Concrete Gravity-Based Structure

Construction of the Hebron offshore oil platform

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The Hebron offshore oil development project consists of the following major components (Fig. 1):

- Reinforced concrete gravity-based structure (GBS);
- Topsides structure with all systems and equipment required to support drilling, processing, utilities, and living quarters; and
- Offshore oil loading system (OLS) with a looped pipeline and two separate loading stations about 2 km (1.2 miles) from the GBS.

The platform will be installed in a water depth of approximately 93 m (305 ft) on the Grand Banks, 340 km (211 miles) from St. John's, NL, Canada, and close to the existing Terra Nova, White Rose, and Hibernia platforms.

The GBS (Fig. 2) is designed to support the topsides structure and will rest on the ocean floor, held in place by gravity. In addition to resisting icebergs and other environmental loads, the GBS provides storage for 1.2 million barrels of crude oil and accommodates 52 drilling conductors, risers/J-tubes, and other mechanical outfitting systems. The GBS was constructed at Bull Arm (Fig. 3) in Great Mosquito Cove, approximately 150 km (93 miles) northwest of St. John's. The lower portion of the GBS (up to an elevation of 27.5 m [90 ft]) was constructed in a dry dock created by building a bund wall and dewatering the site behind it. Subsequently, the dry dock was flooded, the bund wall removed, and the GBS base (weighing about 180,000 tonnes [198,420 tons]) towed about 3 km (1.9 miles) to a deep water site (Fig. 4). At the deep water site, the GBS was held in place with nine mooring lines and the remaining construction was completed while the GBS was afloat.

The topsides structure was fabricated in modules at various Newfoundland and Labrador locations and in South Korea. These modules are being integrated at Bull Arm and the completed topsides structure will be mated with the GBS while it is floating at the deep water site. The mated

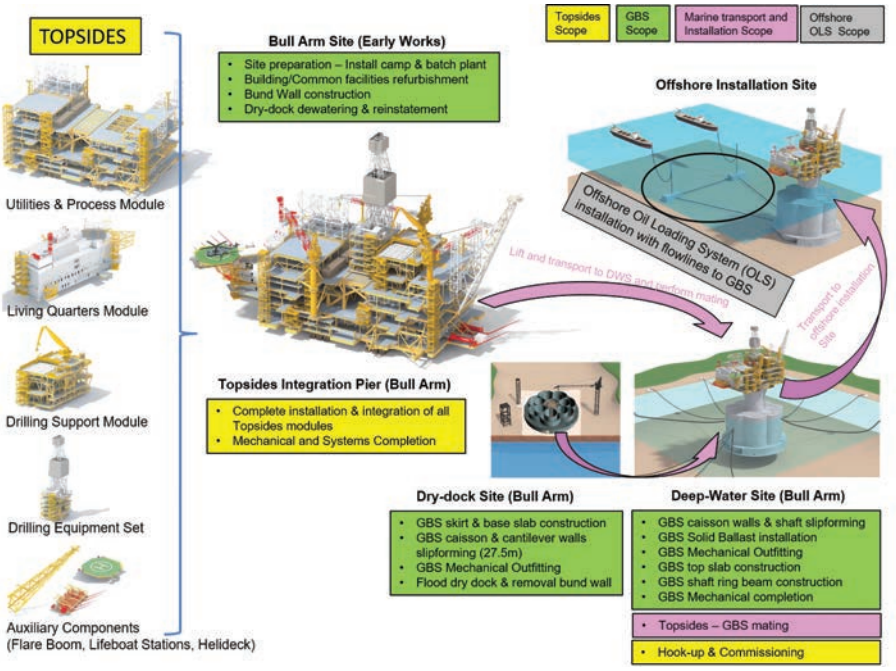
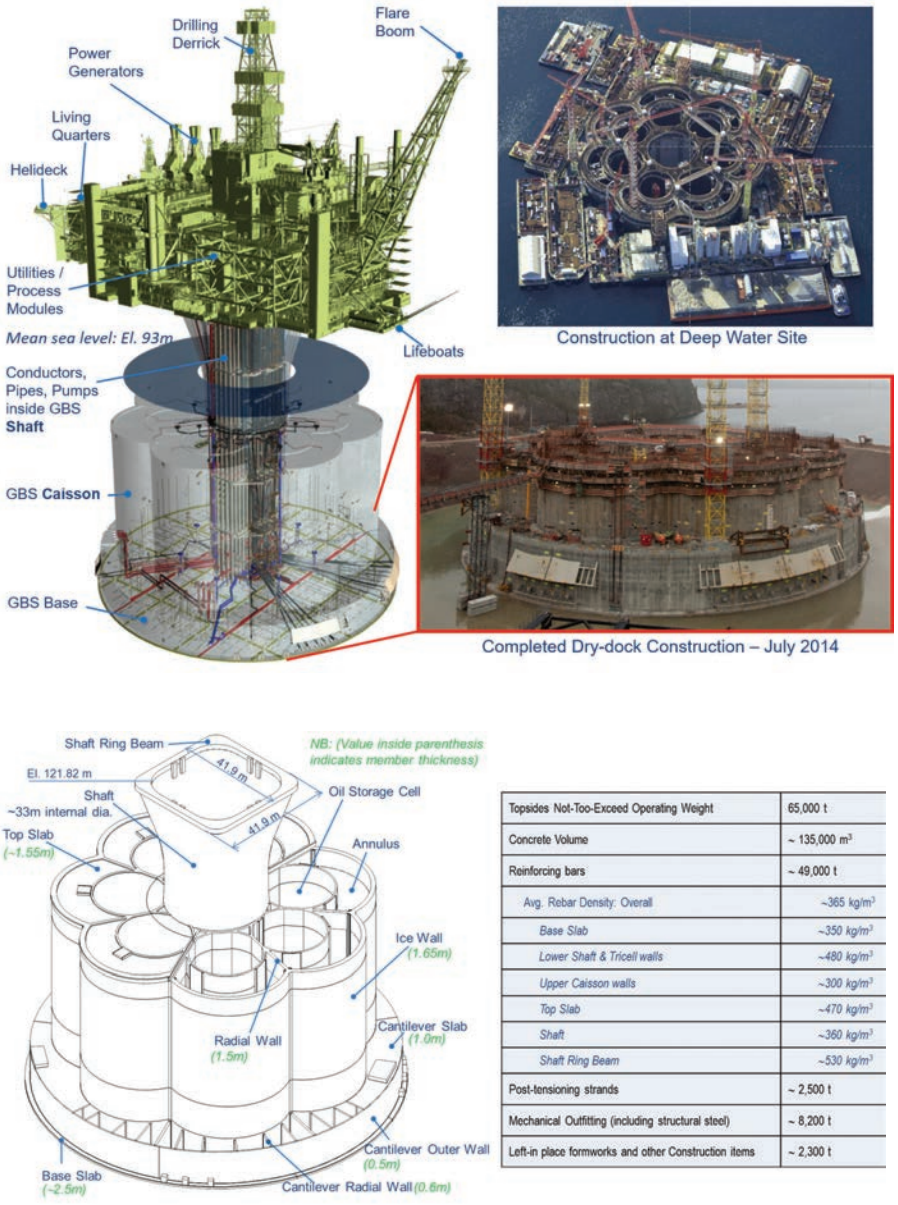


Fig. 1: Major components of the Hebron offshore oil development project



Topsides Not-Too-Exceed Operating Weight	65,000 t
Concrete Volume	~ 135,000 m³
Reinforcing bars	~ 49,000 t
Avg. Rebar Density: Overall	*~365 kg/m³*
Base Slab	*~350 kg/m³*
Lower Shaft & Tricell walls	*~480 kg/m³*
Upper Caisson walls	*~300 kg/m³*
Top Slab	*~470 kg/m³*
Shaft	*~360 kg/m³*
Shaft Ring Beam	*~530 kg/m³*
Post-tensioning strands	~ 2,500 t
Mechanical Outfitting (including structural steel)	~ 8,200 t
Left-in place formworks and other Construction items	~ 2,300 t

Fig. 2: The GBS was designed to support the topsides structure, resist iceberg impacts and other environmental loads, provide storage for 1.2 million barrels of crude oil, and accommodate various mechanical outfitting systems (Note: 1 m = 3.3 ft; 1 tonne = 1.1 ton; 1 m³ = 1.3 yd³; 1 kg/m³ = 1.7 lb/yd³)

platform will then be towed offshore and installed at the production location.

Construction of the GBS is expected to be completed in 2016. During the operation phase, crude oil stored in the GBS will be offloaded to tankers via the OLS and transported to market.

Unique Characteristics

Compared to typical buildings and bridges, there are several distinctive characteristics of the Hebron GBS that are challenging in design and construction:

- Massive size and complex geometry with a significant number of "Disturbed" or D-Regions;
- Various floating stages during construction and marine operations—tow-out from the dry dock, mating with the topsides structure, and tow-out of the completed platform to the installation location (about 300 nautical miles over 1 to 2 weeks) and final ballasting to set the platform on the seabed;
- Heavily stressed and reinforced—average reinforcement density of over 300 kg/m³ (19 lb/ft³) compared to 75 to 150 kg/m³ (5 to 9 lb/ft³) for typical concrete buildings and bridges. This is partly because the cross-sectional thicknesses have to be limited to minimize the weight of the GBS to allow it to float during the various construction and installation phases;
- Harsh environmental conditions leading to very large loads—the 100-year return period wave height of about 28 m (92 ft) was calculated to result in a design base shear of 1600 MN (359,700 kip) and local

Unique Terminology

Deformation tubes—Thick-wall steel tubes welded to a base plate, placed on the GBS below the topsides footings to act as a flexible bearing. These bearings plastically deform as load is transferred from topsides to GBS during mating, thereby eliminating unexpected higher-than-design forces at the connection points.

J-tubes—Pipes or conduits that have the shape of the letter "J" (consists of the bottom bend and vertical conduit). This allows the future pulling of a flow line or an electrical cable from the seabed through the J-tubes and connecting it to the topsides.

Pump caissons—Caissons within GBS that house crude oil, seawater, and fire water pumps. The pumps can therefore be pulled up into the topsides for servicing rather than entering the GBS shaft.

Shale chutes—Drill-cuttings disposal pipes that run from the drill platform in the topsides down through the GBS and exit the GBS above the seabed.

wave impact pressures up to 2.2 MPa (320 psi) over a 50 m^2 (538 ft^2) area; and the 10,000-year return period iceberg produced impact loads of up to 500 MN (1.5 MPa [220 psi] local pressure) and governed the design of the outer walls. The only other GBS designed to resist iceberg impact loading was Hibernia;

- Requirement for oil storage cells to be leak-tight—strict leak-tightness design criteria were specified for oil-storage cell walls and roof subjected to differential pressure and temperature (seawater at −2°C [28.4°F] and oil at 50°C [122°F]). This required significant amounts of post-tensioning as well as other special measures, such as liners, to prevent leakage;
- Significant number of access openings and pipe penetrations—replacement of reinforcement that needed to be cut at openings further increased the local reinforcement density to about 600 kg/m^3 (37 lb/ft^3) in some areas; and
- Support the topsides structure's operational weight of 65,000 tonnes (71,650 tons) at four connection points on a single shaft.

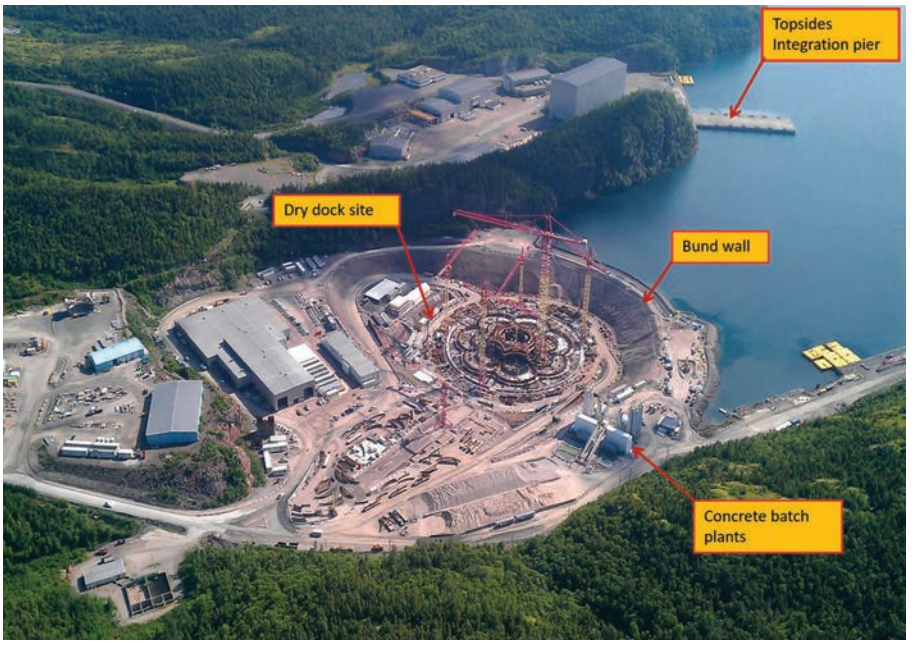


Fig. 3: The lower portion of the GBS (up to elevation of 27.5 m [90 ft]) was constructed in a dry dock created by building a bund wall and dewatering the site behind it, Bull Arm, NL, Canada

Fig. 4: After filling of the dry dock with water, the GBS was towed to the deep water site

Main Features

The GBS (Fig. 2) consists of a base, a caisson, and a single shaft supporting the topsides structure. The overall diameter of the base slab (130 m [430 ft]) was governed by stability requirements and soil-bearing capacity. The caisson houses seven oil storage cells, which are protected against icebergs by an exterior reinforced concrete wall (ice wall). The space between the ice wall and the storage wall (about 13 m [43 ft] wide annulus) was used for ballasting and was sized to satisfy buoyancy requirements. The shaft has an internal diameter of about 33 m (108 ft) and houses 52 drilling conductors, risers/J-tubes, and other mechanical outfitting systems.

The base and top slabs of the GBS were designed as unstiffened flat plates. The base plate thickness of up to 3.2 m (10.5 ft) was generally governed by the hydrostatic pressure resulting from the 116 m (380 ft) draft during mating with the topsides.

At the lower part of the caisson, cantilever walls were used to stiffen the edge of foundation and provide buoyancy during float-out from the approximately 16 m (52 ft) deep dry dock. The height of the GBS that could be constructed in the dry dock was limited by the dock's depth (the GBS draft had to be less than the dock depth).

All oil-storage cell walls were post-tensioned in both vertical and horizontal directions. The ice wall was post-tensioned in the vertical direction only. The post-tensioning ducts were completely filled with grout. The cell walls

project slightly above the top slab to provide space for the vertical post-tensioning anchorage, as anchorage within the top slab thickness would have resulted in clashes with the dense horizontal reinforcement in the slab.

The shaft flares out from a circular section at its base (optimal for minimizing wave loading) to a 41.9 x 41.9 m (161 x 161 ft) section at the top to meet the required layout to support the topsides structure. Each connection point for the topsides structure comprises sixteen 140 mm (5.5 in.) diameter pretensioned bolts. The connection points also include deformation tubes designed to deform and minimize local stress concentrations as the weight of the topsides structure is transferred to the GBS during mating. To complete the connections with the topside structure, high-strength grout (up to 95 MPa [13,800 psi]) is placed below the topsides base plate and the bolts are tensioned.

Approximately 222,000 tonnes (244,710 tons) of solid ballast (approximately 10 m [33 ft] thickness) was placed at the bottom of oil storage and annulus cells. This provides floating stability during the towing of the mated platform as well as improved sliding resistance of the GBS after installation on the seabed.

Mechanical Outfitting

The mechanical outfitting systems (total weight about 8200 tonnes [9040 tons]), Fig. 5, include piping, equipment, and structures required during both the temporary and permanent phases of the GBS and its future removal (decommissioning). These are risers/J-tubes, pump caissons, mechanical and electrical systems, and five structural deck frames inside the shaft at elevations 26, 50, 71, 98, and 118 m (85, 164, 233, 322, and 387 ft) to support the piping systems.

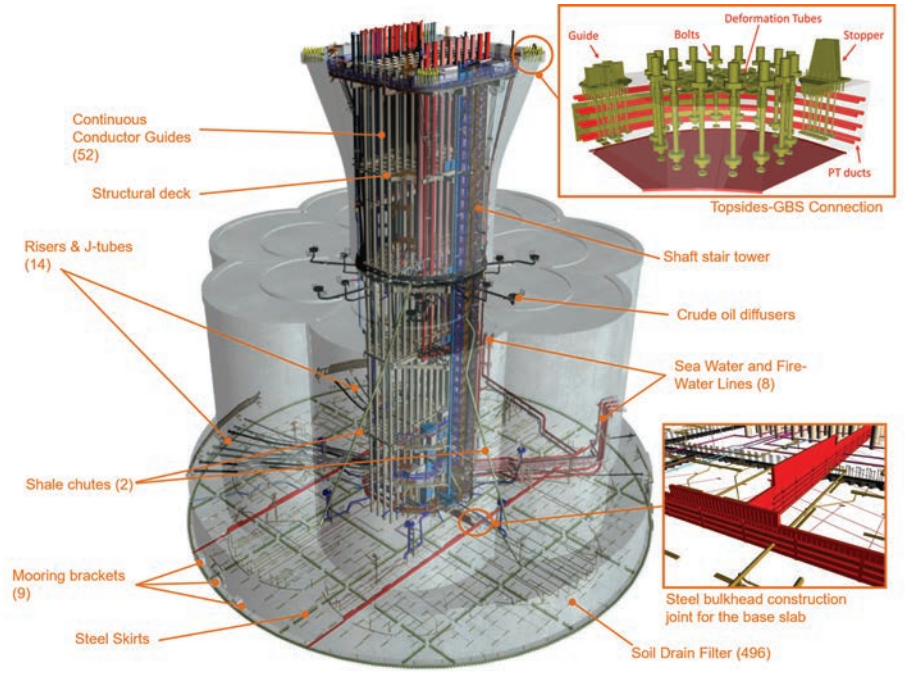


Fig. 5: Mechanical outfitting systems with total weight of about 8200 tonnes (9040 tons)

Analysis and Design

The overall analysis was based on a Global Finite Element Analysis (GFEA) using solid elements. Most analyses were based on linear elastic material behavior, which allowed the use of the superposition principle to determine internal forces at the various locations within the GBS. The nonlinear behavior of reinforced concrete was accounted for during post-processing via the code-checking process.

In addition to strut-and-tie models, local nonlinear finite element analyses were used for design of "D" regions to properly account for redistribution of forces after concrete cracking.

Reinforced concrete was designed based on Norwegian Standard NS 3473:2003[1] and in accordance with the limit states approach specified in ISO 19900-06[2] and 19903-07.[3] Concrete materials were in accordance with the requirements of CAN/CSA A23.1/23.2-09.[4]

The GBS was designed for a 50-year life cycle, including the ability to be removed (remain structurally intact with adequate floating stability) at the end of its functional life.

Design for construction phases

Unlike normal building structures, the various construction/temporary phases imposed significant forces on the GBS in addition to the normal operating phase (as a completed structure). Many parts of the GBS were governed by loading during the construction/temporary phases. For example, the base slab design was governed by water pressure during topsides/GBS mating, while the caisson walls close to the construction joint at an elevation of 27.5 m (90 ft) were governed by the tensile stresses during tow-out from the dry dock to the deep water site. Different GFEA were performed to properly capture internal stresses resulting from the different GBS construction phases. It was therefore crucial to establish construction sequences prior to the start of the analysis/design process.

Design for iceberg impact

Iceberg impact loads on the GBS were developed using a state-of-the-art probabilistic analysis that incorporated Monte Carlo simulations (to capture Type I uncertainties) in combination with Logic Tree Analysis (to capture Type II uncertainties).[5]

This method is similar to the industry approach used for defining seismic hazards where, in the absence of data, expert opinion may significantly differ on important parameters. It has the advantage of ensuring stability of the derived loads because the range of

values for the unknown parameters is covered in the logic tree.

The derived 10,000-year return period iceberg impact load was based on no iceberg management and used information on iceberg size, drift speed, strength, shape, and other variables.

To account for the redistribution of internal forces due to cracking of concrete, an advanced nonlinear finite element analysis (NLFEA) was used for design of ice walls. The use of NLFEA resulted in a reduction of reinforcement of approximately 3500 tonnes (3860 tons), a reduction in post-tensioning cables of approximately 700 tonnes (770 tons), and improved constructibility relative to design using linear analysis.

Design for waves

Wave loads were determined using a combination of wave model tests (1:50 scale) and analytical methods, such as diffraction theory using WADAM, a hydrodynamic analysis software package for calculating wave structure interaction for fixed and floating structures of arbitrary shape. Dynamic analyses were also employed to account for the effect of wave impact on the shaft and the resulting inertial forces of the topsides. Separate wave model tests were run to evaluate wave impact loading on the GBS shaft and on the underside of the topsides.[6]

NLFEA accounting for concrete cracking was used to determine more realistic internal forces at the base of the shaft, which resulted in optimized vertical reinforcement at that location.

Design for seismic events

While the overall rate of seismicity in eastern Canada is low, infrequent earthquakes up to moment magnitude of M = 7.3 (the size of events in terms of how much energy is released) have been recorded in this region (for example, the 1929 Grand Banks earthquake).

As such, the platform was designed to withstand seismic events at two levels: Strength Level Event and Ductility Level Event, associated with 300- and 3000-year return periods, respectively.

The platform was analyzed using an integrated seismic soil structure interaction approach. Accelerations at key locations within the platform were provided to the GBS and topsides engineering, procurement, and construction (EPC) contractors to perform their own separate dynamic analysis. This approach resulted in a more efficient and consistent design process allowing each EPC contractor to tailor the level of detailing for each component of interest.[7]

Foundation design

Failure modes considered in design were bearing, overturning, and sliding (which governed). To increase the sliding resistance, 500 mm (20 in.) deep steel skirts (Fig. 6), which penetrated the weaker topmost soil layer, were installed below the base slab. The skirts were fabricated from 10 mm (0.4 in.) thick corrugated steel plate and arranged in an orthogonal pattern bounded by a circular shape along the outer edge. The skirts were welded into a horizontal top plate, which was anchored to the base slab with approximately 650 and 800 mm (25.6 and 31.5 in.) long T-headed bars. Furthermore, the GBS base slab was cast directly against a coarse aggregate bed in the dry dock to increase roughness between the slab and the seabed.

Under-base grouting was not needed because the geophysical survey data indicated an almost-flat seabed. As a result, the base slab was designed to resist local peak soil pressures, which in turn governed reinforcement density for several localized areas.

The underside of the base slab was fitted with soil drain filters (connected to the sea) to prevent possible pore pressure buildup in the soil under the GBS.

Materials

Concrete

Concrete mixture design criteria were as follows:

- Compressive strength of 65 MPa (9430 psi);
- Maximum water-cementitious materials ratio (*w*/*cm*) of 0.4;
- Cementitious material content between 360 and 450 kg/m^3 (610 to 760 lb/yd^3);
- Maximum silica fume and fly ash content of 8 and 30% of total cementitious material, respectively;
- Chloride diffusion coefficient (per ASTM C1556, "Standard Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion") of 4×10^{-12} m^2/s;
- High slump of 240 mm (9.5 in.) to deal with congested reinforcement;
- Resistant to freezing and thawing for the splash zone (2 to 4% air entrainment); and
- Low heat of hydration (large section thicknesses).

Two identical independently operated, fully automatic batching plants were used for concrete production at dry dock and deep water site locations. The concrete batch plants were installed on shore during the dry dock construction phase and later relocated to a barge for the deep water site construction phase. Similarly, aggregate and cement were stored onshore initially, then moved to barges for the deep water site phase.

Fig. 6: Steel skirts with T-headed bars were installed below the base slab to increase sliding resistance

Solid ballast

Solid ballast with a specified density of 3500 kg/m^3 (220 lb/ft^3) was placed as a slurry mixture comprising iron ore, fly ash, cement, and high-range water-reducing admixtures. All solid ballast installation was performed at the deep water site using concrete pumps.

The solid ballast was designed to have sufficient stiffness to ensure stability during marine operations (deep submergence test and towing to the field) but still be flexible enough to prevent large lateral pressures on the GBS walls and minimize loads on the piping embedded in the solid ballast.

Reinforcement

Grade 500W weldable deformed bars per the requirements of CSA A23.1-09 were used for reinforcement. T-headed bars were used to improve anchorage, eliminate hooks, and reduce congestion. The minimum concrete cover for steel reinforcement (including stirrups) was 50 mm (2 in.) in the splash zone and 40 mm (1.6 in.) in the submerged zone.

Prestressing strands conforming to ASTM A416/A416M, "Standard Specification for Low-Relaxation, Seven-Wire Steel Strand for Prestressed Concrete," and ASTM A722/A722M, "Standard Specification for High-Strength Steel Bars for Prestressed Concrete," were used.

Mechanical outfitting

Carbon steel, high-density polyethylene (HDPE), 6Mo, and titanium were used for various piping systems, mainly depending on environmental exposure (submerged in seawater, located in the splash zone, or located in the atmosphere), service temperature, and service life (temporary or permanent). Additional protection for some pipes was provided by applying epoxy-based paint. Metallic pipes were protected with thermal spray aluminum in the splash zone.

Construction

GBS was constructed in the sequences shown in Fig. 7. After the steel skirts were installed, the base slab was placed in four sections. The concrete was pumped from the batching plants and discharged using placement booms at the slab location.

Cost-effective and innovative vertical steel-panel bulkheads with horizontal corrugations (Fig. 8) were used as formwork between the sections because expanded sheet metal was insufficiently robust for a 1.8 and 2.5 m (5.9 to 8.2 ft) high construction joint subjected to high in-plane membrane forces and transverse shear. The steel bulkheads were supported on a concrete strip foundation that was cast to a height above the bottom reinforcement layers. A steel mesh was used above the bulkheads to allow access to the top layers of reinforcement. Headed studs and steel ribs were welded to the bulkheads to resist in-plane forces and to ensure proper bonding between the bulkhead and the concrete. To ensure watertightness, a two-component low-viscosity epoxy was injected into the joint through hoses installed at several locations over the depth of the bulkhead.

All walls were constructed using the slipforming technique—formwork panels were continuously moved upward using hydraulic pumps and yokes. This approach allows uninterrupted concrete placement, reinforcing bar installation, and minor surface repair. Slipforming allowed walls with high reinforcement density (Fig. 9) to be placed cost effectively, minimized the construction schedule, and improved leak-tightness as most construction joints were eliminated.

The caisson was constructed in three sequences (Fig. 7). Up to an elevation of 27.5 m, the caisson walls were slipformed in the dry dock in two sequences: central shaft and tricells, and storage cells and ice walls. The rest of the walls (with an elevation of 27.5 to 71 m) were slipformed in one continuous placement while the GBS was floating at the deep water site. This is believed to be the second largest slipforming operation in history, incorporating approximately 15,000 tonnes (16,530 tons) of reinforcing bar and about 50,000 m^3 (65,400 yd^3) of concrete over a 34-day period. The formwork used for this deep water site slipforming would stretch over 2 km (1.2 miles).

Deep water site construction required 12 support barges stationed around the GBS, as shown in Fig. 10 (including material lay-down barges, batch plants barge, access and office barge, chain tension barges, and several shuttle barges). In addition,

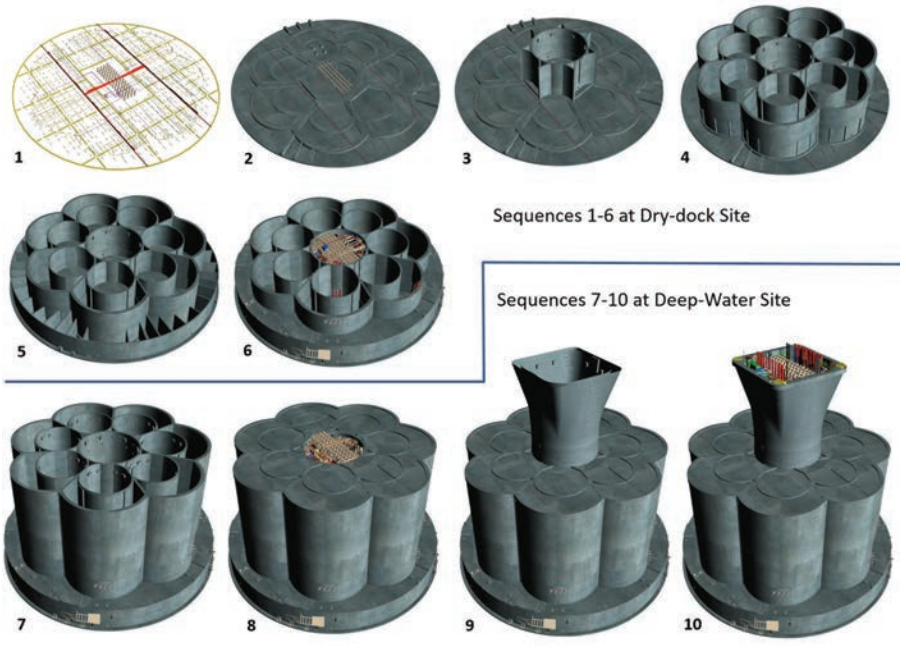


Fig. 7: Construction sequences for dry-dock and deep-water sites

Fig. 8: Steel-panel bulkheads with horizontal corrugations were used as a form between sections

Fig. 9: Slipforming of walls with high-density reinforcement helped to minimize schedule and improve leak-tightness by eliminating most construction joints

shuttle boats and passenger ferries were used to transport materials and personnel.

Over 20 million construction hours have been executed during construction of the GBS without a single lost time injury, which is an outstanding achievement given the construction complexity, harsh winter weather conditions, and work over water. This was only possible due to rigorous planning of the work and full buy-in and involvement of the craft workers in the safety program.

Acknowledgments

The authors are merely acting as chroniclers of the design and construc-

Fig. 10: Construction at deep-water site required 12 support barges stationed around the GBS

tion activities. Full credit is deserved by all the personnel who worked on the design and construction of the platform and successfully met design challenges on this project.

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Note: Additional information on the ASTM standards discussed in this article can be found at **www.astm.org**.

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