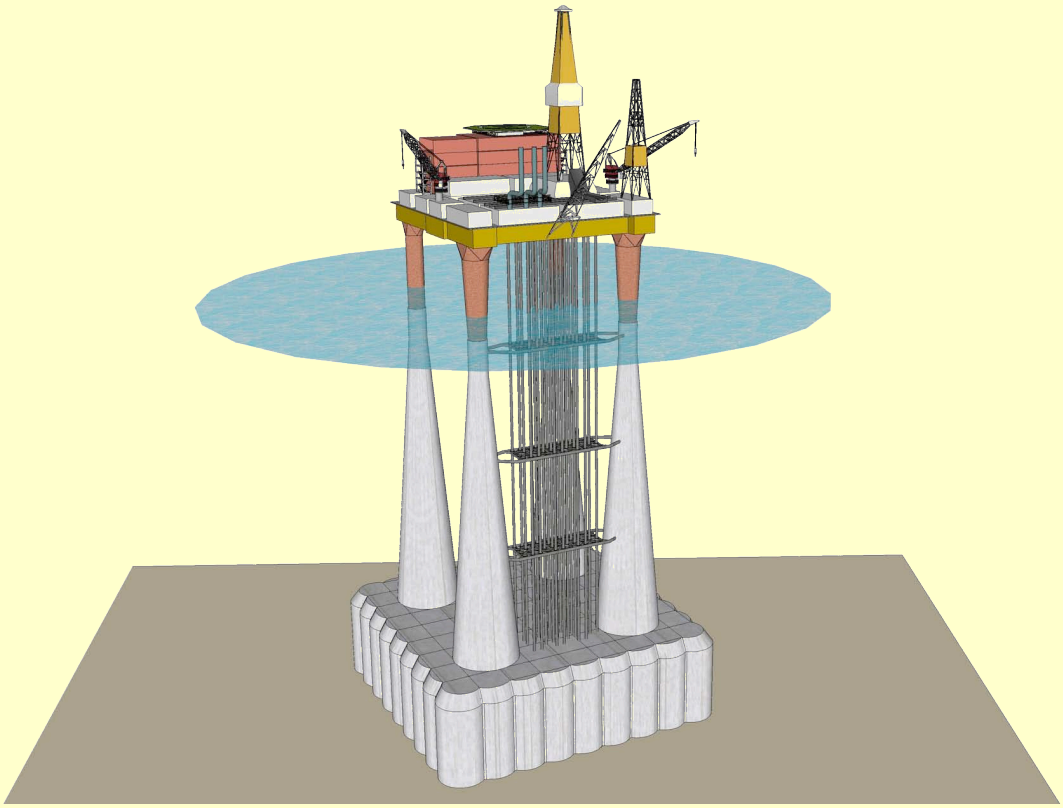
FairfieldEnergy

Dunlin Alpha Decommissioning



Concrete Gravity Base

In Situ Decommissioning

Options for Derogation

November 2011

Dunlin Alpha Decommissioning

Fairfield Document Approval Record

Document Title and Issue Date
Dunlin Alpha Decommissioning Concrete Gravity Base In Situ Decommissioning Options for Derogation First issue date: 28 November 2011

First Approver	Company Position	Signature	Date
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1. Executive summary

This report has been prepared by Fairfield Energy, operator of the Dunlin Cluster of fields, to consider the options for leaving the Dunlin Alpha concrete gravity base (CGB) wholly or partly in place at its current location. These concepts have been considered as part of a possible platform decommissioning programme after cessation of hydrocarbon production from the Dunlin field, which is not anticipated until 2018 or later. It is recognised that such decommissioning options would require derogation to be granted by the UK regulator, the Department of Energy and Climate Change.

As it is not possible to refloat the Dunlin Alpha platform and tow it to another location, any attempt to deconstruct the CGB would have to be carried out offshore at the current location in the field. A separate report presents the technical analysis related to refloating the platform.

A further report evaluates the possibility of removing the CGB by completely deconstructing it on location and returning the materials to shore for recycling or disposal. That report concludes that it is not possible, for a combination of technical feasibility and health and safety reasons, to achieve complete removal of the CGB. It should be noted that, to date, other large concrete platforms that have been decommissioned have been granted derogations to be left in place (with topsides removed) as no viable methods have been found to deconstruct them on location in a controlled way.

Therefore, this report presents the options for wholly or partly leaving the CGB in place at its current location. These options can be viewed as derogation cases under the decommissioning criteria set out in OSPAR Decision 98/3.

Fairfield Energy has investigated three options for leaving the CGB wholly or partly in place. Two of these options, involving controlled demolition of the CGB legs and leaving the resulting debris on the seabed, have been eliminated as they do not comply with current position of the UK regulator.

A third option does meet the criteria which could be considered for derogation. For this option, the CGB, with topsides and all external steelwork removed, would be left wholly in place, submerged to 8m below sea level. One or more of the four CGB legs would be fitted with extension towers reaching above the sea surface to carry navigation lights, in accordance with the requirements of the Northern Lighthouse Board. Fairfield Energy would establish a Legacy Trust to provide ongoing funding for the management and maintenance of the navigation lights.

The structure would be marked on relevant nautical charts and the 500m radius exclusion zone afforded to offshore oil and gas installations would be retained.

2. Introduction

2.1 Decommissioning options

The Dunlin cluster of fields, which includes the Dunlin field and its subsea satellites Osprey and Merlin, is located in the UK North Sea, some 500km north-northeast of Aberdeen, and is operated by Fairfield Energy on behalf of itself and MCX, a subsidiary of Mitsubishi Corporation. Details of the fields and the facilities are given in Appendix A.

The Dunlin Alpha platform, known as Dunlin A, came into operation in 1978 and acts as the production hub for the fields. Dunlin A is a concrete gravity base (CGB) structure supporting a steel topsides deck and production facilities, shown in Figure 2.1 below. Section 3 provides a brief description of the Dunlin A platform.

Figure 2.1a Dunlin A platform

When an offshore installation has reached the end of its economic life as a production facility, it is required to be decommissioned. The UK has a comprehensive regime controlling the decommissioning of offshore oil and gas installations, which favours re-use, recycling or final disposal on land of offshore facilities. These provisions are requirements of European Union Directives, UK legislation and the OSPAR Commission (Ref 1). It is not anticipated that Dunlin A will cease production before 2018 at the earliest. However, as a reasonable and prudent operator, Fairfield Energy is currently engaged in determining and evaluating its decommissioning commitments.

For the Dunlin cluster of fields, the decommissioning of the Dunlin A CGB, which weighs 320,000 tonnes, is the most significant area of decommissioning activity. Wider decommissioning issues related to Dunlin A, Osprey and Merlin,

for example well abandonment and the decommissioning of pipelines, subsea equipment and the removal of the Dunlin A platform topsides, will be addressed in the Decommissioning Programmes for the fields. Environmental Impact Assessments (EIAs) will also be produced for the decommissioning of the fields' facilities and pipelines.

Fairfield Energy is considering seven options for decommissioning the CGB, six of which were presented to stakeholders on 21 January 2010 in Aberdeen as part of the public consultation process; a seventh option was been added in July 2011. These options are described in Appendix B.

This report focuses on three of the seven options, each of them based on leaving the CGB wholly or partly in place, with the topsides removed. These options are referred to as in situ decommissioning.

Separate reports address the other four options and can be viewed at http://www.fairfield-energy.com/pages/view/decommissioning.

One of these is a technical report on the possibility of refloating the CGB prior to taking it to another location (Ref. 2). That report concludes that the CGB cannot be refloated due to technical challenges and associated risks. Therefore any CGB deconstruction work that might be possible for in situ decommissioning would have to be performed offshore in the Dunlin field.

Another of the reports has concluded that the complete removal of the CGB is not possible due to a combination of technical feasibility and health and safety reasons (Ref. 3). The latter report should be read in conjunction with this report, to provide a full technical background to methods for cutting the legs of the CGB.

This report is structured to present a short introduction to the relevant legislation covering offshore decommissioning, an overview of the Dunlin A platform, the options for leaving it wholly or partly in place, and an assessment of those options with conclusions drawn. The findings of independent expert consultant Atkins in support of this assessment are included in Appendices C and D. Details of Atkins decommissioning capability are included in Appendix E.

2.2 OSPAR Guidance

The OSPAR Convention is the current legal instrument guiding international co-operation on the protection of the marine environment of the North-East Atlantic. Work under the Convention is managed by the OSPAR Commission, made up of representatives of the Governments of 15 Contracting Parties and the European Commission, representing the European Community (Ref. 1). OSPAR produces decisions, recommendations and other agreements to act as guidance relating to marine operations, including decommissioning of oil and gas installations.

OSPAR Decision 98/3 covers the disposal of disused offshore installations (Ref. 4). Decision 98/3, which came into force in February 1999 and has been accepted by the UK Government, states that: '*Reuse, recycling or final disposal on land will generally be the preferred option for the decommissioning of offshore installations in the maritime area.*' Decision 98/3 also states that: '*The dumping at sea, and the leaving wholly or partly in place of disused offshore installations is prohibited.*'

However, the Decision also recognises that the decommissioning of concrete installations is likely to present particular problems, and therefore, where it can

be shown '*that there are significant reasons why an alternative disposal method is preferable to re-use or recycling or final disposal on land'*, a permit may be issued for a concrete installation to be '*dumped or left wholly or partly in place*'. In these circumstances, the '*competent authority*' has the power to grant an exemption from the general requirements. This exemption, or permit, is known as derogation.

In this respect, the competent authority in the UK is the Department of Energy and Climate Change (DECC). DECC provides the UK industry with guidance on the decommissioning process (Ref. 5) to help operators to comply with the Decommissioning of Offshore Installations and Pipelines under the Petroleum Act 1998, which addresses the process for derogation application under OSPAR 98/3. However, it should be noted that under this UK legislation, DECC does not accept that concrete installations can be dumped at sea, either at their original location or elsewhere.

OSPAR defines a concrete installation as being an offshore installation constructed wholly or mainly of concrete. Dunlin A is supported by a 320,000 tonnes CGB and falls within this definition.

DECC guidance on decommissioning also states: *'As with other installations, the topsides of concrete installations must be returned to shore for re-use, recycling or disposal'.* Fairfield Energy fully accepts this obligation, hence the Dunlin Alpha Decommissioning Programme is being prepared in compliance with the presumption that the topsides will be removed and returned to shore for re-use, recycling or disposal.

2.3 IMO Guidance

The International Maritime Organisation (IMO), headquartered in London, sets the standards and guidelines for the removal of offshore installations worldwide. The 1989 IMO Guidelines require the complete removal of all structures in water depths less than 100m and weighing less than 4000 tonnes. Those structures in deeper waters can be partially removed, leaving a minimum 55m of clear water above the structure for the safety of navigation. Many of these IMO guidelines have been embraced by OSPAR Decision 98/3 in respect to offshore installations, as defined by the UK Petroleum Act 1998, but some remain relevant, namely:

- An unobstructed water column of at least 55m must be provided above the remains of any partially removed installation to ensure safety of navigation.
- The position, surveyed depth and dimensions of any installation not entirely removed should be indicated on nautical charts and any remains, where necessary, properly marked with aids to navigation.
- The person responsible for maintaining the aids to navigation and for monitoring the condition of any remaining material should be identified.
- It should be clear where liability lies for meeting any future claims for damages.

3. Dunlin A platform

For reference, some of the key facts about the platform and CGB are summarised below. A detailed description of the Dunlin A platform is given in Appendix A.

3.1 Platform structure

Design and construction of the Dunlin A CGB structure was carried out by the Anglo Dutch Offshore Concrete (ANDOC) contractors' consortium in The Netherlands during the 1970s. The method of platform construction is described in Appendix A. The Dunlin A platform was installed in 1977 and, after the drilling of initial wells, oil production began in 1978. The CGB was not designed to be refloated for eventual removal.

The platform sits in 151m of water. To give an appreciation of scale, Figure 3.1a shows a graphic representation of the platform in comparison with the Big Ben clock tower in London, which is 96m high.

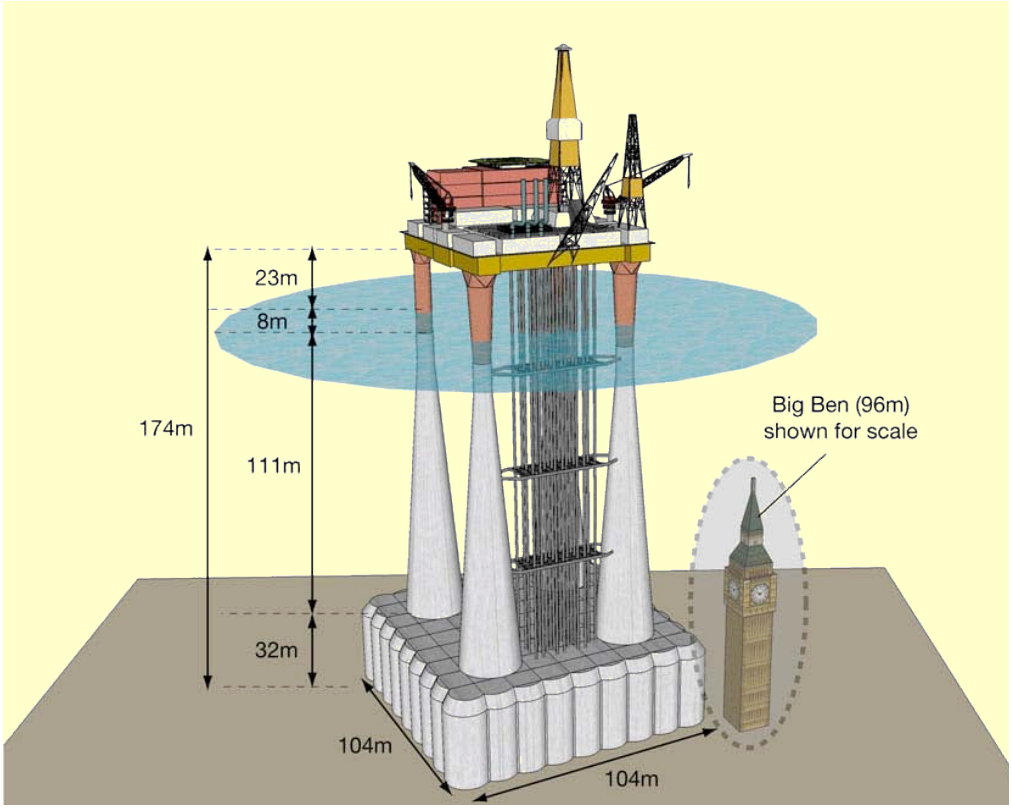

Figure 3.1a Dunlin A compared with Big Ben for scale

Figure 3.1b below shows the main components of the platform.

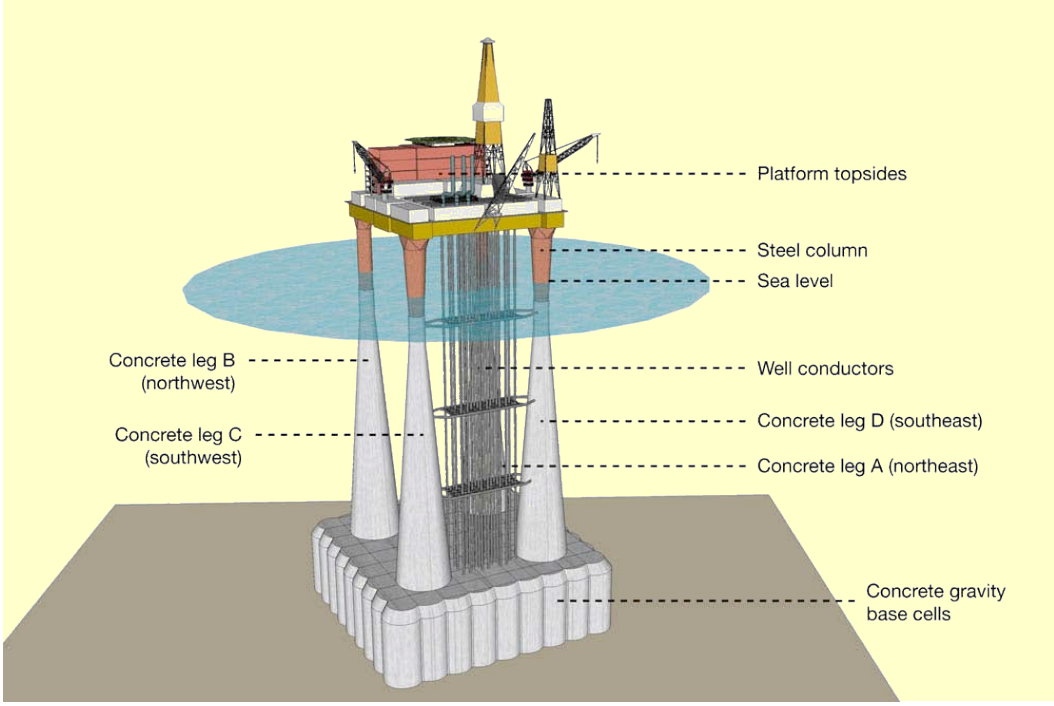

Figure 3.1b Dunlin A platform main components

The main components of the Dunlin A platform are:

- A CGB support structure sitting on the seabed. The CGB has a square base from which four concrete legs rise up to 8m below sea level.
- Four steel columns, attached to the tops of the concrete legs at 8m below sea level, rising to 23m above sea level. The concrete legs and steel columns are hollow and are designed to be maintained as dry spaces for piping and equipment.
- A steel deck and modular topsides, supported on the steel columns.
- Well conductors to convey hydrocarbons from the Dunlin reservoir to the topsides. The conductors are supported by three steel guide frames between Legs C & D.

The Dunlin A CGB weighs approximately 320,000 tonnes, including internal equipment in the legs and solid ballast in the CGB base, while the topsides weighs a further 20,000 tonnes. It is anticipated that the platform's topsides will be removed by lifting it from the CGB as one structure or in parts. While the offshore industry's current maximum heavy lift vessel capability is approximately 14,000 tonnes, there are plans to develop lift concepts with up to 40,000 tonnes capacity in the time scale envisaged for the decommissioning of Dunlin, which would be capable of removing the topsides in a single lift.

As the CGB cannot be lifted or refloated for towing to shore, Fairfield Energy is considering in situ decommissioning of the CGB by leaving the concrete structure wholly or partly in place.

3.2 CGB legs and columns

The CGB base is 104m square and 32m deep. This is subdivided internally by concrete walls into 81 cells, 75 of which were used to assist oil-water separation until 1995, after which the platform's operating practice changed and these cells were no longer used. The other six cells in the base are used for the cooling of the well conductors using circulated seawater. A detailed description of the base is included in Appendix A.

Rising up from the roof of the base cells are four reinforced concrete legs, each 111m high. These reduce in outside diameter from 22.6m at the bottom to 6.6m at the top, where they join the steel columns at 8m below sea level. The legs are designed as hollow shafts, with concrete walls generally being 700mm thick but increasing to 1200mm at the top and the bottom. Each of the concrete legs weighs approximately 7600 tonnes. See Figure 3.2a.

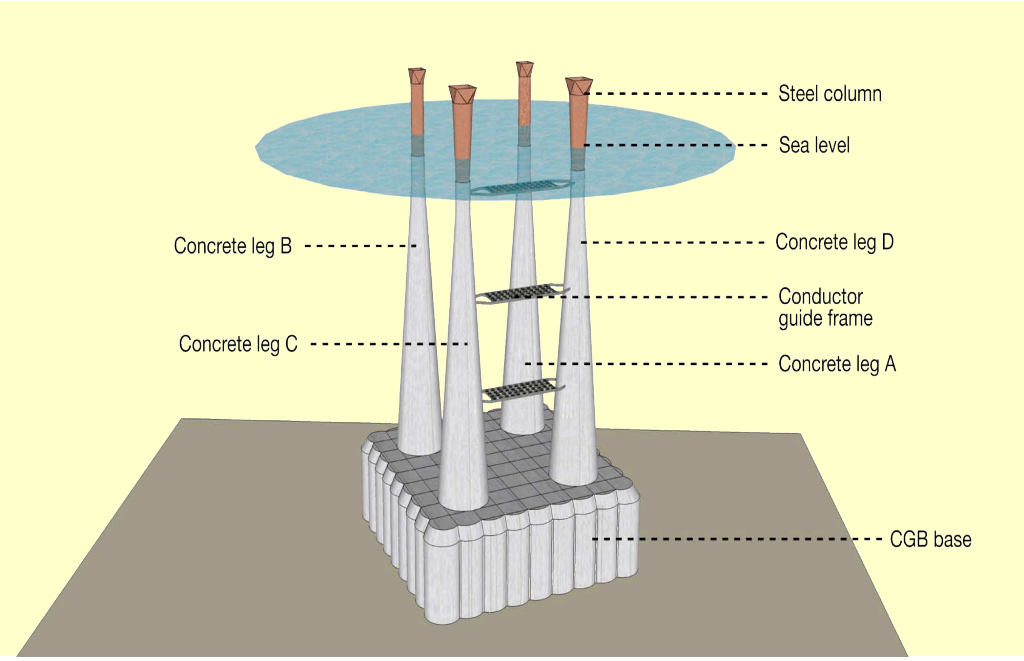

Figure 3.2a CGB with topsides and conductors removed

Four steel columns constructed from stiffened steel plate extend 31m from the top of the concrete legs, rising beyond the sea surface to the underside of the topsides deck. These columns are bolted and grouted into the top of the concrete legs. The steel columns C and D weigh some 500 tonnes each and taper from approximately 6m diameter at the top of the concrete legs to approximately 8.7m square at the underside of the deck. The other two columns on Legs A and B weigh approximately 300 tonnes each and are 5.4m in diameter, changing to a square section at the deck underside. See Figure 3.2b below.

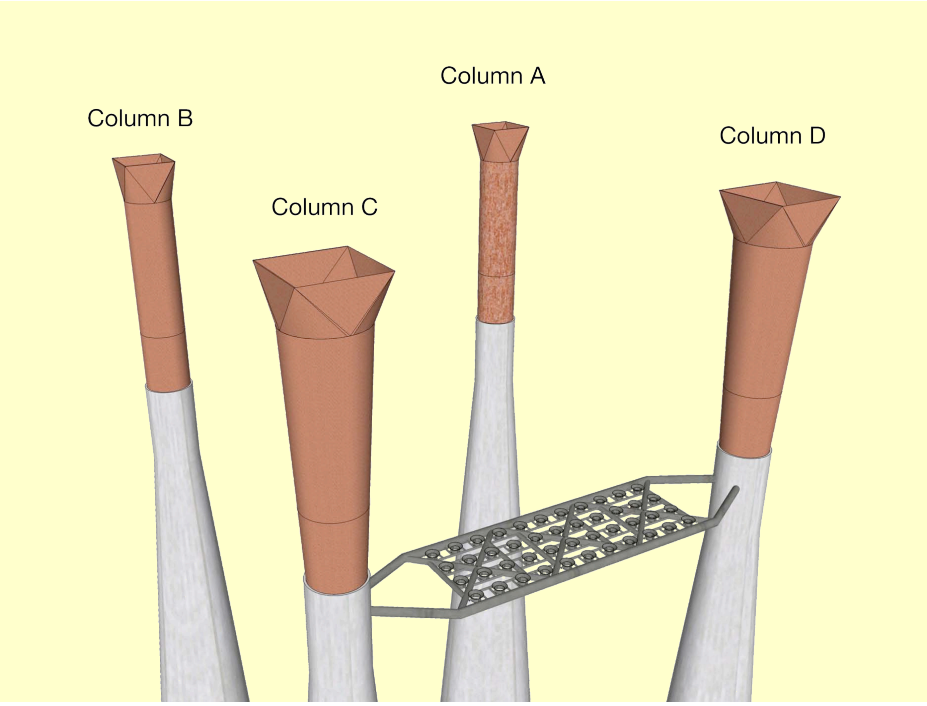

Figure 3.2b Steel columns at the tops of the concrete legs

The Leg A and Leg B steel columns are connected to the concrete legs with one external row of 40 bolts, 50mm in diameter, plus two internal rows of 40 bolts of the same size (120 bolts in total per leg). The Leg C and Leg D steel columns are connected to the concrete towers with two external and two internal rows of bolts (160 bolts in total per leg). The bolted connection is shown in section in Figure 3.2c below.

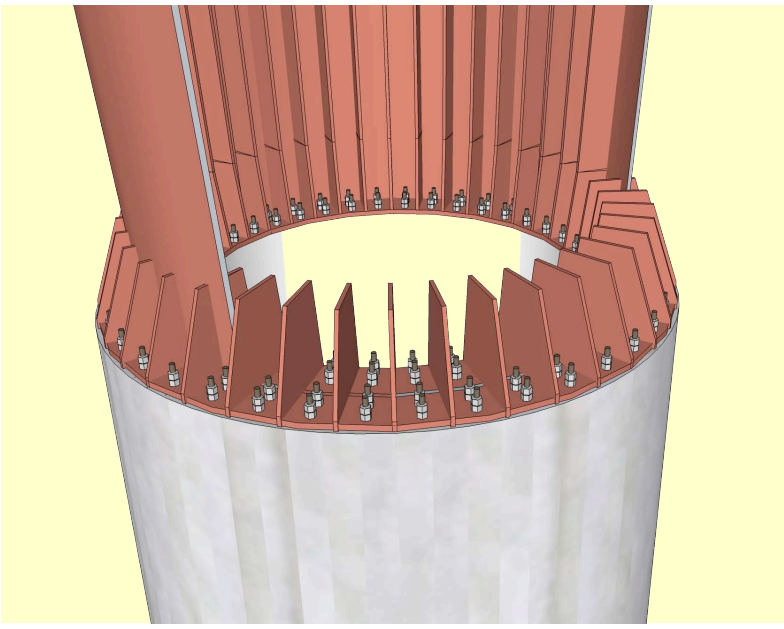

Figure 3.2c. Bolted connection between concrete leg and steel column

Spanning between Legs C and D are three horizontal guide frames. The function of these frames is to provide horizontal support to the 48 well conductors against wave action forces. Each of the three frames weighs approximately 200 tonnes.

Equipment and pipework (up to 36 inch in diameter) are distributed within the four legs, in different combinations. In addition, access stairways, lift shafts, platforms and service openings extend from the top of the legs down to the roof of the CGB base (there is no means of access into the CGB base for personnel). The complexity of the equipment arrangements inside the legs is too great to include within this document. However, a series of engineering drawings that convey the nature of the internal equipment in the legs can be viewed at:

http://www.fairfield-energy.com/pages/view/Dunlin CGB legs internals

For reference, the principal pipework lines and diameters which pass through the legs include:

Leg A
Seawater standpipe 36 inch
Conductor cooling water supply 6 inch
Conductor cooling water return 6 inch

Leg B
Four 16 inch/28 inch oil lines

Leg C
2 x 24 inch risers
1 x 20 inch risers
2 x 16 inch risers
4 x 14 inch J-tubes
2 x 10 inch J-tubes

Leg D
2 x 20 inch risers
4 x 14 inch J-tubes
2 x 10 inch J-tubes

Risers are pipelines which enter the base of the CGB and connect to the platform topsides, conveying oil, gas and water, for example the main 24 inch diameter oil export line from Dunlin A. J-tubes also enter the CGB base and connect to the topsides, and act as conduits for other lines, for example flowlines carrying fluids to and from subsea fields, control umbilicals and power cables.

4. Offshore decommissioning options for derogation

If derogation is granted for the CGB to be left wholly or partly in place, the base of the CGB with its matrix of internal cells would remain on the seabed. Over the course of time, as the concrete in the structure degraded and the reinforcing steel corroded, the cells contents would enter the surrounding marine environment. A detailed assessment of the cells contents and their potential environmental impacts has been conducted independently by consultants Intertek METOC, and the findings have been fully reported (Ref. 6). Intertek METOC concludes that the residual contents of the CGB cells, which are almost entirely filled with seawater, will not pose an unacceptable risk when the contents are eventually released from the CGB and become exposed to the environment.

The release of the cells contents will be further addressed in the EIA that will be produced as part of the Decommissioning Programmes for the Dunlin field. The EIA will also address the environmental impacts, in regard to the decommissioning options, for the drill cuttings accumulation on and around the CGB base. The potential for cumulative combined impacts from cells contents and drill cuttings will also be considered in the EIA. (Note that drill cuttings have been omitted from the diagrams that follow for clarity).

Prior to decommissioning the CGB, the topsides would be removed and taken to shore for dismantling and recycling. Topsides removal is not covered in this report but will be addressed in the detailed Decommissioning Programme for the Dunlin cluster.

It is also assumed that the following activities would have been completed as a precondition of derogation being granted to leave the CGB wholly or partly in place:

- Wells plugged and abandoned
- Well tubing terminated; well conductors and guide frames removed
- Subsea flowlines, water injection lines and cables disconnected from the platform as necessary

Following the completion of the preparatory tasks outlined above, three potential decommissioning options have been considered. These options fall within the framework set out by OSPAR Decision 98/3 for the CGB to remain wholly or partly in place, subject to derogation being granted. These are:

- Leave all of the CGB in place, submerged 8m below sea level
- Leave part of the CGB in place, submerged 55m below sea level
- Leave part of the CGB in place, submerged 110m below sea level

The three options, referred to as derogation cases, are discussed in the following sections. It should be noted that pipework and equipment inside the legs would not be removed for these options. Further details of underwater cutting methods can be viewed in the associated Fairfield Energy report on In Situ Deconstruction of the CGB (Ref. 3).

4.1 CGB remaining wholly in place (8m below sea level)

Figure 4.1a illustrates the CGB if derogation were to be granted for the concrete structure to remain wholly in place, terminating at a depth 8m below sea level.

The steel columns, conductor guide frames and all external steelwork would be removed. The remaining concrete structure would be entirely submerged with 8m water cover at low tide. As this would not comply with IMO guidance for providing 55m of clear water above the CGB, it is proposed that a 31m-tall extension tower be installed on the top of one or more legs to carry navigation aids. The navigation lights would be located 23m above sea level, around the same level of the current Dunlin A navigation lights. The structure would be marked on relevant nautical charts and the 500m radius exclusion zone afforded to offshore oil and gas installations would be retained.

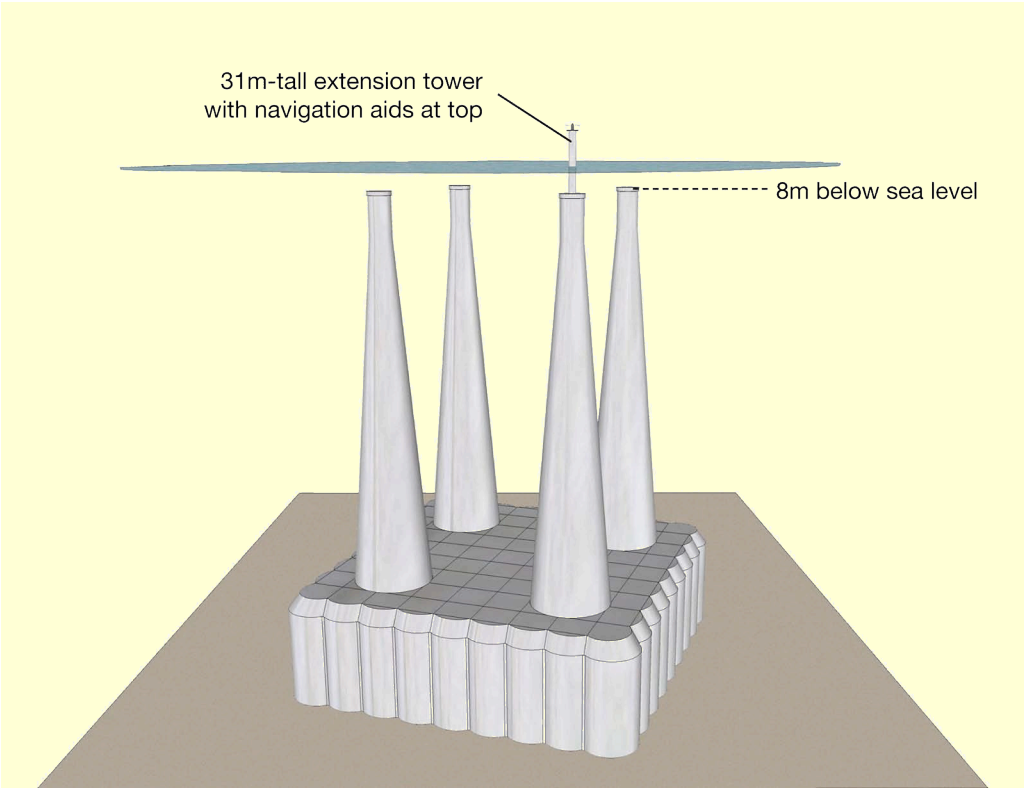

Figure 4.1a. CGB left wholly in place and marked with navigation aids

Removal of the steel columns may be achieved using diamond wire cutting. The cut would be made at around 8m below sea level, just above the bolted connection between each column and leg. At this level, the columns are 5.4m to 6m in diameter, and the columns each weigh 300 tonnes to 500 tonnes. The severed columns would be lifted from the legs by crane vessel and transferred to a barge for transportation to shore for recycling.

This operation would present some challenges for cutting in the splash zone. Lifting the steel columns would be well within the capacity of lifting operations regularly carried out in the offshore industry.

Conceptual engineering for the subsequent attachment of one or more navigation aids towers to the tops of the legs has been conducted by Atkins and

is presented in Appendix C. This demonstrates that the installation of a suitable navigation lights tower, capable of long term service in the northern North Sea environment and with access for periodic maintenance, is achievable in practice. The lights would be powered by electricity generated by a combination of wind and solar power.

Over a very long period of time, environmental loading on the legs of the CGB would cause fatigue in the legs, and in addition, the concrete and/or reinforcing steel in the CGB would degrade in the seawater environment. The end result would be the progressive collapse of the legs.

The addition of a tower to a leg to carry navigation lights 23m above sea level would increase the fatigue loading on that leg, due to:

- An increase in overall height and mass of the leg structure, which would increase the bending moments acting on the leg. The tapered shape of the leg would result in the maximum bending moment occurring at a point well above the base of the leg.
- An increase in the forces acting on the leg as the structure would be exposed to environmental loads through the wave zone at sea level and wind loading on the structure above sea level.

Structural analysis of the submerged CGB conducted by Atkins concludes that with a navigation tower attached to a leg, it would be at least 250 years before the leg failed due to fatigue (see Appendix D). This is likely to occur some 23m below sea level at the conical to cylindrical section of the leg, where the leg cross section is at a minimum, as indicated in Figure 4.1b.

The legs not carrying navigation aids would also fail in this way over time. These legs would be subjected to lower environmental loading due to the absence of the navigation tower, hence their failure would occur significantly beyond 250 years.

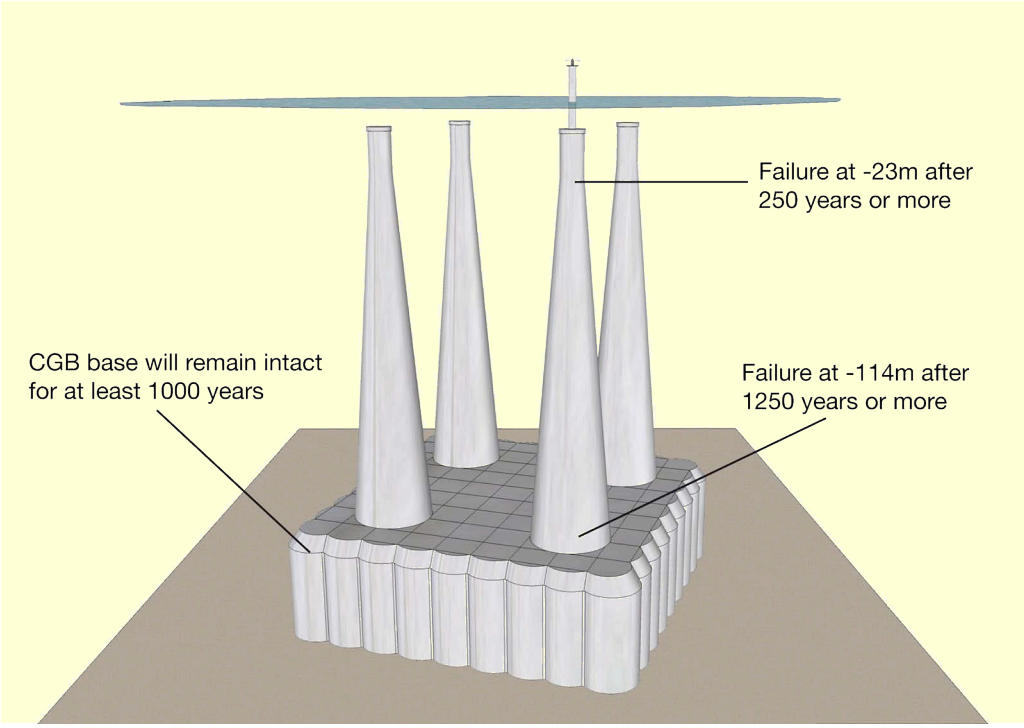

Figure 4.1b. Predicted durations for concrete failure in the CGB

In the event of a fatigue failure at around 23m below sea level in the leg carrying the navigation aids, the remainder of the leg would reach a height above 55m below sea level, and would therefore not comply with current IMO guidelines for minimum navigable water depth. The navigation aids would no longer be visible.

The expected life of the legs at lower levels would be significantly longer. At water depths greater than 20m the structure is not exposed to significant cyclic environmental loads, hence the ultimate structural failure mechanism for the lower leg sections would likely be due to concrete degradation and steel reinforcement corrosion. It is estimated that at a level some 5m above the base of the CGB (113m below sea level), leg failure would not occur for 1250 years or longer.

It is also possible that the support to a leg could be lost due to the failure of the base of the CGB containing the cells. However, the robustness of the base and its probable degradation rate suggest persistent endurance for more than 1000 years.

4.2 CGB remaining partly in place (55m below sea level)

For this derogation option, the steel columns, conductor guide frames and all external steelwork would first be removed. The four concrete legs of the CGB would then be removed above 55m below sea level to comply with IMO guidance for providing 55m of clear water above the submerged structure. The structure would be marked on all nautical charts but would not be provided with a 500m exclusion zone (under current legislation) or surface navigation aids to alert shipping.

To achieve this outcome, one of two approaches could be considered:

- The concrete legs would be cut through at 55m below sea level and the cut sections would be lifted onto a barge and transported to shore.

or:

- A break would be made in the concrete legs at 55m below sea level and the sections of the legs above would be allowed to fall to the seabed.

For the first approach, it would be necessary to cut the concrete legs in a controlled stable operation, with the upper concrete sections, each weighing around 2600 tonnes, being attached to a heavy lift crane vessel in readiness for lifting from the sea. The result is shown conceptually in Figure 4.2a.

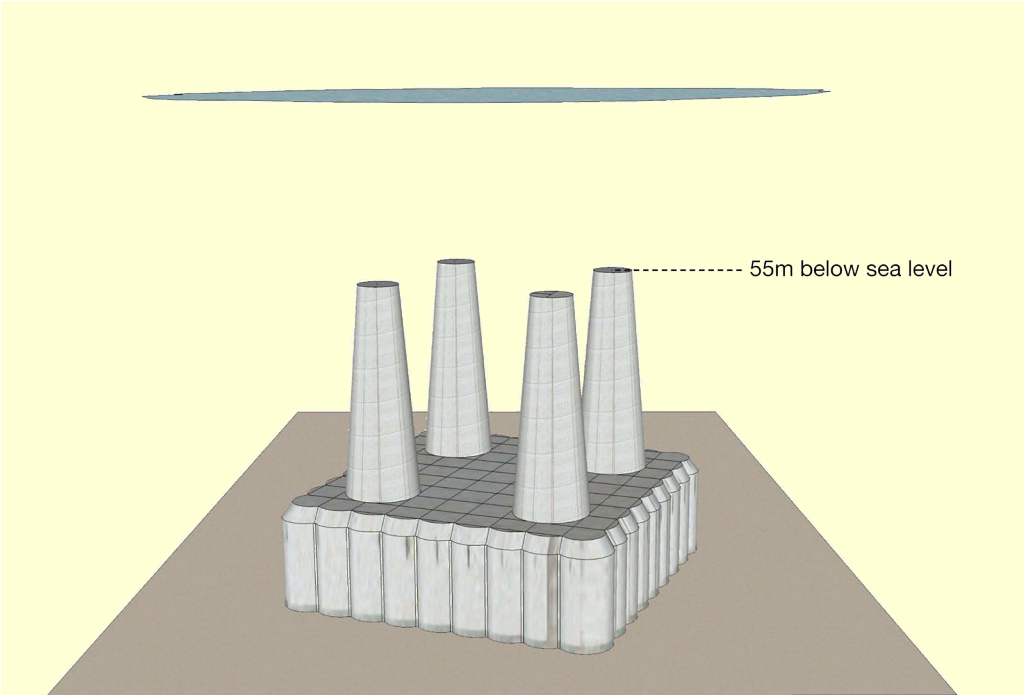

Figure 4.2a. CGB left partly in place with legs cut at 55m below sea level

However, this operation would not be possible due to:

- The inability of underwater cutting technology, including diamond wire cutting, to completely severe the concrete and all steel internals with certainty.

- Health and safety risks preventing personnel working inside the legs before or during cutting operations to remove steel internals.
- The unacceptable risks involved in trying to stabilise the upper section of the leg during cutting by supporting it from a heavy lift crane vessel.
- The impracticality of attempting to stabilise the leg section during cutting by use of a large underwater bracing support system, which would present significant technical and marine operational challenges.
- The inability to float the cut leg section away from the lower leg due to a lack of inherent floating stability and the impractically of attaching very large external buoyancy tanks, and the inability of the concrete structure to withstand elevated internal air pressure.

These factors and a review of cutting technologies are reported in more detail in the associated report on complete removal of the CGB (Ref. 3).

For the second approach of reducing the height of the CGB to 55m below sea level, a different method of removing the upper section of a leg might be considered in concept, referred to as 'controlled demolition'. Instead of attempting to cut through the entire cross section of a leg prior to a crane lift, a break in the leg would be made by first removing a wedge shaped section of concrete, approximately 2m in height at its maximum, from one side of the leg. This could be achieved by making two cuts using diamond wire cutting technology arranged to cut through the thickness of the concrete wall without making contact with internal pipework and equipment. A third similar cut would then be made from the other side of the leg to intersect the created gap. This is shown conceptually in Figure 4.2b.

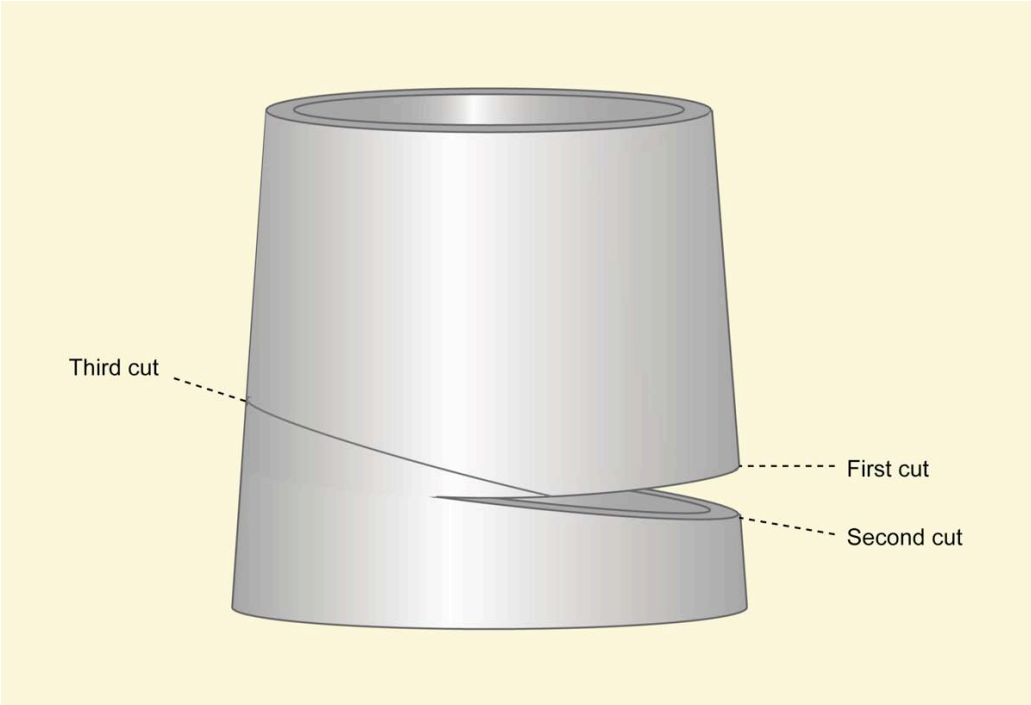

Figure 4.2b. Conceptual cutting method for severing CGB leg.

With the final cut almost complete, the upper leg section would be made to fall by surface tugs pulling it over. Alternatively, explosive charges might be used to induce structural failure and controlled demolition. With both methods the

internal steel and pipework in the legs would be bent and sheared by the falling upper leg section.

In these circumstances it would not be possible to predict with certainty how the upper section of a leg would fall from the 55m level and where it might land due to environmental loads acting on the upper section.. Furthermore, the impact of the leg sections on the base of the CGB and seabed would most likely cause the sections to fragment. The net result of this would be a structure terminating around 55m below sea level without being marked with navigation aids, with debris scattered on the seabed. This result is show conceptually in Figure 4.2c. (The severed pipework and steelwork is not shown on this diagram due to its small scale).

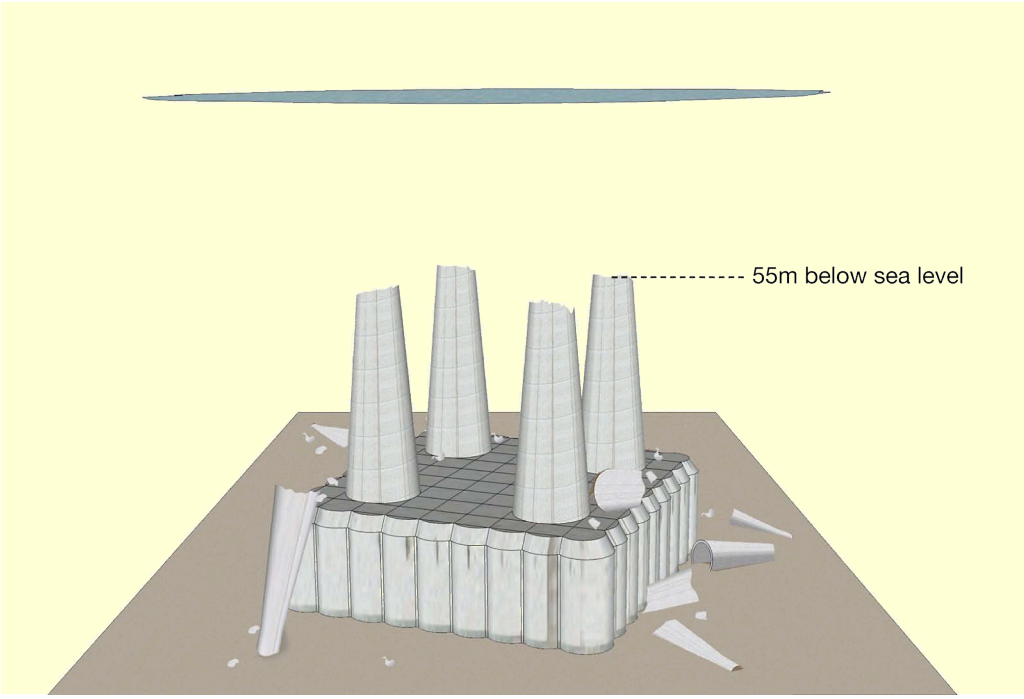

Figure 4.2c. CGB left partly in place by controlled demolition of legs at 55m below sea level

Recovery of the debris would entail protracted saturation diving programmes. Fairfield Energy considers that the risks to divers handling potentially unstable fallen sections of leg represent an unacceptable health and safety risk with little or no benefit to the environment, and such operations would therefore not be sanctioned by Fairfield Energy.

In this condition the remaining structure submerged 55m below sea level would not be exposed to significant cyclic environmental loads and the ultimate failure of the structure would likely result from concrete degradation and the corrosion of reinforcing steel. It is therefore anticipated that complete structural collapse of the lower leg sections would not occur for 1250 years or more, with the base of the CGB likely to endure even longer. To bring further understanding to the three CGB derogation options, Fairfield Energy has commissioned and sponsored two PhD research projects to investigate reinforced concrete degradation in water depths up to 150m in saline marine environments. This work is anticipated to be completed in 2014. (Ref. 7)

4.3 CGB remaining partly in place (110m below sea level)

A third conceptual option for in situ decommissioning of the CGB, if derogation were to be granted for the structure to remain partly in place, would be to demolish the legs at a depth of 110m below sea level. This level is above the drill cuttings accumulation on the CGB base.

The steel columns, conductor guide frames and all external steelwork would be removed. The four concrete legs of the CGB would be severed at 110m below sea level and allowed to fall to the seabed. The structure would be marked on all nautical charts but would not be provided with a 500m exclusion zone (under current legislation) or surface navigation aids to alert shipping.

To achieve this outcome, breaks would be made in the concrete legs in a similar way to that described in Section 4.2, by the controlled demolition method of first removing a wedge shaped section of concrete from one side of the leg using diamond wire cutting, and then cutting towards this from the other side. With the final cut almost complete, the upper leg section would be made to fall by surface tugs pulling it over. Alternatively, explosive charges might be used to induce structural failure and controlled demolition. With both methods the internal steel and pipework in the legs would be bent and sheared by the falling upper leg section.

The direction of fall of the leg sections would be more predictable when cut at this lower level, but they would most likely fragment when they reached the seabed, as shown conceptually in Figure 4.3. (The severed pipework and steelwork is not shown on this diagram due to its small scale).

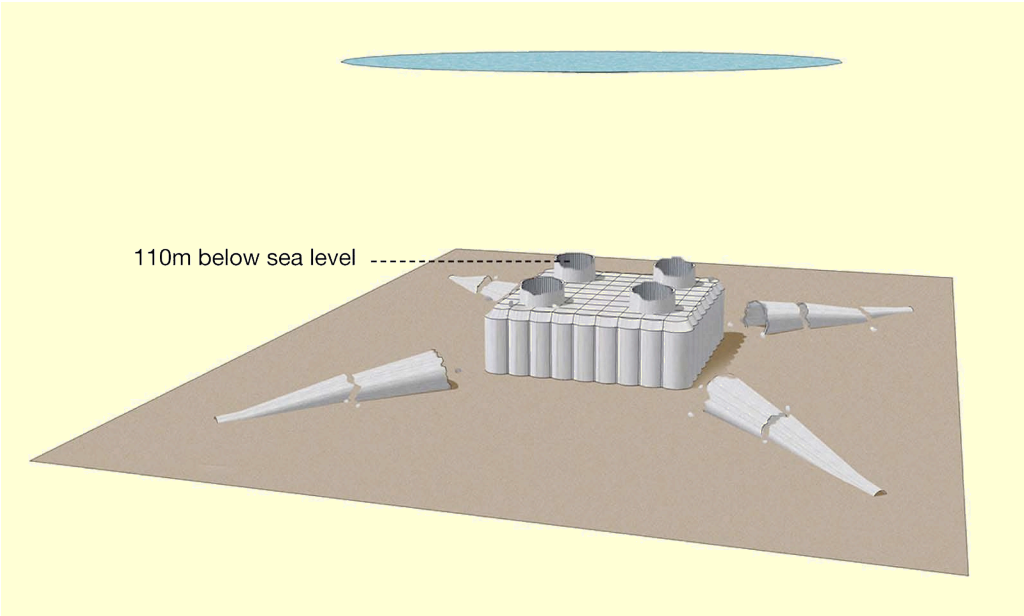

Figure 4.3. CGB left partly in place by controlled demolition of legs at 110m below sea level

As explained in Section 4.2, recovery of the debris would entail protracted saturation diving programmes with associated unacceptable health and safety risks, and such operations would not be sanctioned by Fairfield Energy.

In this condition the remaining structure would not be exposed to significant cyclic environmental loads and the ultimate failure of the structure would likely result from concrete degradation and the corrosion of reinforcing steel. As stated in Section 4.2, it is anticipated that complete collapse of the CGB base structure would not occur for at least 1000 years.

It should be noted that the final outcome of the controlled demolition approach would achieve the same overall outcome of the option to leave the CGB wholly in place, submerged to 8m below sea level. For the latter option, the legs would degrade over the long term and collapse until all of the structure became randomly distributed as debris on the seabed. This would be most likely to occur in stages, with the CGB legs becoming progressively more submerged. However, the structure would probably pass through a period when the legs reached above 55m below sea level, and therefore would not comply with IMO Guidance, but they would not be marked as the navigation tower(s) would fall with the legs.

In this respect, the option to remove the legs at 110m below sea level by controlled demolition could be regarded as an approach which would, in theory, achieve the ultimate condition, but in a more controlled and predictable manner.

5. Discussion

The assessment presented in Section 4 suggests the three options for wholly or partly leaving the Dunlin A CGB in place. Each of these options would require derogation to be granted.

With reference to the current wording of OSPAR Decision 98/3, leaving the structure wholly in place is permissible where it can be shown '*that there are significant reasons why an alternative disposal method is preferable to re-use or recycling or final disposal on land*'. Therefore, subject to demonstrating that there are significant reasons why an alternative disposal method is preferable, along with the installation of navigation lights on the structure, the CGB could be left wholly in place, submerged 8m below sea level at low tide.

With reference to this derogation option, in order to comply with IMO guidance, an extension tower would be installed on the top of one or more legs to carry navigation lights. The navigation lights would be located 23m above sea level, at around the same level of the current Dunlin A navigation lights. Fairfield Energy has held discussions with the Northern Lighthouse Board, based in Edinburgh, to develop an appropriate statement of requirements and specification for the navigation lights. It is anticipated that the marine marking requirement will be consistent with the navigation aid equipment installed on other previously derogated CGB structures in the North Sea, including structures in the Frigg field (MCP01, CDP1, TCP1 and TP1) as well as the Ekofisk Tank in the Ekofisk field. Additionally, Fairfield Energy would establish a Legacy Trust to provide ongoing funding for the management and maintenance of the navigation lights.

Due to a combination of structural fatigue, induced by environmental forces, and naturally occurring reinforced concrete degradation, the legs of the CGB will collapse over time. However, this is not predicted to occur for at least 250 years. Analysis suggests that in this failed condition, compliance with the current IMO guidance for providing 55m of clear water above the submerged obstruction is unlikely to be satisfied. However, it is not possible for commercial organisations to make financial provision for remedial undertakings centuries in the future, nor can the guidelines and regulations that might prevail at that time be predicted.

The CGB legs would continue to fail progressively over time, increasing the water cover as the overall height of the structure reduced, and decreasing the collision risk for shipping. Debris from the leg sections would be scattered on top of the CGB base and/or on the seabed in the immediate vicinity of Dunlin A.

With reference to leaving the CGB partly in place by reducing the height of the legs to either 55m or 110m below sea level, technical study work commissioned by Fairfield Energy has shown that to do this and remove the cut sections of legs to shore is not viable (Ref. 3). As an alternative, a conceptual approach to achieve controlled demolition of the legs has been investigated, which would result in the legs being reduced to debris on the seabed in the vicinity of the CGB. This approach would require further technical work to evaluate its viability. However, although under OSPAR Decision 98/3 the disposal of the substructure of a concrete installation at a deepwater licensed site is an option, this must be considered in light of the UK Government announcements at the time of OSPAR Decision 98/3 when ministers stated that there would be no toppling and no local or remote dumping of offshore installations. Therefore,

Fairfield Energy can see no benefit at this stage in developing the required methodologies or technologies for controlled demolition. However, controlled demolition of the legs and leaving the debris on the seabed, if permitted, would significantly improve the safety of surface vessels while also reducing residual third party liabilities for Fairfield Energy.

Fairfield Energy considers that the risks to divers handling potentially unstable fallen sections of leg and the need for protracted saturation diving programmes for debris recovery operations represents an unacceptable health and safety risk with little or no benefit to the environment. Such operations would therefore not be sanctioned by the company.

While controlled demolition of the legs is currently viewed as being unacceptable by DECC for derogation under OSPAR Decision 98/3, it should be noted that the final outcome of controlled demolition at 110m below sea level would achieve, in the short term and in a more controlled and predictable manner, the same long term outcome of the option to leave the CGB wholly in place, submerged to 8m below sea level, followed by the structure's collapse over a lengthy time period, measured in centuries.

6. Conclusion

Fairfield Energy has investigated three options for leaving the Dunlin A CGB wholly or partly in place. The company accepts that any derogation application submitted to the UK regulator, the Department of Energy and Climate Change, must comply with the current requirements of OSPAR Decision 98/3 and those of DECC Guidelines. The latter effectively eliminate controlled demolition and leaving debris from the legs on the seabed.

Therefore, Fairfield Energy concludes that the only viable derogation option for the Dunlin A CGB is to leave the structure wholly in place, submerged to 8m below sea level, with the installation of one or more leg extensions reaching above the sea surface to support navigation lights, as may be required by the Northern Lighthouse Board. This outcome is shown conceptually in Figure 6.1, with drill cuttings shown.

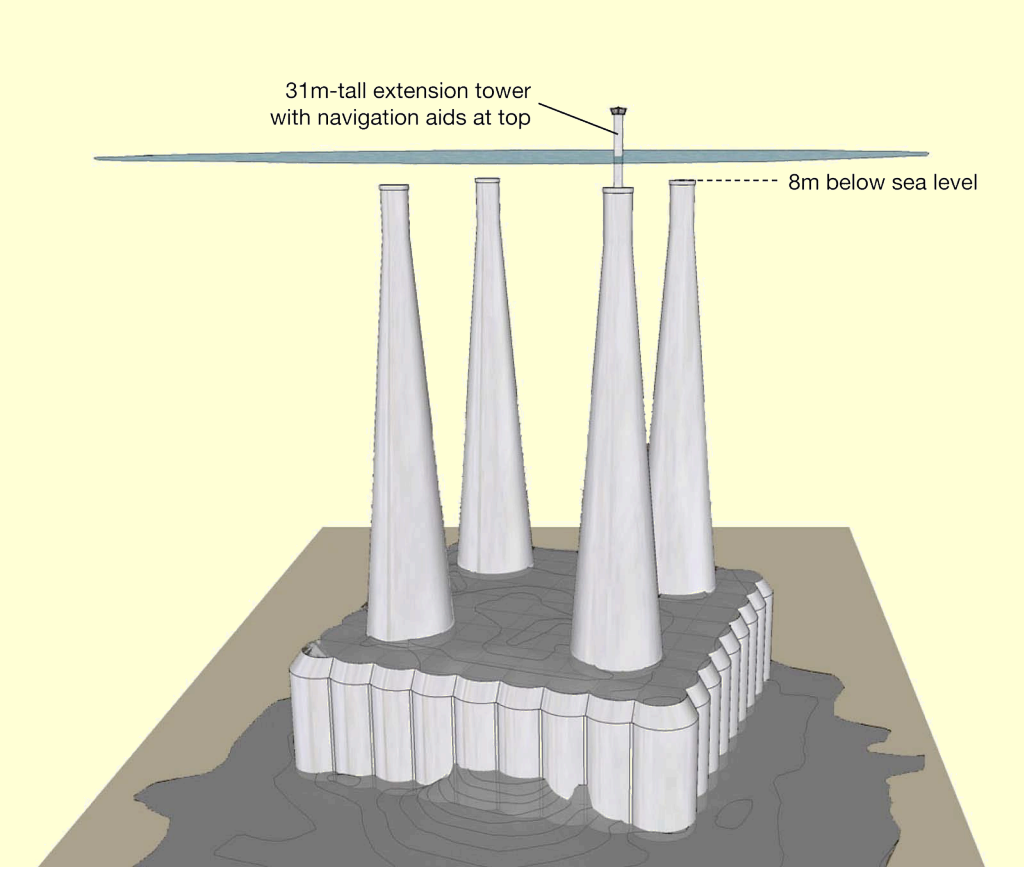

Figure 6.1. CGB decommissioned at 8m below sea level, fitted with navigation aids on one leg

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Appendix A

Dunlin field and surrounding area

Appendix A

Dunlin field and surrounding area

Contents

A.1 Dunlin cluster

The Dunlin cluster of fields is located in the UK sector of the North Sea, and is operated by Fairfield Betula Limited (FBL) and Fairfield Fagus Ltd (FFL), both of which are wholly owned subsidiaries of Fairfield Energy Ltd. The licence interests in the Dunlin cluster are collectively owned by FBL and FFL (70%) and MCX Limited (30%), a wholly owned subsidiary of Mitsubishi Corporation.

The Dunlin cluster of fields is located in Blocks 211/23 and 211/24 of the UK Continental Shelf, some 500km north-northeast of Aberdeen within the East Shetland Basin, and 11.2km from the boundary line with Norway. (Figure A.1a).

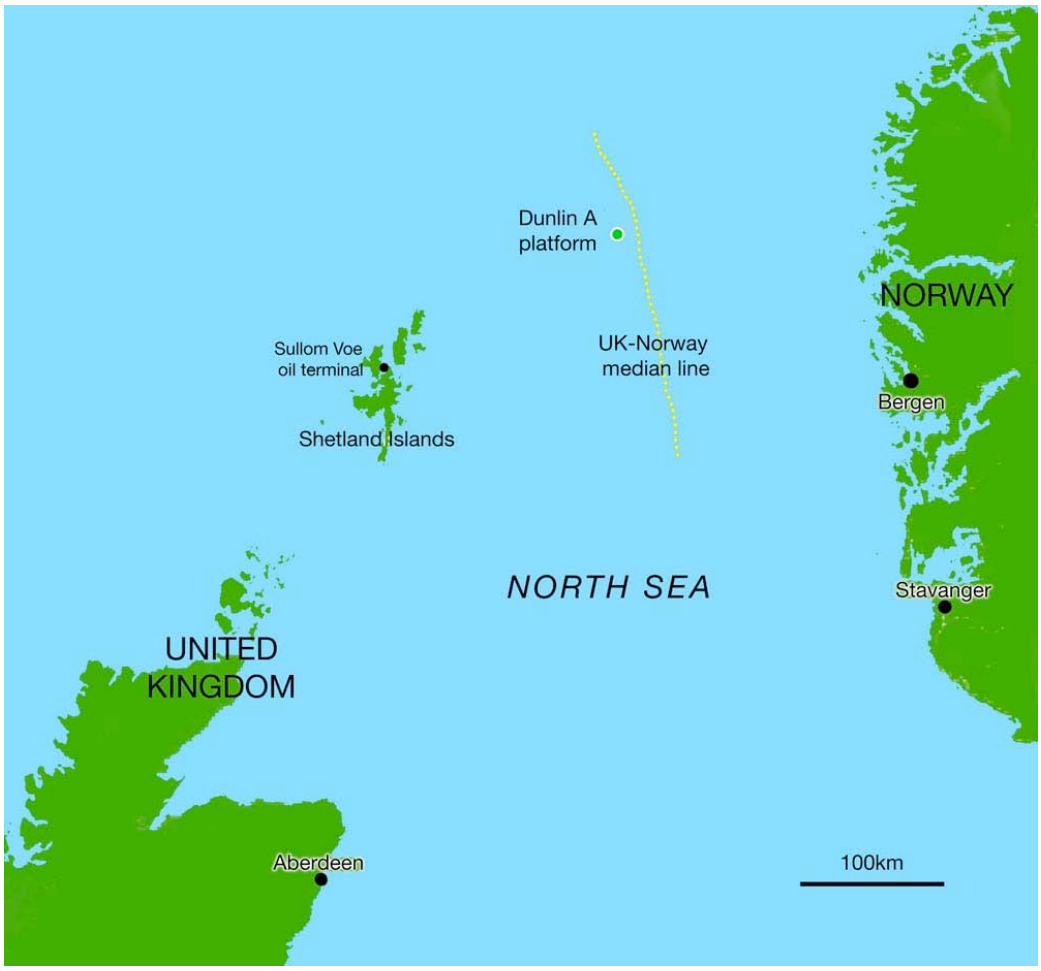

Figure A.1a Dunlin field location map

The Dunlin cluster comprises the Dunlin, Dunlin South West (operated by FBL), Osprey and Merlin fields (operated by FFL). The Dunlin Alpha platform, normally referred to as Dunlin A, stands on the seabed above the Dunlin field. The Dunlin A platform is a fixed installation, serving as a production facility for the Dunlin, Dunlin South West, Osprey and Merlin fields. Oil production from the fields is exported from Dunlin A via pipeline to the Cormorant A platform, and from there by pipeline to the Sullom Voe oil terminal in the Shetland Islands.

The main Dunlin hydrocarbon reservoir is reached from wells located on the Dunlin A platform. Dunlin South West is a separate hydrocarbons accumulation, also reached by wells from the Dunlin A platform.

The Merlin and Osprey fields are separate reservoirs, accessed by subsea wells located on the seabed. These fields are 'tied back' to the Dunlin A platform by a set of seabed pipelines and control lines.

Dunlin A also acts as a pumping station for crude oil imports from the Thistle and Murchison fields, which, after being combined with production from the Dunlin cluster, are also exported via the Dunlin/Cormorant export pipeline.

The nearest manned installation to the Dunlin facility is the Thistle A platform, approximately 12km to the north.

The general arrangement of the Dunlin cluster facilities and pipelines is shown in Figure A.1b.

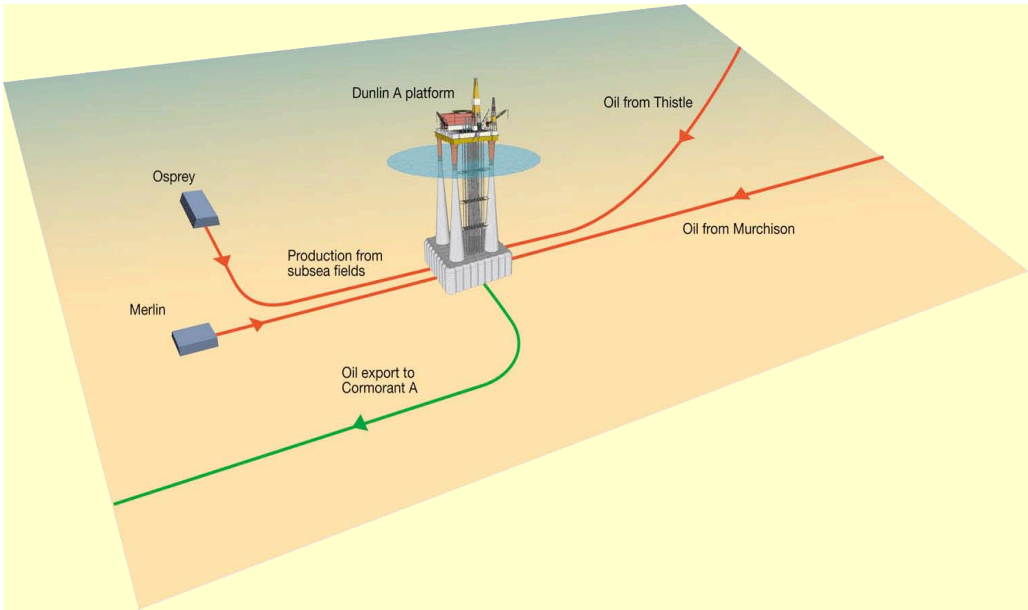

Figure A.1b Dunlin, Osprey and Merlin facilities

The Dunlin A platform was installed in 1977 and production started in 1978. Production began from Osprey in 1991 and from Merlin in 1997.

The Dunlin A platform, located in 151m of water, consists of a four-legged concrete gravity base (CGB) substructure with topsides supported by a steel box girder frame, as shown in Figure A.1c.

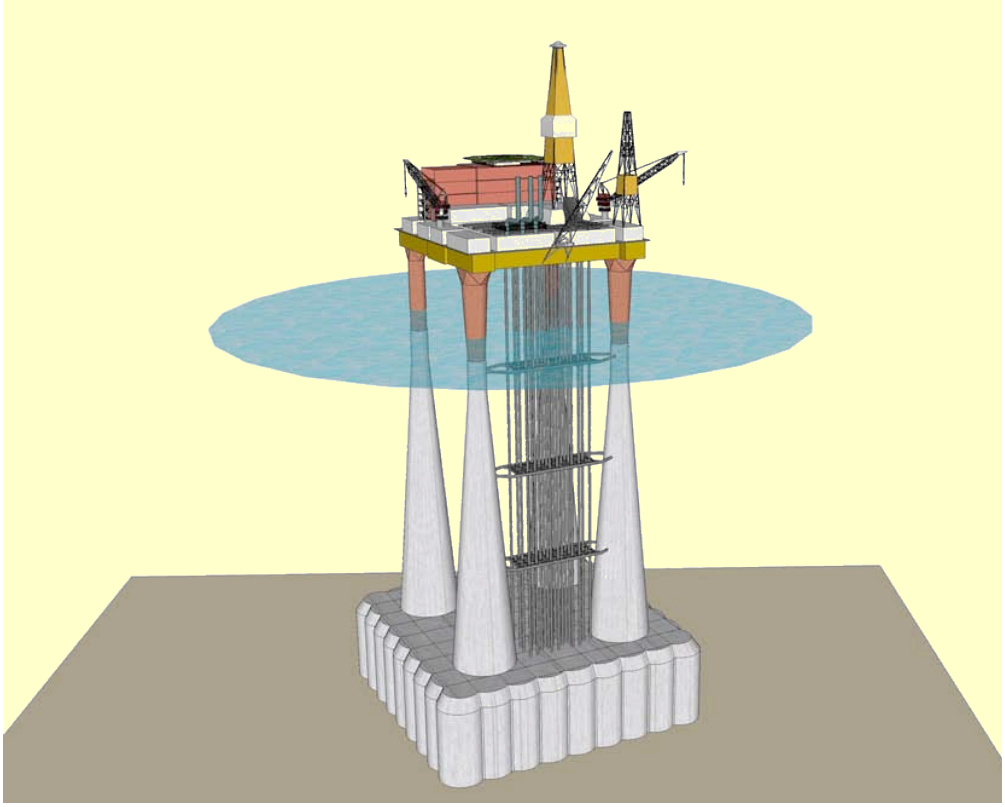

Figure A.1c Dunlin A platform

The installation was designed to:

- Serve as a production facility for the Dunlin, Osprey and Merlin fields.
- Serve as a drilling facility for the Dunlin fields.
- Provide separation of oil and water within the CGB.
- Accept oil imported from Thistle A and Murchison A, prior to onward transmission to Cormorant A via pipeline.

The Dunlin A CGB design basis was developed to satisfy several competing criteria, namely:

- Location of the construction yard in shallow water in The Netherlands.
- Provision of sufficient self-buoyancy for the towed voyage to the field.
- Seabed and environmental conditions at the field.
- Topsides load to be supported.

Two separate 16 inch diameter pipelines import oil from Murchison A and Thistle A to Dunlin A, while a 24 inch diameter oil export line runs from Dunlin A to Cormorant A. Additionally, two 8 inch diameter subsea pipelines associated with the Merlin and Osprey developments are routed to Dunlin A.

The Osprey field facilities consist of two subsea drilling templates and a subsea manifold located some 7km north of Dunlin A in water depths ranging from 155m to 165m.

The Merlin field facilities consist of three subsea production wells and a water injection well, located 7km west of Dunlin A in water depths ranging from 155m to 165m.

A 23km long, 119mm diameter electric power cable runs in a trench from the Brent C platform to Dunlin A to supply the latter with part of its power requirements.

A detailed description of the Dunlin A platform is given in Section A.4.

A.2 Environmental aspects

This section presents a summary of the general environmental conditions around the Dunlin A platform.

To evaluate any likely impact of the options considered for the decommissioning process, the present day environmental conditions need to be understood. The current environmental status reflects historical operational and disposal practices of the offshore and marine industries. Over time, the results of these activities have been modified by the effects of wind, wave and tidal currents, both on the seabed and in the water column.

The meteorological conditions of the region are characterised by rapidly changing weather conditions. Wind direction is commonly from the south and southwest throughout the year, but north and northeast winds can dominate between May and August.

The significant wave height ranges from 8.7m (monthly) to 11.4m (annually) with the maximum 100-year significant wave height estimated to be 15.6m.

The water current patterns in the area are complex, with strong non-tidal currents interacting with relatively weak tidal flows. Water currents in the area predominantly flow from the northeast to southwest although this is less apparent at greater water depths where current velocities decrease.

The seabed surface around Dunlin A consists of fine to gravelly sands with some shell debris. The surface is characterised by a number of natural and man-made features including minor depressions, cobbles and small boulders, extensive anchor scarring, rock dumps, and items of debris.

A drill cuttings accumulation covers part of the Dunlin A CGB structure and adjacent seabed. The cuttings were generated from the start of drilling activities in 1978.

Any potential effects of the Dunlin cluster development on the biological environment are expected to be localised and confined to organisms that live in or on the seabed and, to a lesser extent, in the water column. The marine life includes:

- Plankton - The plankton community around the Dunlin area is typical of that found in the northern North Sea
- Seabed communities - The seabed surface around Dunlin A platform supports a diverse range of animal communities, with no clear dominant species. Bristle worms (polychaetes) make up the majority of recorded species.
- Coral - *Lophelia pertusa* is a coral which develops on hard surfaces in cold, dark, nutrient-rich waters between 100m to 400m deep. It has been observed on parts of the CGB. This species is important as it is protected under the European Habitats Directive 1992, Annex II.

- Fish - Fish catch statistics, compiled by the Marine and Fisheries Agency, show that the area around the Dunlin A platform is dominated by the open water (pelagic) species Atlantic mackerel and the near seabed (demersal) species Atlantic haddock and Atlantic cod. Catches also include whiting, saithe, pollack, plaice, turbot, halibut, lemon sole, megrim and the Norway lobster.
- Sea birds - A number of the bird species likely to be present in the Dunlin area are protected. Species observed include fulmars, guillemots, gannets, kittiwakes, puffins and razorbills.
- Marine mammals - Marine mammals observed in the waters surrounding the Dunlin A platform include whales, dolphins and seals. A number of these mammals are protected under the Habitats Directive, Annex II. The minke whale, killer whale and pilot whale have been sighted in the vicinity of the Dunlin platform on a more regular basis than other cetacean species.

A.3 Socio-economic aspects

The Dunlin A platform stands in open seas with the nearest surface structures being the Thistle, Murchison, Cormorant and Brent platforms. Shipping activity in the area is of low density, primarily related to vessels passing between Aberdeen and offshore facilities in the northern North Sea. Fishing vessels are also likely to be present in this area.

A.4 Dunlin Alpha platform

A.4.1 Introduction

Design and construction of the Dunlin A CGB structure was carried out by the Anglo Dutch Offshore Concrete (ANDOC) contractors' consortium in The Netherlands during the 1970s. The Dunlin A platform was installed in 1977 and, after the drilling of initial wells, oil production began in 1978.

The platform base is 104m square and the platform is over 200m high from the seabed to the top of the drilling derrick. The CGB weighs approximately 320,000 tonnes, including internal equipment and solid ballast in the CGB base, while the topsides weighs a further 20,000 tonnes. The CGB was not designed to be eventually refloated.

To give an appreciation of scale, Figure A.4.1a shows a graphic representation of the platform in comparison with the Big Ben clock tower in London, which is 96m high.

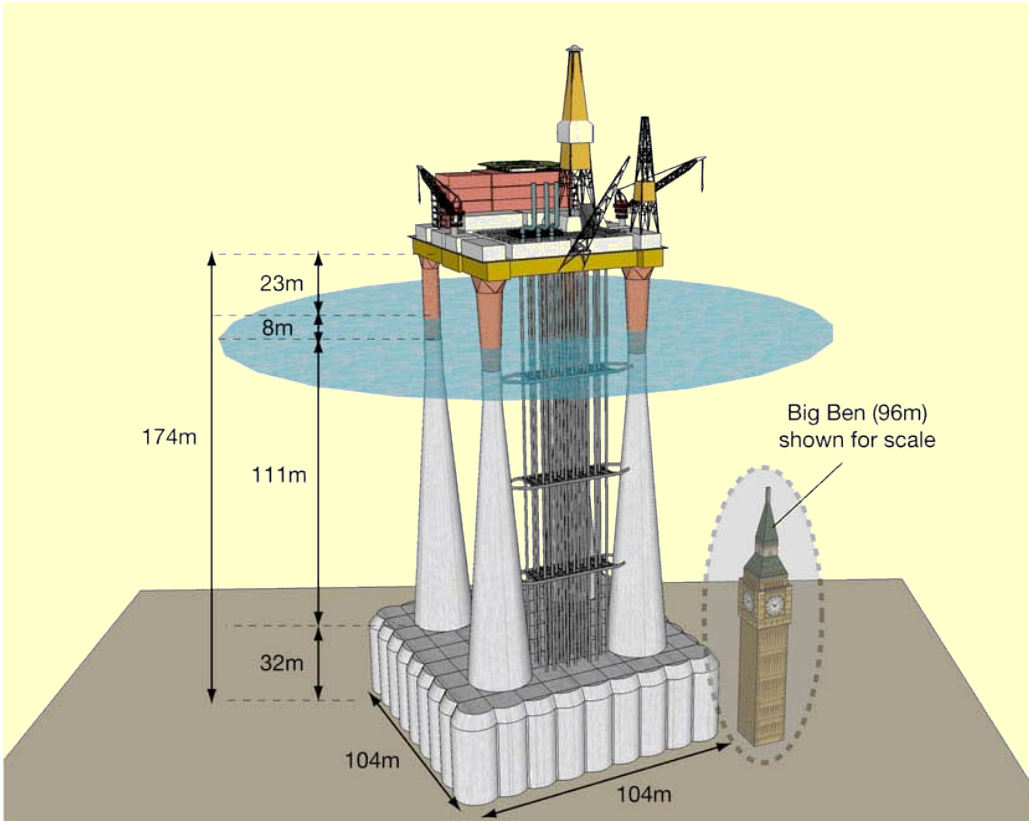

Figure A.4.1a Dunlin A compared with Big Ben for scale

Figure A.4.1b below shows the main components of the platform.

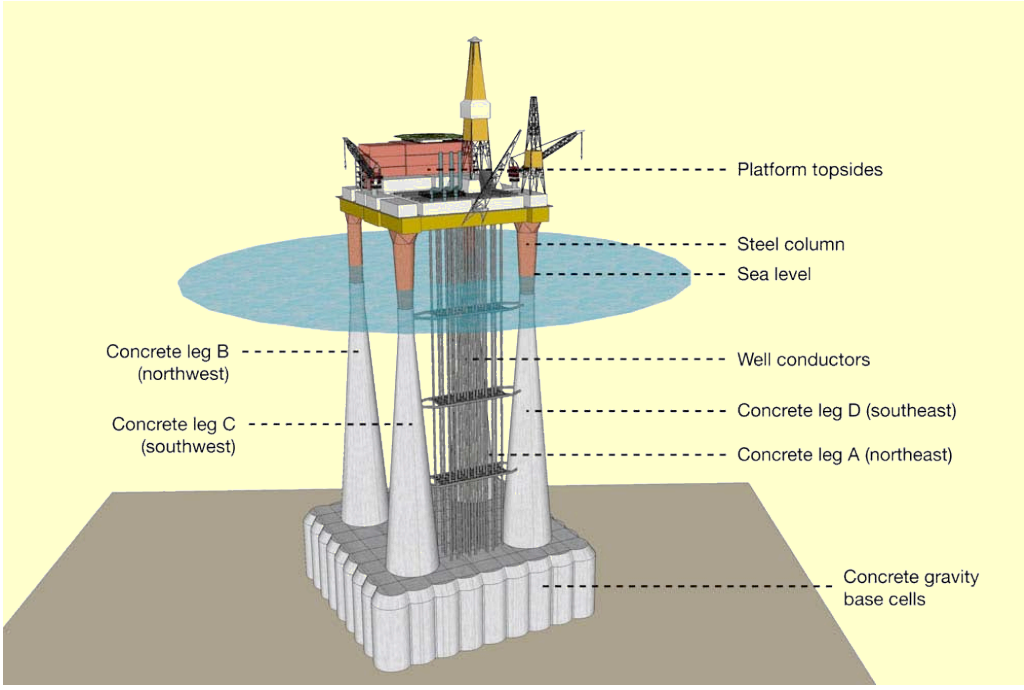

Figure A.4.1b Dunlin A platform main components

The platform was designed as a drilling and production installation. The 20,000 tonnes topsides includes the following facilities:

- Drilling
- Oil and gas processing and metering
- Produced water treatment and water reinjection
- Power generation, utility and safety systems
- Oil export pumping
- Personnel accommodation for 129 people
- Helideck

The platform was designed to accommodate 48 wells. Well fluids pass from the subsurface reservoir to the topsides within steel pipes, one per well, referred to as well conductors. The conductors are held in three steel guide frames located between platform Legs C and D.

A.4.2 Concrete gravity base structure

The CGB extends from the seabed to 8m below sea level where the tops of the concrete legs are joined to the steel superstructure. The CGB, including internal equipment and solid ballast in the base, weighs approximately 320,000 tonnes.

The base of the CGB, which is 32m high, is divided into 81 compartments, referred to as cells, arranged in a 9 x 9 matrix as shown in Figure 4.2a.

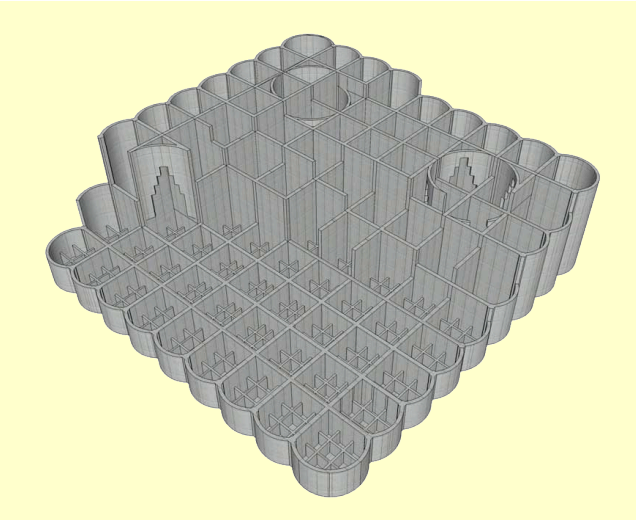

Figure 4.2a Cells in the CGB (cutaway view)

Of the 81 cells, the original purpose of 75 of these was to provide additional separation of oil and water prior to oil export. The remaining six cells, located between Legs C and D, were not used for oil and water separation and are filled with seawater. The 48 well conductors pass through the six cells, each conductor being protected by an outer carbon steel sleeve throughout the height of the cells. The six cells were designed to allow seawater to be pumped around them to cool the conductors.

Each cell is 11m square. Inside the bottom of each cell, secondary 4m-high concrete walls reinforce the base and sub-divide the bottom of each cell into nine open-topped compartments. All open-topped compartments in all of the cells were filled with ballast prior to the closure of the cells with convex concrete roofs.

A stiffened steel plate wall runs around the perimeter of the base to form a skirt, and penetrates the seabed to a depth of 4m. Two further steel walls run underneath the base slab of the CGB in each direction, creating nine sub-base compartments. See Figure 4.2b below.

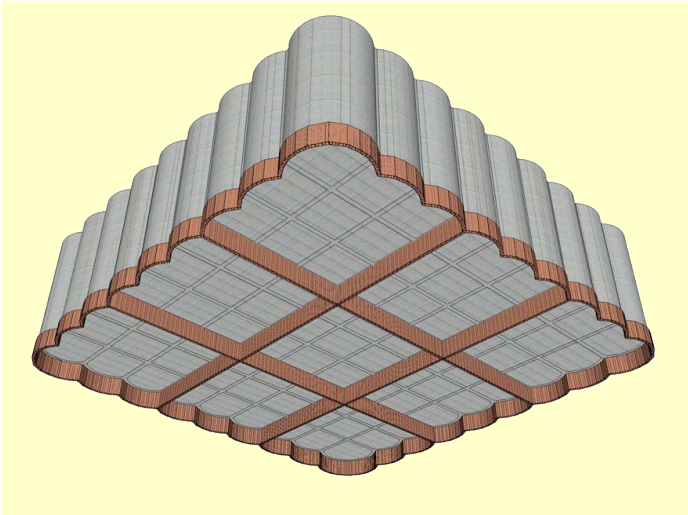

Figure 4.2b Steel skirt and walls

Rising up from the roof of the base cells are four concrete legs, each 111m high. These reduce in outside diameter from 22.6m at the bottom to 6.6m at the top, where they join the steel superstructure at 8m below sea level. The legs are designed as hollow shafts, with concrete walls generally being 700mm thick but increasing to 1200mm at the top and the bottom. Each of the concrete legs weighs approximately 7600 tonnes.

Four steel columns constructed from stiffened steel plate extend 31m from the top of the concrete legs, rising beyond the sea surface to the underside of the topsides deck. These columns are bolted and grouted into the top of the concrete legs. The steel columns C and D weigh some 500 tonnes each and taper from approximately 6m diameter at the top of the concrete legs to approximately 8.7m square at the underside of the deck. The other two columns (Legs A and B) weigh approximately 300 tonnes each and are 5.4m diameter changing to a square section at the deck underside. See Figure 4.2c below.

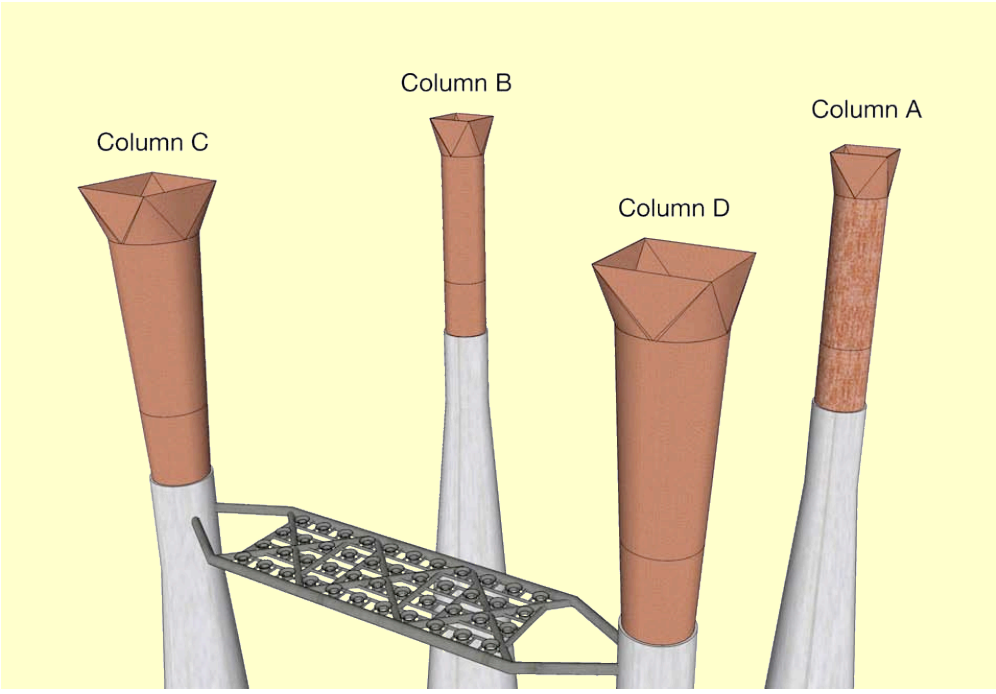

Figure 4.2c Steel columns at the tops of the concrete legs

Equipment and pipework are distributed within the legs, in different combinations. Access stairways, lift shafts, platforms and service openings extend from the top of the legs down to the base of the structure.

Spanning between Legs C and D are three horizontal guide frames which hold the well conductors in a 12 x 4 matrix. The function of these frames is to provide horizontal support to the well conductors against wave action forces. Each of the three frames weighs approximately 200 tonnes.

The deck structure above the steel columns consists of a lattice of steel box girders approximately 85m by 67m in plan. The lattice is 6m deep and is equipped with a deck at top and bottom to support equipment. The deck structure also supports a number of modules which contain drilling facilities, production and utilities equipment and accommodation units.

A.4.3 Platform lifecycle

A.4.3.1 Platform construction and installation

The Dunlin A CGB base slab and cell walls were constructed in a purpose-excavated dock in The Netherlands using conventional civil engineering construction methods for casting concrete walls. After the cell walls were completed, the dock was flooded and the structure was floated into deeper water. The cell roofs were then completed using pre-cast concrete shaped sections to support the cell roof concrete while it dried.

The concrete legs were then constructed using a slip-form method. In this type of construction wet concrete is poured continuously into moulds. The moulds are continuously moved slowly upwards, using jacks, while the concrete at the bottom of the mould sets.

Pipework, pumps, manifolds and access steelwork, required for the installation and operation of the platform, were installed at their design locations during the construction programme.

With the base cells and legs completed the platform was towed approximately 850km to a Norwegian deepwater fjord. Solid granular ballast was added in the base of the cells up to the level of the 4m-high secondary walls within the cells.

By controlled introduction of seawater into the base cells, the structure was submerged to a draught where the water level was near the top of the legs. The steel columns were lifted on to the top of the legs using floating crane vessels and bolted into position.

The deck structure was fabricated in sections in The Netherlands. These were assembled into a single structure on supports over water. Following this, production equipment and other facilities were installed on the deck. A transportation barge was floated between the supports. By deballasting the barge it rose in the water to pick up the deck structure.

The barge was then towed to the Norwegian fjord, with the deck structure onboard. Here, with the CGB submerged to allow the deck to be floated over the legs, the deck was installed on top of the CGB steel columns by a process which reversed that for loading it onto the transportation barge. By carefully deballasting the submerged structure and ballasting the transportation barge simultaneously the deck load was transferred to the CGB.

At this stage, additional topsides modules were installed on the deck using floating cranes before the platform was further deballasted to its towing draught. The platform was subsequently towed a distance of 400km by seven ocean-going tugs to the Dunlin field location in the North Sea.

Once on location the platform was positioned accurately and more seawater was added to the cell bases under careful control, until the platform touched the seabed. The final flooding of the cells caused the steel skirts around the base to penetrate fully into the seabed. Any water trapped within the underbase compartments formed by the steel skirts escaped through preinstalled vent lines in the base.

Platform installation was completed by pumping cement grout, through preinstalled grout lines, under the base to fill any spaces present between the base slab and the seabed. The grout displaced any trapped seawater via the vent lines. Following the completion of the grouting operations the grout lines and vent lines were left grout-filled.

Once installation of the platform was complete, the drilling module was installed on the topsides and drilling of the wells began.

A.4.3.2 Concrete gravity base operational history

The Dunlin A CGB was installed during late summer of 1977. Following completion of the initial drilling phase, crude oil production started in 1978.

For the early period of Dunlin operation (1978-1995), fluids from Dunlin's production wells were first passed through separation vessels on the topsides to separate gas from liquids. The liquids (oil and produced water) were then piped through Leg B of the CGB to those cells in the base of the CGB designated for oil and water separation

In the base of the CGB, the 75 oil and water separation cells are configured as four separate groups, A-D, as shown in Figure A.4.3a below. The cell groups provided gravity separation of oil and water, and were operated in a sequence, as follows:

- One cell group was used as a receiving volume taking fluids from the final stage of topsides separation.
- Two cells groups were used for further oil and water separation.
- The fourth cell group, where separation had progressed furthest, acted as the source of dry export quality oil, which was pumped to the topsides for fiscal metering and export.

By means of pipework and valves, these operations could be cycled around the cell groups in turn.

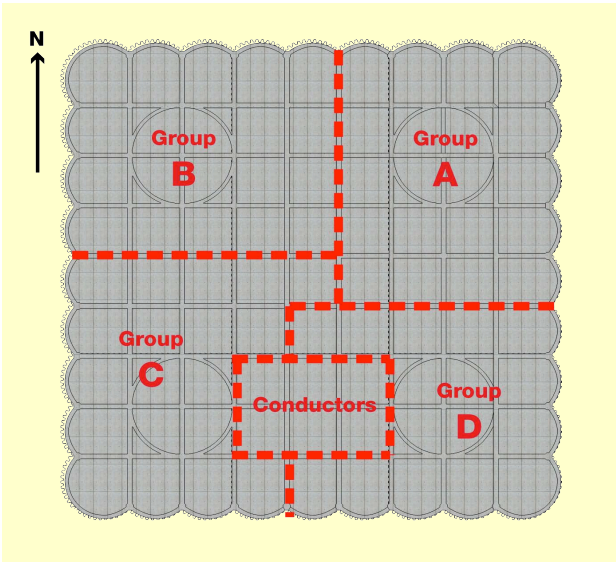

Figure A.4.3a Cells arranged in five groups

As the oil and produced water entered the CGB cells, displaced export oil was returned to the topsides and pumped into the oil export pipeline.

While the oil was contained in the upper part of the four cell groups as four separate oil volumes, the produced water below the oil was in effect one large single volume. This was achieved by interconnecting ports in the walls of the cells at low levels to allow water to move between the four groups. The water

could be pumped from the base of the cells and was returned to the topsides for treatment to meet licensed quality standards prior to discharge to the sea.

The cells have a large volume, designed to give the CGB the necessary self-buoyancy during the towing phases. As a result, the cells provided long liquid retention times for the production fluids when the cells were used for oil and water separation. The long retention times produced a very quiescent flow environment, allowing very effective oil and water separation to occur. Combined with the fact that oil volume in the cells was minimised for commercial reasons, the resultant oil volume in the tanks was relatively low compared to water volume.

A fifth group of six cells between Legs C & D surrounds the platform's 48 production well conductors. This group was not used for oil and water separation. Pumped seawater continuously circulates through this conductor cell group to remove heat arising from the well conductors. Control of the temperature gradient between the conductor cell group, the surrounding cells and the sea is necessary to maintain the structural integrity of the CGB.

This method of operation continued until the Dunlin A topsides separation facilities were modified after 1995 to allow three-phase separation of oil, gas and water on the topsides, thereby eliminating the need to use the CGB cells on a routine basis for oil and water separation. From then on, the cells generally remained water-filled. There were occasional exceptions to this when the cells were used occasionally to hold produced fluids during platform startup to allow the topsides production system to warm up sufficiently to meet oil export specification. This would occur some four to six times a year for a duration of about eight hours. On other occasion, the cells were used to receive and separate oil from process vessel flush water prior to periodic platform maintenance shutdowns; or as security should the Cormorant A receiving systems shut down temporarily and close the export pipeline path from Dunlin.

The commissioning of the Osprey and Merlin fields occurred in 1991 and 1997 respectively. During this period the CGB cells were occasionally used if the circumstances outlined above required this. Fluids produced during startup of Osprey and Merlin were routed to the CGB cells until such time that their arrival temperature had risen sufficiently to allow effective oil and water separation, and to achieve statutory discharge standards.

Following such events, fluids diverted to the cells were subsequently returned to the topsides and passed through the process system during stable operating conditions. Although the CGB cells were not in routine use, the cells contained some residual oil trapped at the tops of the cells (known as 'attic oil').

From the late 1990s, failures in pipework installed in Legs A and B of Dunlin A, together with minor leaks through the concrete floors of the legs, began to occur. Where pipework was not encapsulated in concrete and where appropriate isolation could be achieved, pipework repairs were undertaken.

In 1999 attic oil leaked from the cells below Leg A through the concrete into the leg. The leak probably followed the path of a redundant vent line, and a significant volume of oil and liberated gas was released into Leg A. The leak into Leg A required the leg to be flooded with seawater, in accordance with the Dunlin A Concrete Structure Emergency Procedures Manual, to reduce the differential pressure and oil ingress rate across the leak path. The leak stopped and oil contained within Leg A was subsequently recovered through the process system. Attempts were made in 2003 to seal the leak path to enable Leg A to be pumped dry prior to effecting permanent repairs. However, the leak could

only be controlled by maintaining seawater in Leg A at a level about 70m above seabed. This continues to prevent access for permanent repairs.

In 2004 attic oil leaked into Leg B as a result of pipework failure due to corrosion in a section of an oil pipeline running between the topsides and the cells (known as a 'rundown line'). As with Leg A, the contained volume of oil was subsequently recovered, but in this case it was possible to repair the pipework.

In order to remove the potential for further oil and gas ingress, a project for the removal of attic oil, and the permanent decommissioning of the CGB cells and associated rundown and oil export lines, was successfully undertaken in 2006/7 by the platform's then operator, Shell. This effectively isolated the CGB cells from the process system, making any occasional use of the cells, as described above, impossible.

However, the partial flooding of Leg A to 70m above the seabed level continues to be maintained to prevent ingress of liquids from the CGB cells into Leg A.

In accordance with the Dunlin A Concrete Structure Emergency Procedures Manual, if further flooding of Leg A occurs, Legs B, C and D must also be flooded to avoid the generation of tensile loads which could have the potential to cause cracking in the roofs of the cells beneath the other legs.

It is possible Legs B, C and D may also experience water ingress over time.

A.4.3.3 Drilling history

Following the drilling of nine exploration and appraisal wells in the Dunlin field prior to platform installation, the first platform development wells were drilled soon after the Dunlin A platform was installed in 1977. In all, the Dunlin A platform has 48 well slots. A number of wells have been re-drilled to access other parts of the reservoir.

The drilling programme has resulted in a total well stock of 34 production and 10 water injection wells, plus one drill cuttings reinjection well (now out of use).

The Dunlin South West hydrocarbon accumulation was developed with an extended reach well drilled from the Dunlin A platform in 1996. In 1998 a second producing well was drilled into Dunlin South West.

In 1997 an unsuccessful (dry) well was drilled in an attempt to appraise and possibly develop the untested Dunlin North West prospect. The well was subsequently plugged.

A.4.3.4 Drill cuttings

As a well is drilled, the rotating cutting tool (the drill bit) must be cooled because this generates significant heat when grinding into the rock. In addition, the resulting rock chips, or 'cuttings', must be removed from the well. Furthermore, as the well gets deeper it is necessary to have sufficient hydraulic pressure at the drill bit in order to overcome any gas pockets or oil pressure encountered as the drilling proceeds to its target depth.

All of these requirements are met by using a drilling fluid, circulating from the topsides drilling rig into the well and returning to the surface, carrying the drill cuttings with it. The fluid is known as drilling mud, a heavier-than-water mixture of oils, synthetic polymers, water and natural clays which are mixed in various proportions to suit the well conditions during the drilling phase.

At the drilling rig on the topsides, the drilling mud is separated from the rock cuttings and the mud is recycled. The cuttings are discharged down a chute

beneath the platform towards the seabed. Inevitably, rock cuttings have a thin film of drilling mud adhering to them.

For Dunlin A platform drilling, a bentonite water-based drilling mud was used to drill all the top sections of the wells, with a mix of water-based drilling muds and oil-based drilling muds used in the deeper well sections.

A drill cuttings accumulation covers part of the Dunlin A CGB structure, sitting on the cell roofs beneath the well conductors, and spills on to the adjacent seabed on that side of the platform's base, as shown in the impression in Figure A.4.3b below.

The Dunlin platform has several years of its production life still to run and further drilling from the platform is likely. However, any future Dunlin drilling programme will require cuttings to be shipped to shore for disposal, therefore the current drill cuttings accumulation will not change.

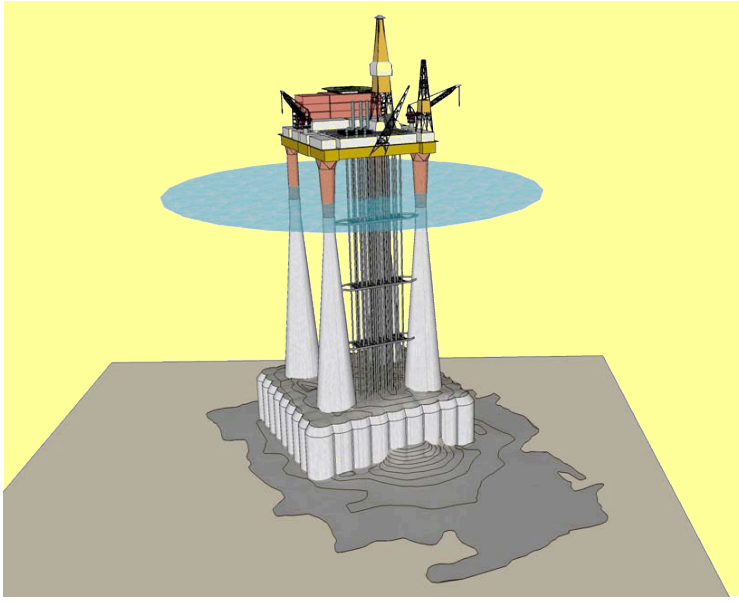

Figure A.4.3b Dunlin A showing a representation of the drill cuttings accumulation at the CGB base

In general, current drilling practice precludes the use of oil-based muds (muds with mineral oils as the base fluid) and the offshore discharge of drill cuttings. However, prior to 1991, this was not the case and therefore oil-based muds were used for some wells on Dunlin A, hence the cuttings accumulation is likely to contain hydrocarbons which might have the potential to affect marine ecosystems.

Chemical analyses of samples from the Dunlin A drill cuttings accumulation are not available but early drilling operations in the field would have used similar fluids to those used for the nearby Brent field. Data for the Brent cuttings accumulation are available. In summary, for Dunlin A the hydrocarbon content is likely to be in the range 30-150g/kg near to the platform, reducing to below 15g/kg at a distance of approximately 100m from the platform.

There have been a number of physical surveys of the cuttings accumulation at Dunlin A to measure its extent and to estimate the probable volume of the deposited material. The latest was undertaken in 1996 by Shell, the former Operator of the Dunlin field. The survey data are shown in Table 4.3 below.

Survey	Cuttings accumulation CGB	Cuttings accumulation on seabed
Volume	4000m^3	10,300m^3
Maximum thickness	Approx 4m	Approx 11m
Surface area	3300m^2	22,000m^2 (worst case estimate)

Table 4.3 Estimated size of the Dunlin A drill cuttings accumulation

The European protocol OSPAR Recommendation 2006/5 on a Management Regime for Offshore Cuttings Piles sets the Best Environmental Practice (BEP) criteria for managing drill cuttings accumulations.

There are two key criteria in the recommendation, which if exceeded, indicate action should be taken to mitigate the environmental effects of drill cuttings accumulations.

The first of these criteria relates to the rate of oil loss from the cuttings to the water column over time. Applying the OSPAR criterion, Dunlin A showed a predicted rate of oil loss to the water column of approximately five tonnes per year, which was below the 10 tonnes per year OSPAR threshold value.

The second criterion relates to the environmental persistence of hydrocarbons over the area of seabed. For Dunlin A this is approximately 125km^2-year, well below the OSPAR threshold value of 500km^2-year.

A.4.3.5 Cells contents

The operational life of the CGB cells has been described earlier (Section A.4.3.2).

The cells system was removed from service in 2006/2007 after an attic oil programme successfully removed mobile oil trapped at the top of the cells. The attic oil programme recovered oil trapped in the spaces in each cell above the oil outlet ports by displacement with carbon dioxide (CO_2) gas. In three of the four cell groups (B, C & D), the CO_2 was removed chemically from the spaces by dosing the seawater in the cells with potassium hydroxide. The cells have been left filled with the treated seawater and the pipework was filled with a gel to inhibit corrosion.

In cell group A, a leak in the cell wall prevented the chemical removal of the CO_2, hence natural scavenging of CO_2 from the seawater has been relied on. The pipework was sealed through injection of buoyant wax particles.

The classes of materials inside the cells include:

- Treated seawater (accounting for 95-97 per cent of the contents volume)
- Inert granular ballast
- Inorganic minerals (clays and sands) originating from the well fluids.
- Hydrocarbons which may have settled at the base of the cells and adhered to the cell internal surfaces.
- Inorganic precipitates (e.g. scales and sediment) formed by reactions in the cells.

- Inorganic material such as trace metals and normally-occurring radioactive material.
- Oil-soluble materials introduced through platform operations.

The contents of the CGB cells and their potential environmental impact if released have been evaluated independently by Intertek METOC. This report concludes that the residual contents of the CGB will not pose an unacceptable risk to the environment. The report can be viewed at http://www.fairfield-energy.com/pages/view/dunlin-cells-contents-impact-assessment.

Appendix B

Concrete gravity base decommissioning options

Appendix B

Concrete gravity base decommissioning options

Contents

B.1 Introduction

A brief description of the theoretical options for decommissioning the Dunlin A CGB is presented in this appendix, without comment on their feasibility or relative merits. Six of these options were presented to stakeholders on 21 January 2010 in Aberdeen, as part of a public consultation process; a seventh option was added in July 2011.

The seven theoretical decommissioning options for the Dunlin A CGB are as follows:

- Re-use of the platform at its current location

Plus three total removal options:

- Refloat and tow the platform for re-use at another location
- Refloat and tow the platform inshore for deconstruction and onshore recycling and disposal of materials
- Complete in situ (at current location) deconstruction of the platform for removal to shore and onshore recycling and disposal of materials

Plus three in situ decommissioning options, leaving the CGB wholly or partly in place:

- Topsides and steel columns removed to 8m below sea level, with the remaining structure marked by navigation aids mounted on a vertical extension to one CGB leg.
- Topsides, steel columns and concrete legs removed to 55m below sea level to provide clear water for navigation, as required by the International Maritime Organization (IMO).
- Topsides removed followed by controlled collapse of the four CGB legs to the seabed. (This option was added to the theoretical options in July 2011, and was not presented to stakeholders on 21 January 2010 in Aberdeen)

For all the above options, prior to topsides removal, topsides production facilities and pipelines would be flushed to remove hydrocarbons, and wells plugged and abandoned prior to cutting and removing the well conductors either below seabed level or above the CGB cell roofs. All activities would be carried out in compliance with Best Environmental Practice and relevant regulations, and pipelines would be addressed in accordance with UK Department of Energy and Climate Change (DECC) guidelines.

Each of the above decommissioning options is addressed in detail in separate Fairfield Energy study reports. These may be accessed through the Dunlin website at: http://www.fairfield-energy.com/pages/view/dunlin-study-reports. The diagrams in Sections B.6 to B.8 show a representation of the drill cuttings accumulation on the CGB base and seabed.

B.2 Re-use at current location

The end of the economic life of the Dunlin A facilities will be defined by the exhaustion of recoverable hydrocarbon reserves in the catchment area. Therefore any future re-use of the platform would be for a non-hydrocarbon venture. This assumes the design life of the CGB could be extended, and would require replacement of the current topsides.

Regardless of the type of new use (for example, carbon dioxide sequestration or wind power generation), at the end of the new use the CGB would still remain in place and would require decommissioning at some future date.

B.3 Refloat and tow for re-use at a new location

This option is only likely to occur should another use arise at the end of Dunlin's field life. Furthermore, re-use represents a postponement of the final decommissioning operation rather than a genuine decommissioning option.

B.4 Refloat and tow inshore for deconstruction and disposal

The offshore industry's current maximum heavy lift vessel capability is approximately 14,000 tonnes. While there are current plans to develop lift concepts with up to 40,000 tonnes capacity, there are no anticipated plans to develop a vessel with sufficient capacity to lift the 320,000 tonnes CGB. Consequently, buoyancy must be used to refloat Dunlin A from its current location.

The platform could be relocated with topsides in place, although the additional weight of the topsides at the highest point of the installation would make the refloat of the structure significantly more challenging. Whether the topsides was removed offshore, or left in place and subsequently removed inshore, the topsides would be dismantled and the materials recycled.

The CGB would be deconstructed in stages, the final stages requiring a dry dock. The concrete generated by this process could be recycled.

The sequence of activities required to refloat and deconstruct the CGB inshore would be as follows:

- Remove topsides and take to shore, or leave in place
- Remove drill cuttings accumulation
- Refloat platform
- Remove and treat ballast water
- Transport to inshore deep water location
- Remove topsides if still in place
- Partially deconstruct the CGB inshore and remove solid ballast
- Move partially deconstructed platform into dry dock
- Complete deconstruction and disposal onshore

B.5 In situ deconstruction

In order to deconstruct the Dunlin A platform in its present location (in situ), the following activities would be necessary:

- Remove topsides
- Remove conductor support frames
- Remove drill cuttings accumulation
- Remove ballast water in cells
- Remove concrete legs by cutting and lifting by floating crane
- Cut and remove cell roof sections in pieces capable of lifting by floating crane
- Cut and remove cell wall sections in pieces capable of lifting by floating crane
- Cut and remove cell floor sections in pieces capable of lifting by floating crane
- Cut and remove skirt sections in pieces capable of lifting by floating crane
- Clear seabed of all debris

Because of the complex geometry of the wall intersections and the thickness of the concrete sections in the CGB, this option would require development of technologically advanced remotely operated subsea cutting tools and methods, and new bracing methods for the concrete legs during the cutting operations.

B.6 In situ decommissioning to 8m below sea level

In some cases, under both the DECC Guidelines and OSPAR Decision 98/3, concrete gravity base platforms installed before 1999 can be decommissioned by removing the topsides and leaving some or all of the main concrete gravity base in situ.

One approach for in situ decommissioning of Dunlin A would be to remove the topsides and all external platform steelwork, and the steel columns at the top of the legs, and leave the entire concrete structure in place. The tops of the concrete legs would be at 8m below sea level. As this would provide no navigable water over the CGB, the structure would require marking with navigation warning devices, as required by the IMO.

The activities required for this option would include:

- Remove topsides
- Remove conductor guide frames
- Remove steel columns to 8m below sea level
- Install a vertical extension to one leg to support navigation warning devices above sea level

The resulting structure is shown in Figure B.6.

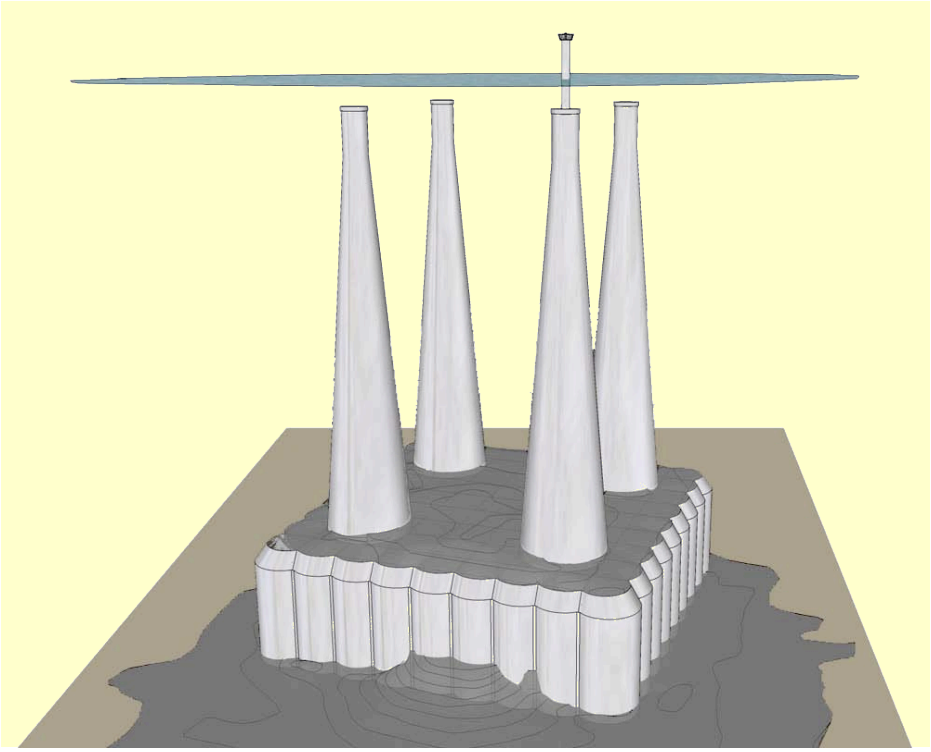

Figure B.6 In situ decommissioning to 8m below sea level

B.7 In situ decommissioning to 55m below sea level

For in situ decommissioning of Dunlin A, a second approach would be to remove the topsides and the upper part of the legs to give 55m clear water below sea level, to provide freely navigable water over the remaining parts of the structure, as required by the IMO.

The activities required for this option would include:

- Remove topsides
- Remove conductor guide frames
- Cut and remove legs to 55m below sea level, requiring restraint of partially cut legs while completing the cutting and lifting of the freed section.

The resulting structure is shown in Figure B.7.

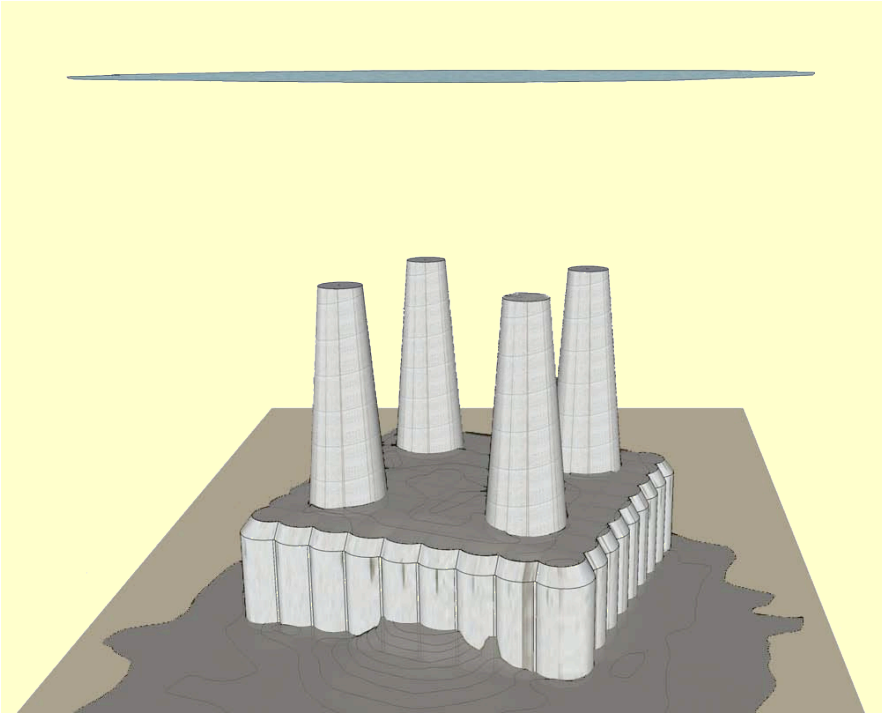

Figure B.7 In situ decommissioning to 55m below sea level

B.8. In situ decommissioning to 110m below sea level

A third approach for the in situ decommissioning of the Dunlin A CGB would be to remove the topsides, followed by conducting a controlled collapse of the legs as illustrated in Figure B.8. The activities required for this option would include:

- Remove topsides
- Remove conductor guide frames
- Deploy explosive charges to create collapse of the legs in a controlled and predictable manner at a level around 10m above the CGB base. An alternative method of collapsing the legs would be to use diamond wire cutting technology to progressively cut the leg wall sections segmentally (i.e. not through the leg cross section) In both cases the internal pipework would be bent or sheared as each 7600 tonne leg fell to the seabed.

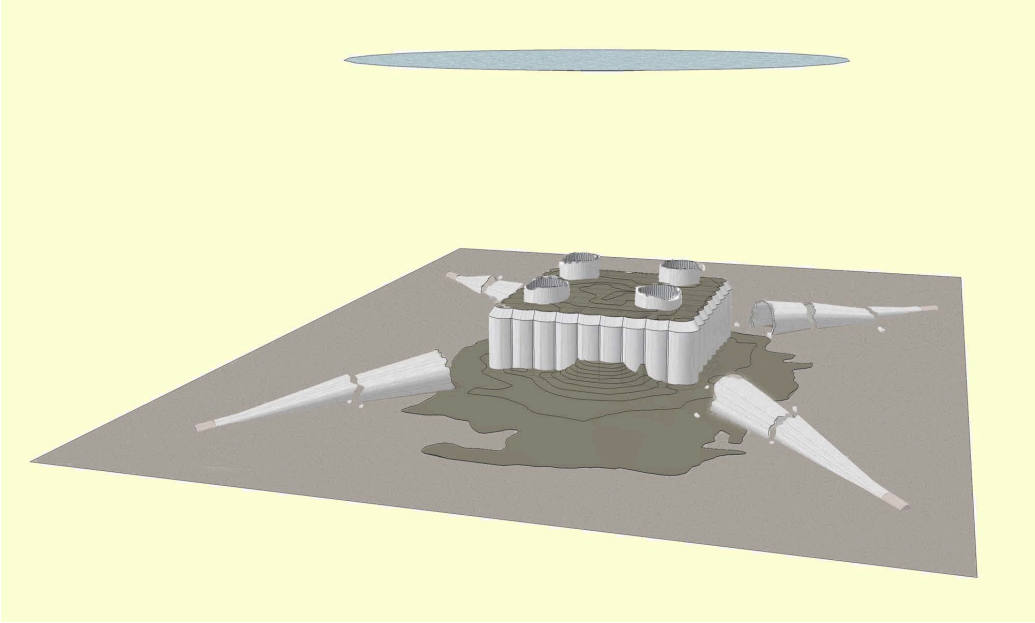

Figure B.8. In situ decommissioning to 110m below sea level

Appendix C

Review of navigation aids installation by Atkins

ATKINS

Fairfield Energy Ltd.

Dunlin A Navaid Post-Decommissioning Study

Final Report

Report No: 5073937-001-ER-01
Issue: 06

Issue Date: November 2011

Dunlin A Navaid Post-Decommissioning Study

Final Report

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DOCUMENT ISSUE CONTROL SHEET

Fairfield Energy Ltd.
Dunlin A Navaid Post-Decommissioning Study

Issue	Date	Purpose	Rev	Prepared	Checked	Approved
1	June 2008	For Internal Review	Draft	PM/TH	PW	TH
2	Sept 2008	Issued to Client	2	PM/TH	WT	TH
3	29 Sept 2008	Final Issue	3	PM/TH	WT	TH
4	22 May 2009	Additional text added	4	TH	PM	TH
5	Oct 2011	Incorporating Client request	5	CG/AM	AM	TH
6	Nov 2011	Figure numbering corrected	6	AM	AM	TH

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EXECUTIVE SUMMARY

Fairfield Energy Ltd commissioned Atkins to evaluate the options for installing a navigation aid on top of one of the Dunlin A CGB legs, should the CGB be decommissioned in place and submerged to 8m below sea level. This report presents Atkins' conclusions and recommendations.

The evaluation has shown that it would be possible to build and install a navigation aid unit on top of one of the CGB legs. Atkins has carried out a conceptual design exercise to demonstrate this. A prestressed reinforced concrete structure would be required to be installed on the leg to support the navigation aids, which would be designed as a zero-maintenance structure for a target period of 100 years.

The navigation aid systems could be similar to the Liaaen AtoN system which has been installed by operator Total on the decommissioned East Frigg CGB platforms in the North Sea. Maintenance of the system would be carried using helicopter support.

It is estimated that the legs of the Dunlin Alpha CGB are likely to remain intact for several centuries before the concrete begins to degrade and the legs being to collapse [9].

Mounting navigation aids on tethered buoyed structures has not been considered as discussions between Fairfield and the Northern Lighthouse Board have determined that use of buoys in open waters would not be acceptable as the buoys could break free, creating shipping hazards and leaving the CGB unmarked.

1. INTRODUCTION

1.1 BACKGROUND

Fairfield Energy (Fairfield) is operator of the Dunlin Cluster of fields in the UK sector of the North Sea, which includes the Dunlin Alpha concrete gravity base (CGB) platform (Figure 1).

Figure 1 : Dunlin Alpha

Fairfield has started to review future decommissioning options. It is considered likely that the Dunlin Alpha CGB will be granted derogation and will be left in situ. However, it is assumed that the UK regulator, the Department of Energy and Climate Change, would require all of the external steel work to be removed. The concrete/steel interface on the legs is circa 8m below lowest astronomical tide (LAT), so that the structure would need to be marked by navigation aids (navaids) above the sea surface if the CGB is left in situ submerged to 8m below sea level.

Fairfield requested Atkins to identify available options to site a navigation aid package on the CGB structure. The navigation aid could be mounted on a suitable form of extension to a single concrete leg. Mounting navigation aids on tethered buoyed structures has not been considered as discussions between Fairfield and the Northern Lighthouse Board have determined that use of buoys in open waters would not be

acceptable as the buoys could break free, creating shipping hazards and leaving the CGB unmarked.

1.2 SCOPE OF WORK

The scope of work undertaken was as follows

1. Data acquisition and familiarisation

2 Generation and development of selected ideas. This involved approximate sizing of structural members, listing of pros/cons and decisions to continue or discard certain options. Conceptual designs were produced for selected options, and design sketches prepared, along with preparation of methods of installation and dismantling, assessment of likely failure mechanisms, some more detailed development of selected options, and an assessment of life expectancy.

3. Estimation of weights for selected options.

4. Preparation of this report for presentation to Fairfield Energy Ltd.

2. PLATFORM DESCRIPTION

2.1 GENERAL ARRANGEMENT

A general arrangement of the Dunlin A substructure is shown in Figure 2. There are four main legs extending upwards from the CGB base, which support a steel topsides structure. Two of the legs (C and D) are also interconnected by conductor guide frames that reduce the free span of the conductors. The remaining two legs (A and B) contain services.

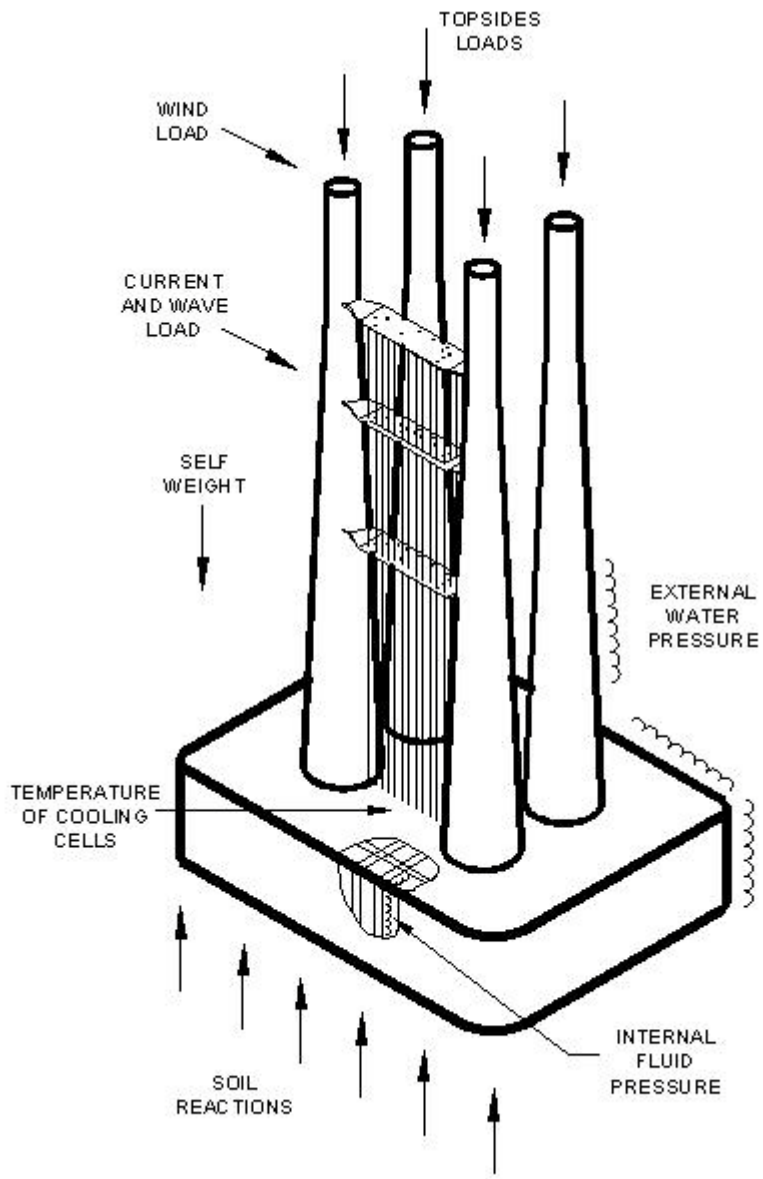

Figure 2 : Dunlin A Substructure

The water depth at Dunlin A is 151.0 m above the structural datum at the sea bed (the underside of the concrete base slab, ignoring skirts).

The structure in operation is designed for the combined effects of the self weight, topsides load, wind, wave and current loads, differential pressures and temperature loads, as shown above.

Due to the geometry of the topsides, the majority of the topsides load is concentrated on conductor legs C and D, which also resist wave loads from the conductors.

Key elevations and dimensions for Leg B are given in Table 1.

DESCRIPTION	ELEVATION (M ABOVE DATUM)	COLUMN OD (M)	COLUMN ID (M)	THICKNESS (M)
Top of ring beam	142.890	6.600	4.200	1.200
Bottom of ring beam	139.000	6.600	4.800	0.900
Varying diameter and thickness of leg	130.000	6.534	5.110	0.712
	128.000	6.814	5.438	0.688
	120.000	7.986	?6.748	0.619
	110.000	9.652	8.382	0.635
	100.000	11.320	10.018	0.651
	90.000	12.986	11.652	0.667
	80.000	14.652	13.288	0.682
	70.000	18.318	14.922	0.698
	60.000	17.984	16.556	0.714
	50.000	19.650	18.190	0.730
	40.000	20.984	19.770	0.607
Top of CGB base	32.000	22.650	20.250	1.200

Table 1: Leg B Elevations

2.2 CONCRETE PROPERTIES

The concrete grade for the CGB is such that the characteristic and nominal properties in accordance with NS3473 [12] or DNV [11] rules are as follows:

- Cube strength (at 28 days) of 45 MPa
- Nominal compressive strength of 28 MPa
- Characteristic tensile strength is 2.95 MPa
- Nominal tensile strength is 2.0 MPa

However, the concrete has aged at least 30 years. During this period, the increase in strength is certain to be at least 20 % (typical concrete cores indicate up to 40 % increase). This increase in strength may be calculated in accordance with MC90. A cube strength of 55 MPa has been used in other reports [9]. It is quite probable that these design concrete strengths are conservative, at least for the material between the reinforcement layers, where better compaction has occurred.

The concrete modulus has been taken as 30,000 MPa, similar to that used by Aker Kvaerner [13]. Poisson's Ratio for concrete is 0.2. The density for plain concrete was taken as 2,400 kg/m^3 and for reinforced concrete was taken as 2,600 kg/m^3.

An assessment of the eventual failure mechanisms of the platform due to degradation of the structural concrete has been carried out by Atkins [9]. This assessment assumed that the platform would be decommissioned and the concrete gravity base left in place with the steel topsides, columns and conductors removed. Additionally it was assumed that either Leg A or Leg B would be modified to support navigation aids.

2.3 STEEL REINFORCEMENT

The steel reinforcement in the CGB has a yield stress of 410 MPa. The steel reinforcement modulus has been taken as 200,000 MPa in accordance with DNV rules [11]. Poisson's Ratio is 0.3.

Reinforcing steel areas vary with elevation along the legs. These was taken from the civil drawings, checked against the Aker Kvaerner shaft report [13].

Cover to the main hoop (outer) reinforcement is a minimum of 50 mm along the legs. Cover to the vertical (inner) reinforcement is a minimum of 80 mm.

2.4 PRESTRESSING SYSTEM

Prestressing is arranged horizontally and vertically in the walls of the base. There is no hoop prestressing in the legs, other than as provided by drawdown of internal water levels relative to the external water level, and at the conductor framing supports. The vertical post-tensioned prestressing system [10] comprises an increasing number of tendons with depth down the leg. For Legs A and B:

- at the top of the column – 68 number 12 x 0.5" Freyssinet cables;
- at 110 m elevation – 64 number 12 x 0.5" Freyssinet cables;
- below 100 m elevation – 96 number 12 x 0.5" Freyssinet cables.

The tendon proof stress is 1700 MPa. The tendon effective modulus was taken as 196,000 MPa. The area of each 0.5" strand is 98.7 mm^2, and of each 12 strand cable is 1184 mm^2.

The nominal strain in the prestressing tendons has been quoted as 0.00477, giving a stress of 935 MPa (55 % of the proof stress) and a total load per cable of 1106 kN. Prestressing loads at the top and bottom of the column were therefore taken as given in Table 2.

ELEVATION (M ABOVE DATUM)	CABLES	PRESTRESS LOAD
128-139	68	71.9 MN
110	64	67.6 MN
32-100	96	101.5 MN

Table 2 : Prestressing Loads in Legs A and B

2.5 DECOMMISSIONED ARRANGEMENT

The assumed decommissioned state of the platform with have the topsides, conductors, guide frames and steel columns removed. In addition to this, the legs will be flooded.

3. BASIS OF DESIGN

3.1 DESIGN CODES OF PRACTICE

The following design codes and sources of data were used for this study

- OSPARCOM Guidance [14]
- DTI (DBERR) Guidance
- DNV [11]
- BS8110 [17]
- MC90 [18]
- NS3473 [16]
- Concrete in the Oceans Data [19]

3.2 DESIGN BASIS FOR NAVAIDS

3.2.1 Navaid Dimensions and Weights, Maintenance Requirements

One possible proprietary navaid device considered, suggested by Fairfield, is the Liaaen Aid to Navigation (AtoN) unit, similar to that installed by operator Total on the decommissioned East Frigg CGB platforms. Details of this equipment are presented in Appendix A. Figure 3 and 4 illustrate the as-left configuration at Frigg.

Figure 3 : East Frigg TCP2 Concrete capped with AtoN Navaid on top

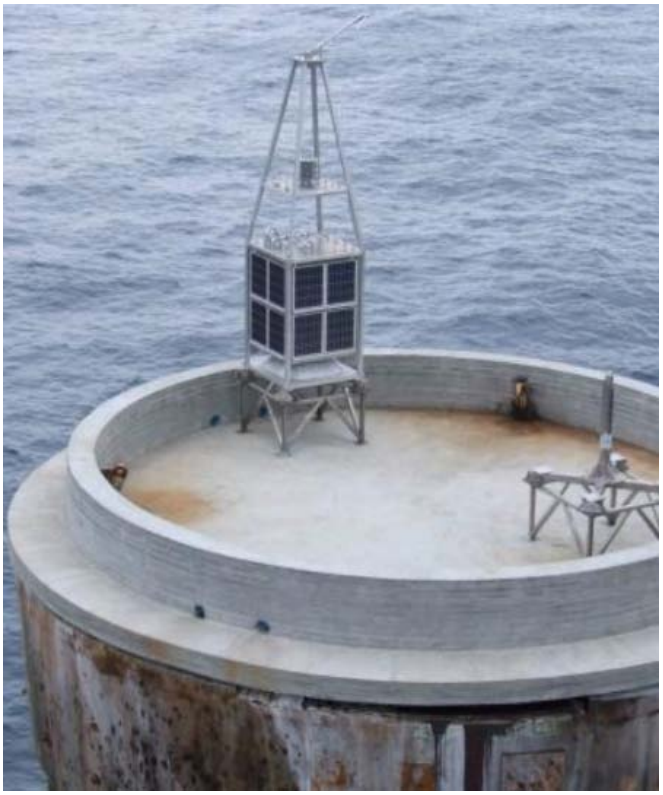

Figure 4 : Detail of Cap Slab and AtoN Support Units

3.2.2 Navaid Design Criteria

If it were attached to the Dunlin CGB, the navaid would be required to be located at a height above mean sea level (MSL) sufficient to clear the 10,000 year wave crest elevation, and withstand associated tide, long term settlement and water surface effects, and the effect of water flow over the CGB base structure. The maximum 10,000 year crest elevation above MSL is given by the environmental report [22] as 21.45 m.

For extended offshore life (>100 years), the design wave and current loading on the navaid support structure and the leg below it, would normally be based on 10,000 year environmental data. However, such information is not given in the environmental report [22] and design loading has therefore been based on 100 year data with a suitable factor to allow for increased loading. This factor has been based on the ratio of crest elevations squared, equal to $(21.45/18.40)^2 = 1.36$.

3.3 POTENTIAL FUTURE DEGRADATION MECHANISMS

The assessment found that initial failure of the leg carrying the navaid is most likely to occur circa 20m below sea water level at the transition between the conical to cylindrical sections of the leg, where the leg cross section is at a minimum [9]. This failure was predicted by Atkins to occur approximately 250 years or more after the installation of the CGB and would result in the toppling of the upper leg. This failure mechanism may result in a significant dropped object risk to the integrity of the cells below.

4. CONCEPTUAL OPTIONS

4.1 PLATFORM CONFIGURATION

In accordance with OSPAR, it is envisaged that derogation could be sought so that the CGB structure could be left in a '*Lighthouse'* configuration, with the topsides removed.

However, since the remaining structure would be surface-piercing, or within 55 m of the surface, it is required by regulations that the CGB be marked by navigation aids such as a Liaaen AtoN navaid unit, similar to that shown in Figure 5.

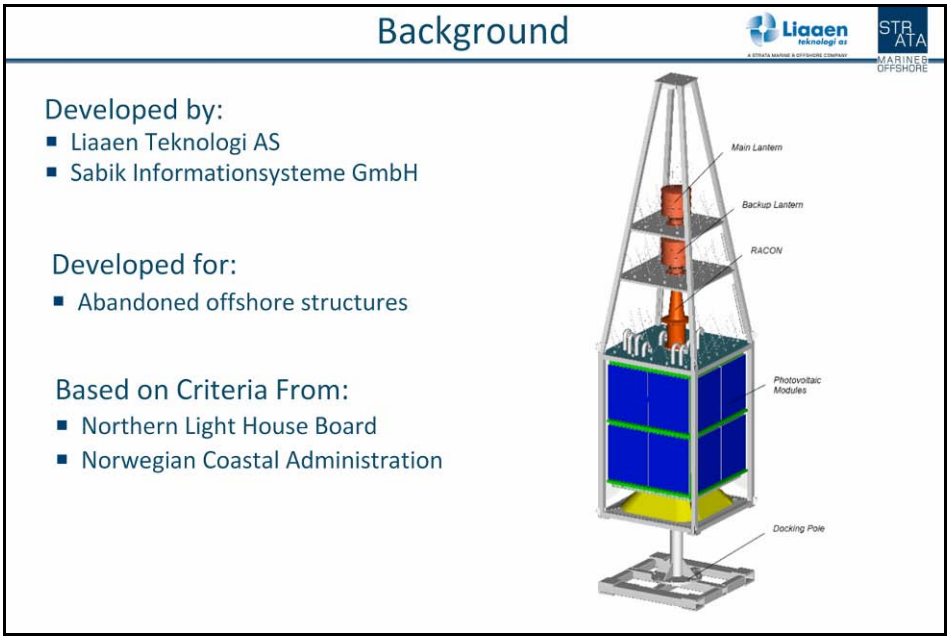

Figure 5 : Liaaen AtoN navaid unit

Only one such navaid unit and support would be required to be operational at any given time. If fixed to the structure, the navaid would need to be at a level of about +151+25 = +176 m to remain above the 10,000-year return wave crest. The 25 m elevation includes an allowance for tide, CGB "caisson" effects and long term settlement / global warming.

The structural configuration at Dunlin is different from Condeep or other concrete structures, such as Frigg. Issues specific to Dunlin Alpha are as follows:

- Because the concrete-steel transition at Dunlin is lower than other structures (i.e., below the water level), support of this navaid suggests that the leg would likely need to be built up.
- If all of the steel column is to be removed, this would involve underwater cutting to reduce the structure to the concrete ring beam and attachment of navaids and plugs underwater. However, options to leave steelwork in place to degrade might also be possible if deemed acceptable to OSPAR regulations.
- The upper leg diameter for the Dunlin CGB is relatively small, making removal of structures from inside the legs even more difficult than for other concrete structures.

- The legs are normally dry internally, and the conical section has a significant taper, so that the effect of external water pressure will be to provide significant compression and increase the leg bending capacity. This capacity would be significantly reduced if the leg were flooded, although normally adverse local bending at the base of the legs would be avoided.

4.2 MATERIALS SELECTION

Materials selected for the navaid support structure would need to be durable for a significant number of years. Common construction materials are listed below:

1. Carbon steels are generally discounted because of high corrosion rates and significant maintenance requirements.
2. If steel is to be selected, then stainless steel would be preferred as this limits corrosion.
3. Concrete and grout have very good offshore durability, and are the most likely materials to be used.
4. Glass Reinforced Epoxy (GRE) appears possible, but has a maximum life of only 20 years in seawater, breaking down due to osmotic delamination.
5. Aluminium or other metals are discounted, as the material weight is not significant and fatigue characteristics are generally worse than for steels.

Material selection should also avoid pollution of the environment due to long term degradation. Concrete is perhaps the least invasive material. However, eventual collapse of large concrete structures would be more likely to cause dropped object damage to the cells in the CGB, and introduce more risk of leakage from the cells, if not inert by then.

4.3 INSTALLATION AND MAINTENANCE CONSIDERATIONS

Consideration has been given to the ease of installation and maintenance of the navaid and its supporting structure.

The Liaeen device is relatively light in weight (see Appendix A - Details of Liaeen Teknologi A/S Navaid Package) and is intended to be replaced by helicopter. This is considered to be the optimum solution, since the entire device could be replaced easily for refurbishment onshore.

At Frigg, there is room to lay down one navaid while handling the other (original or replacement unit). This requires the helicopter to carry only one navaid at any time. Such laydown is not directly available for Dunlin due to the top of the leg being below water. A dual mounting scheme could be considered at the top of the navaid support structure, otherwise a more capable helicopter would be necessary.

4.4 LIGHTHOUSE OPTIONS

A number of options for extending the leg upwards to support the navaid package have been considered.

Options A1, A2, B, C, D and E are all fixed structures. The concepts are based on the use of prestressed reinforced concrete for corrosion resistance and maximum offshore life. The use of steel, GRE and other materials has been discounted on the basis of cost and durability. The options are presented in Figure 6 to Figure 11 below.

Option A1 (see Figure 6) was the starting point but proved to be too slender for the hydrodynamic loading and dynamic response.

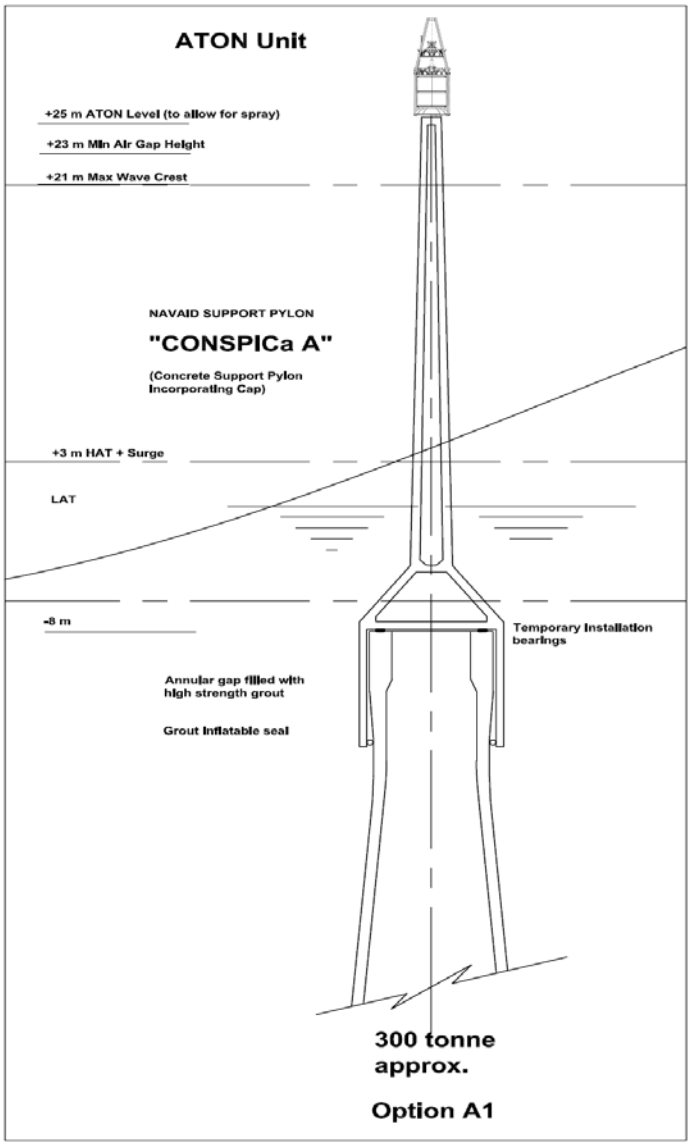

Figure 6 : Lighthouse Mode Option A1

Option A1 was discarded in favour of ***Option A2*** which proved to be capable of being prestressed with a practical number of tendons to prevent cracking and resultant deterioration (see Figure 7). This scheme comprises a concrete leg integral with a cap that can be grouted over the existing top of shaft ring beam. However, it could only easily be deployed on legs A and B because the other legs have out-of-circular prestressed concrete ring beams for the conductor framing, which would prevent easy installation of the “overshot” connection. This may limit life expectancy, as legs A and B are not as strong as legs C and D.

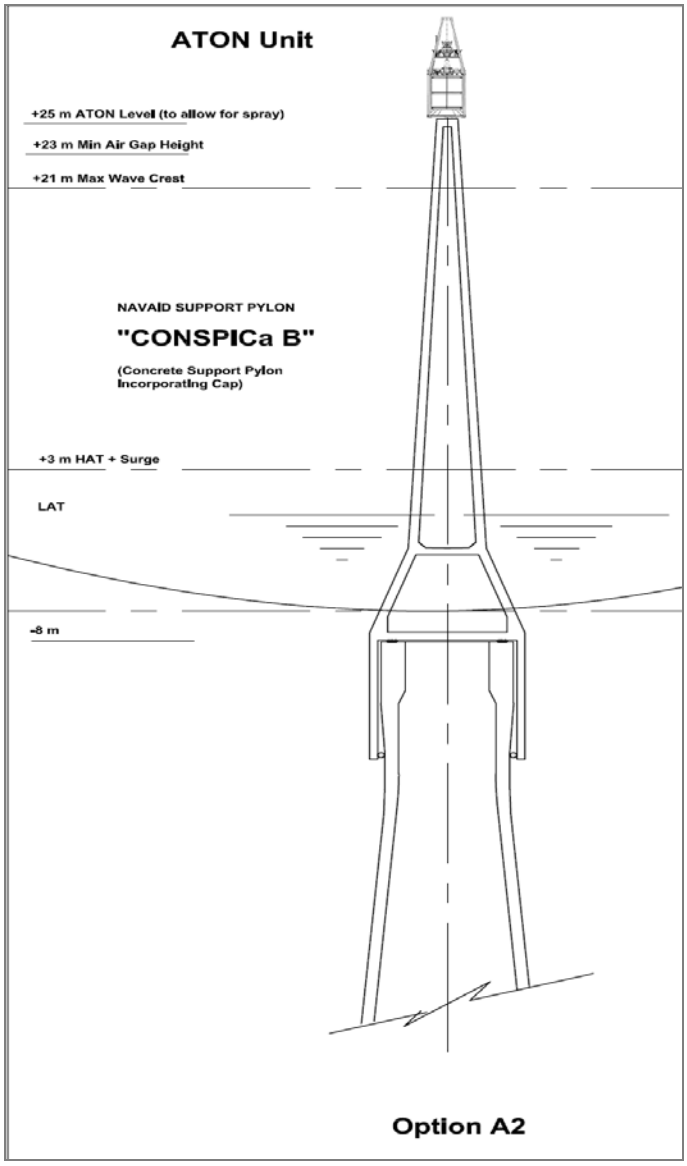

Figure 7 : Lighthouse Mode Option A2

Option B is a variant of Option A2 (see Figure 8) with the lower section being inserted into the top of the shaft and grouted within it, as opposed to the cap detail. It is not considered to be as durable a structural solution as the "overshot" A2 connection detail, as it has a smaller diameter at the interface and does not prevent long term wave erosion at the externally exposed (grouted) joint. However, it would permit the navaid support to be installed more readily on any leg, and installation of legs C and D would improve the life expectancy of the structure, due to their increased capacity to carry additional wave load.

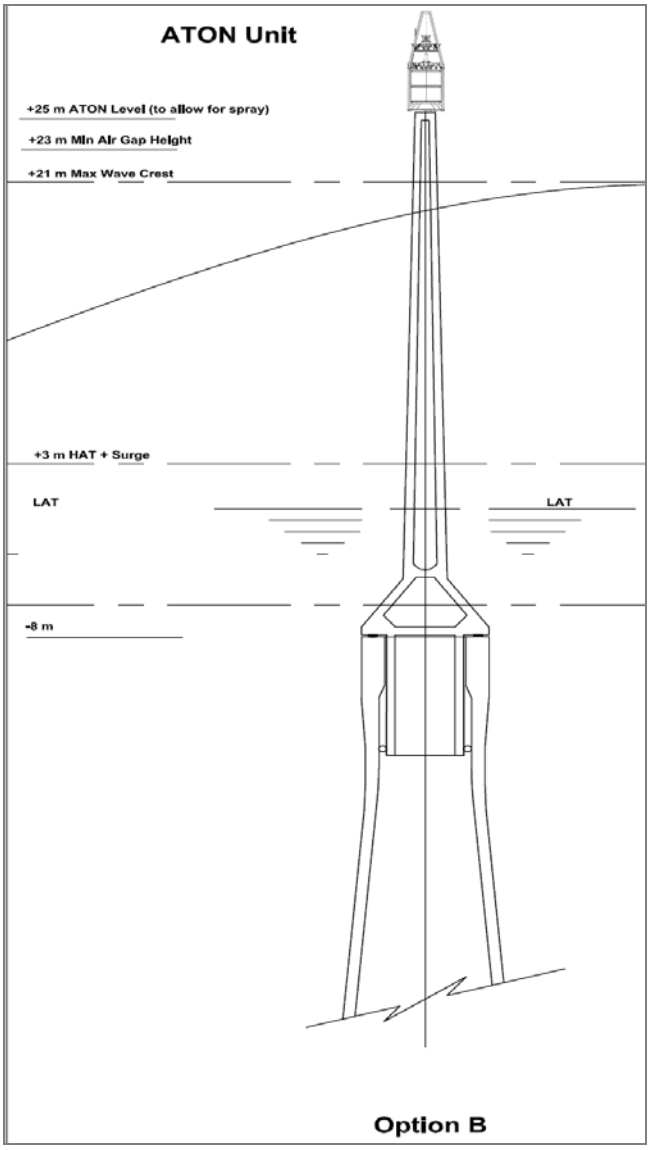

Figure 8 : Lighthouse Mode Option B

Option C was conceived as a solution which could possibly be constructed internally within the steel column, using this to provide a dry environment and to support the concrete or its formwork during pouring (see Figure 9). This would avoid the need for a heavy lift installation of a pre-cast unit such as A2 and would avoid the complication of a subsea grouted connection. It could be constructed in advance of deck removal and would not require the column to be cut off subsea. The steel that surrounds the concrete leg would be left to degrade with time, supported by the concrete leg within. There are therefore several benefits to this option, but the drawback is the cost of significant offshore operations. It would have larger wave loading then Option B and therefore be not as strong.

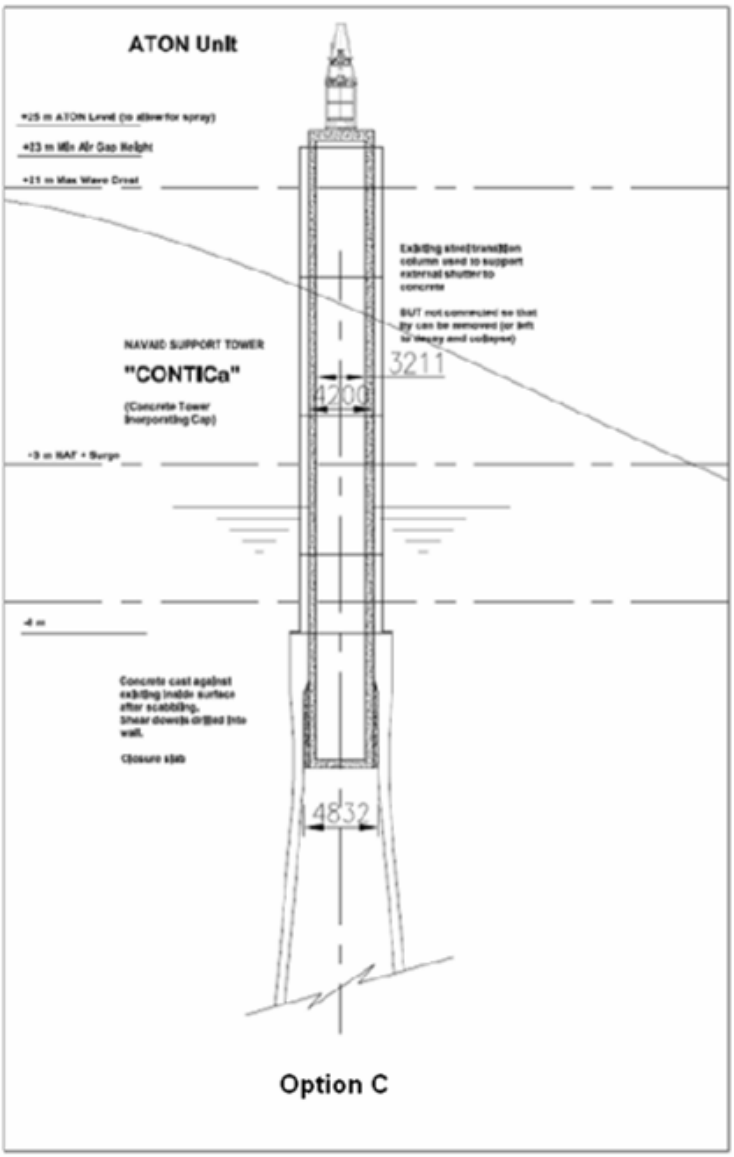

Figure 9: Lighthouse Mode Option C

Option D was conceived as a "concrete jacket" or braced flagpole stabbed into concrete or stainless steel socket piles bolted and stressed onto the external wall of the leg (see Figure 10). On top of this a variant of Option A2 would be stabbed and grouted. This is an inelegant and structurally inefficient solution and was rejected from any further development or costing. However, a variant of this, a propped 'flagpole', is considered feasible. A tripod base structure was envisaged, connected to the prestressed upper ring beam on either of legs C and D, possibly using the existing conductor framing anchorages. The chief disadvantage of any of these structures is the material selection, as concrete is not good for this type of frame structure and stainless steel and GRE are costly or of poor life expectancy.

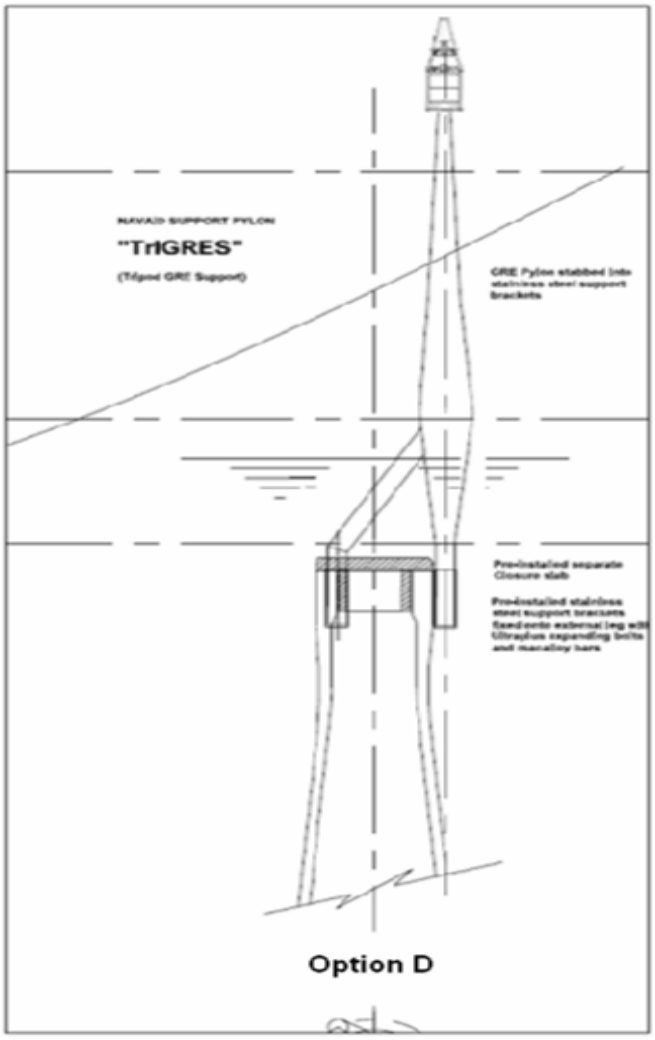

Figure 10 : Lighthouse Mode Option D

Option E is a variant of Option A2 with the modification that its lower section is sized to encompass the bottom half of the steel column (see Figure 11). This would allow the steel column to be removed down to the level of the highest astronomical tide (or possibly lower) without the need for underwater operations. The lower portion of the steel column would then be encapsulated within the concrete lighthouse structure. It would not form part of the structural system, but would be protected by the concrete around it. The advantage here is the lack of subsea cutting, but the disadvantage is a larger and heavier navaid column that would be more difficult to install, particularly over the conical steel columns on legs C and D.

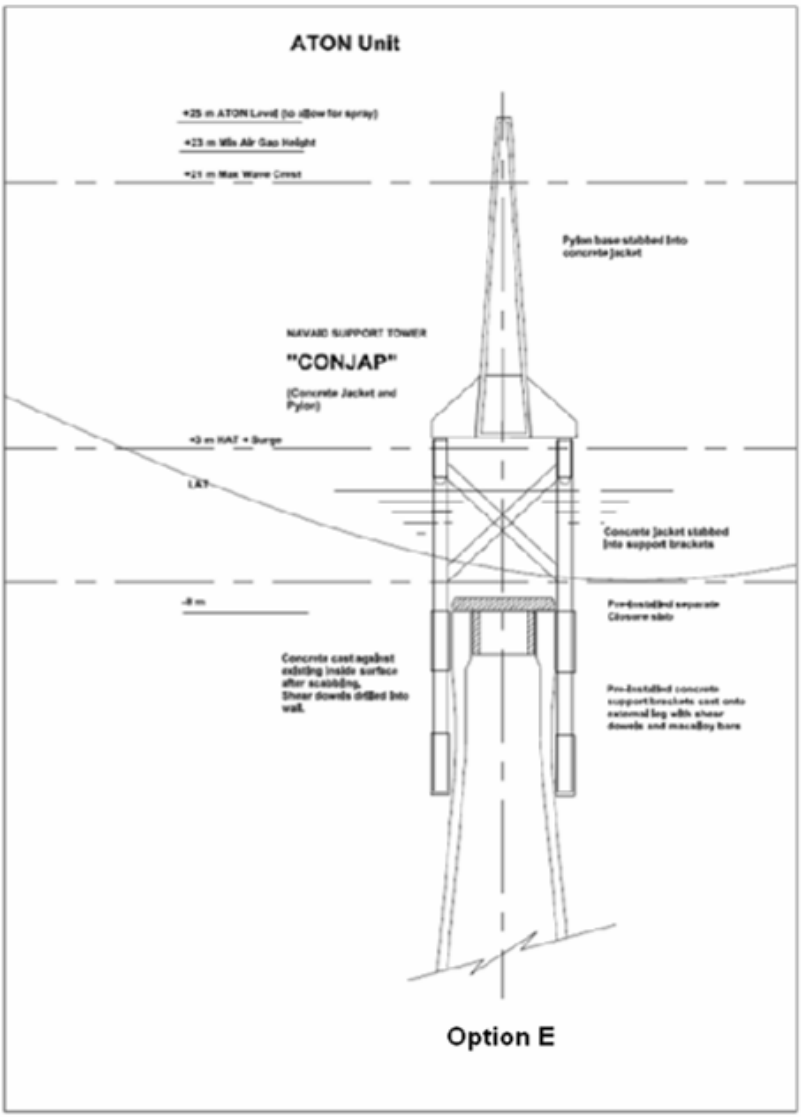

Figure 11: Lighthouse Mode Option E

5. SELECTION AND DEVELOPMENT OF OPTIONS

5.1 CONCEPT SELECTION

Table 3 lists the pros and cons of the various options presented in Section 4:

Option	Pros	Cons
A1	Slender, lightweight, low lift weight	Not strong enough, discounted
A2	Durable larger leg construction Easy to prestress and fabricate	Difficult to fit to legs C and D
B	Can be connected to any leg Lighter construction than A2	Water egress into grout bond Weaker leg connection than A2
C	Construction in the dry No cutting of steel underwater	Debris from degrading steel shaft Weaker leg connection than A2 Lots of offshore fabrication More wave loading.
D	Prefabricated units	Low strength concrete in tension Underwater connections to leg Difficult to construct, low strength Inelegant solution, discounted
E	No cutting of steel underwater Possible grouting from dry interior	Heavier construction and lift Difficulties fitting over steel shaft, particularly for legs C and D

Table 3 : Pros and Cons of Lighthouse Options

On the basis of the above, Options A1 and D are discounted.

5.2 LIGHTHOUSE CONCEPT DESIGN

For lighthouse Option A2, a simple computer model of the leg with the navaid support was generated and analysed using the ASASNL program, as a transient dynamic analysis. This model is illustrated in Figure 12.

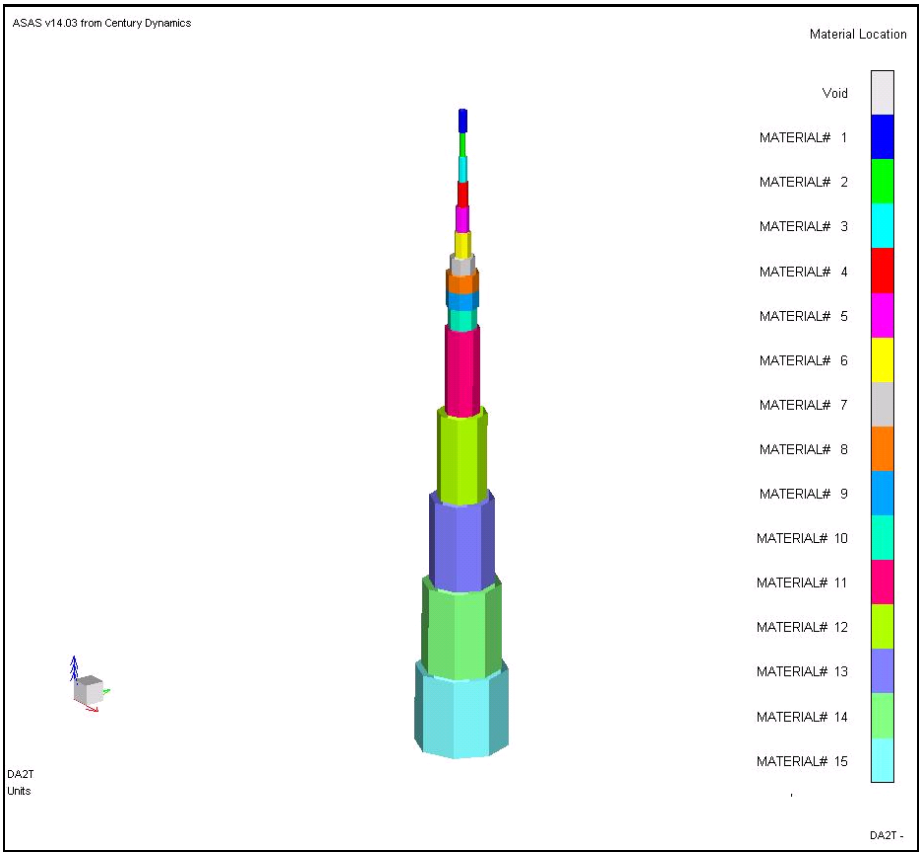

Figure 12: Dunlin Leg Model with Lighthouse Option A2 Installed

The 100-year storm (wave, tide and cu rrent) conditions were considered, but the structure was designed to have sufficient reserve strength for 10,000-year return wave loads. The sea bed was artificially set at the top of the CGB base (151 – 32 = 119 m) to allow for the base causing increased water velocities.

The maximum lateral displacement of the leg at the navaid was 241 mm (see Figure 13).

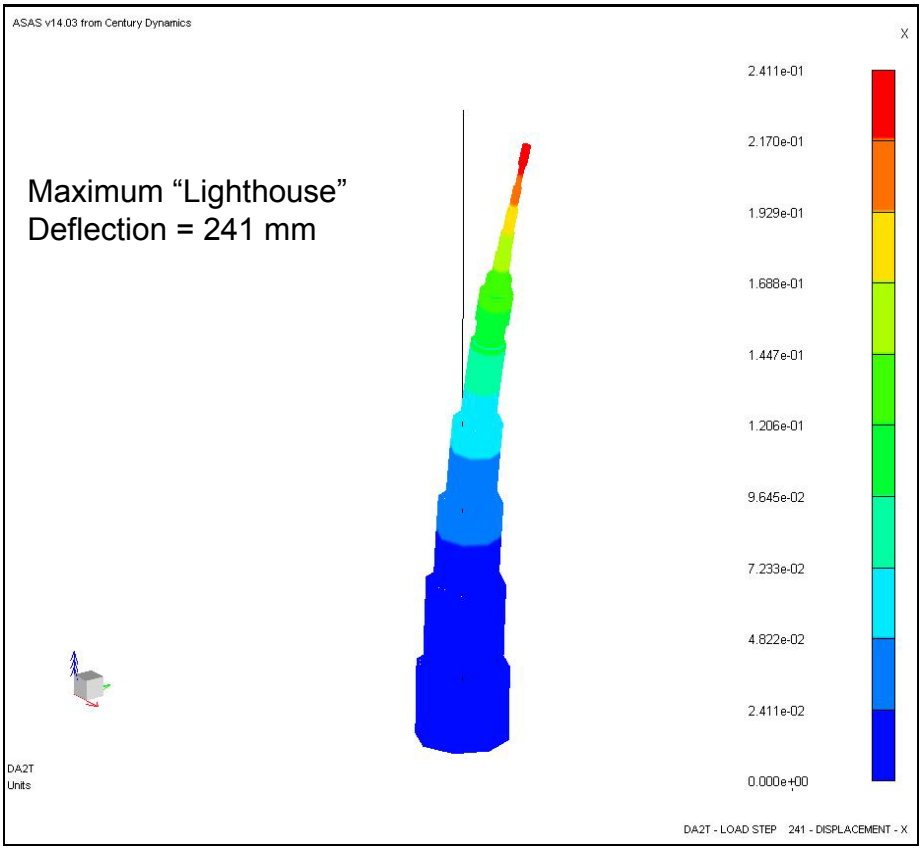

Figure 13: Displacement of Dunlin Leg in 100-year Storm Conditions

Figure 14 shows that the maximum shaft bending moment was 1393 MNm at the CGB roof, and 18.2 MNm at the base of the navaid support leg (just above the leg ring beam). This latter moment, even when allowing for the increase to 10,000-year load conditions, does not require an unreasonable section size.

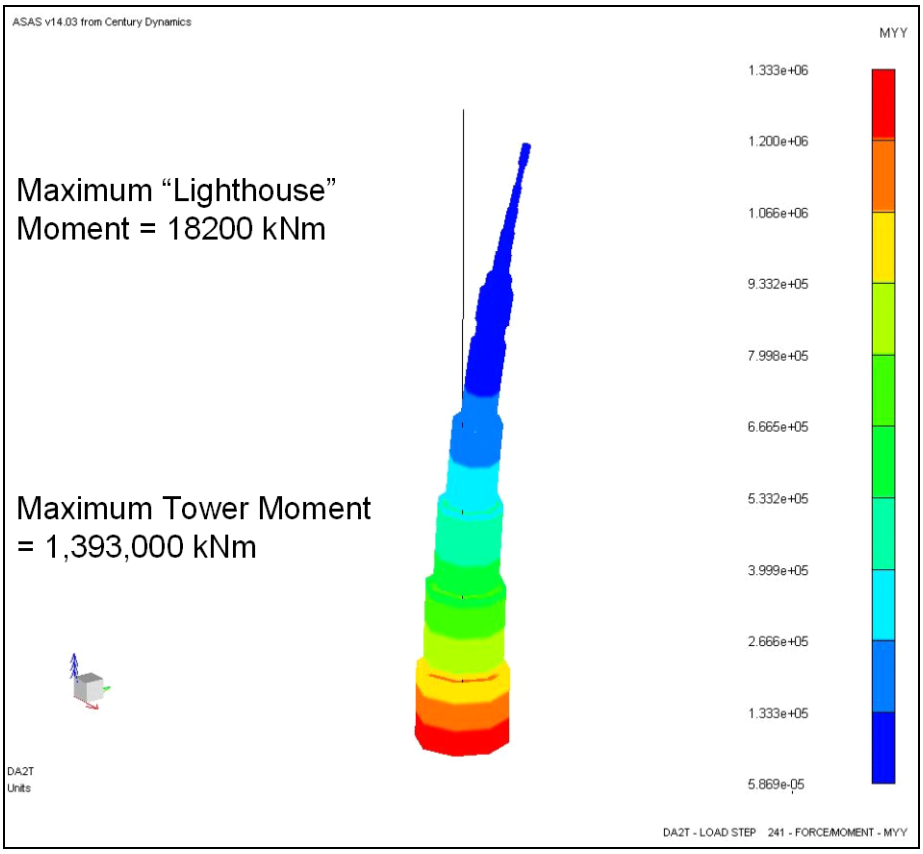

Figure 14 : Moments along Dunlin Leg under 100-year Storm Conditions

The time history of response of the leg indicated that the upper section (the navaid support) would experience its maximum moment a significant time after the leg base (see Figure 15). The reason for this is that the lower leg is dominated by inertial loading, whereas the upper leg experiences mostly drag. The drag loading peaks at the wave crest, one quarter of a wave cycle after the inertial loads.

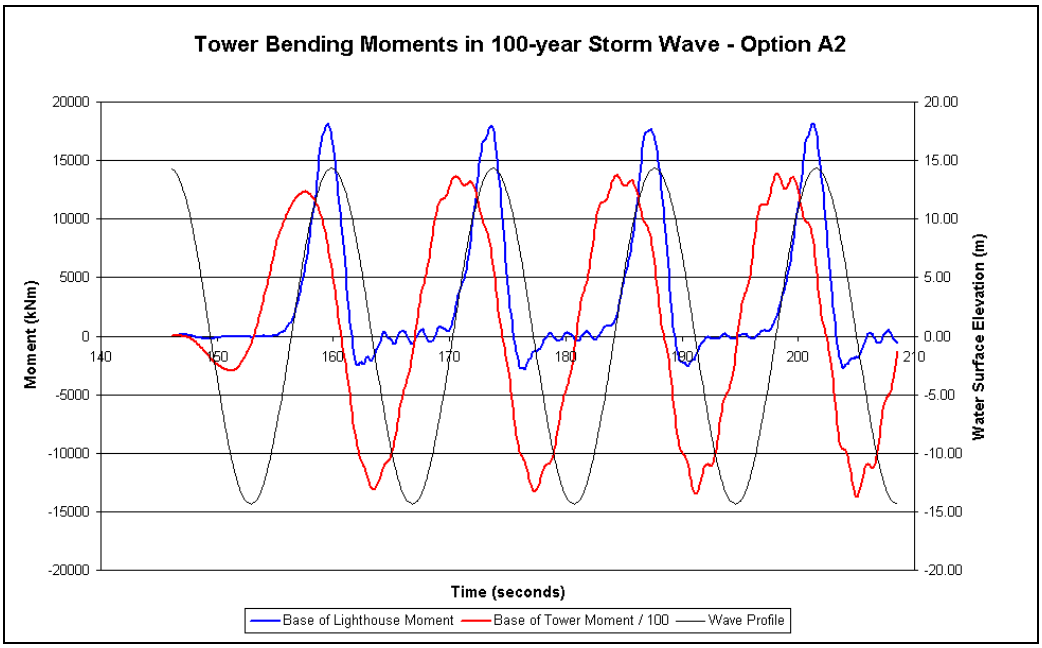

Figure 15 : Time History of Leg Bending Moments

The leg design moment for 10,000-year ultimate limit state (ULS) conditions is 1393 x 1.36 x 1.30 = 2462 MNm. This compares favourably with the 3040 to 4140 MNm reported by others [10] for an in-place condition with the conductors and conductor guide frames removed. The load from the 48 conductors therefore contributes a significant percentage (over half) of the in-place design load on the legs. The leg ULS capacity after ageing is given in [5,6,7,8] as 4360 MNm just above the CGB base roof.

The above analysis is simplistic, but does demonstrate that legs C and D without the conductors should have sufficient strength to resist extreme 10,000-year wave loads, even with the navaid in place, as long as there is no degradation of reinforced, prestressed concrete strength. However, there will be such degradation over time, and the leg is expected to lose strength due to the mechanisms identified in Section 3. It has been estimated [1] that this residual life may be several hundreds of years prior to the leg toppling, as long as the structure is not subjected to a significant accidental event (such as ship impact).

6. CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

1 It would be possible to build and install a navigation aid unit on top of one of the CGB legs. Atkins has carried out a conceptual design exercise to demonstrate this.

2 The installation of a navigation aid unit such as the Liaaen AtoN would require a substantial structure to be placed on top of one of the CGB legs in order to position the lights and beacons at a level which will survive the long term wave climate.

3 In order to achieve a zero-maintenance structure, for a target period of 100 years, a solution using prestressed reinforced concrete would be required.

4 The most practical "Lighthouse" solutions are Options A2 or B for long term low maintenance cost and risk

5 Option A2 is limited to legs A and B only

RECOMMENDATIONS

1 Proceed to develop Options A2 or B.

2 Carry out a brief study of the probable and possible technology changes in GPS navigation, and the International Maritime Organisation recommendations and Rules for the Prevention of Collision at Sea over the next 20 years.

3 Consider ULS, FLS and ALS conditions for navaid support tower in FEED phase.

7. REFERENCES

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APPENDIX A - DETAILS OF LIAEEN TEKNOLOGI A/S NAVAID PACKAGE

Liaaen teknologi as

VOL 1 of 1

FINAL DOCUMENTATION FOR DESIGN AND CONSTRUCTION OF AID TO NAVIGATION FOR TP1 FRIGG 5 / FRIGG 6

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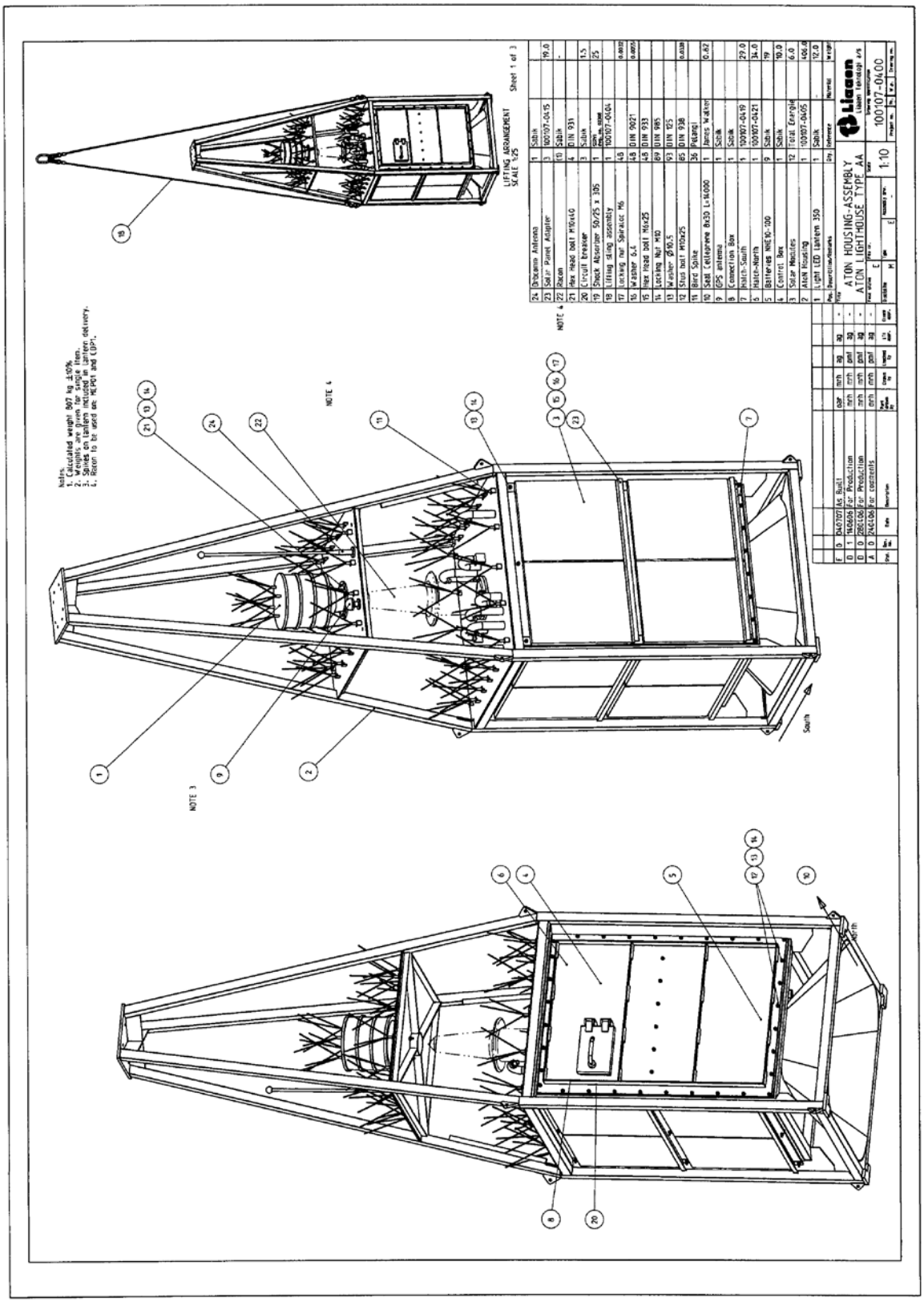
Project no.	**: 100107**
Order no.	**: 1005685**
Doc. no.	**: 007**
Date	**: 30.08.2007**
Rev	**: 00**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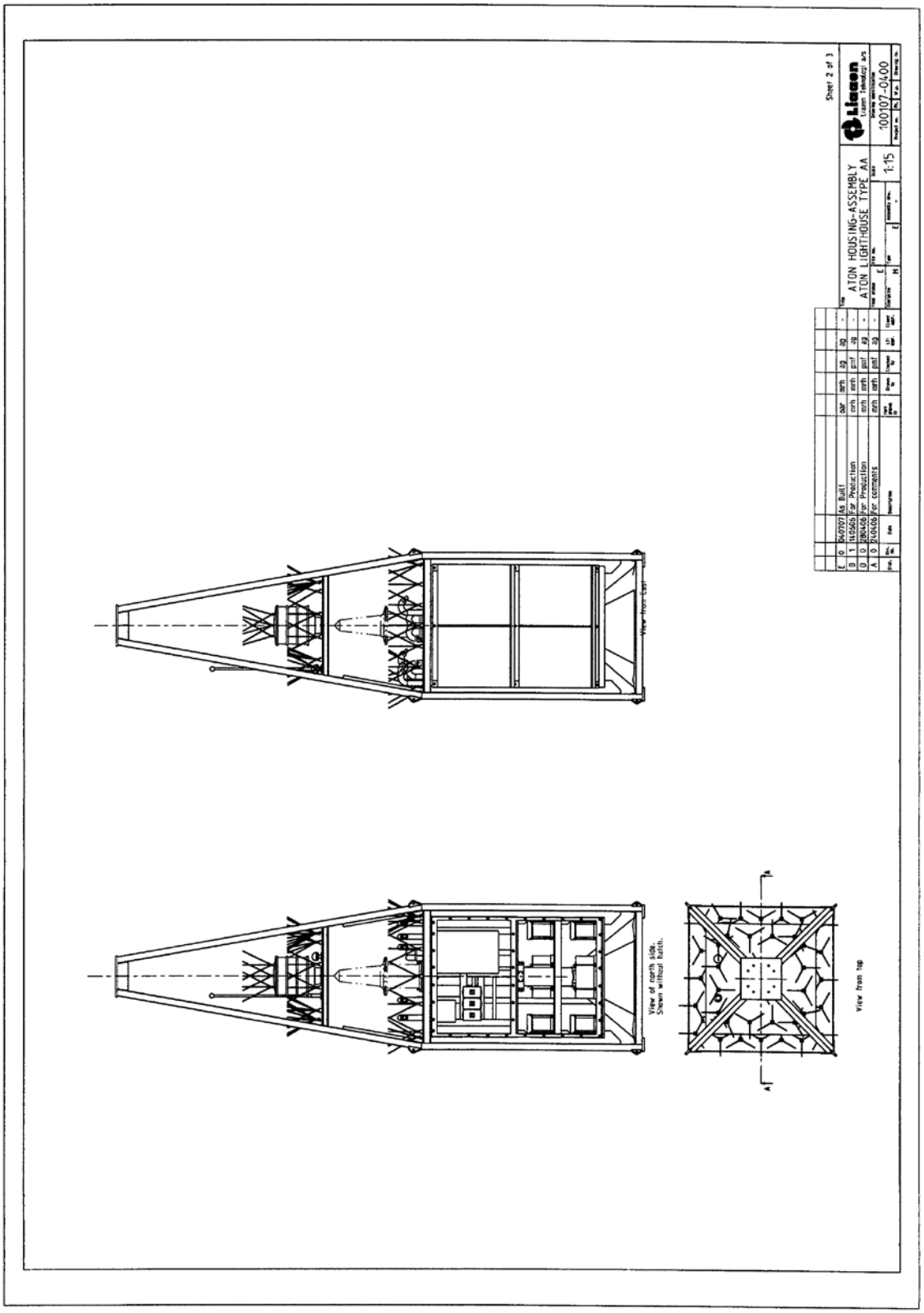
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View from top
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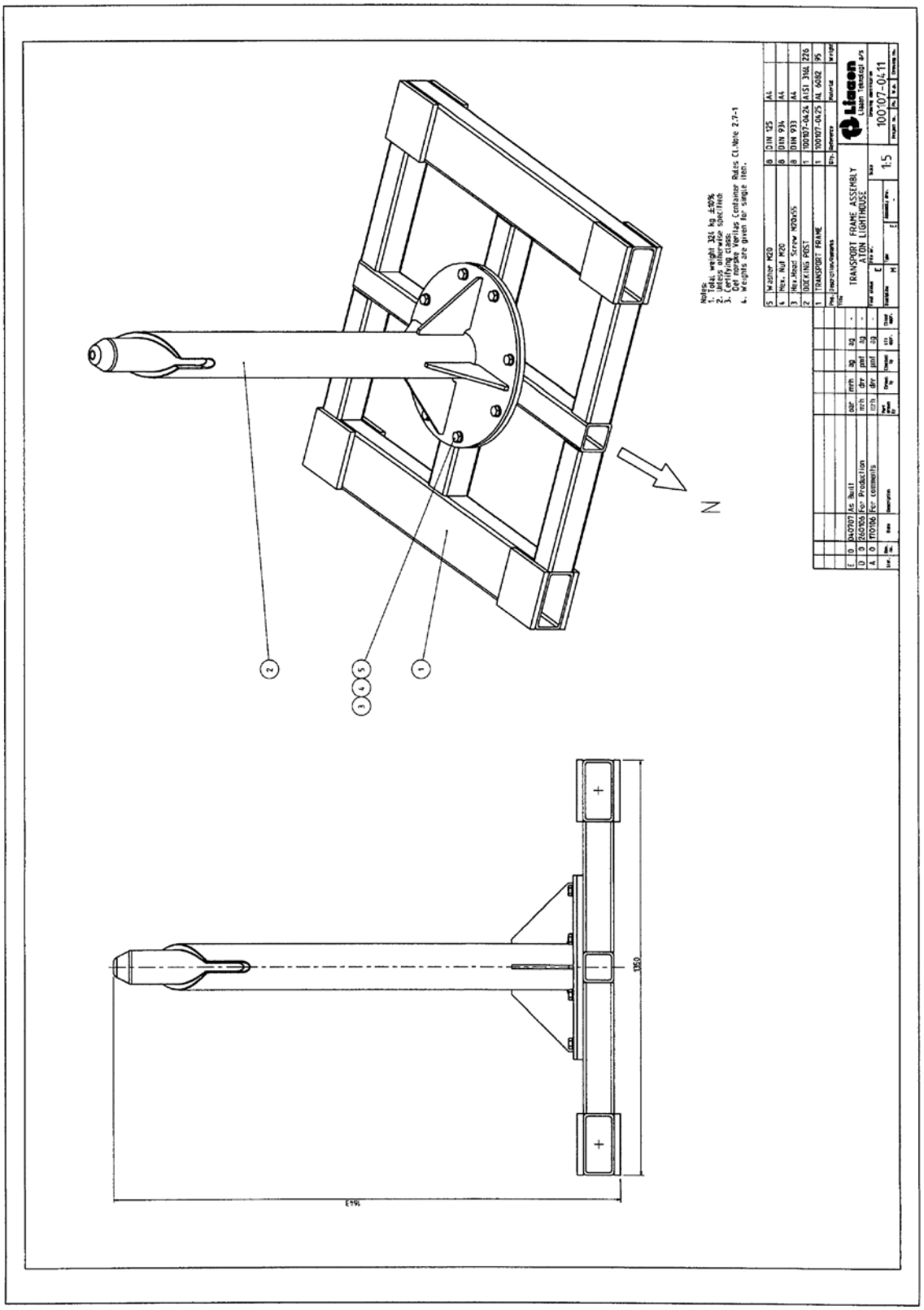
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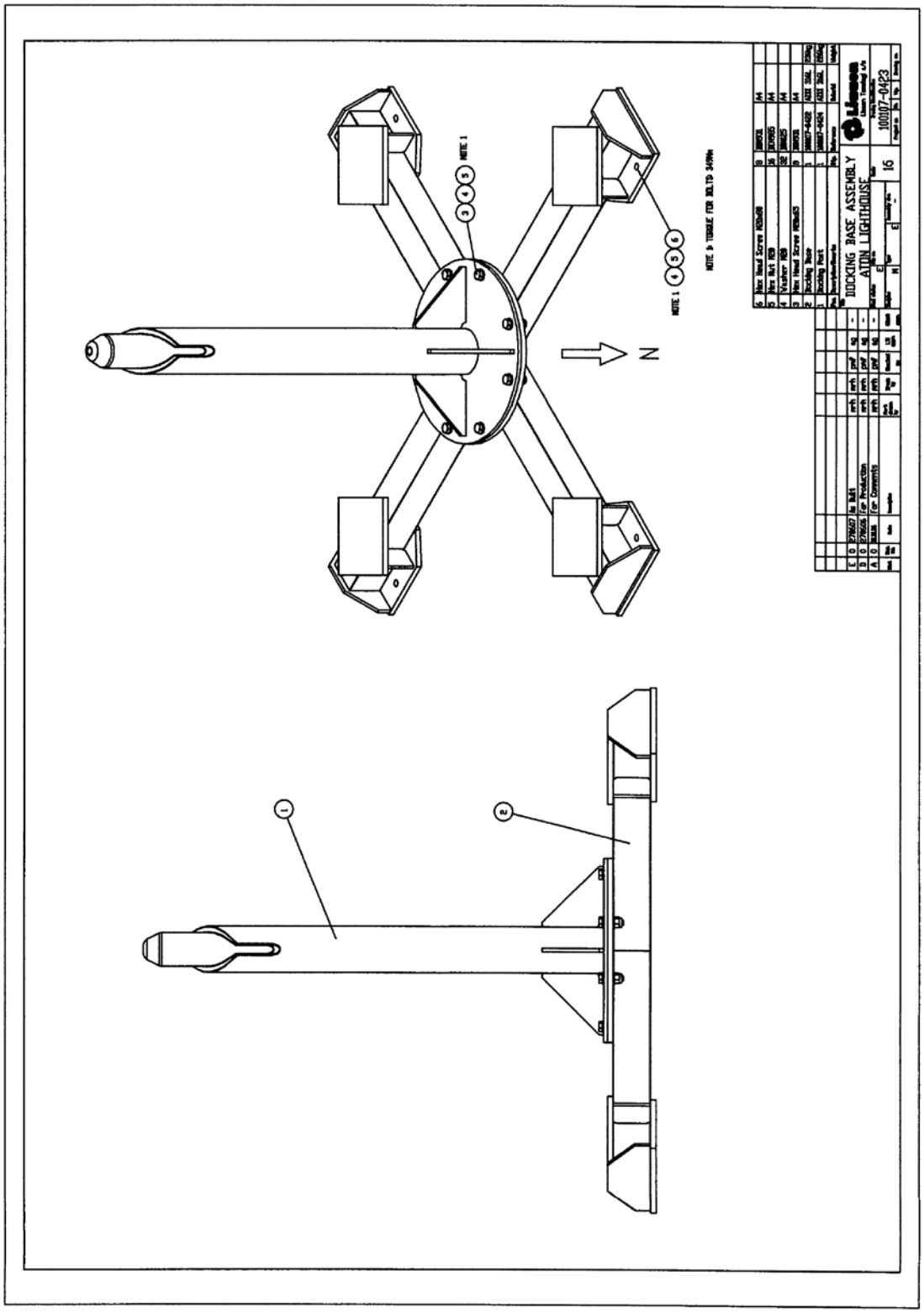
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Appendix D

Review of concrete degradation by Atkins

Dunlin Alpha Platform Concrete Degradation Technical Review

5073937-ER-04 Issue 5

October 2011

Plan Design Enable

ATKINS

Dunlin Alpha Platform

Concrete Degradation Technical Review

prepared for Fairfield Energy Limited

Abstract

The life expectancy of the Dunlin Alpha concrete gravity base (CGB) has been assessed for the circumstances of the platform being decommissioned and the CGB left in place with the steel topsides, steel columns and well conductors removed. The four legs of the CGB would terminate at 8m below sea level, and one of the legs would be modified to support navigation aids as required by the International Maritime Organization.

It is recognised that the concrete and steel materials in the CGB structure would degrade over time, and would be subjected to fatigue, ultimately leading to the failure and collapse of the CGB. Fairfield has commissioned Atkins to evaluate the failure mechanisms, and determine the life expectancy of the CGB.

The methodology predicts that an initial failure of the CGB leg carrying the navigation aids is most likely to occur around 20m below sea water level at the transition between the conical to cylindrical sections of the leg, where the leg cross section is at a minimum. This failure is predicted to occur around 250 years after the installation of the CGB and would result in the toppling of the upper leg in a direction that cannot be foreseen. This failure mechanism could result in a significant dropped object risk to the integrity of the cells below.

The legs not carrying the navigation aid would also fail at a similar location. However, due to removal of the steel column there will be smaller loading from the waves and therefore they would be expected to last longer. The additional duration has not been quantified in this study.

It is also possible that support to the navaid leg could be lost at its base due to failure of the CGB cell structure, causing a loss of support to the leg and collapse of the leg. The robustness of the base caisson structure and its probable degradation rates suggests persistent endurance for more than 1000 years but further study would be required to increase confidence in this statement.

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Document History

JOB NUMBER: 5073937			DOCUMENT REF: 5073937-ER-04 Issue 5			
0	Draft Report	TH				27/11/09
1	For Client Comment	AM	TH	PM	TH	30/11/09
2	Client Comments Added	TH			TH	24/12/09
3	Further Comments	TH	PM	SJ	TH	08/01/10
4	Client updates	AM	AM	PM		24/02/11
5	Further Client updates	AM	KR	AM	TH	31/10/11
Rev	Purpose Description	Originated	Checked	Reviewed	Authorised	Date

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1. Introduction

Fairfield Energy is operator of the Dunlin Cluster of fields in the North Sea, which includes the Dunlin Alpha platform (Figure 1-1). Dunlin Alpha consists of a concrete gravity base supporting a steel topsides.

Figure 1-1 Dunlin Alpha Platform

As part of its programme of work relating to the decommissioning of the platform, Fairfield has identified possible options to leave the CGB in situ, with topsides removed, which would require derogation under OSPAR Decision 98/3. One of the derogation options would be to remove the steel columns from the top of the concrete legs, which would give the remaining concrete structure 8m clearance below lowest astronomical tide (LAT). This would require at least one of the four legs to be marked with a navigation aid (navaid) to meet IMO Guidelines.

A separate report by Atkins [1] has suggested means of mounting a navaid on one of the four legs. The most probable means of achieving this is by means of a concrete mast mounted on either Leg A or Leg B, cantilevering up to at least the 10,000-year wave crest from the top of the concrete leg at -8m. All legs would be flooded.

It is recognised that the concrete and steel materials in the CGB structure would degrade over time, and would be subjected to fatigue, ultimately leading to the failure and collapse of the CGB. Fairfield has commissioned Atkins to evaluate the failure mechanisms, and determine the life expectancy of the CGB.

The objectives of this report are to:

- Assess the degradation and fatigue performance of the leg supporting the navaid
- Assess the degradation and fatigue performance of the legs without the navaid
- Review the likely degradation and fatigue performance of the base of the CGB
- Conclude on the likely life expectancy of the CGB

The likely failure mechanisms are:

- Degradation (spalling of the concrete, corrosion of the reinforcement bars (rebars) and prestress)
- Fatigue of the concrete, rebars and tendons
- Eventual damage to, or collapse of, the legs in severe weather.

These mechanisms are discussed in the following sections. A more detailed technical discussion of concrete degradation and failure mechanisms is presented in Appendices A and B of this report.

2. Glossary

This section briefly describes some of the specific terms used for concrete gravity base structures.

- **Cathodic Protection:** A method of slowing/preventing corrosion of steel by providing sacrificial anodes (zinc or aluminium) which have a higher galvanic potential than steel.
- **Concrete Grade (C45/C50):** Concrete strength, or Grade, is usually expressed (in design codes) in the form C#, where # denotes the "concrete cube strength" in MPa (Newtons per millimetre squared). The grade represents the minimum strength of the concrete after 28 days (from original construction date). Therefore, a concrete of grade C45 would be expected to have a cube strength of at least 45 MPa.
- **Depassivation:** Steel embedded in concrete is initially protected from corrosion by the alkaline chemistry of the concrete. When the cover layer to the concrete is cracked sufficiently to expose the steel, or chlorides have penetrated through, then this protection is lost and the steel is said to be depassivated.
- **Drawdown System:** A method of keeping the base concrete in compression by maintaining a lower water pressure inside the cells than outside.
- **Haunch:** In the context of Dunlin Alpha, the haunch is the section of the leg where the inner diameter starts to taper. Moving upward from the base of the legs (where they attach into the cells) there is a constant inner diameter (ID) and decreasing outer diameter (OD), therefore the wall section gets thinner until the ID starts to taper also. This is shown below.

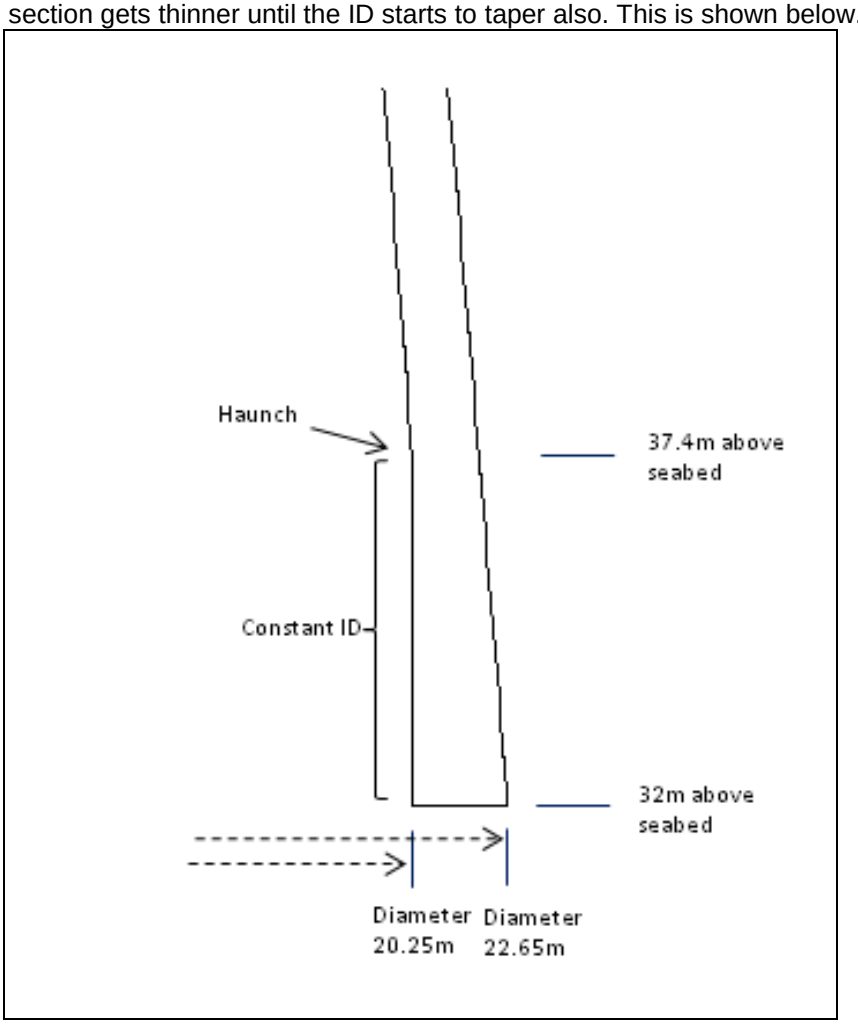

Figure 2-1 Schematic of leg

- **LAT (Lowest Astronomical Tide):** This is the minimum water level during a tidal cycle (not including weather related effects). This water level often used for reference purposes on nautical charts or offshore construction drawings.
- **Portal Frame Action:** This is basically a bracing effect of the deck onto the legs, or technically, the transfer of moment from one leg to the other via the deck structure. The stiffer the legs, the greater the moment that can be transferred. Due to the narrower top concrete sections and steel columns, the portal frame action is not large on Dunlin Alpha.
- **Prestress:** Concrete is much stronger when it is being squeezed (compression) than when it is being pulled apart (tension). Therefore, in most concrete structures steel bars and tendons (or wire strands) are embedded in the concrete to provide additional strength under tensile loads (see Rebars). In order to prevent the concrete ever having to resist being pulled apart, it is often pre-squeezed so it remains in compression at all times. This is done using prestress tendons. These tendons, typically running through the centre of the concrete section, are tightened up once the concrete has set, using powerful jacks, effectively compressing the concrete so that under normal loading the concrete has significant compression, and is therefore operating in its strongest state.
- **S-N Curves:** These are curves that give the relationship between the range of stress in a material (S) and the number of cycles (N) of that stress-range required to cause the material to break (failure). These failures are called fatigue failures, and are based on long term actions rather than single events.
- **Spalling:** The process of the concrete cover outside the rebars breaking off and exposing the steel. Usually caused by corrosion of the rebars.
- **SRB (Sulphate-reducing bacteria):** SRBs are bacteria that obtain their energy by oxidizing organic compounds or molecular hydrogen whilst reducing sulphate to create hydrogen sulphide. See Section A.5.
- **Steepness (of a wave):** The steepness of a wave is the wave height divided by the wave length. Typically, the steeper a wave, the larger the force it exerts when it hits a structure. Waves are limited in steepness; if they become too steep, they break. A steepness between 1:12 and 1:16 is a reasonable value for very large waves.
- **Rebars (Reinforcement bars):** As discussed under Prestress, concrete is much stronger when it is being squeezed (compression) than when it is being pulled apart (tension). Rebars are steel bars embedded into the concrete (see Figure 2-2) to carry the tensile loads instead of the concrete carrying this load. There are two kinds of rebars; vertical rebars running up the legs and hoop rebars running around the legs. Rebars normally come in pairs (inner and outer) and are on both internal and external faces of the concrete, usually about 50 – 80mm in from the outside (this distance is known as the 'cover'). A rebar is "gripped" by the concrete that it is embedded in and takes any tensile load arising such that cracking of the concrete is controlled and limited. Rebars control and reduce cracking but do not prevent it. Prestress tendons limit cracking when used in conjunction with rebars by adding compression.

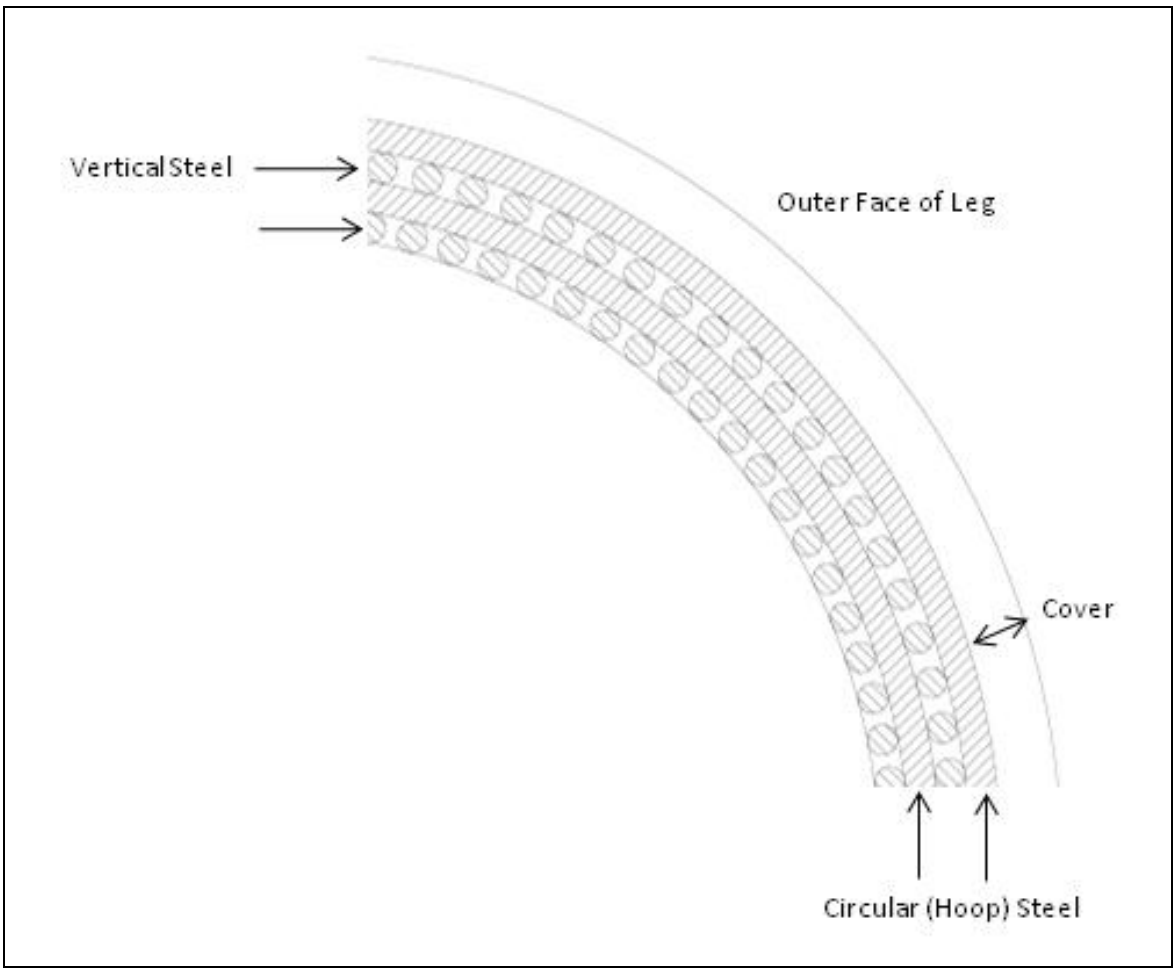

Figure 2-2 Diagram of Rebars

3. Concrete Strength and Degradation Mechanisms

3.1 Concrete Strength

The long term strength of concrete in the legs has been taken as 55 MPa, in accordance with Appendix A. This value allows for:

- Uncertainty in the original concrete strength, variably quoted as C45 and C50
- Increase in strength due to ageing
- Possible reduction in strength based on tests on pressurised concrete

It is recommended that the strength of concrete used in the design of a future navaid support structure should be optimised for durability of this structure. It is considered possible to design a navaid support tower that will last at least as long as the original CGB legs, so that the navaid support is not the critical component of the CGB, despite being located partly in the splash zone.

Research carried out in the COIN [12] programme in the 1970s showed that the effect of acidity produced from the action of sulphate reducing bacteria (SRB) present in the residual hydrocarbon-rich ballast water in the base of the CGB, would result in a loss of only 10% of concrete strength, and a loss of thickness of only 1mm in the concrete cover zone. Thus it is likely that the endurance of containment of the CGB base, even when subjected to both external and internal corrosion processes, would be not less than 1000 years.

3.2 Degradation Mechanisms

The most vulnerable sections of offshore concrete structures in terms of long-term durability are the leg areas around the splash zone, as discussed in Appendix A. However, loss of concrete and steel section can occur at any depth, as a result of the following known mechanisms:

- Carbonation and chloride attack into the concrete, resulting in depassivation of the reinforcement when these mechanisms permeate the concrete to reach the layers of steel rebars
- Spalling of the concrete due to volumetric expansion caused by the generation of corrosion products around the steel
- Loss of rebar cross-section caused by the same corrosion mechanisms
- Accumulated fatigue damage to the concrete and reinforcement, this process accelerating as stresses increase due to the above degradation mechanisms, and due to pitting of the steel caused by corrosion
- Similar corrosion and fatigue mechanisms on the prestress tendons and their anchorages, resulting in reductions in prestress and loss of section capacity

The design of the navaid support structure should reflect and compensate for the above mechanisms.. The life of this new design could be maximised in several ways, including increased cover, higher quality concrete, reinforcement coating systems, and the provision / retention of a cathodic protection system.

A further degradation mechanism that should be considered is the long-term effect of SRBs in the CGB base cells and in the drill cuttings on the cells roof. It is difficult to predict these effects because of lack of data. Very limited test data indicates that for the likely conditions in concrete storage tanks, the depth of attack is likely to be less than 70mm over a 100 year period. This may therefore have an effect in the very long term, and would require further research.

3.3 Degradation Rates

The above data on degradation mechanisms has been compiled into a series of tables showing the likely time intervals from platform installation for different levels of concrete and reinforcement loss to occur. In compiling these tables, the following have been considered:

- The time taken for carbonation and chloride attack to penetrate down to the reinforcement
- The time for rust formation to cause cracking and spalling of the concrete cover
- The rate of corrosion of the reinforcement and subsequent loss of steel section

Table 3-1 andTable 3-2 show the expected times (in years) for the above processes to occur for the exterior and interior faces of the legs in submerged regions. The tables are colour graded to give a visual representation of the times involved, with red being relatively quickly and green the longest time period. The splash zone has been discounted, as only the navaid tower would penetrate the water surface. Its design is unknown at this stage, but should be such that this component would not be not critical.

Sequence		Exterior Face - Cover 50 mm	Outer Circ. Bars	Outer Vert. Bars
Intact	Phase 0	Carbonation & Chloride permeation removes passivisation to outside of rebar.	70	100
	Phase 1	Carbonation & Chloride permeation removes passivisation of full rebar perimeter	100	130
Damaged	Phase 2	Cracking of cover zone by corrosion volume	130	160
	Phase 3	Spalling of cover to expose rebar	160	190
	Phase 4	Rebar diameter reduced by 10%	240	270
	Phase 5	Rebar diameter reduced by 50%	560	590
	Phase 6	Rebar diameter reduced by 100%	960	990

Table 3-1 Disintegration Mechanism Timescale Estimate (years) – Submerged Leg Exterior

Sequence		Interior Face - Cover 50 mm	Inner Circ. Bars	Inner Vert. Bars
Intact	Phase 0	Carbonation & Chloride permeation removes passivisation to outside of rebar.	70	100
	Phase 1	Carbonation & Chloride permeation removes passivisation of full rebar perimeter	100	130
Damaged	Phase 2	Cracking of cover zone by corrosion volume	130	160
	Phase 3	Spalling of cover to expose rebar	180	210
	Phase 4	Rebar diameter reduced by 10%	260	290
	Phase 5	Rebar diameter reduced by 50%	580	610
	Phase 6	Rebar diameter reduced by 100%	980	1010

Table 3-2 Disintegration Mechanism Timescale Estimate (years) – Submerged Leg Interior

The information in Table 3-1 and Table 3-2 has been used to analyse specific locations along a leg. This considers the net loss of concrete and steel due to degradation. An extract from this list, for a location at 37.4m above the seabed, which is just above the base of the leg haunch, is given in **Table 3-3**. It does not, at this stage, include the effects of fatigue.

Time (years)	Degradation Mechanism	% concrete lost	% rebar lost
160	Spalling of exterior concrete to hoop steel	5.6%	0.0%
180	Spalling of interior concrete to hoop steel	55.1%	0.0%
270	10% loss of vertical bars on outer face	55.1%	5.0%
290	10% loss of vertical bars on inner face	55.1%	10.0%
590	50% loss of vertical bars on outer face	55.1%	30.0%
610	50% loss of vertical bars on inner face	55.1%	50.0%
990	100% loss of vertical bars on outer face	55.1%	75.0%
1010	100% loss of vertical bars on inner face	55.1%	100.0%

Table 3-3: Chronological percentage loss of section, El +37.4m, vertical steel

The degradation would be marginally faster at the outside face as this is exposed to constantly changing seawater. Corrosion is expected to be slower on the interior face due to reduced oxygen levels. The percentage of concrete lost conservatively assumes complete loss of all concrete cover due to spalling. The above list has been used as the basis of degradation models of the concrete and steel, to be combined with fatigue section losses, using the methodology presented in Section 4.

3.4 Prestress Losses

Long term loss of prestress is also predicted. The time for depassivation, corrosion and resultant stress corrosion cracking of the prestress tendon steel at a leg section would be longer than that for reinforcement loss, due to the tendons being set deep inside the concrete, and protected by ducts and grout.

Somewhat faster degradation mechanisms would be expected at the tendon anchorages, similar to those for reinforcement. Since the anchorages are located some way from critical sections, this would only be a concern if the tendons are ungrouted. It has been suggested by the UK's Health and Safety Executive (see Appendix A) that potentially 5-10% of prestress tendons in a structure built in this era may be ungrouted, or partially grouted.

In addition to the above, the process of prestress relaxation would continue. Although most of the design prestress relaxation would have occurred by Cessation of Production (CoP) in the Dunlin field, some further losses may be expected in the centuries that follow, perhaps of the order of 4-5%.

At critical leg sections, such as the haunch, where bending moments are high but the wall thickness is thinner than further down the leg, design prestress values have been used for the period up to CoP for calculation purposes. After CoP a further 15% loss of prestress has been applied to the time until depassivation would reach the tendons, after which the loss of prestress would be relatively rapid.

4. Combined Fatigue / Degradation Life Prediction

4.1 Introduction

In order to predict the life of any component of the CGB, it is necessary to consider how the strength of the structure would change with time. As described in Section 3, there would be anticipated changes to the strength of the concrete, prestressing forces, spalling of the concrete and corrosion processes that must be considered.

Fatigue processes would cause additional loss of section in both the steel and the concrete, and long term reduction in prestress. As noted above, fatigue would accelerate as the effective section and prestress reduce, resulting in higher stresses for any particular wave action. The rate of fatigue degradation would also be expected to increase once the corrosion process began on the reinforcement, due to local pitting.

Additionally, changes in the configuration of the structure would alter its ability to resist wave loads, such as removal of the topsides and of the well conductors. Following the end of normal operations, the topsides would be removed and the drawdown system decommissioned. This would result in an effective reduction in the strength of the structure due to loss of compression in the concrete. Removal of the portal frame action between legs, afforded by the deck (this is the effect that the deck has in stiffening the legs by connecting them) would be expected to be a minor effect at Dunlin Alpha due to the highly tapered legs. Removal of the conductors would compensate for this to some degree, by reducing the load on the CGB, particularly on Legs C and D.

Ultimate failure of a leg section or the base supporting the leg would be expected to occur as a result of the above processes weakening the structure, followed by a severe environmental event causing overload. There is also a risk, difficult to predict in the very long term, of impact on surface piercing legs due to errant vessels. Such an accident could significantly reduce the endurance time of the legs by causing damage to the legs. However, it is thought this threat remains small and assumed every effort would be made to reduce it.

The wave loading would be most critical on the leg that supports or restrains the navaid structure. It is likely that this structure would be located either on Leg A or Leg B [1], since these do not have the complexity and additional loading caused by the conductor guide support rings. Despite Legs C and D carrying substantial additional conductor loads throughout the operational life of the structure, there is no appreciable difference in the design of the four legs. Legs C and D may therefore have been subjected to more cracking and greater fatigue during the producing life of the platform, with the result that Legs A and B are likely to have degraded less by the time of CoP.

4.2 Methodology

Atkins has developed a methodology that combines all of the above mechanisms in a spreadsheet-based analysis of critical locations in the structure. A flow chart of the analysis process, incorporating configuration changes, prestress loss, fatigue, degradation and load-based strength models, is shown diagrammatically in Figure 4-1. Each step is discussed further in Appendix B.

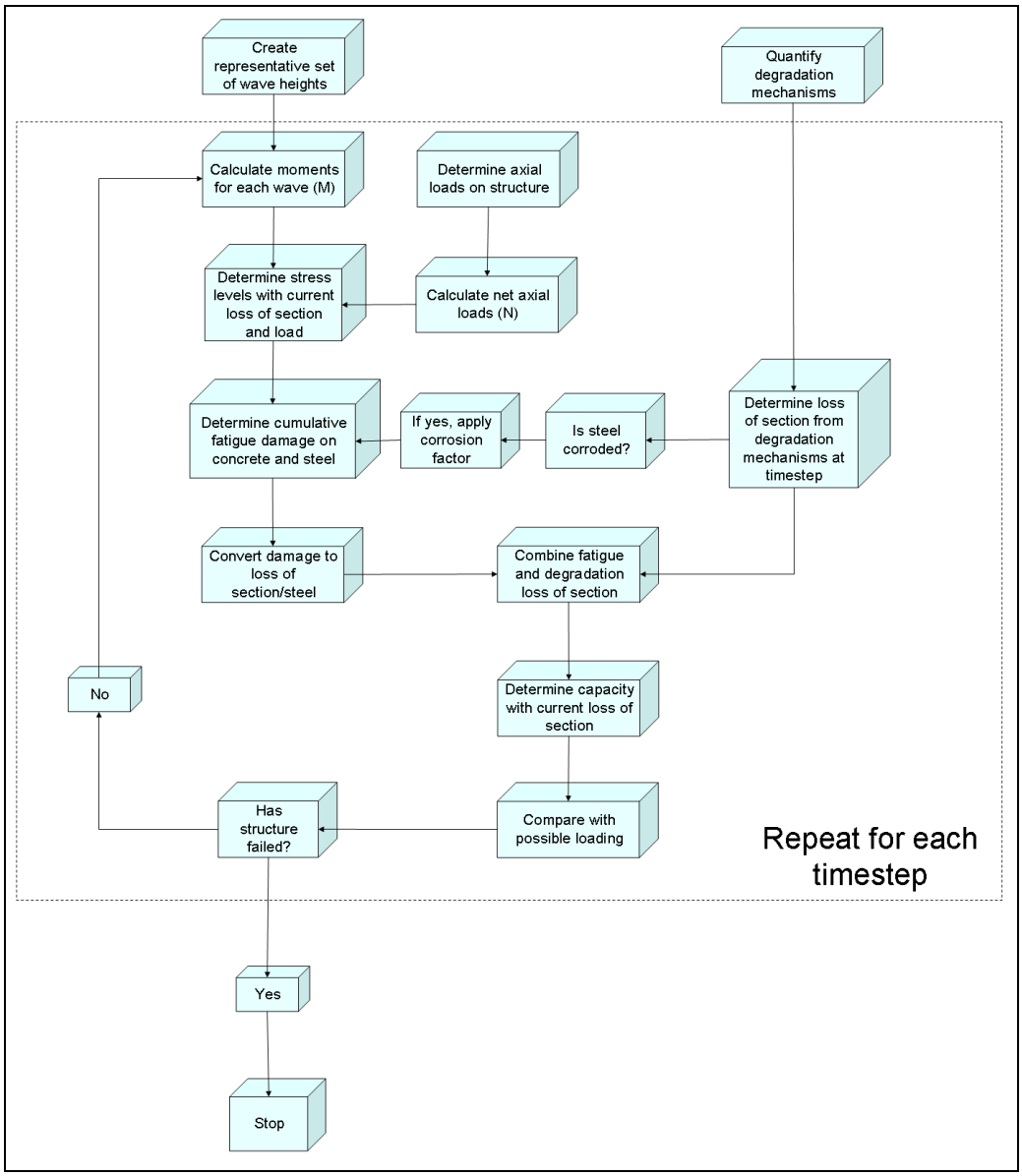

Figure 4-1 Flow Chart of Analysis process

Starting with the condition of the platform as first installed, a series of time-steps were taken with the above process repeated at each stage. The strength of the leg may decrease at each stage, due to degradation or fatigue, and eventually it may drop below the calculated strength required to resist the largest wave expected in 100 years. This is considered the end point. Alternatively, the process is continued to approximately 2000 years, which is considered significantly long to be beyond the scope of the method and any concerns regarding collapse.

Reference to the S-N curves for concrete [5], shows that the fatigue life will be very sensitive to the strength of the concrete. A representative long term strength of C55 was used throughout (see Section 3.1).

As stated above, degradation and fatigue mechanisms are related as increasing degradation causes higher stresses in the remaining section and therefore accelerates the fatigue process. Conversely, fatigue damage resulting from additional concrete cracking that will marginally speed the depassivation process, would have only a minor effect on degradation mechanisms, the methodology used is not sensitive to this minor effect, and it has therefore been ignored.

A concrete material safety factor of 1.2 is included in all fatigue and capacity calculations in accordance with accepted guidance [5] for fatigue and accidental limit states (FLS and ALS). This is intended to allow for the difference between in situ and test strengths, and fabrication tolerances. The latter are likely to have little effect as the leg is of large diameter, but some allowance is needed for varying support stiffness from the CGB base structure beneath. For this reason, the full safety factors of 1.2 have been included.

4.3 Locations Checked

This methodology was applied to two locations on the legs of the CGB:

1. At the base of the legs, at elevation +37.4m above seabed, just above the thicker section of the leg at the roof
2. At the transition between the cylindrical and conical parts of the legs, at +128m elevation, where wave loading is at its greatest on the smallest section size.

Location 1, the base of the legs was selected as the leg moments are highest just above the roof [7]. Location 2, the higher leg location, was chosen because this is the most heavily loaded location on the narrow concrete section below the steel columns. These are both show graphically in Figure 5-3

Legs A and B were selected as the most critical legs as one of these would have the navaid structure installed. As the navaid tower would rise up through sea level, it would attract greater wave loading.

Consideration was also given to a basic cylindrical navaid tower in the splash zone, and to the CGB base structure. However, the main focus of this report is to investigate the most likely initial collapse mechanisms. The robustness of the base caisson structure and its probable degradation rates suggests persistent endurance for more than 1000 years.

5. Collapse Mechanism and Life Expectancy

5.1 Splash Zone

The installed navaid would penetrate the water surface. The most severe degradation conditions would be expected to occur in the splash zone, between 3m below and 6m above LAT. Breakdown of the navaid supporting structure would occur faster on the outer face due to contact with moving, aerated water, causing faster steel corrosion.

The loads on this part of the structure could be significant. Wave action would cause pressure fluctuations in hoop stress, which combined with local impact loading and changes in bending moment would produce fatigue damage. Once the degradation process had begun, there would be the potential for water to penetrate into cracks and through holes. Freeze-thaw cycles could also occur in this region, further affecting the integrity and increasing cracking.

However, the navaid support structure itself has not been considered in this report as this would be a new structure and could be specifically designed for longevity. As previously noted, it is thought that

when this structure is designed it could be designed to be less critical to failure compared with the original leg structure below.

5.2 Base of Legs

The integrity of the base of Leg A / B has been assessed using the method described in Section 4. The critical part of the leg was assumed to be just above the thicker haunched region at 37.4m above the seabed, as this has large moments but is thinner than the concrete below.

Results are shown in Figure 5-1, in the form of the remaining moment capacity of the leg, versus the time since the CGB was installed. Also shown is the moment caused by the worst expected wave in 100 years, to give an indication of the required capacity. The higher capacity over the early years is with the topsides deck intact, full design prestress and axial compression due to pressure differentials across the leg wall (the leg has been assumed to be unflooded up to the time of CoP).

The graph shows that the expected life of the leg at this depth would be very long, approaching 1500 years. Over this period, it is likely that the 100-year wave would be exceeded on many occasions, perhaps significantly. A probabilistic approach to this is suggested, as discussed in Appendix B. As a result, it would therefore be prudent to consider a somewhat lower life expectancy, perhaps 1250 years.

For the 55 MPa concrete considered, the primary failure mechanism would be degradation, with loss of section due to spalling and corrosion only combining with fatigue damage towards the end of life. After 1500 years all of the reinforcing bars and most of the prestressing tendons would have been lost, to corrosion and fatigue respectively. This would result in a relatively slow reduction in the leg section capacity. At this point the structure would become very sensitive to loss of prestress with a complete loss of prestress causing the moment capacity to drop below the 100-year wave moment.

Local wall buckling, where a small discrete section of the wall around the circumference is weakened sufficiently to have little or no capacity and which would accelerate the collapse process, has not been considered. This would require a detailed assessment, that was not deemed appropriate for this level of study, and the loss of capacity would be dependent on the size and location of the buckling.

The relative longevity of the leg at this level of the structure is attributed to the low wave induced moments. The applied wave loading produces a moment which is one third of the magnitude of the initial leg capacity. This would result in minimal concrete fatigue and would allow a significant loss of material section to occur before leg failure would be likely.

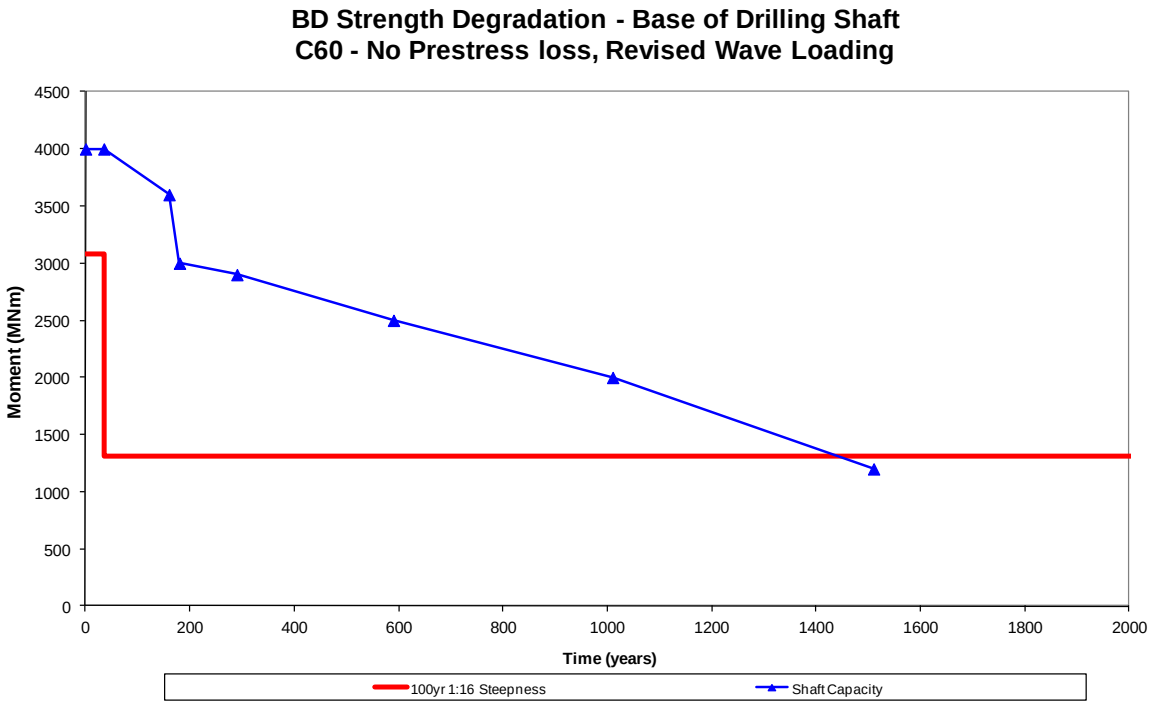

Figure 5-1 Base of leg capacity versus time – Grade C55 concrete

5.3 Leg Conical to Cylindrical Transition

The integrity of the lower end of the cylindrical region of Leg A / B was also assessed using the degradation / fatigue method. Results are shown in Figure 5-2, in the form of remaining section capacity versus time, compared with the 100-year wave induced moment.

The life expectancy at this leg level would be less than at the base of the leg, with the 100-year wave load exceeding the capacity in about 325 years. Once again, because of the probability of this 100-year loading being exceeded over this period, it is recommended that this reported life expectancy be reduced, to perhaps 250 years.

Compared with the base of the leg, fatigue is relatively significant here. Removing the conductor array does not significantly reduce the wave-induced bending moments at this location and the introduction of the navaid tower would result in significant wave moments being experienced at high wave heights. More importantly, the removal of the deck, while reducing wind loads, does reduce the capacity by decreasing the beneficial axial load. This axial load comes from the deck weight and portal frame action, which would significantly reduce bending at this section in the operating condition.

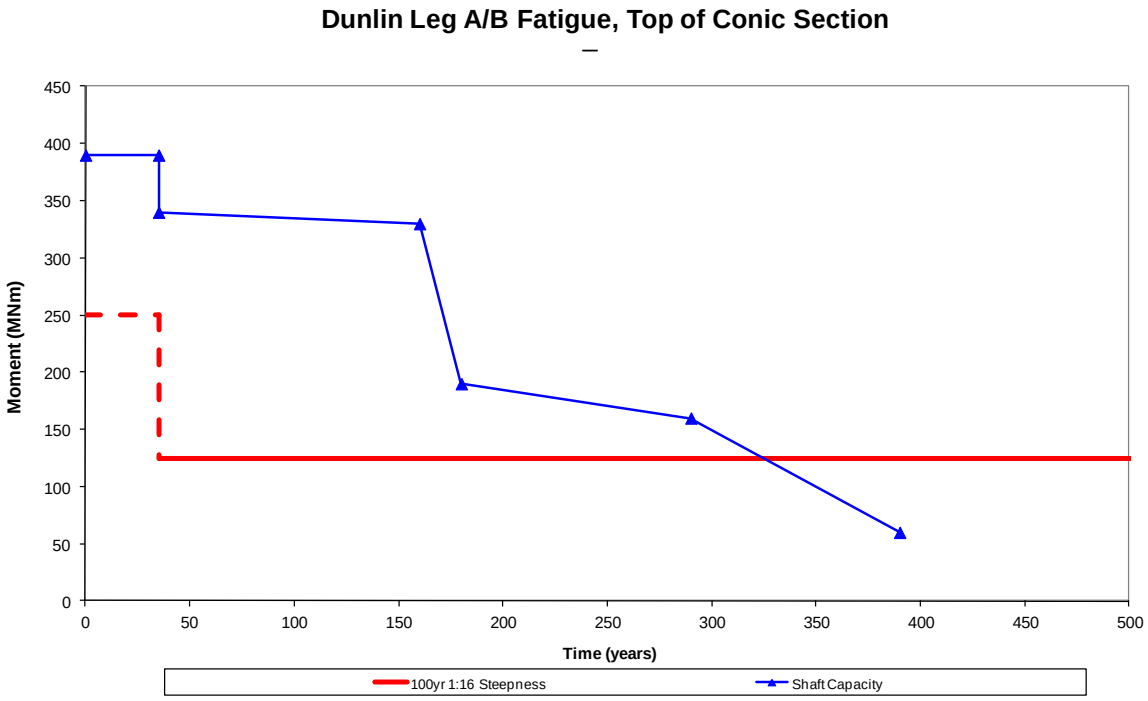

Figure 5-2 Lower end of cylindrical leg capacity versus time – Grade C55 concrete

5.4 Base Structure

The concrete base structure below the legs is subjected to a different loading regime than the legs, in that the wave and current drag forces on the base of the CGB are much reduced and tend to produce bending in the roof slab, rather than the fluctuating axial load found in the legs. Buckling collapse of one of the flat internal walls is also possible.

Following decommissioning, the highest stresses in the base structure would occur where the legs join the base. At these locations, the roof, cathedral, leg and inner walls of the base all intersect, resulting in complex local stress patterns. Away from the leg junctions, the applied forces become small as the structure was essentially designed to contain the pressure differentials during installation and in service. In a decommissioned state, there would be no pressure differentials and hence no significant forces to accelerate the degradation of the structure.

However, this loss of pressure differential and reduction in beneficial compression load from the drawdown system would significantly reduce the strength of the base structure. As noted previously, concrete performs much better in compression, and the drawdown system provides approximately 10 MPa of compression in the base structure. Additional prestress and hydrostatic pressures provide between 1.0-2.0 MPa and 1.2-1.5 MPa of compression each respectively. Removing the drawdown therefore significantly reduces the beneficial compression. However, in the decommissioned state the stresses caused by thermal loading (i.e. oil storage or production) will be removed, and loading from wave action is also significantly reduced. Therefore, although the beneficial compression from drawdown has been removed, the loading has also reduced.

A further risk would be that of dropped object damage from falling objects or from vessels. The dropped object impact risk would be present during partial deconstruction operations (e.g. deck removal) and afterwards as the upper residual structure would begin to decay and disintegrate. The possible impact from trawling would remain, but in real terms would present a low risk of damage to the CGB structural walls because of their very great strength compared to the maximum energy from a trawl board impact.

The effect of SRBs, arising from residual hydrocarbons and cuttings, on the inside and outside of the CGB cells has been discussed in Section 3.2. This is another mechanism that could reduce strength in this region.

No analysis of the cell structure has been possible at this time, but a qualitative judgement has been made on the basis of the above. It is considered very likely that loss of support for the legs would occur in a shorter timescale than base of leg failure, probably as a result of degradation / fatigue failure of the roof structure adjacent to the legs. Collapse of the internal walls under the legs would also be possible, particularly towards the centre of the structure where these are thinner and susceptible to loss of cover due to spalling.

The timescale for these mechanisms is uncertain, but it is considered likely to be similar to the timescales shown in Section 3 associated with degradation of the upper part of the legs, at the conical to cylindrical transition. The same mechanisms apply, namely, concrete and steel fatigue superimposed on general degradation in a submerged environment. Although the induced loads in the base would be relatively high, there would be be little oxygen in the cells and in the drilling cuttings, with the result that corrosion processes would be slow.

5.5 Conclusions

As noted previously, two key locations were considered:

1. At the base of the legs, at elevation +37.4m above seabed, just above the thicker section of the leg at the roof
2. At the transition between the cylindrical and conical parts of the legs, at +128m elevation, where wave loading is at its greatest on the smallest section size.

These are both shown in Figure 5-3.

On the basis of the above, it is considered likely that the leg supporting the navaid would be the first to fail, due to the increased wave load caused by the navaid tower (see Figure 5-3**)**. This relative weakness could be minimised by using Leg A or B for the navaid (these legs do not have the ring beams for the conductor guide frames, that increase the loads on Legs C and D), and by making the navaid supporting structure as transparent to waves as possible.

Assuming that the navaid support is adequately designed, first failure would be expected to be at the conical to cylindrical leg transition (Location 1), where the moments in the smallest diameter cylindrical section are highest. Collapse of this upper leg could be as soon as 250 years after initial CGB installation, and it could result in a significant dropped object risk should the upper leg and navaid support (or parts thereof) fall on the cell structure below.

Collapse of the other legs not supporting the navaid would very probably take longer to occur, as these legs would not penetrate the water surface. However, it is conceivable that one or more of these legs could be hit by the toppling navaid leg or other unseen force prior to these collapse times.

The majority of wave loading occurs in the wave crest, and by removing the section of structure cutting the water surface, the loading is reduced and predicted time to failure significantly increased both at the 128m and 37.4m locations. This increase has not been calculated at this stage. Similarly, a failure at 128m on the leg with the navaid will reduce wave loading and extend the 1250 year prediction. It should be noted though that if failure occurs at the 128m level, an additional method of marking the structure must be instigated.

It is also possible that support to the navaid leg could be lost at its base due to failure in the CGB cells structure. The consequences would be loss of support to the leg, and collapse and damage to the

surrounding base structure as the leg toppled. The robustness of the base caisson structure and its probable degradation rates suggests persistent endurance for more than 1000 years but further study would be required to increase confidence in this statement.

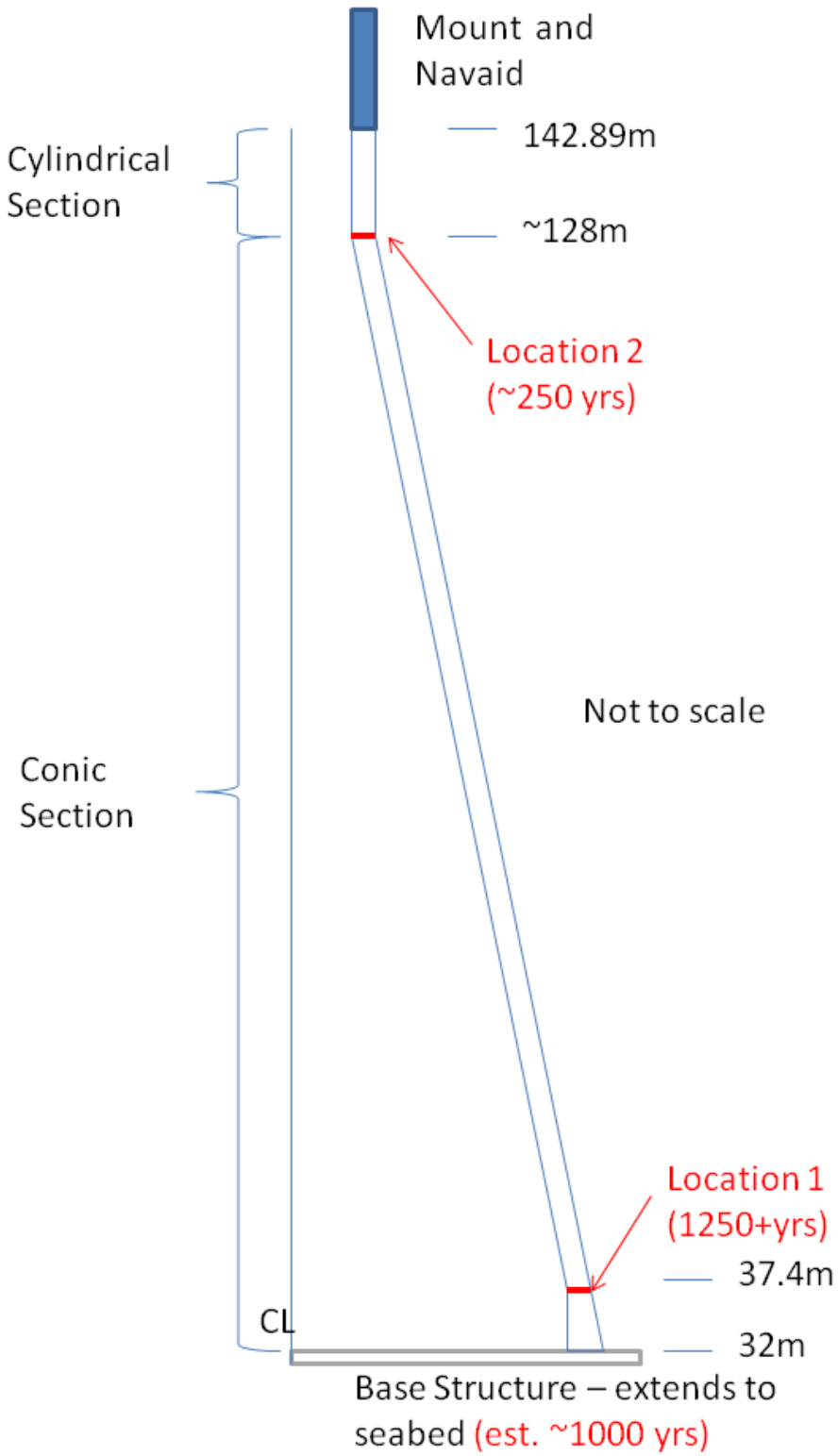

Figure 5-3 Concrete leg locations and predicted time to failure

6. References

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Appendix A Strength and Degradation Mechanisms

A.1 Degradation Mechanisms

A detailed description of the processes associated with deterioration, breakdown and disintegration of concrete and reinforcement is contained in a UKCS concrete substructure JIP [2], and is summarised in the following sections.

Based on these degradation mechanisms, the probable failure of components the Dunlin Alpha legs is predicted, and the sequence of failure of the substructure is therefore qualitatively evaluated.

A.2 Concrete Strength and Ageing

The nominal cube strength of the concrete used for Dunlin Alpha is alternately referenced as C45 and C50 in the Aker report [7]. However, the concrete has aged and will continue ageing once decommissioned and left in place. Typical offshore concrete cores indicate 25% increase in strength due to this ageing, perhaps even more.

The MC90 [6] code predicts a strength increase from 50 MPa to 63 MPa over 30 years. Most of this ageing benefit occurs in the first 30 years, and the strength does not increase significantly beyond this stage, as shown in Figure A-1. However, MC90 also includes for a reduction in concrete strength where the concrete is in sustained compression. This can compensate for the ageing effect.

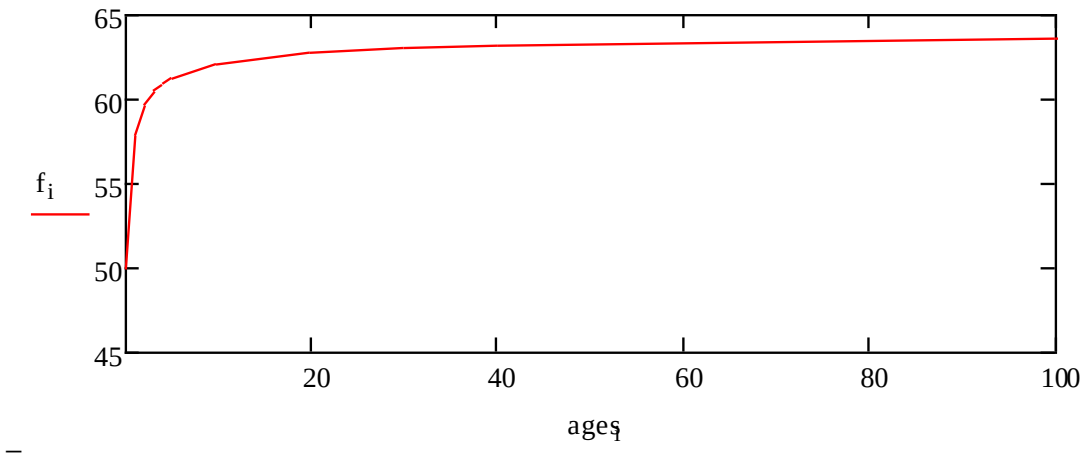


Figure A-1 Initial aging of C50 concrete, calculated from MC90 code

Evidence of a relatively small ‘long-term’ gain in compressive strength is supported by cores taken from the Brent platforms. These suggest that the aged concrete strength is an average of 60 MPa, but the initial strength was greater than planned, with a mean value of 57 MPa (from cube tests). It is also

known from tests [2] that concrete immersed in seawater at significant pressure can demonstrate a reduction in strength, as described in the following section.

A.3 Chemical and Pressure Effects on Concrete

The possibility of chemical attack of concrete by seawater has been well researched and the results show that there is good evidence for minimal long term degradation from chemical attack, particularly for the high strength and quality concretes used offshore. Indeed the results indicate that protective layers of brucite and aragonite form which enhance the protection of the concrete.

However, results from several programmes show that there is evidence for long term loss in compressive strength (up to 20%) after exposure periods of 8 years in deep seawater (~100m), compared to samples stored in shallow water. Although the mechanism is not well understood, evidence from several studies was found for micro-cracking around coarse aggregate particles. There is no data for the effects of longer term exposure, beyond the 8 year test periods, making it very difficult to predict the loss after much longer periods. The above cracking may even have occurred during retrieval of the sample from depth, and the concrete may actually be stronger in-situ.

The possibility of strength reduction must therefore be considered, which will negate the increase in strength due to ageing. In this study, a concrete strength of 55 MPa has been considered, somewhat lower that the calculated aged strength for the minimum C45 concrete and considerably less than the aged strength for C50 concrete.

A.4 Corrosion of Reinforcement

Corrosion mechanisms and rates depend on level with respect to seawater, as illustrated in Figure A-2 below:

- Corrosion Zone 1 is the atmospheric zone, and the upper part of the splash zone. This zone is similar in many respects to land based concrete structures, and high quality cover will limit any corrosion.
- Corrosion Zone 2 is the upper middle 'splash zone'. Corrosion is more likely to occur Zone 1, but again should be limited, particularly where there is good quality cover present. Conditions favour the formation of hydrated ferric oxides which are highly expansive and lead to cracking and spalling, making any corrosion visible at early stages.
- Corrosion Zone 3 comprises the middle to lower part of the 'splash zone', which means that it is wetted on many occasions, resulting in lower resistivities. As a result there is the opportunity for more extensive macro-cells to form, and because the zone is exposed to air for long periods there is a good availability of oxygen. If passivity of the reinforcing steel should break down, moderate to severe corrosion can occur. Any CP system is partially effective, and this zone can provide efficient cathodes to active areas in Zone 4.

- Corrosion Zone 4 is primarily the underwater zone, and extreme lower part of the splash zone. The concrete in this zone is permanently wet and is therefore of low resistance, but the availability of oxygen is generally limited. Corrosion rates are likely to be less than 0.01mm/year. However corrosion can occur at suitable sites (cracked areas) provided there is a galvanic link to an anode with access to oxygen, probably in Zone 3. A CP system should be effective in this zone, and is recognised as the prime control system to manage localised corrosion in this region.

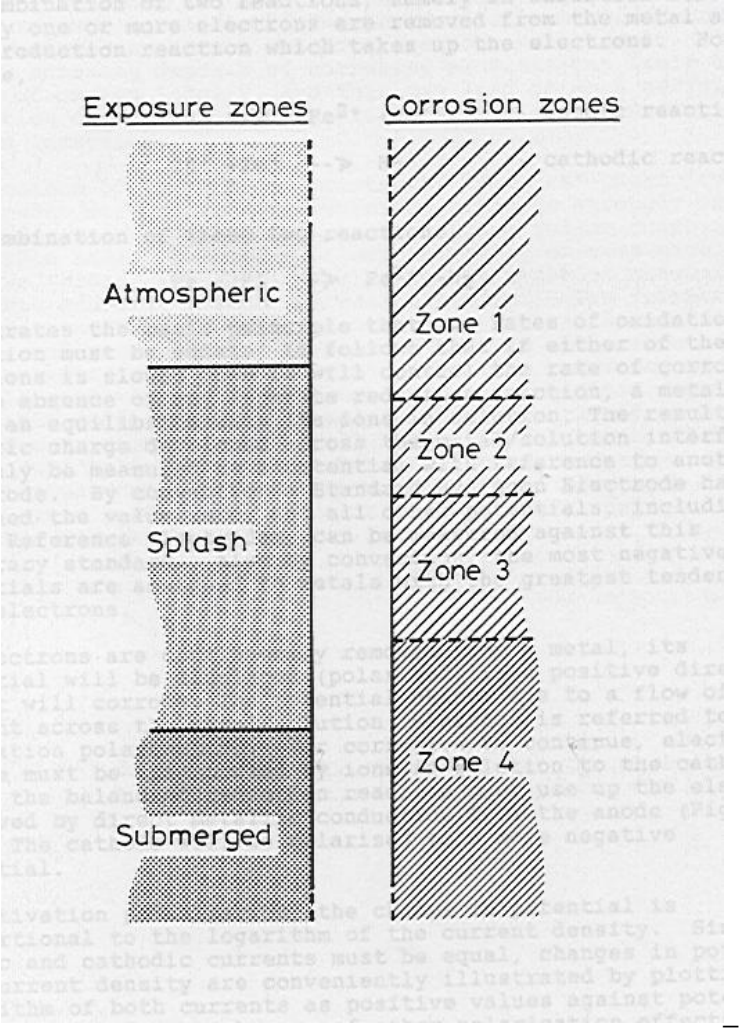


Figure A-2 Corrosion Zones in Marine Concrete Structures

A.4.1 Corrosion in Splash & Tidal Zones

It is widely recognised that reinforcement in Zones 2 & 3 is most vulnerable from corrosion due to the plentiful supply of chlorides from the seawater and also oxygen to fuel the cathode. The Dunlin Alpha CGB does not feature concrete in the splash zone (the legs only extend up to LAT-8m). However, the

navaid support columns will extend up through the splash zone, and degradation in this region will need to be considered during the navaid support design.

These zones are also the ones most vulnerable to ship collision damage which could lead to cracking and loss of cover, and the potential for corrosion to occur at damaged sites. A carbonation and chloride attack rate of about 1mm per year is not unlikely. If the cover depth in Zones 2 & 3 in Dunlin Alpha navaid support structure is set at 70mm, by implication the protection against general rebar corrosion should endure for 70 years after installation.

A cathodic protection system normally provided to protect attached steelwork also protects the steel reinforcement to some extent, and tests indicate that it would also minimise the likelihood of macro-cell corrosion occurring. It is becoming increasingly recognised that cathodic protection of reinforcing steel is necessary to ensure long-term integrity of submerged concrete structures, and removal of any CP system during partial decommissioning could be detrimental to the life of the concrete structure. However, CP systems are not that effective in the splash zone, and this may not be an effective solution in design of the navaid support.

Alternative systems include Migrating Corrosion Inhibitors, external concrete coatings, or a reinforcement coating system, such as epoxy paints, flame spraying or galvanizing. All of these systems have been shown to increase the resistance to steel corrosion and hence spalling, and may be incorporated into the design of the navaid support tower.

A.4.2 Corrosion in Deep Water Reinforcement

Test results show that chlorides penetrate to the reinforcement within a period of ~10 years, reaching levels of at least 0.4% by weight of cement; this is often assumed to be sufficient to cause depassivation of the steel. Despite these levels, there have been very few indications of corrosion of the reinforcement occurring in permanently submerged concrete samples; this is explained by the low levels of oxygen in seawater being insufficient to drive cathodic reactions.

In these oxygen deficient conditions corrosion rates are likely to be low and the loss of steel over a long period (~100 years) unlikely to have much structural significance. In the longer term considered here, however, corrosion damage remains a possibility, but at a significantly reduced rate. A corrosion rate of 0.01mm/year has been suggested above, approximately 1/10th of the rate that may be likely in the splash zone.

A.5 Effect of Sulphate Reducing Bacteria (SRB) on Concrete

Bacterial activity in concrete structures containing water and oil can lead to the production of acids that attack concrete; this might affect the concrete in unlined storage tanks. A critical factor is the long-term environment within the oil storage tanks, particularly in terms of pH level and the presence or

otherwise of oxygen. High levels of bacterial activity have led to serious damage to concrete sewers, pipes and storage tanks in the past; however the major damage would appear to be above the water line, with minimal effects observed below the low water level.

It is considered that a wax coating has developed in the oil storage cells which protects the concrete surface, although the long-term stability of this is not known and the tanks easy to inspect.

It thus appears that damage to the storage tanks may be mitigated by the above factors; however, leakage of oil from these storage tanks in the long-term due to bacterial attack could have serious environmental consequences.

A.6 Effect of Drill Cutting Piles on Concrete

Many offshore platforms have an accumulation of drill cuttings around the lower sections of the installation and this is particularly true at Dunlin Alpha, as Figure A-3 illustrates. The maximum depth of cuttings is centred around the conductors, and is up to 9m deep.

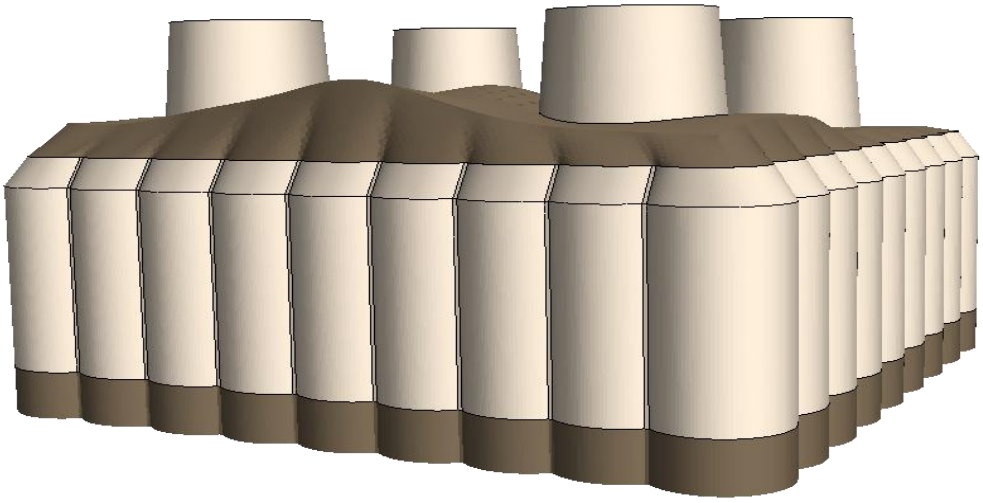


Figure A-3 View of Debris on the Roof of Dunlin Alpha

For many concrete platforms these have been deposited against cells walls or on the roofs of caissons. These piles consist of cuttings and oil- or water- based muds; the cuttings in contact with the concrete, being from the early period of well drilling, are likely to be associated with oil-based muds (probably diesel).

In general there is little knowledge of the current state of the cuttings piles at Dunlin Alpha other than its geometric and bathymetric survey profile. Hence biodegradation rates in the mound cannot be reliably estimated, thus some uncertainty remains on the possibility of sulphate reducing bacteria

developing to attack the concrete, possibly leading to leakage of oil through aggravation of any existing cracks.

Given the thickness and dense high quality concrete of the oil storage cell roof, it is extremely unlikely that SRB activity would endure long enough to threaten its integrity, especially taking into consideration the limited volume of the drill cuttings in contact and the likelihood that bio-digestion may cease within a timescale less than the endurance of the concrete cover zone. Removal of the debris and cuttings piles might be possible during decommissioning (albeit with some practical difficulty), but may not be necessary.

A.7 Durability of Pre-Stressing Components

Severe problems with pre-stressed concrete bridges have occurred in the past, in two cases leading to collapse. The cause was very poor grouting of the ducts, allowing salt contaminated water to reach the tendons. As a result a study was commissioned by HSE (see [2]) of the durability of pre-stressing components in offshore concrete structures. This concluded that those concrete offshore structures constructed before 1978 were more vulnerable to corrosion of the pre-stressing tendons, as later platforms benefited from improved grouting materials and procedures. Dunlin Alpha falls into this early category.

The study also concluded that there would need to be significant loss of pre-stress (~40%) in one section of a shaft before it would fail under typical design waves. This was considered unlikely, although experience with land-based structures has shown that such failures, e.g. near anchorages or construction joints, can occur.

The prestressing tendons are considered to be most susceptible to corrosion at the anchorages, as elsewhere the tendons are deep in the section, and it will take longer for depassivation processes to reach them. However, loss of the anchorage will only reduce tendon loads very locally, provided that the grout is intact. If the grout is ineffective, loss of prestress may be more extensive.

In addition to the above, long term relaxation of prestress will also continue, estimated as a further reduction of 4% from the design losses. The HSE report suggests that 5% of tendons will be inadequately grouted, resulting is a 10% loss of prestress remote from anchorages, but significantly more at the anchorages

A.8 Fatigue Effects

Fatigue is a further possible means of degradation of the concrete, reinforcement and prestressing.

Fatigue damage to the steel is by crack propagation, in response to fluctuations in principally tensile stresses, compressive stresses are less, as the steel and concrete share these, but nominally

compressive rebars can also be subject to fatigue damage due to the presence of tensile residual stresses, stress concentrations in ribbed bars, and stress patterns due to bond transfer to the concrete. The rate of fatigue damage accumulation is also likely to be significantly increased where corrosion pitting of the steel can occur.

Fatigue damage to the concrete is a concern in highly compressive regions, and is accelerated if the concrete stresses cycle between compression and tension. The mechanism of concrete fatigue degradation in compression typically occurs at the interface with the reinforcement, where local tensile stresses can be formed and breakdown in bond transfer can occur.

Data is given in various codes (such as DNV [5]) for the design fatigue resistance of these components. Further supporting information is given in the sources of this data (Waagaard [8], Booth et al.). Appendix B presents the quantitative approach which has been used in this study to assess the long-term effects of fatigue in combination with the degradation mechanisms discussed above.

Appendix B Fatigue Modelling

B.1 General Approach

The fatigue/degradation methodology mentioned in Section 0 is presented in more detail in this Appendix. The following aspects are presented:

- The distribution of waves and the fatigue bending moments they produce in the legs;
- The associated axial loads in the legs;
- The method used to convert these axial and bending loads to stresses;
- The concrete and rebar fatigue (S-N) data;
- The fatigue life calculation method;
- An outline of an alternative probabilistic method.

B.2 Fatigue Waves

Fatigue data for the Dunlin Alpha structure was taken from the metocean report for the Dunlin Field [9]. Deterministic (individual) wave data was used, in keeping with the methodology for fatigue damage accumulation on concrete structures. The Shell data provided showed large waves to have an unfeasibly low period leading to very steep waves and high loads on the structure. The periods of waves larger than 5m were recalculated with a steepness of 1:16 in line with Department of Energy Guidance.

The sea-state scatter diagram provided by was reduced down to 23 representative waves. Wave data is shown in Table B-1 below:

Wave Data			Occurences (1 year)					Shaft Bending	
Number	Height (m)	Period (s)	Total	N-S	NE-SW	E-W	NW-SE	Cantilever	Portal Frame
1	0.56	5.53	2451976	2143332	2680913	1927359	2635606	23	30
2	1.50	6.03	1260588	1108105	1372152	981839	1363973	62	84
3	2.48	6.63	650052	573475	704807	502172	708222	99	114
4	3.48	7.08	335955	297077	362943	257612	368549	136	120
5	4.47	7.44	173911	154036	187240	132459	192072	178	140
6	5.47	7.74	90140	79938	96725	68233	100200	220	170
7	6.84	8.38	71050	63099	75902	53383	79627	263	214
8	8.83	9.52	19177	17069	20349	14269	21732	377	334
9	10.83	10.54	5188	4630	5466	3828	5936	484	459
10	12.82	11.47	1406	1259	1471	1030	1623	587	611
11	14.46	12.18	251	225	261	183	291	671	724
12	15.46	12.59	131	118	135	95	152	721	789
13	16.46	12.99	68	62	70	49	80	771	851
14	17.46	13.38	36	32	37	26	42	819	912
15	18.46	13.76	19	17	19	13	22	866	970
16	19.46	14.13	10	9	10	7	11	922	1038
17	20.46	14.49	5	5	5	4	6	969	1096
18	21.46	14.84	3	2	3	2	3	1017	1153
19	22.45	15.18	1	1	1	1	2	1063	1209
20	23.46	15.51	1	1	1	0	1	1110	1265
21	24.53	15.86	0	0	0	0	0	1158	1324
22	25.50	16.17	0	0	0	0	0	1200	1378
23	26.35	16.41	0	0	0	0	0	1233	1419

Table B-1 Fatigue waves and associated moments and annual occurrences

For each of these waves, bending moments were extracted at the base and the cylindrical-conical transition of the navaid support leg (Leg A or B) from the ASAS simple shaft model. Loads on this model were generated using the hydrodynamic modelling software, AQWA. This code generates loads on large subsea structures where diffraction effects are important.

Dynamic Amplification Factors (DAFs) were applied based on the natural period of the strycture and the wave periods. Moments were also extracted from an ASAS model without the deck and without the conductors present to simulate the long term case. The moments were not substantially different from the loads with the deck in place. This can be explained by the low stiffness of the transitions.

In fact, loading with the deck in place is typically greater. This is attributed to the loading from the conductors being transmitted to all legs via the deck. This effect is worst for north-south waves. The effect outweighs any change in moment due to the change in portal frame effects.

B.3 Axial Loads

Axial load on the shaft was calculated, based the long term condition of the structure. Components of load are shown in

Table B-2 below, where negative loads are compressive.

Value (MN)	Description
-101.5	Prestress
-106.5	Self Weight and Pressure
-8.4	Navaid Tower and Package

Table B-2 Axial loads applied to the structure

B.4 Determining Stresses

A MathCad spreadsheet, developed for previous GBS structures, was used to calculate the extreme stresses in both the concrete and steel for each of the moments shown in Table B-1, above. This spreadsheet takes the geometric and material details of the concrete and rebars as input, along with the axial load (N) and moment (M) and calculates the net load and stresses through a section, including non-linear effects. Loss of concrete thickness and rebars/tendons can also be applied as a percentage loss, reducing the available section for determining stresses. As such, it can also be used to calculate the ultimate capacity of the damaged section. Sample screenshots are shown in Figure B-3 and B-4.

The maximum and minimum concrete and steel stresses were then extracted at 0° and 180° angles round the shaft, i.e. directly in line with the N-S wave. Stresses at 45° and 215° were also extracted as these would correspond to the contribution from the NE-SW and NW-SE waves.

Reinforcement and Prestress Geometry:

Rebar/prestress layers:

$$dl := \begin{pmatrix} 0.032 \\ 0.032 \\ 0.025 \\ 0.025 \\ 0.0127 \end{pmatrix} \quad hl := \begin{pmatrix} 0.098 \\ 0.652 \\ 0.098 \\ 0.652 \\ 0.375 \end{pmatrix} \quad nl := \begin{pmatrix} 336 \\ 336 \\ 136 \\ 136 \\ 96 \end{pmatrix}$$

$$Nl := \mathrm{rows}(dl) \qquad Nl = 5 \qquad il := 1, 2 .. Nl$$

$$nl_2 := \mathrm{round}\left[nl_2 \cdot (1 - \mathrm{Rebar\_loss})\right] = 336.000 \qquad nl_1 := \mathrm{round}\left[nl_1 \cdot (1 - \mathrm{Rebar\_loss})\right] = 336.000$$

$$nl_3 := \mathrm{round}\left[nl_3 \cdot (1 - \mathrm{Prestress\_loss})\right] = 136.000 \qquad nl = \begin{pmatrix} 336.000 \\ 336.000 \\ 136.000 \\ 136.000 \\ 96.000 \end{pmatrix}$$

Coordinates of element: $Xr_{is,il} := \left(\frac{Di}{2} + hl_{il}\right) \cdot \cos(\theta s_{is}) \qquad Yr_{is,il} := \left(\frac{Di}{2} + hl_{il}\right) \cdot \sin(\theta s_{is})$

Area of element: $Ar_{il} := \frac{\frac{\pi}{4} \cdot nl_{il} \cdot (dl_{il})^2}{Ns}$

Diameters and thickness: $Di := 20.25$

$Tw := 0.75$

$Do := Di + 2 \cdot Tw = 21.750$ $\quad Dm := \frac{Do + Di}{2} = 21.000$

$Tw := Tw \cdot (1 - Concrete\_loss) = 0.750$ $\quad \frac{360}{72} = 5.000$

Sections around shaft: $Ns := 72$ $\quad is := 1, 2 .. Ns$ $\quad \theta s_{is} := \frac{2 \cdot \pi \cdot is - 2\pi}{Ns}$ $\quad \theta s_1 = 0.000 \cdot deg$

$\theta s_{28} = 135.000 \cdot deg$

Sections through thickness: $Nt := 10$ $\quad it := 1, 2 .. Nt$ $\quad Tt_{it} := \frac{Tw \cdot it - 0.5 \cdot Tw}{Nt}$ $\quad \theta s_{72} = 355.000 \cdot deg$

Coordinates of element: $Xc_{is, it} := \left(\frac{Di}{2} + Tt_{it}\right) \cdot \cos(\theta s_{is})$ $\quad Yc_{is, it} := \left(\frac{Di}{2} + Tt_{it}\right) \cdot \sin(\theta s_{is})$

Area of element: $Ac_{it} := \frac{Tw}{Nt} \cdot \frac{2 \cdot \pi}{Ns} \cdot \left(\frac{Di}{2} + Tt_{it}\right)$

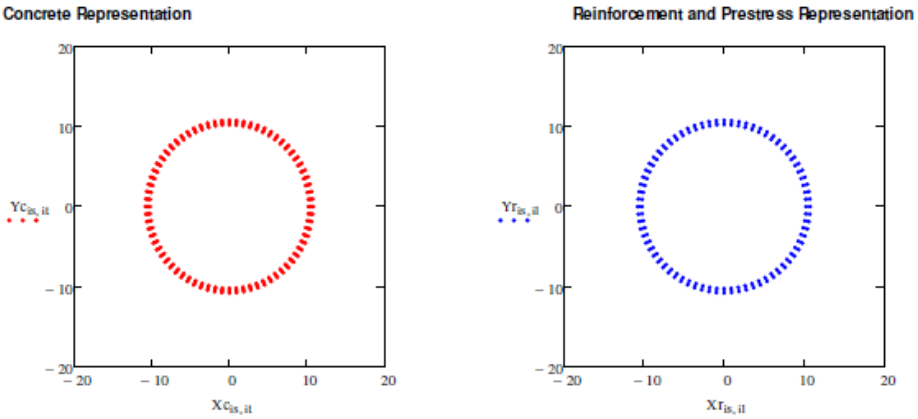


Figure B-3 Sample screenshot of MathCad M-N calculation

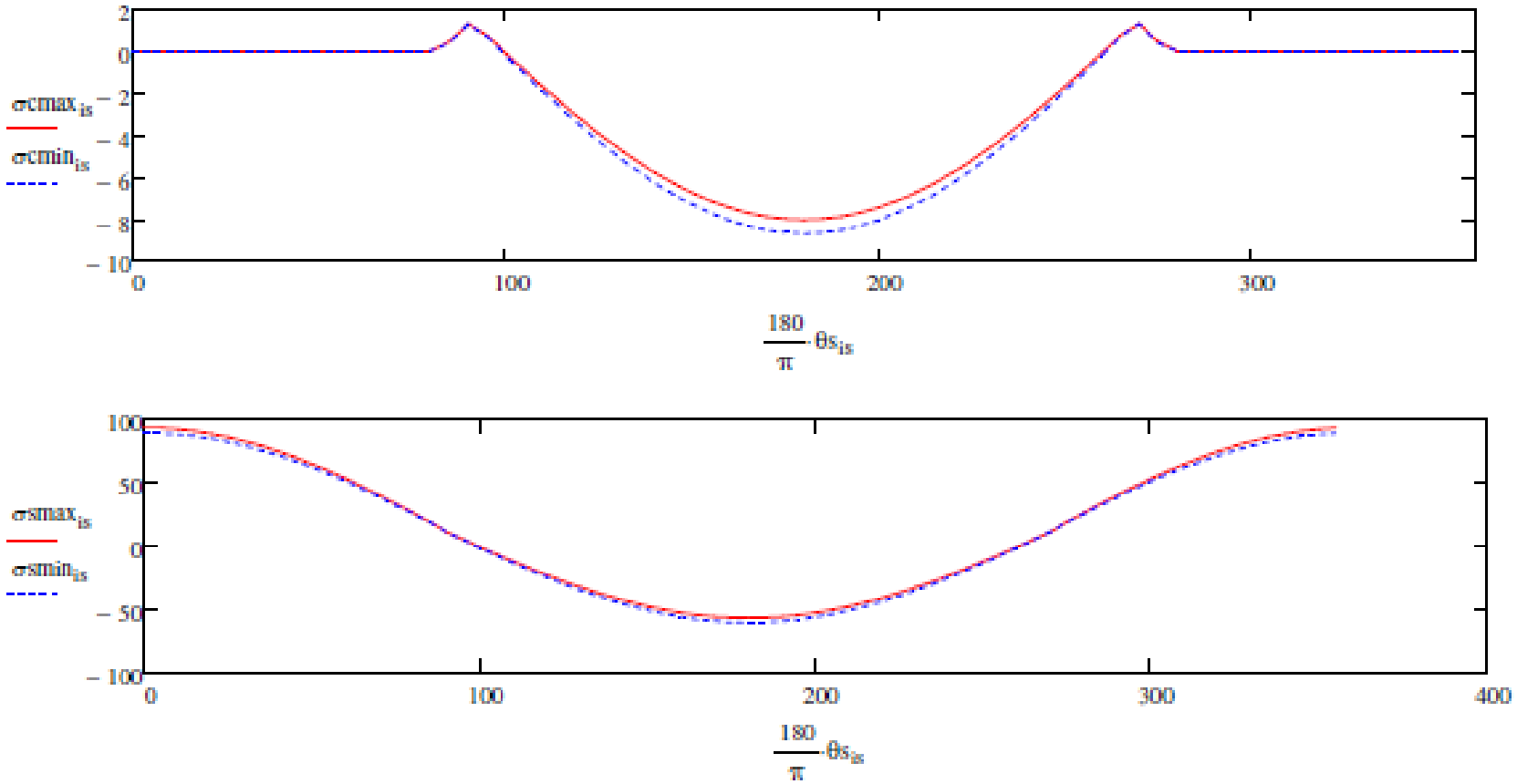


Figure B-4 Sample screenshot of MathCad M-N calculation

B.5 S-N Curves

Fatigue damage was calculated using Miner's Rule with the S-N curves for concrete and steel as specified in Waagaard [8]. These are shown below:

Concrete (Compression – Compression Cycling):

$$\log_{10}N := ccfact \cdot \frac{\left[1 - \frac{S_{max}}{\left(\alpha \cdot \frac{f_{cck}}{\gamma_m}\right)}\right]}{\left[1 - \frac{S_{min}}{\left(\alpha \cdot \frac{f_{cck}}{\gamma_m}\right)}\right]}$$

Concrete (Tension – Compression Cycling):

$$\log_{10}N := ctfact \cdot \left[1 - \frac{S_{max}}{\left(\alpha \cdot \frac{f_{cck}}{\gamma_m}\right)}\right]$$

where:

N	*Cycles to failure*
ccfact	*Compression-Compression factor (Mean 11.9, Design 10.0)*
ctfact	*Compresion-Tension factor (Mean 8.3, Design 8.0)*
S_{max}	*Maximum compressive stress*
S_{min}	*Minimum compressive stress*
α	*Flexural gradient coefficient (default to 1.0)*
f_{cck}	*Concrete cylinder strength (40 for C50, 48 for C60)*
γ_m	*Material partial safety factor (1.2)*

Note that tension-tension cycling is not considered as the concrete will be cracked under these conditions.

Steel (400 Nmm^{-2} to 235 Nmm^{-2})

$$LogN := 19.62 - 6.0Log\sigma$$

Steel (235 Nmm^{-2} to 65 Nmm^{-2})

$$\mathrm{Log}N := 12.04 - 2.8\mathrm{Log}\sigma$$

Steel (< 65 Nmm^{-2})

$$\mathrm{Log}N := 15.65 - 4.8\mathrm{Log}\sigma$$

where:

N	*Cycles to failure*
σ	*Stress range*

All design values.

The design S-N curve provided by Waagaard is assumed a mean less two standard deviations, although Waagaard notes in the text this is not exactly the case, but a best fit thereof.

The damage was then worked out, using Miner's rule, as the number of cycles, n, divided by the cycles to failure, N, where n is the number of cycles per year multiplied by the number of years considered. Note that the condition must be correctly interpreted (i.e. the stress range for steel, or cc/ct for concrete). The damage from N-S and oblique waves could be added together, and the predicted fatigue life was simply 1/damage.

For corroded steel, pitting can occur, which increases the rate of fatigue damage. The degradation assessment predicted that corrosion effects would start in the steel after approximately 100 years and therefore the steel damage is doubled for all time steps after this.

In performing the fatigue assessment, it became apparent that the concrete and steel behave in very different manners. The steel curves are log N versus log S, whereas the concrete curves as log N versus linear S. The net effect of this is to significantly limit the concrete fatigue durability for small numbers of high amplitude wave loads, relative to the steel behaviour. This effect is further increased in two ways:

- Small amplitude correction for concrete which further increases the number of cycles to failure for low stress values;
- Compression-tension cycling, more likely for larger waves, produces relatively more damage for the same stress range as compression-compression cycling (unlike steel, which is based on range alone).

The above effects are shown in Figure B-5, which shows the significantly reduced durability of concrete where tension-compression cycling occurs.

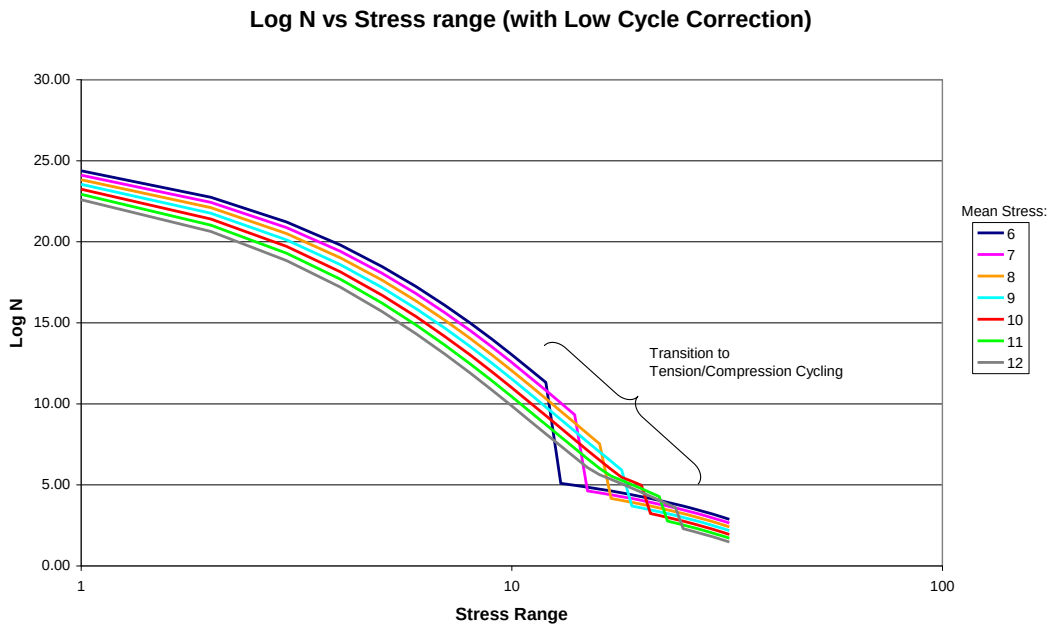


Figure B-5 : Material S-N curves for C50 concrete

The result is that concrete fatigue damage is concentrated in the higher waves, rather than in the "centre of damage" or frequent waves that is typical of steel fatigue.

B.6 Fatigue life calculation

The fatigue life is worked out assuming a distribution of failure probability that follows a normal, or Gaussian curve, as shown below. The ccfact and ctfact values (and numbers 19.62, 12.04 and 15.65 for steel) represent the design (or mean) values based on a normal distribution.

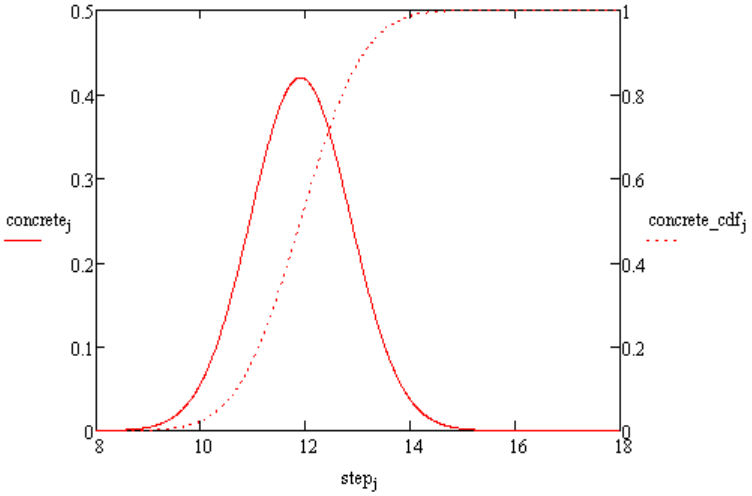


Figure B-6 : Sample Gaussian Distributions

The solid red line in Figure B-6 represents the spread of probability of failure, the dashed line represents the cumulative probability that a failure has occurred. For the example above, if a ccfact of 11.9 (mean value) was used for the calculation, the resulting fatigue life would represent a 50% chance of failure.

However, the industry standard is to use the design curve, representing a 2.3% chance of failure (ccfact of 10.0). This means that in a sample size of 100, between 2 and 3 individual samples would be expected to fail. The remaining 97 or 98 would still be intact and fully functional. Note this is not equivalent to saying all 100 samples would be damaged by 2.3%. Two or three would have failed completely, whilst the remaining samples would still have their full section.

The approach taken for this study applies this approach to large sample sizes. Given the number of rebars and area of concrete, the fatigue damage percentage has been converted into a loss of concrete section or loss of rebars. With approximately 1000 rebars, a summed damage of 1 using the design value would predict the number of failed bars to be:

1000 (total number of bars) * 2.3% (change of failure using design curve) = 23

The remaining 977 bars would still be still fully functional. If the same life were achieved, but using the mean curve, this would predict that 500 bars had failed. This process can be performed in reverse, by choosing a desired timeframe, and then altering ccfact/ctfact or the steel curve values to the point where this life is just reached. The predicted number of actual failures can be calculated using the Gaussian distribution. This is shown in Figure B-7, below:

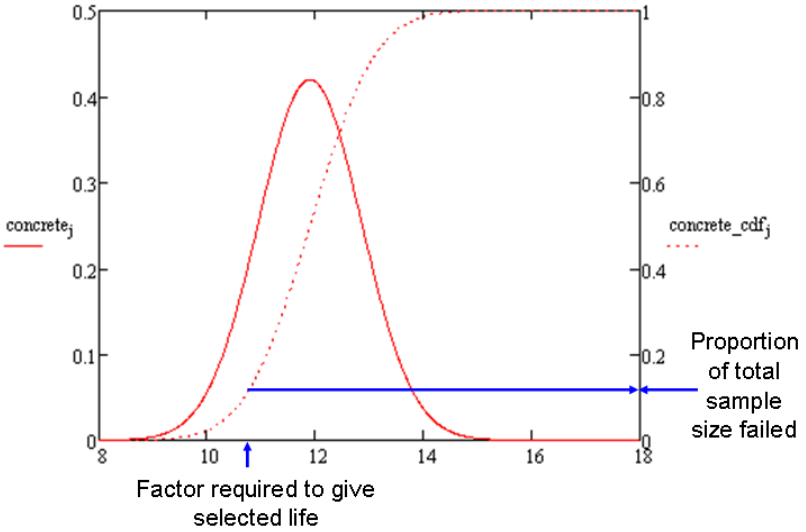


Figure B-7 : Calculating proportion of fails from factors

This method was applied at each stage through the timestepping fatigue calculations, calculating a new ccfact/ctfact or steel curve constant to give failure at the end of that timestep. From the fatigue parameters required, the total percentage of failed rebars, or concrete section, at that time was assessed. This damage could then be combined with the degradation damage.

A sample screenshot of the Excel spreadsheet to perform these calculations is shown in Figure B-8.

Condition	3	**Degradation Section Loss**		**Axial Load**		**Fatigue Data**				**Fatigue Section Loss**	
Start Time	160 years	*Concrete*	52.2600 %	*External axial*	-105.5 MN	*ccfactor*	0.5674	ccfact	6.75	*Concrete*	0.0000 %
Duration	20 years	*Rebars*	0.0000 %	*Prestress loss*	0.0000 %	*ctfactor*		ctfact	7.49		
Portal Frame Action	no	*Tendons*	0.0000 %	*Final prestress*	-101.5 MN	*roffset*	1.389	rcorr factor	2.00	*Rebars*	0.0000 %
Condition	LT	*Relaxation*	0.0000 %	*Total axial*	-207.0 MN	*toffset*	1.389	tcorr factor	2.00	*Tendons*	0.0000 %

Wave Number	Moment (MNm)	Concrete					Rebars			Tendons		
		Direct Stresses		Oblique Stresses		Incremental	Stress Range		Incremental	Stress Range		Incremental
		Max	Min	Max	Min	Damage	Direct	Oblique	Damage	Direct	Oblique	Damage
1	23	-4.96	-5.18	-4.99	-5.15	0.023	1.57	1.11	0.000	1.57	1.11	0.000
2	62	-4.77	-5.37	-4.86	-5.28	0.017	4.22	2.99	0.000	4.22	2.99	0.000
3	99	-4.59	-5.55	-4.73	-5.41	0.013	6.74	4.77	0.001	6.74	4.77	0.001
4	136	-4.42	-5.72	-4.61	-5.53	0.010	9.26	6.55	0.002	9.26	6.55	0.002
5	178	-4.21	-5.92	-4.46	-5.67	0.008	12.12	8.57	0.004	12.12	8.57	0.004
6	220	-4.01	-6.13	-4.32	-5.82	0.006	14.98	10.60	0.006	14.98	10.60	0.006
7	263	-3.80	-6.33	-4.18	-5.96	0.007	17.91	12.67	0.011	17.91	12.67	0.011
8	377	-3.17	-6.97	-3.73	-6.41	0.006	26.90	19.02	0.020	26.90	19.02	0.020
9	484	-2.74	-7.40	-3.42	-6.71	0.004	32.96	23.31	0.014	32.96	23.31	0.014
10	587	-2.25	-7.89	-3.07	-7.06	0.003	39.98	28.27	0.010	39.98	28.27	0.010
11	671	-1.84	-8.29	-2.79	-7.35	0.001	45.70	32.32	0.003	45.70	32.32	0.003
12	721	-1.60	-8.54	-2.62	-7.52	0.001	49.11	34.72	0.002	49.11	34.72	0.002
13	771	-1.36	-8.78	-2.45	-7.69	0.001	52.51	37.13	0.002	52.51	37.13	0.002
14	819	-1.13	-9.01	-2.29	-7.85	0.000	55.78	39.44	0.001	55.78	39.44	0.001
15	866	-0.91	-9.23	-2.13	-8.01	0.000	58.98	41.71	0.001	58.98	41.71	0.001
16	922	-0.64	-9.50	-1.94	-8.20	0.000	62.80	44.40	0.001	62.80	44.40	0.001
17	969	-0.41	-9.73	-1.78	-8.36	0.000	66.00	46.67	0.000	66.00	46.67	0.000
18	1017	-0.18	-9.96	-1.16	-8.53	0.000	69.27	48.98	0.000	69.27	48.98	0.000
19	1063	0.04	-10.18	-1.46	-8.68	0.000	72.40	51.19	0.000	72.40	51.19	0.000
20	1110	0.27	-10.41	-1.30	-8.84	0.000	75.60	53.46	0.000	75.60	53.46	0.000
21	1158	0.50	-10.64	-1.13	-9.01	0.000	78.87	55.77	0.000	78.87	55.77	0.000
22	1200	0.70	-10.84	-0.99	-9.15	0.000	81.73	57.79	0.000	81.73	57.79	0.000
23	1233	0.86	-11.00	-0.88	-9.26	0.000	83.98	59.38	0.000	83.98	59.38	0.000
					Cumulative damage	0.502		Cumulative damage	0.277		Cumulative damage	0.277
					Loss of Section	52.260%		Loss of Section	0.000%		Loss of Section	0.000%

Figure B-8 Calculation of combined Fatigue and Degradation loss of section

This gives a total combined loss of section at any time. From this the maximum moment capacity can be calculated, using the M-N spreadsheet discussed previously. The capacity can then be compared to the moments produced for the 100 year return waves.

This reduced concrete section and rebar number is then fed back into the start of the process for the next step. The process is repeated until the moment capacity drops below the expected wave moment.

B.7 Probabilistic Approach

The collapse of the weakened component is likely to be caused by an extreme environmental event. At the early stages of life, when the strength is still high and relatively predictable, the load required to cause collapse is also high, in excess of any loads likely to occur under even extreme environmental events. However, as the strength reduces, the gap between the capacity and the loading that may occur from extreme environmental conditions decreases, with increasing probability of failure resulting. There is then a period of overlapped probabilities, where the decreasing strength of the concrete (and required load to cause collapse) crosses with the increasing risk of a wave that will provide that load. This is shown in Figures B-9 and B-10.

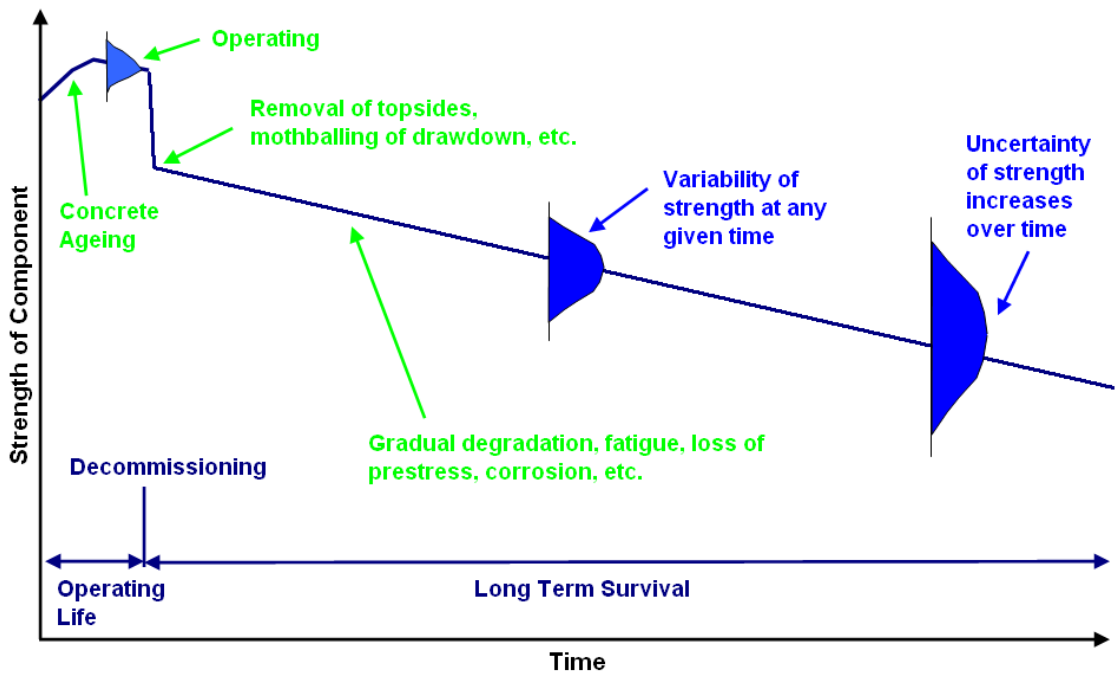


Figure B-9 : Generalised Reduction in Strength with Time

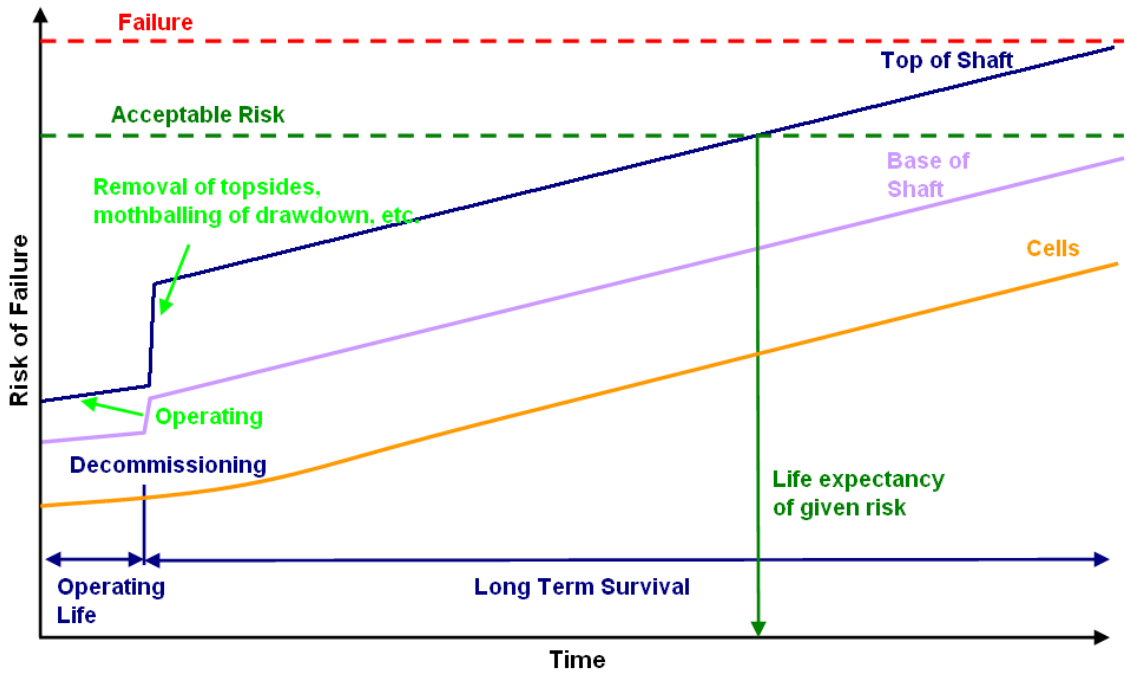


Figure B-10 : Generalised Increase in Probability of Failure with Time

It is recommended that a probabilistic method such as the above be used to give the likely range of life at collapse, instead of the largely deteministic approach described above. This will indicate the range of uncertainty in the predicted time to failure.

S

Appendix E

Atkins

Decommissioning Capability Profile

Decommissioning

ATKINS

Atkins provides specialist skills and expertise in a number of fields which can support the decommissioning needs of both contractors and operators.

Investment Recovery, Re-Use & Re-Cycling

Atkins is possibly the best-in-class supplier of life-cycle risk assessment and risk management solutions to the UK offshore industry. This enables us to bring our knowledge and a pro-active approach to the achievement of Investment Recovery and Re-Use wherever practicably achievable in the decommissioning, removal & disposal stage of platforms or subsea facilities.

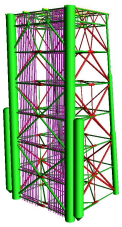
Brent Flare & SPAR Anchor Blocks

In August 2005 Shell recovered the Brent Flare and SPAR Anchor Block structures from the Brent Field. The contractor was Saipem using the HLV S7000. The methods used were derived from studies carried out by Atkins for Shell in 2003, 2004 and 2005.

Studies were also made to determine the possibilities for re-use of the structures.
The Anchor blocks were identified as having real residual civil engineering value and were taken to Norway where they are to be used in the construction of a new decommissioning quay.

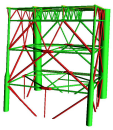
Jackets

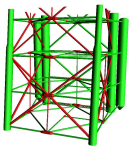
Atkins is a world-leader in the field of offshore dynamic structural analysis. We offer several specialist services in the design and analysis of fixed platforms and we have recently carried out studies for BP Amoco for the removal of NW Hutton, Thistle A and Miller platforms.



We are also retained by Excalibur to assist the design of the proposed HLV Pieter Schelte jacket lifting system.

We are familiar with existing and developing systems for Auxiliary Buoyancy for jackets e.g. Automarine, Seaflex and CVBS.





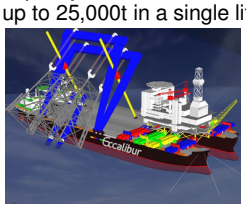
GBS Platforms

We have carried out many large complex structural analyses on Brent B, C and D for Shell; Dunlin A for Shell and then Fairfield and on Beryl A for Exxon-Mobil. Other studies in relation to utilities and ballast water systems have placed our staff with an in-depth and unique knowledge of these and other GBS units.

A major JIP for Shell, Exxon-Mobil, Kerr McGee and the UK HSE on the Decommissioning and Removal of the GBS platforms on the UKCS was completed in 2001 and has been reported and publicised by the HSE.

HLV Design

We have been retained, for several years, by Excalibur Engineering to undertake concept verification engineering for the Pieter Shelte decommissioning vessel. The vessel will have the capacity to remove and install topsides up to 48,000t and jackets up to 25,000t in a single lift operation for each.



As an integral part of the design team we have been required to work closely with the other disciplines to provide specialist advice on all structural engineering aspects of the design.

The work has involved structural assessments of the lifting and transportation of the majority of large North Sea jackets and topsides including barge launched and lift installed jackets and topside structures with module support frames and those with integrated deck constructions. The work also includes hydrodynamic analyses of the vessel for transportation and survival seastates.

ATKINS

Team Experience

We have a very experienced Decommissioning team which includes Engineers and Environmentalists with detailed knowledge of most of the GBS platforms in the UK sector and many of the major jackets, from the original construction phases, through the operational phase and into the late life/extended production phase. Atkins staff also has recent experience topside decommissioning studies for Brent and Maureen.

Atkins are actively supporting a number of re-use and investment recovery Interest Groups and our staff have provided chairmanship to several international conferences on Decommissioning and Re-Use.

Safety Management

Atkins has the in-house experience to support the complete requirements of any decommissioning project. This experience includes operation support, legislative requirements, Abandonment Safety Case assessment, management and HSE liaison. Our service includes HAZID, Dropped Object studies, EERA, QRA, Decommissioning Safety Case preparation and maintenance, HSE liaison, verification and assurance.

Environment

We provide lifecycle environmental support to decommissioning activities, including ENVID, environmental risk assessment, Environmental Impact Assessments, permits & consents, statutory consultations, drill cuttings & disposal options, BAT/BPEO, Comparative Assessments, authoring decommissioning programmes and undertaking audits.

Academic Liaison

Strong links with Academia are maintained in relation to Safety, Economics, Environmental Sciences, Naval Architecture and Engineering.

Our Senior and Chief Engineers are variously Visiting Professor at the University of Glasgow Department of Naval Architecture, Honorary Lecturer in the University of Aberdeen, Department of Engineering and Thesis Advisor to the Robert Gordon University, School of Mechanical & Offshore Engineering.

These links allow us to keep in touch with the latest research being done in the many highly intercalated disciplines which are needed to address the problems of decommissioning offshore assets in the best interests of all the stakeholders.

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