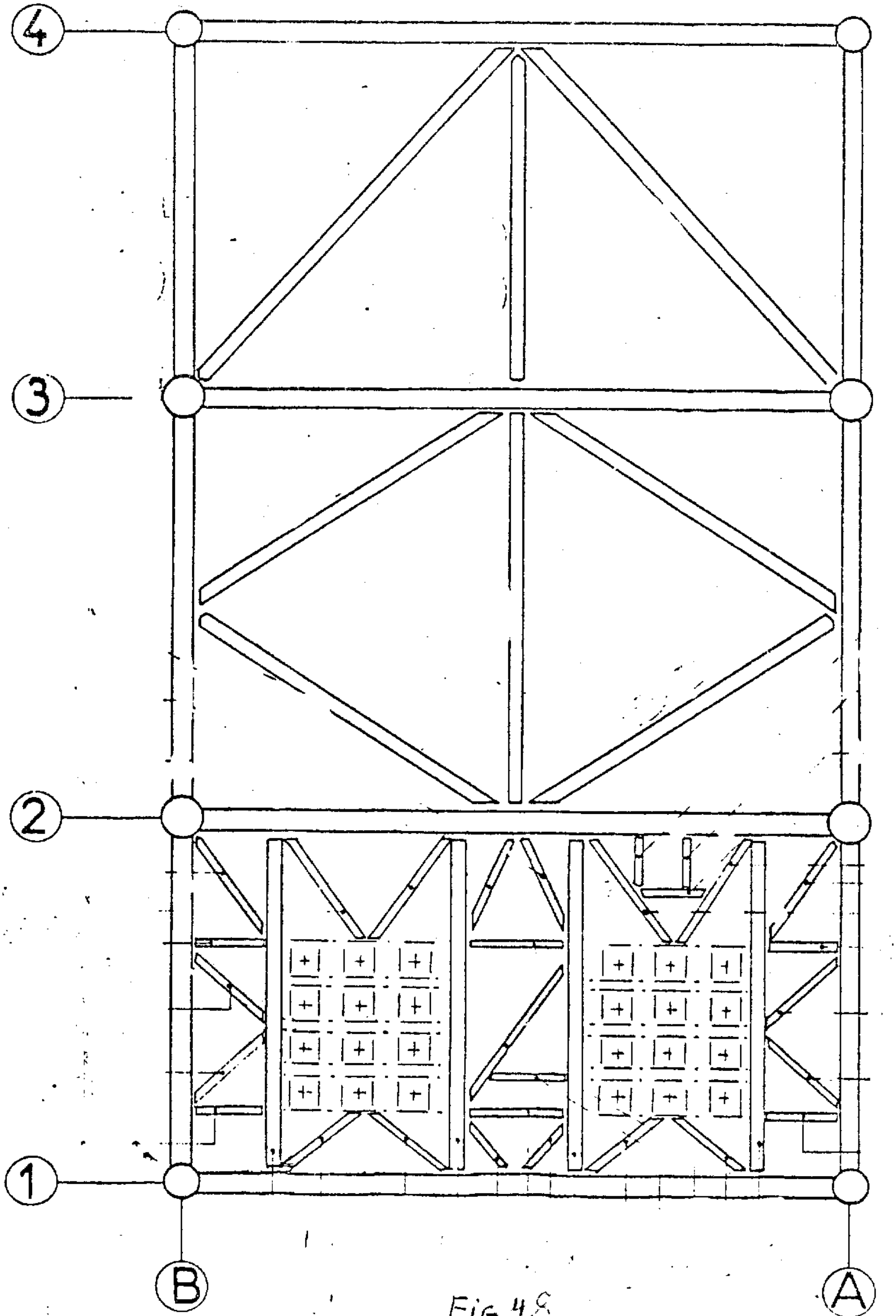


Fig 4.8

Siège : 49b avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (-) 11430	Dessiné :	Le 19.08.75
		Visa :	Echelle : 1/250
		N° Fo 454.699 8/42	

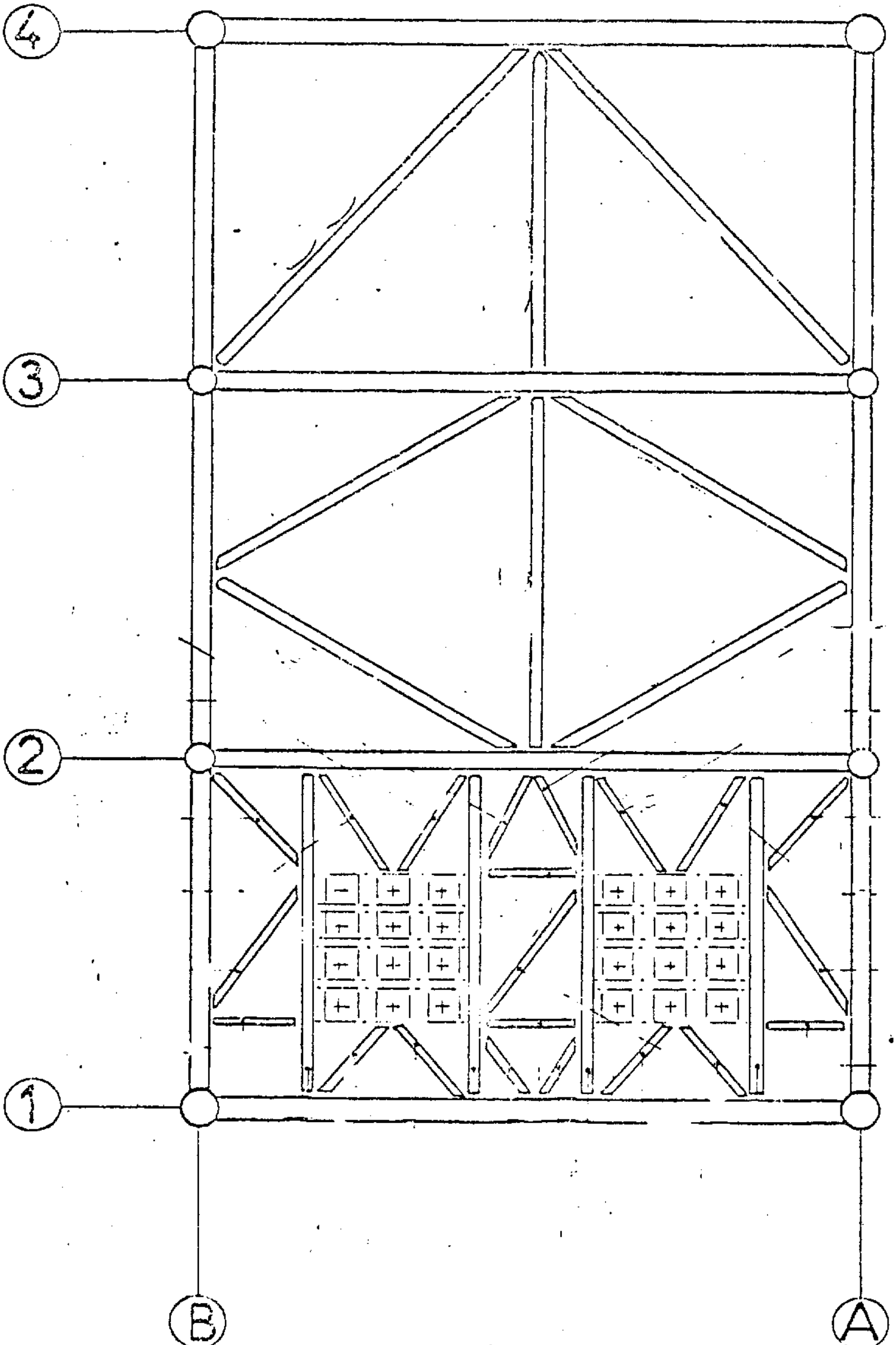


Fig 4.9

Siège : 49b avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (-)28955	Dessiné : Le : 18.08.75 Visa : Echelle : 1/300 N° Fo 454.699 9/18

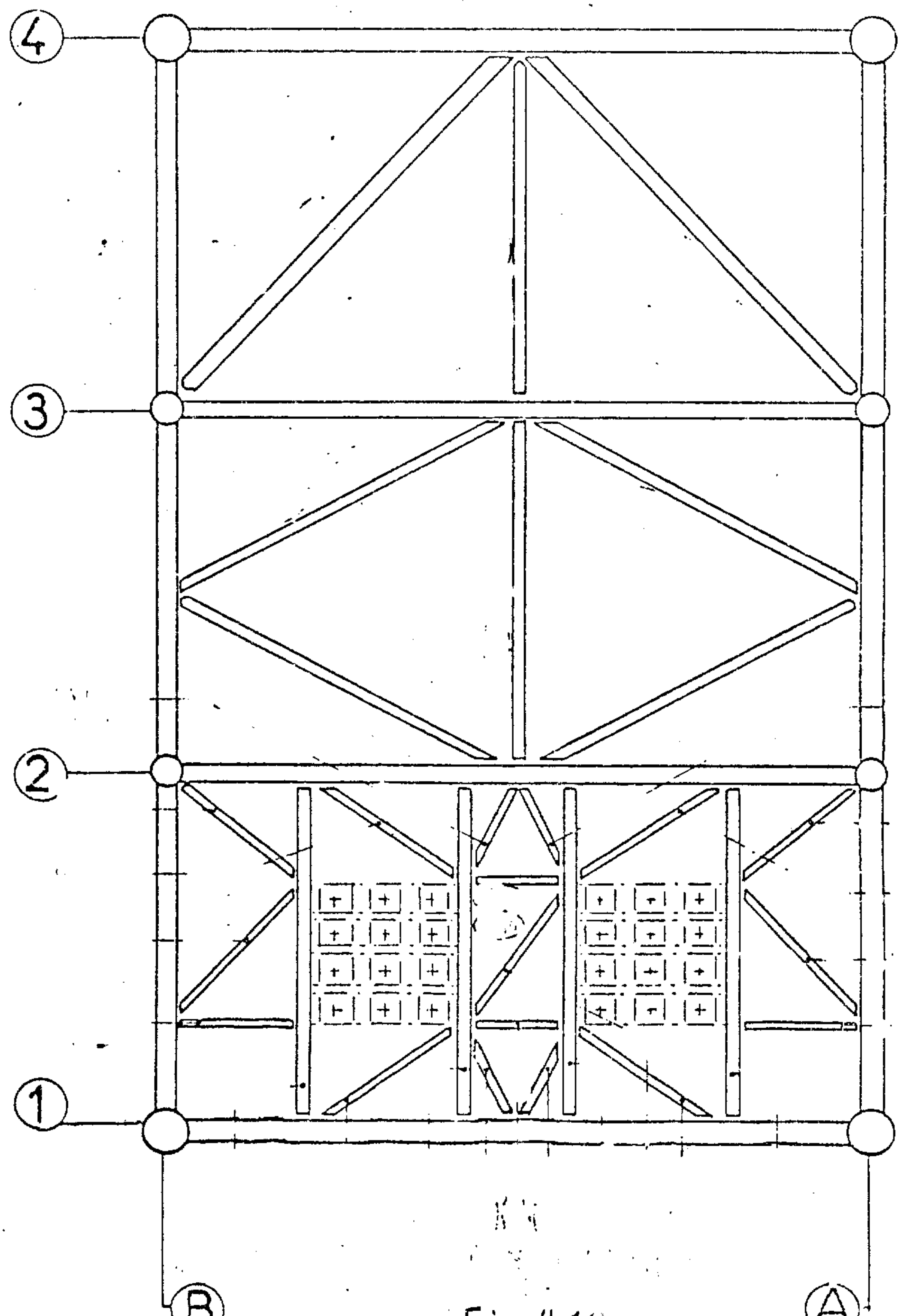


Fig 4.10

Siège : 4bis avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (-) 50290	Dessiné : / Le 21.08.75 Visa : / Echelle : 1/300 N° Fo 454 699 10/12

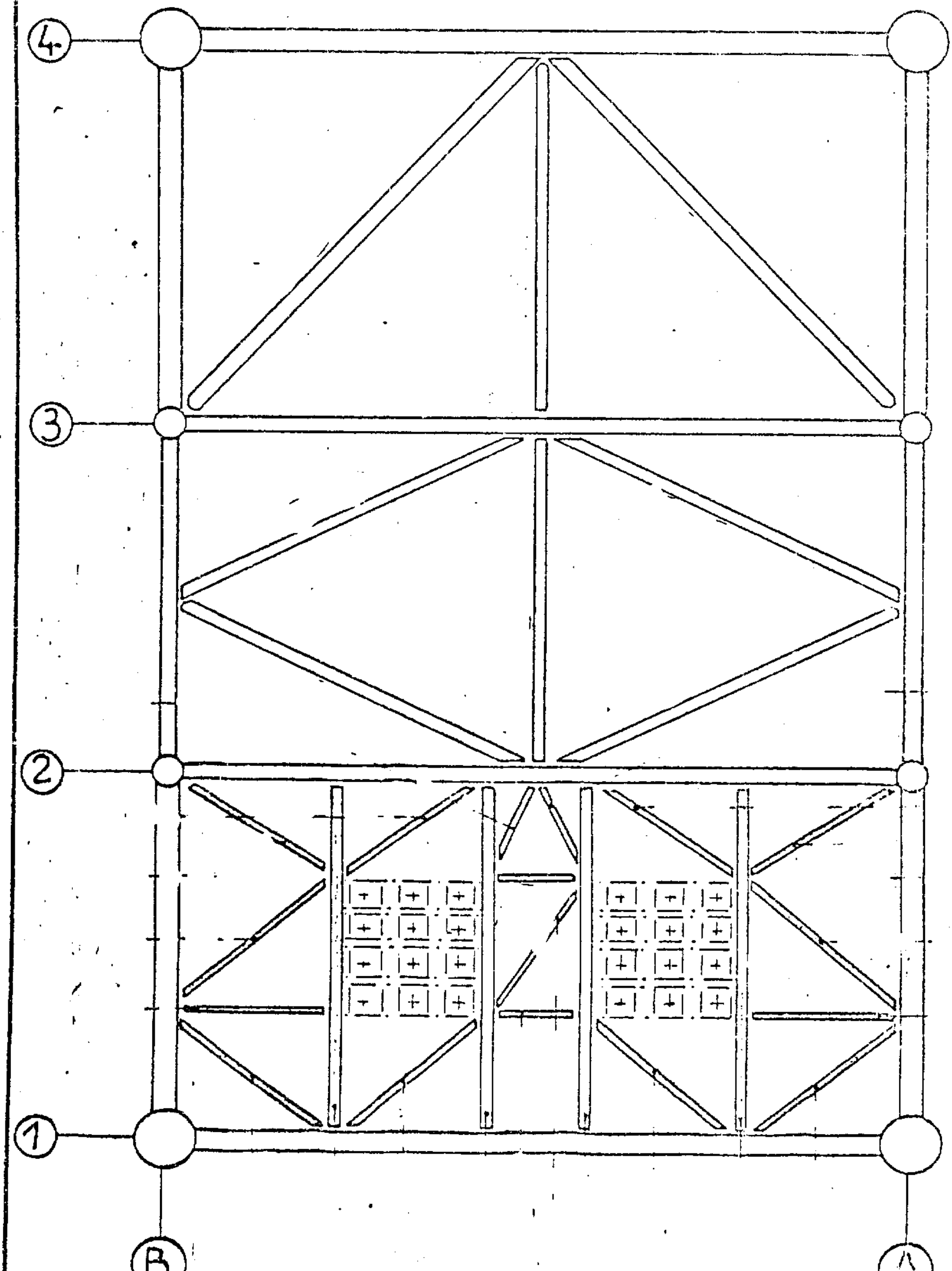


Fig 4.14

Siège : 49bis avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (-)70865	Dessiné : Le Visa : Echelle : 1/300 N° Fo

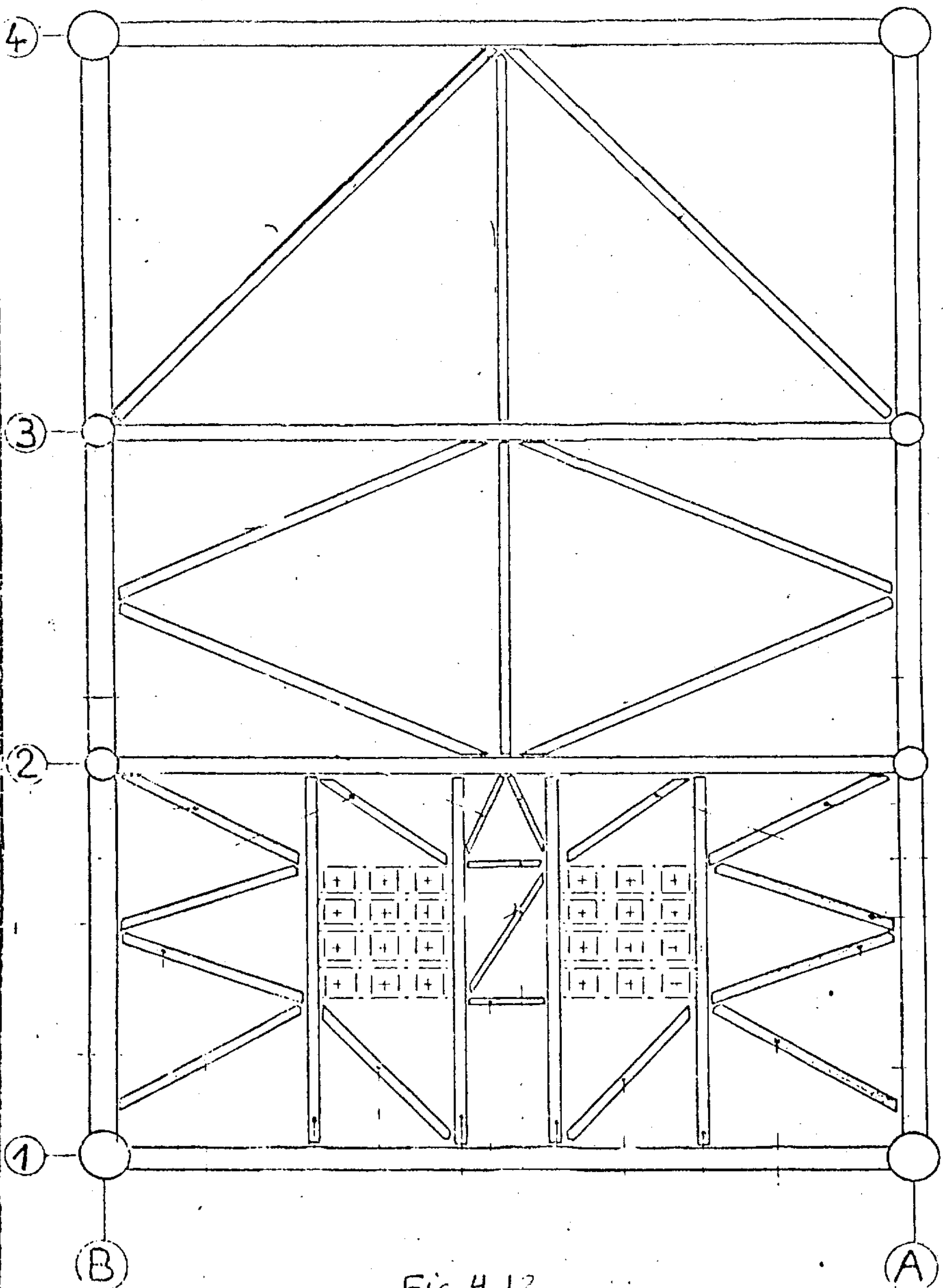


Fig 4.12

Siège : 40 bis avenue Hoche - PARIS Établissement de CHERBOURG	FRIGG DP2 Niveau (-) 100000	Dessiné ... le 21.08.75 Visa : ... Echelle 1/300 N° Fo 454.699 12/12

5. Special Considerations Including Fabrication and Installation

5.1 Jacket

The following figures point out those areas which based on DnV's inspection during fabrication and installation are considered critical. Also some areas which, based on DnV's general knowledge of the structure as well as the detailed design is considered to be most critical.

Joints where the brace end or stub thickness is equal to or exceeds 50mm must be thoroughly inspected in the future. This inspection was agreed upon during the design and fabrication of the structure and rigorous in-service inspection of these welds was a fundamental basis for DnV's acceptance of these joints. It is essential that these welds be regularely and frequently inspected, and since these weldments have comparatively small critical crack sizes, emphasis must be laid on detecting any cracks at an early stage.

These joints are marked with a tripple circle (◎).

The shear plates between corner legs and pile sleeve transfer the loads from the jacket corner legs into the piles. These plates are highly stressed in the upper and lower 5.0m region. The void enclosed by the shear plates, the corner leg and the pile sleeve is also filled with sea water. Although water is theoretically prevented from circulating in/out it is considered essential that the thickness of these plates be regularely measured.

It is also important that the weld between the shear plates and pile sleeve as well as between shear plates and corner legs and incoming braces be regularely inspected in the upper and lower 5 m lengths.

Also the weld between the yoke plates and the shear plates, upper and lower, are highly stressed.

The relevant welds are shown on Fig. 5.13.

Generally, the leg joints at mud line elevation (-)100m, are also highly stressed and should be inspected accordingly. These joints have been incorporated here because any irregularity in the modelling of the piles in the computer calculations may greatly effect the stress level in these joints.

The welds between the support frame legs and the jacket legs or piles experienced repeated cracking during field welding. These butt welds are relatively highly stressed and due to the repeated repair of these welds it is considered necessary that they be subjected to regular inspection.

In order to support buoyancy tanks and later on guide the piles, pile guides were installed around each corner leg. These pile-guides installed on elevation (+)6.095m and also on elevation (+)21.485m were removed after installation of piles. In order to ascertain that no cracks develop at these locations, regular inspection should be conducted, particularly in an initial phase. Also the welds attaching boat landings and barge bumpers should be inspected.

It is believed that general measurements of wall thicknesses will be done. This appendix does not give any further suggestion as to where such measurements should take place. It is assumed, however, that appropriate areas be selected based on the previous figures, e.g. those defining critical areas with respect to stresses.

It is also assumed that special attention be paid to the members in the splash zone, and not at least flooded members and compartments. Flooded compartments and members are defined in the following figures.

5.2 Deck Support Frame

The deck support frame is generally not very highly stressed.

However, the joints between deck legs and deck beams are of utmost importance and furthermore fabrication of these connections experienced some problems. It is thus essential that these welds be regularely inspected.

For further details of critical areas see Fig. 5.14.

Attachments and members welded to the flanges of the deck support frame involve areas susceptible to fatigue cracking. This applies to areas where walkways, deck modules, and (+)21.485m horizontal members are welded to the flanges of the deck beams. Also the areas where the storage tanks and pump house is welded in should be regularely inspected.

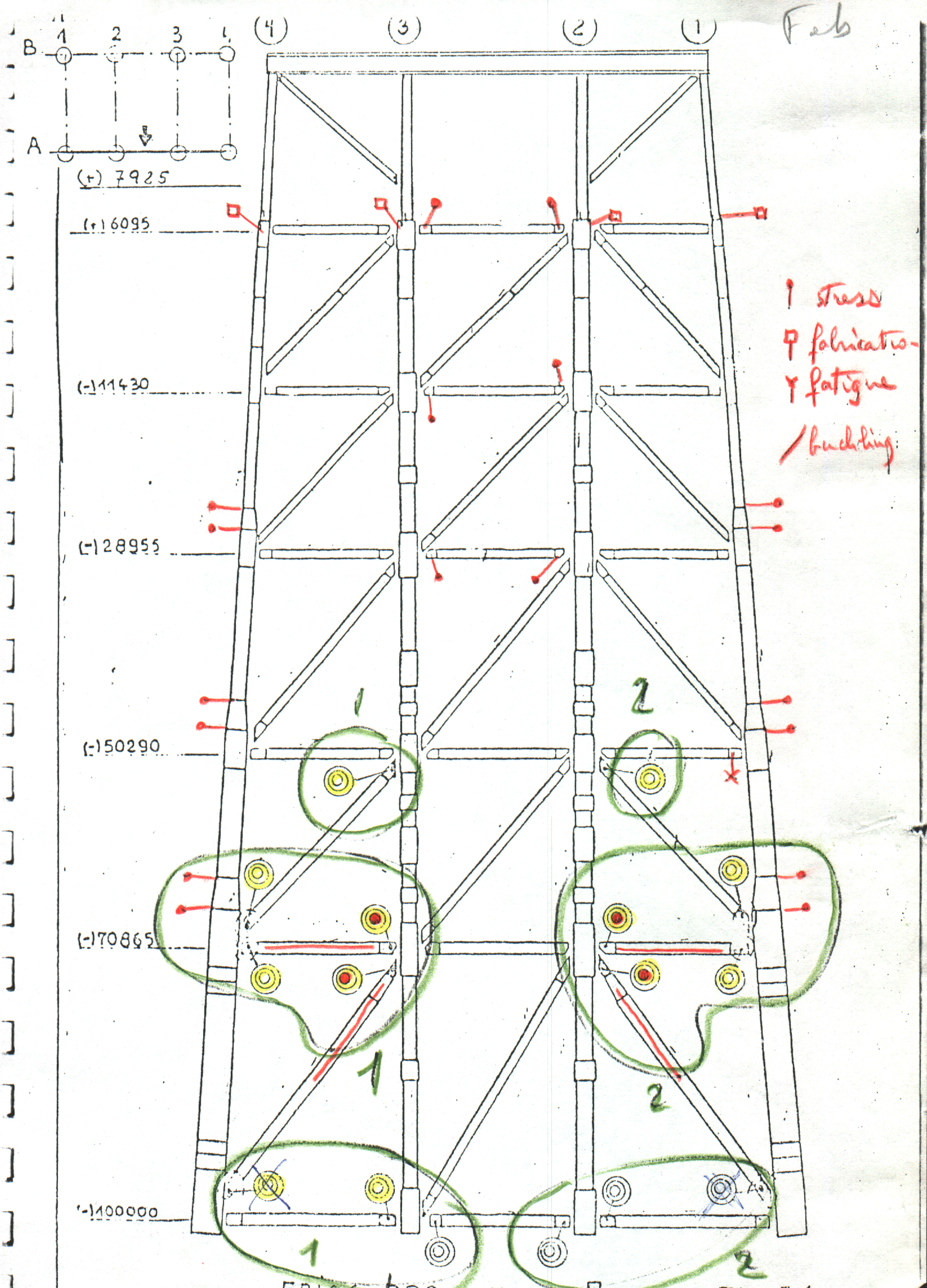
5.3 Production Modules

The production modules A, B, C and D are generally not very highly stressed.
However, some truss member in module D are marginally loaded. These members are shown on Fig. 5.15.

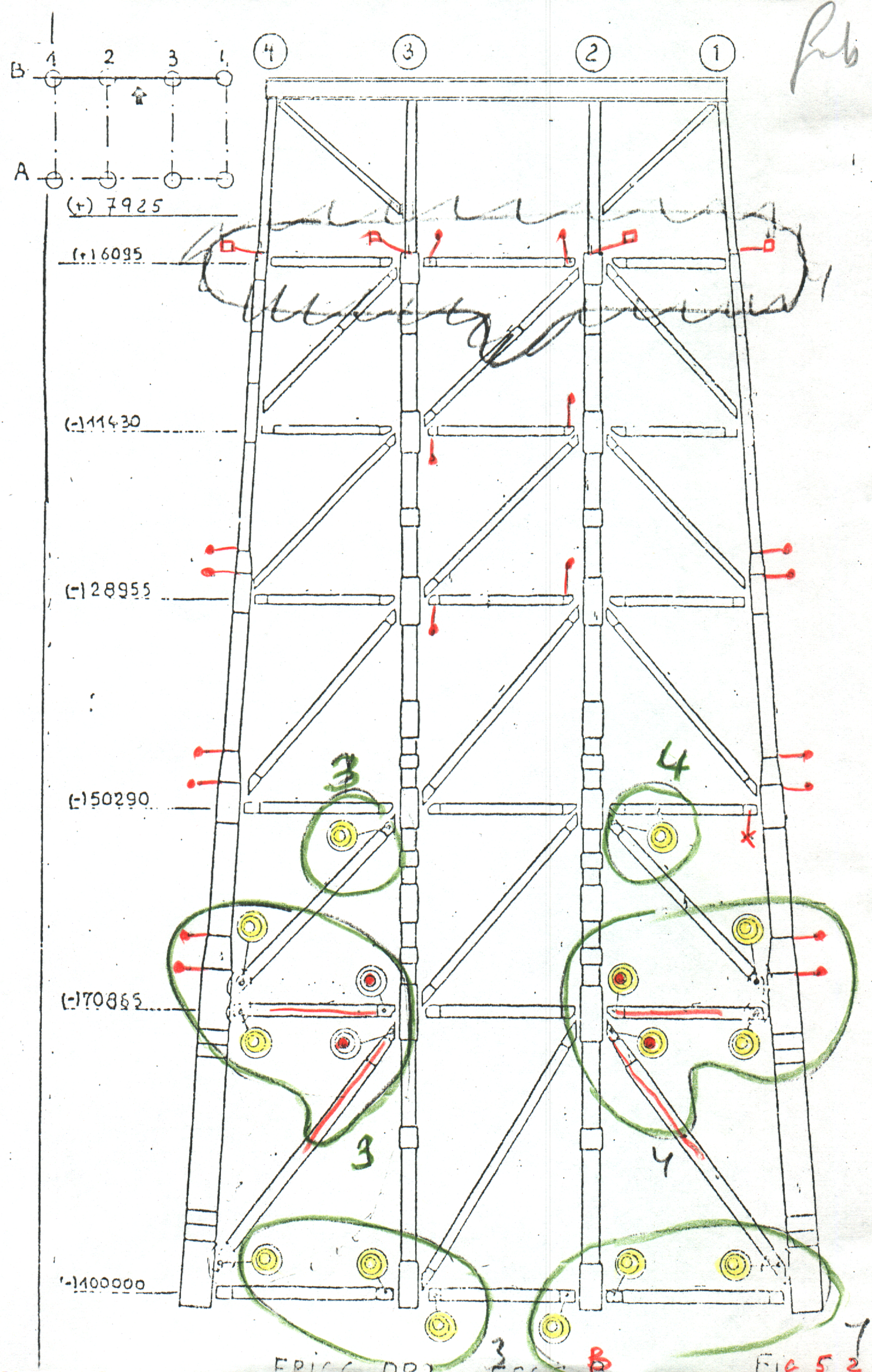
Similar members on the remaining modules (A, B and C) should also be inspected regularely.

Also, module supports should be subjected to regular inspection. This applies to the welds between shim plates and module structure and between shim plates and deck support frame.

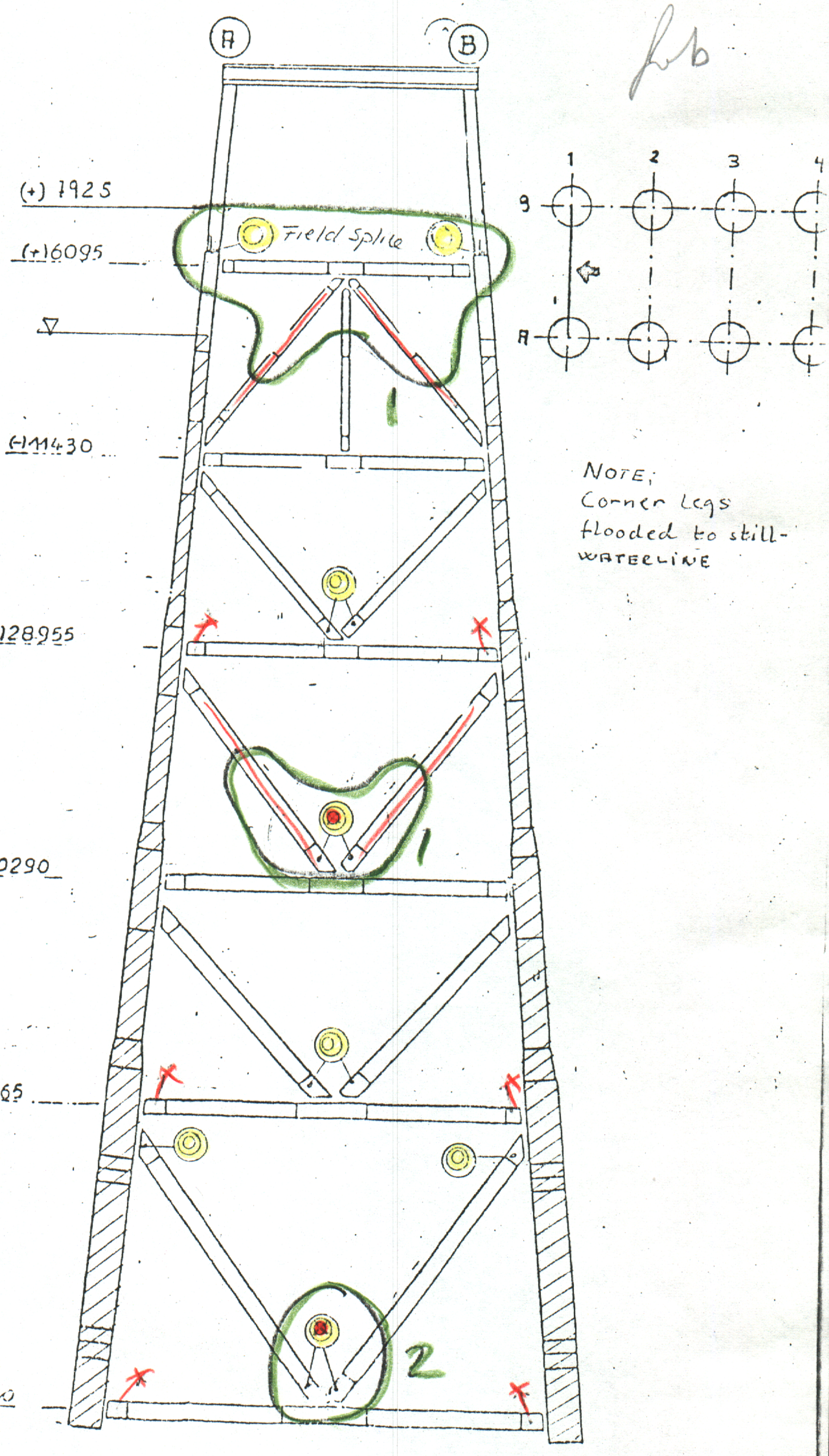
In module A also the skid beams should be inspected, particularly the areas supporting the sub-structure skid beams.



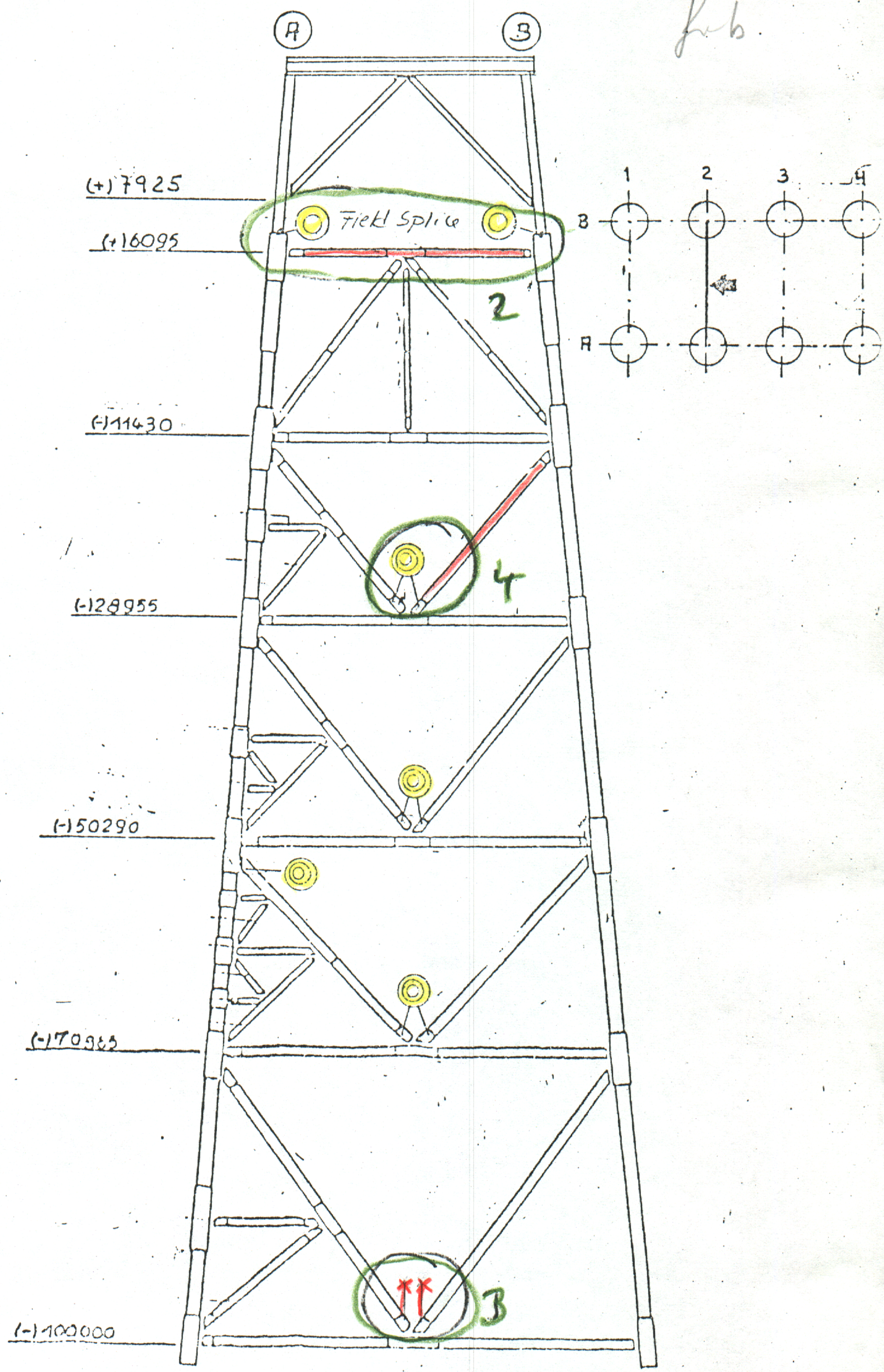
FRIGG DP2 FACE A FIG 5.1



4
3
2
1
B
A
(+) 7925
(+) 16095
(-) 11430
(-) 28955
(-) 50290
(-) 70865
(-) 100000
3
4
3
4

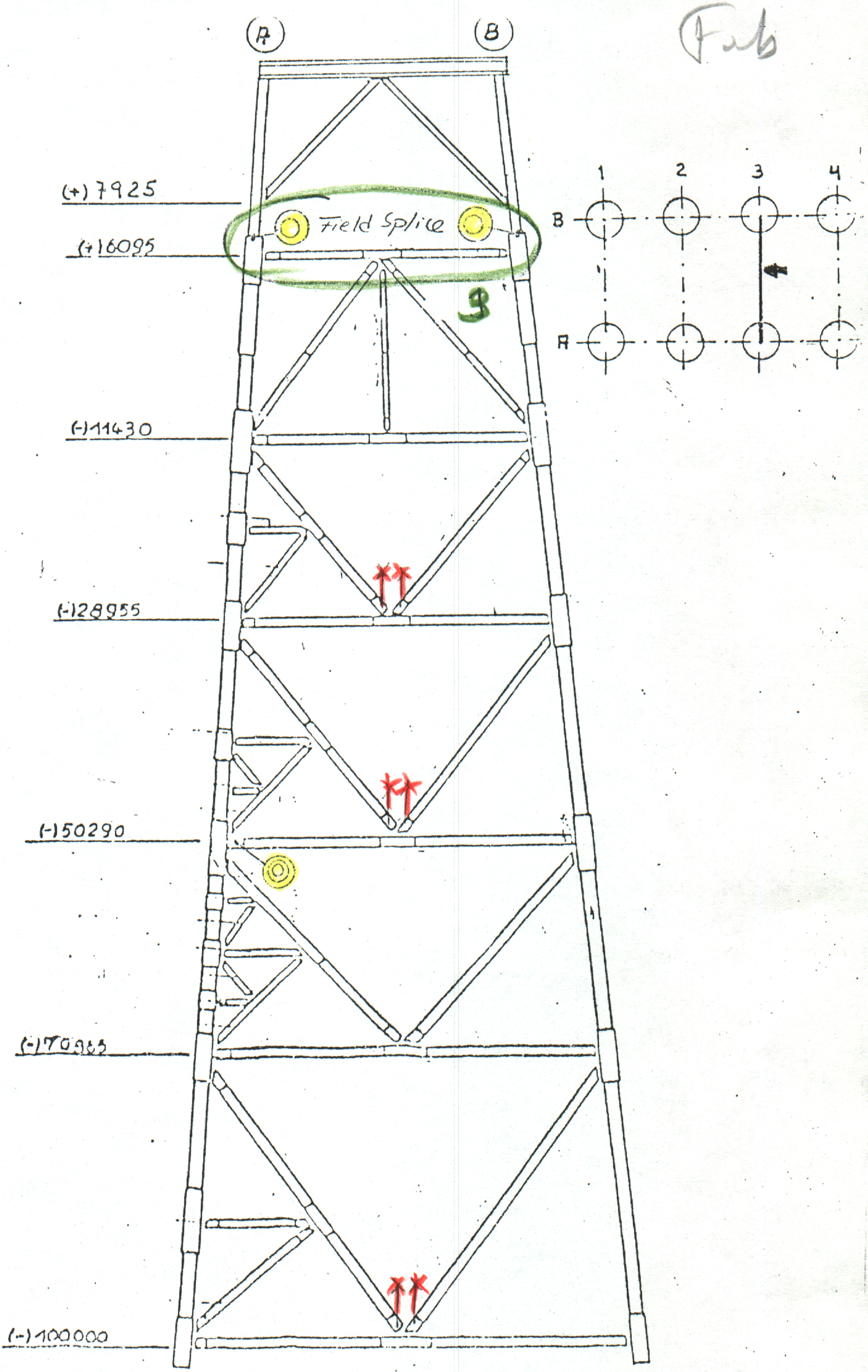


FRIGG DP2 FILE 1 Fig. 5.3



FRIGG DP2 FILE 2 Fig 54

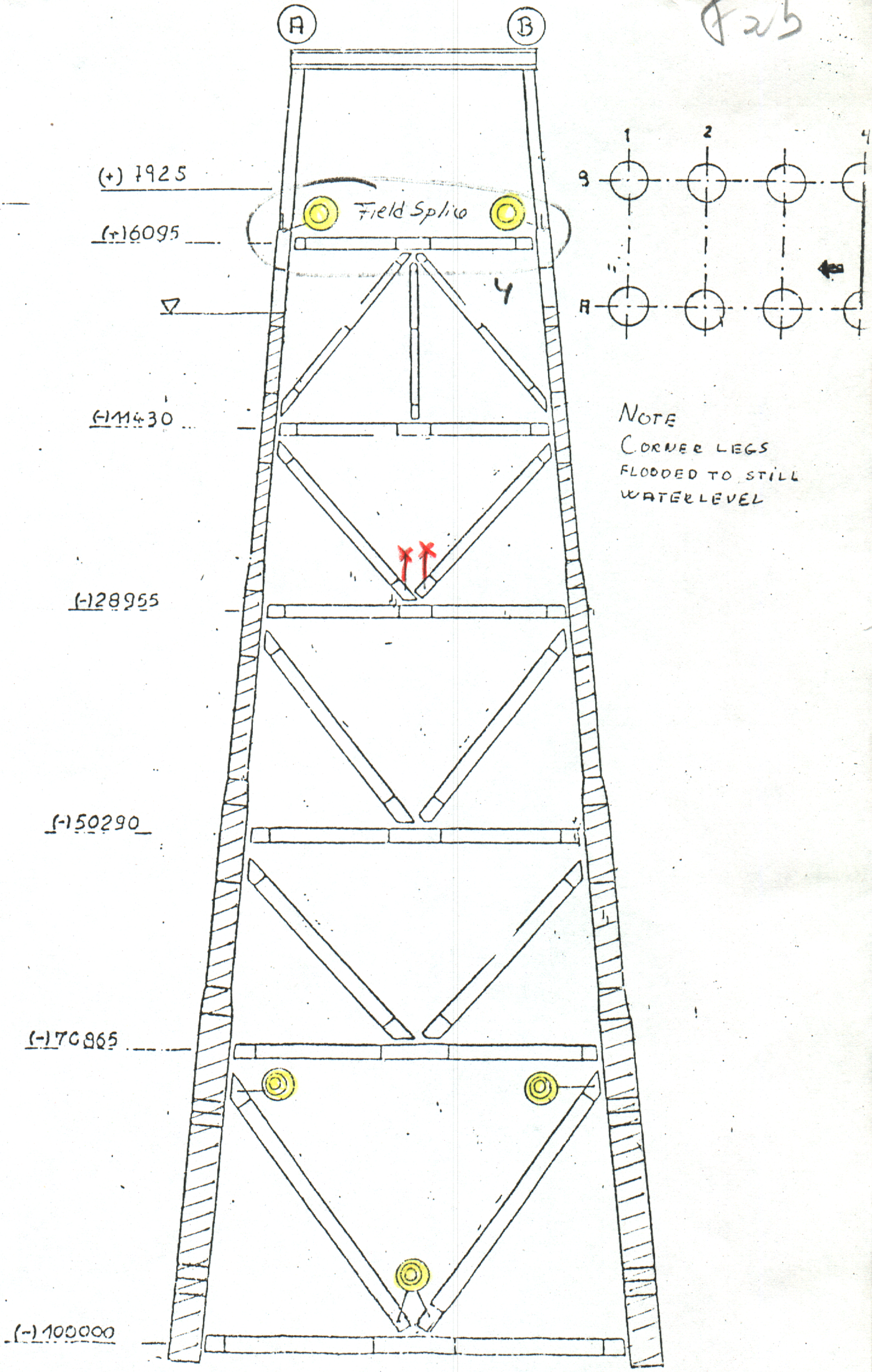
Fab



FRIGG DP2 FILE 3

FIG. 5.5

Fab



NOTE
CORNER LEGS
FLOODED TO STILL
WATERLEVEL

FRIGG DP2 FILE 4

Fig 56

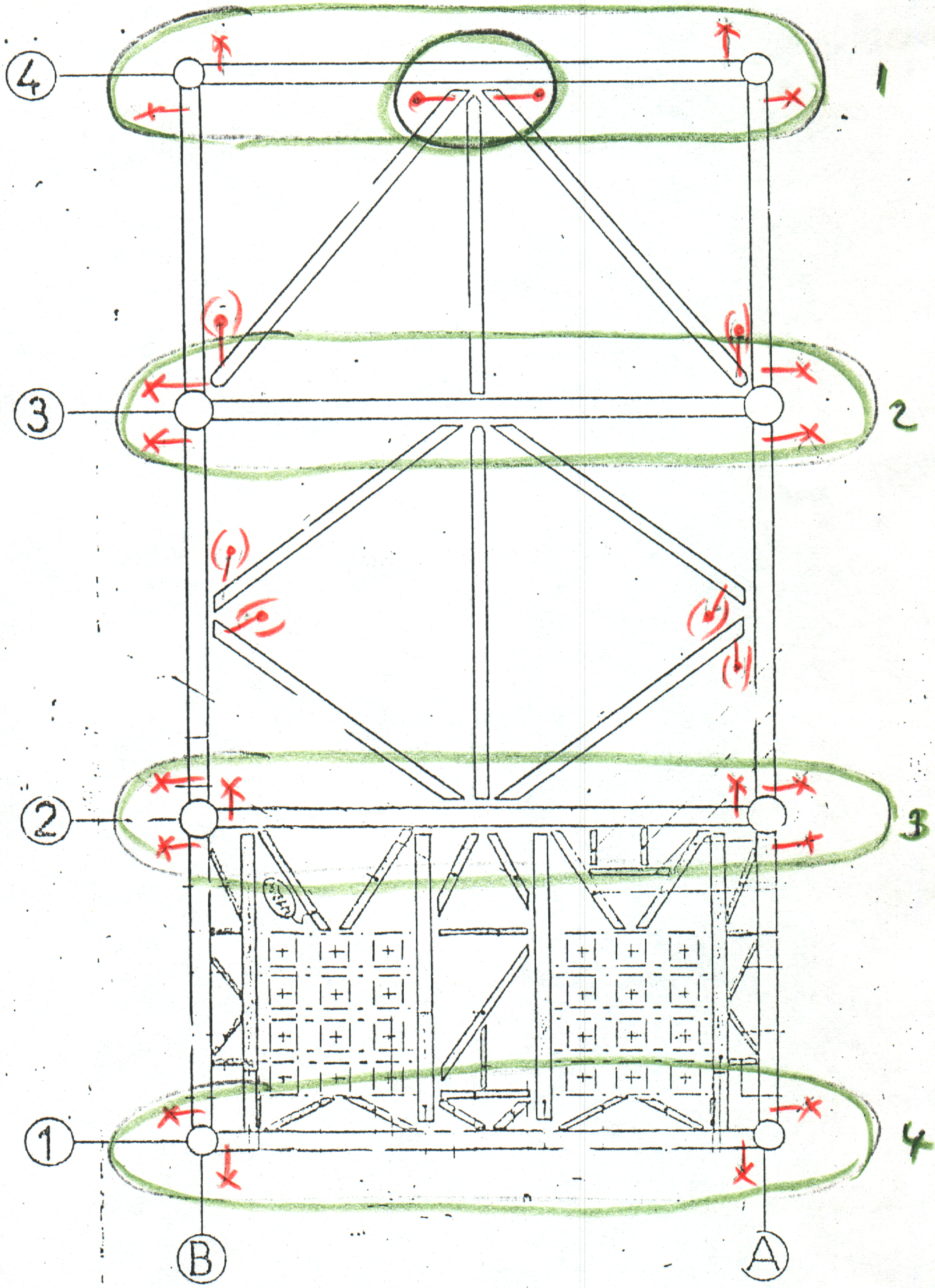


Fig 5.7

Siège : 4 bis avenue Hoche - PARIS Établissement de CHERBOURG	FRIGG DP2 Niveau (+) 6095	Dessiné : — Le : 13.08.75 Visa : — Échelle : 1/250 No Fo 457 699 1/3

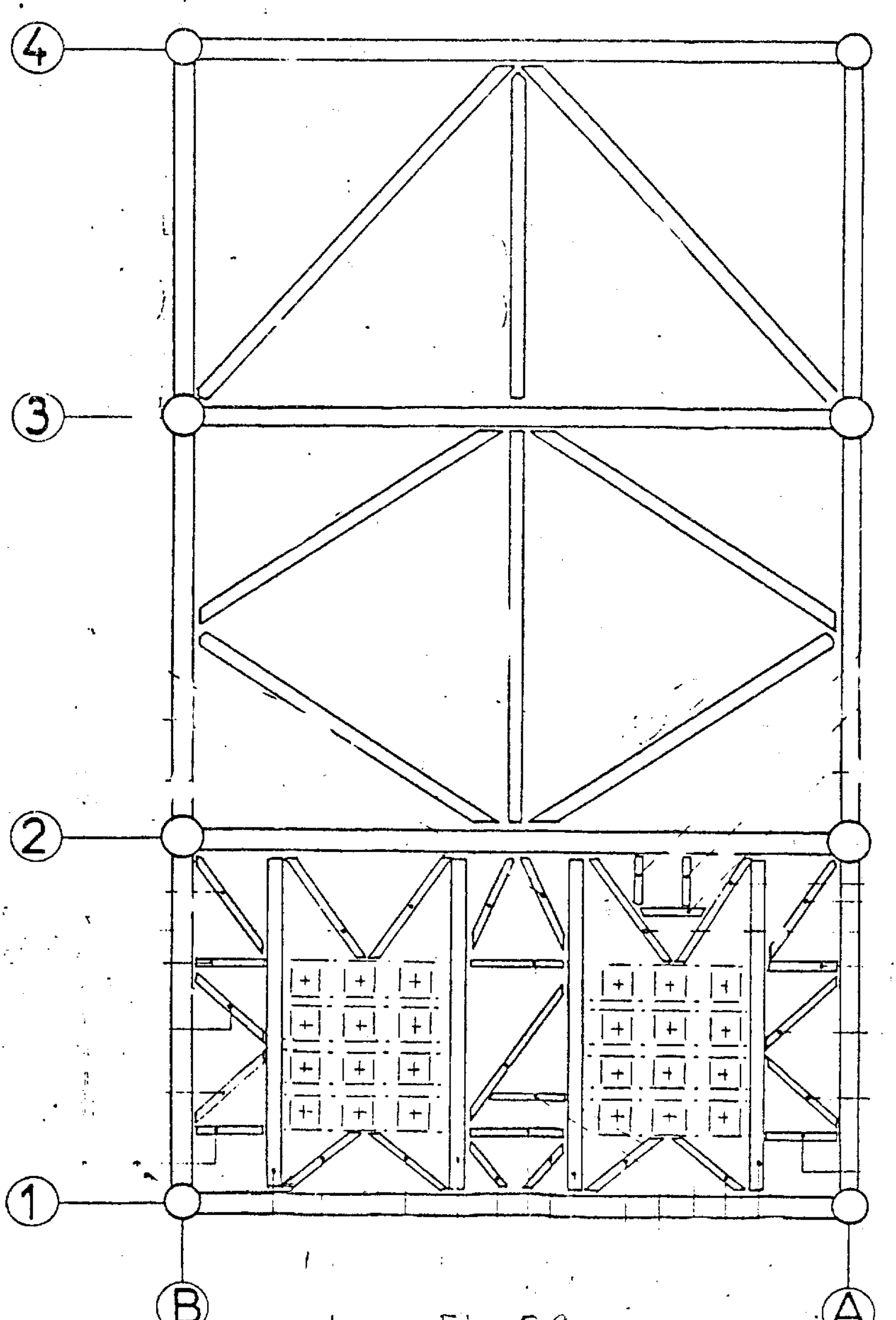


Fig. 5.8

Siège : 49b avenue Hoche - PARIS
Etablissement de CHERBOURG

FRIGG DP2
Niveau (-) 11430

Dessiné :	Le 19.08.75
Visa :	Echelle 1/250
Nº Fo 454.699 8/12	

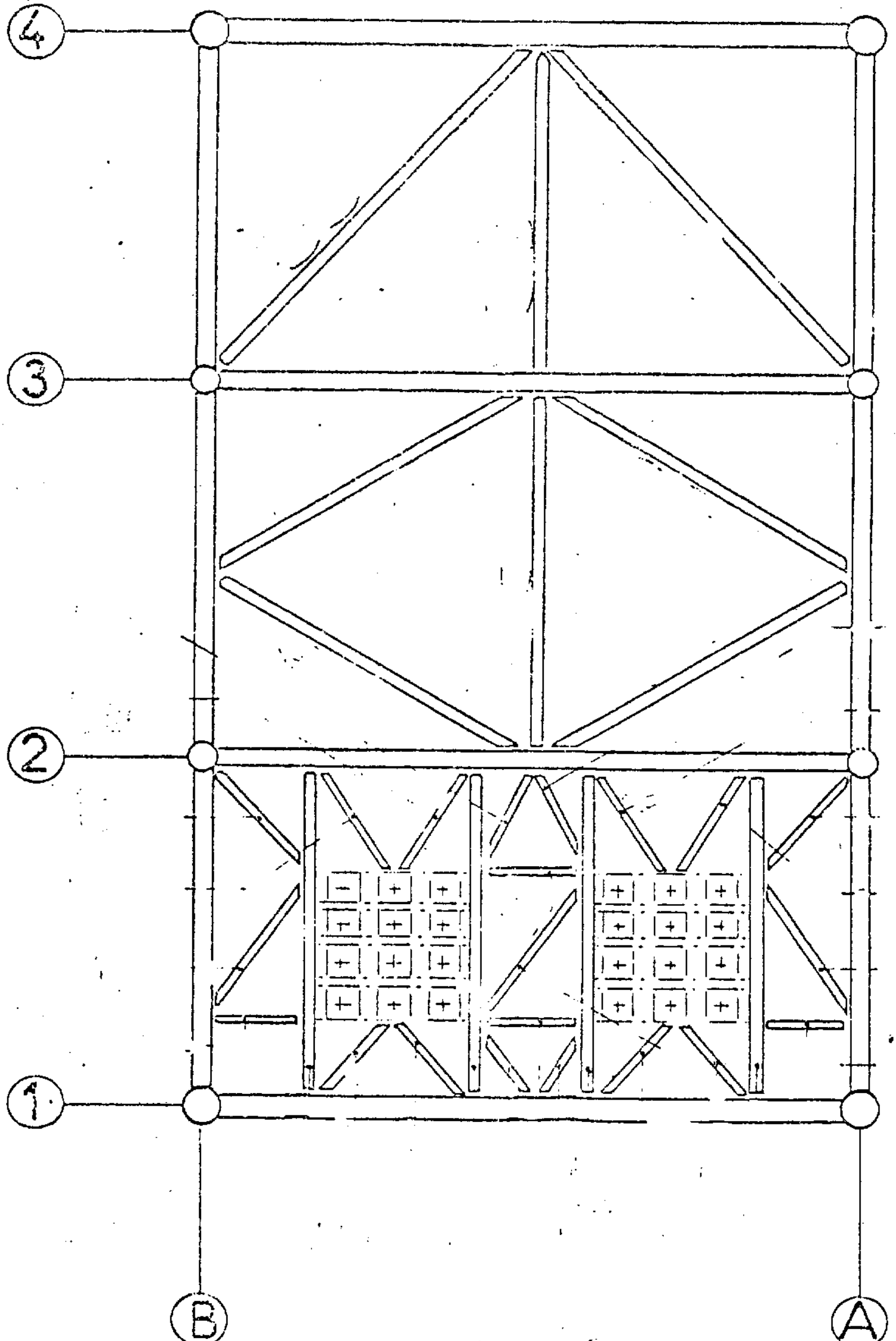


Fig S.9

FRIGG DP2

Niveau (-)28955

Siège : 49b avenue Hoche - PARIS
Etablissement de CHERBOURG

Dessiné :	Le : 18.08.75
Visa :	Echelle : 1/300
N° Fo 457.699 9/12	

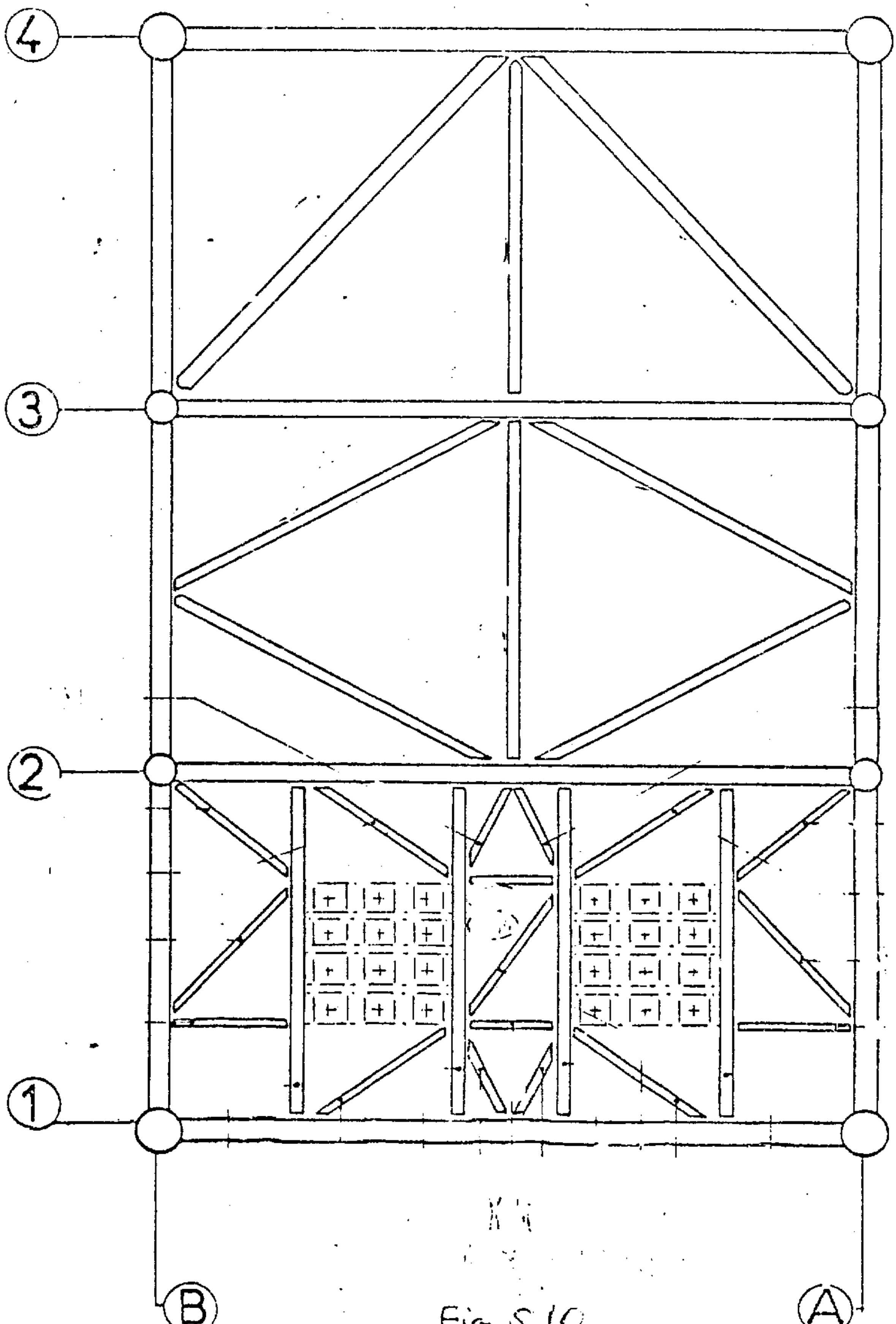


Fig. S.10

Siège : 46 bis avenue Hoche - PARIS
Établissement de CHERBOURG

FRIGG DP2

Niveau (-) 50290

Dessiné :	Le : 21.08.75
Visa :	Échelle : 1/300
Nº Fo 454 699 10/12	

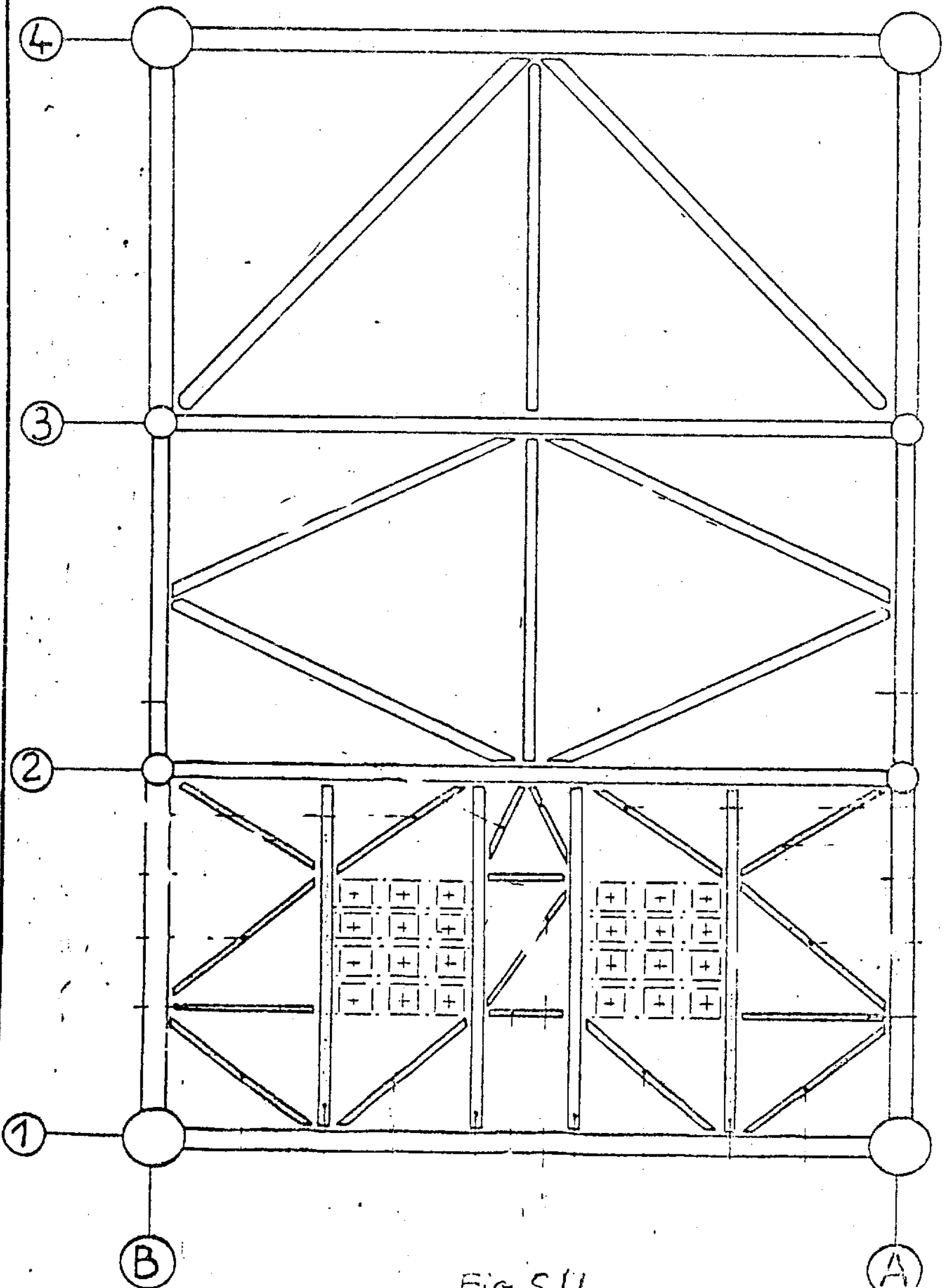


Fig 5.11

Siège : 45bis avenue Hoche - PARIS Etablissement de CHERBOURG	FRIGG DP2 Niveau (-)170865	Dessiné : [illegible] Le [illegible] Visa : Echelle : 1/300 N° F6 [illegible]

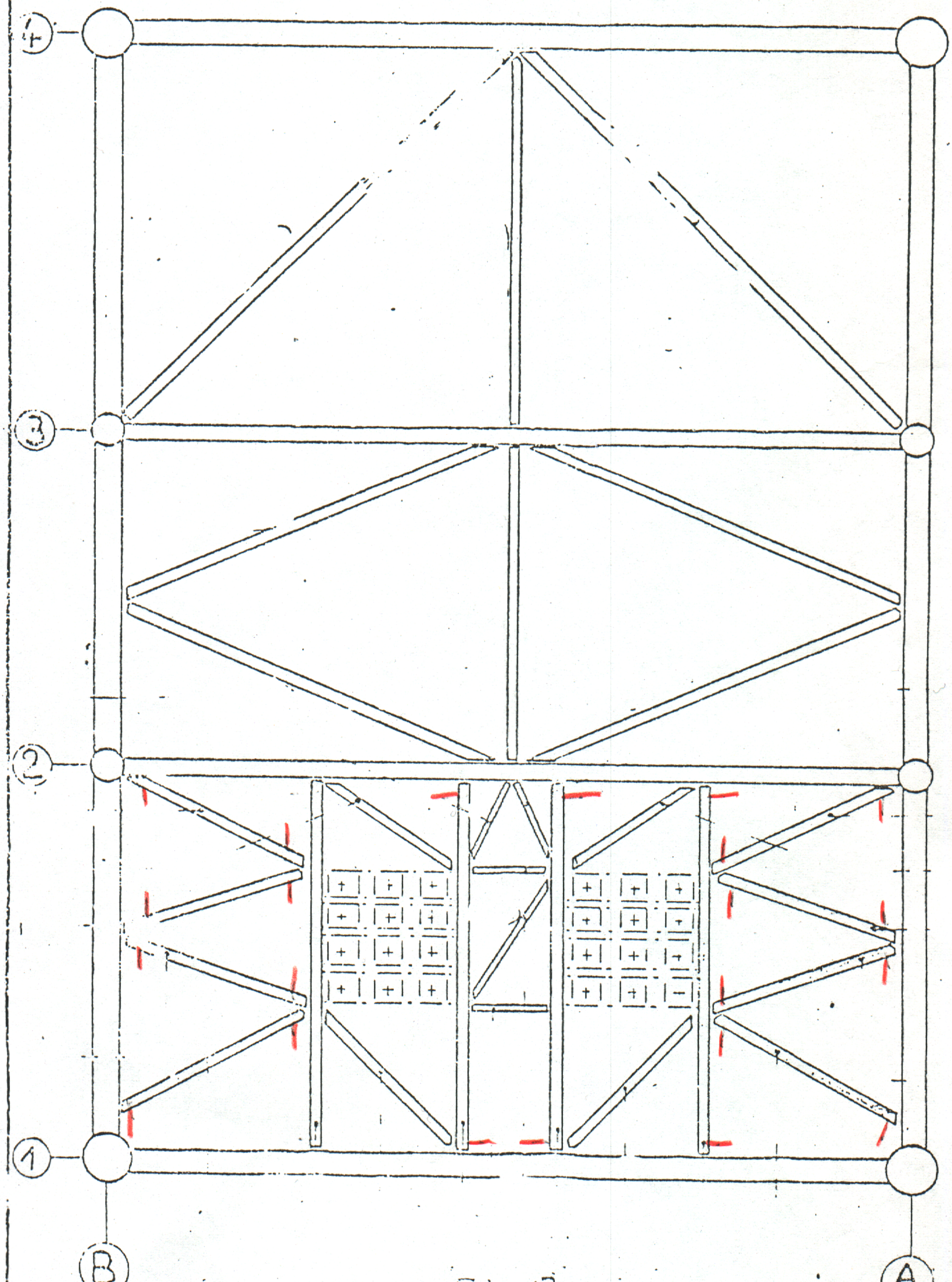
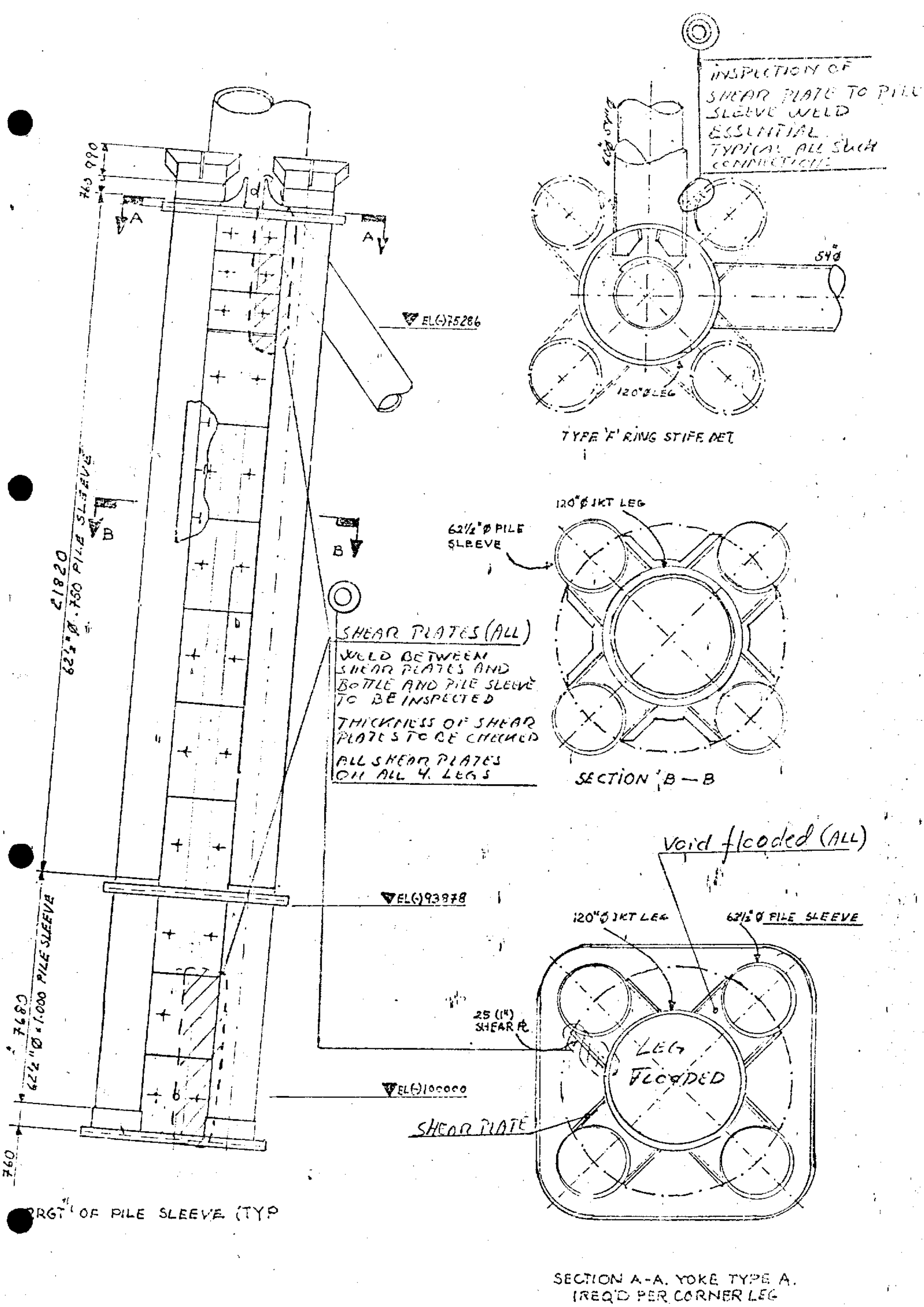


Fig. 5.12

FRIGG DP2
Niveau (-) 100000

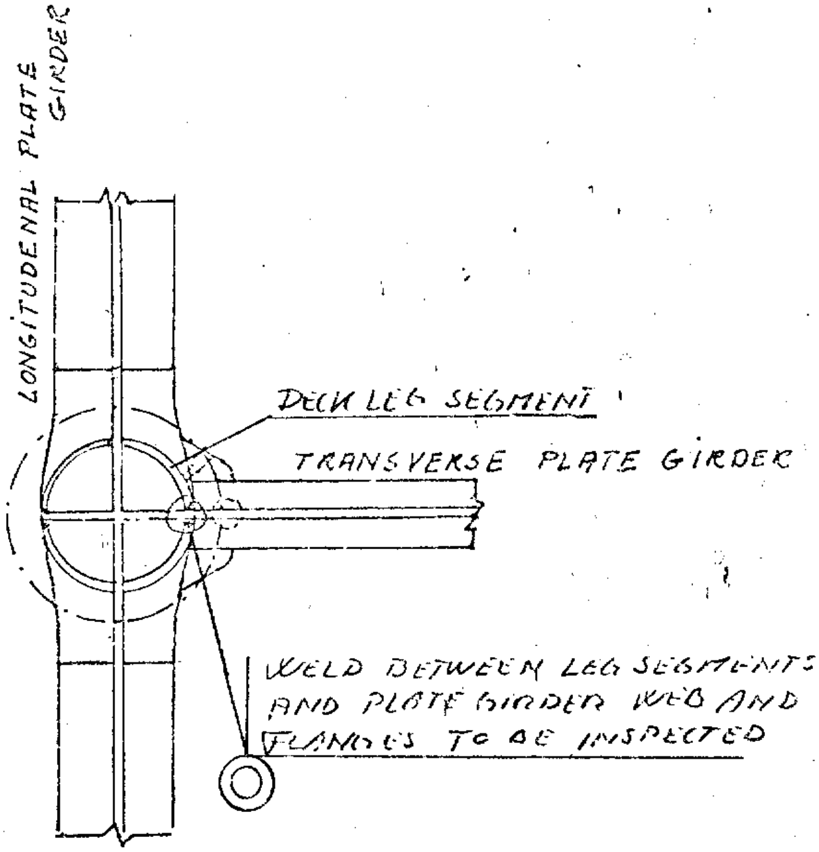
Siège : 49b avenue Hoche - PARIS
Etablissement de CHERBOURG

Dessiné	Le 21.08.75
Visa :	Echelle 1/300
N° Fo 454.633	12/12

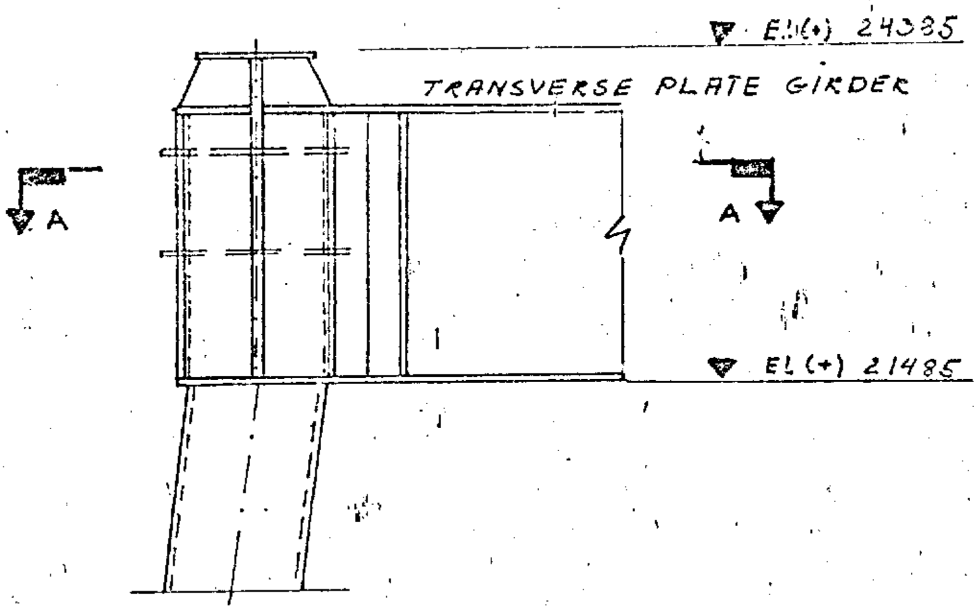


INSPECTION OF SHEAR PLATE TO PILE SLEEVE WELD ESSENTIAL. TYPICAL ALL SUCH CONNECTIONS
540
120"Ø LEG
TYPE 'F' RING STIFF DET.
A
A
990
760
EL(-)75286
21820
62½"Ø · 750 PILE SLEEVE
B
B
120"Ø JKT LEG
62½"Ø PILE SLEEVE
SHEAR PLATES (ALL)
WELD BETWEEN SHEAR PLATES AND BOTTLE AND PILE SLEEVE TO BE INSPECTED
THICKNESS OF SHEAR PLATES TO BE CHECKED
ALL SHEAR PLATES ON ALL 4 LEGS
SECTION B — B
Void flooded (ALL)
EL(-)93878
120"Ø JKT LEG
62½"Ø PILE SLEEVE
25 (1") SHEAR PL
LEG FLOODED
7680
62½"Ø x 1.000 PILE SLEEVE
EL(-)100000
SHEAR PLATE
760
RGT OF PILE SLEEVE (TYP
SECTION A-A. YOKE TYPE A.
1 REQ'D PER CORNER LEG

FIG 5.13

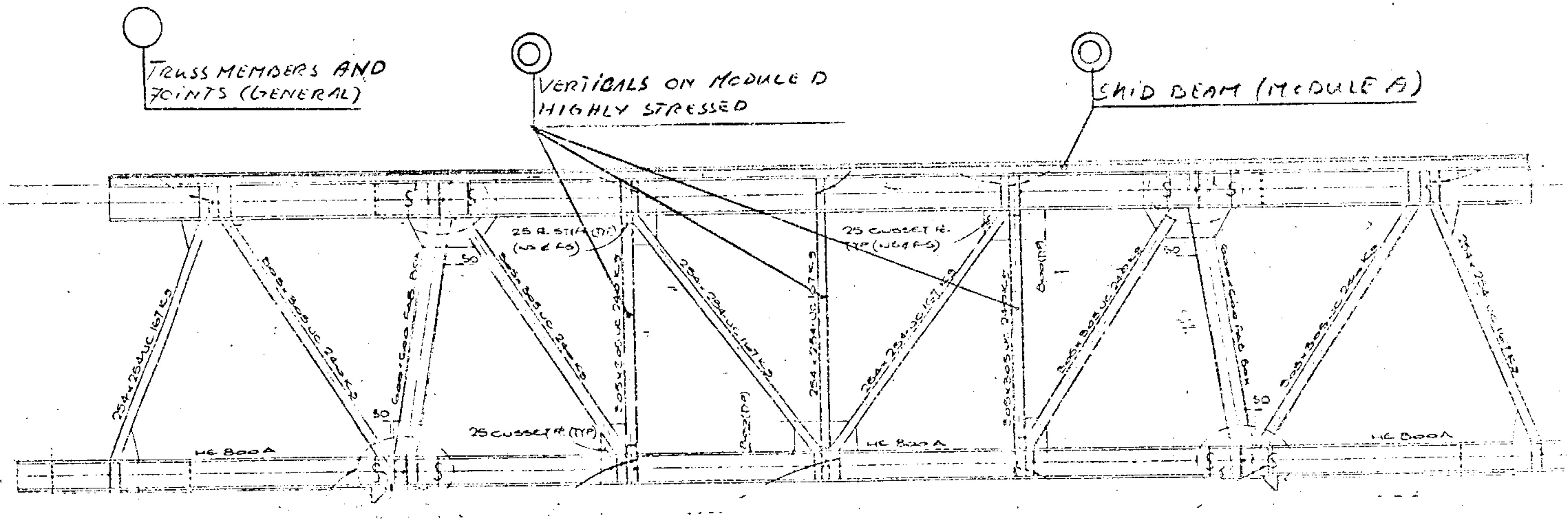


SECTION A-A



TYPICAL DECK LEG - PLATE GIRDER CONNECTION

FIG. 5.14



MODULE TRUSS (TYPICAL)

Fig 5.15

APPENDIX NO. 7

to

DnV REPORT NO.601636/12

SUMMARY OF PILE INSTALLATION

1. Introduction

This note contains a summary of the observations made by DnV in connection with a review of the various records received conserning the installation of Frigg DP2 Piles.

The tow first paragraphs briefly outline the observations for the Main- and Insert Piles. The third point discuss in more detail the grouting of the Piles.

2. Main Piles

2.1 Pile Driving

All main piles have been installed. The piles were driven with either a Vulcan 060 or a Vulcan 560 hammer to sufficient penetration without any need for relief drilling. The piles in B4 corner met considerable greater driving resistance than the others.

2.2 All driven piles have been completely grouted to the jacket.
Grout samples were obtained from the slurry return and the sepcified grout density was confirmed during the operations. Grout strength tests seem to yield somewhat low results.

3. Insert Piles

3.1 All insert piles have been installed to design depth-approx. 385 ft. below mudline. The drilling of the holes and inserting of the piles were done without encountering deviations from expected behaviour. Grouting of the piles caused, however, some problems. The following are reported.

Pile A4.3 Slow return at end of 1. stage
Pile A4.4 Slow return at end of 1. stage
Pile B1.1 1. stage grouted in two steps. 1 step to tip of main pile, 2. step to 2. stage level.
Pile B1.4 1. stage grouted in several steps in order to establish a plug inside main pile. Fron this plug the annulus was grouted in one stage.
Pile B4.2 2. stage grotued in two steps.

All these observations indicate a possibility of soil fracturing by hydraulic pressure. The procedure of tracing the grout by means of a radioactive source was not completely successful. However, the presence of grout has been confirmed at the end of each stage by sampling of return flow.

A review of the grouting records gives evidence for that corrections were taken against the problems met, and all probable soil fractures were sealed off. The increased numbers of steps in the grouting prosess is not expected to affect the pile capacity to any essential degree.

Compressive strength test results from the grout samples do not meet required specifications. The effect of low strength test results are discussed in the following chapter. Otherwise the insert piles are installed in an acceptable way.

4. Pile grouting

4.1 For grouting of the main piles to the jacket legs and the insert piles to the main piles the use of a grout mix with compressive strength of 2000 psi have been assumed in the original structural design calculations of pile foundation (McDermott-Hudson, "Design Calculations, Piles", Nov.1975). DnV's review of the calculations revealed a need for a greater number of shear lugs and/or a higher grout strength in order to meet the requirements of the applicable standards.

(DnV's telex no. 2865, dated 1976-03-02) McDermott-Hudson specified the use of cement grout with compressive strength of 4000 psi and minimum bond strength of 90 psi. With the increased grout strength and the proposed grouting procedure (McDermott-Hudson, "Installation procedure", Revision 1, May 1976) DnV was satisfied with the degree of safety incorporated in the design of pile foundation.

4.2 Due to installation difficulties and some commercial problems three types of cement were used for pile grouting on the platform: (i) for primary piles grouting, a grout mix based on DIACEMOIL cement was used.

4.3 In order to ascertain compliance of grout produced with the strength required, the quality control programme stipulated sampling at the site of compression test speciments for subsequent laboratory testing.
To assure an acceptable safety level for transfer of loads through grout in the annulus between steel piles the following strength requirements were specified:
for case (i) compressive strength of 4000 psi after 28 days for case (ii) minimum bond strength of 180 psi after 28 days.

4.4 In order to verify the bond strength achieved by grouting of the annulus between steel piles a report on tests performed at Elf's Central Laboratory has been submitted to DnV - "Procedures and Equipemnts for Testing Cement slurries (and results of DIACEMOIL Cement slurries), SNEA(P), Report no. 2051-6/5.249-JC/cp".

The documentation received give no information on subjects such as:

- how the samples were prepared, giving details on: mix proportions and mixing procedures adopted, the method of filling of the annular space with grout, curing conditions, etc.

- surface condition of steel tubes used in the tests

- number of samples tested, i.e. the statistical basis on which the recommended shear bond strength had been assessed.

- other basic properties of the diacemoil grout mix, such as shrinkage, free water, etc.

- failure modes observed.

A detailed knowledge of the above particulars is important for a correct interpretation of the test results. Moreover, in order to study the relevance of the tests performed to simulate load transfer taking place in the annulus between large diameter piles, and to evaluate whether the ultimate bond stress assessed gived a reliable value, influence fo the following items - all known to have a prounced effect on bond strength - should be further investigated:

(i) the influence of sample geometry on bond strength including:

- the effects of samples different length to diameter ratio

- the effects of diameter to wall thickness ratio

- the effects of sample's size

(ii) the influence of mechanical disturbance from jacket movement while the grout is setting.

(iii) the effects of contamination of the grout by drilling mud, presence of a mud film on the piles, and water intermixing during placing of the grout.

(iv) the influence of compressive and tension pile loading on the grout connection and the effects of repeated loading.

(v) long term field service performance of DIACEMCIOL grout.

4.5 Review of the received pile grouting records and tests certificates disclose that the grout mix produced failed to meet the strength requirements for the following primary piles: A1-1, A4-1, A4-2, A4-3, B1-2, B1-3, B4-1 and B4-2. The average compressive strengths as low as 2857, 2665 and 2732 psi for piles A4-2, B1-3 and B4-2 was recorded which mean a discrepancy of as much as 30% from the required strength.

4.6. Acceptance of the pile foundation on DP2 platform is qualified with the following items:

(i) Primary pile grouting; New calculations which demonstrate sufficent degree of safety for load transfer between the main piles and the jacket legs applying the compressive strength actually recorded should be submitted to DnV for approval.

(ii) Insert pile grouting; The actual bond strength of the DIACEMOIL grout should be adequately documented.

APPENDIX NO. 8

to

DnV REPORT NO. 601636/12

FINAL SURVEY REPORT FROM FRIGG FIELD

Det norske Veritas

INSPECTION SERVICES, WEST NORWAY

ADDRESS: CARL KONOWSGT. 36, N-5031 LAKSEVÅG, NORWAY TELEPHONE: (05) 26 00 12

TELEX: 42 913 dnvbg n

TECHNICAL REPORT

VERITAS Report No. 601636/77/36/1	Subject Group
Date 17.11.77	
Department 36	Project No. 601636
Title of Report: INSTALLATION DP2 STRUCTURE FRIGG FIELD PHASE II	Approved by: M. OLSEN
Client/Sponsor of project: NORWEGIAN PETROLEUM DIRECTORATE	Client/Sponsor ref.
Work carried out by: SULEN / DAHLBERG	Reporters sign.

Summary

This report is a summary of the inspection work carried out offshore by DnV-surveyors from DnV-Bergen office.

The report covers all phases of installation of the Jacket - Piles - Deck Modules. It does not cover risers or hook-up of production equipment.

4 Indexing terms

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Date of last rev.	Rev. No.	Number of pages

INTRODUCTION

This report is a summary of the work done offshore by DnV-Bergen during the installation of DP2 structure on the Frigg Field.

The report covers:

1. Launching and installation
2. Pile - Installation - Main
3. Deck Support Frame
4. Welding - fieldsplice DSF/JACKET
5. Temporary modules - installation
6. Pile installation - insert
7. Production module installation
8. Miscellaneous

1. LAUNCHING AND INSTALLATION

The DP2 Jacket was towed out from Cherbourg, France to the Frigg Field on barge Intermac 600. It was launched in the Norwegian part on the Frigg field at 0700 on May 11th, 1976. After upending and towing to location it was set down on the bottom at 1400 the same day, using approximately 7 hours to complete the operation. The final location is:

N 59° 53' 10.08"
E 02° 04' 20,60"
ORIENTATION 332,5° TRUE NORTH.

The operation was supervised by Oceanic Contractors. After the four tugboats had been released from DP2, the barge DB22 moved in and anchored up at the Jacket's A-side. A underwater survey was carried out, but did not reveal any structural damages.

2. PILE INSTALLATION - MAIN

The pile installation started with mainpile A3 and continued with A2 - B2 - B3. When the four centerpiles reached penetration (21 m), the Jacket was levelled by floating A-side, then shimmed off and welded. The welds were subject to magnetic particle inspection and found acceptable. The four corner main pile A1-4, B1-4, A4-4 and B4-4 were then driven to penetration (18 m). Before any further piledriving was carried out, the eight piles now driven were grouted to the structure. The Grouting was carried out in one stage except the grouting of centerpile B3. Due to problems with the minipacker on bottom this pile was grouted in two stages. Samples from the grout during grouting were taken. Six cubes 4" x 4" x 4" were made at the beginning and the end of each stage. Together with the sample a PILE GROUTING RECORD was written and signed by the local DnV-surveyor. This was sent to DnV-headoffice for evaluation. The samples were sent to laboratory for 7 and 28 days' test.

When this grouting operation was completed, the Temporary Work Deck was installed on June 24th on the Jacket's four centerlegs. The plan was to install a ringer crane on the Temporary Work Deck to do the lifting operation for the next twelve main piles. However, they never installed the ringer crane on the Temporary Work Deck due to good weather and good workmanship of the crew on barge DB22. Thus all the mainpiles were installed from DB22. The installation, including grouting of the twenty main piles, sixteen corners and four centers, was completed on July 14th, 1976.

The Temporary Work Deck was removed from the Jacket by DB22 on July 20th, 1976.

All PILE DRIVING RECORDS were signed by the local DnV-surveyor on the barge and sent to DnV-headoffice for evaluation. No jetting or drilling was carried out during installation. All piles reached penetration within limit of design.

3. DECK SUPPORT FRAME

After the Temporary Work Deck was was removed, the four center-legs were cut and bevelled in order to install the Deck Support Frame. When all necessary preparation had been carried out, barge DB22 left the Jacket making way for a bigger barge,PM27, which was selected to perform the lifting of the Deck Support Frame. The lift was carried out in the night between August 3rd and 4th, 1976. Immediately after the lift was finished, PM27 left the Frigg Field and DB22 came back and anchored up at A-side on DP2 to do the welding of the Deck Support Frame/ Jacket field splice.

4. WELDING - FIELDSPLICE DSF/JACKET

The eight fieldsplices between DSF/JACKET were welded according to accepted welding procedure, prescribing minimum 150°C pre-heat and ESAB OK 73.68 electrodes. All welders on DB22 were certified. ASME 6G.

After completion of the eight welds magnetic particle inspection and ultrasonic inspection were carried out and five of the welds had to be locally repaired, e.g. cracks and slagline. Because of doubtful qualifications and interpretation of results, one of the ultrasonic technicians was replaced. When all repairs were completed, one of DnV's own ultrasonic experts carried out the ultrasonic inspection of all the eight welds. The welds are acceptable to ASME VIII.

5. TEMPORARY MODULES INSTALLATION

Between August 10th and 20th, 1976 eight temporary work modules were installed to do the insert piling work. No living accomodation existed on DP2 before November 2nd, 1976. In the meantime DB22 and Aker H3 rig "Treasure Hunter" were hotel and supply rig. The temporary quarter/helideck module was lifted onto the platform by barge ETPM 1601 on October 31st, 1976.

6. PILE INSTALLATION - INSERT

The installation of the sixteen insert piles started on 19th September, 1976 and was completed on 15th December, 1976. The work was carried out by Oceanic Contractors and supervised by ELF. No major problems occurred during installation. Each pile was welded according to accepted welding procedures, and each weld was subjected to ultrasonic and magnetic particle inspection. All welds are accepted accoring to ASME VIII. The insert pile was lowered down in a predrilled hole 117 m into the soil. It was then grouted in two stages accoring to procedure. Twelve cube samples were taken from each stage. The PILE GROUTING RECORD was signed by local DnV-surveyor and sent to DnV-headoffice for evaluation.

7. PRODUCTION MODULES INSTALLATION

When the work with the insert piles terminated, all temporary modules were lifted off DP2 by barge ETPM 1601 in order to install the permanent Production Modules. The A and B Production Modules were lifted onto DP2 on January 17th and 18th, 1976, and C and D on February 9th, 1976. When skidding of the modules were completed, welding between modules and DSF started. After completion of the welds first time cracks/ lammellar tearing were found in the base material. Due to contract completion Oceanic Contractors demobilized and the hook-up contractor U.I.E. took over the repair work. After welding procedures were established and accepted U.I.E executed the repairwork to all parties' satisfaction.

8. MISCELLANEOUS

Some outstanding work from yard was done offshore. Stiffening of conductor panel in accordance with drawing no. ELN 2144 sheet no. T 158 at el. + 20' was done by Oceanic Contractors. All welding were subject to magnetic particle or ultrasonic examination. Walkways, handrail and ladders were installed. General clean up and removing of temporary structures and attachments were done. Magnetic particle examination was carried out to the satisfaction of the DnV-surveyor. All exposed areas and fieldwelds were sandblasted and painted. Boatbumper and boatlanding were installed according to drawings. After all heavy construction works were completed and support-rig left, a underwater survey of the structure was carried out by a submersible (RCV), but it did not reveal any damages. Where nothing else is specified, the inspection offshore by DnV-surveyors is carried out in accordance with ASME section V and section VIII.

ELF-NORGE FRIGG FIELD
FIXED OFFSHORE STRUCTURES
FABRICATION SPECIFICATION REVISION 2

ELF-NORGE FRIGG FIELD
DRILL PLATFORM NO 2
INSTALLATION PROCEDURE REVISION 1

DNV TECHNICAL NOTE FOR FIXED OFFSHORE STRUCTURES.

5. Special Considerations Including Fabrication and Installation

5.1. Jacket

The following figures 5.1 to 5.13 point out those areas which based on DnV's inspection during fabrication and installation are considered critical.

5.1.1 Heavy Wall Joints

Joints where the brace end or stub thickness is equal to or exceeds 50 mm must be thoroughly inspected in the future. This inspection was agreed upon during the design and fabrication of the structure and rigorous in-service inspection of these welds was a fundamental basis for DnV's acceptance of these joints. It is essential that these welds be regularely and frequently inspected, and, since these weldments have comparatively small critical crack sizes, emphasis must be laid on detecting any cracks at an early stage.

These joints are marked with a tripple circle (◎)

5.1.2 Shear Plates

The shear plates between corner legs and pile sleeve transfer the loads from the jacket corner legs into the piles. These plates are highly stressed in the upper and lower 5,0 m region.
The void enclosed by the shear plates, the corner leg and the pile sleeve is also filled with sea water. Although water is theoretically prevented from circulating in/out it is considered essential that the thickness of these plates be regularely measured.

It is also important that the weld between the shear plates and pile sleeve as well as between shear plates and corner legs and incoming braces be regularely inspected in the upper and lower 5 m lengths.

Also the weld between the yoke plates and the shear plates, upper and lower, are highly stressed.

The relevant welds are shown on Fig. 5.13.

5.1.3. Bottom Horizontals

Generally, the leg joints at mud line elevation (-)100m, are also highly stressed and should be inspected accordingly. These joints have been incorporated here because any irregularity in the modelling of the piles in the computer calculations may greatly effect the stress level in these joints.

5.1.4. Jacket/Support Frame Field Splice

The welds between the Support Frame legs and the jacket legs or piles experienced repeated cracking during field welding. These butt welds are relatively highly stressed and due to the repeated repair of these welds it is considered necessary that they be subjected to regular inspection.

5.1.4. Attachments

In order to support buoyancy tanks and later on guide the piles, pile guides were installed around each corner leg.
The pile-guides installed on elevation (+)6.095 m and also on elevation (+)21.485 m were removed after installation of piles.
In order to ascertain that no cracks develop at these locations, regular inspection should be conducted, particularly in an initial phase. Also the welds attaching boat landings and barge bumpers should be inspected.

5.1.6 Corrosion

It is believed that general measurements of wall thicknesses will be done. This appendix does not give any detailed suggestion as to where such measurements should take place. It is assumed, however, that appropriate areas be selected based on the previous figures, e.g. those defining critical areas with respect to stresses.

It is also assumed that special attention be paid to the members in the splash zone, and not at least flooded members and compartments. Flooded compartments and members are defined in the following figures.

5.2. Deck Support Frame

This chapter (5.2) and Fig. 5.14 point out those areas of the Deck Support Frame which based on DnV's inspection during fabrication as well as partly also the design review, are considered critical.

5.2.1. Deck Legs/Deck Beam Joints

The deck support frame is generally not very highly stressed.

However, the joints between deck legs and deck beams are of utmost importance and furthermore fabrication of these connections experienced some problems.
It is thus essential that these welds be regularely inspected.

For further details of critical areas see Fig. 5.14.

5.2.2. Attachments

Attachments and members welded to the flanges of the deck support frame involve areas suspectible to fatigue cracking. This applies to areas where walkways, deck modules, and (+)21.485m horizontal members are welded to the flanges of the deck beams. Also the areas where the storage tanks and pump house is welded in should be regularely inspected.

5.3. Production Modules

This chapter (5.3) and Fig. 5.15 point out those areas of the Production Modules which based on DnV's design review are considered most critical.

5.3.1 Module Beams

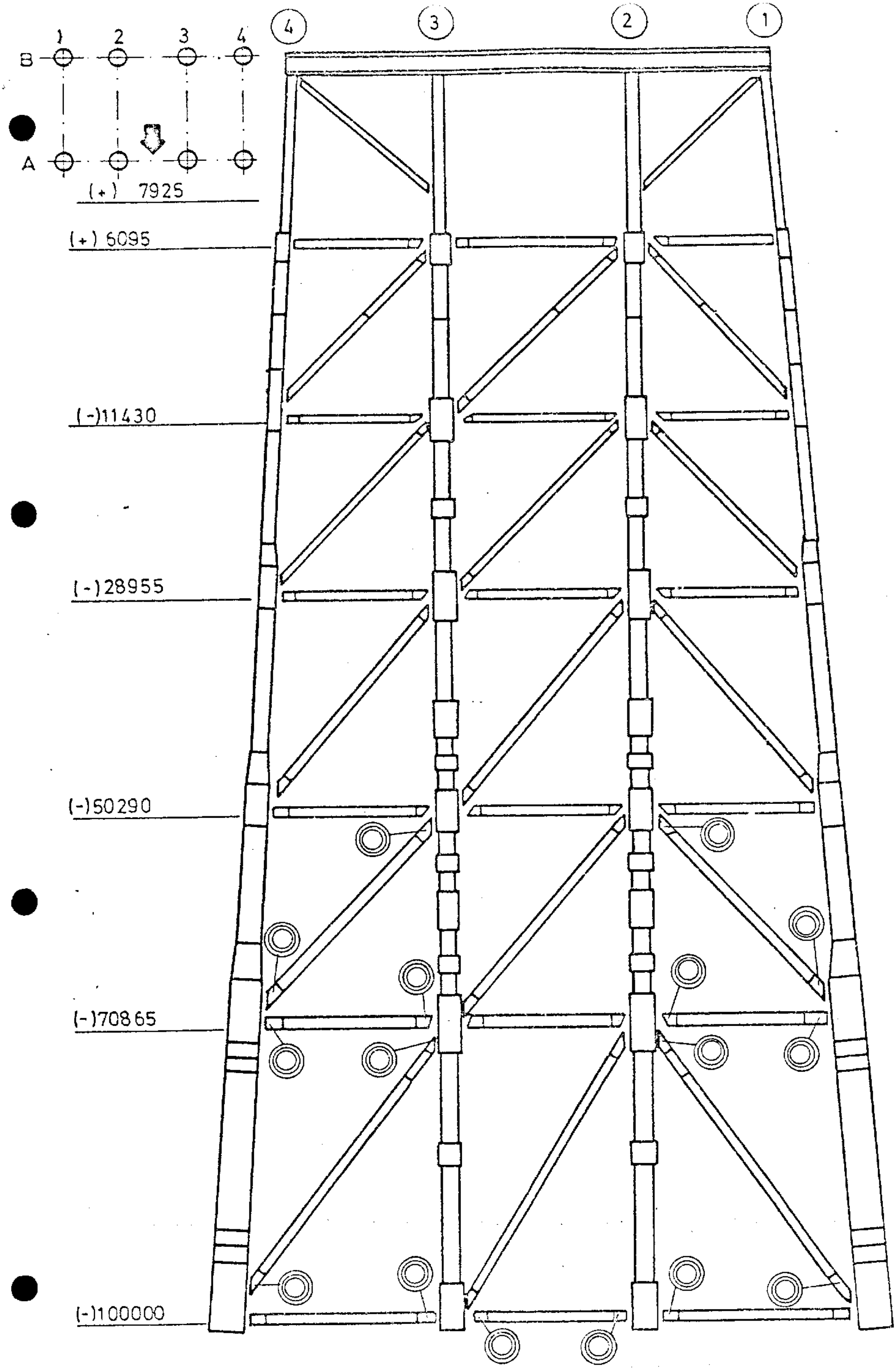
The production modules A, B, C and D are generally not very highly stressed.
However, some truss member in module D are marginally loaded.
These members are shown on Fig. 5.15.

Similar members on the remaining modules (A, B and C) should also be inspected regularely.

5.3.2. Module Supports

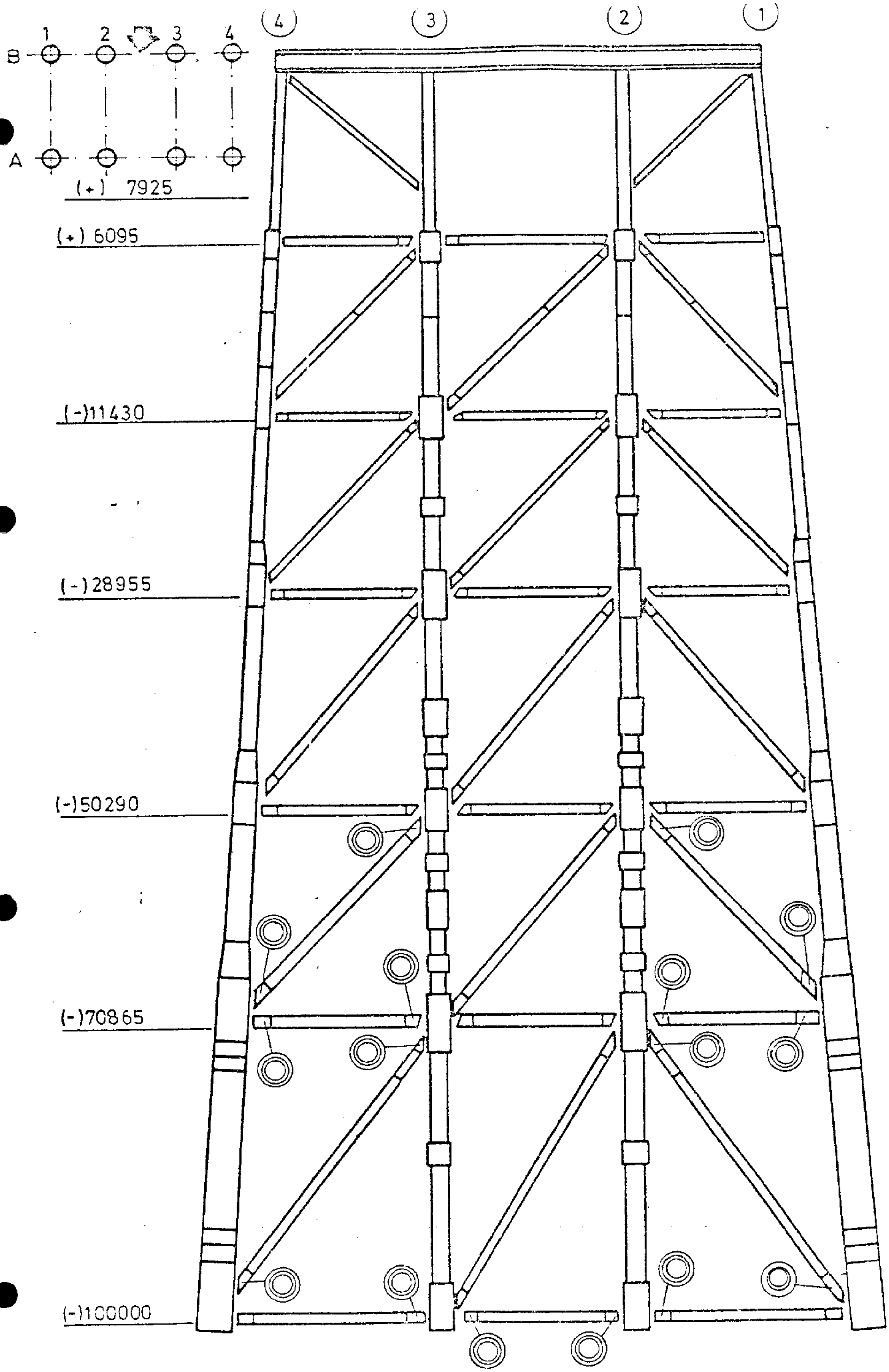
Also module supports should be subjected to regular inspection. This applies to the welds between shim plates and module structure and between shim plates and deck support frame.

In module A also the skid beams should be inspected, particularly the areas supporting the sub-structure skid beams.



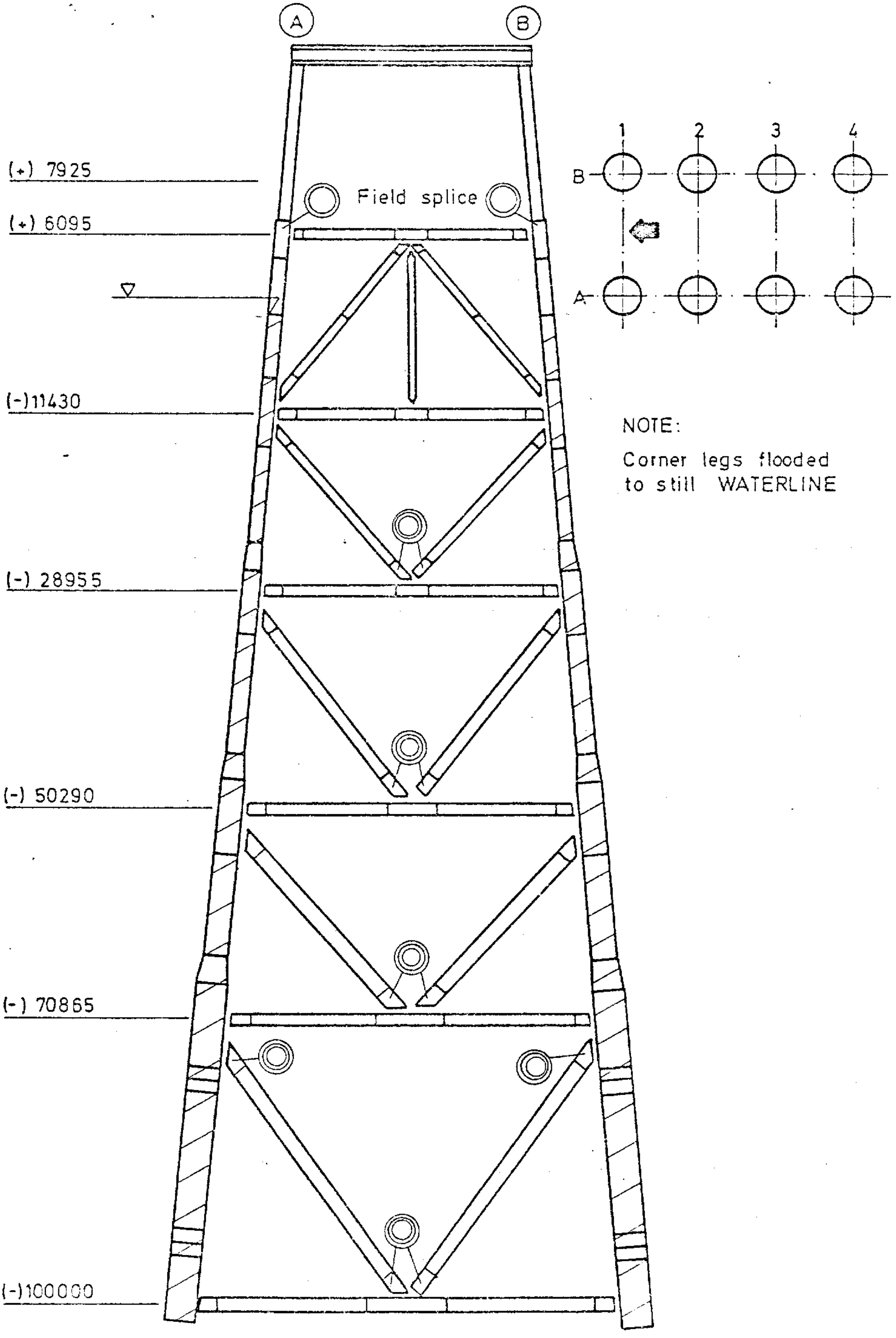
FRIGG DP2   COL. LINE A

FIG. 5.1

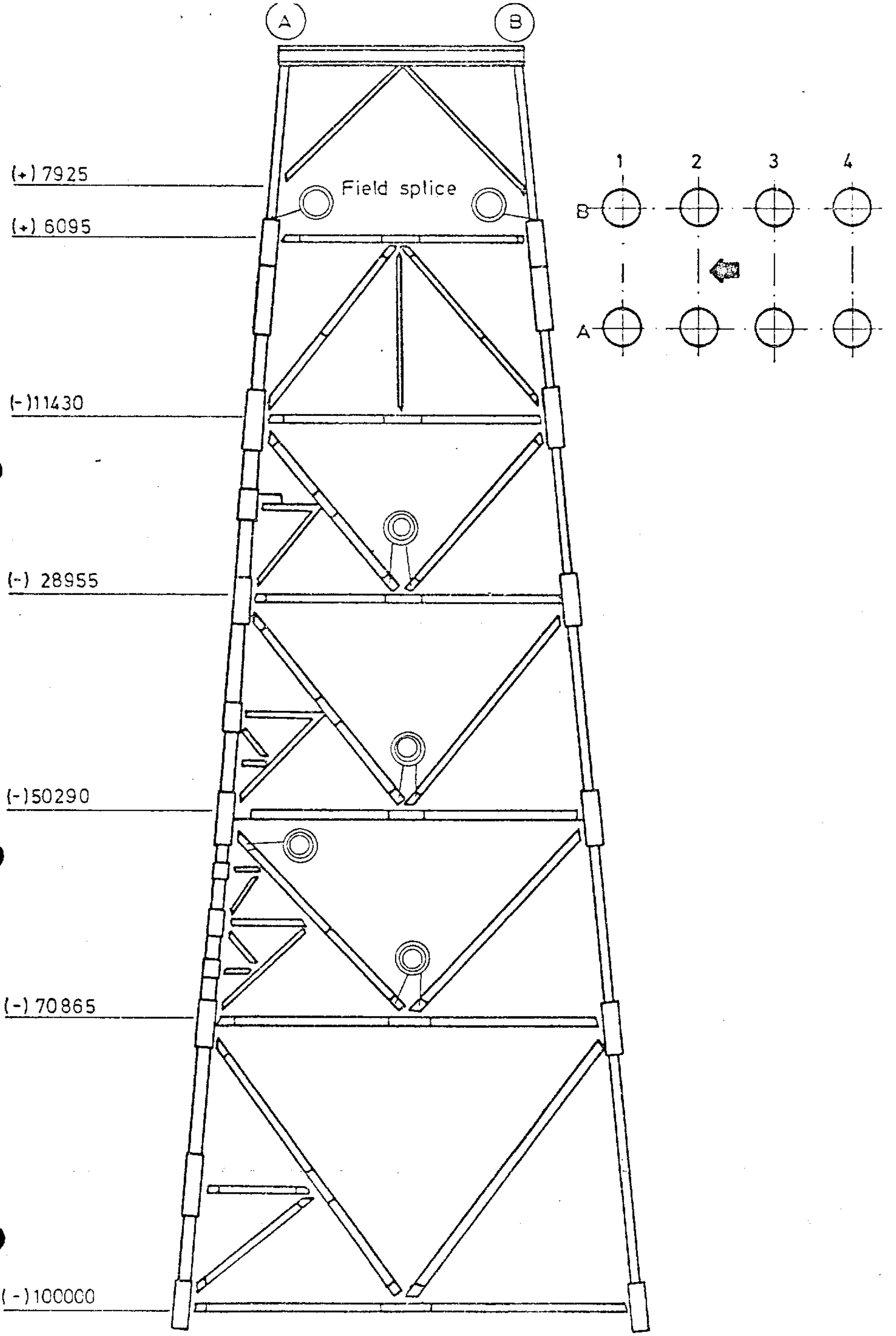


FRIGG DP2 COL. LINE B

FIG.5.2



FRIGG DP2 COL LINE 1



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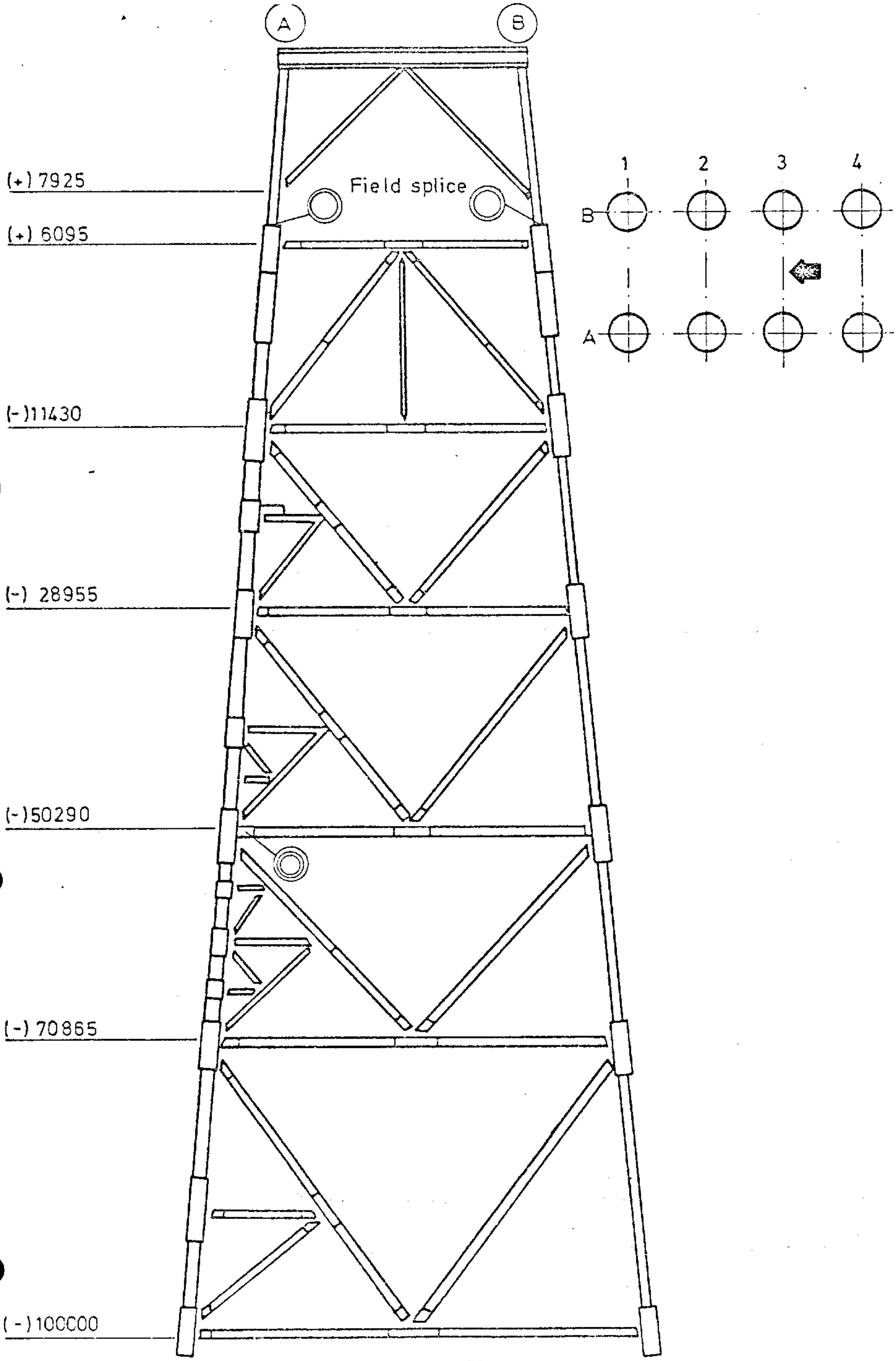
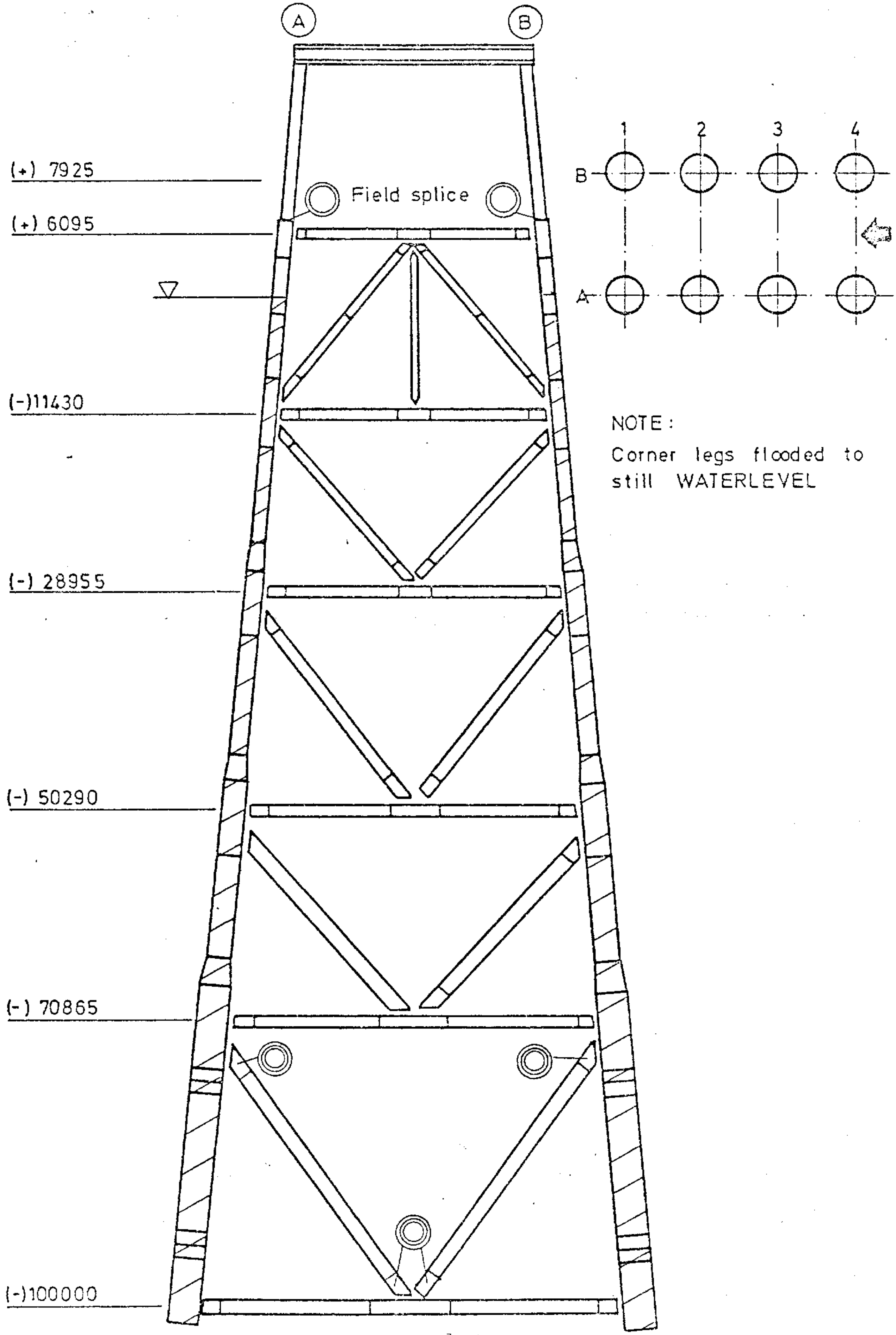
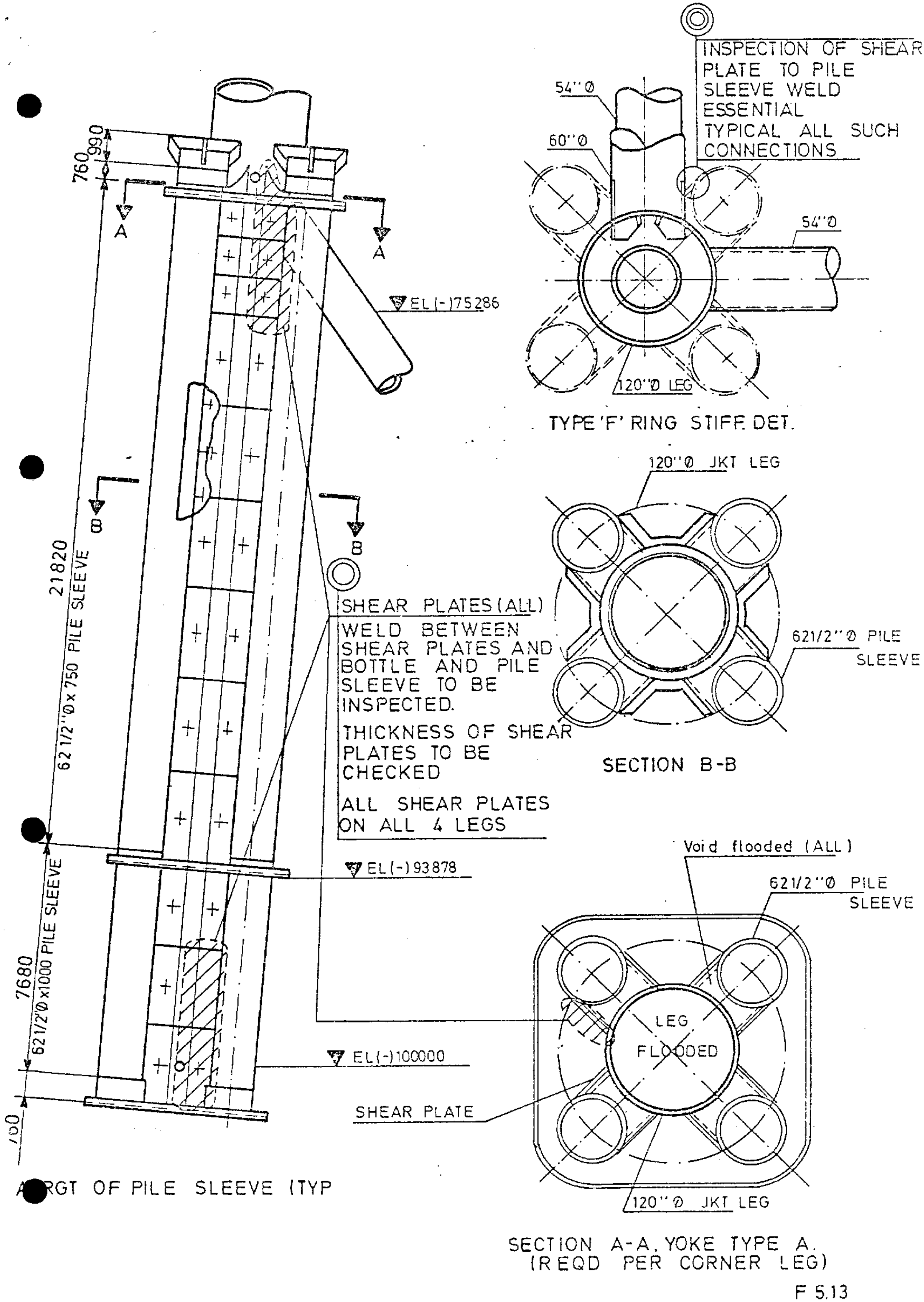


FRIGG DP2 COL. LINE 3

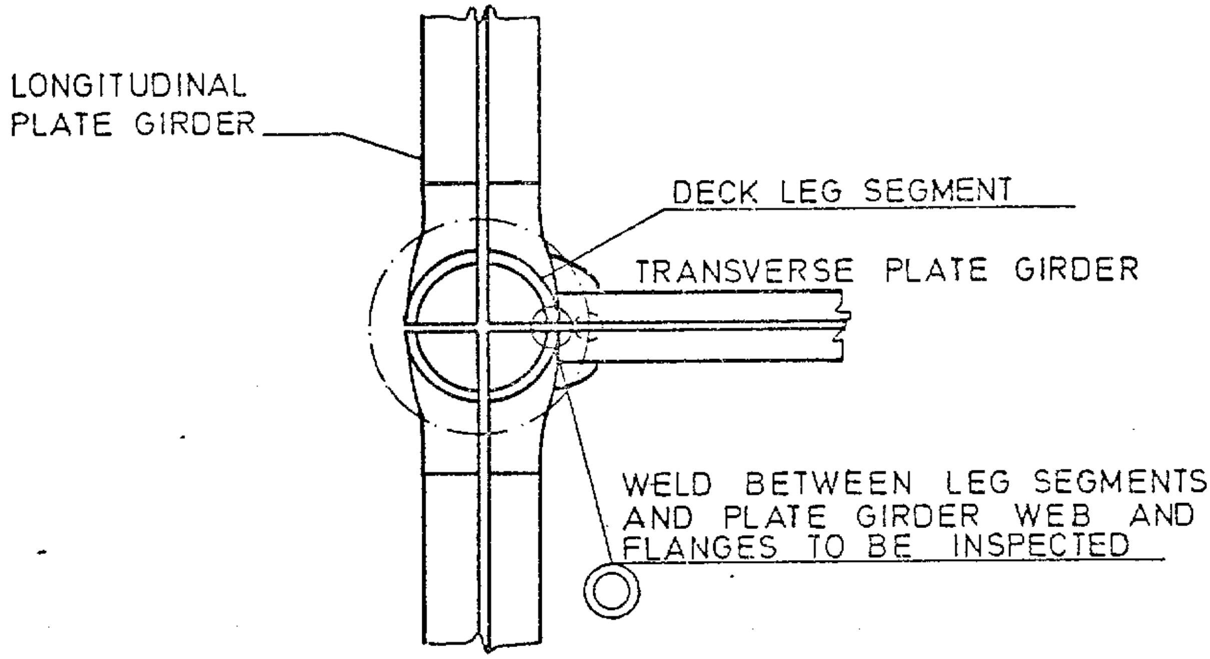


FRIGG DP2 COL. LINE 4

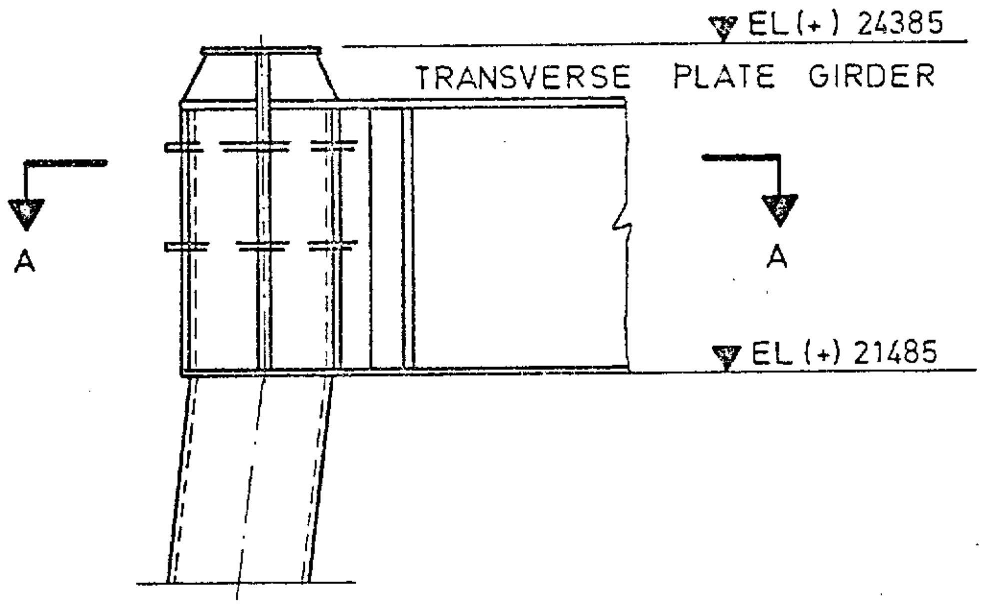


INSPECTION OF SHEAR PLATE TO PILE SLEEVE WELD ESSENTIAL
TYPICAL ALL SUCH CONNECTIONS
54"Ø
60"Ø
54"Ø
120"Ø LEG
TYPE 'F' RING STIFF. DET.
990
760
A
A
EL(-)75286
B
B
21820
62 1/2"Ø x 750 PILE SLEEVE
SHEAR PLATES (ALL)
WELD BETWEEN SHEAR PLATES AND BOTTLE AND PILE SLEEVE TO BE INSPECTED.
THICKNESS OF SHEAR PLATES TO BE CHECKED
ALL SHEAR PLATES ON ALL 4 LEGS
120"Ø JKT LEG
621/2"Ø PILE SLEEVE
SECTION B-B
EL(-)93878
7680
621/2"Ø x1000 PILE SLEEVE
EL(-)100000
Void flooded (ALL)
621/2"Ø PILE SLEEVE
LEG FLOODED
SHEAR PLATE
120"Ø JKT LEG
ARGT OF PILE SLEEVE (TYP
SECTION A-A, YOKE TYPE A. (REQD PER CORNER LEG)

F 5.13

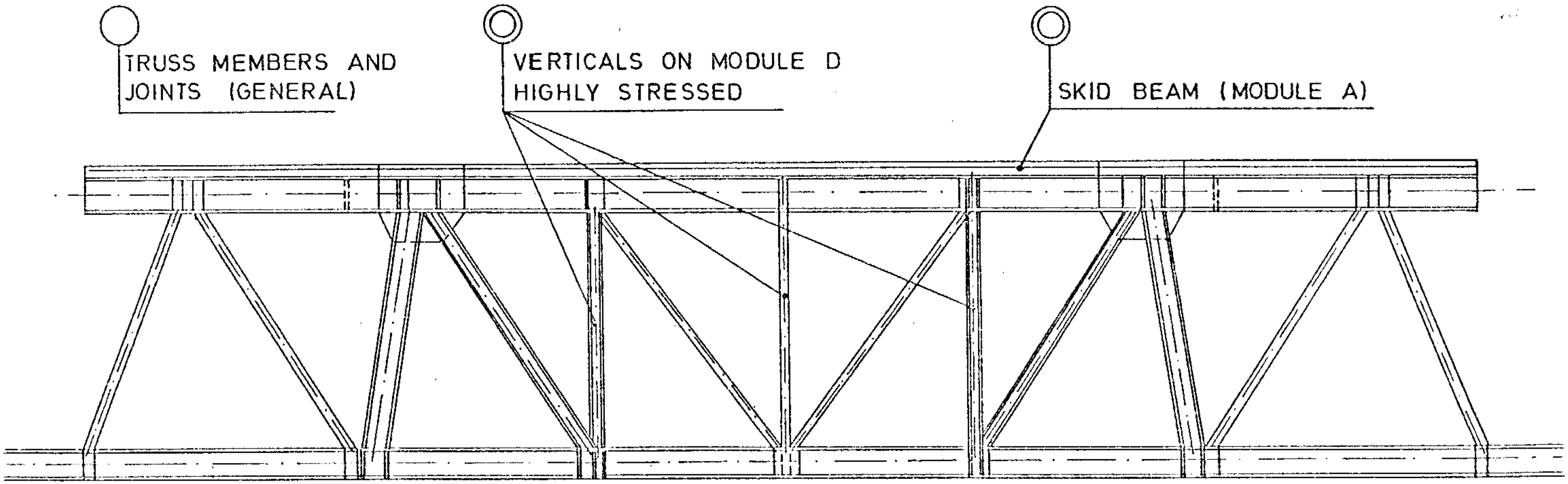


SECTION A-A



TYPICAL DECK LEG PLATE GIRDER CONNECTION

FIG. 5.14



MODULE TRUSS (TYPICAL)

FIG 5.15