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Offshore and Marine Concrete
Structures: Past, Present, and Future

SP-337

Editor:
Mohammad S. Khan



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Offshore and Marine Concrete Structures: Past, Present, and Future

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PREFACE

Offshore and marine concrete structures have not received enough attention in the recent past, at least in the United States. The complexity and safety concerns associated with these structures are such that they probably need more attention compared to many other types of concrete structures. Also, offshore and marine concrete structures are so global in nature that there is a higher need for better coordination and synchronization of design, construction, inspection, and maintenance practices in different parts of the world.

A two-part session, titled “Offshore and Marine Concrete Structures: Past, Present, and Future,” was held at the Spring 2019 ACI Concrete Convention and Exposition on March 24-28 in Quebec City, Quebec, Canada. The session, sponsored by ACI Committee 357, Offshore and Marine Concrete Structures, highlighted accomplishments of the past, current state-of-the-practice, and a path for the future. This ACI Special Publication (SP) is a compilation of select papers presented at the session. The efforts of all the reviewers in assuring the quality of this publication is greatly acknowledged.

Mohammad S. Khan, Ph.D. P.E.
Editor

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Design and Construction Overview of Offshore Concrete Gravity-Based-Structures: Past, Present, and Future

Widianto, Jameel Khalifa, Erik Åldstedt, Kåre O. Hæreid, Kjell Tore Fosså

Synopsis: An offshore concrete Gravity-Based-Structure (GBS) is a massive concrete structure placed on the seafloor and held in place strictly by its own weight, without need for anchors. This paper focuses on concrete GBSs used as the base of integrated oil drilling and production platforms. The summary of key distinct structural features of several major GBSs, since the first Ekofisk GBS (installed in the North Sea, offshore Norway, in 1973) until the latest Hebron GBS (installed in the Grand Banks, Canada, in 2017), is presented. This paper also discusses several unique loads that GBSs have to resist. An overview of structural analysis and design methodology is described in detail. Key considerations for preliminary sizing of GBS structural components are presented. Typical construction phases, methods, and the importance of constructability are explained. Finally, potential future research topics that would result in a more cost-effective offshore concrete GBS are discussed.

Keywords: Floating Construction, Reinforced Concrete Design, D-regions, Structural Analysis, Offshore Structures, Strut-and-tie, Wave loads

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INTRODUCTION

An offshore concrete Gravity Based Structure (GBS) is a massive concrete structure placed on the seafloor and held in place strictly by its own weight, without need for piles or anchors. This paper focuses on concrete GBSs used as the base of integrated oil drilling and production platforms (topsides). The topsides typically includes drilling, production, processing, and accommodation facilities. Table 1 provides a summary of key distinct structural features of several major GBSs, starting with the first, Ekofisk GBS (installed in the North Sea, offshore Norway, in 1973), until the latest Hebron GBS (installed in the Grand Banks, Canada, in 2017). Several major GBSs are also shown in Figure 1.

General Characteristics

In contrast to typical buildings and bridges, there are several distinct features of offshore GBSs that pose unique challenges in the structural analysis, design and construction:

- They are typically massive in size, have a complex geometry and contain a significant number of "Disturbed" or D-regions. Typical examples of D-regions are locations with abrupt changes in dimensions (change in thickness or an opening in a wall), locations where concentrated loads are applied (such as post-tensioning anchorages), or deep beams where simple beam theory does not apply.
- The structural elements are heavily loaded and highly reinforced, with average reinforcement density of over 300 kg/m³ (19 lb/ft³) compared to 75-150 kg/m³ (5-9 lb/ft³) for typical buildings and bridges. This is partly because the member thicknesses have to be limited to minimize GBS weight to allow it to float during the various construction and installation phases.
- They contain many access openings and pipe penetrations that require additional reinforcement around them to replace the reinforcement interrupted by these openings. This further increases the local reinforcement density that can be as high as 600 kg/m³ (37 lb/ft³).
- They are typically located in harsh climates leading to enormous wave loads (>1,000 MN (225,000 kip)) and sometimes ice/iceberg impact loads (>500 MN (112,000 kip)).
- Some GBSs also have a requirement to store oil, hence be leak-tight. This subjects many structural members to large differential pressures and temperatures that requires post-tensioning and other

measures to prevent leakage, such as special reinforcement detailing to minimize cracking and crack-widths.

- To minimize expensive construction offshore, GBS construction is initiated in a dry dock. The partially built GBS is then moved to a nearshore location with adequate water depth, where the construction is completed. As a result, a GBS has various floating stages during construction as well as various complex marine operations to allow transport to, and installation at, the field location.

Furthermore, each GBS is unique with regard to its location, water depth, soil characteristics, environmental conditions, functional requirements, topsides weight and hydrocarbon processing requirements. As such, GBSs generally present some design challenges that may not have been addressed in the past, requiring new design techniques in some instances.

LOADS

Following are several unique loads (compared to typical buildings/bridges) that GBSs have to resist:

- **Hydrostatic loads** – A significant portion of a typical GBS is constructed while it is floating, which leads to hydrostatic loads acting on the structure both from the outside as well as ballasting loads internally. These loads can be extremely high, such as when a GBS is submerged during mating the GBS with the topsides (as much as 100 m (330 ft) of water head).
- **Hydrodynamic loads** – Wave action can result in very large base shear and overturning moments on the GBS as well as high local pressures on the shafts. These forces and pressures are generally estimated by analytical methods (such as Morison equation or Computational Fluid Dynamic (CFD)) as well as model tests in a wave basin. Model tests are particularly important when the GBS geometry is unusual. For example, several model tests were conducted for the Hebron GBS since the caisson was submerged with relatively shallow water depth above it (Oberlies et. al, 2014). This caused the waves to accelerate and break above the caisson and impact the shaft with much higher pressure than expected, see Figure 2. The waves also run up the shaft impacting the underside of the topsides. The large shaft diameter caused the waves to split around both sides of the shaft and collide at the back of the shaft, creating an upward jet (rooster tail effect) that impacted the underside of the topsides (Widianto et. al, 2018). Such phenomena can be better approximated by model testing.
- **Ice/Iceberg loads** – GBSs located in the arctic and sub-arctic regions can be subjected to large loads from sea ice or icebergs, see Fig. 3. For example, at the Hebron GBS location in the North Atlantic, historical data indicates presence of over 2,000 icebergs annually for some years. The load imparted to the structure by icebergs depends on various parameters: ice crushing strength, drift speed, shape and draft. Such loads for the Hebron GBS were developed using state-of-the art probabilistic analysis that satisfied ISO and covered a range of expert opinions for Type II uncertainty. Iceberg loads were developed for two return periods: 100-year and 10,000-year per ISO requirements (Widianto et. al, 2013).
- **Topsides loads** – The primary function of the GBS is to support the topsides structure. Topsides can weigh as much as 65,000 tonnes (71,650 tons) with few support points that impose very large unbalanced reaction forces on the GBS shaft (~20,000 tonnes (22,000 tons) for Hebron GBS).
- **Oil storage loads** – Since the density of oil is less than water, cell storage walls and roof are subjected to large pressure differential. Stored oil is generally hot (~50°C) to allow it to flow. This is significantly higher than the surrounding sea water temperature, causing thermal gradients that introduce large thermal strains/forces in the structure. Various combinations of water and oil storage in the cells need to be considered in developing the loads due to oil storage.
- **Impact loads** – Supply barges and vessels may impact the outer parts of the GBS applying highly concentrated local loads to the walls and slabs. Such impact loads may occur either during construction or in operation. Furthermore, drill bits and conductor pipes may fall and hit the internal pipes carrying oil and water during the operation phase. Protective structures are usually provided to prevent these pipes from damage.
- **Miscellaneous loads** – These include loads on mooring/towing brackets (during various floating stages and marine operations) and loads on embedment plates supporting various mechanical outfitting. Moreover, where pipes penetrate the concrete walls, anchor loads are generated due to restrained thermal expansion/contraction as well as the interaction between pipes embedded in the solid ballast and concrete walls during various construction and operation phases.

STRUCTURAL ANALYSIS AND DESIGN

Generally, the first step in the analysis and design of a GBS is to develop the overall geometry. In order to assure a robust and cost efficient concept for an offshore GBS, preliminary field data and functional requirements are processed in iterative steps until a satisfactory concept is found, as shown in Figure 4. Subsequently, structural analysis is performed on the chosen concept to determine internal forces. Finally, structural design is performed to ensure that all elements can resist these internal forces for the different limit states per applicable design codes and standards.

Overall Geometry

As shown in Figure 1, a GBS typically consists of a lower caisson built up by a number of circular cells, straight walls, or by a combination of cells and walls. One or more circular cylinders are extended above the caisson to a safe level above the water level forming the shafts. The bottom and top of the caisson is usually closed by domes or by slabs.

Following are some key considerations for sizing the GBS (i.e., developing an overall geometry):

- Soil properties: Impacts the foundation system and dimensions
- Oil storage, if relevant: Governs the caisson volume/size
- Topside weight during tow-out and in operation: Governs the dimensions to satisfy floating and geotechnical stability
- Water depth at site: Governs the height of the GBS
- Water depth during tow out: Governs the caisson dimensions to satisfy floating stability. The governing condition is typically the shallowest water depth in the tow route, in which the GBS will need to be de-ballasted to meet the required under keel clearance requirement.
- Dynamics (Eigenvalue): Governs slenderness, etc.
- Execution / construction technique: Regular, symmetrical structures that can be slipformed are preferred
- Material properties: Affect the dimensions and the execution process.

The following sections provide more details on proportioning specific elements of the GBS.

Caissons

The overall geometry of the caisson cells (including diameter, number of cells, and height) is typically chosen to enable a sound support system for the topsides structure, to fit the required space for all mechanical outfitting in the shafts, and to fulfil requirements for floating stability, geotechnical stability, and oil storage volume (if any). Sometimes, when the GBS is to be installed in deep water requiring long shafts, the diameter of the caisson has to be increased to provide a sound base for the shaft structure.

The high differential pressure during deck mating phase in combination with constructability consideration determines the thickness of the cell walls, which is typically kept constant over the cell height to simplify slip-forming. For outer cell walls, second order effects normally govern the thickness. Some of the shaft cells may require extra thickening locally to transfer shaft loads to the caisson and enable proper anchoring of vertical post-tensioning cables from the shafts.

For many GBSs with inshore deck mating, one-cell damage stability is generally required. This implies that the platform shall remain stable if one watertight compartment is subjected to uncontrolled flooding. The metacentric height must be large enough to prevent excessive inclination of the GBS when subjected to environmental loading in the accidental situation. This requirement led to the selection of 30 cells of smaller diameter for Oseberg A caisson. By increasing the number of cells, the volume of a potentially flooded cell was reduced and the structure maintained necessary buoyancy while afloat. It also had a large enough base area for geotechnical stability.

Skirts/Foundation

The caisson typically rests on skirts, which are the walls extended below the base and embedded in the seafloor. For some structures, steel has been used for these skirts.

GBSs located on sand typically have short skirts, which prevent scour due to current and provide geotechnical stability. GBSs resting on soft clay typically require long skirts that penetrate the clay to an adequate level to provide the required geotechnical stability. The first GBS utilizing long skirts was Gullfaks C (foot-print area = 16,200 m² (174,000 ft²); cell diameter = 28 m (92 ft)) with skirts penetrating 21 m (69 ft) below the seabed. Subsequently, Troll A was designed with skirts penetrating the soil to an even greater depth of 36 m (120 ft).

To resist global overturning due to environmental loading, a GBS resting on sand must have a large enough base area and sufficient self-weight to avoid tension at the “heel” (least compressed area) of the base. Whereas, GBSs installed on clay can utilize tension at the “heel” of the base (that can be resisted by the long skirts), thereby reducing the size of the base. If floating stability requirements do not govern, utilizing tension at “the heel” is an efficient method for reducing the caisson size.

Domes

Historically, domes have been used as the base and top of the cells as this is an optimal shape to resist hydrostatic pressures – internal forces are mainly compression, which concrete is very good at resisting. For some GBSs, slabs have been used instead of domes as they are easier to construct even though they have a lower capacity and require more reinforcement to resist the bending and shear forces. Slabs may be chosen when the hydrostatic differential pressures are low (50 m (160 ft) or less water head). The distance between the caisson walls or the dome diameter also influences this decision.

The radius of the domes should be large enough to allow the main reinforcement to be placed without pre-bending and require minimal need for formwork on the upper face, both for lower and upper domes. The lower domes are normally exposed to higher loading than the upper ones. If resting on sand, the lower dome is loaded by both water pressure and loading from the sand (vertical and horizontal) since the void between the lower domes and the soil is filled with grout.

Loading during deck-mating normally governs the thickness of the domes. By choosing a ratio of the radius of cell to the radius of dome ($R_{\text{cell}}/R_{\text{dome}}$) close to 0.8, most of the requirements can generally be fulfilled. For example, this ratio for Gullfaks C is ~0.8 for both the lower ($14\text{m}/17.6\text{m} = 46\text{ft}/57.7\text{ft}$) and the upper ($14\text{m}/17.5\text{m} = 46\text{ft}/57.4\text{ft}$) domes. Similarly, for Statfjord C, this ratio is 0.81 ($11.5\text{m}/14.2\text{m} = 37.7\text{ft}/46.6\text{ft}$) for the lower domes and 0.72 ($11.5\text{m}/15.9\text{m} = 37.7\text{ft}/52.2\text{ft}$) for the upper domes. Reducing the R_{dome} will create constructability issues and increasing R_{dome} will increase loading from the dome to the ring beam, which typically needs to be post-tensioned to resist the dome compressive forces.

Shafts

The upper part of the shaft is sized to support the topsides structure and accommodate all the mechanical outfitting within the GBS. The diameter of the shafts penetrating the water surface is minimized to reduce the wave loading while still providing a sufficient space for all mechanical outfitting. The thickness of the lower part of a shaft is normally governed by a combination of self-weight, environmental loading or deck mating.

While most shafts during operation are filled with sea-water and connected to the sea, the shaft is typically kept empty when it contains gas exporting piping, in order to avoid risk of explosion in case of a gas leak and pressure build-up.

Structural Analysis

The Ekofisk tank structure (installed in 1973) was the first offshore concrete GBS designed to store crude oil. The structural analysis of the tank walls and outer perforated walls was carried out on main-frame computers using a combination of in-plane frame and space frame models. The foundation structure, composed of crossing beams and upper and lower slabs, was analysed using a frame-model computer program and yield line theory, respectively. The verification work of the Ekofisk tank structure was based on the Finite Element Method (FEM), which was being developed at the Norwegian University of Science and Technology, University of California at Berkeley and University of Stuttgart at that time.

Since then, the analysis approach has evolved and it has been common practice to perform a global finite element analysis (FEA) of the structure to determine internal forces. Specialized or local analyses are also conducted to study structural elements of special interest or subjected to extreme loads. These are described further in the following sections.

Global Analysis

Global FEAs determine the internal strains and forces (in-plane axial and shear forces, torsion and bending moments, out-of-plane shear forces) throughout the structure when subjected to various loads and boundary conditions, see Figure 5. The global analyses account for various construction phases including floating phases as well as operation and eventual decommissioning and removal phases. For the operation phase when the GBS is resting on the seafloor, the soil response is included as soil reaction forces.

Some construction phases, e.g. the out-of-dock phase when the structure is incomplete, have often been analyzed as separate FEA models of only the partially-built parts. However, for the most recent Hebron project, the global FEA model was used for the out-of-dock phase as well with the stiffness and mass of the parts not yet built reduced to essentially zero.

Linear elastic material properties have typically been used as a basis for global analyses, allowing the use of superposition. As a result, analyses of a limited number of individual load cases can be performed, which can then be combined together to develop the large number of load combinations required for checking the various limit-states. The member stiffness is based on un-cracked concrete section properties. Some non-linear behavior due to cracking or other effects may be accounted for by reducing the stiffness of selected members. Specialized non-linear analyses may be used in some instances and are described in a subsequent section.

Shell and solid finite elements have typically been used in these global analyses with varying degrees of freedom (DOFs) as shown in Table 2. Simplified beam or shell element models have often been used for dynamic analyses to reduce the DOFs. In the 1990s (for example, Troll A project), input data were prepared by the design team, whereas the time-consuming solving of the large FEA equilibrium equations was performed elsewhere on main-frame supercomputers. Thereafter, the post-processing and design work were carried out on Personal Computers (PCs) by the design team. Today, the whole analysis and design process is automated and can be carried out on standard laptops.

Generally, the size of the global models used in analysis have steadily increased over time, primarily due to advancement in the development of computers through the years. Table 2 indicates that the DOFs generally also increase with concrete volume. It is important to note however, that the number of DOFs required for an accurate analysis is also dependent on the geometry, height/width ratio of the GBS, and member thickness. While the 4-node shell and 8-node solid elements are relatively simple first order finite elements, their accuracy has been improved over time by various condensation and integration techniques. Furthermore, the enormous increase of computing power at relatively low cost, has made possible the use of higher order element types and the very large number of DOFs used in such analyses these days.

Local Analysis

A local analysis using a more refined finite element mesh with a more accurate material model may be required for certain local areas and/or load conditions to obtain more realistic internal forces. Some FEA programs provide sub-modelling techniques which can be useful to refine the mesh in local areas. Local analysis methods may include non-linear finite element analysis techniques, cut out models, and strut-and-tie models (see section on strut and tie models below).

Some examples of local analysis include topsides-GBS connections, connections between various elements of GBS (D-region, such as between walls, wall and base, etc.), pipe penetrations and other items identified in the Miscellaneous Design section.

Non-Linear Analysis

Material Non-Linearity

Nonlinear material behavior is primarily caused by cracking of the concrete in tension, nonlinear stress-strain relationship of concrete in compression and yielding of reinforcement. Prior to about 1990s, almost no nonlinear analyses were performed. Since 1990s however, non-linear analyses have been used to an increasing extent for vital parts of many GBSs, especially under extreme loads. Examples of analyses using material non-linearity performed in the Hebron GBS are described in Widiyanto et al., 2020.

Geometric non-linearity – Implosion and second order effects

Enormous hydrostatic pressures typically produce high compressive forces, which may result in large $P-\Delta$ effects that could lead to a catastrophic failure of the walls due to implosion. Potential failure modes include material failure and structural instability. The stability is checked on the basis of a rational analysis of the behavior of the structure, including the influence of loads and second order effects produced by deformations. The latter is evaluated by taking into account possible cracking, the effect of the reinforcement on the rigidity of the member, creep effects, and the effects of possible geometrical imperfections, including out-of-roundness due to construction tolerances. Since the forces due to prestressing reinforcement do not contribute to the second order moments due to “internal equilibrium”, these effects are disregarded. The creep values are generally small due to the damp environment.

Implosion checks may be performed in various ways, using simplified or refined shell analyses, out-of-roundness data based on codes or previous experience, and simplified code-based or accurate cross-sectional capacities for concrete and reinforcement. In general, the computer programs discussed below can be used for refined implosion checks.

Computer programs

In addition to linear structural FEAs, many FEA programs can handle non-linear structural analyses, offering various concrete element types simulating concrete cracking and steel reinforcement modelling including yielding. Several examples of finite element programs for nonlinear analyses are ABAQUS, ADINA, ANSYS, DIANA, LS-DYNA and SOFiSTiK. ABAQUS and ANSYS have frequently been used for FEA of offshore concrete structures.

ABAQUS:

The nonlinear material curve for concrete and the reinforcement are provided as input to the analysis model. The steel reinforcement is specified as layers in the different directions and at correct locations within the concrete cross-sections. ABAQUS offers two different material models for concrete: “The Concrete Smeared Cracking model” in ABAQUS Implicit and the “The Brittle Cracking model” in ABAQUS Explicit. The former has been used with great success for shell finite elements.

ANSYS:

The material model may be independent of the finite element itself. Thereby, the same material model can be used for various types of elements. The shape functions of the elements may be of the first or second order. The reinforcement is introduced as separate elements, as single rebars or as smeared reinforcement. In general, it is favorable to use elements with first order shape functions for non-linear analyses. The use of a fine mesh with regular elements is important to avoid convergence problems.

Design

Design Standards

At the start of the design process of the Ekofisk platform (the first major GBS) the basis for the design was not completely agreed upon by the design team. Consequently, there were many discussions on various design provisions, such as transverse shear capacity in slabs with water pressure in flexural cracks and leak-tightness criteria in various design phases. The leak-tightness criteria in the serviceability limit state (SLS) used in the design of this GBS ended up in the so-called “Oslo Criteria”, which later on was included in the 1974 edition of the “DNV Rules for the design, construction and inspection of fixed offshore structures.”

Since 2006, the structural analyses and design of GBSs, to a large extent, have been based on the ISO standard “Fixed concrete offshore structures, ISO 19903:2006”. Nevertheless, the principles of this ISO standard were adopted in design of GBSs even before 2006, e.g. Troll A (1990 - 1995). The Norwegian concrete design code, NS 3473.E:2003, has also frequently been used in design of many GBSs. These standards define limit states and load combinations, including load factors for concrete structures.

Limit states

Typical limit states are as follows:

- Ultimate Limit State (ULS): Factored loads are checked against ultimate capacities including the resistance factor
- Serviceability Limit State (SLS): Unfactored loads are used to calculate and limit crack-widths for durability, depth of cracking for leak-tightness, deformation/deflection limits, and vibration limits.
- Fatigue Limit State (FLS): Considers the effect of cyclic loading due to wave action and oil filling cycles
- Accidental Limit State (ALS): Unfactored loads due to extreme loads are checked against member resistance to ensure life-safety and prevent release of hydrocarbons.

General design

General design follows the common rules for design of beams, plates and shells in the following order:

- The results of the global FEA are integrated into the eight stress resultants or internal forces shown in Figure 5
- Reinforcement quantities are calculated to resist these internal forces (ULS) for the given element thickness accounting for the non-linear behavior of reinforced concrete
- The capacity of the concrete cross-sections are checked
- Various other limits states are checked and reinforcement increased if required
- Drawings are produced indicating optimal arrangement of reinforcement

This effort can be extremely time consuming for the very large number of load combinations that need to be checked at a vast number of design points. As such, several post-processing and design computer programs have been developed since the 1970s. Normally, these programs are linked to a commercial FEA program to automate the overall process.

In the early design programs, tensile reinforcement design was based on the sum of the tensile stresses in uncracked concrete sections, while separate compression checks were carried out for concrete. In the subsequent versions of programs, design forces and moments have been based on cracked sections, i.e. non-linear material properties, allowing for equilibrium iterations and compatibility between strains and curvatures.

Transverse shear forces are typically not included in the equilibrium iterations, but treated separately using a simple beam shear design approach.

Water pressure may intrude into cracks when the GBS is submerged causing additional tensile forces in the cracked member and requiring more reinforcement. Water pressure in cracks reduces the beneficial compression from the corresponding external water pressure applied at the outer face of the structure. Such water pressure in cracks is accounted for by adding a tensile force in the design checks for in-plane stresses, transverse shear, crack width, and leak-tightness calculations.

Imposed deformations may be due to temperature, creep, shrinkage and differential settlements. In a GBS with oil storage, the thermal effects are significant hence cannot be ignored. Currently, they are treated as strains, not forces. These thermal strains are added to the calculated strains due to imposed loads to perform the various limit-states checks. While thermal strains have limited impact on ULS capacity, they can significantly impact the crack-width calculations and leak-tightness checks. If the total membrane force becomes tension, the imposed deformations also reduces the transverse shear capacity.

The presentation of the results from the post-processing and design programs is usually tailor-made to facilitate their use in the design work and drawing production. The results are presented as tables, diagrams and plots, such as:

- Stress-strain diagrams for concrete and steel reinforcement
- Tables and diagrams for forces and moments
- Details for the design of one section (point) in the structure
- Utilization Ratios (URs) for concrete and reinforcement
- Governing loads and limit states
- Reinforcement charts, which may be used as a basis for reinforcement drawing production

Miscellaneous design

Miscellaneous design typically includes following items:

- Pipe penetrations in walls and slabs
- Support for mechanical outfitting and embedment plates
- Towing brackets and mooring supports
- Construction crane support
- Impact loading from barges and other vessels
- Accidental conditions such as explosions and impact from dropped objects

Local FEA models, hand-calculations, spreadsheets, tailor-made programs, are generally used for designing such items.

The design of walls at pipe penetrations is normally carried out by strut-and-tie models, stress field models or based on classical solutions shown in handbooks. Note that the design of reinforcement at a circular opening in a wall may be quite complicated when the wall is subjected to in-plane forces combined with out-of-plane shear forces and bending moments.

For dropped objects it is normally necessary to provide protection covers above pipes, cables, etc. The design for impact loading from barges and vessels normally imply a check of concrete dimensions while reinforcement amounts are governed by other loads.

Strut-and-tie and stress field design models

GBS elements can be divided into beam/slab-like regions (B-regions) and disturbed/discontinuity regions (D-regions). As described previously in the General Design section, B-regions are designed by rules for beams and slabs, for which the strains are assumed to be linear across the depth of the member.

D-regions are found adjacent to abrupt changes in loading at concentrated loads and reactions, or adjacent to abrupt changes in geometry, such as holes or changes in cross-section. In the D-regions, the strain distributions are not linear, and typical sectional analysis would not apply or provide safe designs. In such cases, strut-and-tie models representing local stress fields (both tension and compression) may be more appropriate, see Figure 6. This approach also allows a rational method to determine reinforcement detailing including anchorage requirements.

MECHANICAL OUTFITTING

In order to meet all functional requirements, concrete offshore structures also contain of a significant number of mechanical systems, which include piping, equipment, mechanical and electrical systems, and structures required during both the temporary and permanent phases of the GBS and its future removal (decommissioning), see Figure 7. These systems have a significant impact on the GBS design and construction as the weight of mechanical outfitting can be about 20% of the weight of the topsides structure.

From a design perspective, mechanical systems require many penetrations through concrete walls. The area around the penetration has a significantly higher amount of reinforcement, due to “cut-and-replaced” reinforcement. This increased reinforcement, while local, could reduce the overall slipforming rate thereby increasing risk of concrete surface defects. In addition, special consideration is required for ensuring leak-tightness at penetrations.

From a construction perspective, the sequence of mechanical outfitting systems installation (including commissioning and hydro-testing) must be coordinated with concrete placement. Concurrent construction would require improved planning and worker management.

CONSTRUCTION**Construction Phases**

As mentioned earlier, a GBS is typically constructed away from the permanent installed location in a protected location or near shore. Typical construction phases include:

- Construction in a dry-dock
- Construction while floating at an inshore location (deep water site)
- Transportation or towing the GBS from the dry dock to the deep water site
- Mating the GBS with the topsides and tow to the offshore field location
- Installation of the structure at its offshore location by ballasting
- Removal after completion of operating period to either another site or for demolition

Figure 8 shows the various construction phases of Hebron platform:

1. Completion of base slab reinforcement in the dry-dock
2. Completion of the GBS structure to a height of 27.5 m (90 ft) built in the dry-dock (picture taken prior to filling the dry-dock)
3. Tow from the dry-dock site to the deep water site (about 3 km (1.9 mi) tow)
4. Construction at the deep water site
5. Completion of GBS construction at the deep water construction site
6. Topsides-GBS mating – the GBS submerged to 6m freeboard and the topsides floated by 2 barges to the GBS location, subsequently the GBS was de-ballasted to carry the topsides
7. The completed platform ready to be towed to the final installation location – tug configuration different at various towing stages
8. Platform installed at the final location

Construction Method

The most common construction method used on concrete offshore GBSs is slipforming, in which the formwork panels are continuously raised by hydraulic pumps and yokes. This approach allows uninterrupted concrete placement, reinforcing bar installation, and minor surface repair. Slipforming allows walls to be constructed cost effectively, minimizing construction schedule and improving leak-tightness as most construction joints are eliminated. A typical slipform assembly is shown in the Figure 9. The largest slipforming operation in history is believed to have been carried out for the Gullfaks C GBS at the deep water site (56 m (184 ft) high walls of 24 cells, each with ~28 m (~92 ft) inner diameter), incorporating approximately 24,000 tonnes (26,500 tons) of reinforcing bar and about 114,000 m³ (149,000 yd³) of concrete over a 42-day period.

For the early platforms, the slipform panels were made of wood, but these were changed to steel panels by 1980. In the beginning, concrete was distributed by using crane and bucket, but now concrete is typically pumped to the slipform.

Slipforming relies on many interdependent items: a suitable concrete mix with predictable properties, a distribution system, and a strong slipform assembly. Slipforming requires detailed planning with regard to the execution, manpower and logistics. When started, the slipform operation is a continuous operation that may last for several days or weeks. Multiple work shifts are required so that the work can continue uninterrupted day and night. ACI 347 provides some guidelines on slipforming.

The slipform wall panels are held together by a steel yoke structure, see Figure 9. Working decks for concrete, reinforcement and installation of block-outs are installed at the top of the slipform panel. A lower deck is installed under the working deck for inspection of the slipformed concrete wall. The necessary follow-up work and curing measures are carried out from the lower deck. Additional decks and suspended scaffolds may also be installed depending on the size and complexity of the slip.

For horizontal slabs or domes on an offshore structure, more traditional construction methods are used with concrete placement from crane and bucket or directly pumped to the area. A robust formwork at the base are installed prior to the rebar installation. Often the dome formwork were pre-made and lifted in place.

Constructability

The design of a GBS requires comprehensive constructability input from the construction team throughout the design process starting from the conceptual phase. The constructability effort and feedback improves execution certainty and helps eliminate potential schedule delays.

Generally, constructability aspects are systematically considered starting with the concrete mix design development program prior to initiating detailed design and continuing throughout the construction period. For examples:

- Concrete mix design and test program are developed early to ensure that concrete is flowable without segregation to minimize the risk of honeycombing in dense reinforcement
- Temperature increase is calculated in mass concrete, and special measures are implemented as needed, such as cooling of the concrete and changing the mix design
- T-headed reinforcing bars are used to minimize congestion
- Regular review of design drawings by the construction team especially prior to each slipform or major concrete placement
- Numerous full-scale mock-ups of several complex elements of the GBS are generally required. These are small parts of GBS built at full-scale model using the same procedures, equipment and materials as planned for the actual structure. Lessons-learned from these mock-ups are fed back to the design team to improve constructability. The mock-ups also allow the opportunity to train the work force on these complex operations prior to actual construction.

MULTIDISCIPLINE INTERFACES

The design and construction of a GBS involves numerous interfaces between various disciplines. Some examples are: the interface between the GBS and topsides design teams, the interfaces between the various design disciplines, and the interface between the design and construction team. This section provides more details on the topsides and GBS interface.

Topsides – GBS Interface

One of the key functional requirements of the GBS is to support topsides. In order to meet the overall project schedule, the design of the GBS typically starts prior to the completion of topsides design. Since the engineering, procurement, and construction of GBS and topsides are almost always carried out by different contractors, management of interfaces between these contractors is critical, and a formal interface agreement (IA) process is typically followed.

A couple of examples of IAs between topsides and GBS are as follows:

- Center-of-gravity envelope and not-to-exceed weight for the topsides: In addition to the final topsides-GBS connection design, this information is also critical for floating stability during various marine operations.
- Crude storage cells pipes routing, which determines the penetration locations in concrete walls.

SUGGESTED FUTURE RESEARCH

This section discusses potential future research topics that would result in a more cost-effective offshore concrete GBS. Topics are proposed that are relevant to both analysis/design and construction. They address many limitations that are present in the current analysis and design approaches: more efficient implementation of nonlinear finite element analysis (NLFEA), further improvement in crack-width calculations for heavily-reinforced, thick concrete elements, and further advancement in leak-tightness calculation and acceptance criteria. They also include some construction aspects, in particular, further improvement in the slipforming operation. In addition, there is also a considerable focus on durable concrete structures, which is discussed in Fosså and Widiyanto (2020).

Structural Analysis and Design

The principle of linear superposition is not valid for non-linear analyses. Therefore, it is not possible to run several independent nonlinear analyses and add the load effects obtained from these analyses to obtain the total design combinations - as typically done with elastic analyses results. In other words, the nonlinear analyses must be based on a complete design combination for the actual scenarios to be studied, such as wave load from one direction, combined with concurrent other loads.

Typical procedure in performing a linear analysis is to assume the concrete dimensions and using a post-processing program revise the amount of reinforcement until it meets code requirements. It would be technically very complicated and time consuming to vary the reinforcement and carry out a nonlinear analysis simultaneously.

For known concrete dimensions and amount of reinforcement (determined through linear analysis and post-processing), it is currently possible to carry out a non-linear analysis covering the following aspects:

- thermal transient analysis with results transferred to a structural analysis,
- transient structural analysis,
- nonlinear material model for concrete and reinforcement,
- smeared reinforcement,
- nonlinear boundary conditions, contact elements,
- second order effects and large displacements, and
- full Newton-Raphson equation solver.

Results of such non-linear analyses are displacements, gaps between structure and foundation, crack patterns, etc. (Lie et al, 2018). A similar analysis can be performed for a structure subjected to temperature, waves and various wave effects, current, wind, ice loads, earthquake, and inertia forces. Soil-structure interaction may also be taken into account.

In future, the NLFEA described above may be carried out for various limit states. For instance, more accurate concrete stresses, cracking of concrete, and reinforcement stresses may be calculated for SLS as part of leak-tightness checks.

Although it is possible to perform a full transient dynamic analysis in the time domain for this type of analysis today, there is still a need for further improvement of these such methods. It is especially important to improve the stability of the computer models.

The real benefits from implementing those complex NLFEAs is to better capture the behaviour of structural concrete in the ULS and ALS (including redistribution of forces), which would result in more-optimized concrete

section (including potential reduction of reinforcement), which would further improve constructability and construction schedule, resulting in more cost-effective structures.

Crack-width Calculation

Crack-width calculation is important to ensure durability and leak-tightness of liquid containment structures. Crack-width provisions in various codes and standards are mainly semi-empirical, based on test results of relatively small-scale “beam” type specimens. Additionally, none of these test specimens had multiple layers of transverse reinforcement similar to those commonly used in offshore concrete GBS.

American Concrete Institute codes (e.g., ACI 318, ACI 350) typically control crack widths by limiting reinforcement stresses and spacing. While other codes and standards, such as NS 3473.E:2003 (Concrete Structures: Design and Detailing Rules), provide a method to explicitly calculate crack-widths. The NS 3473.E:2003 provisions include cases where the transverse reinforcement is fixed (welded) to the main reinforcement and the principal strain deviates from the reinforcement direction. However, to the authors’ knowledge, there is no code that provides guidelines to calculate crack-widths in thick, heavily reinforced concrete members subjected to complex loading similar to offshore concrete GBSs. Using currently available simplified formulas, in many cases, results in additional reinforcement beyond that required for strength, increasing rebar density and negatively impacting constructability.

Therefore, it is important to further improve the crack-width calculation formula to account for the following aspects:

- Thick concrete members (in the order of 3 m (10 ft) or so)
- Several layers of bundled orthogonal reinforcement at each face (for example, 32 mm (1.25 in.) rebars spaced at 150 mm (6 in.))
- Principal strain not aligned with the reinforcement direction

It is also important to calibrate new formulas using NLFEA with advanced material models and full-scale laboratory tests.

The implementation of this approach would better capture the behavior of the GBS elements in the SLS and result in more cost-effective and constructible GBSs.

Leak-Tightness Calculation

Currently available guidance (e.g., NS 3473.E:2003, DNVGL-ST-C502) to ensure leak-tightness is typically based on the following:

- Limiting rebar stresses
- Minimum depth of compression zone
- Limiting membrane tensile stresses

This approach is mainly empirical and can sometimes lead to highly congested reinforcement layouts for structural elements subjected to complex loading. The following future research areas could potentially improve practicality/constructability without increasing risk of leakage:

- More realistic limits based on nonlinear analysis with advanced material modelling (for crack propagation, accounting various transient thermal loading) and calibrated with full-scale experimental tests
- Cost-effective durable materials for barriers/liners (subjected to oil with fluctuating temperatures) with special details at the termination points that are more constructible and prevent seepage, based on full-scale testing

Construction

Slipforming Improvements

The concrete used in a slipform operation needs to be specifically designed for slipforming with regard to fresh properties and in particular setting time. This topic is further discussed in Fosså and Widiyanto (2020). Future advancement of the slipforming technology should focus on improving the slipforming equipment. This should also include improvements that reduce the risk of poor quality in the finished concrete surface

One of the key challenges for slipforming the heavily-reinforced concrete offshore GBSs is the slow slipforming rate and the resulting increased friction between the hardening concrete and the panel. This may be improved by using a low friction coating/material in the sliding zone. So far, materials like Teflon, hard plastic, and other low friction materials tested have not been robust enough to resist the wearing action from the concrete. An alternative

solution that can break the bond between the concrete and the slipform panel is to start the lift with a small downwards push before lifting upwards as usual, but requires further development of the lifting equipment.

Lately, the height of the slipforming yoke has been increased from 800 to 1000 mm (31 to 39 in.), which improves the installation of the horizontal rebar and embedments, and slipforming rate. Improving the slipforming rate would reduce the risk of concrete adhering to the slipforming panel and the associated surface damage, which would eliminate expensive repair costs and ultimately result in more schedule certainty and a cost-effective GBS.

SUMMARY

An offshore concrete Gravity-Based-Structure (GBS) is a massive concrete structure placed on the seafloor and held in place strictly by its own weight, without need for anchors. In contrast to typical buildings and bridges, there are several distinct features of an offshore GBS that pose unique challenges in the structural analysis, design and construction. From analysis and design perspective, there have been significant improvement in analysis method and tools since the first GBS installed in 1973. From construction perspective, there have also been some improvement in slipforming method. However, in order to further improve the cost effectiveness of concrete offshore structures, several future research topics are suggested to continue further development in analysis, design, and construction aspects.

REFERENCES

- ACI 347. 2014. Guide to Formwork for Concrete. American Concrete Institute, Farmington Hills, Michigan.
- Det Norske Veritas. 1974. Rules for the design, construction and inspection of fixed offshore structures. Oslo, Norway.
- DNVGL-ST-C502. 2018. Offshore Concrete Structures.
- ISO 19903-2006. 2006. Petroleum and Natural Gas Industries - Fixed Concrete Offshore Structures. International Organization for Standardization.
- Lie R., Aasheim E. E. and Engen M. 2018. Thermal Cracking of a Concrete Arch Dam. FE analyses with a 3D non-linear material model for concrete. Proceedings of the 14th ICOLD International Benchmark Workshop on Numerical Analysis of Dams, Stockholm.
- Norwegian Standard, NS 3473.E:2003, "Concrete structures – Design and Detailing Rules," Standard Norge, Norway, 2003, 128 pp.
- Oberlies, R., Khalifa, J., Huang, J., Hetland, S., Younan, A., Overstake, M., and Slocum, S. 2014. Determination of Wave Impact Loads for the Hebron Gravity Based Structure (GBS). Presented at the 33th International Conference on Offshore Mechanics & Arctic Engineering, San Francisco, California, USA, 8-13 June. OMAE2014-23503.
- Widianto, Chichester, J., Younan, A., Khalifa, J., Komperla, K., and Bidne, K. 2018. Hebron Platform: Innovative Design and Efficient Execution. Proc. Offshore Technology Conference, Houston, Texas, 30 April – 4 May, OTC-28803-MS.
- Widianto, Khalifa, J., Younan, A., Karlsson, T., Stuckey, P., and Gjørven, A. 2013. Design of Hebron Gravity Based Structure for Iceberg Impact. Proceedings of the Twenty-third (2013) International Offshore and Polar Engineering, Anchorage, Alaska, USA, 30 June – 5 July. ISOPE-I-13-036.
- Widianto, Khalifa, J., Taborda, G., and Bidne, K. 2016. Concrete Gravity-Based Structure: Construction of the Hebron Offshore Oil Platform. Concrete International, American Concrete Institute, 38(6):29-36.
- Widianto, Khalifa, J., Hæreid, K.O., Fosså, K.T., and Gjørven A. 2020. Hebron Offshore Concrete Gravity-Based-Structure: Novel Design and Construction Techniques. ACI Special Publication, ACI.
- Fosså, K.T. and Widianto. 2020. Concrete Mix Design Development for Offshore Structures. ACI Special Publication, ACI.

Table 1 – Summary of major offshore Gravity Based Structures (GBS)

Year Delivered	Platform	Average rebar density kg/m ³	Concrete Grade MPa	Concrete Volume m ³	Location	Water depth m	Key features
1973	Ekofisk 1	145	C45	80,000	North Sea	70	First major GBS, has 9 storage cells and an outer perforated concrete breakwater wall as part of the tank structure.
1975	Beryl A	175	C45	52,000	North Sea	118	
1975	Brent B	150	C45	64,000	North Sea	140	
1975	Frigg CDP-1		C50	60,000	North Sea	98	
1976	Brent D	185	C50	68,000	North Sea	140	
1976	Frigg MP-2		C50	60,000	North Sea	94	
1976	Frigg TP-1			49,000	North Sea	104	
1977	Statfjord A	180	C50	87,000	North Sea	145	
1977	Frigg TCP-2	225	C50	50,000	North Sea	104	
1977	Dunlin A			90,000	North Sea	153	
1978	Brent C			105,000	North Sea	141	Has more number of cells (30) and a slightly larger footprint than Gullfaks C. The main reason for 30 cells is to satisfy the requirement for one cell damage stability. Slipform length approximately 3,650 m.
1978	Cormorant A			120,000	North Sea	149	
1978	Ninian Central		C55	140,000	North Sea	136	
1981	Statfjord B	255	C55	140,000	North Sea	145	
1984	Statfjord C	255	C55	130,000	North Sea	145	
1986	Gullfaks A	260	C55	125,000	North Sea	135	
1987	Gullfaks B	240	C55	101,000	North Sea	141	
1988	Oseberg A	240	C60	116,000	North Sea	109	
1989	Ekofisk PB			105,000	North Sea	75	
1989	Gullfaks C	260	C70	244,000	North Sea	216	- Heaviest manmade object ever moved (1,500,000t including solid ballast and water ballast during tow-to-field) - the largest slipforming operation in history (performed at the Deep-Water Construction site) - first GBS with long concrete skirts (21m penetration)
1991	Sleipner A1	290	C65	75,000	North Sea	82	Not installed because it did not survive submergence tests. Reasons of failure believed to include: under estimation of shear force by the FEA and post-processing programs, and inadequate detailing of transverse reinforcement.
1993	Sleipner A2	430	C65	75,000	North Sea	82	Replacement of Sleipner A1 with updated FEA and detailed review of all post-processing design.
1993	Draugen	270	C70	80,000	North Sea	251	First major GBS with a single shaft
1995	Heidrun TLP	350	C60	58,000	North Sea	345	Floating platform anchored to the seabed (tension leg platform)
1995	Troll A	400	C80	245,000	North Sea	303	- Tallest structure ever moved (overall height of 472m, weighing 684,000t (1,200,000t with ballast) - Has the longest concrete skirts (36m penetration)
1996	West Tuna	480	C50/C60	29,000	Bass Strait	61	- First platform designed for iceberg impact
1997	Bream B	430		11000	Bass Strait	61	
1997	Hibernia	510	C70	165,000	Grand Banks	80	
2005	Sakhalin II - LUN-A	440	C60	35,500	Sea of Okhotsk	48	
2005	Sakhalin II - PA-B	520	C60	28,000	Sea of Okhotsk	31	
2008	Adriatic LNG Terminal	290	B45	85,000	Adriatic Sea	29	
			B50 (General) B70 (Ice abrasion zone)				
2011	Sakhalin I - Arkutun Dag	380		52,300	Sea of Okhotsk	35	
2017	Hebron	365	B65	135,000	Grand Banks	93	- Heaviest marine float-over and Topsides-GBS mating operation - Heaviest topsides supported on a single shaft - Designed for iceberg impact

Note: Concrete grade Cxx refer to cube strength (MPa) and Bxx to cylinder strength (MPa)

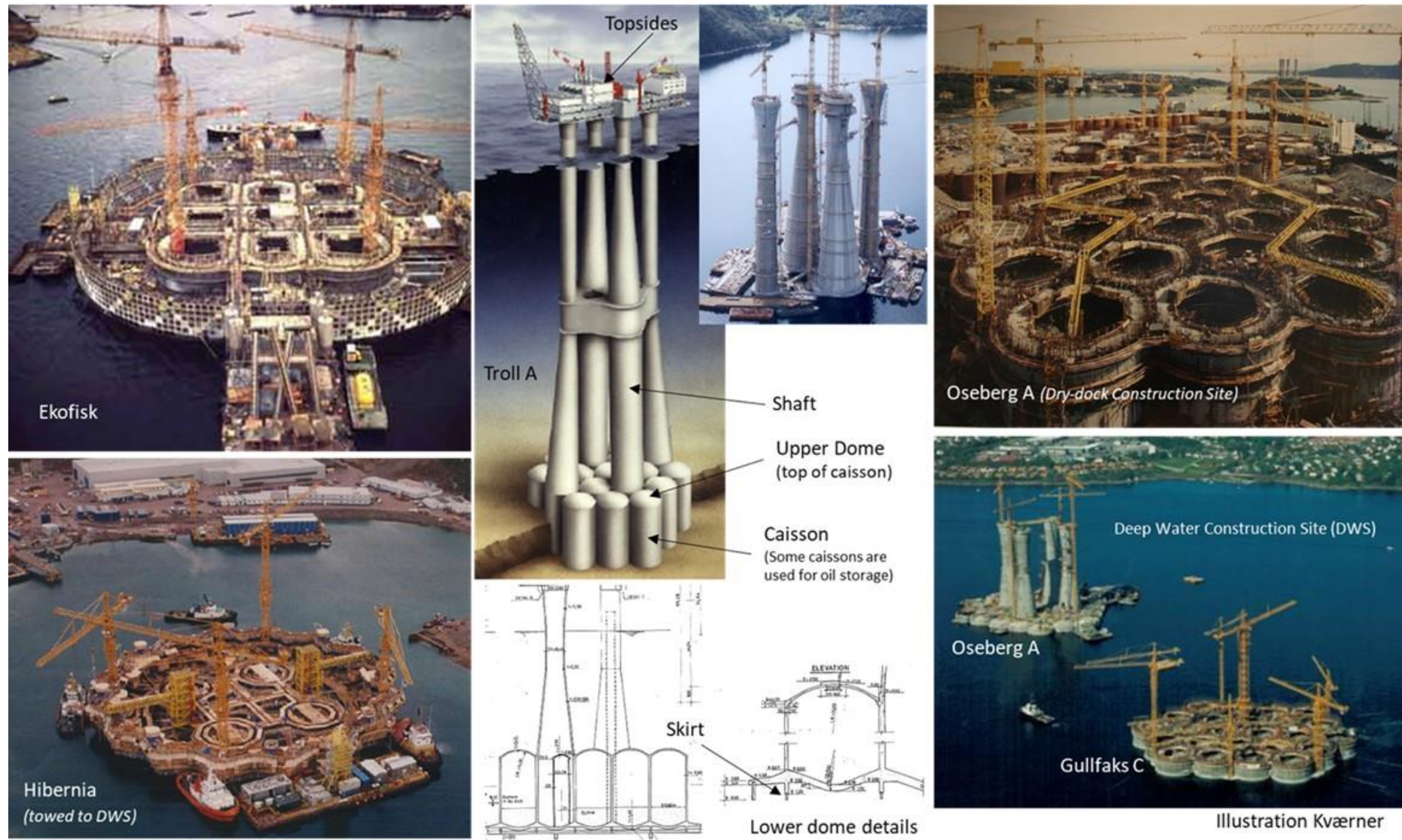


Fig. 1 - Several major GBS and features

Table 2 - Finite Element types and number used for global FEA of selected GBSs

Project	Concrete volume (m ³)	Element type	Number of elements	DOFs (unknowns)	Comments
Gullfaks C (1989)	244 000	20 node Solid	31,400	610,000	Only parts of shafts are included
Troll A (1995)	245 000	8 node Solid	170,000	950,000	
Sakhalin II LUN-A (2005)	35 500	8 node Solid 4 node Shell	12,600 73,700	427,000	Solid elements in intersection and top of Shaft.
Sakhalin II PA-B (2005)	28 000	8 node Solid 4 node Shell	9,200 120,000	721,000	Solid elements in intersection and top of Shaft.
Adriatic LNG (2008)	95 000	8 node Solid	246,000	1,410,000	
Sakhalin I Arkutun-Dagi (2011)	52 300	8 node Solid 4 node Shell	728,000 79,300	1,740,000	Shell elements in skirts.
Hibernia (1997)	165,000	20 node solid	42,000	840,000	
Hebron (2017)	135 000	8 node Solid 4 node Shell	455,000 3,480	1,990,000	Shell elements in skirts.

**Fig. 2** – Determination of wave-structure interaction on a GBS**Fig. 3** - Typical icebergs in the North Atlantic

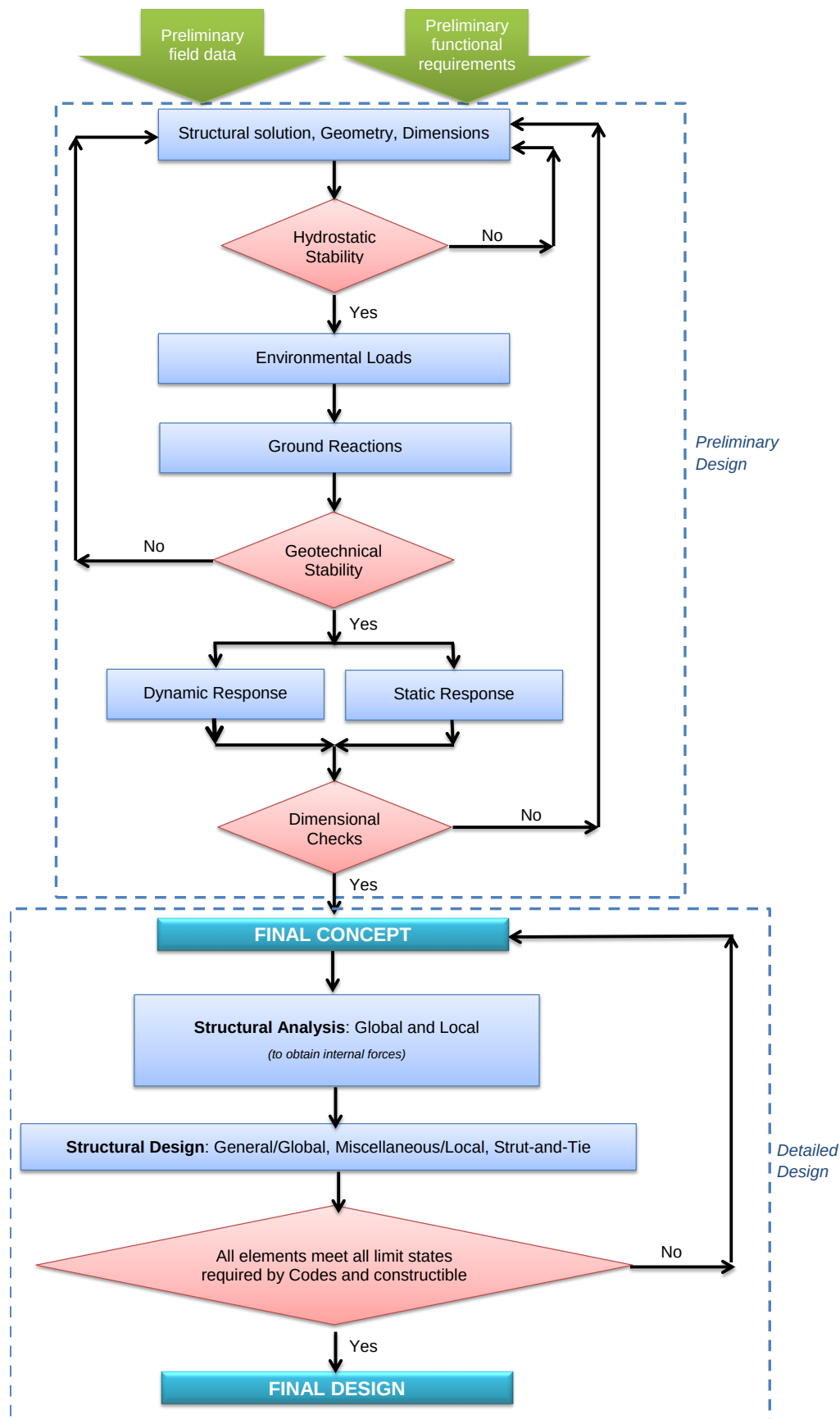


Fig. 4 - Iterative procedure to develop Final Design

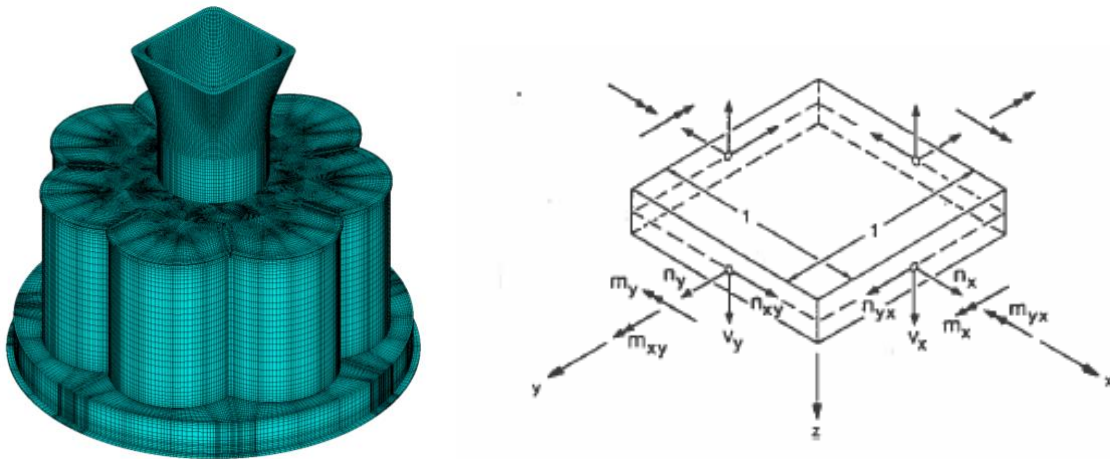


Fig. 5 – Finite element model

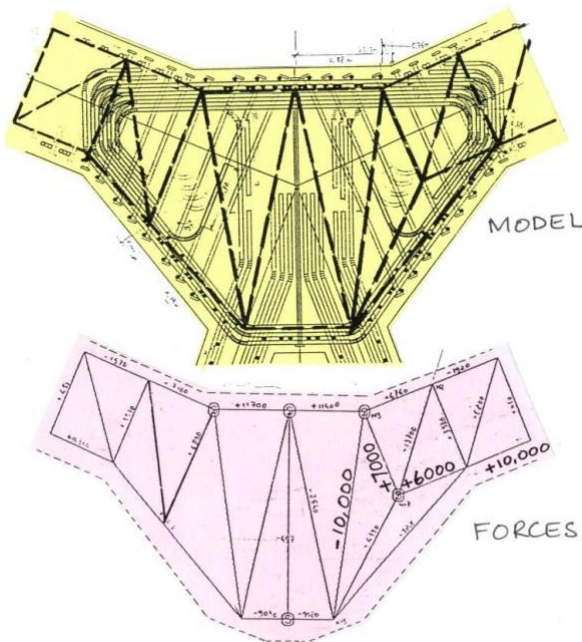


Fig. 6 – Strut-and-tie model

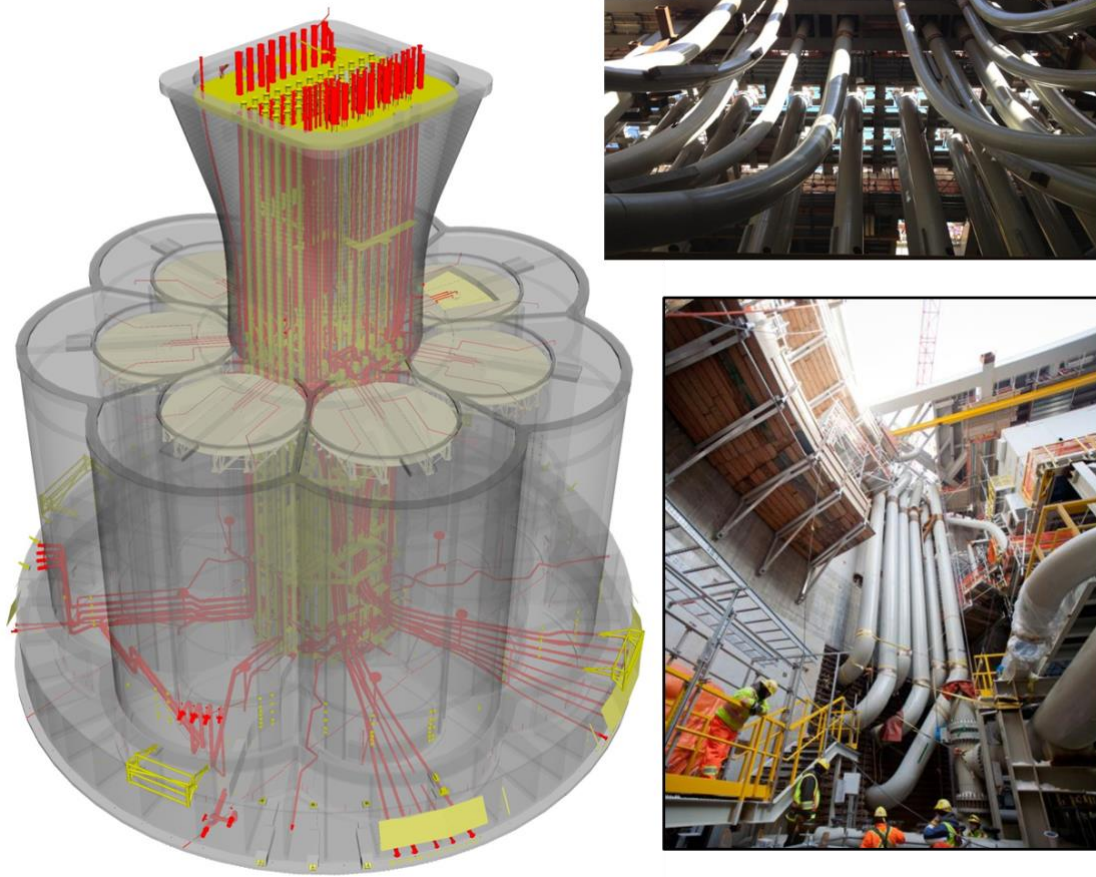


Fig. 7 – Mechanical outfitting



Fig. 8 – Construction phases –Hebron Platform

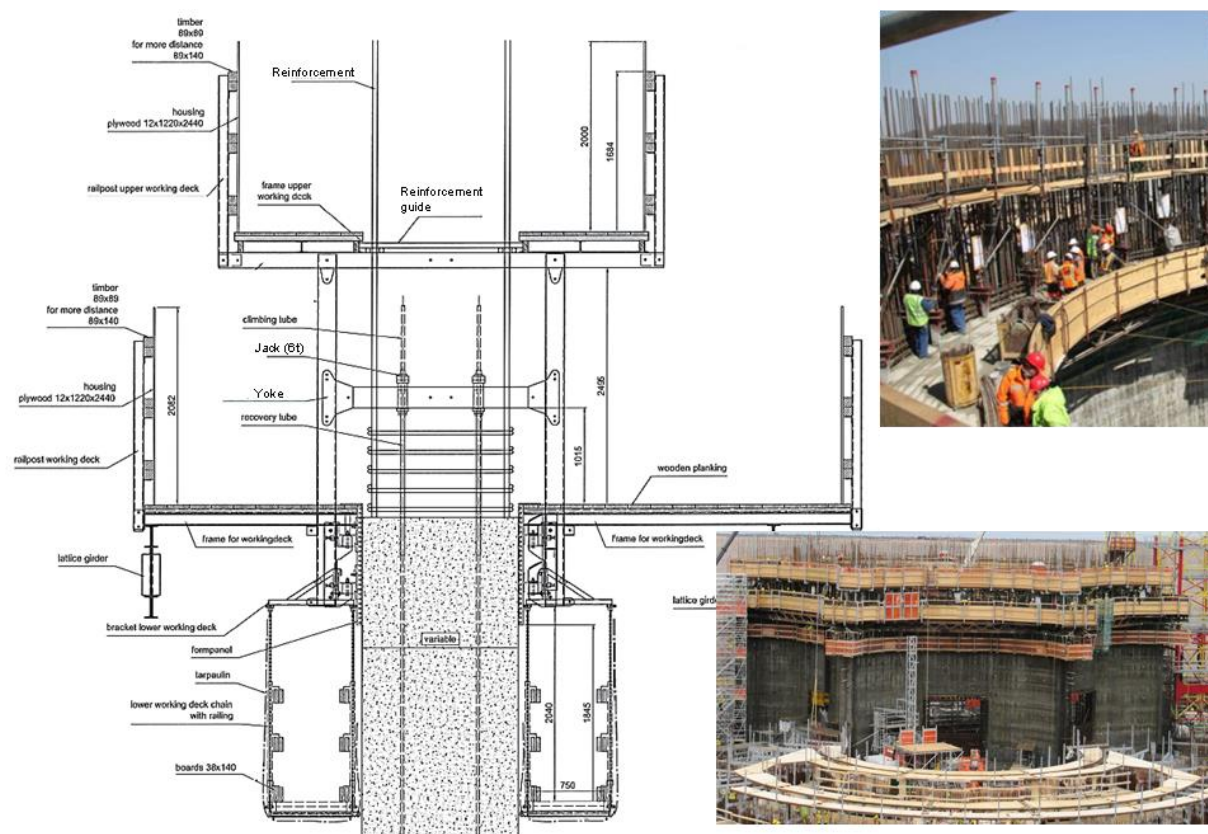


Fig. 9 - Typical slipform assembly (left) and work at the slipform (right)

Hebron Offshore Concrete Gravity-Based-Structure: Novel Design and Construction Techniques

Widianto, Jameel Khalifa, Kåre O. Hæreid, Kjell Tore Fosså, Anton Gjørven

Synopsis: The Hebron platform is the latest major offshore integrated oil drilling and production platform supported by a concrete gravity-based-structure (GBS). It was successfully installed in the Grand Banks (offshore Newfoundland) in June 2017. The design of the platform was challenged by arctic-like and extreme metocean conditions. This paper presents development of extreme loads on the GBS such as 10,000-year iceberg impact and wave loads. It also describes novel design and construction techniques used, which resulted in a capital-efficient platform.

From an analysis and design perspective, in addition to linear-elastic finite element analysis typically used in design of offshore concrete GBS, the innovative use of non-linear finite element analysis (NLFEA) technique to calculate internal forces is presented. Such analyses more accurately capture the structural behavior and result in more realistic internal forces. In addition, a new crack-width calculation method accounting for the effect of a significant number of layers of transverse reinforcement was implemented. Also, a novel method to assess the complex interactions between solid ballast, embedded pipes, and concrete structures was applied.

From a construction perspective, the use of slipforming panels that are taller than those used in past GBSs and a system to allow slipforming of the shaft wall with a complex geometry and curvature, that is much larger than that employed in the past GBS, are presented. A novel method to minimize the risk of concrete adhering to slipforming panels by cooling the panels with cold water is presented. An innovative method to ensure that high-strength grout completely filled the space underneath one of the largest Topsides footings is discussed. Full-scale constructability tests of various complex GBS components, which provided invaluable information for design, increased execution certainty, and improved construction safety, is presented.

Keywords: Hebron, Offshore, Design, Construction, Gravity Based, Slipforming, Crack Width, Grouting

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INTRODUCTION

The Hebron platform is the most recent major offshore integrated oil drilling and production platform supported by a concrete gravity-based-structure (GBS), see Figure 1. It was successfully installed in 93 m (305 ft) water depth in the Grand Banks (~340 km (211 miles) offshore Newfoundland) in June 2017. The design of the platform was challenged by arctic-like and extreme metocean conditions. This paper presents the development of extreme loads such as iceberg impact and wave impact on the GBS, and the implementation of various innovative design and construction techniques that resulted in a capital-efficient platform.

NOVEL DESIGN METHODS

This section discusses several novel methods implemented in the design of the Hebron GBS. These include development of design loads through advanced statistical analysis and model tests, Non-Linear Finite Element Analysis Methods, crack width calculation method, foundation design approach, and a novel design approach for the complex interactions between solid ballast, embedded pipes, and concrete structural components

Development of Design Loads

Unlike typical buildings located in urban areas, environmental loads for offshore structures, such as iceberg impact and wave impact, are typically not mentioned in codes/standards and are more difficult to quantify. This section presents novel approaches implemented during design of the Hebron GBS to calculate iceberg and wave impact loads.

Advanced Statistical Analysis (for development of iceberg impact loads)

Icebergs drift into the area where the Hebron platform has been installed. The International Ice Patrol's website archive of icebergs dating back to 1900's shows annual iceberg counts drifting south of latitude 48° N ranging from zero to more than 2,000. As such, it was essential to design the Hebron platform to resist iceberg impact loads.

Icebergs vary in size, drift speed, shape, length, draft, etc., therefore, development of iceberg impact loads requires an advanced statistical analysis. The ISO 19906 standard requires that two levels of iceberg impacts be considered

in design: the 100-year return period event as well as the 10,000-year return period event. For Hebron, the governing impact load was for the 10,000-year return period (~486 MN (109,300 kip)), which is considered a very rare Abnormal-Level ice event per ISO 19906.

Iceberg design loads were estimated using the Iceberg Load Software (ILS). This software uses a probabilistic algorithm, which incorporates Monte Carlo simulations for distribution of global impact forces. The methodology takes into account iceberg size, drift speed, strength, shape, and other variables. Sea state conditions and associated hydrodynamic effects are also included. In addition to Type I uncertainty captured by Monte Carlo simulations, Type II uncertainty was considered via a logic tree analysis to capture a range of expert opinions and to add further conservatism to the load generating recipe.

Design loads were estimated for both a population of icebergs and a population of ice islands by running two separate analyses (one for each population). The total analysis cases included 96 logic tree branches, and for each branch, one million iceberg and one million ice island impacts were simulated. The impact forces were sorted and plotted in exceedance form to create a hazard curve for each branch. All 96 hazard curves were based on 192 million impacts (2 million impacts $\times$ 96 cases) occurring over the entire GBS.

The impact force calculation was based on a kinetic energy approach in which energy is dissipated due to ice crushing and iceberg rotation caused by eccentric impacts. Pressure-area relationships were developed to determine global loads. Local ice pressures were calculated accounting for the size of the loaded area, and the duration and frequency of the impacts. The rare nature of iceberg impacts was accounted for by adopting a Poisson process to model the arrival rate of icebergs.

Individual iceberg impacts with maximum forces very close to the 10,000-year design load were extracted from the simulation and force-time curves were generated using the time domain model using principles of ice mechanics. A more detailed description of the method and resulting loads are provided in Widiyanto et al. (2013).

Novel Model Tests (for development of wave impact loads)

The Hebron platform is located in an area characterized by harsh storms with large waves and high winds. The geometry of the Hebron GBS has an effect on the behavior of the incident waves with regards to their likelihood of breaking onto the shaft. As such, wave impact loads are best estimated through model testing because of several limitations of analytical methods.

Three separate model tests of the structure (1:50 scale using Froude scaling) in storm waves were executed to estimate GBS global loads as well as local wave loads on the GBS shaft. These tests required significant planning and design of the model, wave environment and instrumentation in order to properly satisfy the test objectives. Careful consideration was given to model stiffness and the force and pressure sensing equipment selection and calibration. Additionally, the accuracy and repeatability of the waves in the test basin was crucial for collecting reliable data. The results of the tests showed that the measured wave impact loads on the structure were highly variable, requiring a long-term response based method to quantify design loads on an annual exceedance basis.

A specialized methodology was employed to post-process the data from the test program to develop design loads. This approach provided a method for combining probabilities of sea-states and wave impacts into a joint probability distribution that can be used to derive yearly exceedance probabilities of wave impacts. As such, design values were estimated based on the probability of a particular response instead of the probability of a specific input (i.e., sea-state return period). More details can be found in Oberlies et al. (2014).

Non-Linear Finite Element Analysis (NLFEA)

Internal forces used to design offshore GBSs are typically calculated using linear-elastic finite element analysis, allowing superposition, which is convenient for handling a large number of load cases and combinations required for checking the various limit-states. However for Hebron, a non-linear finite element analysis (NLFEA) technique was employed to calculate internal forces for selected extreme load cases. For example, NLFEA was used for the 10,000-year return period iceberg load on the exterior caisson walls, as well as wave impact loading (both 100-year and 10,000-year return periods) on the shaft.

Compared to linear-elastic analysis, some of the benefits of NLFEA are as follows:

Under some extreme load cases associated with Ultimate Limit State (ULS) and Accidental Limit State (ALS), internal forces calculated using the linear-elastic FEA can result in unrealistically high stresses in localized areas (whereas in reality, yielding would occur and re-distribute the forces over a larger area). Designing to these high stresses would require much larger reinforcement amounts, which would negatively affect constructability, cost,

and schedule. On the other hand, NLFEA redistributes the internal forces better, allowing local yielding and a reduction in peak stresses. This usually results in a reduction of rebars in an already heavily-reinforced area with a small increase of rebars in adjacent areas that are less-reinforced. Note that even though NLFEA was used for selected extreme load cases (ULS and ALS), the Serviceability Limit State (SLS) calculations were performed by linear elastic FEA to satisfy crack-width and leak-tightness criteria.

Ice Walls Subjected to Iceberg impact loads

When subjected to 10,000-year iceberg impact loading, the Hebron GBS was designed to be highly-utilized, consistent with the ALS check per ISO 19900. The ALS structural check requires that accidental loading does not lead to a complete loss of integrity of the structure. The intention of this limit state is to ensure that even if damage occurs, sufficient structural integrity is maintained under specified environmental conditions to safely evacuate personnel. Acceptable damage could include large strains and displacements.

The NLFEA captured the redistribution in internal forces due the cracking of concrete under tension, allowing the ice wall to carry more load as membrane compression rather than bending. This was much more efficient since concrete has a greater capacity to resist compression. Non-linear behavior also reduced the peak forces/moments, thereby reducing the amount of reinforcement. Compared to linear elastic analysis, the use of NLFEA in ice walls design resulted in improved constructability and construction safety, and considerable cost and schedule savings: ~3,500 tonnes (3860 tons) less reinforcement; ~700 tonnes (770 tons) reduction in post-tensioning cables. Details of this NLFEA are presented in Widiyanto et al. (2013).

Shaft Subjected to Wave impact loads

The use of NLFEA in the shaft reduced the peak tension in shaft walls due to overall bending, thereby allowing a reduction of the reinforcement amount by ~300 tonnes (330 tons). More importantly, the number of layers of vertical reinforcement on each face of the shaft wall were reduced from two to only one, resulting in a more constructible shaft wall. In the most densely reinforced area of the shaft, the amount of reinforcement was reduced by about 50%. This was crucial for successfully slipforming these walls, because of the following reasons:

- The overall slipforming rate is governed by the time required to install reinforcement at the most densely reinforced area because all slipform panels move together during the slipforming operation.
- The time to install reinforcement can be decreased if its amount can be reduced, allowing an increase in the overall slipform rate. In particular, a reduction in number of reinforcement layers of the flared shaft (where vertical rebars needed to be bent to follow the shaft curvature) greatly improved constructability.
- Increasing the slipforming rate reduces the risk of concrete sticking to the slipforming panels, hence reduces the potential of surface damage, which is critical for successful slipforming.

Local Analyses using Non-Linear Material Models

The following local structural analyses were performed using non-linear material models:

- **Built in stresses during construction stages:** These analyses simulated the various casting sequences during floating phases. Non-linear analyses utilizing birth and death technique were used for calculation of built-in stresses in the structure. The birth and death technique allows reactivation and deactivation of selected elements. Deactivation of elements reduces the element stiffness to zero, which eliminates the effect of the deactivated elements on the model even though they are technically still present. This technique was used in simulating the construction sequence, in which temporary construction supports were removed (death) and stiffness of newly cast structural elements was taken into account (birth).
- **Modified global FEA for tension release in shaft tricell walls:** During the installation case in which the shaft does not have any ballast, while all other cells (caissons and annuli) are ballasted (a combination of solid ballast and water ballast), global FEA results showed that large tension and shear forces were present in the shaft tricell walls. These large tension and shear forces were mainly due to “hanging” load condition (i.e., large upward hydrostatic pressure at the underside of the shaft base slab in conjunction with large net downward forces at all surrounding cells). Using NLFEA, tension was released in the tricell walls. The redistribution of forces resulted in some additional compressive stresses in the shaft wall.
- **Transient temperature structural analysis for oil filling in storage cells:** In order to prevent waxing, crude oil is heated to ~50°C-70°C prior to being stored in the GBS oil-storage cells. One of critical load cases is the first oil filling, prior to which all the GBS oil storage cells are filled with much colder sea-water, resulting in significant thermal loads. This load condition was analyzed using a refined local NLFEA, which is important to accurately capture any concrete cracking associated with thermal loads. The transient temperature effects from load filling were simulated in the time domain. The temperature for each load step was transferred to an identical structural model for stress evaluation purposes.

- **Connection between ice-wall and the base slab:** During the Topsides-GBS mating, the GBS was submerged to a ~115m (377 ft) draft. The linear FEA gave relatively large bending moments (vertical stresses) at the connection between ice-wall and the base slab. The non-linear analysis allowed a reduction in these design moments.
- **Crack-width calculation:** NLFEA was employed for crack-width calculations for thick members reinforced with several layers of bundled rebars in each direction and is described in the following section.

Crack Width Calculation Method

Crack-width provisions in various codes and standards are semi-empirical, based on test results of relatively small-scale “beam” type specimens. None of the test specimens had multiple layers of transverse reinforcement similar to the reinforcement layouts used in the Hebron GBS. For example, Figure 2 shows the cross-section of Hebron GBS base slab around the drilling conductor penetration. The bottom face reinforcement consists of 10 layers of bundled 32 mm (1.25 in.) diameter rebars spaced at 200 mm (8 in.) in each direction.

The currently used semi-empirical formulas generally require additional reinforcement for crack width control beyond that required by the ULS. In areas where multiple reinforcement layers in each direction are needed for ULS, additional rebars to satisfy crack width control can only be placed further away from the concrete surface. This not only reduces the effectiveness of the added rebars but also negatively impact constructability and increases the risk of honeycombing.

Therefore, a new method for calculating crack widths in thick elements with multiple reinforcement layers was implemented in the design of the Hebron GBS. This method accounted for the crack-initiation effect from several layers of transverse reinforcement that are not fixed/welded to the main reinforcement, and was validated by a parametric study using NLFEA. This new method did not require any additional reinforcement beyond what was needed for the ultimate limit state, which improved constructability.

NS 3473 Provisions for Crack Width Calculation

The design of the Hebron GBS is based on NS 3473, the Norwegian concrete code. As such, all GBS concrete elements were calculated using rules and parameters recommended by NS 3473.

The basic principle for crack width calculations in NS 3473 is to find a characteristic strain tensor ($\epsilon_x, \epsilon_y, \epsilon_{xy}$) on the tension side of the concrete by considering non-linear material properties for concrete and rebars with different diameter, orientation and center distances within each layer. Shear transfer that takes place between the rebars and concrete is also taken into account.

The characteristic crack spacing for cracks normal to the direction of the reinforcement is determined by the Eqn. (1):

$$s_{rk} = 1.7 s_{rm} = 1.7 \left(s_{ro} + \frac{k_c A_{cef}}{\sum \left(\frac{\pi \phi \tau_{bk}}{f_{tk} k_b} \right)} \right) \quad (1)$$

The 1.7 factor accounts for the statistical variation (maximum expected crack width versus the mean crack width)

length with an assumed loss of bond = concrete cover c

where:

The 1.7-factor accounts for the statistical variation (maximum expected or characteristic crack spacing, s_{rk} , versus the mean calculated crack spacing s_{rm}).

“x” is the contribution from the rebars to the crack spacing, s_{rm} (the more rebars, the lesser s_{rm}). The summation encompasses reinforcement located within A_{cef} .

A_{cef} is the effective concrete section

ϕ is the diameter of the reinforcement

$\frac{f_{tk}}{\tau_{bk}}$ is effective relation between tension and bond strength (=0.75 for ribbed bars).

k_c is coefficient to account for the strain distribution in the cracked section

k_b is coefficient to account for reduced bond for bundled reinforcement

For a general case with a concrete shell element, where the principal strain deviates from the reinforcement direction, the crack spacing in the principal strain direction may be calculated from the following simplified Eqn. (2):

$$s_{rm} = \left(\frac{1}{\frac{\sin v}{s_{rmx}} + \frac{\cos v}{s_{rmy}}} \right) \quad (2)$$

where:

v is the angle between the principle strain and the y-direction considering the bars to be placed in the x and y direction, respectively

s_{rmx} and s_{rmy} are the calculated crack spacing in the x and y directions, respectively

The characteristic crack width is then calculated by simply multiplying the maximum principal strain by the characteristic crack spacing, s_{rk} ($=1.7 s_{rm}$).

NS 3473 has provisions for calculating crack spacing in cases where transverse rebars are fixed (i.e., welded) to the longitudinal rebars, but it does not provide any guidance in cases where several layers of bundled reinforcement are not fixed to the longitudinal rebars, similar to those in the Hebron base slab.

Section A.15.2.3 of NS 3473 provides some basis to application of engineering judgment. It states “...the calculated crack widths will not always be in accordance with the actual crack widths, which could be larger or smaller than the calculated values. Calculation of crack widths is therefore assumed to be approximate and the requirements which are given should not be regarded as absolute...”

Parametric Study using NLFEA

A parametric analytical study was performed during the detailed design of Hebron GBS base slab in order to understand the effect of heavy transverse reinforcement on crack-widths. The analysis model utilized a linear elastic material for concrete in compression and a failure model for the concrete in tension that allowed the concrete to take tensile stresses up to a certain level as described in concrete codes/standards for various concrete strengths.

This study was performed by utilizing the sub-modelling technique within ABAQUS. A simplified model consisting of a slice through the 2.4m thick base slab on the Hebron GBS was developed. The concrete material model “*BRITTLE CRACKING” was used in ABAQUS. The tension softening curve was scaled such that maximum tensile stress were in accordance to NS 3473.

This parametric study using NLFEA demonstrated the following:

- Without transverse reinforcement the development of cracks appeared to be randomly distributed. The distance between crack openings reaching the concrete surface was irregular and the calculation of crack spacing could be based on statistics, i.e., the 1.7-factor in Eqn. (1).
- When at least 3-layers of bundled transverse reinforcement were present, the cracks developed more regularly and crack spacing appeared to be related to the presence of transverse rebars. This could be a result of the transverse rebars creating a “weakness” in the tensile zone of concrete. Therefore, the use of a statistical factor lower than 1.7 in Eqn. (1) was reasonable. This “crack initiator” effect was not so dominant with one or two layers of transverse rebars, which is also consistent with observations in lab tests of scaled beams with stirrups (i.e., the effect of stirrups as crack initiators was often noted, but was considered insignificant).

Based on the results of this parametric study, the following engineering judgement was implemented in the design of thick concrete members reinforced with more than 3 layers of bundled 32 mm (1.25 in.) diameter rebars spaced at 200mm (8 in.) (center-to-center) in each direction:

- Average crack spacing: $s_{rm} = 200$ mm (8 in.).
- Cracking could develop anywhere between inside of bundles to outside of bundles (130 mm – 270 mm (5.1 in. – 10.6 in.)), therefore the characteristic crack spacing $s_{rk} = 270$ mm (10.6 in.) was chosen.
- This is about 20% less than $1.7 \times s_{rm} = 340$ mm (13.4 in.), where 1.7 is a code factor to account for randomness as shown in Eqn. (1).

Foundation Design

Hebron GBS has a flat, circular base slab with 500 mm (20 in.) deep skirts (elements that penetrate the soil). Several key considerations related to the foundation design optimization for the Hebron GBS are as follows (Tistel et al. (2015)):

- Optimization of skirt depth
Corrugated steel skirts were used mainly to improve horizontal sliding resistance by transferring the loads deeper into the soil where it is stronger. From a constructability perspective, shorter skirts are more cost effective because they require shallower trenches in the dry-dock construction site, minimize the negative impact from skirt length to the draft requirement during tow, and are subjected to smaller stresses during soil penetration at installation (hence avoiding thicker skirts). Furthermore, the GBS base slab was cast directly on a coarse aggregate bed in the dry dock, in order to improve roughness and sliding resistance.
- Elimination of under base grouting
High resolution surveys of seabed bathymetry and a comprehensive assessment of soil properties concluded that the platform could be installed directly on the seabed without performing expensive offshore under base grouting. FEA was used to calculate local soil stresses against the base slab that may result from any minor seabed unevenness and the base slab was designed to accommodate these peak stresses. While this required some additional reinforcement in the base slab, it was still far more cost-effective than performing under base grouting.
- Elimination of scour protection
GBSs located on sand are in some cases subjected to scour, especially around corners and edges of the structure where the seabed current is amplified. The circular shaped foundation for Hebron was favorable when considering scour. Furthermore, a comprehensive scour risk evaluation was carried out for Hebron including assessment of seabed material (grain size), historical current velocities, and experience from a nearby GBS. It was concluded that installing scour protection around the Hebron GBS was not needed. Instead, a comprehensive periodic and after-storm scour inspection plan was implemented.

Design for Interactions among Solid Ballast, Embedded Pipes, and Concrete Structural Components

Solid Ballast (SB) Material

The purpose of solid ballast (SB) is to improve floating stability (lower the vertical center of gravity) and to increase the on-bottom weight (to achieve sufficient geotechnical stability) after platform installation. Some the key requirements for SB were as follows:

- Target unit weight of 3,500 kg/m³ (220 lb/ft³)
- No segregation and degradation during placement
- Stability of SB after placement, to avoid any SB movement during various floating phases
- A minimum angle of repose of the submerged consolidated SB of 25 degrees

In order to meet the floating stability and on-bottom weight requirements, SB was installed at the bottom of all 7 annulus cells (up to ~11.3m (37.1 ft) high) and all 7 oil storage cells (up to ~11.4m (37.4 ft) high). Most pipes at the bottom of annulus and oil storage cells were embedded within the SB. Figure 3 shows a plan and elevation view of pipes on the bottom of a typical cell.

In order to meet the above requirements, the SB mix was constituted of iron ore, fly ash, water, cement, silica-fume, retarder, super-plastisizer, and stabilizer. In order to understand the behaviour of SB at various stages, the SB materials were tested both in the laboratory and in a site mock-up.

Mock-up tests showed that SB mix behaved quite similar to “concrete”, with curing processes and hardening in the early phases of ballast placement, and a larger material stiffness in the long term. Therefore, the interaction between solid ballast, embedded pipes, and concrete structural components are complex and presented design challenges.

Interaction between Solid Ballast (SB) and Concrete Structural Components

SB mock-up tests demonstrated that the magnitude and distribution of the horizontal pressure exerted on the walls, for a period of time after the placement of the SB, was different than the expected hydrostatic pressure distribution. The increased pressures were assumed to result from thermal expansion of the SB material during the curing process.

Some key design considerations were:

- SB pressure on the concrete wall during filling,
- Design of SB filling sequence and the required amount of ballast water in adjacent cells during SB filling to minimize the impact of differential pressure on concrete walls,
- Effect of SB stiffness on the walls (i.e., fixation / clamping effect), especially under maximum water pressure (during Topsides-GBS mating) resulting in the largest wall deformation,
- Effects of the locked-in stresses and deformations from the solid ballast filling.

Interaction between Solid Ballast, Embedded Pipes, and Concrete Structural Components

Similar to concrete, SB undergoes a “curing” process from “slurry” to a “stiff” material. During SB placement, the load on pipes changes from downward loading (when the SB is placed on the top of the pipe) to a buoyant upward loading when the cell is filled. In order to minimize the forces on the pipe (and its reaction force at the concrete walls support points) and its deformation, SB filling was done incrementally in layers. This way, the SB below selected pipes would achieve a certain stiffness and provide support to that pipe, prior to further filling. Incremental analysis was performed with the appropriate SB properties at various time intervals after placement (Figure 4) to calculate loads on pipes more accurately.

In some cases, when a more rapid filling sequence was required, supporting structures were installed at certain intervals to support the lower pipes, as shown in Figure 3.

A full analysis of a pipe response, combining the load effects from SB placement and operational thermal loads was performed. In order to minimize the loads on pipes and concrete walls, a “pipe-in-pipe” solution (Figure 5) was implemented for some pipes with significant thermal loading during operation. Curved pipes were enclosed in larger secondary pipes, which allowed unrestrained displacement.

In addition, the “shearing effect” of pipes at caisson wall penetrations was also analysed. FEA with non-linear material properties was utilized to determine the relative displacement between the SB and concrete walls (especially under maximum deformation during Topsides-GBS mating). When the calculated relative displacement was quite large, the pipes were wrapped by flexible materials to accommodate relative movement.

NOVEL CONSTRUCTION TECHNIQUES

This section discusses several novel techniques implemented during construction of the Hebron GBS, which includes state-of-the-art slipforming, grouting of Topsides-GBS connections, full-scale mock-ups, and innovative construction joint in the base slab. The various construction phases of Hebron GBS are described in Widiyanto et al., 2016 and 2020.

State-of-the-Art Slipforming

Slipforming was used to construct all of the vertical elements of the Hebron GBS. The slipform set-up consisted of a working deck for installation of block-outs, embedments and horizontal rebar, a top deck for installation of vertical re-bar, and a hanging deck for inspecting the slipformed concrete surfaces. Wheelbarrows were used to unload concrete into hoses at the top deck. These hoses conveyed the concrete to the working deck where it was placed and vibrated. The novel parts of the slipform assembly were:

- Yoke height of 1 m (39 in.) (normally 800 mm (31 in.)). This enabled an earlier start with rebar and embedment installation at the working deck.
- Cooling system on the slipform panel (Figure 6). This closed system made it possible to continuously cool the slipform panel during slipforming without disturbing work at the hanging deck. The system consisted of steel tubes attached to the slipform panel with cold water circulating through them to keep the panel surface cold. The low temperature prevented concrete lumps from adhering to the panels thereby improving the concrete surface quality even with slow slipform rates during hot weather conditions.
- The conical slipform system (including star-girder arrangement) used for slipforming of the GBS shaft above the top slab (which has a unique and complex geometry), as shown in Figures 7 and 8. Note that Sections A-A and B-B in Figure 9 are taken from Figure 7. The conical rig itself was not new, but the geometry change during this slipform was considerable. It started with an outer shaft diameter of 36 m (118 ft) with a wall thickness of 1.5 m (4.9 ft). At the top of the slipform operation, the shaft ended in a square shape with outer dimensions of 53.1m (174 ft) (75 m (246 ft) diagonally) with a wall thickness 4.52m (14.8 ft) (at the thickest point in the four corners).

State-of-the-Art Grouting and Tubular Bearing at Topsides-GBS Connections

The single shaft had four topsides support points with very high reaction forces (~20,000 tonnes (22,000 tons)). The diameter of Topsides base plate is ~5 m (16.4 ft). To safely transfer these reaction forces, a grout with a compressive strength of 80 MPa (11,600 psi) with a 90% minimum contact area was required. Furthermore, the grout had to be flowable such that it could be placed without need for vibration due to limited access below the very large baseplate. The grout also had to completely fill the ~120 mm (4.7 in.) gap below the baseplate (after Topsides installation) to provide the requisite contact area.

Commercially available grout materials and placement methods were unable to fulfill these stringent requirements. As such, full-scale trials were conducted in Denmark using a specialty contractor to test various grout material enhancements and placement techniques to determine the most appropriate material and placement method.

The eventual grouting of the topsides connection was performed in several steps, as shown in Figure 9. In the first grouting stage prior to Topsides installation, a 130 mm (5.1 in.) thick grout layer was placed reaching the top of the tubular bearing base plate. After Topsides installation, the remaining gap was filled by grouting in two layers that were placed within a few hours of each other. These two steps were necessary in order to minimize formation of air bubbles, thus obtaining a contact area between that grout and baseplate of over 90%.

Figure 9-1 shows the tubular bearings at the completion of stage 1 grouting. They were installed at each support point to uniformly distribute the stresses (hence prevent local stress concentrations) during TS-GBS mating. They also provided temporary support to the Topsides (vertically by direct bearing and laterally by friction) until the grout could be placed and cured. In order to ensure that the tubular bearings had sufficient coefficient of friction to resist the lateral load (combination of wind load and the horizontal component of the Topsides weight when the platform is inclined), a special finish was applied on the upper half of the deformation tubes. Friction tests were conducted to ensure that the coefficient of friction was sufficient to resist the lateral load but also not too high to limit the amount of lock-in stress during Topsides-GBS mating. Horizontal displacement occurs during mating when the global Topsides deformation changes from sagging (when it is carried by the two barges, shown in Widiyanto et. al. (2020)) to hogging (when it is supported by the GBS). The number of tubular bearings per support was based on the calculated Topsides support reactions to obtain about the same total deformation at all supports.

Figures 9-2 and 9-3 show the movement of Topsides base plate during Topsides-GBS mating. Figure 9-4 shows the tight space around the TS-GBS connection, after the TS-GBS mating is completed and prior to Stage 2 grouting. This tight space made the grouting operation quite challenging. Figure 9-5 shows the deformation of tubular bearing after TS-GBS mating. Figure 9-6 shows a partition to divide the entire grouting area into smaller areas, to minimize the risk of air bubble formation. The partition was able to accommodate any vertical movement from the TS-GBS mating load transfer (and deformation of tubular bearings). Figure 9-7 shows a grout form prior to Stage 2 grouting. Figure 9-8 shows the top surface of layer 1 of stage 2 grouting and stinger used for grouting.

Full-scale Mock-ups

Full-scale mock-ups are small parts of GBS built at full-scale using the same procedures, equipment and materials as planned for the actual structure. In addition to regular and early review of design drawings, full scale mock-ups of several complex elements of the GBS were conducted and lessons-learned were fed back to the design team to make the design more constructible. The mock-ups also allowed an opportunity to train the work force on these complex operations prior to actual construction. A few important mock-ups (Figure 10) used were:

- Base slab with a thickness of up to 3.2m (10.5ft) containing multiple reinforcement layers (up to 10 layers per concrete face). Some of the main findings were:
 - Concrete could be placed/vibrated with no honeycombing or voids,
 - The full thickness could be placed in one layer eliminating horizontal construction joints,
 - Orthogonal horizontal reinforcement layout could be used instead of a more complicated radial layout
- Slipforming walls with high reinforcement and embedment density: Verified that all required reinforcement and embedments could be placed while still allowing the slipform to move at a reasonable rate.
- Topsides-GBS connection: Verified that the high-strength grout (80 MPa (11,600 psi)) would completely fill the space between the underside of topsides baseplate and the GBS ring-beam (described in section above).

- Post-tensioning duct grouting: Verified that the grout will completely fill the very long (80 m (262 ft)) vertical ducts as well as horizontal ducts.

Innovative Construction Joint in Base Slab

In order to improve the construction schedule for the GBS base slab, cost-effective and innovative vertical steel bulkheads with horizontal corrugations were used as formwork between the various sections of base slab (poured in 4 sections), as shown in Figure 11. The corrugated vertical steel panels were designed to be left-in place and detailed to ensure leak-tightness. Compared to conventional formwork that would require stripping and surface preparation after concrete had set (very time consuming for a thick slab with many layers of dense reinforcement), the use of left-in-place formwork resulted in a simpler and shorter construction period.

The corrugated steel-panel was needed because the commonly used expanded sheet metal could not transfer the high in-plane membrane forces and transverse shear forces that the 1.8 m (5.9 ft) and 2.5 m (8.2 ft) high construction joints were subjected to. The steel bulkheads were supported on a concrete strip foundation cast to encase the bottom layers of horizontal reinforcement. Ties anchored in the bottom layer of reinforcement were used to provide horizontal support. A steel mesh was used above the bulkheads to allow for the top layers of horizontal reinforcement to go through. Headed studs and steel ribs were welded to the bulkheads to resist in-plane forces and to enhance bonding between the bulkhead and the concrete. To ensure water tightness, a two-component low-viscosity epoxy was injected into the joint through hoses installed at several locations over the depth of the bulkhead.

SUMMARY

The Hebron platform is the most recent major offshore integrated oil drilling and production platform supported by a concrete gravity-based-structure (GBS). It was successfully installed in the Grand Banks (offshore Newfoundland) in June 2017. The design of the platform was challenged by arctic-like and extreme metocean conditions. This paper presents development of extreme loads on the GBS and describes novel design and construction techniques used, which resulted in a capital-efficient platform.

The magnitude of environmental loads for offshore structures are typically not specified in codes/standards and are more difficult to quantify. In design of Hebron GBS, advanced statistical analyses with a logic tree were utilized in developing iceberg impact loads and novel model tests were performed in developing wave impact loads.

Innovative non-linear finite element analysis (NLFEA) techniques were used to calculate internal forces for selected structural elements and load cases, such as for designing ice walls under 10,000-year iceberg impact load and shaft under wave impact loads. NLFEA resulted in more realistic internal forces, reduced reinforcement amounts, and improved constructability. In addition, a new crack width calculation method that accounts for the presence of a significant number of layers of transverse reinforcement was applied, which also further optimized the reinforcement quantities. Complex analyses to account for interaction among solid ballast, embedded pipes, and concrete structural components were utilized.

Several innovative slipforming techniques were employed including panels and yoke height that were taller than those used in the past to improve the slipforming rate. A specially designed conical slipforming rig with a star-girder arrangement was used to slipform the shaft that had a complex geometry change and curvature that was much larger than used for past GBSs. In order to minimize the risk of concrete lumps adhering to the slipform panels especially during hot weather conditions, cooling system on the slipform panel was utilized.

In order to ensure that high-strength would grout completely fill the space underneath one of the largest Topsides baseplate, grouting was performed in several steps. Full-scale mock-ups of several complex elements of the GBS were conducted and lessons-learned were fed-back to the design team to improve constructability, increase execution certainty, and improve construction safety. Innovative vertical steel bulkheads with horizontal corrugations were used as left-in-place and leak-tight formwork between the various sections of base slab.

REFERENCES

Canadian Standards Association. 2006. CAN/CSA-ISO 19900:06 Petroleum and Natural Gas Industries — General Requirements for Offshore Structures. 28 pp.

Canadian Standards Association. 2010. CAN/CSA-ISO 19906:10 Petroleum and Natural Gas Industries — Arctic offshore structures.

Norwegian Standard, NS 3473E:2003, "Concrete structures – Design and Detailing Rules," Standard Norge, Norway, 2003, 128 pp.

Oberlies, R., Khalifa, J., Huang, J., et al. 2014. Determination of Wave Impact Loads for the Hebron Gravity Based Structure (GBS). Presented at the 33th International Conference on Offshore Mechanics & Arctic Engineering, San Francisco, California, USA, 8-13 June. OMAE2014-23503.

Tistel, J., Eiksund, G. R., Hermstad, J., et al. 2015. Gravity Based Structure Foundation Design and Optimization Opportunities. Presented at the Twenty-fifth (2015) International Ocean and Polar Engineering Conference, Kona, Hawaii, USA 21-26 June. ISOPE-I-15-767.

Widianto, Khalifa, J., Younan, A., Karlsson, T., Stuckey, P., Gjørven, A. 2013. Design of Hebron Gravity Based Structure for Iceberg Impact. Proceedings of the Twenty-third (2013) International Offshore and Polar Engineering, Anchorage, Alaska, USA, 30 June – 5 July. ISOPE-I-13-036.

Widianto, Khalifa, J., Taborda, G., Bidne, K. 2016. Concrete Gravity-Based Structure: Construction of the Hebron Offshore Oil Platform. Concrete International, American Concrete Institute, 38(6):29-36.

Widianto, Khalifa, J., Åldstedt, E., Hæreid, K.O., Fosså, K.T. 2020. Design and Construction Overview of Offshore Concrete Gravity-Based-Structures: Past, Present, and Future. ACI Special Publication, ACI.

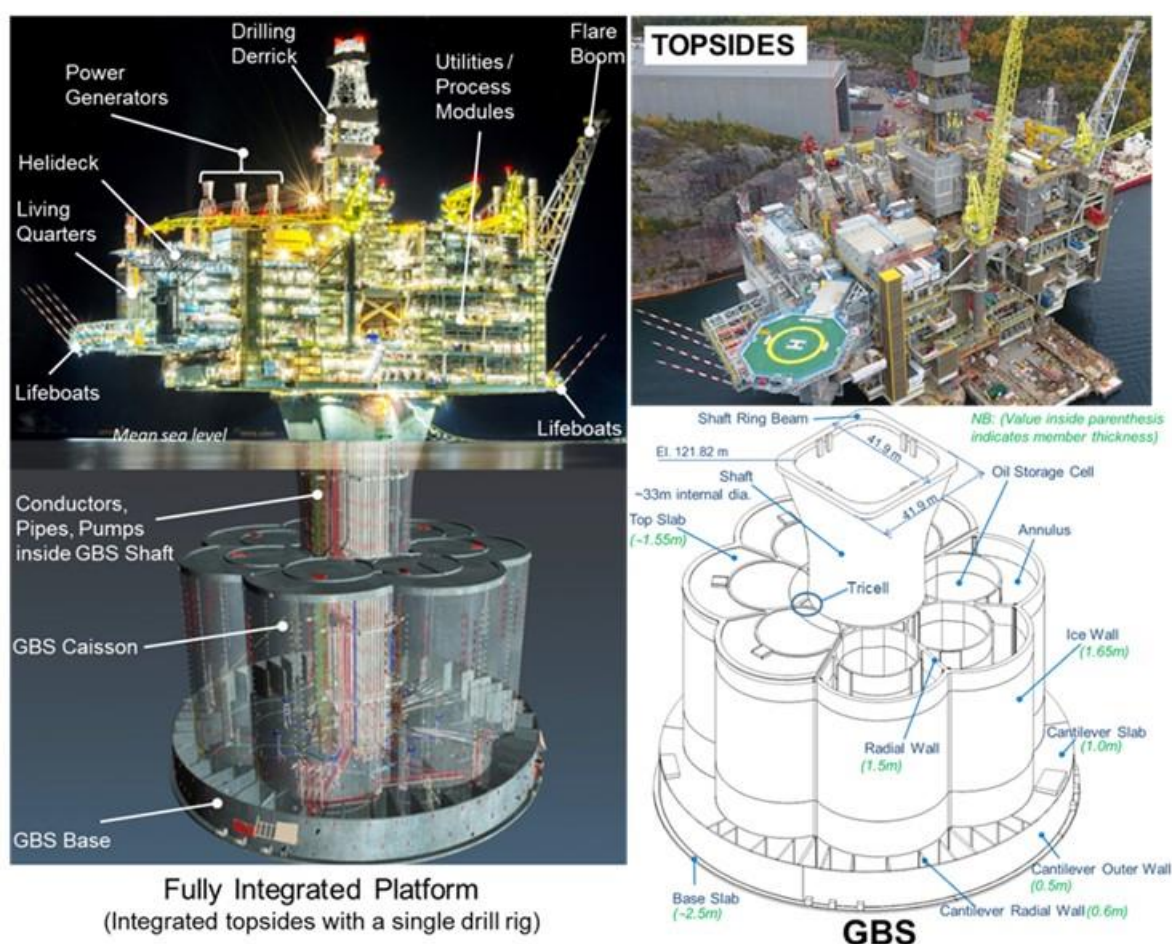


Fig. 1 – Hebron Oil Drilling, Production, and Storage Platform

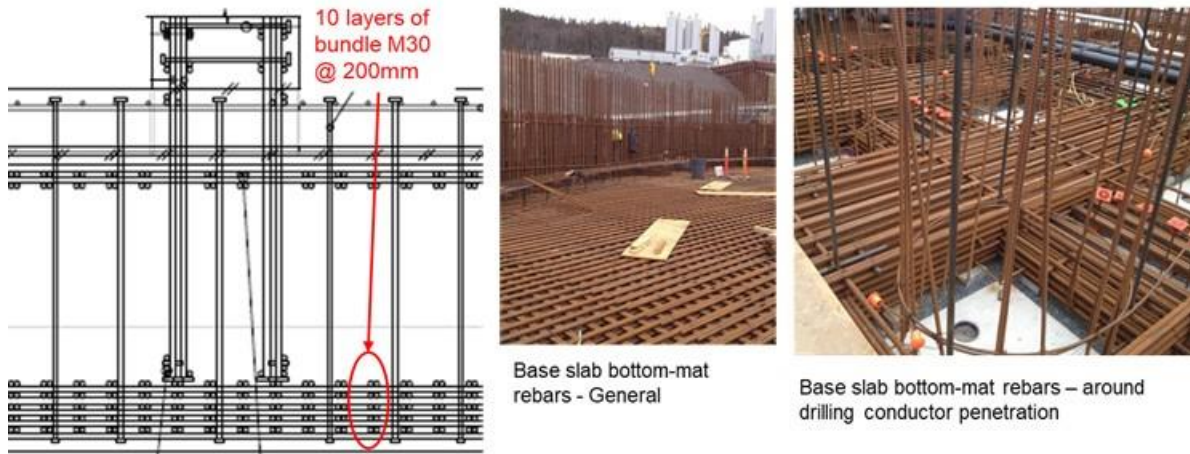


Fig. 2 – Base Slab Reinforcing Bars

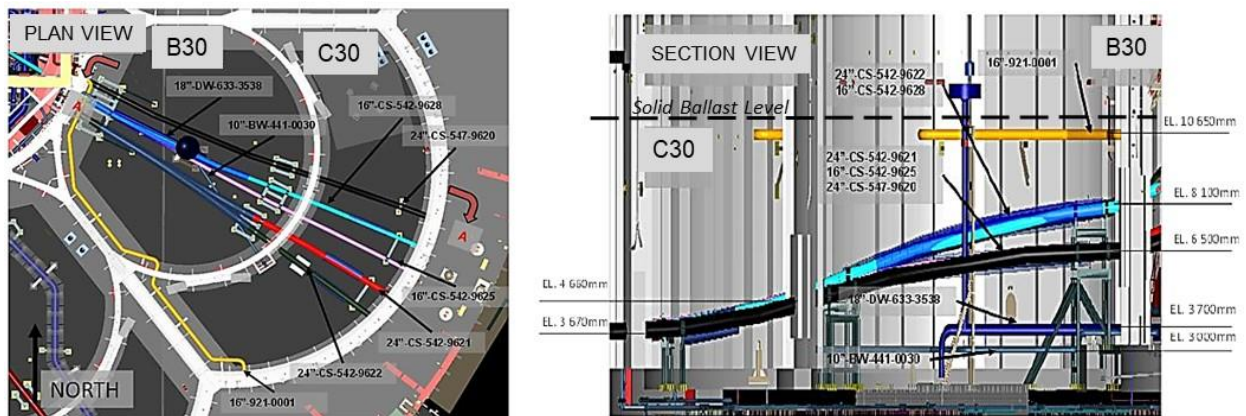


Fig. 3 – Embedded pipes within SB

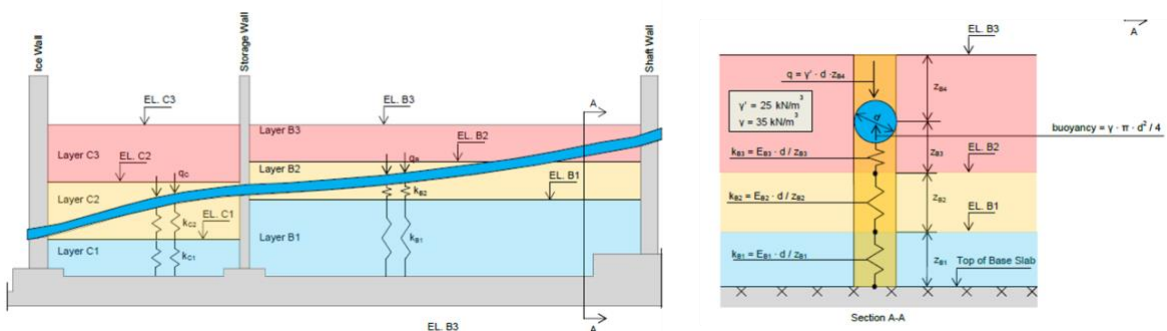


Fig. 4 – Illustration of incremental SB filling

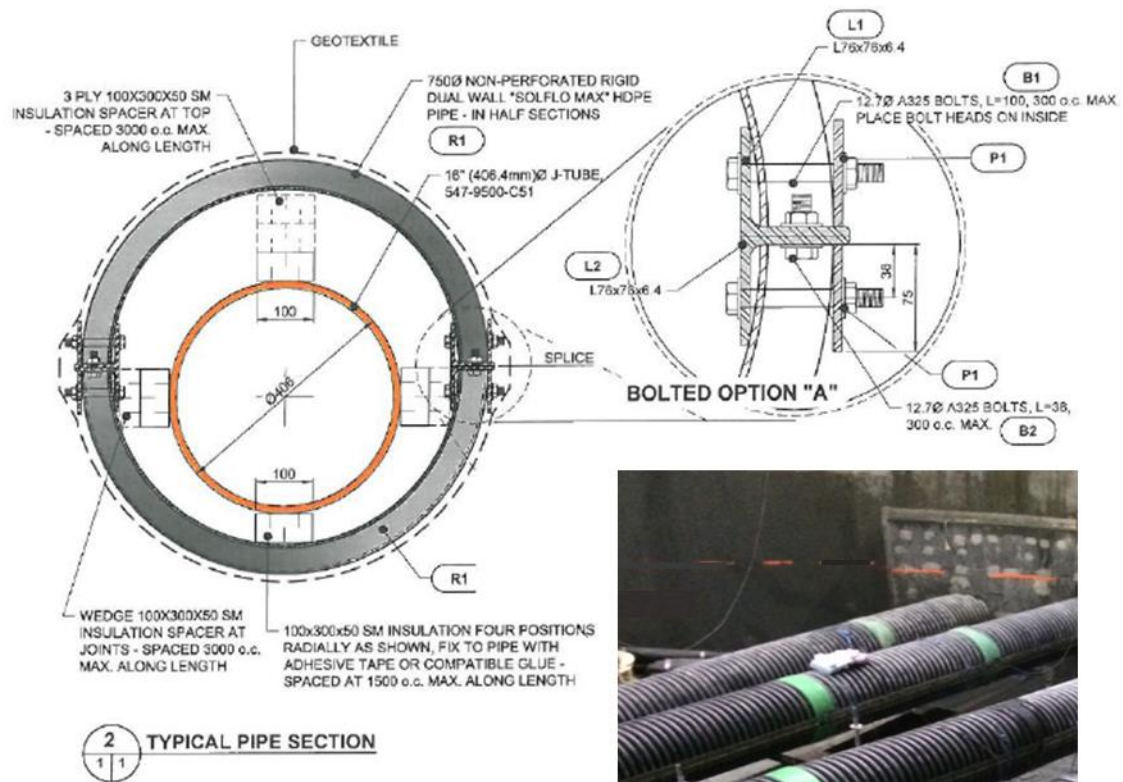


Fig. 5 – Example of pipe-in-pipe solution

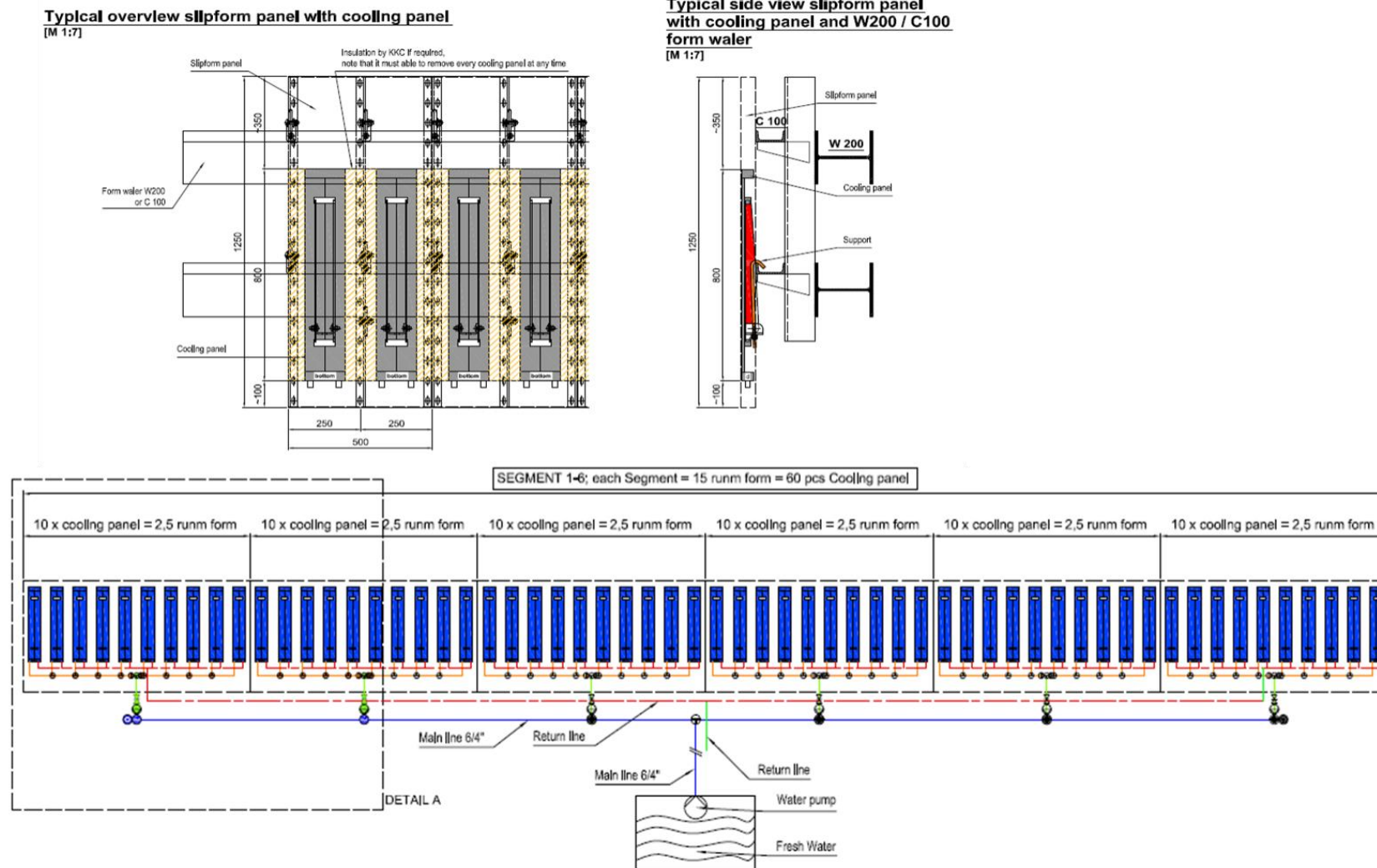


Fig. 6 – Sketch of the water cooling system of the slipform panels



Fig. 7 – GBS shaft unique geometry and star-girder arrangement for shaft slipforming