PILED VS GRAVITY OFFSHORE STRUCTURES FOR SEISMICALLY ACTIVE ARCTIC REGIONS

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ABSTRACT

Two designs of steel offshore structures for seismically active ice infested regions are presented and compared on cost, weight, ease of fabrication and installation.

The comparison is developed for a site with 30m water depth, moderate one year ice, and earthquakes up to Richter strength eight.

Gravity structures are found to have low installation costs and are attractive for sites with strong seabed conditions and a short weather window.

The piled structure considered is a conventional jacket with a conical ice shroud. The piled structure has a much lower dynamic mass, generates lower earthquake forces and is preferred for sites with weak seabed conditions.

NOMENCLATURE

ρ_i	Density of ice
ρ_w	Density of seawater
g	Acceleration due to gravity
H_o	Horizontal force to fail an ice sheet in bending on an upward breaking cone.
σ_c	Compressive strength of ice (uniaxial)
σ_F	Flexural ice strength
t	Ice thickness
E	Ice modulus of elasticity
z	Height attained by broken ice on an inclined surface
b	Ice sheet or structure width
V_b	Vertical ice load on an inclined surface
α	Underwater slope angle of ridge keel to horizontal
D	Structure width or diameter
D_c	Cone diameter at the still water level
K	Contact factor : $K = 1$ for drift ice $K = 1.4$ for outbreak load calculations
F_s	Force to shear off the unconsolidated portion of a pressure ridge
ϕ	Angle of internal friction of unconsolidated rubble ice (assumed 45°)
B	Width of unconsolidated portion of a ridge
F_c	Penetration force to fail consolidated ice in crushing.
I	Indentation factor $I = 2.5$ for narrow structures D/t 10: $I = 1$ otherwise
m	Shape factor $m = 1$ for circular surfaces
h_k	Depth of unconsolidated portion of a ridge
h_c	Thickness of consolidated portion of a ridge
C_u	Undrained shear strength of cohesive soil
P_o	Effective vertical stress in soil
K_o	Earth pressure coefficient
OD	Outside diameter

1.0 INTRODUCTION

There are several Arctic offshore areas such as the West coast of Canada or Alaska or the East cost of the USSR which suffer one year sea ice and severe earthquakes.

This paper evaluates structures for such areas. A one year sea ice of mean thickness 1 metre rafting to 2 metres, is assumed present for up to 9 months of the year. Ice pressure ridges will also occur due to the movement of the ice which may have keels extending to 27 metres below the mean water level with sails of up to 5 metres in height. Tidal fluctuations and upwelling of water through cracks in the ice may result in ice up to 4 metres thick local to a fixed platform.

The design wave in these areas is typically of height 12 metres, which occurs in the Summer.

The design earthquake is taken to be of a strength equivalent to Richter scale eight. The soil conditions are assumed to be poor corresponding to a 30m layer of fine sand overlying normally consolidated clay.

To minimise wave and ice loads cut water plane area in the ice zone should be minimised. The example site required 49 conductors at 3m spacing (centre to centre). To minimise water plane area and avoid assymmetric loading, a square 7 x 7 grid was assumed for this comparison.

The dry topside weight, was assumed to be 20,000 tonnes which determines the hydrostatic characteristics of the gravity structure during transportation and installation. The operating topside weight was taken to be 29,000 tonnes for both concepts examined.

2.0 CHOICE OF CONCEPTS

The concepts considered for the ice/earthquake site and the reasons for selection of two concepts for more detailed discussion are:

Floating or Fixed:-

Floating structures have limited payload and may suffer from "down-time" (when oil cannot be produced) due to riser and mooring system limitations. Contemporary mooring systems cannot easily withstand the sustained horizontal ice loads expected at the example site.

Multi-Leg or Single Leg Structures:-

For the purposes of selection of ice resistant platforms a multi-leg structure is one where there is more than one ice protective cone (or shroud) at the water-line with ice able to flow between. In this sense the internally braced monopod shown in Figure 1 is essentially a one legged structure.

For 30m water depth with the ice conditions prevailing the one-legged or monopod structure is preferred because of the lower ice and earthquake loads for a given topside configuration.

Structural analysis of two deck designs for multi-legged structures revealed high stresses in the deck as the deck not only has to support the topsides, but also has to transmit loads between the legs.

The uncertainties associated with ice blockage between legs in the ice loading also discouraged the selection of the multi-legged variants.

Gravity or Piled:-

The effectiveness of a steel gravity platform depends on the prevailing soil conditions. Hence it is wise to examine the possibility of piling the structure into underlying strong soil layers that are available to steel structure designs.

Liquefaction of the soil beneath the structure may also occur during earthquakes [3, 5, 6 & 7]. However, the short weather window available for deck installation and piling may make a gravity structure preferrable because it carries the deck and topsides to site. Both variants have been considered.

Steel or Concrete:-

For given soil conditions, concrete gravity structures have inferior earthquake performance to steel gravity structures, simply because of the lower strength and lower density of the material. A preliminary concrete design for the environmental conditions at the site displaced 260,000 tonnes had a base diameter of 136m. The lower density of concrete results in higher immersed volume for a given weight and hence more solid ballast must be included in the design in order to give the same nett foundation pressure. This higher immersed volume results in increased buoyancy and added mass (which is proportional to immersed volume) and hence the overall dimensions of the structure must be increased. Earthquake loads increase with increasing immersed volume necessitating higher base dimensions to resist sliding.

The two structures considered are shown in Figures 1 and 2.

1) Internally braced monopod (Fig. 1) which is a tubular steel jacket, supported by piles, with a conical ice shroud.

2) Stiffened plate monopod (Fig. 2) with a gravity base.

3.0 DESIGN CONSIDERATIONS

3.1 Environmental Conditions

The two designs were based on the following environmental conditions:-

a)	Water Depth	30 m
b)	Tidal range	2 m
c)	Design wave height	12 m
d)	Design wave period	10 - 12 secs
e)	Ice thickness	2.0 m (rafted ice)
	floe velocity	1.0 m/sec
	sheet velocity	0.1 - 0.2 m/sec
	Ice thickness local to structure	4.0m
	Ice Ridge	4m high, with 27m deep keel considered equivalent to 5.4m consolidated ice + 25.6m unconsolidated moving at 0.1 - 0.2 m/s
f)	Earthquake	8 (Richter scale)
	Effective Ground Acceleration	0.15g
g)	Soils allowable bearing pressure	200 - 300 KN/m^2
	Effective angle of internal friction	27.5°

3.2 Foundations

Soil conditions were conservatively assumed to be loose sand overlying normally consolidated clay. Clearly, if glaciers overlaid the soils during a pre-historic ice age and there has been no subsequent deposition, then clay layers would be overconsolidated. Also, slow deposition of sand layers would cause sand layers to be dense. However for comparison, a loose sand of internal friction angle 27.5° was assumed to extend to 30m below seabed. The underlying clay was assumed to have an undrained strength to effective vertical stress ratio (Cu/Po) of 0.20.

For such conditions, limiting bearing pressure of 200-300 KN/m^2 was assumed for the gravity foundation. Sands with a relative density of less than 50% may be prone to liquefaction [3, 4]. To avoid the possibility of liquefaction under earthquake conditions, the ratio of shear to normal stress on the sand must be limited. For illustration, the Ekofisk tank in the North Sea is designed for a maximum stress ratio at the base of 0.4 under extreme storm conditions [5]. A maximum stress ratio of 0.7 is likely to cause initial liquefaction of a sand within 10 cycles of loading unless the initial relative density is at least 75% [6].

A piled foundation also needs to be designed for the possibility of loss of strength of surface soils. However, the stress in the sand will be less and therefore liquefaction is less likely. Even if the surface sand layer did liquefy, the piled foundation will still have a significant reserve of capacity from the axial capacity of the piles in the clay layers and portal action lateral capacity.

Assuming an earth pressure coefficient (Ko) of 0.8 and a submerged soil weight of 8 KN/m^3, the ultimate axial capacity of a 3m diameter pipe pile may be conservatively estimated from API RP2A [2] as 20MN at 45m penetration, 33MN at 60m penetration, and 50MN at 75m penetration. Lateral capacities of piles may be assessed either from elastic stresses in the pile wall linked to a non-linear analysis of the soil response or by reference to an appropriate factor of safety against fully developed foundation failure. Lateral forces per pile were conservatively limited to 5MN on 2.134m (84") OD piles, 6.5MN on 2.438m (96") piles and 10MN on 3m (118.1") OD piles.

4.0 LOAD COMBINATIONS

The following load case combinations were considered:-

1. Dead load, live load, wind, current, 100 year design wave.
2. Dead load, live load, wind, current, maximum ice loading.
3. Dead load, live load, wind, current, earthquake.
4. Dead load, live load, wind, current, earthquake, ice break out load.
5. Dead load, live load, wind, current, 10 year significant wave, ice floe collision.
6. Dead load, live load, wind, current, ship collision.

Combination load cases 1 to 3 are catastrophic events each with a low probability of occurrence. The 100 year design wave, maximum ice loading and design earthquake are assumed not to occur concurrently. Furthermore ice pressure ridge impact is a short term impulsive loading not considered to occur during an earthquake. Load case combinations 1, 3, 5 and 6 are well documented in the literature [1, 2, 13] and so discussion will be limited to load combinations 2 and 4.

4.1 Ice Loading

Two main ice loading conditions were considered in this design comparison:-

i) Ice ridge loads whereby a first year ice pressure ridge impacts the structure. In such ridges only the upper part is consolidated, the keel consisting of unconsolidated loosely accumulated ice blocks. The two parts are assumed to fail in different modes, the consolidated part failing as level ice and the keel being sheared off. The idealised ridge section is shown in Figure 3.

   For the purposes of calculation the ridge is considered equivalent to 5.4m of consolidated level ice and a further 25.6m of unconsolidated ice which fails in shear on contact with the structure.

ii) The outbreak case whereby the struture is completely frozen into level or rafted ice or the consolidated part of a pressure ridge. It is assumed there is no adfreeze bond between the ice and structure due to tidal action and effective coating of the jacket cone. The surrounding ice (of 4m thickness) is assumed to fail in shear.

In calculating the loading the following formulae (from [8])were used:-

The horizontal force, H_b , required to fail level ice rafted ice or the consolidated part of a pressure ridge in bending at an upward breaking cone is given by:

$$H_b = A_4\left(A_1 \times 0{\cdot}67\sigma_F t^2 + A_2\rho_w g t D^2 + A_3\rho_w g t\left(D_c^2 - D^2\right)\right)K \quad \cdots 1)$$

The vertical force Vb is

$$V_b = \left(B_1 H_b + B_2\rho_w g t\left(D_c^2 - D^2\right)\right)K \qquad \ldots 2)$$

Where A1,A2, A3, A4. B1.B2 are parameters derived in [9] from the plastic limit analysis dependent on θ the angle of the surface to the horizontal.

The other parameters are defined in the nomenclature section.

The force required to shear the unconsolidated portion of a ridge, F_s can be calculated according to:

$$F_s = \tfrac{1}{3} B h_k^2 \left(\rho_w - \rho_i\right) g \tan^2\left(45^\circ + \phi/2\right)\tan\phi \quad \cdots 3)$$

The force required to fail level ice, rafted ice or the unconsolidated portion of a pressure ridge Fc is given by:

$$F_c = I m K \sigma_c D_c t \qquad \ldots 4)$$

where

K is the contact factor taken to be 0.5 for drift ice.

The values taken for the physical parameters are given in Table 1.

TABLE 1

Ice Physical Properties (KPA)

	Level Ice	Rafted Ice	Ridge Ice	Unconsolidated Ridge Keel
σ_c	3000	2500	1500	90
σ_f	800	700	400	-

ρ_i = 0.917 t/m^3 ρ_w = 1.025 t/m^3

E = 3000 MPa B = 33m

α = 45° hc = 5.4m

σ_c = Uniaxial compressive ice strength (crushing failure)

σ_f = Flexural ice strength established by downward breaking Cantilever beam tests. A value of 0.67 σ_f should be used for upward breaking conical structures as the warmer lower ice layer is weaker in resisting tension than the colder upper layer in contact with the air.

Ice ridge impact loads were calculated using equations 1, 2, 3 and 4 with the idealisation given in Figure 3 with h_K = 21.6m.

Outbreak loads were calculated assuming 4m of rafted ice surrounding the structure with no adfreeze bond and crushing of the level ice sheet and consolidated portion of ice ridges during outbreak using equation 4.

4.2 Winter Earthquake Loads

One of the critical load combinations to be considered is the occurrence of an earthquake during the frozen in condition. Because of tidal action and the effectiveness of surface coating and heating systems the ice is assumed not be be adfrozen to the structure. Rather it is assumed that cracks exist and that motion of the structure results in shearing of the ice with subsequent crushing depending on the motion time history. The ice is assumed not to be moved during the earthquake in sympathy with the sea bed.

Estimates of ice loads in these conditions are inevitably uncertain because the ice loading as the very high relative accelerations of the structure and ice during an earthquake. In this study strain rate dependency of ice properties have been conservatively ignored.

Consideration of the harmonic displscement of the structure, as a component of earthquake response, gives an acceleration opposed to the structure displacement and an inertia load in the same direction as the displacement. A straight-forward interpretation of this result would give an inertia load opposing the ice loads. The authors consider however that given the broad band nature of an earthquake spectrum advantage should not be taken of this apparent cancellation of loading and the inertia and ice outbreak loads (from Section 3.1) are considered conservatively to re-inforce one another.

The inertia loads were calculated and applied [2] with an effective horizontal ground acceleration, corresponding to a structure natural period of 1.85 seconds for the pile structure, of 0.15g (together with an acceleration of 2/3 the effective ground acceleration in an orthogonal horizontal direction). A vertical inertia load corresponding to half the effective ground acceleration was applied in the most unfavourable direction (upwards). The added mass or mass of water entrained with the moving structure was added to the structural mass before calculation of the inertia loads.

The soil conditions for both cases were assumed to be weak soils of class C. Liquefaction was assumed not to occur as it would probably result in a permanent tilt and loss of air gap. In these conditions the gravity structure would not be appropriate.

The load summaries for both piled and gravity structures are given in Tables 2 and 3.

5.0 PILED STEEL STRUCTURE

5.1 General Remarks

A piled steel jacket may be protected from ice by means of a conical shroud which extends from 6m above LAT to 7m below LAT. The structure could have 3, 4, 6 or 8 sides. For comparison, an 8 sided structure, the "Octagonal braced monopod" (Figure 1) was assumed. Each leg is supported by a small cluster of three piles. Less conservative soil data would enable the number of piles to be reduced. The conical ice shroud is supported off the plan bracings at ELs. +6m and -7m.

As the ice shroud does not extend down to mudline, the conductors and other appurtenances are protected from ice floes and ridges, without attracting the full added mass of the volume enclosed by an extended cone. Slots through the cone may be used as guides for pile installation. The pile guide slots through the cone would need to be sealed after pile installation. The cone is set at an angle of 60° to the horizontal. The possible ingress of unconsolidated or broken ice from ridge keels into the area beneath the cone is prevented by a grid of small diameter members.

The octagonal framed monopod does not directly suit installation by launching off a barge or by direct lifting. Launching off a barge would required the provision of two parallel launch legs. Direct lifting would require a substructure weight somewhat less than the 9000 tonnes of the concept substructure unless one of the largest of the present generation of SSCV's (McDermott DB102 or Micoperi M7000) were available. Therefore, it may be necessary to use temporary buoyancy to tow the structure in an upright orientation, or possibly to transport upright and launch from a launch barge.

Hydrostatic and stability calculations indicate that the structure can be floated out vertically by the attachment of temporary buoyancy chambers to the lower part of each of the eight legs. Cylindrical chambers 8m in diameter and 15m in length will give adequate buoyancy and stability for float out at a draught of 13.5 metres. These will be removed after installation and may be re-used or used for auxiliary storage, although they would then need to be designed against the hydrostatic pressure loads present on the seabed.

A conservative estimate for the very worst soil condition suggests that 24 piles 3,000mm O.D. x 80mm (or 2,438mm O.D. x 100mm) will be sufficient to withstand the envisaged loading possibly piling through guides in the ice-cone. If conditions allow, fewer oiles would allow the use of leg piles and piles interior to the cone perimeter.

A slightly modified configuration may be barge launched or even installed with a crane if the lift capacity is available.

5.2 Loads

Loads on the structure were calculated using the environmental conditions given in Section 3.1 and conservative assumptions where uncertainty in load calculations methods exist.

Ice ridge loads were calculated assuming an equivalent level ice thickness of 5.4m failing in bending together with the unconsolidated keel failing in shear.

Wave loads were evaluated for the design maximum wave height using the Morison equation which gives conservative loads as compared with a full diffraction analysis. As this loadcase does not design the structure this approach was chosen for simplicity.

Earthquake loads were calculated using the API response Spectrum approach [2] with an effective ground acceleration of 0.15g (g = acceleration due to gravity). During a major earthquake the structure was assumed to be surrounded by rafted ice of 4m thickness which fails in far field buckling. "Break-out" forces thus produced were applied simultaneously with the earthquake loads. For this structure the sea-bed was assumed to liquefy to a depth of 30m, giving no support to the piles above the clay layer. A load summary table is given in Table 2.

The earthquake and ice breakout loads were not the critical load case for this structure because of the reduced added mass resulting from the open lattice geometry below the ice sheet.

Structurally the platform was analysed by applying a load of 170 MN horizontally and 70 MN vertically at the still water level together with a vertical load of 300 MN applied at the MSF level to represent the superposition of topside and ice ridge loads.

Stresses were evaluated assuming a nominal wall thickness of 50mm for each brace. Resulting utilisation factors were found to be acceptable.

	VERTICAL LOAD (MN) (upward loads -ve)	BASE SHEAR (MN)	OTM (MN-m)
TOPSIDES DEAD WEIGHT BALLAST	398	-	-
BUOYANCY	-59	-	-
NETT STILL WATER	339	-	-
WIND, WAVE & CURRENT	-16	82	1734
ICE RIDGE	* 70	170	3230
EARTHQUAKE & ICE BREAKOUT	*- 29	104	4869

TABLE 2 DESIGN FORCES ON INTERNALLY BRACED MONOPOD

* Vertical loads are associated with the energy required to fail the ice in bending and the energy required to push the broken ice up the cone.

6.0 STEEL GRAVITY STRUCTURE

6.1 General Remarks

The steel gravity platform (Figure 2) was designed to withstand environmental ice and wave loads and to transport the topsides from the fabrication area, following deck mating, to the offshore installation site.

The chosen structure has a minimum diameter of 52 metres to ensure stability during deck mating and platform installation without the need for temporary buoyancy.

6.2 Loads

Ice loads were calculated assuming ice ridge loading, break out from the adfreeze condition and assuming far field buckling of the ice. Ice ridge loading was evaluated using Ralston's plastic limit state method [9]. Maximum adfreeze loads were calculated considering 4m thick rafted ice.

Wave loads were evaluated for the design maximum wave height of 12 metres associated with a period of 10 to 12 seconds. The wave length was found to be 150m and a vertical force was calculated directly from the dynamic pressure field.

Earthquake loads were calculated assuming a total horizontal ground acceleration of 0.18g and a vertical acceleration of 0.075g. Ship and ice floe collision loads were also calculated. A load summary table is given in Table 4.

Offshore structures are commonly checked for accidental impacts by supply boats at up to 2 m/s. Assuming an added mass factor of 1.4 for broadside impact, an accidental broadside impact by a 3,000 tonne supply boat would have an impact energy of 8.4 MJ. This energy will be absorbed by denting, bending and distortion of the structure, and damage to the boat hitting the structure. The energy absorbed by the boat may be estimated by integrating the force-indentation curves presented by DnV [12]. If the structure is stiff, so that most of the energy is absorbed by the boat, then the DnV curves for broadside impact predict a maximum force of 19.4 MN to absorb the 8.4 MJ of energy. For global design, 19.4 MN is much less than ice, storm or earthquake loads (See Table 4).

6.3 Weights

During transportation, the structure is designed to have the following characteristics:-

i) 20,000 tonnes of dry topsides
ii) a maximum draught at float-out of 7m to comply with fabrication yard requirements
iii) a vertical centre of gravity of dry topsides above the base of 70 metres.

In addition to the basic steel weight of the structure, estimated to be approximately 35,000 tonnes, allowances were made for items such as Mechanical and Electrical Systems (1,000 tonnes), conductor frames (300 tonnes), appurtenances (1,000 tonnes), and anodes (2,500 tonnes). Hence the total weight of the structure, including basic steel weight, was estimated to be about 40,000 tonnes.

6.4 Transportation and Installation

The proposed transportation and installation sequence for the steel gravity platform is as follows:-

i) Float out the steel gravity substructure from a dry dock, at a draught of 7.0m, utilising compressed air in the skirts.

ii) At a sheltered deep water site, ballast the structure with seawater to 41m draught and mate the topside structure.

iii) Deballast the platform to a draught of 9.5m and tow to site.

iv) Install the platform at the location using seawater ballast.

v) Add the solid ballast required for stability and grout below the base of the structure.

For initial stability, the heel righting moments and wind overturning moments were calculated for three conditions:-

i) Towout
ii) Installation
iii) Deck mating

For deck mating, one solution is to use a pair of launch barges. The deck mating takes place by de-ballasting the steel gravity structure. In order to obtain satisfactory stability at the deeper draughts solid ballast of 6,700 tonnes is placed in the base of the fabrication site giving a VCG (vertical centre of gravity) of 27.8m above the base and a dry weight of 84,400 tonnes total.

Table 3a gives the atability parameters for the structure for all relevant draughts during deck mating, transportation and installation.

Draught (m)	Buoyancy (t)	Water Ballast (t)	VCG (m)	KM (m)	GM (m)
9.5	84420	0	27.8	79.9	52.1
30.0	218433	134013	17.0	17.0	+1.3
36.0	234846	150326	16.1	16.1	+0.4
41.0	245714	161294	15.8	17.0	+1.2

TABLE 3a INITIAL STABILITY (6,700t SOLID BALLAST)

GM is the metacentric height
KM is the elevation of the metacentre above the keel

Table 3b shows the variation of righting and wind overturning moment at the deck mating draught of 41m. Similar dynamic stability curves were generated for towing draught (9.5m) and installation draught (30m).

Heel Angle (deg)	Righting Moment (T.m.)	Wind Moment (T.m.)
0.0	0.0	383.0
1.0	6844.3	384.0
5.0	34297.6	381.0
10.0	69082.7	373.0
15.0	103360.7	361.0
20.0	132871.6	347.0

TABLE 3b DECK MATING - 41.0m DRAUGHT

6.5 Structural Analysis

A finite element structural analysis was performed for the steel gravity structure. The finite element mesh consisted of 416 semi-loof elements, each one had a nominal thickness, defined as that thickness which gives the same section properties as the actual stiffened plate.

For the analysis, the gravity structure was restrained laterally at all nodes in the inner and outer rings of the base.

For the part of the structure, near the mean ice level, the maximum nominal stress was found to be 18 N/mm^2, indicating satisfactory strength with local stress concentration of up to 10.

Plate buckling checks and local stress checks have also b een performed.

A load summary is given in Table 4.

	VERTICAL LOAD (MN) (upward loads -ve)	BASE SHEAR (MN)	OTM (MN-m)
TOPSIDES		-	-
DEAD WEIGHT	441	-	-
BALLAST		-	-
BUOYANCY	-243	-	-
NETT STILL WATER	198	-	-
WIND, WAVE & CURRENT	- 68.5	150	3094
ICE RIDGE	70	170	3230
EARTHQUAKE & ICE BREAKOUT	- 46	163	6357

TABLE 4 DESIGN FORCES ON GRAVITY BASE MONOPOD

7.0 COMPARISON AND CONCLUSIONS

In view of the foregoing the piled steel structure has the following important advantages over the gravity based structure:-

i) The foundation loads are reduced by 50% in earthquakes because of the reduced mass and added mass.

ii) The steel weight is considerably reduced.

iii) The piled structure can resist foundation liquefaction.

iv) Welded tubular construction is well understood in terms of its long term fatigue behaviour and stress concentration characteristics.

v) The internal bracing amy be easily extended downwards to provide designs for structures in deeper waters without disproportionate increases of steel weight.

The octagonal internally braced monopod has a jacket weight of 9,000 tonnes, a deck weight of 2,600 tonnes, and a piling weight of 7,000 tonnes. Assuming total fabrication costs of $4,500/tonne for jacket and deck and $1,500/tonne for piling, the fabrication cost of the octagonal framed substructure is estimated as $62.7 million. By comparison, the 42,000 tonne steel gravity structure has an estimated fabrication cost of $168 million assuming a slightly lower fabrication rate of $4,000/tonne.

Installation costs for the steel gravity structure are estimated to be of the order of $5 million as opposed to the order of $20 million for the octagonal internally braced monopod.

Therefore total fabrication and installation costs are of the order of $83 million for the octagonal framed monopod, and $173 million for the steel gravity structure. Therefore, the octagonal internally braced monopod will show a saving in overall costs so long as the additional hook-up costs do not exceed $90 million.

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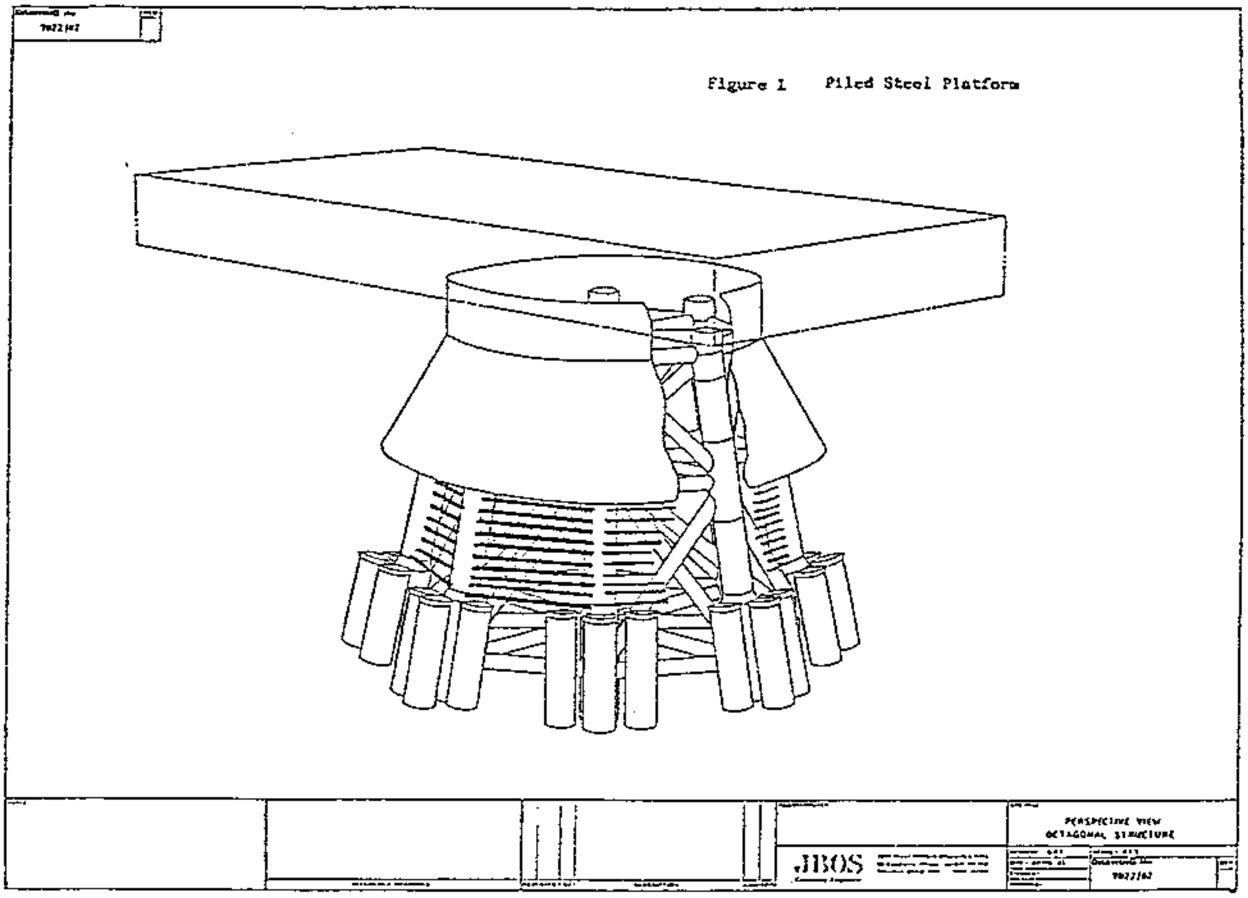

Figure 1 Piled Steel Platform

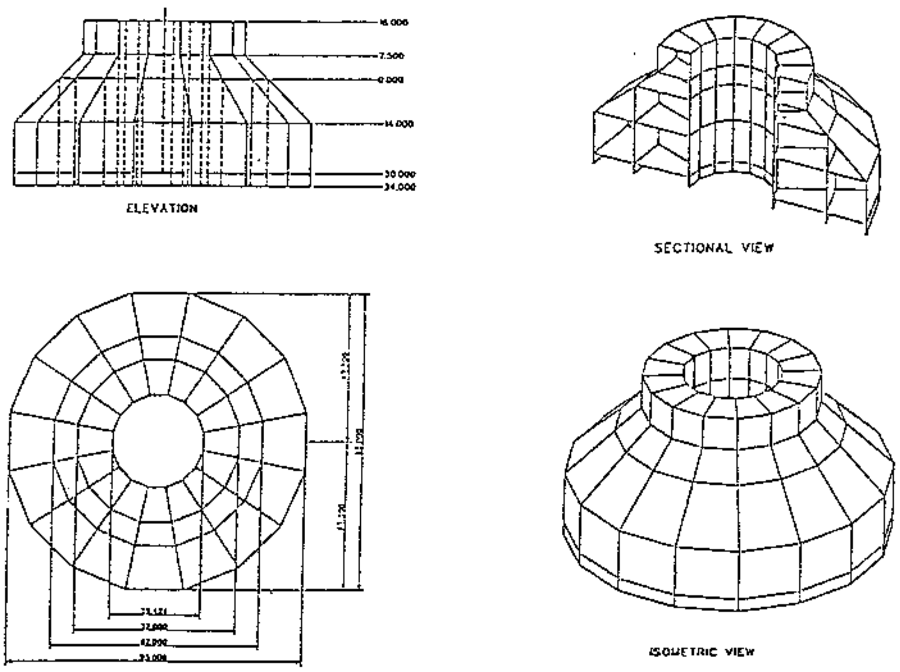

FIGURE 2.

STEEL GRAVITY STRUCTURE

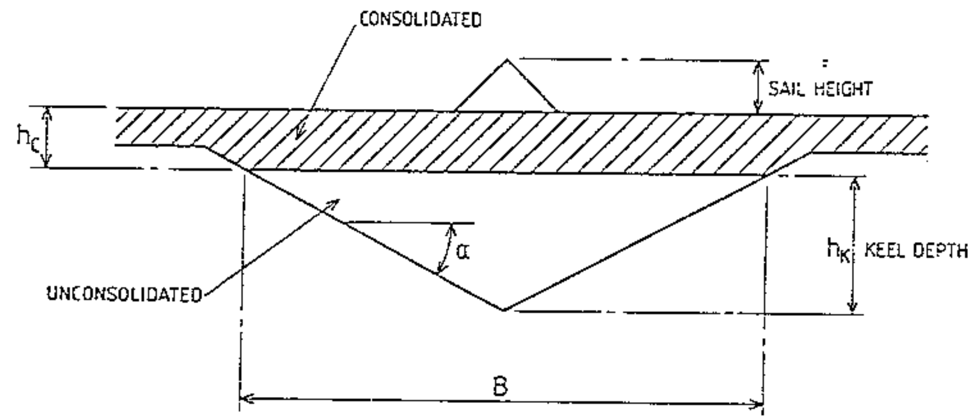

Fig. 3 Idealised Ice Ridge Section