C2. Structural engineering in arctic regions

Objekttyp: **Group**

Zeitschrift: **IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht**

Band (Jahr): **12 (1984)**

PDF erstellt am: **15.09.2024**

THEME C2

Structural Engineering in Arctic Regions

Structures de génie civil dans les régions arctiques

Konstruktiver Ingenieurbau in arktischen Regionen

Chairman:	G.F. Fox, USA
Coordinator:	M. Virlogeux, France
General Reporter:	P. Jumppanen, Finland

The theme has been introduced by the General Reporter in the Introductory Report of the 12th Congress, available at the IABSE Secretariat.

Bridge Construction under Severe Natural Climatic Conditions in the USSR

Construction de ponts dans les régions à climat rigoureux en URSS

Der Brückenbau in den Regionen der UdSSR mit harten Klimabedingungen

V.N. SAFONOV
Civil Engineer
Gosstroy
Moscow, USSR

K.S. SILIN
Dr. Sci.
All-Union Res. Inst. of Transp. Constr.
Moscow, USSR

G.P. SOLOVYEV
Dr. Sci.
Glavmostostroy, Min. of Transp. Constr.
Moscow, USSR

A.V. CHERNYSHOV
Civil Eng.
Glavtransproject, Min. of Transp. Constr.
Moscow, USSR

SUMMARY
The paper deals with the experience recently gained through engineering design and construction of bridges in the North and East regions of the USSR. Scientific researches have formed the basis for the design solutions selected. Minimum environment disturbance, use of materials and structures of the required quality as well as elaboration of special technology for work execution make it possible to develop economically efficient structures of bridges and their approaches.

RESUME
Le rapport concerne l'expérience dans la construction des ponts dans les régions du nord et de l'est de l'URSS, accumulée ces dernières années. Les projets retenus sont basés sur des études scientifiques. Les dommages minima à l'environnement, l'utilisation de matériaux et de structures de bonne qualité ainsi que l'étude des techniques de réalisation des travaux permettent de créer des structures solides et fiables de ponts et de leurs accès, du point de vue économique.

ZUSAMMENFASSUNG
Im Vortrag wird die in den letzten Jahren in den nördlichen und östlichen Regionen der UdSSR gewonnene Erfahrung im Brückenbau erläutert. Als Grundlage der angenommenen Entwurfslösungen dienten die wissenschaftlichen Forschungen. Die minimale Störung der örtlichen Bedingungen, die Anwendung der Baustoffe und die geforderte Qualität der Konstruktionsart sowie die spezielle Entwicklung der Ausführungstechnologie gewährleisten das Schaffen von zuverlässigen und wirtschaftlichen Brückenkonstruktionen und deren Zufahrten.

1. INTRODUCTION

The major part of the USSR territory, especially Siberia, is characterized by long winter periods with low temperatures, unfavourable engineering and geological conditions and lack of good local communications. Duration of low temperature periods (up to -60°C and lower) amounts to 200 days a year. The ground is covered with snow from September-October till April-May.

Permafrost occupies almost half of the USSR territory. The depth of permafrost is between 0.5 m and 4-4.5 km, the thickness varies from a few meters to as much as 1.5 km. In some regions the areas are swampy,the mountaneous regions are characterized by seismic activity. Many streams and rivers freeze up to the bottom, some of them are covered by ice crust.

In such extremely difficult conditions bridges reliability, durability and efficiency can be achieved by the following measures:

- use of such pier structures the erection of which would slightly affect natural conditions; use of machines and mechanizms for work execution;
- use of more precast elements;
- reduction of material consumption by material strength and grade improvement;
- conversion of construction sites into erection ones.

In severe climate regions the most complex problem is to ensure reliability and durability of short span bridges and to increase essentially their efficiency by material and labour cost reduction. Such bridges amount to more than 90 % by quantity and more than 70 % by total length of all the bridges being built.

The route of Baikal-Amur railway with an overall length of over 3000 km crosses the areas unfavourable for construction, i.e. severe climate, mountaneous relief, frozen soils of various properties, icings, high seismicity, etc.

Low-alloy steels and frost-resistant concrete have been used to ensure the required bridge reliability. As to foundations, the introduction of new structures and methods of work execution, slightly affecting the natutal conditions, have ensured the foundation stability in permafrost. In particular, fill material instead of ground cut-off was used for construction site planning. Water accumulation in front of bridges and several water courses passage through one opening were not allowed.

During the period of mass construction of bridges new codes of practice for design and construction of bridge foundations in permafrost have been refined or developed.

Fig. 1. Construction of foundation pit for massive pier

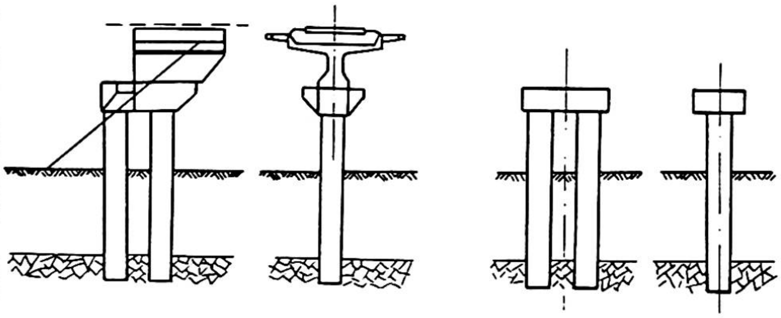


Fig. 2. Industrial construction of columnar pier

Fig. 3. Columnar piers bridge

Fig. 4. Completely precast columnar pier

2. BRIDGE PIERS

Traditional structures of piers with massive foundations appeared to be inadequately reliable and inefficient for BAM bridges. Such piers construction required much manual labour consumption for deep trenching to prevent the possibility of inadmissible pier settlement and inclination in case of frozen and ice saturated soils thawing. Occurence of such soils is usually at the depth of 5 m from the ground surface (Fig. 1).
Deep trenching required not only a great volume of earth and concrete works, but resulted in considerable disturbance of natural conditions. Due to the latter the force of frost heave increased and icings appeared in new places. All these factors could significantly reduce the reliability and durability of short span bridges. Moreover, the use of traditional piers couldn't ensure the completion of the project by the time fixed. Therefore, industrial structures of piers have been developed and widely used for short span bridges (Fig. 2).
Reinforced concrete columns of 0,8 m diameter were used as load bearing elements of these piers. They were sunk into holes predrilled in frozen and rock soils and on top integrated with precast cross beam. The superstructure and precast caps (for abutments) were placed directly on the latter (Fig. 3).

Every pier of short span bridge consists of up to 6 of such columns, being at the same time the members both of the foundation and the pier (Fig. 4).

The holes were bored by either percussion-churn drill (Fig. 5) or rotary drill (Fig. 6) depending on local ground conditions.

Precast members (columns, cross beams and caps) were manufactured at plants, delivered to the site by constructed railway section and then by road transport (Fig. 7). Piers were erected by general purpose cranes of 25-30 t capacity.

Cement-sandy grout was used for columns fixation in holes.
Columns and caps were cast in place by conventional methods (Fig. 8).

Use of columnar piers reduced concrete consumption by 2-4 times, cut down labour cost and shortened the erection period by 1,5-2 times and reduced the volume of earth works up to 10 times. All the above mentioned was achieved due to exclusion of foundation pit, use of precast elements and sharp reduction in concrete volume compared to the traditional piers.

The major part of BAM route crosses seismic regions with designed seismicity of 7 degree (measured on 12-degree scale).

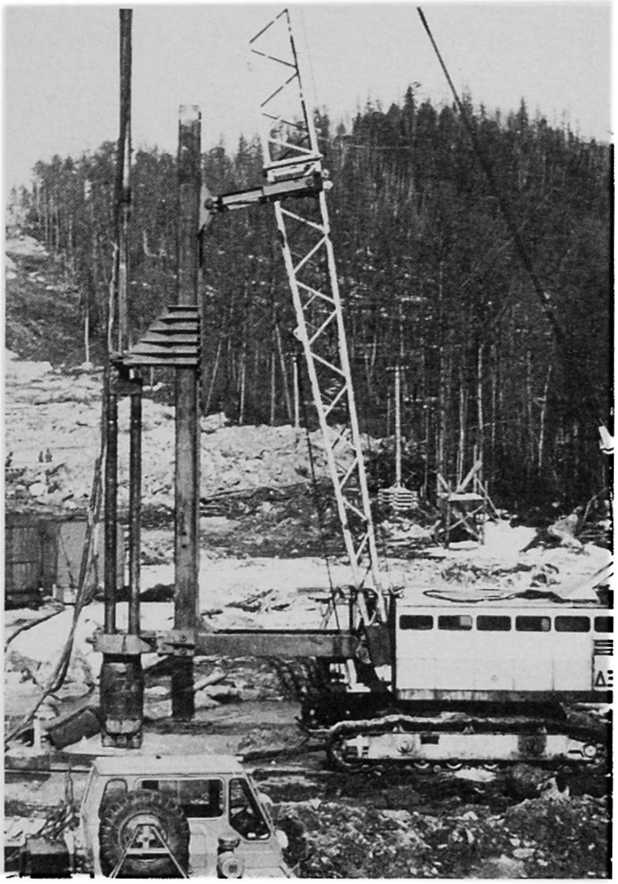


Fig. 6. Holes boring by rotary drill

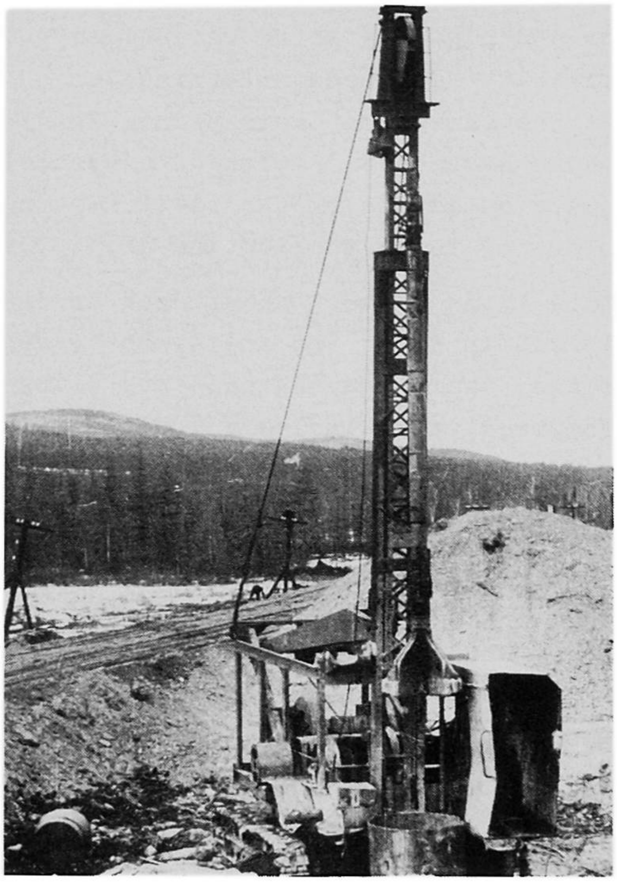
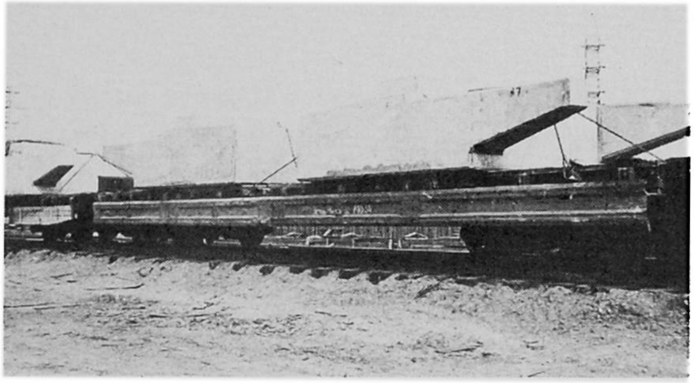


Fig. 5. Holes boring by percussion-churn drill



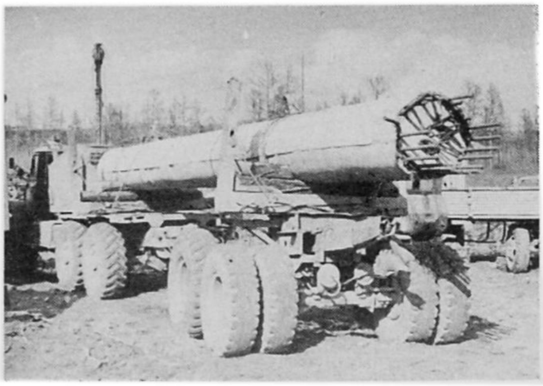


Fig. 7. a) Prefabricated pier members delivery by railway
b) Reinforced concrete columns delivery by road transport

A number of investigations have been conducted to increase bridge efficiency and high degree of reliability. Thus, the conditions for train springing rigidity have been determined.
At the same time basic principles for the design of bridge aseismic elements have been developed. A special construction of bearings and earthquake resistant bridge piers have been developed for more typical conditions of their application (Fig. 9).

More than 2000 short span bridges with columnar piers have been built in BAM in different soils. Thus, grounds were conserved in their frozen state, their bearing capacity being high in columns base carrying large compressive train loads (up to 3.5 MN) and pull out loads under the effect of frost heave forces (up to 1.5 MN) (Fig. 10).

Columnar piers are more reliable compared to traditional ones.
There have been not a single case of frost heave or such piers settlement in a great number of highway and BAM railway bridges, while at the same period inadmissible settlements and distortion of traditional piers with ordinary foundations have been observed in some bridges.

At the present time in connection with the Western Siberian fuel-power complex development highway and railway bridges are being extensively built in Western Siberia. Here permafrost extends from the Arctic circle to the North and is regarded as high-temperature. The climate is featured by long and cold winter (the temperature falls up to $-60^{o}C$) and short and cold summer. Amounts of snow in the region are quite small. But permanent strong winds cause snow-drift accumulation in low-lands and near different obstructions such as road embankments and engineering structures. Such snow-drifts warm the area they cover and contribute to permafrost temperature rise and even to its full melting. This factor has been taken into account in the pier foundation design. The piers for these regions have been designed with due regard to the experience gained during BAM construction.

3.SUPERSTRUCTURES

Precast concrete has become the main building material for superstructures of BAM bridges with spans length up to 27 m. Superstructures without prestressing and meeting the requirements of standard design corrected for low temperature regions have been used for spans of up to 15 m length. Prestressed, superstructures 18.7;23.6 and 27.6 m long specially designed for the North conditions have been used for spans exceeding 15 m. The above superstructures consisting of twin girders are designed for seismic regions with earthquake magnitude up to 9.

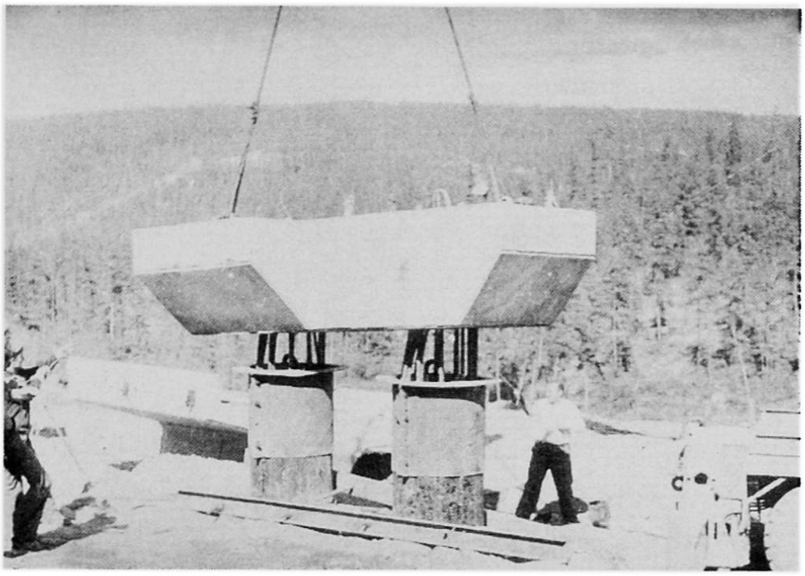


Fig. 8. Columns and caps casting

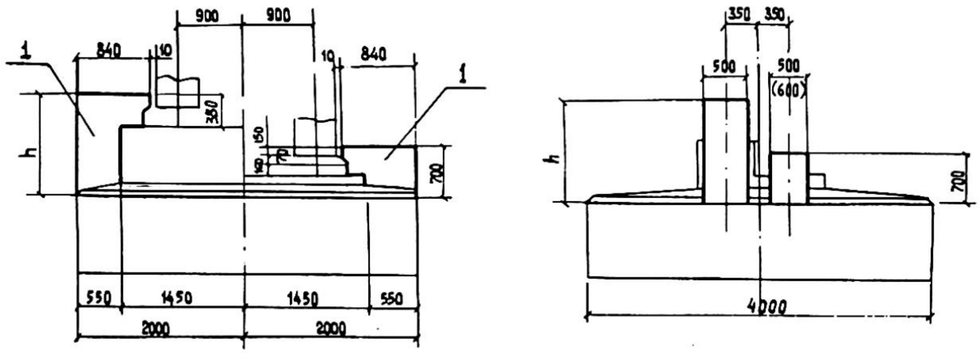
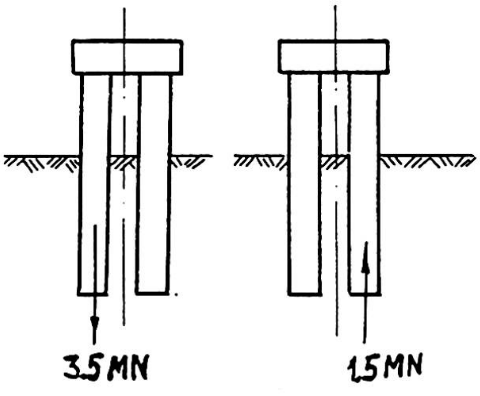


Fig. 9. Aseismic elements. 1 – stoppers



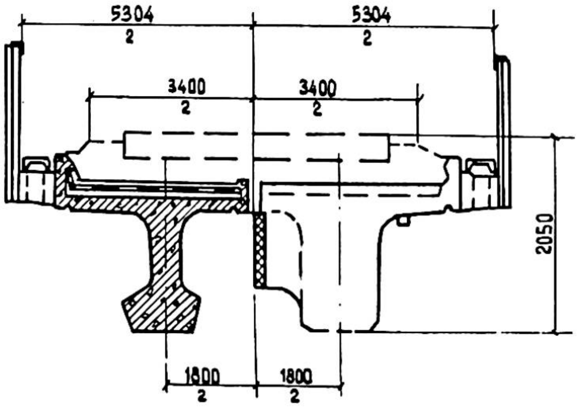


Fig.11. Railway bridge superstructures designed for the North conditions

Fig.10. Pier loading scheme

Design temperature	– below –40°
Magnitude of earthquakes	– M – up to 9
Concrete class	– 400
Frost resistance	– 300

They carry one railway track on straight and curved sections, radius of curvature being 300 m and more. The width of ballast bed is 3.4 m (Fig. 11).

The superstructures considered are practically similar to those used in usual service conditions by their formwork dimensions, main prestressing reinforcement, method for prestressing. The difference is in the use of prestressed stirrups, made of high strength bar reinforcement, at the end girder sections, the use of reinforcing steel of special class and the application of special waterproofing materials and concrete of higher quality (Fig. 12).

Improved crack resistance of superstructures and their service reliability under extreme conditions have been achieved due to more precise analysis methods, use of cold-resistant materials and extra requirements for the technology of structures fabrication, their transportation and erection.

Steel consumption for prestressed concrete superstructures with 23.6 and 27.6 m spans designed for BAM conditions is 2.8-3 times less compared with steel superstructures, the cost of their plant fabrication being approximately the same. Their advantage in comparison with composite superstructures is in complete prefabrication, practically eliminating concrete "wet" works on construction site.

Riveted steel superstructures made up of hot rolled carbon steel have proved to be the best possible solution for the most severe conditions of old Trans-Siberian railway construction.

Refusal from rivetting and industry reorganization for manufacture of welded superstructures for the North conditions required a great deal of research and design work. The activity of researchers and designers has led to the following main results.

Only low alloy heat treated steels with yield point of 350-400 MPa are used as building material for welded superstructures.

All shop connections in steel bridges are being carried out by automatic submerged arc welding, while field connections, as a rule, - by high strength bolts (Fig. 13). However, there are examples of urban bridges built in Siberia (steel and composite box girders among them) where all-welded and composite bolted-welded joints have been used. In the latter case beam flanges are welded and webs are connected by plates on high strength bolts.

The main improved feature of railway truss bridge superstructures designed for the North conditions is elimination of breaks in deck and its composite action with main trusses.

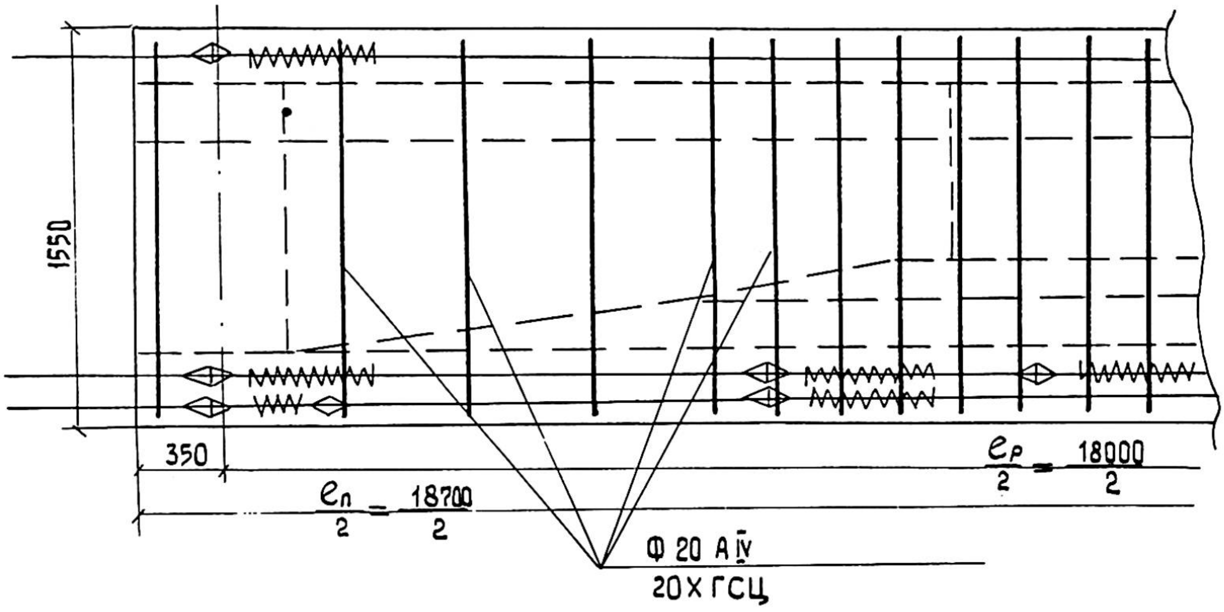


Fig.12. Reinforcement of superstructure

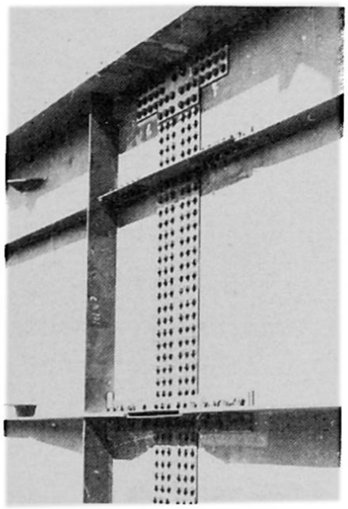


Fig.13. Superstructure field joint by means of high strength bolts

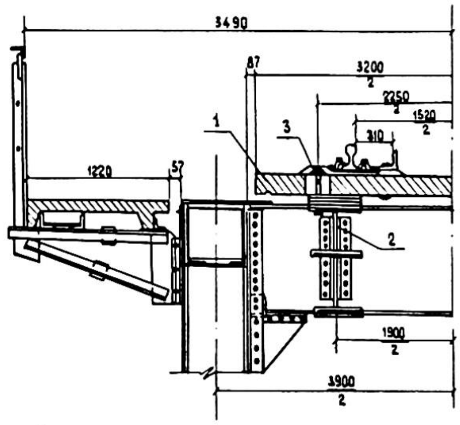


Fig.14. Deck elements connection by high strength bolts

1 – deck slab
2 – stringer
3 – high strength bolt

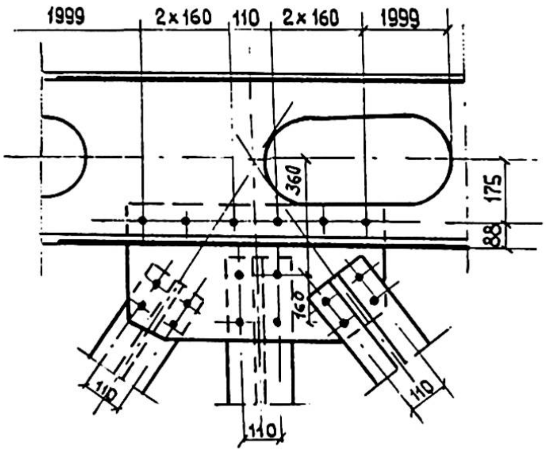


Fig.15. Bracings connection by high strength bolts

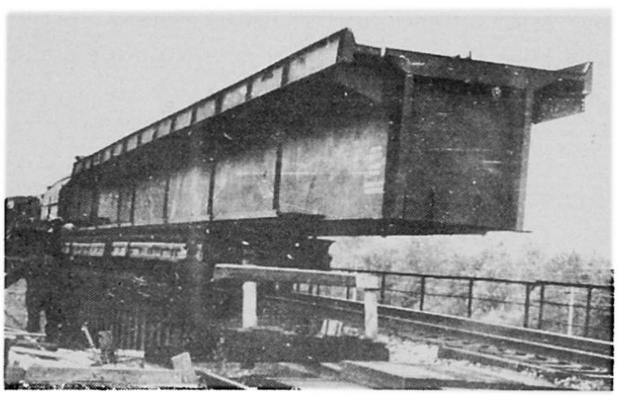


Fig.16. Box girder superstructure coated with stainless steel protecting ballast bed from corrosion

For this purpose the procedures for three-dimensional analysis of these structures have been developed (Fig. 14).

Special precautions against stress concentrators in superstructures designed for the North conditions have been made. As a result fastening of connecting joint plates and elements of main trusses and deck bracing was carried out by means of high strength bolts (Fig. 15).

The analysis methods have been refined to ensure the equal strength of constructive elements. Thus, bending moments of stiffening joints have been taken into consideration in three-dimensional analysis. Endurance of all the elements, joints and details have been thoroughly checked.

Plate girder superstructures with ballasted floor have been considerably improved: integration of precast concrete slab with steel beams by means of high strength bolts and slab units glued connection have been introduced in composite superstructures; to avoid water-proofing repair in steel box girder superstructures the interior surface of ballast pocket was protected from corrosion by the stainless steel layer applied either when rolling or by metallizing methods (Fig. 16).

All these measures have made it possible to achieve high reliability of bolted-welded connections of superstructures for BAM and all-welded superstructures for urban bridges in Siberia.

4. EXAMPLES OF BRIDGES CONSTRUCTION

Two track bridge may serve as a typical example of railway bridges designed for the North conditions (Fig. 17). This deck bridge with superstructure made up of steel 15 XCHD has the scheme 4 x 66 m. The deck and main trusses are of composite action when ballastless tied track is used. Truss superstructures are composed of welded members field connected with high strength bolts.

Superstructures erection by cantilever method with the application of connecting elements was adopted for the most railway bridges similar by construction. Aseismic protection consists of shock absorbing springs in the superstructures upper bearings, superstructures hinged joints and edge stoppers from the both sides of the superstructures. Hinged joints prevent the latter from failure in the case of superstructure shift relative to piers (Fig. 18).

Side stoppers are designed for prevention of superstructure side displacement and tilting.

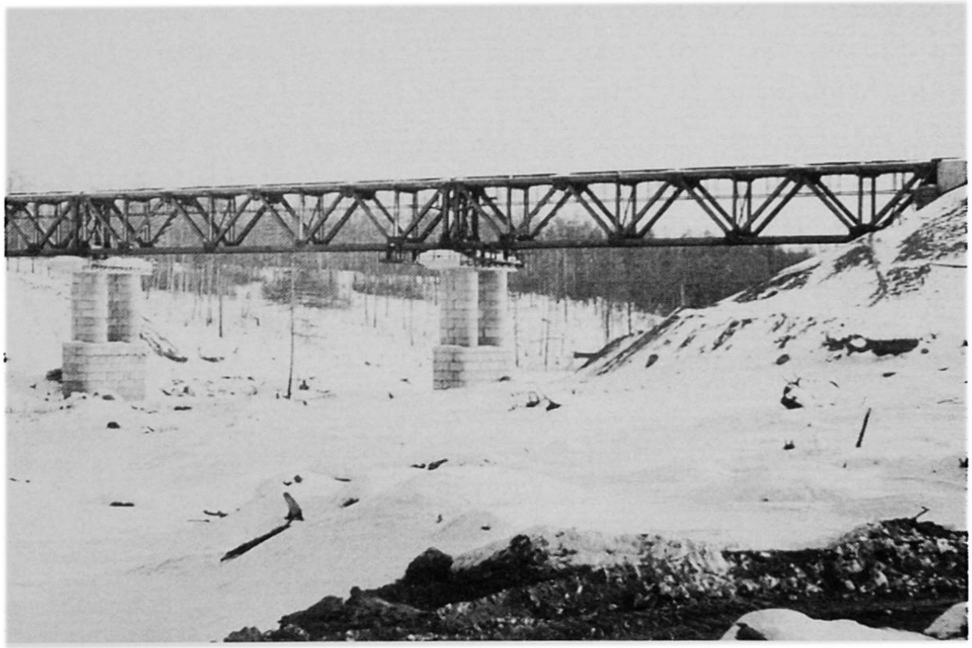
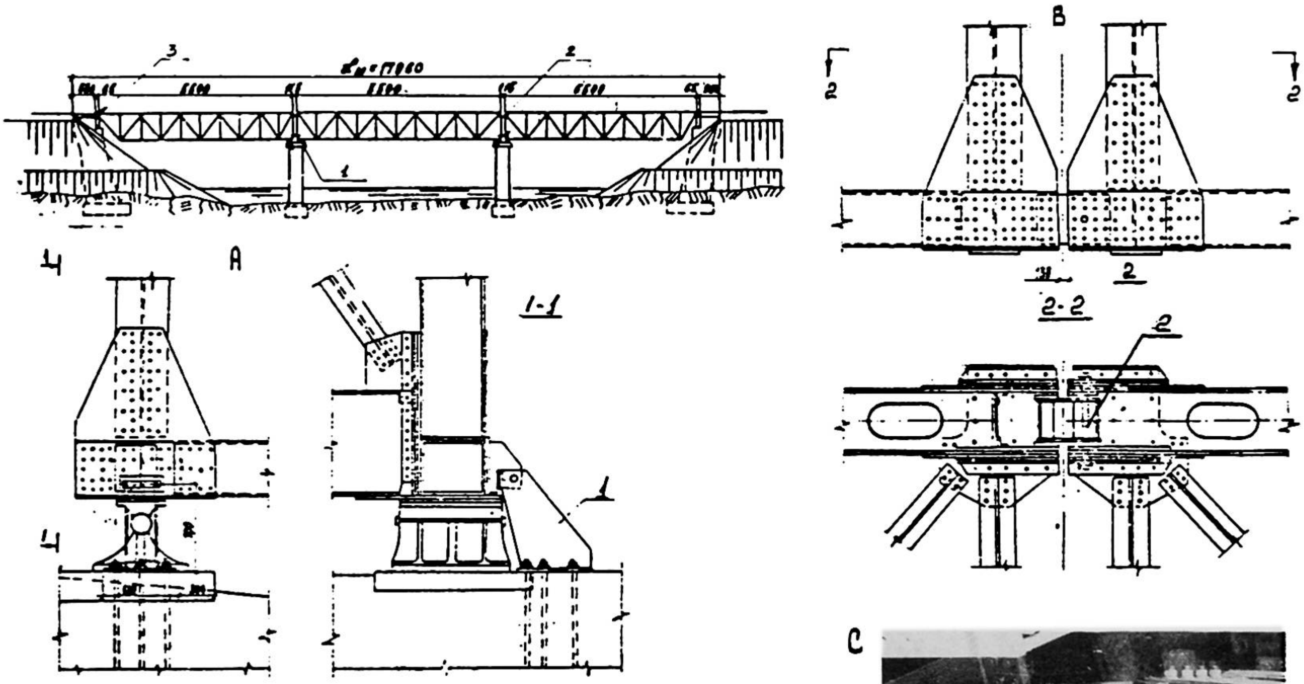


Fig.17. Truss deck bridge



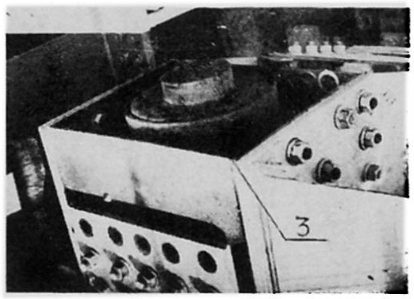


Fig. 18. Bridge aseismic devices

1 – truss bearings displacement stoppers
2 – bonding devices
3 – damping devices

A – Construction of bearing displacement stoppers

B – Bonding device

C – Damping device erection

Two-lane bridge of the scheme 12+2x87+2x159+4x87+12 and 9.0 m roadway, built over the river with severe ice phenomena, may serve as an example of highway bridges for the North regions (Fig. 19).

The deep channel section is bridges by continuous truss superstructure of 2 x 159 m. The prefabrication technology of the structure stipulated the use of conductor devices for railway superstructures.

The 159 m length of navigation spans was dictated by skew river crossing. Innavigable river section with 87 m long spans is bridged by continuous girder with solid web and orthotropic deck slab.

Steel 15 XCHD was used as building material for all spans. The yield point of the above steel is 350 MPa. Steel 10 XCHD was used for the heaviest members of 2 x 159 m continuous superstructures.

The erection of continuous superstructures was carried out by cantilever method. Bridge piers were placed on deep foundations made up of reinforced concrete 1.6 m diameter shells, except for superstructure central pier with foundation composed of inclined steel shells accounting for large channel depth. The depth of shells for the central pier is up to 50 m from low water level.

Some data on two urban bridges construction under North conditions are presented below.

The bridge design of one of them incorporates six-lane roadway of 22.5 m width and two pedestrian walkways of 2.25 m width. The bridge is built in seismic region with earthquake magnitude M=8. The river span is bridged by three-span (100.5+146.0+100.5 m) continuous steel box girder superstructure with orthotropic deck slab (Fig. 20).

The superstructure is of all welded construction. Its cross section consists of two boxes with the upper orthotropic deck slab and bottom ribbed slab made up of steel 10 XCHD hardenable by heat treatment, its yield point being 400 MPa (Fig. 21).

Steel superstructure was erected by cantilever method with the application of compound joints - high strength bolts were used for the web and butt welding for the flanges.

Deck pavement of steel superstructure consists of asphalt concrete layer of 7 cm thickness placed over epoxy-bitumen insulation layer. Sticking insulation with the application of bitumen was used for reinforced concrete approach spans.

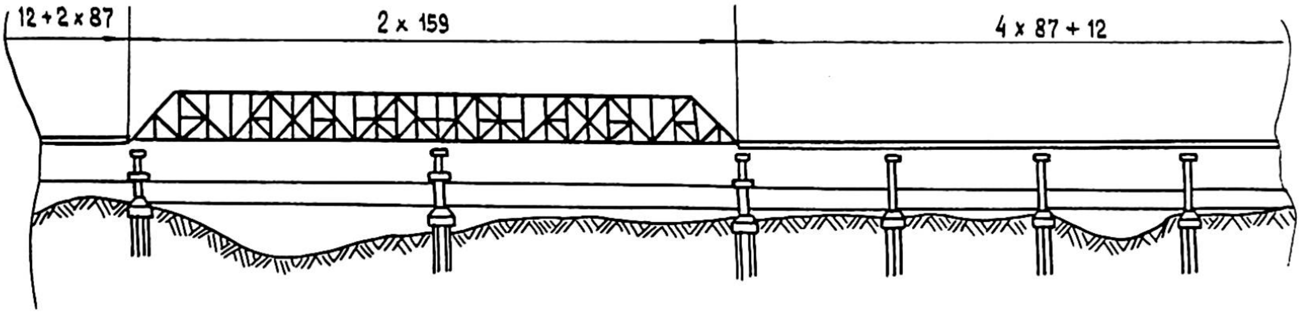


Fig.19. Highway bridge over the river with severe ice phenomena

Fig.22. Continuous composite superstructure

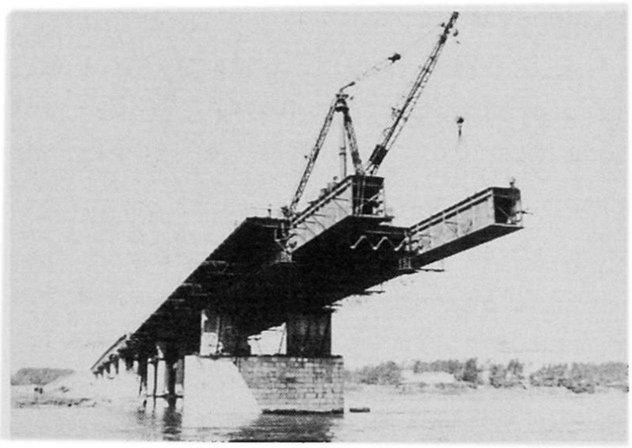


Fig.20. Continuous box girder superstructure with orthotropic deck slab

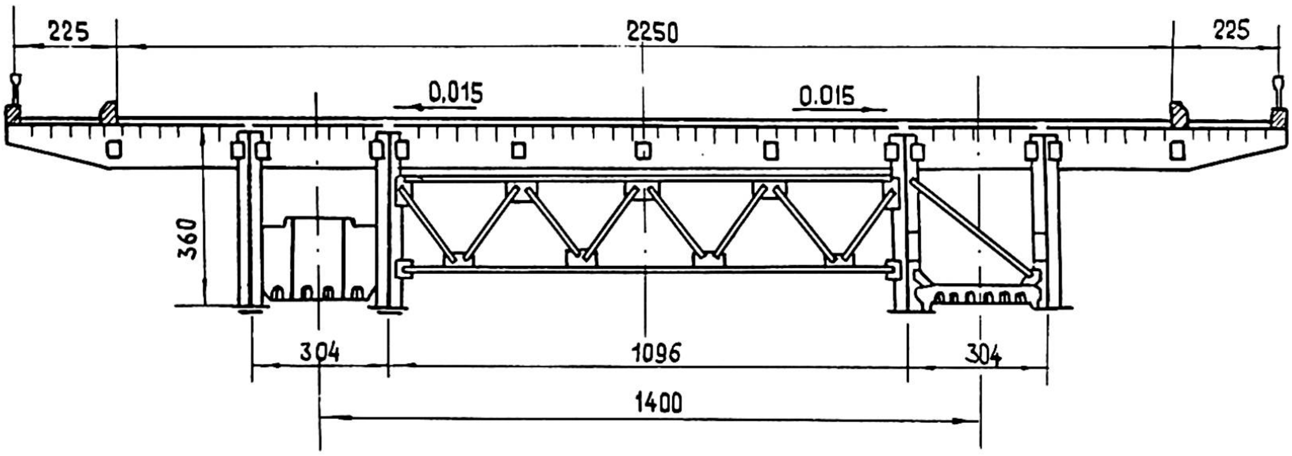


Fig.21. Box girder superstructure

The design of the second urban bridge incorporates six-lane roadway of 25.5 m overall width. Composite continuous superstructure consists of 6-span box girder with the longest span of 126 m (Fig. 22), made up of steel 10 XCHD, hardenable by heat treatment. Its yield point is 400 MPa.

Reinforced concrete slabs are of composite action with the main 3.3 m deep girders forming two boxes 7.0 m wide each (Fig. 23). Support sedtions of reinforced concrete deck are prestressed by wire strands, consisting of 48 wires of 5 mm diameter, with tensile strength of 1700 MPa.

Erection joints of main girders units of the length up to 29.8 m are of twc types:
a) compound ones, when webs are joined by high strength bolts and flanges - by butt welding;
b) all welded joints.

Waterproofing of the deck is made up of butil rubber, pavement consists of 7 cm thick asphalt concrete layer. Bridge piers rest on foundations composed of 1.6 m diameter reinforced concrete shells (Fig. 24).

5. BRIDGE APPROACHES

The junction point of abutment piers and approach embankments is considered to be important component of bridge crossings. Here, the proper conditions for smooth car traffic should be ensured. In this connection approach embankments should be designed accoding to the principle of permafrost level rise up to embankment foot as minimum (Fig. 25).

Such solution seems to be accessible when crushed stone - sandy soils are used for embankment bottom. Existing moss vegetative cover should be preserved in this case accounting for its action as natural heat insulating layer.

In the events when embankment height may be lowered accoding to the requirements for longitudinal profile designing, such lowering should be substantiated by thermal-technical analysis to prevent permafrost horizon from thawing and depression.

Placement of thermal insulating layer of either moss or peat, or some other material such as foam plastic, expanded polystyrene, etc. into embankment foot is considered to be additional measure preventing permafrost upper layer from melting. Moreover, the use of materials mentioned is more preferable as they make possible prevention of unfavourable effect of moss removal or peat mining on environment (vegetative cover).

The above measures being taken, pavement ultimate settlement not exceeding

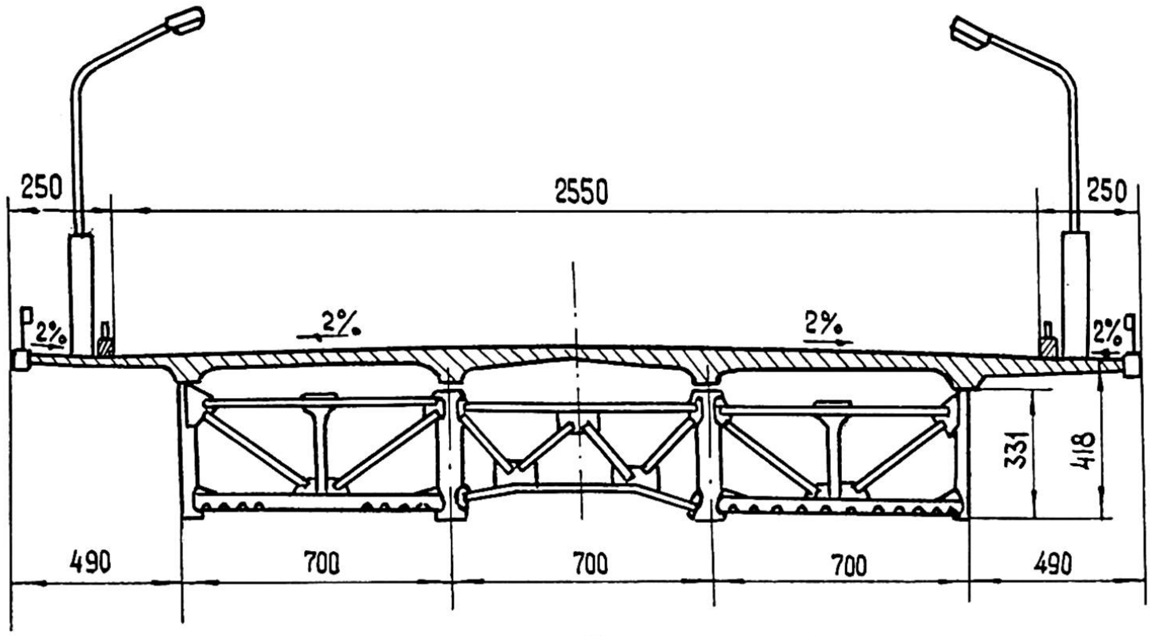


Fig.23. Box girder composite superstructure

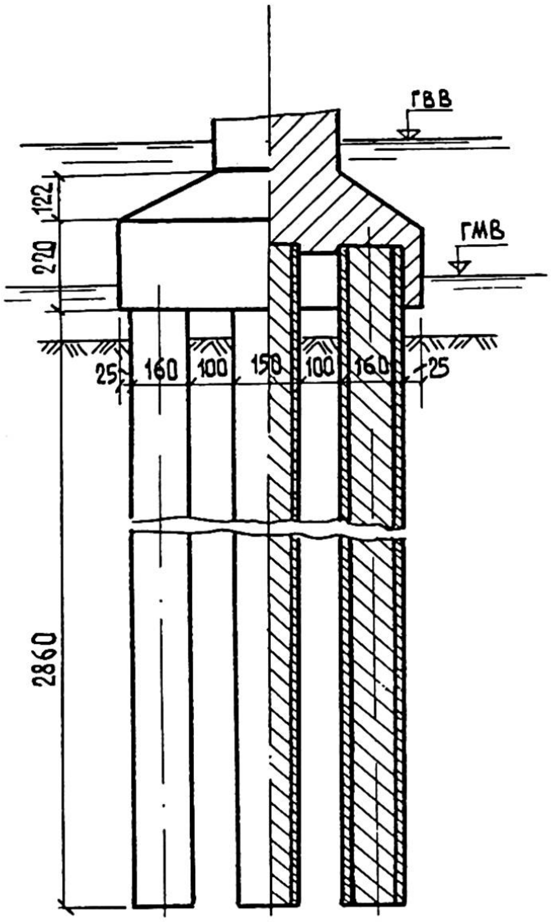


Fig.24. Bridge pier foundation made up of 1,6 m diameter shelled pile

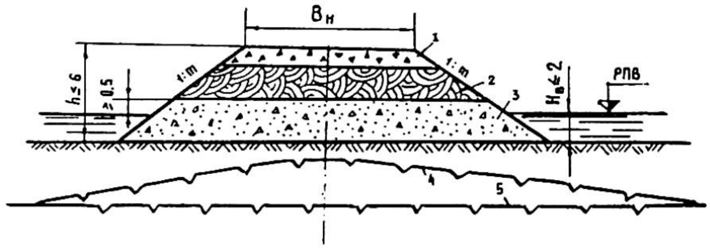


Fig.25. Cross section of approach embankment

1 – crushed stone or gravel
2 – clay soil
3 – noncemented fragmental soil
4 – permafrost upper horizon after
5 – permafrost upper horizon before embankment erection
РПВ – surface water design level

40 mm - for asphalt concrete covering and 20 mm - for cement covering would be ensured.

Pavement consisting of such materials as gravel, crushed stone,etc., settlements of up to 100 mm are admissible.

Careful check should be carried out for determination and design of embankment height to prevent its surface from snow accumulation. The check may be, for instance, made by the following formula:

$$H_{min} = K \cdot h_{sn} + K_1 \cdot \Delta h$$

where K - maximum snow depth local correction, h - maximum snow depth along the route to be designed, K_1 - local relief coefficient, Δh - minimum embankment elevation over snow cover of plain terrain.

Thus, great experience has been gained by the Soviet specialists through engineering design and construction of highway and railway bridges under severe climatic conditions in the North regions of the USSR. This experience has made it possible to develop more efficient bridge structures and industrial technology for their construction.

Requirements for steels used for bridges welded superstructures in different climatic regions according to the Codes of Practice of the USSR

Steel Grade	Thickness,mm	Mechanical properties of all grades steel in tension			Impact strength(J/cm^2) of below indicated steel grades at the following temperatures (°C)					Bending test
		Temporary tensile strength, MPa	Yield point, MPa	Relative elongation,%	1 -40	2 -60	3 -70	1 and 2 +20 after strain ageing	3 -20 after strain ageing	in cold state up to sides parallelism for all grades steel
			not less than							
15XCHD	8-32	500-700	350	21	30	30	30	30	30	d = 2a
	33-60	480-680	340	21	-	30	30	30	30	d = 2a
10XCHD	8-15	540-700	400	19	40	30	30	30	30	d = 2a
	16-32	540-680	400	19	-	30	30	30	30	d = 2a
	33-40	520-660	400	19	-	30	30	30	30	d = 2a

Requirements for concrete to be used in bridge structures, designed for the North conditions, according to the Codes of Practice of the USSR

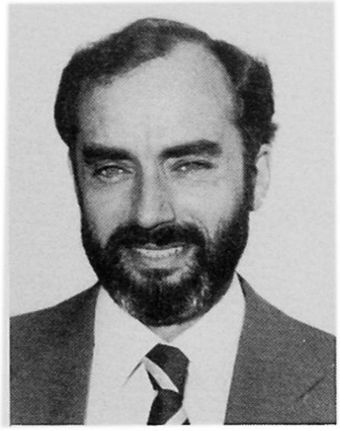
Material	Material properties	Requirements for properties of materials to be used in structures	
		reinforced concrete	concrete
Concrete	Strength quality	200-300 foundation,unprestressed piles 300-400 prestressed structures, piers in the zone of variable water level 400 superstructures of long span bridges, shells	150
	Frost resistance quality	300	300-400 for structures in the zone of variable water level and protective layer 200-in the other cases
	Use of chemical additives	Use is obligatory at Frost resist. 300	

Rebuilding of Australia's Three Antarctic Stations

Reconstruction des trois bases australiennes en Antarctique

Wiederaufbau der drei australischen Forschungsstationen in der Antarktis

Robert A. McEWAN
Structural Engineer
Comm. Dep. of Housing and Constr.
Melbourne, Australia



Robert McEwan, born 1946, graduated as Bachelor Civil Engineering (1969) and Master Engineering Science (1971) at Melbourne University. For the next 10 years he was involved in the design of a wide range of structures. Since 1980 he has been Structural Engineer in Charge, Antarctic Section.

SUMMARY
Antarctica is the coldest, windiest, driest and most remote continent on earth and Australia is at present completely rebuilding its three permanent stations. This paper discusses the Rebuilding Program with particular emphasis on the development of the building system, structural design philosophy and solutions adopted for footings, framing systems, cladding systems, and associated testing and development work.

RESUME
L'Antarctique est le continent le plus froid, le plus sec, le plus isolé et le plus balayé par les vents. L'Australie a entrepris de reconstruire entièrement ses trois bases permanentes sur ce continent. L'article examine le programme de reconstruction, et met particulièrement l'accent sur les méthodes de construction, les motivations qui ont présidé à la conception des nouvelles structures, ainsi que les solutions adoptées pour les fondations, les systèmes de coffrage et de revêtement, les contrôles de qualité et la mise au point des techniques.

ZUSAMMENFASSUNG
Die Antarktis ist der kälteste, windigste, trockenste und abgelegenste Ort auf der Erde. Zur Zeit werden die drei festen australischen Forschungsstationen neu aufgebaut. Der Beitrag beschreibt das Bauprogramm und geht insbesondere auf die Entwicklung des Bausystems und die Konstruktionsprinzipien ein. Das Fundationssystem, die Ausbildung der Tragstruktur, die Verkleidung und die damit zusammenhängenden Qualitätsprüfungen und Entwicklungsarbeiten werden beschrieben.

1. ANTARCTICA - BACKGROUND

Antarctica is the highest, coldest, stormiest and driest continent on earth. It is an immense ice dome of more than 13 million square kilometres overlying a rock mass much of which is depressed below sea level. At its highest point the ice is over 4 kilometres above sea level. While ice predominates, small areas of bare rock occur. In mid winter the continent experiences continuous darkness whilst in mid summer it experiences continuous daylight. Australia has been active in research and exploration in Antarctica for the past 80 years. In 1936 Australia became responsible for the Australian Antarctic Territory - almost half the total area of the continent.

2. AUSTRALIA'S ANTARCTIC STATIONS

The three permanent Australian Stations on the Antarctic continent at Casey, Davis and Mawson are located on coastal ice free rocky outcrops. The ambient temperature at each station ranges from $-40^{o}C$ to $+5^{o}C$, with air of a very low specific humidity. The sites are subjected to winds of up to 280 kph, which carry fine dry drift snow and occasionally, wet snow and sleet in summer. For approximately 9 months of the year, wind blown drift snow covers each site. During summer this snow usually melts leaving all buildings free of snow. Access to these stations is by ship for 3 months during summer when the sea ice temporarily breaks up.

Mawson station is located on solid granite rock. Davis station is located on moderately weathered metamorphic rock (mainly gneiss), crossed with numerous basic dykes (mainly dolerite) and covered in many places with moraine deposits ranging in size from fine silty sands to large boulders. Casey station is located on slightly weathered rock (mainly gneiss) and moraine deposits.

3. BUILDING SYSTEM

3.1 Historical Development

Building design philosphy for Australia's Antarctic stations has been continually developed since the establishment of Mawson Station in 1954. Buildings erected before 1976 at Mawson and Davis were relatively small and comprised post tensioned, load bearing insulated panels with external guys; they were located directly on ground and were designed for rapid erection during the short summer season. However, the designs suffered shortcomings including inadequate vapour barrier provision, inability to replace damaged external panels, noise and vibration problems, access difficulty for snow clearing equipment due to the guys, and the congested sites increasing the problems of snow drifting. At Casey, a similar building design philosophy was adopted, although elevated above ground to preclude snow drift accumulation, deterioration was accelerated by exposure to salt spray. By mid 1970's, buildings and services at all stations had deteriorated to an unacceptable standard and new building systems were investigated.

3.2 New Building System

The main features of the new building system are:

- larger and more efficient buildings;

- braced steel framed structures on concrete footings anchored to the ground, without need for external guying;
- external insulated sandwich panels which are durable and readily removed for maintenance;
- provision of vapour barriers on the inside and moisture barriers on the outside of buildings, and prevention of 'cold paths';
- maximum use of standard and prefabricated components, within the constraints of the available shipping and material handling facilities;
- simplified construction details to minimise the extent of site labour during the brief summer outside construction period (3 to 4 months);
- trial erection of building structures in mainland Australia to minimise the risk of delays to work on site;
- careful location of buildings minimising the drift problem. All buildings (rectangular in plan) are oriented with their long sides parallel to the prevailing (strong) winds, which carry the drift. Doors, windows and openings are only permitted on these sides, and drift snow generally accumulates either end of the building and tends to blow clear on the sides;
- fire prevention, detection and control.

3.3 Rebuilding Program

In 1981 the Australian Government gave approval to redevelop the three stations at a cost of A$58m (1980), with completion planned for 1990. The Department of Housing and Construction assumed full responsibility for program management, design, fabrication, construction and commissioning of all facilities.

Separate buildings are being provided at each Station for Operations, Living, Science, Services, Power Houses, Workshops, Communications, etc. Other facilities include roads, bulk fuel depots, cargo handling facilities, and helipads. Feasibility studies are continuing on the construction of airstrips.

All buildings are linked by external service reticulation comprising separate, insulated and heat-traced pipes and electrical cable trays supported above ground on structural scaffold tube supports and anchors. These service lines are crossed by road and pedestrian bridges. Service lines over ice are supported on timber piles.

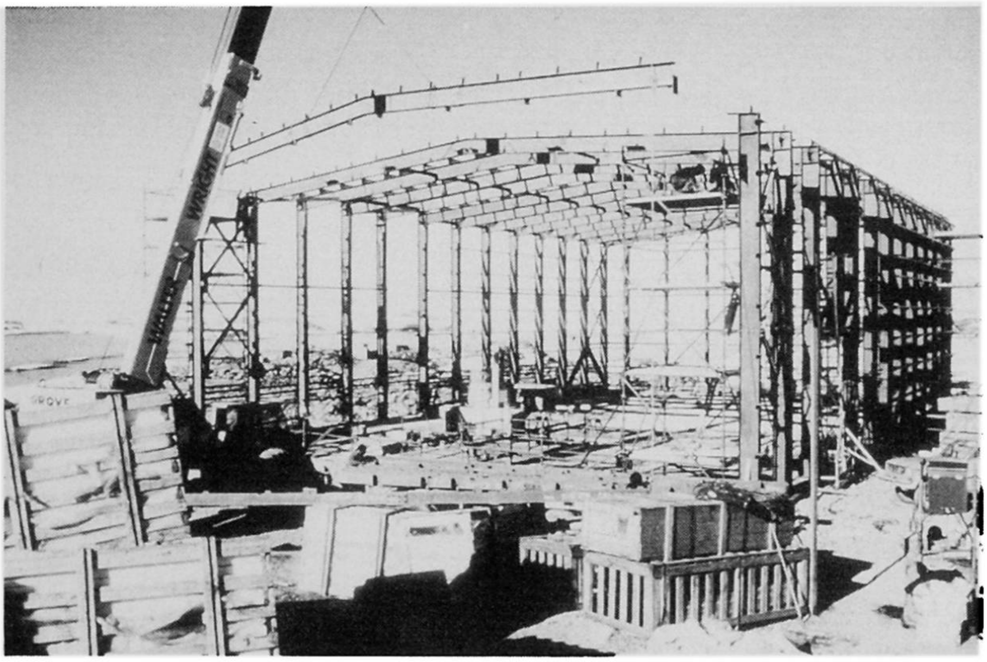


Fig. 1. Casey store under construction

4. WIND LOAD DESIGN PHILOSPHY

4.1 Wind Loads

Wind loads are the dominant design loads on structures at Australia's three Stations and detailed wind load design guidelines have been prepared. Wind loadings are based on a 2-3 sec. gust using the 'quasi-static' approach in Australian Standard AS 1170 Part 2, and using a statistical analysis of daily maximum extreme values recorded at each Station.

The strong wind (prevailing wind) at each Station is unidirectional and is taken to act within an arc of 45^{o}. The weak wind (of lower magnitude) acts outside this arc. The 25 year return period local wind velocity at 10.0m height for Terrain Category 2 at each Station is:

- strong wind 72 m/sec.
- weak wind 48 m/sec.

The design wind velocity is adjusted for local terrain and height of structure.

4.2 Limit State Design

The Limit States used for all Antarctic structures are:

- Ultimate Limit State: the wind velocity used for strength requirements has a 1.5% probability of occurrence during the 25 year life of the structure.
- Serviceability Limit State: the 25 year return period wind velocity is used for serviceability requirements.

4.3 Wind Pressures

Since internal wind pressures are dependent on wind direction and building penetrations, the following design policy on penetrations was formulated after extensive testing:

- doors, windows and openings are not permitted on the windward wall for the strong wind;
- doors may be opened at any time, for both Limit States;
- windows may be fractured by flying debris for Ultimate Limit State only;
- cladding remains intact for both Limit States.

A study of the latest experimental data resulted in the adoption of higher values of both average and local external pressures on walls and roofs, than those recommended in AS 1170 Part 2 1981.

5. FOUNDATIONS

5.1 Site Investigation

A continuing Geotechnical Investigation Program is examining the following critical aspects:

- Ground Conditions: differ markedly from Station to Station (Section 2). Frozen ground has three important properties:
  - impermeability: resulting in poor drainage;
  - ground ice content: bonding the soil particles together, which when melted can cause soft areas (Casey and Davis);

thermal sensitivity: alteration to existing state of thermal equilibrium can cause settlement of structures, although this problem is minimal for moraine material.

Ground Temperatures: have been monitored to a depth of 10.0m. Frozen ground generally consists of two layers: the upper (or active) layer, and the lower (or permafrost) layer, which is the perenially frozen layer. The greatest fluctuation of temperatures occurs in the active layer and is potentially the principal cause of damage to structures. Below 10.0m the temperature remains fairly constant at -9^{o}C whilst above this depth the temperatures are influenced by the seasonal variation of air temperature, the maximum depth of thaw being 1.0m approximately.

- Ground Compaction: tests on moraine and crushed rock.
- Ground Level Survey.
- Water Quality: studies performed at Casey and Davis.

5.2 Substructure Types

There are two main types of substructure used:

- Slabs: for heated buildings the concrete ground floor (Fig. 2) is cast directly onto 150mm thick polystyrene insulation with penetrations in the insulation to permit support of the slab directly on the ground (usually via rubber bearing pads); whilst for unheated buildings the insulation is deleted.
- Pedestal/Shear Wall: these heated buildings have suspended ground floors supported off pedestals (braced by shear walls), to cater for extensive subfloor services and/or steeply sloping sites.

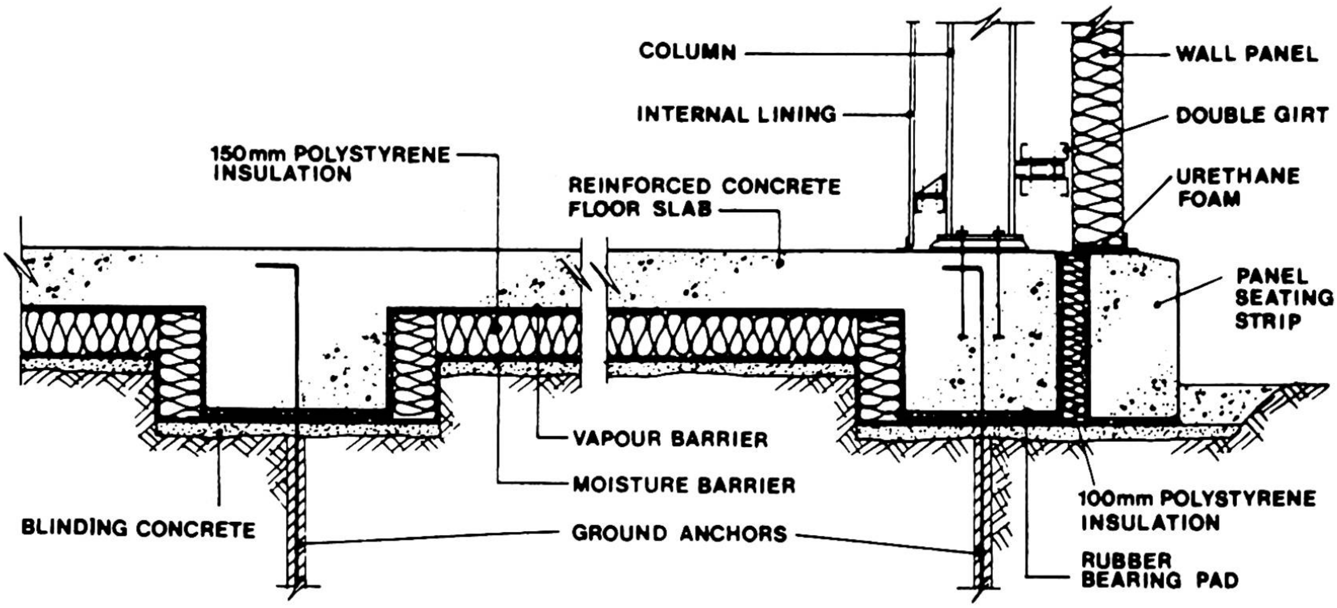


Fig. 2. Typical substructure for heated building

5.3 Ground Anchors

In view of the extreme wind velocities at each Station a system of ground anchors has been developed to reduce the mass of concrete footings. Extensive laboratory and site tests proved epoxy, sulphur and a range of mechanically expanding and split-set anchors performed well in sound rock, whilst a frozen cement grout system performed well at all Stations. However in 1982 a magnesium phosphate chemical grout became available. This grout is high strength, rapid setting and self curing at temperatures down to -10^{o}C, and gave excellent results in tests at all Stations. The ground anchor system adopted is:

- Y20 reinforcing bar - grouted length 3.0m in 64mm dia. hole;
- magnesium phosphate grout mixed with 10% water;
- proof testing of a percentage of all anchors on each structure;
- the min. ultimate pullout loads are 80kN (Casey and Davis) and 130kN (Mawson). Limit State load combinations have been developed for both strength and stability.

6. FRAMING SYSTEM

6.1 General

The structural system consists of a structural steel frame which is fully braced at roof level and on all four walls to satisfy strength and serviceability requirements. The modular frame spacing is based on the cladding panel module of 1230mm (1200mm wide panel and 30mm gap). Cost comparisons of alternative structural systems (including secondary steelwork) resulted in the adoption of the following frame spacings:

- small buildings 2460mm spacing
- medium buildings 3690mm spacing
- large store-type buildings 4920mm spacing

6.2 Details

The steelwork for buildings is completely trial erected in mainland Australia to minimise site problems. Steelwork is shop welded (in Australia) and field bolted (in Antarctica). All electrodes used are of a low hydrogen type and strict requirements are laid down for welding procedures.

Hot rolled steelwork complies with AS 1204 Grade 250 (min. yield 250 MPa) and is painted with self-curing inorganic zinc silicate, except for exposed steelwork which is hot dipped galvanised. Purlin and girt sections are cold rolled zinc coated steel conforming to AS 1397-G450-Z300 (min. yield 450 MPa).

Panel bolts, purlin/girt bolts and holding down bolts are commercial grade 4.6 to AS. 1111 (min. tensile strength 400 MPa). All other structural bolts are high strength grade 8.8 to AS. 1252 (min. tensile strength 800 MPa). All bolts are hot dip galvanised, except for panel bolts which are electroplated zinc.

7. CLADDING SYSTEM

The cladding system consists of sandwich panels which are fixed to a supporting framework of double "C" section purlins and girts (Fig. 3). The cladding panels, produced on a continuous laminating machine, are 1200mm wide and 150mm thick, and consist of polystyrene foam sandwiched between two 0.6mm thick precoated "Zincalume" steel sheets. The rigid cellular polystyrene is Class M, in accordance with AS 1366, Part 3. The purlins and girts consist of two cold formed steel "C" sections back to back and separated by 50mm wide rectangular hollow sections each containing a captive nut for the panel fixing bolt.

Extensive testing has been completed on the individual components and full scale tests on the complete roof and wall system (including windows and doors), using a suction box. These tests showed that:

- correct oven drying of polystyrene foam is critical;
- correct glueing of polystyrene foam butt joints and foam/steel skin joints is also critical;
- panels with unbonded butt joints suffer premature shear failure at these joints;
- dynamic loading effects are insignificant;
- panels have good impact properties;
- strict quality control testing is essential during panel manufacture since their strength is very dependent on the standard of workmanship.

Delamination of some older panels has been a problem and thermal cycling tests are planned. Tests have been performed on the double purlin/girt system to confirm the joining requirements of the composite system.

The main design considerations for the cladding system are:

- bending and shear of the polystyrene cladding panels;
- pullout of the panel fixing bolts and plates through the panel;
- lateral buckling of the double purlins/girts.

In zones of high external wind pressure special attention is paid to purlin/girt spacing (550mm max.), elsewhere across the building the maximum purlin/girt spacing is 1100mm. The roofs are kept snow free by the continual winds and minimal roof projections. The dead load of wall panels is supported by the concrete panel seating strip, and not by the girts.

Windows are triple glazed factory sealed units and are generally fitted into panels during prefabrication. The doors have been developed from standard cool room doors with special latches and gaskets.

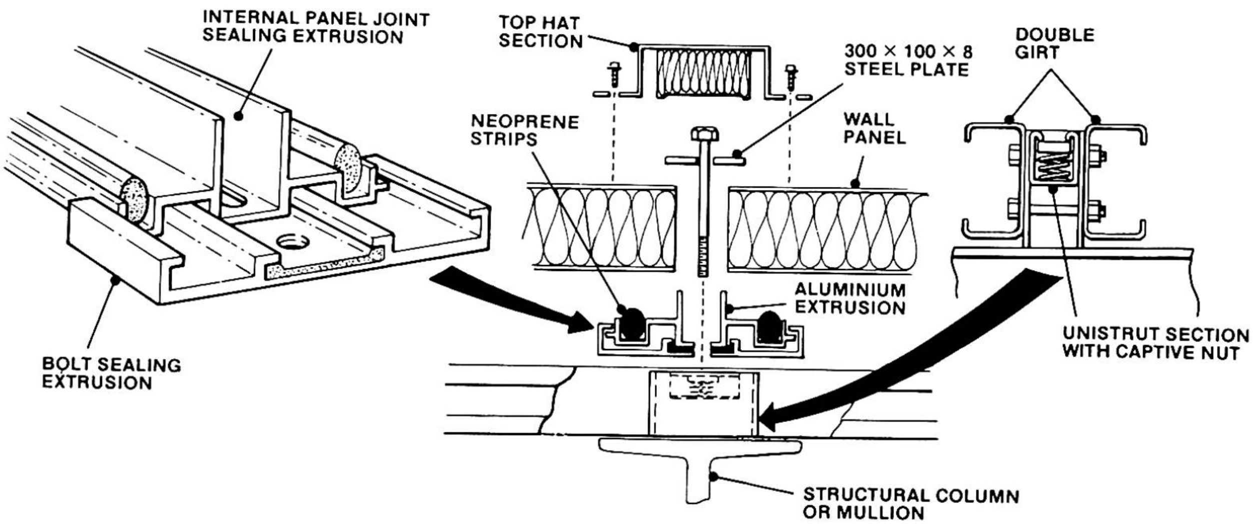


Fig. 3. Typical wall panel fixing system

8. CONCRETE

8.1 General

Successful techniques [1] are being used in Antarctica to ensure the efficient production of sound concrete structures. Concrete has proved eminently suitable for structural footings, heavily loaded floors, suspended floors and machine bases. Difficulties in concreting arise because of the short outdoor construction period available and the harsh environment. Insitu concrete is minimized by the use of pre-cast concrete (floors and bridges) and ground anchors (instead of mass concrete). Detailed cost comparisions of the relative merits of precast and insitu concrete in Antarctica have been undertaken.

Material for in-situ concrete is supplied to Antarctica in pre-mix form in 40kg bags (multiwall paper bags inside heat sealed polythene bags), and delivered in waterproof lined 2 tonne cases. A strict quality control program covers all aspects of material and mix design, manufacture, packaging and crating. The main requirements of the pre-mix concrete specification are:

- min. cement content (Type B - high early strength) is 7.5kg per bag;
- concrete characteristic strength to AS 1480 at 7 days with entrained air content of $5\pm1\%$ is 30Mpa;
- max. moisture content after bagging is 0.25%;
- slump $70\pm$ 10mm.

8.2 Site Concreting

Concrete admixtures (air entraining agent and accelerator) and water are added to the pre-mix on site in pre-determined quantities. Air entraining admixtures improve workability and freeze/thaw resistance of the concrete, whilst accelerating admixtures (chloride free) hasten initial set and strength development. Concrete strengths of 25MPa at 14 days are consistently being obtained.

Detailed instructions are contained in an Erection Manual on all aspects of concreting, particularly subgrade preparation, mixing, placing, compaction, finishing, curing and quality control. Concrete is never placed at ambient temperatures below $-10^{o}C$. The mixing water is heated to $40^{o}C$ approximately, and the concrete mixed in 0.8 cu.m. agitator trucks and placed at 10^{o} to $15^{o}C$. The placed concrete must be kept above $10^{o}C$ for a minimum prehardening period to ensure a strength of 7MPa (min.) to prevent permanent damage from freezing. Concrete curing is continued for a min. of 7 days, to ensure attainment of its design strength, using some of these techniques:

- preventing early stripping of formwork;
- covering with non-porous polythene membrane, insulation blankets and tarpaulins;
- application of hot air (above the membrane) or low pressure steam inside enclosure around members - not generally needed;
- care to prevent thermal shock after curing.

A detailed quality control program is used on site and full details are forwarded to the design office regarding:

- batching;
- site sampling and testing (inc. slump and air entrainment);
- cylinder and cube testing;
- monitoring of concrete temperatures during mixing, placing and curing.

8.3 Formwork

The following types of formwork are used:

- A modular system of steel framed and stiffened plywood panels.
- Pre-insulated sandwich panel of polystyrene foam core and structural plywood. This system is used for pedestals, walls etc., where there exists a high ratio of surface area to volume of concrete.
- Ribbed steel decking for suspended floors in lieu of precast panels.

8.4 Reinforcement

The reinforcement generally used is Tempcore bar, ideally suited to use in Antarctica with its good ductility, toughness and high strength. Tempcore complies with AS 1302 Grade 410Y and has a 410MPa min. yield strength and low carbon equivalent. Precast concrete is reinforced with Tempcore bar or prestressed using low relaxation stress relieved super grade strand to AS 1311.

9. CONCLUSION

The complete rebuilding of Australia's three Antarctic Stations is a challenging task in view of their remote location and the severe environmental and logistical constraints. The success of the Rebuilding Program is dependent on good planning and management, innovative and cost effective design, detailed and accurate documentation, technical standards based on a testing and development program and site feedback, and strict quality control and supervision.

10. ACKNOWLEDGEMENT

The author would like to thank the Department of Housing and Construction for its support and encouragement in the preparation of this paper, and for granting permission for its publication.

11. REFERENCES

1. McEWAN R., GOSBELL K. Concreting Practices at Australian Antarctic Stations. SCAR Antarctic Logistics Simposium Leningrad U.S.S.R., June, 1982.

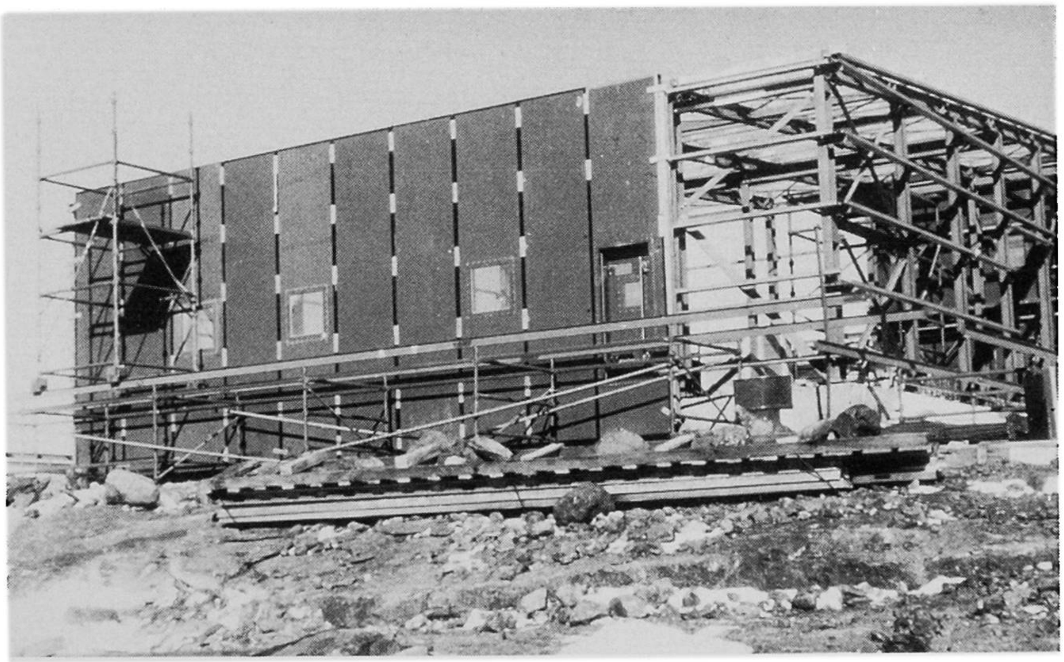


Fig. 4. Mawson Tank House under construction

Leere Seite
Blank page
Page vide

Comportement des sols et des structures en zone arctique

Verhalten von Baugrund und Bauwerken in der arktischen Zone

Behaviour of Soils and Structures in Arctic Regions

Dominique BLANCHARD
Laboratoire Central des
Ponts et Chaussées
Paris, France

Michel FREMOND
Laboratiore Central des
Ponts et Chaussées
Paris, France

Peter J. WILLIAMS
Director
Carleton University
Ottawa, ON, Canada

RESUME
Les effets du gel du sol autour d'un gazoduc de 18 mètres de long ont été observés au cours d'une expérience comportant de nombreuses mesures. Le gonflement, les déplacements du sol et les contraintes dans le tube sont continuellement enregistrés. Les expérimentations sont par ailleurs comparées avec des modèles numériques. L'un d'entre eux est décrit ici. Il tient compte des aspects thermique, hydraulique et mécanique. Il permet en particulier de calculer le gonflement.

ZUSAMMENFASSUNG
Die Auswirkungen der Vereisung des Bodens im Bereich einer 18 Meter langen Gastransportleitung wurden beobachtet und eine Vielzahl von Messungen durchgeführt. Die Hebungen und Verschiebungen des Bodens sowie die Dehnungen im Rohr werden fortwährend registriert. Die Daten werden mit numerischen Modellrechnungen verglichen. Ein Modell wird in diesem Beitrag beschrieben. Es berücksichtigt die thermischen, hydraulischen und mechanischen Aspekte und erlaubt, im besonderen die Hebungen rechnerisch zu erfassen.

SUMMARY
The effects of frozen ground around an 18 m long pipeline have been observed during a test involving numerous measurements. The soil movements and the stresses in the pipe have been continuously monitored. Tests have been compared with numerical models, one of which is described in the article. This model considers thermal hydraulic and mechanical aspects. It allows, in particular, to calculate the ground heave.

1 – INTRODUCTION

Le trait essentiel des problèmes de géotechnique nordique est la présence constante d'eau et de glace dans les sols. Un oléoduc chaud fait fondre la glace présente en quantité largement supérieure au volume normal des vides dans le pergélisol. Des subsidences parfois dommageables pour les structures en résultent. A titre de remède on pourrait envisager d'utiliser un tube froid. En fait la congélation qui en résulterait induirait des problèmes encore plus complexes. De plus le pergélisol est souvent discontinu : des sols gelés alternent sur de courtes distances avec des sols dégelés. Comme il est impossible de maîtriser la température des tubes sur de tels intervalles, la connaissance des effets du froid sur les sols est capitale [14].

Les ingénieurs civils, en particulier les ingénieurs routiers [8], connaissent bien les effets du gel, surtout ceux dus au gonflement des sols et à leur perte de portance au dégel. Tous ces phénomènes sont influencés par la vitesse et l'intensité de la congélation, le type de sol et son humidité et quantité d'autres facteurs.

Les effets dus à l'eau sont prépondérants. Le confinement de l'eau dans les pores, la proximité et la nature des parois des pores provoquent :

(a) un abaissement progressif de la température de congélation de l'eau au fur et à mesure que l'eau liquide se transforme en glace. Les sols gelés contiennent donc de l'eau liquide à des températures inférieures de quelques degrés à 0° C ;

(b) une perméabilité notable de la zone gelée en raison de la présence de cette eau [2] ;

(c) un potentiel (relié à l'énergie libre de Gibbs) pour l'eau du sol gelé plus faible que celui de l'eau du sol non gelé voisin [5], [15] il en résulte,

(d) un important déplacement d'eau vers les parties les plus froides du sol gelé provoquant un gonflement notable du sol [13].

Les ingénieurs civils sont confrontés à cette mouvance du sol. La protection du réseau routier français contre l'action du gel [8], les projets de gazoducs [3], [14] dans le grand nord canadien ont montré l'importance de ces problèmes et l'intérêt d'études scientifiques de base. Les ingénieurs doivent en effet comprendre et maîtriser le gonflement qui peut infliger des déformations destructrices aux structures.

Une étude rationnelle de l'aspect mécanique du gel des sols devait donc être entreprise.

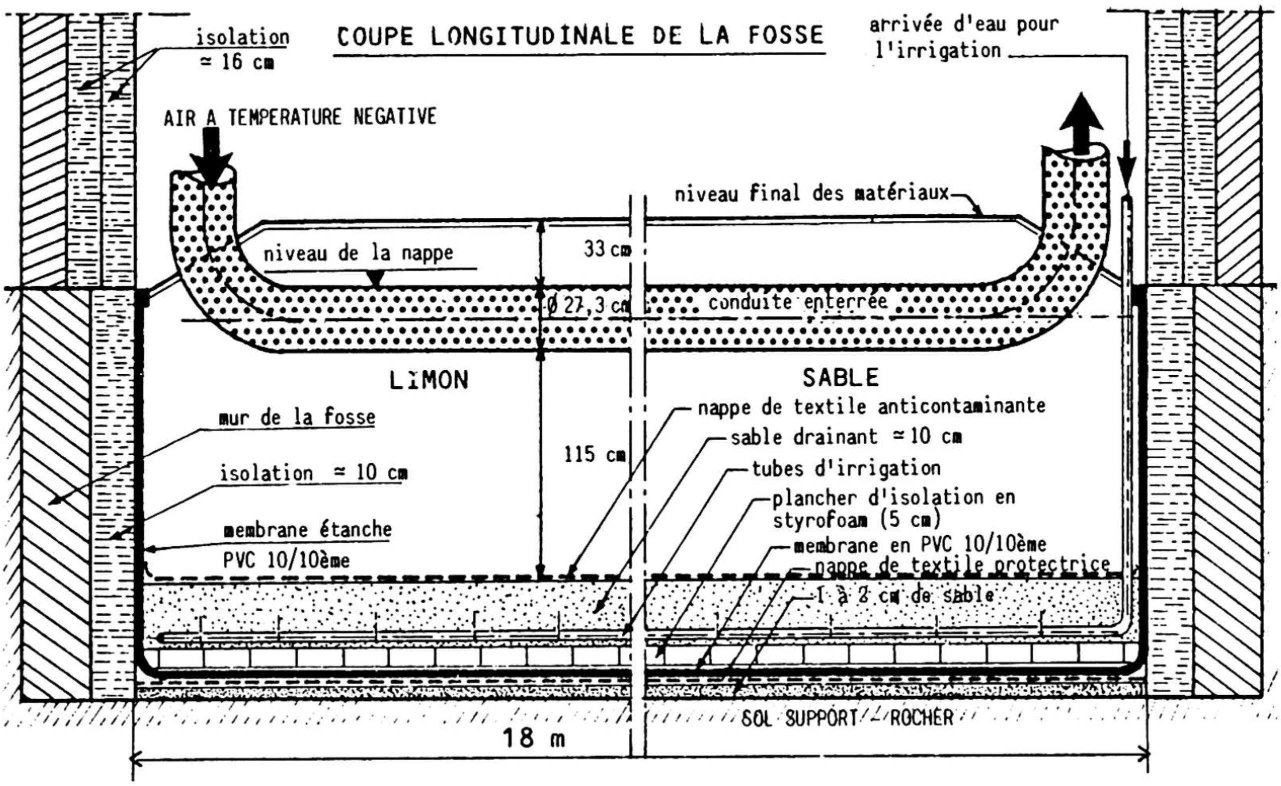


Fig. 1 — Expérience franco-canadienne. Coupe longitudinale

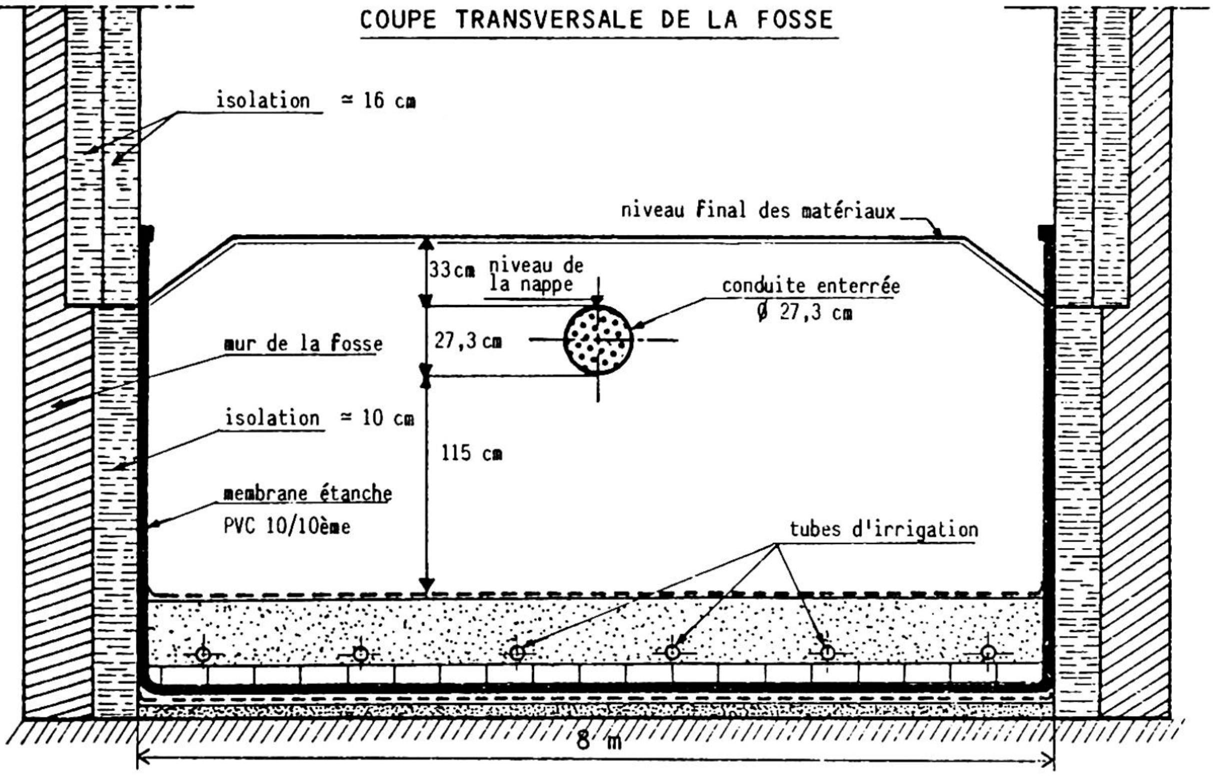


Fig. 2 – Expérience franco-canadienne. Coupe transversale.

Les ingénieurs civils sont confrontés à cette mouvance du sol. La protection du réseau routier français contre l'action du gel [8], les projets de gazoducs [3], [14] dans le grand nord canadien ont montré l'importance de ces problèmes et l'intérêt d'études scientifiques de base. Les ingénieurs doivent en effet comprendre et maîtriser le gonflement qui peut infliger des déformations destructrices aux structures.

Une étude rationnelle de l'aspect mécanique du gel des sols devait donc être entreprise.

2 – UNE ETUDE SCIENTIFIQUE

En 1980 on commença une expérience franco-canadienne [6], [7] (voir remerciements) de congélation de sols dans un environnement contrôlé. Une fosse (18 m x 9 m x 2 m) contenant deux sols différents préparés avec soin (sable et limon) est traversée par un gazoduc froid dont on maîtrise la température (figure 1).

La température de l'air ambiant est + 0,75° C, celle de l'air circulant dans le tube -6° C dans l'expérience actuelle. La nappe phréatique est maintenue 30 cm sous le tube de diamètre 273 mm lui-même recouvert de 30 cm de sol. L'expérience est menée de manière scientifique : les divers paramètres sont parfaitement maîtrisés et enregistrés. Ils ne sont pas soumis aux variations erratiques imposées par la nature. Leur influence peut donc être quantifiée de meilleure façon.

Les modèles du gonflement décrits plus loin demandent une bonne connaissance des lois de comportement mécanique, hydraulique et thermique. L'expérience ainsi que des essais en laboratoire au Canada et en France donnent les renseignements nécessaires par l'intermédiaire de nombreux instruments de mesure. Par exemple, la tendance remarquable des sols gelés au fluage est étudiée en fonction de la température et de la composition (en particulier en fonction des teneurs en eau et glace) [9].

Les déformations dans le tube lui-même sont continuellement enregistrées par l'intermédiaire de jauges de déformation et d'un appareil spécial évaluant la courbure du tube mesurée à partir des mouvements de tige en acier soudées sur le tube. Des méthodes optiques conventionnelles sont aussi utilisées. Des déplacements et des efforts dans le tuyau sont présentés sur les figures 3 et 4.

3 – LES MODELES

La physique et les observations décrites ci-dessus impliquent qu'une description rationnelle de la congélation des sols doit inclure à la fois les aspects thermique, hydraulique et mécanique.

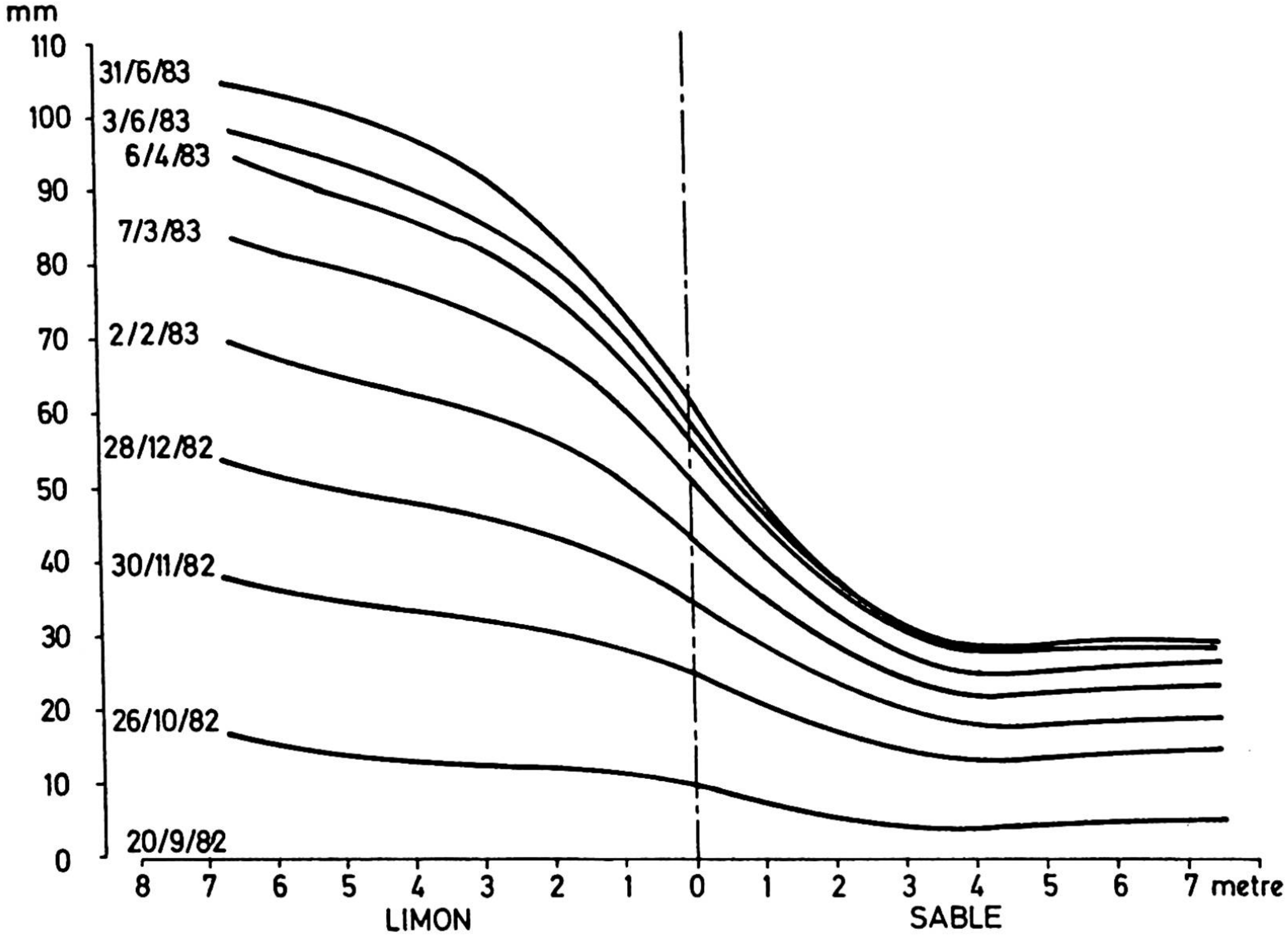


Fig. 3 — Déplacement du tube provoqué par le gonflement du limon et du sable.

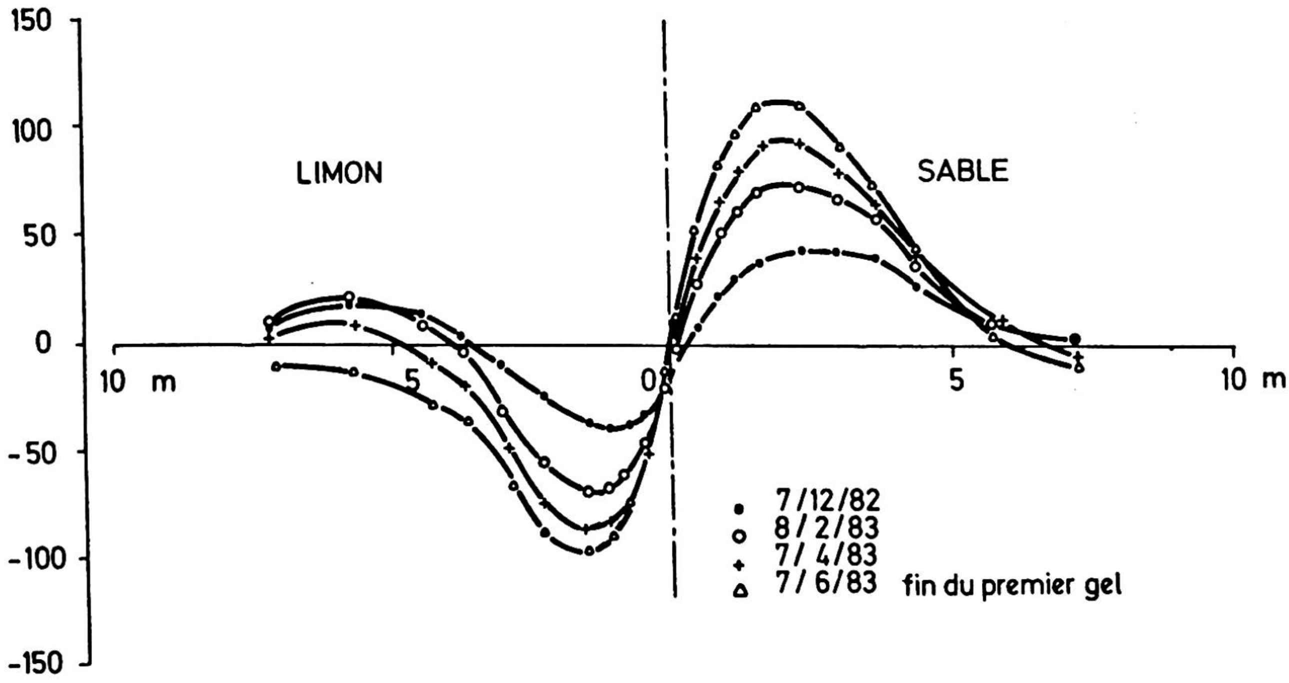


Fig. 4 — Effort dans le tube dû à la flexion provoquée par le gonflement

Les modèles thermiques sont maintenant classiques [4]. Les modèles liant les aspects thermique et hydraulique sont moins nombreux [1], [5] [12]. Des modèles plus spécialisés décrivant l'interaction sol gazoduc enterré ont aussi été mis au point [10], [11]. Le modèle le plus complet que nous présentons maintenant inclut les trois effets thermique, hydraulique et mécanique.

Les équations sont obtenues en utilisant — les lois de conservation :

(a) de la masse de l'eau et de la glace ;

(b) de l'énergie du milieu poreux ;

(c) de la quantité de mouvement du milieu poreux pour les effets mécaniques ;

— les lois de comportement :

(a) loi de Darcy avec une mobilité fonction de la température ;

(b) loi de Fourier avec une conductivité thermique fonction de la température ;

(c) conformément au point a du paragraphe 1, on tient compte de la présence d'eau liquide à des températures strictement inférieures à 0° C. La teneur en eau liquide apparaît naturellement comme une fonction décroissante de la température ;

(d) on suppose que le sol est visco-élastique dans sa partie gelée, le fluage étant du type Norton-Hoff.

On suppose par ailleurs que le sol est saturé et que sa porosité est connue. Les données du modèle sont les caractéristiques des sols et les actions extérieures thermique, hydraulique et mécanique. Les résultats de la simulation numérique sont l'évolution de la température, de la charge hydraulique et des contraintes dans le sol en fonction du temps.

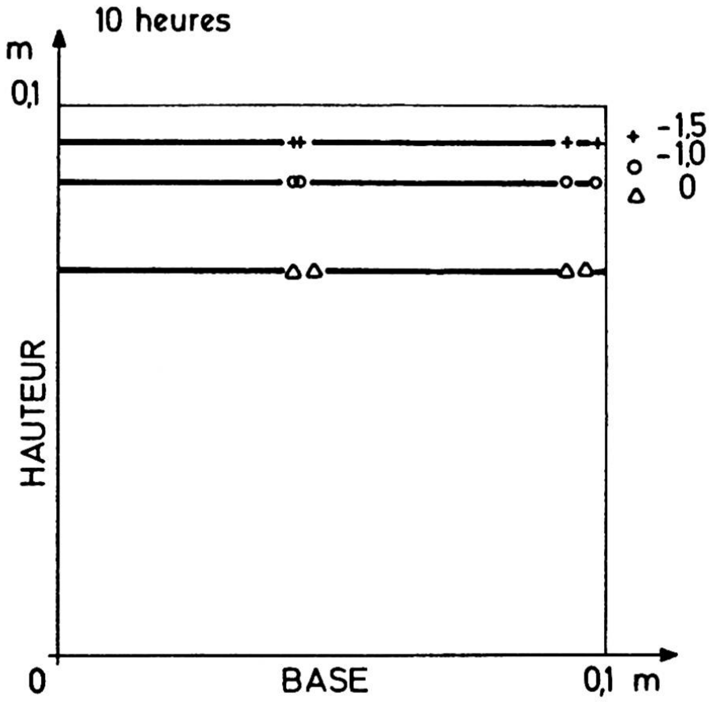


Figure 5 — Isothermes dans un sol congelé en surface.

A titre d'exemple, on examine l'évolution d'un sol non chargé congelé par le maintien d'une température négative (−2° C) à sa surface. La figure 5 représente les isothermes de température dix heures après le début de la simulation. La figure 6 représente la charge hydraulique en fonction de la profondeur. On note la forte diminution du potentiel dans la zone gelée provoquant le drainage de l'eau de la partie non gelée vers les zones les plus froides.

La figure 7 représente le déplacement du sol. On y constate un gonflement matérialisé par les flèches dont la longueur est proportionnelle au gonflement. Le gonflement maximal est de quelques millièmes de millimètre, ce qui est raisonnable dans la situation présente.

On doit remarquer qu'aucune action mécanique n'a été exercée sur le sol dont le gonflement ne résulte que de la congélation. On modélise donc bien un des aspects essentiels de l'action du froid sur les sols. Le modèle peut être utilisé par les ingénieurs pour simuler l'évolution des structures qu'ils projettent. Ces simulations numériques contribuent au dimensionnement rationnel des structures et permettent de prendre en compte les impératifs de sécurité attachés aux conditions extrêmes que l'on rencontre en milieu nordique.

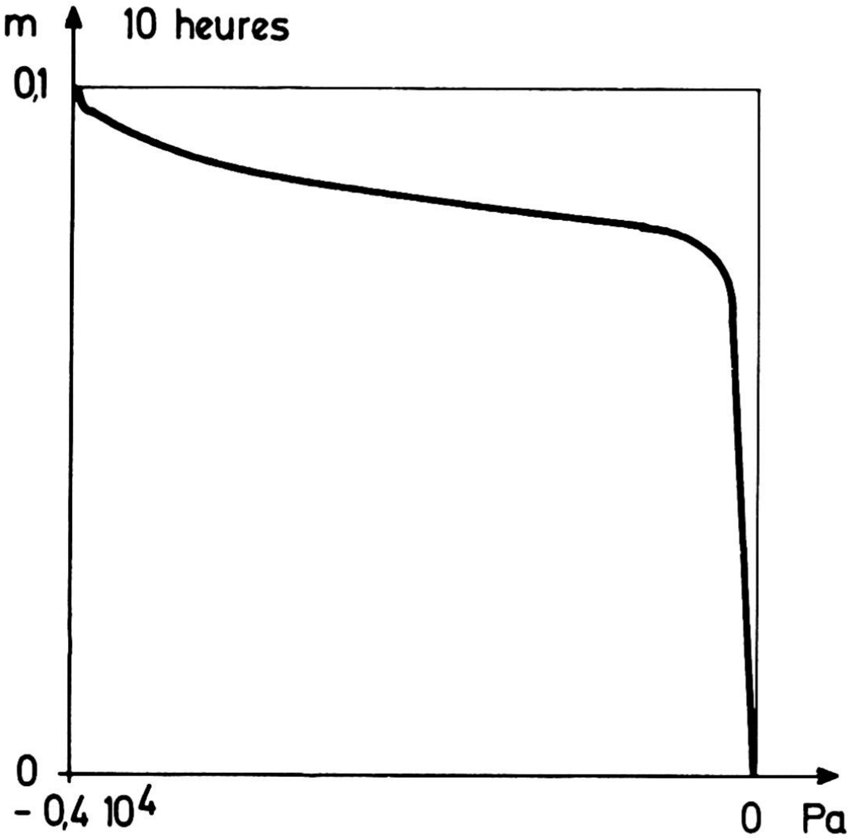


Figure 6 – Charge hydraulique en fonction de la profondeur. Noter la grande dépression due au gel.

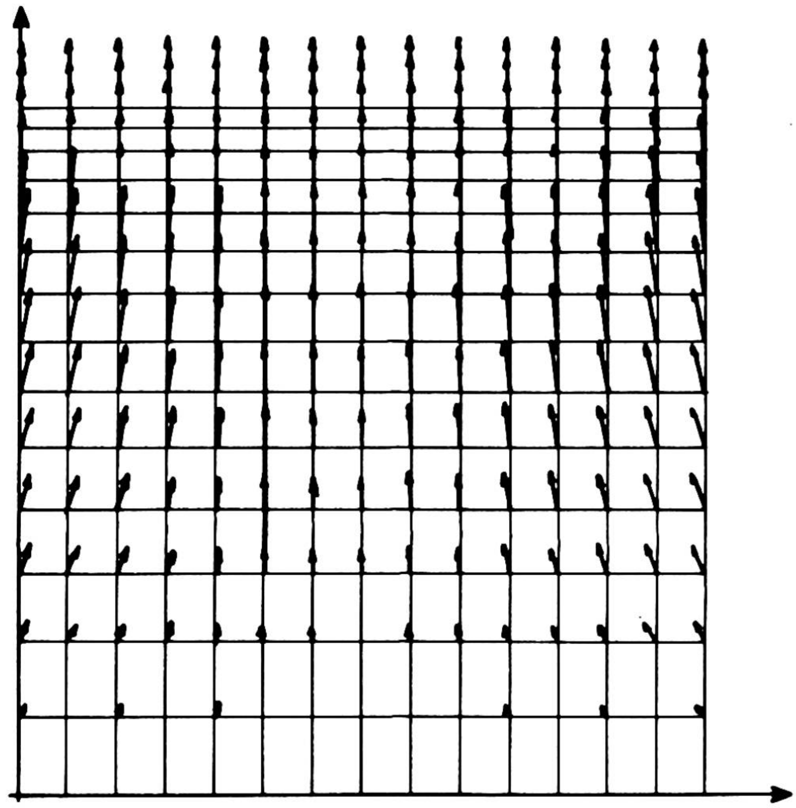


Figure 7 – Gonflements du sol. Noter qu'ils ne sont dus qu'à la congélation.

4 – CONCLUSION

Les expériences telles que celles qui sont menées à Caen clarifient les relations fondamentales gouvernant l'action du gel sur les sols. Elles permettent d'apprécier les données nécessaires à l'utilisation des modèles et par la suite de les valider.

Notons que dès maintenant l'aspect qualitatif du gel des sols est maîtrisé dans la plupart des situations usuelles. La méthode scientifique constamment utilisée, a permis de mettre en lumière les paramètres régissant les phénomènes. L'identification des relations qui les lient conduit aux modèles prévisionnels. Ceux-ci permettent un dimensionnement plus fiable et plus économique de structures civiles, par exemple les gazoducs, dans les régions nordiques.

5 – REMERCIEMENTS

L'expérience Franco-Canadienne a lieu à Caen au Centre de Géomorphologie du Centre National de la Recherche Scientifique. La participation française regroupe le Laboratoire Central des Ponts et Chaussées, le Centre d'Expérimentation Routière et le Centre d'Etude et de Construction des Prototypes du Centre d'Etudes Techniques de l'Equipement (Rouen), le Laboratoire Régional des Ponts et Chaussées de Nancy, le Laboratoire d'Aérothermique (Meudon) et le Centre de Géomorphologie du Centre National de la Recherche Scientifique.

La participation canadienne est subventionnée par le Ministère de l'Energie, des Mines et des Ressources du Canada, par l'intermédiaire d'un contrat avec les Géotechnical Sciences Laboratories de l'Université Carleton. Les participants sont des membres du Corps Professoral du Geography Department, du Ottawa-Carleton Geoscience Centre (M.W. Smith – P.J. Williams) et du Department of Civil Engineering (W.H. Bowes), de l'Université Carleton ; du Centre d'Ingénierie Nordique de l'Ecole Polytechnique (B. Ladanyi). Les participants sont aussi des étudiants diplomés : A. Guichoua, G. Lemaire (Ecole Polytechnique) ; M. Burgess (également au Ministère de l'Energie des Mines et des Ressources) ; S. Dallimore (Geotechnical Science Laboratories) ; D. Halliwell (actuellement à l'Université Mac Master) ; H. Crawford (Université de Western Ontario) ; D. Fisher. Le professeur visitant R. Kettle (Université d'Aston, U.K.) , G. Boyle, technicien (Geotechnical Sciences Laboratories) et beaucoup d'autres participent de façon significative.

BIBLIOGRAPHIE

[1] AGUIRRE-PUENTE, J. ; FEMOND, M. – Propagation du gel dans les milieux poreux. Joint I.U.T.A.M., I.M.U. Symposium on applications of methods of functional analysis to problems of mechanics. Marseille 2-6 Septembre 1975.

[2] AGUIRE-PUENTE, J. ; GRUSSON, J. – Measurement of permeabilities of frozen soils. Proceedings of the Fourth International Conference on Permafrost, Fairbanks, 1983.

[3] BERGER, T.R. – Northern Frontier, Northern Homeland, 1977.

[4] BLANCHARD, D. ; FREMOND, M. – The Stefan problem. Computing without the free boundary. Intern. Jal Num. Method in Engineering, vol. 20, 757-771, 1984.

[5] BLANCHARD, D. ; FREMOND, M. – La succession cryogénique dans la congélation des sols : un modèle macroscopique. Note aux C.R.A.S. Série II, t. 294, pp. 1-4 (4 Janvier 1982).

[6] Bulletin 27 du Centre de Géomorphologie du C.N.R.S. Expérience – Expérimentation – Modélisation – Simulation. Chap. IV. Expérience franco-canadienne, 1983.

[7] BURGESS, M. ; LEMAIRE, G. ; SMITH, M. and WILLIANS, P.J. – Investigation of soil freezing in association with a buried chilled pipeline in a large-scale test facility : Phase 1. Rept. to Energy, Mines and Ressources, Canada, Earth Physics Branch, 1982 (disponible en français).

[8] GEL et DEGEL des CHAUSSEES. Note technique du Laboratoire Central des Ponts et Chaussées, Janvier 1975.

[9] LADANYI, B. – Mechanical Behaviour of Frozen Soils in Mechanics of Structured Media. PT. B. A.P.S. Selvadurai (Ed.). Elsevier, 500 pp., 1981.

[10] LEMAIRE, G. – Etude expérimentale d'un gazoduc enterré soumis à des températures négatives. Mémoire de Maîtrise ès-Science Appliquée. Ecole Polytechnique, Montréal, 1983.

[11] RAULIN, Ph. – Communication personnelle.

[12] RUNCHAL, A.K. – Porfreez : A general purpose ground water flow, heat and mass transport model with freezing, thawing and surface water interaction. A.C.R.I. Technical Note T.N-005, 1982.

[13] SMITH, M.W. – à paraître 1984.

[14] WILLIAMS, P.J. – Pipelines and Permafrost. Longman, London and New York, 1979.

[15] WILLIAMS, P.J. – The surface of the earth. An introduction to geotechnical science. Longman, London and New York, 1982.

Integrated Deck Structures for Arctic Islands

Structures de planchers préfabriqués pour des îles arctiques

Integrierte Arbeitsdecks für Offshore-Bauten in der Arktis

Nat W. KRAHL
Eng. Consult.
Marine Div. Brown & Root
Houston, TX, USA

Phillip A. ABBOTT
Manager of Eng.
Highlands Fabricators
Nigg, Scotland

Valery M. BUSLOV
Eng. Consult.
Marine Div. Brown & Root
Houston, TX, USA

Kailash C. GULATI
Eng. Manager
Marine Div. Brown & Root
Houston, TX, USA

Philip J.M. RAWSTRON
Eng. Manager
Marine Div. Brown & Root
Houston, TX, USA

SUMMARY
This paper presents the results of a feasibility study of placing one or more integrated decks (I-DECKS) on a man-made island for drilling and producing oil and gas in Arctic waters. Separate drilling decks and production decks are proposed. Each deck and its facilities would be prefabricated, skidded onto a barge, towed to the site, and skidded onto the island. Deck footings provide positive compensation for settlements. The paper concludes that the concept is feasible and promises significant cost savings when compared to conventional modular facilities.

RESUME
Cet article présente les résultats d'une étude de faisabilité pour des dalles intégrées (I-DECKS) sur des plates-formes de forage de production de pétrole et de gaz dans les eaux arctiques. On propose des dalles de forage et de production séparées. Chaque dalle est préfabriquée avec ces installations, chargée dans une barge et amenée sur le site. Ensuite de quoi, elle est hissée dans la structure de la plateforme. Un système de réglage permet de compenser les tassements. Cette étude conclut que ce système est faisable et permettra des économies significatives par rapport aux autres systèmes conventionnels.

ZUSAMMENFASSUNG
Der Beitrag präsentiert die Ergebnisse einer Durchführbarkeitsstudie über die Anordnung eines oder mehrerer Zwischendecks auf einer künstlichen Insel für die Förderung und Produktion von Oel und Gas in arktischen Gewässern. Getrennte Decks für die Förderung und die Produktion werden vorgeschlagen. Jedes Deck mit seinen Einrichtungen würde vorfabriziert, auf einen Schleppkahn geschoben, an Ort geschleppt und auf die Insel geschoben. Um Setzungen auszugleichen, sind spezielle Auflager vorgesehen. Der Beitrag schliesst mit der Feststellung, dass das Konzept durchführbar und gegenüber vergleichbaren konventionellen Bauten kostengünstiger ist.

1. INTRODUCTION

1.1 Integrated decks for offshore platforms

In the past 40 years the search for oil and gas resources has extended offshore on a worldwide basis, and as the price of oil and gas has increased, the search has extended into deeper and deeper water and into the hostile environment of the Arctic region.

In conventional construction, separate modules for various drilling and production facilities are prefabricated onshore and then transported to an offshore platform and set on a deck substructure by a marine derrick barge. The modules are then hooked up and operations begin. In the North Sea, for example, 8 to 20 modules must be placed and hooked up, requiring an average of one million man-hours and ten months, at a cost of millions of dollars. To reduce these costs a new integrated concept was developed called HIDECK [1], in which the entire deck is prefabricated in one piece and all facilities are installed on the deck while onshore. The completed structure is then carried offshore on a barge (see Fig. 1) and mated with the supporting platform with an estimated savings of 90-95% of the offshore hookup. HIDECK was used for the deck structure of the Maureen Platform for Phillips Petroleum Company in the U.K. sector of the North Sea, with installation completed in 1984. Other integrated decks have also been used in the North Sea.

1.2 Man-made islands in the Arctic

When oil and gas exploration moved to the ice-covered waters of the Beaufort Sea, engineers were forced to design fixed and floating structures for the tremendous forces exerted by moving ice, sometimes of the order of 500 MN or more on a single structure. As a result the structural solution for most Beaufort Sea sites to date has been to build an artificial island of sand or gravel, and use drilling rigs and other facilities adapted from land-based operations. Modules of a convenient size and weight are prefabricated in Canada or the Lower 48 United States and generally shipped by barge to the island to be placed on the island and hooked up. The same problems of hookup time and cost are encountered on the islands as on offshore platforms, with even greater unit costs because of the severe environment and remote location. Therefore, the same economies as offshore would be achieved by integrating island facilities into a single unit, or several units, if they could be prefabricated, delivered, and installed successfully. This paper presents the results of a study of this specific topic by the Brown & Root Marine Engineering Division.

2. INTEGRATED DECKS FOR DRILLING AND PRODUCTION ON MAN-MADE ISLANDS (I-DECKS)

2.1 Requirements

In order to provide a realistic basis for this study, the assumption wasmade of a facility capable of producing up to 150,000 barrels of oil per day along with some natural gas.

2.2 Facilities design

The required drilling and production complex was studied in two configurations, one for a land-type operation with multiple modules on the island surface, and the other for four large structures in which the facilities could be grouped. Two such structures are devoted to production drilling and supporting facilities (Fig. 2), the others are devoted to production facilities for separation and processing (Fig. 3). For a variety of reasons, both the drilling and the production structure, during several design cycles, evolved into rather narrow, elongated structures with two main levels. The lower level would be completely enclosed while the upper level would support operations which could be individually enclosed as required. Certain items, such as personnel quarters, would be placed on the island separately. A comparison of the surface area of the islands, modular units vs. I-DECK, revealed that the I-DECK layout used only 46% of the area required by the modular units.

2.3 Structural design

The primary structural elements in both structures are full-depth trusses whose chords are part of the floor framing for the decks. On the drilling I-DECK (Fig. 2), longitudinal trusses lie along grid lines A and B, with transverse trusses on grid lines 3 and 7. Three major support points lie along line 3 and three more on line 7. The longitudinal trusses have a central span of 88 m and a cantilever at each end of 27 m, a condition which tends to equalize positive and negative bending moments in the trusses. On the production I-DECK (Fig. 3), longitudinal trusses lie along grid lines A, B, C, and D, with transverse trusses on line 2 and 5. Five major support points lie along each of lines 2 and 5. The longitudinal trusses have a central span of 85 m with a cantilever at each end of 30.5 m. In addition to supporting the live and dead loads for the in-place conditions described above, the structures were also designed to resist the loads that occur during construction, loading onto a barge, transportation on a barge, off-loading onto the island, and jacking to the proper elevation.

2.4 Fabrication

Large trusses such as these are normally fabricated flat on the ground and then rolled up into place for interconnection. Large items of equipment could be placed in each structure as it is built, if desired, or slipped into place later through the large openings in the main trusses before the exterior skin is attached. Fabrication would be performed at a site adjacent to water so that the completed structure could be loaded onto an ocean-going barge.

2.5 Transportation

The drilling I-DECK is estimated to require 3,900 tonnes of structural steel, to have a transport weight of 5,500 tonnes, and to have a total operating weight, in place, of 15,000 tonnes. Comparable figures for the production I-DECK are 4,100 tonnes of structural steel, transport weight of 8,200 tonnes, and an in-place, wet, operating weight of 14,000 tonnes. Using normal fabrication yard procedures, one structure could be skidded longitudinally aboard a barge, and the barge could be towed from the fabrication yard to the island. Computer analysis of transportation accelerations and stresses for a tow from either the West Coast of the United States or from the Far East indicated that the barge and the I-DECK would behave satisfactorily. For this study a barge was chosen with a length of 177 m, a breadth of 49 m, and a lightship displacement of 17,200 tonnes. Barge draft for the production I-DECK would be 3.7 m.

2.6 Installation

Fig. 4 shows the sequence of island construction operations in a schematic manner. The great structural strength of the deep longitudinal trusses is utilized not only in the in-place condition but also during installation. Because of the shallow slope of gravel islands (see Fig. 5), the bow of the barge must stand off a certain distance (estimated as 23 m) from the near edge of the skidways on the island. The I-DECK trusses have more than adequate strength and stiffness to cantilever off the bow of the barge until they touch down on the skidways. Skidding proceeds until the structure is in its final position with major support points directly over the permanent foundations (Fig. 6). A jacking mechanism is provided at each major support point to jack the structure to its final elevation.

The elevated position of the deck has several advantages (1) additional freeboard above the level of the sea and ice, (2) open space beneath the structure which allows the wind to sweep the snow away, and (3) ability to load consumables through a hatch in the lower deck from a vehicle below.

2.7 Foundation design

Most gravel islands to date have been intended only for exploratory drilling and have been classified as "temporary", with a short design life and no need for permanent foundations. For a production island, however, a longer useful life requires the provision of permanent foundations. Since settlements in man-made Arctic islands can be very large, these must be accommodated in some manner. Some of the basic characteristics of the I-DECK make it ideally adaptable to these difficult foundation conditions: (1) the long span between major supports makes it possible for the drilling I-DECKS' foundations to move away from the areas of greatest settlement due to thaw of the permafrost around the wellbore, and (2) the small number of support points and the provision of a separate jacking device at each of these makes it possible to relevel each of the support points as often as necessary to accommodate foundation settlement.

Spread footings are proposed for the permanent foundations. Two strip footings would suffice for supporting one I-DECK. Since each support can be jacked indefinitely, settlement is not a problem, and therefore the most important limitation for allowable bearing pressure on cohesionless soils is completely bypassed, and we observe that the unit bearing capacity of the footing goes up in direct proportion to footing width. Hence for any reasonable footing size such as we would have, there would be a large factor of safety. Also, with spread footings, we have avoided the problems of driving and maintaining piles in fill material and in frozen ground, with associated downdrag and uplift at various times. Finally, if we wish to make one further provision for the worst imaginable circumstances of surface settlement and tilting in the fill supporting the footings, each separate support point in a row of supports could be provided with a separate spread footing. This would allow the load to be taken completely off one footing, if it should, for example, tilt too much, with the transverse truss redistributing the load along that line of footings while the tilted footing itself is reset and releveled. The transverse truss and individual footings would have to be designed for this possibility of course. It should be noted that the footings and skidways could be prefabricated, possibly of steel, and transported to the island, to be installed during the island construction.

3. CONCLUSIONS

The complete scenario for the design, fabrication, transportation, and installation of an integrated deck (I-DECK) on a man-made Arctic island has been studied and has been determined to be feasible. A number of advantages of an integrated deck over the conventional practice of using a large number of small modules have been identified and include the following items:

- hookup man-hours, time, and cost can be tremendously reduced
- island size can be reduced
- number of sealifts can be reduced, thus improving the probability of finding a satisfactory summer "weather window" for installation
- the number and length of utilidors connecting various facilities can be reduced
- the small number of support points and the provision of a separate jacking device at each of these makes it possible to relevel each of the support points as often as necessary
- if we choose, we can provide a separate spread footing at each support point, thereby making possible the resetting and releveling of individual footings, one at a time, in the event of extreme settlement or tilting
- an integrated deck would make possible an easy removal of facilities after the field is depleted.

ACKNOWLEDGMENTS

The authors wish to acknowledge the contributions of the other persons at Brown & Root who participated in this study, particularly S.X. Gunzelman, R.A. Hayes, R.A. Roberts, M.H. Thibodeaux, B.D. Martin, W.F. Peters, A.F. Boyd, J.W. Pratt, and J.O. Davis.

REFERENCES

1. ABBOTT P.A., FARMER L.E., BLIGHT G.J., OSBORNE-MOSS D., A New Integrated Deck Concept, Offshore Technology Conference Paper No. 3879, May 1980.

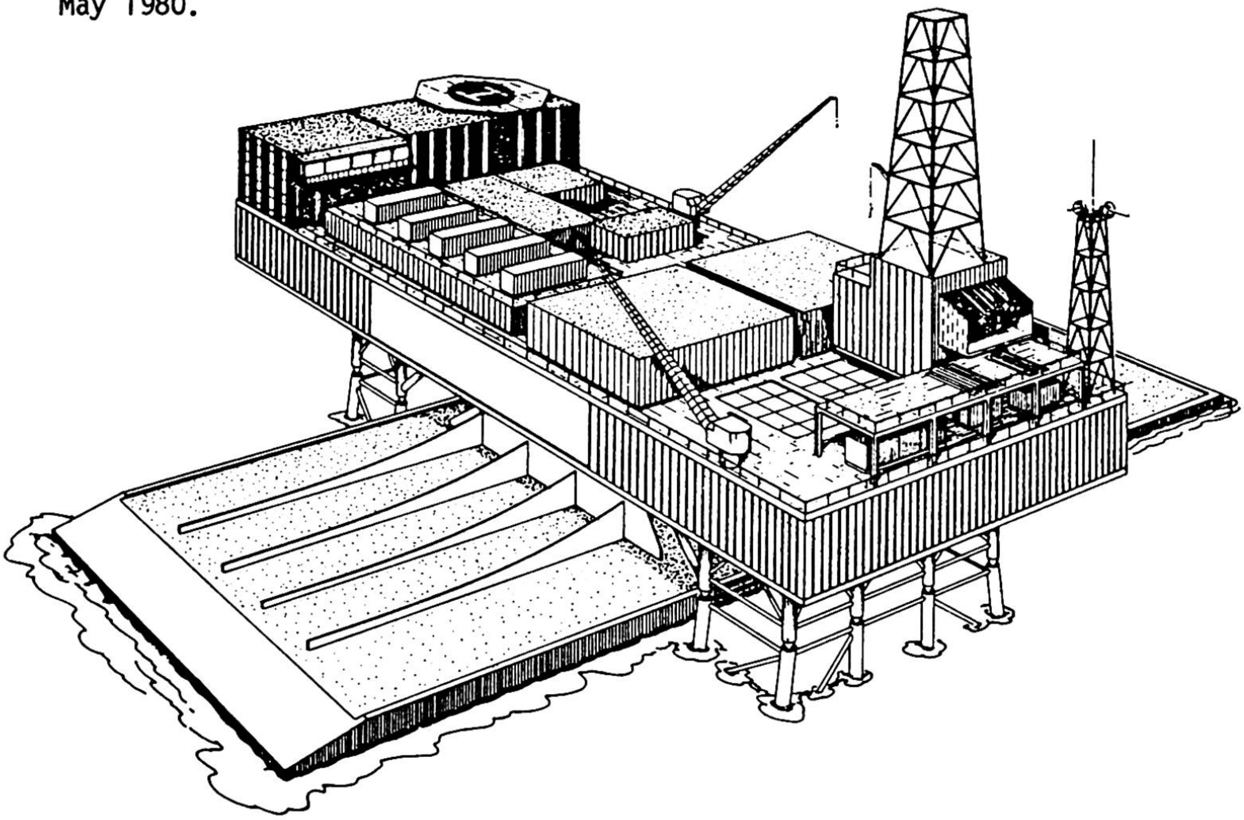


FIGURE 1- HIDECK - INSTALLATION BY BARGE

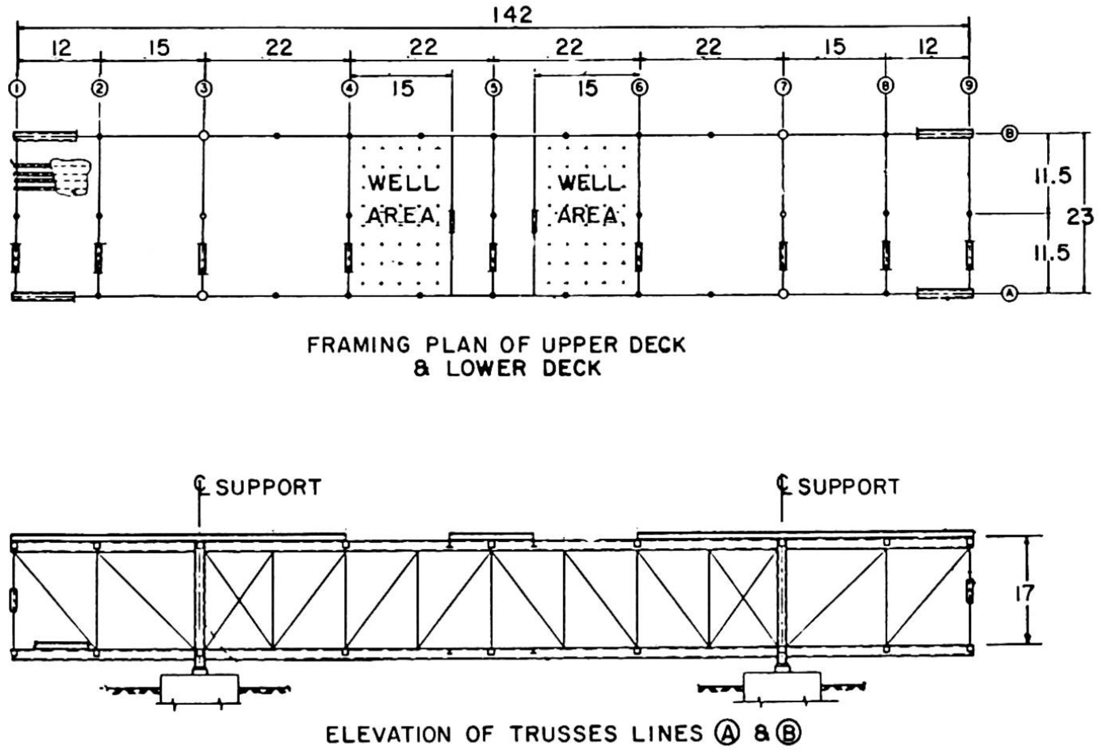


FIGURE 2- DRILLING I-DECK

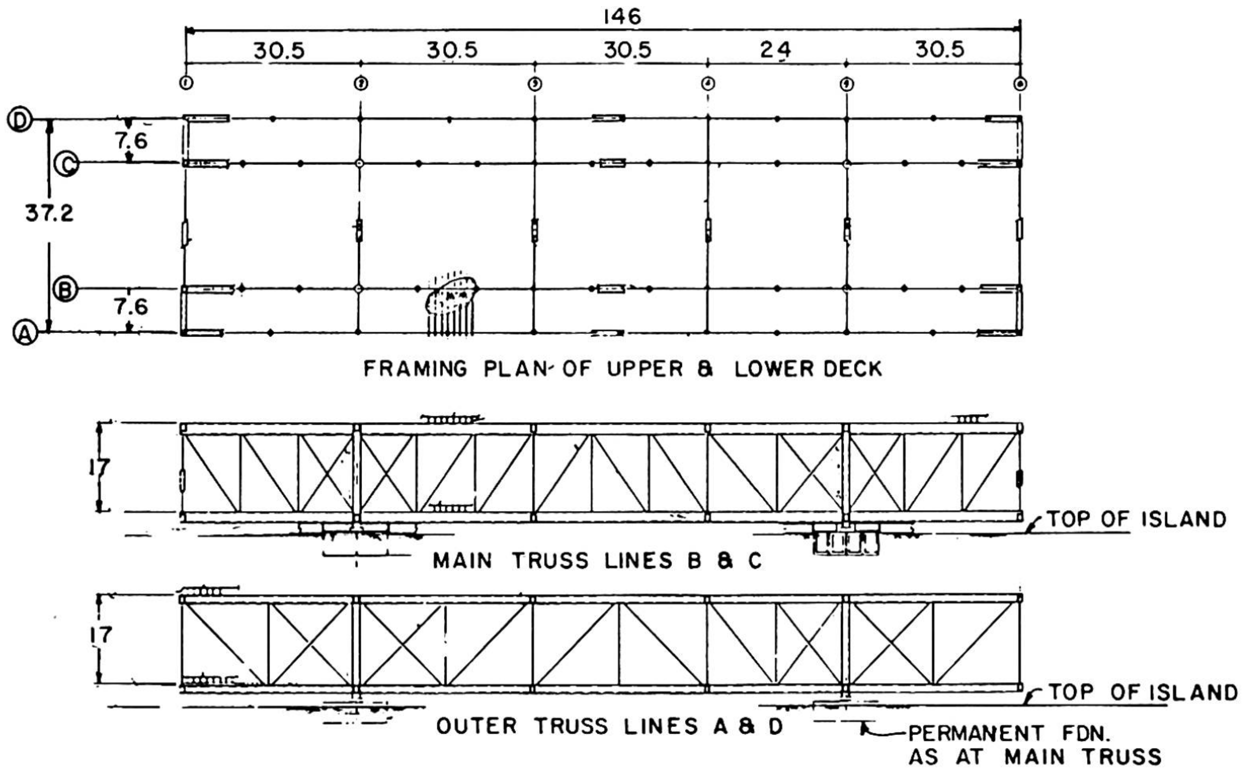


FIGURE 3- PRODUCTION I-DECK

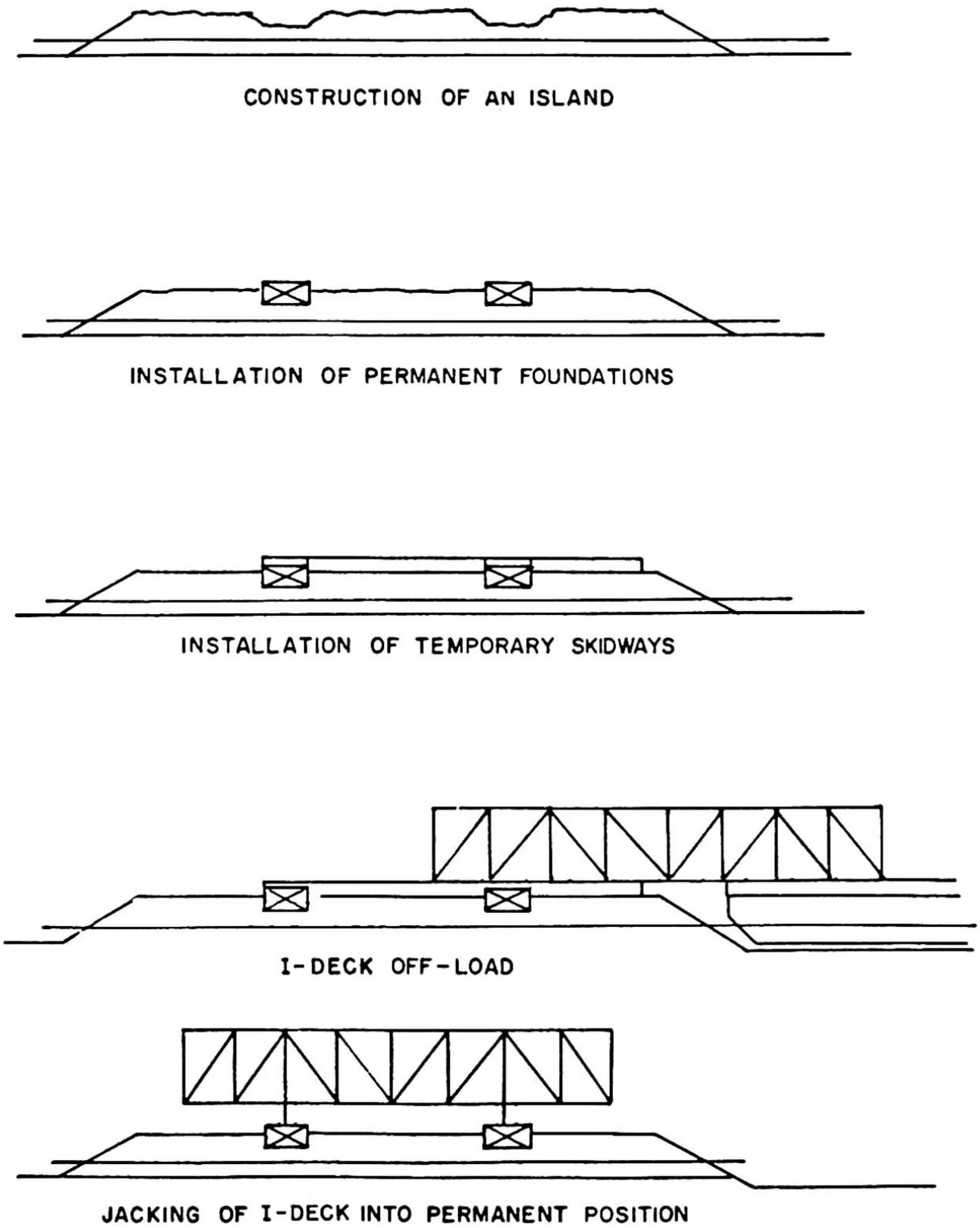


FIGURE 4– ISLAND CONSTRUCTION SEQUENCE (SCHEMATIC)

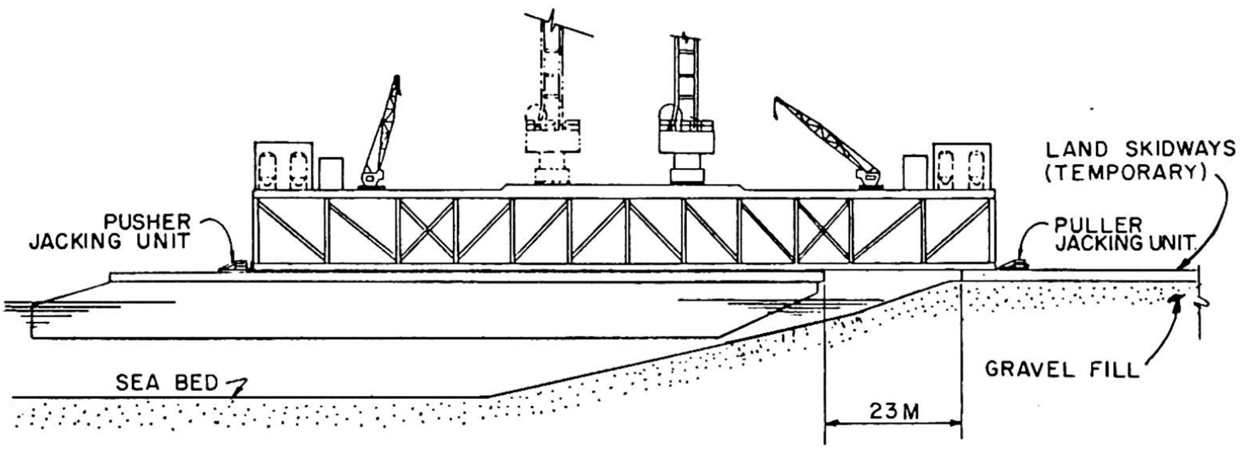


FIGURE 5- TRANSFERRING I-DECK TO ISLAND

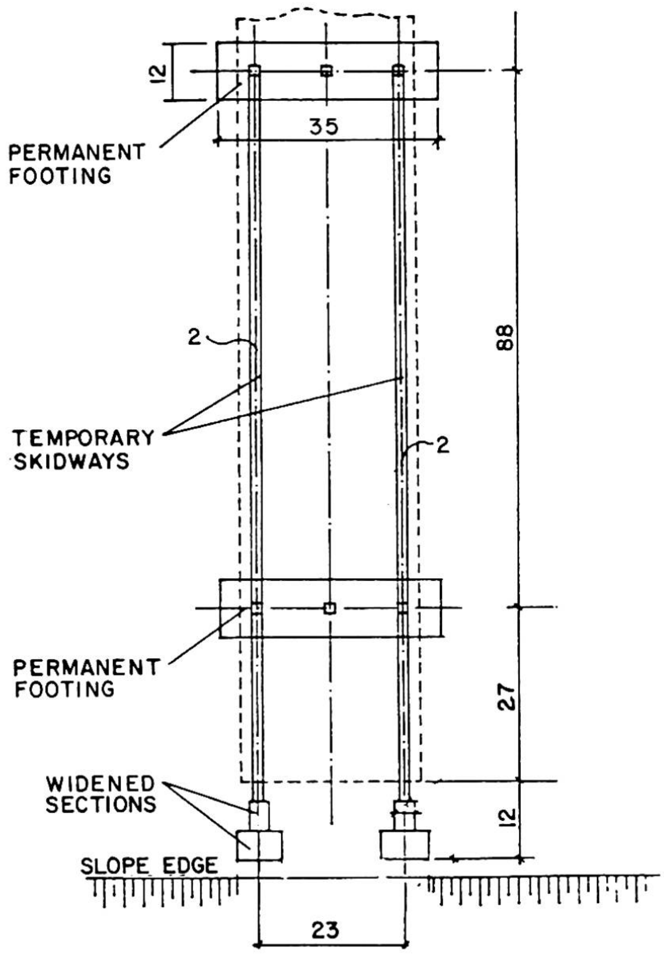


FIGURE 6- LAYOUT OF SKIDWAYS AND FOOTINGS FOR DRILLING I-DECK

Plateforme de forage d'exploration en mer arctique

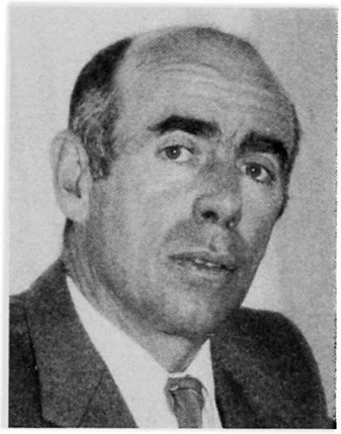
Bohrplattform im Arktischen Meer

Exploratory Drilling Unit for the Arctic Seas

Pierre RICHARD
Directeur Scientifique
Groupe Bouygues
Clamart, France

Né en 1927. A obtenu son diplôme d'Ingénieur de l'Ecole Nationale Supérieure des Arts et Industries de Strasbourg.

Henri MARION
Prés. Dir. Gén.
Bouygues Offshore
Le Plessis Robinson, France



Né en 1933. A obtenu son diplôme d'Ingénieur de l'Ecole Polytechnique.

RESUME

L'article présente une structure conçue pour des forages d'exploration ou de production en mer de Beaufort. Elle est aussi prévue pour d'autres utilisations, comme réservoirs de stockage des hydrocarbures ou terminaux maritimes par exemple. Le principe de la structure spatiale permet un gain de poids pouvant atteindre 25%, ainsi qu'une répartition optimale des efforts, d'où une réduction correspondante des quantités de béton, d'armatures et d'aciers de précontrainte.

ZUSAMMENFASSUNG

Der vorliegende Artikel beschreibt eine Struktur, die für Probebohrungen oder Produktionsbohrungen im Meer von Beaufort entwickelt wurde. Ähnliche Bauwerke können auch für grössere Tiefen angewendet werden, wie vor der Küste von Alaska, oder für andere Anwendungen, wie z.B. die Oel-Lagertanks an den Meeres-Terminals. Das Prinzip der Raumstruktur ermöglicht eine Gewichtseinsparung von bis zu 25% und eine optimale Kräfteverteilung und daher eine Verringerung der Betonmenge, des Baustahls und der Vorspannung.

SUMMARY

As an example, a unit developed for exploration drilling or production in the Beaufort Sea will be presented. This structure, developed for a water depth ranging from 10 to 30 m can be used also to deeper water depths; other Alaskan offshore areas, such as Chukchi Sea and Norton Sound; and other uses, such as crude oil storage tanks and marine terminals. The space frame principle permits an optimum distribution of the forces and, therefore, a minimization of concrete quantities as well as reinforcement and prestressing.

1 - ABSTRACT

As an example, a unit developed for exploration drilling or production in the BEAUFORT SEA will be presented. This structure developed for a water depth ranging from 30 to 80 feet can be extrapoled to deeper water depths, other Alaskan offshore areas such as CHUKCHI SEA and NORTON SOUND for instance and other uses such as crude oil storage tanks, marine terminals,...

The space frame principle permits an optimum distribution of the forces and therefore a minimization of concrete quantities as well as reinforcement and prestressing.

2 - LES DONNEES DU PROBLEME

les forces qui déterminent le dimensionnement proviennent de l'action de la banquise sur la plateforme alors que celle-ci est posée sur le fond pendant les opérations de forage.

2.1 Forces exercées par la glace

2.1.1. Les efforts exercés par la glace ne peuvent être définis dans un article aussi court d'une part, et d'autre part leur détermimation n'a pas un intérêt direct avec la conception de la structure.

il faut cependant savoir que l'on distingue d'une part :

- des effets d'ensemble dûs à la dérive de la glace
- des efforts locaux qui se produisent l'été lors de la débâcle de la banquise.

d'autre part

- des efforts d'ensemble exercés par la glace formée dans l'année
- des efforts d'ensemble exercés par des superpositions de plaques (ridges) constituées donc des glaces de l'année ou des années précédentes.

2.1.2. Les efforts d'ensemble sont également fonction de la longueur des côtés de la structure s'opposant à la dérive de la glace.

La structure est en plan, un polygone inscrit dans un cercle de 120 m de ∅ environ au niveau d'appui au fond de la mer.

Les parois de la structure étant inclinées, les côtés du polygone sont de dimensions variables avec la profondeur de l'eau. Les efforts exercés par la glace sur la structure sont donc fonction de la profondeur d'eau.

Enfin, ces efforts sont fonction de l'inclinaison des parois de la structure, du coefficient de frottement entre les parois et la glace et de l'adhérence éventuelle de la glace sur les parois.

2.1.3. Efforts exercés sur l'ensemble de la structure dans le cas que nous avons étudié - Polygone à 12 côtés - Profondeur d'eau de 30p à 80p.

CHARGES GLOBALES (en kips : unité de charge de 100 livres)

PROFONDEUR DE L'EAU	30 p.	65p.	75p.
ECRASEMENT DE LA GLACE DE PREMIERE ANNEE			
Période de retour de 100 ans	143 980	126 820	120 850
Période de retour de 25 ans	134 750	118 690	113 100
EFFORTS EXERCES PAR UN RIDGE	au-dessous de 60p.		au-dessus de 60p.
Période de retour de 100 ans	112 840		131 170
Période de retour de 25 ans	95 860		108 600

3 - LA STRUCTURE DE LA PLATEFORME

les forces agissant sur cette structure sont très importantes et cependant il est nécessaire que la structure de cette unité soit aussi légère que possible pendant la phase de remorquage précédant sa mise en place, de façon à avoir un tirant d'eau aussi faible que possible : ce qui donne la possibilité de remorquer la plateforme dans des eaux peu profondes au voisinage des côtes, là où les eaux sont libres tôt, à l'approche de l'été.

On augmente ainsi les chances d'une mise en position de la plateforme pendant la courte saison de l'été.

Certaines solutions peuvent être l'emploi du béton léger à haute résistance de l'ordre de 7 000 psi. Le gain de poids par cette technique est de l'ordre de 15 %.

La solution que nous décrivons ici est totalement différente.

La structure est spatiale, composée de barres. Le béton de ces barres est toujours comprimé, soit par effet de la précontrainte, soit par l'action des forces extérieures, ou par la combinaison des deux actions.

De ce fait, il y a intérêt à utiliser des bétons classiques à haute résistance. Le projet actuel utilise un béton à 10 000 psi. Il faut d'ailleurs noter que ce choix de 10 000 psi est prudent, nous savons aujourd'hui exécuter des bétons dont la résistance est de l'ordre de 14 000 psi.

Ces choix conduisent à une réduction de la quantité de béton nécessaire supérieure à 25 %, ce qui est donc plus intéressant en gain de poids que celui donné par les agrégats légers.

D'autre part, la structure spatiale offre une multiplicité de points d'appui pour les parois, le radier et la couverture du volume défini par la structure spatiale.

De ce fait, ces éléments qui représentent une part importante de la quantité de béton nécessaire seront eux aussi plus économiques.

Cette structure spatiale conduit également à une économie de précontrainte et d'acier passif importante.

Il faut noter la forme particulière, plissée, des parois, lesquelles sont soumises à un gradient thermique important dû au fait qu'une partie de cette paroi est immergée, donc à une température voisine de 0°, et l'autre partie (supérieure), à l'air libre, à une température qui peut être inférieure à -50° (fig. 1)

4 - L'EXECUTION

Pour une raison d'économie évidente, la structure spatiale est préfabriquée.

Cette méthode d'exécution permet donc une industrialisation et un contrôle de qualité stricte du béton à haute résistance.

La fig. 2 donne la géométrie de l'objet élémentaire composant la structure. Nous avons fait le choix de préfabriquer ce qui est difficile à exécuter, c'est-à-dire le noeud avec les demi-barres qui y aboutissent et d'assembler ce qui est simple, les barres au milieu de leur longueur, là où les moments de flexion sont extrêmement faibles.

La dimension transversale des barres (Ø de l'ordre de 2') et leurs longueurs 20' environ font que le phénomène de flambement est pratiquement inexistant.

Chaque élément de la structure est stable, ce qui facilite l'assemblage des éléments entre eux sans immobiliser l'èngin de manutention.

Les câbles de précontrainte sont extérieurs au béton, protégés par des gaines étanches et injectés au coulis de ciment. Il n'y a donc pas d'opération d'enfilage difficile.

La fig. 3 donne une schématisation de la structure spatiale. Le radier est également préfabriqué, composé de dalles triangulaires. Les parois sont également préfabriquées par éléments conjugués en forme de caissons, formant ainsi un espace fractionné à la périphérie de la plateforme permettant d'augmenter la sécurité de celle-ci en cas de choc avec une plaque de glace.

Les moules de préfabrication de ces éléments sont clos et le bétonnage se fait par injection du béton.

La qualité de résistance de ce béton est complétée par deux caractéristiques particulièrement intéressantes :

- Porosité 2 à 3 fois plus faible que celle d'un béton classique.

  qualité particulièrement intéressante pour sa résistance au cycle gel - dégel.

- Excellente résistance chimique à l'eau de mer par l'emploi de silica fume pour sa composition.

5 - CONCLUSION

Cette technique de structure que nous développons également pour des ponts notamment nous paraît donc parfaitement adaptée au probléme posé. Elle est en outre très économique.

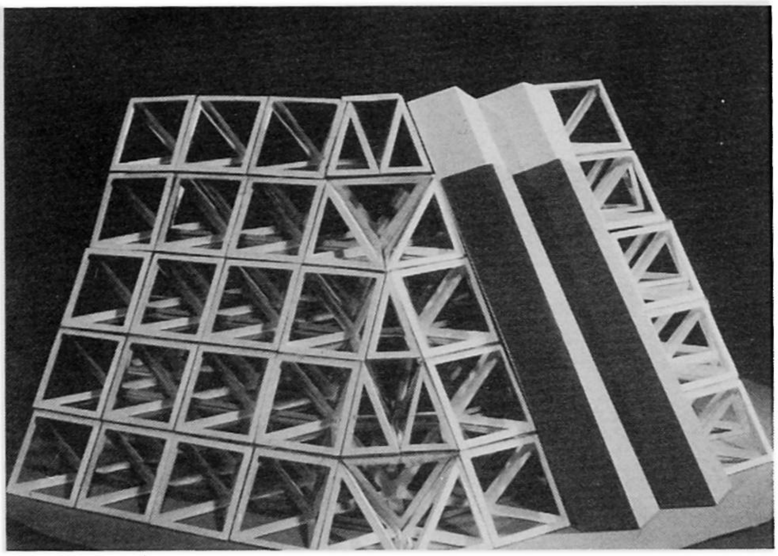


Fig. 1

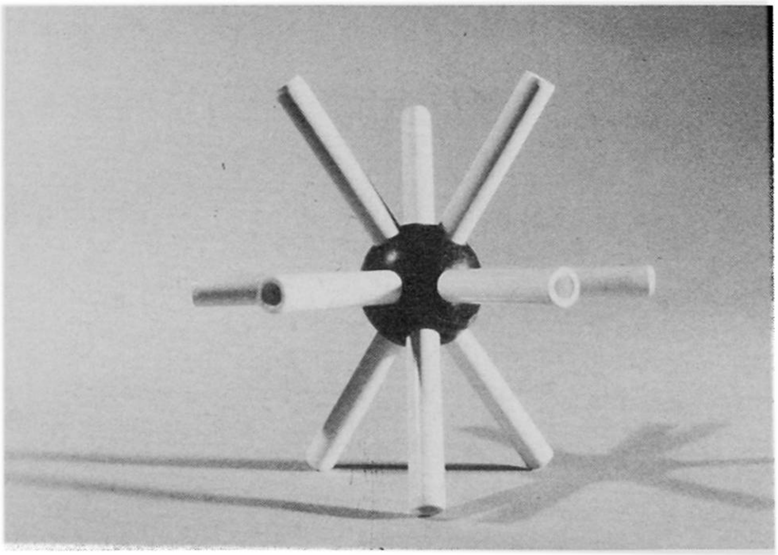


Fig. 2

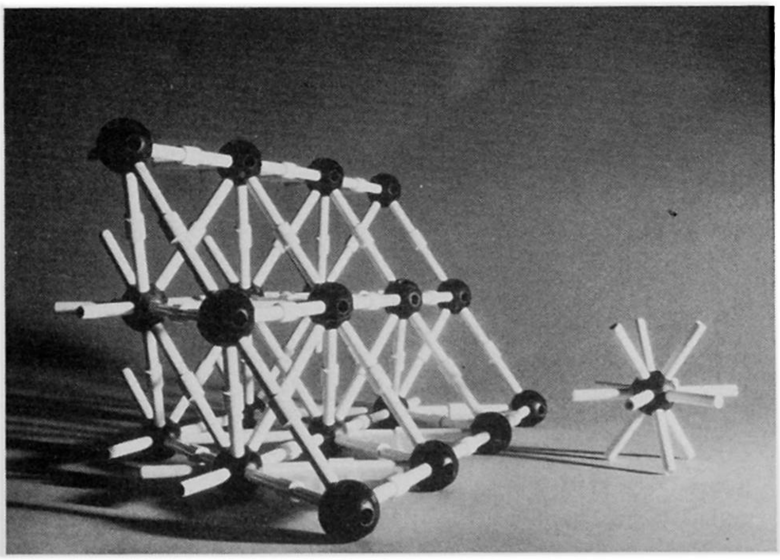


Fig. 3

Leere Seite
Blank page
Page vide

Leere Seite
Blank page
Page vide

Leere Seite
Blank page
Page vide