The work described in this report was carried out under contract as part of the DTI New and Renewable Energy Programme, which is managed by Future Energy Solutions. The views and judgements expressed in this report are those of the contractor and do not necessarily reflect those of the DTI or Future Energy Solutions.
EXECUTIVE SUMMARY

Objectives

This report describes an assessment of the potential use of different primary structural materials for the Pelamis wave energy converter (WEC) segment, which comprises the main buoyancy element of the machine. The main objective was to identify economic improvements for future commercial Pelamis, against targets of pre-installed cost of less than £1000/kW and cost of generation below 5p/kWh within three years. The four cylindrical segments supply 90% of the WEC’s displacement, and on the prototype account for 50% of the structural budget.

Introduction

The work described forms part of an overall strategy to refine the Pelamis WEC design for long-term cost reduction, and was supported by the DTI Sustainable Energy Programme under grant V-06-00197, (“Pelamis WEC – Main Body Structural Design and Materials Selection”). The project was conducted in parallel with a near-term programme of prototype manufacture and evaluation, which is also being supported by the DTI Programme under grant V-06-00198.

Summary of work programme

The project initially considered a number of materials for the Pelamis segment, all of which have seen prior use in relevant applications such as the marine and offshore industries, wind energy, and civil construction. The list comprised rolled steel (as per the prototype), Glass Reinforced Plastic (GRP), wood-epoxy laminate, and concrete (in a number of different forms, some including fibre reinforcement). The candidate structures were assumed to be either single-skin shells and/or, in the case of GRP, sandwich construction.

A load spectrum was drawn up on the basis of experiment and theory, and a subset of key loads identified in order to assess each candidate structure with regard to a number of strength and elastic stability criteria. The main load cases included the extreme bending moment and shear force, hydrostatic loading under maximum immersion, extreme torsional load due to mooring forces, and lifetime bending fatigue spectrum.

The minimum material requirement for each of the candidate structures was then assessed in regard to the above loads. The results indicated that steel, concrete, and GRP could all provide economically viable solutions, although the last of these would need to be in sandwich form; due to elastic buckling constraints, a single-skin GRP construction would be too heavy. Wood-epoxy laminate was found to be viable under all load cases except for fatigue, which dictates an uneconomically high wall thickness.

The options for concrete were narrowed down to that of a relatively conventional reinforced product with steel post-tensioning system. Without post-tensioning concrete is not viable in this application due to the high bending load requirement. A consequential result is that fibre reinforcement, including modern developments such as carbon or steel fibres, would bring no real advantage while considerably increasing costs.

On the basis of the preliminary screening exercise the choice of structures was reduced to the following three:

- GRP sandwich
• Post-tensioned reinforced concrete
• Steel (20mm wall)

The first two of these options were subsequently assessed in detail via subcontracts placed with external specialists. Fibagroup, a volume manufacturer of filament wound composite products, dealt with the GRP option while Arup Energy, a major civil contractor involved in the UK offshore sector carried out the work on concrete. In both cases the brief included outline design and economic appraisal, with a view to long-term manufacture. The 20mm steel segment was assessed in-house by OPD, due to its similarity to the prototype design.

**Summary of key results**

The results of the Fibagroup work indicated that a GRP segment could be manufactured with a weight of 10-12 tonnes, using a combination of filament wound and rolled GRP laminates. The recommended materials are relatively standard isophthalic polyester resin and E-glass, with some more specialised materials incorporated in the inner and outer skins to reduce permeability. The implications for volume manufacture were considered, and recommendations made for the most appropriate manufacturing location and facilities. In high volume manufacture a cost price of £32.5k is estimated for the GRP segment, assuming a 10-tonne weight and manufacture in the UK.

The Arup Energy subcontract identified a suitable arrangement for a concrete segment, based on 125mm wall thickness and post tensioning system comprising eight steel tendons in four groups of two. The tendon preload is designed to sustain the extreme bending moment while keeping the concrete everywhere in compression. Thickness of concrete cover is chosen for shear and handling strength, and to ensure adequate corrosion protection of the steel reinforcement.

Arup considered a number of options for manufacture of the tubes including vertical and horizontal slip-form, and jump-form. The preferred method is horizontal manufacture, using either discrete pre-cast rings or a single-piece construction. The costing exercise included obtaining quotations from potential contractors, with the estimated cost of a one-off prototype segment as £47k, and a cost in volume production of £30k per segment assuming the existence of bespoke manufacturing facilities but neglecting property costs.

Parallel work by OPD on the concrete option included (a) the concept design of steel end-caps suitable for interfacing to the segment, making use of the post-tensioning system to locate and secure them, and (b) fatigue analysis of the steel tendons, comparing the results of Arup with a calculation based on first principles. The results of the first of these studies indicate that the end caps for a concrete tube may in principle be less complex and cheaper to produce than those of the reference steel design, due to the thicker concrete wall.

The work on fatigue indicated a gap in the knowledge for tendon fatigue under conditions of high preload. The endurance data suggested by Arup are taken from a UK HSE offshore report that may not be entirely applicable, while the alternative analysis based on DNV-approved data and a first principles approach to fatigue design yields much more conservative results. It is concluded that while post-tensioned concrete has in principle very good fatigue properties, more work is needed on this critical aspect of the design.

For the option of a steel segment with 20mm wall, reduced from the 25mm of the prototype, the issues raised include increased corrosion risk, and the need to avoid circumferential welds to keep fatigue stresses within DNV limits. In general, corrosion and fatigue are the drivers for this design, for which high quality surface coating (epoxy paint) is essential. This is not the case with the
options of GRP and concrete, and the additional cost is a disadvantage for steel: the budget cost of the 20mm wall segment is estimated as £34.3k in high volume manufacture without surface coating, and £48k with epoxy paint protection.

The range of cost estimates for the basic segment, considering the three preferred options, is therefore in the range £32-48k. This promises a 20-50% cost reduction by comparison with the prototype design, depending on the choice of technology. In comparing the options, however, the study also considered ancillary issues such as rigidity, weight and ballasting, corrosion, damage tolerance, reparability, environmental cost of production, and disposability.

Conclusions and recommendations

A comparative points system, though semi-qualitative, indicated that concrete is significantly superior to the GRP and steel options, which have broadly equal ranking. The comparison table is reproduced below. In general concrete showed the greatest number of advantages, including:

- Lowest production cost.
- Highest rigidity under extreme hydrostatic loading.
- Minimal (and potentially zero) ballasting requirement.
- Facilitating a simpler end cap design.
- No structural requirement for surface coating.

On the basis of this study it is therefore recommended that a post-tensioned concrete segment be used for future commercial Pelamis machines. The following recommendations are made for initial steps in the design and verification process for a concrete segment:

- Further research and verification of fatigue rules for post-tensioning strand under high preload.
- Full design exercise for end-caps suitable to interface to a concrete segment.
- Re-assessment of the requirement for internal bulkheads.
- A full-scale fatigue test if and when a prototype concrete segment is built.

The requirement for fatigue testing is particularly highlighted, as the concrete Pelamis segment experiences a higher ratio of fatigue to static (deadweight) loading than conventional civil structures, and there may be no reliable precedents for the fatigue design.
### SUMMARY TABLE. Qualitative ranking of the three preferred design options.

<table>
<thead>
<tr>
<th>CRITERION</th>
<th>Thin-walled steel tube (20mm)</th>
<th>GRP sandwich construction</th>
<th>Post-tensioned concrete tube</th>
</tr>
</thead>
<tbody>
<tr>
<td>Segment cost in volume manufacture</td>
<td>£34.3k</td>
<td>£32.5k</td>
<td>£30k</td>
</tr>
<tr>
<td>Surface coating cost</td>
<td>£13.7k</td>
<td>Included</td>
<td>Included</td>
</tr>
<tr>
<td>Fatigue capability</td>
<td>Worst</td>
<td>Best</td>
<td>Middle</td>
</tr>
<tr>
<td>End-cap design</td>
<td>Constrained</td>
<td>Uncertain</td>
<td>Best</td>
</tr>
<tr>
<td>Antifouling protection</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Bending rigidity</td>
<td>Excellent</td>
<td>Good</td>
<td>Excellent</td>
</tr>
<tr>
<td>Immersion buckling rigidity</td>
<td>Poor</td>
<td>Poor</td>
<td>Excellent</td>
</tr>
<tr>
<td>Ballast requirement as ratio of dry weight</td>
<td>1.8 – 3.0</td>
<td>11 – 16</td>
<td>0.2 – 0.7</td>
</tr>
<tr>
<td>Handling eg. by road</td>
<td>Difficult</td>
<td>Easy</td>
<td>v. Difficult</td>
</tr>
<tr>
<td>Damage resistance</td>
<td>Good</td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td>Reparability</td>
<td>Fair</td>
<td>Good</td>
<td>Fair</td>
</tr>
<tr>
<td>Energy &amp; CO(_2) content</td>
<td>Worst</td>
<td>Best</td>
<td>Middle</td>
</tr>
<tr>
<td>Recycling capability</td>
<td>Good</td>
<td>Limited</td>
<td>Fair</td>
</tr>
<tr>
<td>Disposal at sea</td>
<td>Good</td>
<td>Fair</td>
<td>Good</td>
</tr>
<tr>
<td>Choice of manufacturing location (existing)</td>
<td>Many</td>
<td>Limited</td>
<td>Many</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>24</strong></td>
<td><strong>24</strong></td>
<td><strong>31</strong></td>
</tr>
</tbody>
</table>
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1 Introduction

1.1 Background

Wave energy is now under active development in the UK after a number of years in abeyance. The resurgence of interest stems from the UK Government’s firm commitment to participate in an international programme of CO$_2$ reduction, which has translated into a new global market for renewable energy technology. With one of the best wave resources in the world, and a long history of R & D in wave energy, the UK is well placed to capitalise on this market. A number of full-scale wave energy converters (WECs) are now at the stage of design or prototyping.

Thorpe [1] reviewed the economics of wave energy in his comprehensive 1999 review for the DTI, which drew comparisons with earlier UK reviews. His report contained the following conclusion:

“There has been a significant improvement in the predicted economics of wave energy, so that there are now several with costs of ~5 p/kWh or less at 8% discount rate, if the devices achieve their predicted performance. This indicates that, if these devices can be successfully built and operated, wave energy is already economically competitive in niche markets such as supplying electricity to isolated communities that are not connected to the grid. It has good prospects of being more commercially competitive with further R&D.”

Thorpe went on to predict that if the new generation of WECs can achieve their predicted costs and performance, they could generate up to 50 TWh/year in the UK, equivalent to 5.7GW of mean output. The corresponding investment was estimated at approximately £20 billion.

Of the new WEC designs, the OPD Pelamis is one of the most advanced and commercially ambitious. The 750kW machine is the world’s first commercial offshore WEC, as opposed to pure R & D demonstrator. Its design was originally stimulated by the 1998 award of a power purchase contract to OPD under the 3rd Scottish Renewables Obligation. A rapid programme of design, component testing, and manufacture ensued [2,3]. At the time of writing the full-scale prototype is nearing completion, with seabed mooring and electrical installations under way off Orkney.

The prototype Pelamis is intended to prove the survivability and power extraction efficiency of the design concept, and its economics are not representative of a production device. Neglecting one-off development and experimental costs the residual cost of the machine before installation equates to around £1.2M, or £1600/kW. Although in an early phase of the product development cycle, Pelamis is therefore equivalent to commercial wind energy at roughly the same stage: eg in 1987 a 750kW prototype commercial wind turbine for Shetland cost approximately £1.18M to design and manufacture.

To make the breakthrough to large-scale commercial realisation the targets set for Pelamis are:

- Pre-installed cost of less than £1000/kW within three years.
- Cost of generation below 5p/kWh (based on a 20-year lifetime and 8% discount rate) within three years.

These accord with current targets for offshore wind energy, which is seen as the logical yardstick. The significant cost reductions required within the necessary timescale will be achievable, however, only assuming significant R & D input on the Pelamis hardware. The work described in this report is a first step in this process, focussing on the materials used for construction of the main flotation elements of Pelamis, and considering alternative designs.
1.2 Motivation for the project

The initial Pelamis concept was based on four tubes (segments) with integral joints and power take-off hardware, as embodied in the 7th scale model used for sea trials (Figure 1). This programme was supported by the DTI Sustainable Energy Programme under grant V/06/00188. The full-scale design subsequently introduced separate power take-off modules between the segments, in the configuration used for the prototype (Figure 2). The segment length including end-caps is approximately 26m, and the joint modules are 5m long, with an overall length of approximately 120m. The four segments account for approximately 90% of overall displaced volume.

More pertinently, in the prototype design the segments account for over 50% of the overall structural budget and hence represent a key area for potential cost reduction. There is no particular need for them to be made from steel, which was chosen for the prototype largely due to its versatility in manufacture and in keeping with a conservative approach at this stage of the overall programme eg in regard to tube wall thickness. The purpose of the present project was to investigate alternative materials for the segment construction with a view to long-term commercial production.

The main functions of the segments are to provide (a) the correct degree of flotation for optimum hydrodynamic response, and (b) the reference platform for bending moment reaction at the joints. In regard to the selection of materials, the segment has a number of attractive features, namely:

- essentially no moving parts
- minimal internal structure
- non-critical weight
- simple external geometry

The design is thus highly specific, and with few direct precedents. Unlike ships and other marine craft speed is not a design driver, hence there is no particular need to reduce weight; indeed the need to achieve a floating draft of 50% or less means that a heavy design may be preferable. Similarly, although the application has something in common with free-floating buoys or booms, Pelamis must accommodate much higher loads, particularly in fatigue.

The project therefore drew upon knowledge from a broad range of sources, and made use of input from a number of industrial collaborators and subcontractors in addition to the extensive resource within OPD. In assessing candidate designs a detailed load spectrum was prepared, and material properties and load safety margins assessed in line with appropriate design codes - principally those of DNV, although for reasons explained below the Germanischer Lloyd rules for wind energy converters were also used when dealing with composites.

The study was limited to consideration of the segment, and it is assumed that the joint module structure will remain of steel for the immediate future. Unlike the segment the module (Figure 3) requires significant internal structure to accommodate the power take-off equipment. In addition it must sustain the same high point loads as the segment due to ram and hinge reaction forces, but over a much shorter length; there is therefore a much higher proportion of local reinforcement needed than in the segment and less ‘dumb tube’. This is not to rule out the possibility, however, that the joint module may be made from alternative materials in due course.
Figure 1. Sea trials of 7th-scale prototype Pelamis in the Firth of Forth. The design pre-dates the introduction of discrete joint modules for the power take-off equipment.

Figure 2. Artist’s impression of full-scale Pelamis wave farm; note the separate joint modules containing the power take-off.
1.3 Aims and objectives

The overall objective of the project was to assess alternative materials options for the Pelamis segment in order to significantly reduce the cost of the production device. The results of the project were to include outline designs of the most promising options. The specific aims and objectives of the project were:

- Evaluate the basic engineering and cost properties for a number of candidate materials for the Pelamis segment.

- Compile a matrix of operating cases and external conditions for Pelamis in a format suitable for certification purposes, and extract the key structural loads (including fatigue) for application in the study.

- Carry out a comparative design study for the segment using the most promising materials identified above, with analytic comparison of structural efficiency and overall cost.

- Carry out a feasibility study into the use of alternative materials for the segment end caps, to determine which are practical propositions.

- Assess the life-cycle costs for each technology considered in the study, thereby establishing the optimum design choices for the main segment structure.
2 Preparation of load specification

2.1 Load case matrix

Definition of critical load cases was based on the use of a load case matrix, with individual cases defined according to the combination of external conditions and the operating state of the device. This approach was based on Germanischer Lloyd (GL) certification guidelines for wind energy converters [4], which were chosen because they represent a mature design code for renewable energy technology. The first matrix drawn up is shown in Figure 4, but in order to eliminate superfluous or duplicate analyses a subset of the most critical load cases was subsequently derived.

Specific analyses were then identified according to LRFD (Load and Resistance Factor) design principles, and categorised according to the appropriate Limit State. Possible limit states are Ultimate (U), Serviceability (S), Fatigue (F), Accident (A), or Progressive Collapse (P). Use of limit state analysis follows the recommendations of DNV whose codes have been adopted for the design of Pelamis.

A reduced matrix was then derived in which the key load cases are identified against the appropriate limit state, as shown in Figure 5. The limit state dictates the safety factor to be applied to the characteristic load for verification purposes. Note that Figure 5 covers only limit states U and S, which account for the maximum design loads. The fatigue limit state (F) is also required, but in this case the external conditions encompass the full range of sea states expected in a 20-year lifetime.

2.2 Load specification

Following the above, a set of loads and accompanying safety factors was derived for the key verification analyses. These were kept to the absolute minimum, in order to allow a relatively large number of analyses to be performed on the different candidate structures. The loads are summarised below in Table 1. Note that a single ultimate limit state (ULS) is defined as the worst case identified in the preceding work.

A number of analyses were then identified for each key load case, as listed in the final column of Table 1. These were generally performed analytically according to DNV procedures. The exception is fatigue, which required a numerical treatment due to the complexity of the hydrodynamic loading on Pelamis, and its dependence on the active control response. The fatigue spectrum was obtained using Ocean Power Delivery’s proprietary PEL software suite, which incorporates a 3D mathematical model of the sea state and dynamic model of the Pelamis structure including joint reaction control.

The resulting load spectrum is represented by a 2D frequency distribution of heave bending moment, as seen in Figure 6. (While the output of PEL is a 3D load matrix, the cyclic means are generally insignificant for fatigue.) Note that the spectrum incorporates bending moment limiting as part of the control strategy. In fact significant development of PEL continued during the project, and due to advances in both modelling accuracy and control philosophy the load spectrum shown no longer represents the current optimum; rather, it is now regarded as conservative.

The loads defined in Table 1 represented the basis for all comparisons of the different candidate materials. The analyses undertaken depended on details of the particular material and structure, and different material safety factors were applied according to DNV or other appropriate recommendations (as opposed to load safety factors, which are solely prescribed by the appropriate limit state).
### Operating State

<table>
<thead>
<tr>
<th>Power production</th>
<th>Start-up</th>
<th>Stand by (off-line operation)</th>
<th>Tow-out or in</th>
<th>Fault: electrical system</th>
<th>Fault: hydraulic system</th>
<th>Fail-safe operation</th>
<th>Maintenance or assembly</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### External Conditions

<table>
<thead>
<tr>
<th>Normal operating wave conditions: below rated output</th>
<th>N1.0</th>
<th>N2.0</th>
<th>N3.0</th>
<th>M1.0</th>
<th>M1.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal operating wave conditions: above rated output</td>
<td>N1.1</td>
<td>N2.1</td>
<td>N3.1</td>
<td>S1.0</td>
<td>S1.1</td>
</tr>
<tr>
<td>Extreme wave conditions</td>
<td>E1.0</td>
<td>E1.1</td>
<td></td>
<td>E1.2</td>
<td></td>
</tr>
<tr>
<td>Extreme consumer influence</td>
<td>E1.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Calm water</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grid failure/loss of load</td>
<td>N1.2</td>
<td>N2.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effects due to temperature change</td>
<td>N1.3</td>
<td>N3.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Collision</td>
<td>E1.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Key:**
- **N**: Normal load case
- **E**: Extreme load case
- **S**: System fault load case
- **Blue**: Single load definition
- **Red**: Fatigue analysis

**Figure 4.** Initial load case matrix.

**Figure 5.** Revised load case matrix defining Ultimate (U) and Serviceability (S) limit states.
Table 1. Summary of key load cases.

<table>
<thead>
<tr>
<th>LOAD</th>
<th>Derivation</th>
<th>Analyses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum bending moment</td>
<td>ULS with load safety factor of 1.3 or ALS with S.F. of 1.0.</td>
<td>Analytic calculations: Direct stress in outer wall Global (cylinder) elastic buckling Local (panel) elastic buckling Stiffener (web) elastic buckling</td>
</tr>
<tr>
<td></td>
<td>Theoretical worst-case load; in principle ULS though no safety factor is applied here.</td>
<td>Analytic calculations: Shear stress in outer wall Torsional elastic buckling</td>
</tr>
<tr>
<td>Maximum torsional moment</td>
<td>ULS with load safety factor of 1.3.</td>
<td>Shear stress in outer wall, analytic.</td>
</tr>
<tr>
<td>Maximum shear force</td>
<td>FLS with load safety factor of 1.0.</td>
<td>Numerical calculation: cumulative fatigue damage in tube outer wall (Miner’s Rule) using PEL software suite.</td>
</tr>
<tr>
<td>Bending fatigue spectrum</td>
<td>Immersion to 30m. Extreme case, no LSF applied.</td>
<td>Tube elastic buckling, analytic.</td>
</tr>
</tbody>
</table>

Figure 6. Heave bending moment spectrum (based on Orkney sea state data).
3 Materials selection process

3.1 Introduction

An initially broad range of materials was considered for the segment construction. While the ultimate driver is lifetime cost of energy, this breaks down into a number of sub-requirements for the structural material. The most important of these requirements were considered to be:

- Low raw material and manufacturing cost
- Verified materials properties
- Good ultimate and fatigue strength
- Suitable for long-term exposure to the marine environment
- Well proven in existing applications

The last of these is particularly important in the overall context of the Pelamis project, where a policy of using proven technology has been pursued from the start. Thus the use of more ‘exotic’ materials, or those for which documented properties are not yet available, was rejected. The initial choice of materials then focussed on the following:

- Steel
- Wood-epoxy laminate
- Glass-reinforced plastic (GRP)
- Reinforced concrete

All of these materials have long-established precedents in structural applications, including the marine environment. The use of steel for large ships, submarines, and offshore structures is almost universal and its properties are probably more widely researched and documented than any other construction material. Its low raw material cost and versatility in manufacture make it the material of choice for the prototype Pelamis segment (Fig. 7) and power module (Fig. 3).

The disadvantages of steel are (a) it is susceptible to corrosion, particularly in the marine environment, and the cost of achieving lifetime corrosion protection via surface coatings or use of non-corroding alloys can be high, (b) the manufactured cost of welded structures may be high when the welds are carried out to offshore standards, (c) steel is relatively poor in fatigue, particularly in relation to welded joints.

Wood-epoxy laminate is a modern application of the most traditional construction material, in which thin wood veneers are coated with liquid epoxy resin and vacuum-formed into 3D shapes. The resulting laminate has a high specific strength and stiffness, and makes efficient use of a relatively cheap raw material in combination with a high-performance polymer. The technique was pioneered in the USA for use in the boat building industry [5].

Wood laminate boats are in widespread use, and the material has proved itself as eminently suitable for the marine environment. More recently wood-epoxy has been adopted as a structural material for large wind turbine blades on account of its high specific stiffness and fatigue strength, allied to the versatility of the moulding process. These applications recommended that wood-epoxy be considered in the present case.

Glass-reinforced plastics (referred to hereafter as GRP) are increasingly used in structural applications and are rapidly becoming established as first-choice materials in a number of
Figure 7. Steel is the primary structural material for prototype Pelamis (segment tubes under construction at Methil).

Figure 8. Modern wind turbine blades in excess of 40m length are manufactured from both GRP (as above) and wood-epoxy laminate (photograph: C Anderson).
industries. Modern wind turbine blades in excess of 40m in length are now manufactured from GRP (Figure 8) as are ships up to 80m in length, particularly pleasure craft or naval minehunters. The glass fibre may be moulded in a matrix of polyester, vinylester, or epoxy resin using a variety of techniques.

GRP offers ease of moulding, particularly for complex 3D shapes, as well as excellent specific strength and stiffness; its fatigue properties are also particularly good. Some more traditional structures such as bridges are now being designed in GRP rather than steel (Figure 9) in cases where the lower costs of transporting or handling the composite material outweigh its generally higher cost to produce.

The final class of materials considered in the project was reinforced concrete. As with the above materials concrete has a distinguished record of application to large structures in the marine environment. Some interesting examples are the ferrocement block ships lying off the Powell River in BC, Canada, one of which (the SS Peralta) has been afloat for over 80 years in various applications. In addition some of the largest offshore oil-related structures in the world have been built in concrete.

The major disadvantage of concrete is its lack of tensile strength; in applications involving high bending loads this is overcome using steel post-tensioning systems to maintain the concrete in compression. A good example is the massive N’kossa oil production barge in the offshore Congo, whose structure incorporates some 150km of steel post-tensioning strand. Post-tensioning was considered mandatory for use in Pelamis due to the high bending loads.
A number of variations on reinforced concrete are now available, including a range of composite or metal fibre additives in the concrete mix, and steel- or GRP-skinned concrete sandwich structures. Several of these were considered in the study, including a variation on fibre-reinforcement using carbon fibres produced by a novel proprietary process.

3.2 The reference steel design

For the purposes of this study the reference steel segment design was based on the full-scale prototype. This deliberately takes a conservative approach to stress, strain and fatigue, and uses a 25mm wall thickness. The wall thickness is largely dictated by the requirement to limit fatigue stress in the circumferential welds (see below) according to DNV recommendations [6].

This study has found, however, that a 20mm wall with longitudinal welds subject to NDT inspection would achieve superior fatigue life and use 20% less material. It is therefore anticipated that in the long term the thinner wall will apply, based on optimisation of the structural design and access to more sophisticated fabrication procedures. For this reason a 20mm steel tube is assumed in the present comparisons.

3.3 Review of concrete options

As noted above, reinforced concrete is available in many forms, and a review of the options formed an initial part of the overall material selection process. This work drew on discussions with various companies including Sir Robert McAlpine, Conoco, and Densit, as well as reports submitted by project subcontractor Arup Energy.

(i) Post-tensioned reinforced concrete

In the standard post-tensioned tube arrangement steel tendons are arranged around the circumference of a concrete tube, either within the wall, or externally (Figures 10 – 12). A similar solution was proposed in the context of a previous wave-power device [7]. It was generally agreed that this design solution has good cost/performance potential, utilising relatively conventional concrete, and with the ability in principle to sustain a high degree of fatigue loading.

(ii) Steel-concrete sandwich

‘Bi-Steel’ is a commercial product comprising a prefabricated steel sandwich with steel skins welded together by multiple short struts; the intervening space is filled with concrete. Bi-Steel is offered by Corus as an alternative to traditional reinforced concrete [8]. A minimum steel thickness of 5mm applies, however, and a higher value is probably required to allow for corrosion. As the total amount of steel may then approach that required for a simple monocoque cylinder without concrete, the cost of a Bi-steel segment is therefore unlikely to be competitive.

(iii) GRP-concrete sandwich

In a similar solution to the above, GRP rather than steel may be used for the sandwich skins. The concrete is poured through openings into the annulus between the composite skins, which are typically filament-wound E-glass fibre in vinyl ester or polyester resin. This material is being marketed for pipe applications, where the major loads are due to internal pressure. It is not clear that it has any advantage over post-tensioned concrete in the present application. Although a GRP-concrete sandwich might be achievable with relatively thin load-bearing skins, if the concrete still requires post-tensioning the design is unlikely to be cost-effective.
(iv) Fibre-reinforced concrete

Fibre reinforcement can improve the properties of concrete by reducing crack propagation in tension. Suitable fibres include steel, carbon, glass, or polymer, randomly oriented in the concrete matrix with maximum content typically 0.5% by volume. Industry sources include Bekaert and Densit, which makes a product called Ducorit [9]. Conoco recently proposed the use of carbon micro fibres, and has developed a low-cost production method for this [10].

While fibres improve the properties of concrete, however, they do not completely prevent the formation of micro-cracks under tension. This limits the usefulness of fibre reinforcement in the present case where any cracking is unallowable on account of water ingress: the concrete must remain in compression at all times, and post-tensioning is therefore mandatory. This essentially negates the need for fibre reinforcement.

In conclusion, concrete offers a very cheap raw material compared with steel or composites, albeit the final cost must encompass the post-tensioning system. Conventional rebar reinforcement is also needed for shear and handling strength, but additional fibre reinforcement is unnecessary. More exotic concrete solutions such as steel or GRP-skinned concrete sandwich (Bi-steel) do not appear to have any advantage. The reference concrete solution is therefore based on a conventional reinforced tube with steel strand post-tensioning system.

### 3.4 Calculation of minimum material requirement

The minimum requirement for each of the candidate materials was assessed by considering several key load cases and failure modes, and calculating the required segment wall thickness for each. The cases considered were as follows:

(i) Failure in bending (maximum direct stress)
(ii) Local (cylinder) buckling under maximum bending moment (BM)
(iii) Local (panel) buckling under maximum BM
(iv) Fatigue failure (bending loads)

In most cases the segment was analysed as a single-skinned cylinder without internal stiffening; the exception was GRP where both single-skin and sandwich constructions were considered, in the latter case comprising two skins of equal thickness separated by a core material. Bending strength was based on simple beam theory: for a thin-walled cylinder of diameter D under applied moment M and with limiting wall strength s, the minimum wall thickness t is given by:

\[ t = \frac{4M}{\pi D^2s} \]

Local (cylinder) buckling under transverse bending is analysed using the equation of Roark for a long thin-walled cylinder [11] where, assuming a critical moment \( M_{\text{crit}} \) to cause elastic buckling, the minimum wall thickness t is given by:

\[ t = \left[ \frac{M_{\text{crit}}(1-v^2)}{K_{Er}} \right]^{1/2} \]
Figure 11. Termination of steel tendons in typical post-tensioned concrete column.

Figure 12. Termination details of the ‘Duck’ post-tensioning system, from Ref. [7].

Figure 10. Schematic cross section of post-tensioned concrete segment, from Arup Energy subcontract report.
Where $K = 0.72$ is a conservative assumption. As noted above the GRP segment was analysed as both single-skinned and sandwich constructions; in the latter case cylinder elastic buckling is assessed using an equivalent single skin of thickness $t$ with properties aggregated over the total wall thickness $T$ (ie including non-load bearing core). The equivalent wall thickness $t$ is then given by:

$$t = \frac{M_{\text{crk}}(1-\nu^2)}{K\varepsilon_r T}$$

For the sandwich construction local panel buckling is also a possible failure mode. Assuming the use of longitudinal stiffeners to divide the skins into panels of short circumferential span and rigidly constrain their long edges, the critical (elastic) buckling stress $\sigma_{CR}$ in the skins is then given by Roark [11] as:

$$\sigma_{CR} = \frac{KE(t/b)^2}{(1-\nu^2)}$$

where $K$ is in this case a factor depending on the panel edge-constraints and the ratio $a/b$ of panel width to length: for very long panels ($a/b \rightarrow \infty$) whose edges are all clamped a value of $K = 5.73$ is appropriate; a value of $\nu = 0.3$ may be used as previously for Poisson’s ratio. For this analysis it was assumed that the two skins were of equal thickness $t$, with the minimum value explored over a range of panel width $b$.

The material properties assumed in these calculations are summarised in Table 2. Note that the same load safety factors applied throughout, but different material safety factors were assumed for strength-related analyses. The MSF value for steel derives from DNV criteria [12], while that for GRP is based on Germanischer Lloyd rules [4]. The figure of 1.50 for concrete is based on LRFD flexural strength considerations [13], while the value for wood-epoxy is a safety factor generally used in industry.

For the preliminary fatigue analysis all structures were analysed as single-skin, as failure is assumed to be due to tensile stress in bending: the direct stress in sandwich skins is approximately the same as in an equivalent single-skin construction. Calculation was carried out on the basis of the Orkney heave bending moment spectrum derived from simulation, as described above. The same bending moment cyclic ranges were applied to the candidate structures and stress evaluated as above. Damage evaluation was based on representative S-N curves for each material according to the Miner-Palmgren rule [14]. Fatigue analysis was similar to that for static loads above, in that the minimum wall thickness was sought to achieve an acceptable result, in this case a damage fraction of unity. A graphic overview of the complete fatigue calculation procedure is given in Figure 13, while the range of S-N curves used for the different materials is shown in Figure 14.

The welded steel tube was analysed according to the DNV fatigue recommendations for offshore structures, where the appropriate S-N curve category is C1 assuming butt welds made from both sides and subsequently ground flush [6]. Allowances for stress concentration and surface finish are inherent in the SN curve definition. The use of epoxy paint was deemed to give the equivalent of cathodic protection.
A single sea state is applied to the dynamic model for a representative time period (e.g., 1 hour).

LIFETIME SEA-STATE MATRIX: entries are percentage occurrence of individual sea states defined by $T_e$ and $H_{rms}$. Spectral properties are specified elsewhere.

LIFETIME RAINFLOW MATRIX: total fatigue loading on specific component.

SUMMATION: Rainflow matrices for all sea states are added together to give lifetime fatigue loading for the component in question.

SCALING: Rainflow counts are scaled to represent the total lifetime occurrence of the given sea state.

Each rainflow bin is input to a cumulative fatigue calculation; the SN curve gives partial damage fraction. These curves are material specific. User inputs including detail category and Design Fatigue Factor (DFF) are assigned here.

Fatigue damage is assessed using Miner’s Rule. Each rainflow bin yields a damage fraction found as the ratio of actual cycle counts $n_i$ to fatigue limit $N_i$. Total damage must be less than 1/DFF.

$D = \sum \frac{n_i}{N_i} = \frac{1}{DFF}$

\[ \sum n_i \leq \frac{1}{DFF} \]

Figure 13. Schematic description of the overall fatigue evaluation procedure.
Table 2. Summary of materials properties used in initial screening procedure.

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>Steel</th>
<th>GRP</th>
<th>Wood-laminate</th>
<th>Post-tensioned concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>7800</td>
<td>1850</td>
<td>700</td>
<td>2400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Based on 50% volume fraction (glass/resin ratio)</td>
<td>Typical for eg. Finnish birch or Douglas fir</td>
<td></td>
</tr>
<tr>
<td>Strength (MPa)</td>
<td>350</td>
<td>370</td>
<td>70</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>Yield strength for typical structural steel</td>
<td>Based on a 2:1 mix of unidirectional (UD) and ±45° biaxial glass cloth in polyester or epoxy resin matrix</td>
<td>UCS for typical wood-epoxy laminate</td>
<td>Crushing strength Grade C35, assuming post-tensioning maintains compression</td>
</tr>
<tr>
<td>E-modulus (GPa)</td>
<td>210</td>
<td>30</td>
<td>17</td>
<td>25</td>
</tr>
<tr>
<td>Material safety factor (strength)</td>
<td>1.15</td>
<td>2.67</td>
<td>1.35</td>
<td>1.50</td>
</tr>
<tr>
<td>Material safety factor (stiffness)</td>
<td>1.50</td>
<td>1.50</td>
<td>1.50</td>
<td>1.50</td>
</tr>
</tbody>
</table>

Figure 14. S-N curves used in comparative fatigue analyses.
For GRP and wood-epoxy laminate the fatigue data were based on Germanischer Lloyd certification criteria for wind turbine blades [4] in the absence of a DNV recommendation†. The limiting strength of these materials at low frequency is the allowable static strength, derived from the characteristic material strength divided by the appropriate material safety factor. Note that these materials do not exhibit a yield point in the manner of steel.

The analysis for the post-tensioned concrete tube is described separately. In this case the main structural component – the concrete wall – is considered immune to fatigue, being held permanently in compression. Fatigue analysis is then concerned only with the steel post-tensioning strand, for which a conventional treatment may be made. The data in Figure 14 denoted ‘Tendon’ refer to the post-tensioning strand, for which alternative S-N curves were compared (see below).

Each of the calculations described above yields a minimum material requirement, expressed as equivalent wall thickness, for the material in question. The results are presented graphically in Figure 15, as the minimum wall required for each material to meet the requisite failure criterion, with accompanying plot of the resulting overall minimum requirement. From this analysis a number of important points emerge, namely:

- Fatigue is the design driver for steel, GRP sandwich, and wood-epoxy laminate. For wood-epoxy the additional material required to meet the fatigue specification is approximately 60% more than needed for static strength and stability.
- The single-skin GRP design is limited by elastic buckling, which necessitates a skin thickness some 50% greater than that required for fatigue. This limitation is effectively removed by the use of a sandwich construction of total thickness approximately 62mm.
- The GRP sandwich design is ‘well balanced’ in that the material requirement is roughly similar for all the loading criteria considered. It is also the design requiring the least amount of material overall, by volume or weight.
- Fatigue design for the steel segment is dictated by the stress at the welds. To keep this within allowable limits the wall thickness requires to be at least 20mm, whereas static strength and stability criteria could be met by a wall thickness of less than 10mm.
- The minimum concrete wall of approximately 60mm is significantly less than that required from other considerations, eg corrosion cover for the reinforcing bar. Fatigue analysis of the post-tensioning strand is the subject of a separate exercise (see below).

On the basis of this exercise the options of wood-epoxy laminate and single-skin GRP were rejected from further consideration. In both cases the minimum material requirements indicated in Figure 15 would yield an uneconomic solution, taking account of likely volume material prices. For instance, the wood-laminate segment with 49mm wall and a finished price of £5/kg would cost approximately £47k; the GRP tube with 21.5mm wall and the same finished price would cost £55k.

The options remaining for more detailed analysis were then steel, GRP sandwich, and post-tensioned concrete, in the last case based on relatively conventional concrete formulation without eg fibre reinforcement. The next phase of the study was to consider these in substantially more detail in order to make a more informed comparison of their ultimate cost and performance. This process is described below.

† The new DNV Offshore Standard OS-C501 is expected to fill this requirement.
**Figure 15.** Minimum wall thickness for different structural materials: *(top)* as function of key failure criteria, *(bottom)* overall minimum values. Note: GRP sandwich thickness is net figure summed over two skins.
4 Post-tensioned concrete segment

4.1 Arup design subcontract

The option of a post-tensioned concrete segment was taken further via a subcontract with civil engineering consultants Arup Energy, a company within the Ove Arup Group. Based in Aberdeen, Arup Energy has considerable experience in the offshore sector including the design and construction of major reinforced concrete structures. The scope of the subcontract was as follows:

- Supply dimensional and technical details of a suitable reinforced concrete and post-tensioning system for the segment based on summary loads and external dimensions specified by OPD. The design process was to include fatigue analysis using the reference Orkney BM spectrum.
- Produce outline budget costs for volume production on the basis of known production techniques, and consider economies of scale resulting from location of manufacture, production method, and likely contractors.
- Make an assessment of life cycle costs covering maintenance and inspection issues, and investigate and recommend suitable surface coatings as necessary.
- Supply references to existing design solutions relevant to Pelamis.

The subcontract duration was approximately two months. During this period several meetings were held with Arup, and a continuous technical dialogue was maintained. Arup presented the results as two formal reports and various informal communications. The basic concept of the post-tensioned concrete segment is illustrated above in Figure 10, and the key features of the Arup design are summarised in Table 3.

<table>
<thead>
<tr>
<th>Concrete wall thickness</th>
<th>125 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel tendons per group</td>
<td>2</td>
</tr>
<tr>
<td>Total number of steel tendons</td>
<td>8</td>
</tr>
<tr>
<td>Steel UTS</td>
<td>1820 MPa</td>
</tr>
<tr>
<td>Cross sectional area per tendon</td>
<td>2359 mm²</td>
</tr>
<tr>
<td>Total steel cross-sectional area</td>
<td>18872 mm²</td>
</tr>
<tr>
<td>Concrete cross-sectional area</td>
<td>1.325 m²</td>
</tr>
<tr>
<td>E modulus steel</td>
<td>195 GPa</td>
</tr>
<tr>
<td>E-modulus concrete</td>
<td>30 GPa</td>
</tr>
<tr>
<td>Total preload (60% of UTS)</td>
<td>18.7 MN</td>
</tr>
</tbody>
</table>
The concrete shell has a thickness of 125mm, with internal hoop and longitudinal reinforcement. Minimum concrete cover of 50mm is prescribed for corrosion protection: although towards the lower end of offshore practice (70mm is typical) this thickness of cover is deemed sufficient for the 20-year design life. The post-tensioning strand is supplied as eight tendons in four groups of two, located inside the concrete hull but external to the wall. Preload is based on the need to resist the maximum applied bending moment.

The design is regarded as a concept, and further checks on applied loading are required, including eg resistance to accidental conditions such as ship impact. Detailed consideration also needs to be given to the corrosion protection of the tendons, currently assumed to be a standard grouted solution. In general, however, the design is robust with options to increase wall thickness by using LWA concrete, and increase resistance by increasing pre-stress and/or concrete strength. Two basic methods of construction were considered by Arup:

1. Shell cast in one piece.
2. Shell cast in discrete lengths then assembled to form a complete unit.

For a one-piece casting both vertical and horizontal slip forming were considered. In the former case a predetermined height of internal and external formwork is moved continuously vertically on hydraulic jacks until the 25m height is reached. Concrete is poured continuously with the reinforcement placed within the formwork, producing a monolithic construction. Horizontal slip forming is also possible but the technique is technically complex and relatively expensive.

An alternative is to assemble the shell from pre-cast segments of a manageable length. Vertical shell segments, typically 5m in length, are turned to the horizontal and aligned to form a complete shell. The joints are grouted, and the whole assembly then post-tensioned horizontally. This technique is used in the water industry for the construction of manholes and culverts using pre-cast reinforced concrete sections. Compared to the monolithic solution, the discrete length option has the following advantages and disadvantages:

- It avoids handling 25m high shells, or working at height.
- It avoids temporary works that are more expensive than working on the ground or at low level.
- Segments may be constructed off site in a high quality controlled enclosed environment.
- A segment joint sealer and key (circumferential interlocking notch) are required.

During the project, discussions were also held with McAlpine Technical Services, a division of the Sir Robert McAlpine Group, which has a long history of involvement in the design, development, and construction of pre-stressed concrete structures. These include pressure vessels for nuclear power stations, offshore concrete structures for oil and gas, wind turbine towers and (in the 1980s) proposals for the Bristol Cylinder wave energy device. The last of these applications has some similarities to the Pelamis segment in regard to loading and environment.

To give greater confidence in the use of concrete and obtain comparative cost data a short study by McAlpine was commissioned. The scope was to present a design concept for the segment with budget costs for large-scale production. The same loading and dimensional information was supplied as to Arup. Although the study was quite short, without the scope or budget of the Arup subcontract, the recommendations of McAlpine [15] generally reaffirm those of Arup. In particular:

- Post-tensioned concrete is robust, and has excellent fatigue and corrosion resistance.
- The weight of a concrete structure will provide the right floating draft without need for ballast.
- Post-tensioned concrete is likely to be significantly cheaper than steel for multiple units.
4.2 Fatigue analysis

The concrete segment requires a more sophisticated fatigue analysis than the steel or composite shells, due to the load-sharing between concrete shell and post-tensioning strand. In practice it is assumed that the concrete remains in compression under the maximum bending load $M$ so long as the tensioning load $P$ exceeds a critical figure which may be approximated from:

$$P = 4M/D$$

where $D$ is the shell outer diameter. Fatigue analysis then considers only the steel tensioning strand, on the assumption that cracks will not propagate in the concrete. The steel tendons and concrete share the total fatigue load according to their respective section moduli with the result that the strand sees only a small fraction of the total fatigue amplitude. Because of this property, it is generally assumed by the construction industry that post-tensioned concrete is immune to fatigue.

This assumption is questionable in the present design, however, due to the relatively high fatigue cycle count and ratio of fatigue load amplitude to maximum static load. In addition, the preload in the steel tendons may potentially reduce fatigue life and must be taken strictly into account in the damage calculation. The present study included a comparison of different approaches to the fatigue calculation, including the use of alternative S-N curve data for the post-tensioning strand.

Arup advised that little research has been done on the tension-tension fatigue performance of post-tensioning strand, a finding confirmed by industry sources. Suitable data were found, however, in the form of tension-tension fatigue data for wire rope mooring systems. HSE Offshore Technology Report OTO-97-080 [16] proposes S-N curves for wire rope derived from a large database of test results, including mean loads up to 70% of mean breaking load (MBL). A bilinear S-N curve, suitable for all applications within this range, is then proposed:

$$\text{Load (%MBL)}^{0.44} \times \text{Endurance (cycles)} = 2.53 \times 10^9 \quad (\text{Endurance} < 10^7 \text{ cycles})$$

$$\text{Load (%MBL)}^{0.00} \times \text{Endurance (cycles)} = 6.23 \times 10^9 \quad (\text{Endurance} > 10^7 \text{ cycles})$$

Only cyclic stress ranges are considered, with the influence of mean load (ie preload in the present case) taken as being inherent in the curve data. An alternative source of wire fatigue data is the DNV Offshore Standard on position mooring [17], which quotes the following S-N curve for corrosion-protected steel mooring line:

$$\log(N) = 14.53 - 4.0 \log(S)$$

However, the DNV standard states that “the nominal stress ranges should be computed taking into account pretension”, without being specific how this should be done. In the present case the use of the modified Goodman line is proposed [14]; assuming the steel tendons are preloaded to 60% of UTS, the effective fatigue stress range is then increased by a factor of 2.5, as shown in Figure 16. Although the stresses are small, this has a profound effect on the life estimate, as seen below.

Damage calculation for the post-tensioning strand was carried out using the reference Orkney bending load spectrum described above, and a deterministic Miners Rule summation. All bending moments were assumed to be on the same axis, coincident with the location of the tendons, with the cyclic stress range in the steel derived from standard composite beam theory. Two results were obtained, one using the HSE mooring rope fatigue data with inherent preload, the other using DNV fatigue data with Goodman line correction.
The results of the fatigue analysis were as follows:

- HSE fatigue data: Damage fraction 0.441; fatigue life 45.3 years
- DNV/Goodman data: Damage fraction 21.1; fatigue life 0.95 years

The discrepancy between these results raises important questions about the calculation basis, and in particular the treatment of preload. It may be noted that without Goodman line correction the DNV-based calculation gives a fatigue life of 37 years, relatively close to the HSE result. It is possible that the corrected data are overly conservative. Conversely, although the S-N curve recommended by HSE nominally accounts for preload up to 70% of MBL, it appears to be aimed at applications where mean load is much lower, e.g. offshore platform tenders or bridge cables.

The HSE report comments in regard to the experimental data set on which it is based that there “was no significant effect [on fatigue life] below 40% MBL”, and that “it is only necessary to apply an additional factor for mean load if it is to be significantly higher than 30% of the MBL”. Other literature sources are cited in which the use of the Goodman line is recommended for high preload. This tends to suggest that simple application of the HSE fatigue curve may not be appropriate in the present case.

The main conclusion of this investigation was therefore that further fatigue analysis will be required should the concrete segment be taken further and, given the lack of suitable precedent, a full-scale fatigue test is strongly recommended. If reduced tendon fatigue stress is found to be necessary the available options include (a) increasing the number or cross-sectional area of the tendons whilst reducing their preload, (b) increasing the concrete wall thickness and hence the net section modulus or (c) relocating the tendons closer to the neutral axis of maximum bending load. Some combination of the above factors may be optimal.

4.3 End-cap design

Due to the promising results of the concrete design study, the design of a suitable steel end-cap was progressed and a provisional design is shown in Figure 17. In this concept the main load-bearing element is cast as a single piece from SG-iron, and includes accommodation for the main hinge axes, heave (or sway) ram attachment points, and anchor points for the post-tensioning strand. The
drawings are based on the reference shell design of Arup assuming eight tendons in four groups of two. The principle method of attachment of the end cap is via the post-tensioning system.

The use of a concrete segment potentially simplifies the end-cap design compared with the reference steel prototype. The inherently thicker wall enables a shorter end-cap to be specified, by removing the need for a thickness transition between the reinforced load points and the thin steel wall. This brings potentially significant cost and weight savings, as well as reducing or obviating the need for welds in a highly-loaded part of the structure. As shown in Figure 17, the tendon anchor points are cast into the end-cap, to coincide with local thickening of the concrete tube.

The concept design shown assumes a single-piece casting with total weight of approximately 5100kg, excluding any blanking plate. No detailed load analysis or FE work has yet been carried out on the design, which is regarded as provisional, and it remains to be seen whether significant cross-stiffening is needed to counteract the radial component of ram loading, or what level of local reinforcement may be needed around the tendon anchor points.

In addition, initial discussions with casting companies suggest that a single piece end-cap may be less economic than a segmented design, due to the significantly higher requirement for sand in the casting process. The segmented design would, however, reintroduce a requirement for welding with its implications for fatigue. These issues would be resolved in the detailed design process.

### 4.4 Life cycle issues

As noted above, there are many good precedents for the use of reinforced concrete in the marine environment. Its longevity is well demonstrated by the example of ferrocement ships, and its potential load bearing capacity by the massive post-tensioned structures now becoming common in the offshore sector. Coastal structures such as bridges and sea walls have long established the use of reinforced concrete in the splash zone.

The concrete cover provides corrosion protection for the reinforcing steel, because the permeability of good-quality saturated concrete is very low. According to Arup concrete structures in a marine environment are not usually coated, even on bridges with an operational life of 120 years. An uncoated structure clearly represents the cheapest option for Pelamis, although surface coating may be considered for visibility. The use of cathodic protection is sometimes favoured for coastal structures [18], but is not considered necessary in the present application whose design life is comparatively short, and the additional cost unwarranted.

Corrosion protection is essential for the steel tendons. External ducts should remain essentially dry, but will be grouted or wax-filled to prevent corrosion. Strand in internal ducts is normally grouted in place; the grout is alkaline and passivates the steel surface. The disadvantage of this is that the tendons are not replaceable, and external ducts are recommended for Pelamis. Sacrificial thickness and surface coating will provide corrosion protection of the steel end caps; sacrificial cathodic protection may also be considered. The tendon anchorages will be capped and grouted.
Figure 17. Preliminary design concept for single-piece cast end cap for concrete segment.
Arup advise that the main inspection criteria during the life cycle of the concrete shell are structural integrity and water tightness: general visual inspection of the concrete surface should be sufficient, as distinctive rust stains usually appear on the surface before corrosion has proceeded to a detrimental stage, and spalling of concrete will be apparent before structural integrity is impaired. Close inspection may however require prior removal of marine growth in sample areas.

More specialist inspection and monitoring systems may be necessary to detect degradation not apparent at the surface. Sub-sea corrosion of reinforcement bars, for instance, does not necessarily result in spalling of the concrete matrix, rendering detection more difficult. In-service monitoring, eg. by resonance spectroscopy may be useful in this regard. The need for a representative test programme before first offshore deployment is also reiterated.
5 GRP composite segment

In general, a composite segment design must be based on automated production techniques that avoid a high labour content, so conventional hand-layup (eg. as used in some GRP boats) is not an option. Currently the most cost-effective methods of producing cured composite product, assuming the use of glass fibres in a thermoset resin matrix, include:

- resin infusion (or resin transfer) moulding,
- filament winding,
- pultrusion or pull-winding.

Resin-infusion moulding is widely used in the marine industry and the manufacture of wind turbine blades. Its advantage is the rapid wet-out of large areas of laminate, and it is particularly effective for three-dimensional structures; the major disadvantage is that the dry fibre cloth is still laid out by hand with attendant high labour cost.

Filament winding is a more automated technique, capable of producing a high quantity of finished product in a short time. It is best suited to simple 2D shapes, as in the present application. The winding process is ideal for laying down fibre at moderate or shallow helix angle, but other techniques must be used when fibres are required in the purely longitudinal direction, as will be the case with Pelamis.

Pultrusion and pull-winding are techniques for producing structural profiles in an entirely automated process. The profiles may be relatively complex, and a high content of unidirectional (UD) fibres is achievable, as well as high fibre volume fraction. The chief disadvantage is the limitation on profile dimensions. Pultruded profiles are, however, available in more or less unlimited lengths.

5.1 Proposed design

The solution proposed for Pelamis is based on a hollow-core sandwich construction, embodying a mix of unidirectional (UD) and $\pm 45^\circ$ biaxial glass in polyester resin, and manufactured primarily by tape winding. The design is shown in Figure 18. The laminate composition achieves low weight by using a 2:1 volume ratio of UD and $\pm 45^\circ$ glass: the UD confers high axial stiffness for bending and buckling strength, with the $\pm 45^\circ$ material to resist shear both direct and due to torsion.

The segment has an effective wall thickness (single-skin equivalent) of 16mm, with equal inner and outer skins of approximately 7mm separated by a 50mm hollow core. The core is stiffened by GRP webs at axial spacing of approximately 180mm, running the full 25m length of the segment. The web wall thickness is 4mm. The laminate composition and sectional dimensions of the GRP composite were based on a number of load analyses, summarised in Table 4.

The resulting design represents the minimum material requirement for strength, elastic stability, and fatigue endurance. The prototype composite tube has an estimated weight of 12.3 tonnes, with an expected weight reduction in final design to 9-10 tonnes (excluding end caps and internal ballasting).
Figure 18. GRP segment shell based on tape-wound inner and outer skins encapsulating a hollow core pre-formed from corrugated GRP sheet.
Table 4. Summary of design loads analysis for the GRP segment.

<table>
<thead>
<tr>
<th>LOAD CASE</th>
<th>DESCRIPTION</th>
<th>Calculated Safety Factor*</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum bending moment</td>
<td>Maximum direct stress in outer skin; load is ALS with safety factor of 1.0, or max. operational load with S.F. of 1.30.</td>
<td>3.3</td>
<td>Direct stress from beam theory.</td>
</tr>
<tr>
<td>Global (elastic) buckling due to max. BM‡</td>
<td>Elastic buckling of thin-walled cylinder under transverse bending; Roark, Item 16. BM as above; cylinder wall properties assumed to be evenly distributed over 62mm thickness.</td>
<td>3.5</td>
<td>Equivalent single-skin cylinder†</td>
</tr>
<tr>
<td>Local (panel) buckling due to max. BM‡</td>
<td>Elastic buckling of long flat panel under longitudinal compression (all edges clamped), Roark Item 1b. Evaluated for panel of 180mm width, 6mm thickness, same BM as above.</td>
<td>1.9</td>
<td>Long panel under axial compression, clamped edges</td>
</tr>
<tr>
<td>Local (web) buckling due to max. BM‡</td>
<td>Elastic buckling of stiffening web, Roark Item 1b. Same analysis as immediately above (including material properties), but panel width 50mm, thickness 4mm.</td>
<td>11</td>
<td>As above</td>
</tr>
<tr>
<td>Maximum shear stress (direct)</td>
<td>Shear force due to differential ram action. Evaluated using DNV Item 2.2.3, for equivalent single wall thickness of 16mm.</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>Shear stress under max. torsional load</td>
<td>Shear stress due to max. torsional load applied by mooring yoke. Analysis according to DNV Item 2.2.2., based on equivalent single wall thickness.</td>
<td>4.0</td>
<td>Highly conservative: this case under review.</td>
</tr>
<tr>
<td>Torsional stability (cylinder elastic buckling)‡</td>
<td>Elastic stability of a long cylinder under pure torsional loading, as per Roark, Item 17a. Distributed material properties are assumed as per global (elastic) buckling above.</td>
<td>1.3</td>
<td>Equivalent single-skin cylinder†</td>
</tr>
<tr>
<td>Fatigue life (bending)</td>
<td>Cumulative damage calculation according to Miner’s Rule, based on the Orkney heave bending moment spectrum.</td>
<td>8.5</td>
<td>Cumulative damage fraction = 0.117, Orkney BM spectrum</td>
</tr>
</tbody>
</table>

* Factor of safety before including material PSF or design fatigue factor DFF, but including load SF
‡ All elastic stability calculations following Roark [11]
† Based on aggregate shell properties; assumes ring stiffeners at 5m intervals (case under review)
5.2 Fibaflo subcontract design

As part of the design process a subcontract was placed with Fibaflo Ltd, a specialist filament-winding company. The proposed design is based strongly on its recommendations, which were submitted in a formal report, and the following is a brief summary of this:

- The preferred material for the primary structure is E-glass fibre in isophthalic polyester resin; this solution is chosen on a cost-performance basis, and for its extensive experience in the marine environment. Epoxy or vinylester resins have higher cost but no great structural advantages.

- The main skins will be applied as a triaxial tape. Although more expensive than pure glass rovings, this achieves the required fibre orientation mix in a single winding operation.

- A profiled section formed from trapezoidal corrugated GRP sheet will provide the hollow core. Foam cores are rejected on the grounds of long-term structural instability, and susceptibility to slamming damage.

- Long-term environmental protection will be achieved by the use of a resin-rich coating incorporating UV inhibitor on the outer surface. It is anticipated that no further coatings will be necessary during the lifetime of the segment.

- The capital cost to set up production in a dedicated facility is estimated as £1.0-1.5M, excluding the building. The cost of the winding mandrel is £120k. Such a facility could manufacture one 25m tube per week with a minimum of 8 production workers. The same equipment could be used to make 2 tubes/week, with a larger workforce.

- Bonding the core panels in place is the most labour-intensive operation, and this issue requires more attention; optimising the process could result in significant cost savings.

An alternative core structure based on pultruded GRP profile was considered at some length, in discussions with commercial pultrusion companies. The concept is shown in Figure 19. Axial stiffness is provided by the high content of UD fibre in the profiles, which are overwound with an outer skin of biaxial glass fibre. The profiles are bonded at their edges with adhesive paste (typically epoxy or methacrylate), with bond to the outer skin made during lamination.

The advantages of the concept are the availability of pultrusion profile the full length of the segment, high dimensional accuracy and fibre volume fraction, and versatility in terms of the fibre orientation. Tooling costs are however high for a profile of reasonable width, which is necessary in the present case to avoid high layup costs or handling. Fibaflo expressed some concern about the long-term stability of the pultrusion and its bond with the outer skin, even though there is now a wide range of applications of pultruded sections in high performance structural use.
5.2 End-cap design

Three potential end-cap attachment methods were identified by Fibaflo, and presented as concept designs in their final report. No detailed loading analysis of the end caps was carried out at this stage. The three options were:

(i) Attachment method 1: the GRP segment is laminated directly to a conventional steel end cap, which is profiled to facilitate tapering of both the core and end cap profiles.

(ii) Attachment method 2: post-tensioned GRP tube. Internal tie bars are used to maintain contact between the GRP segment and steel end cap in all load conditions.

(iii) Composite end cap: a more radical option involving replacement of the cast steel end-cap with a composite equivalent. This option would require RTM or other moulding processes, rather than filament winding.

Sketches of options 1 and 2 are shown in Figure 20. Of the two, Option 1 may be preferable on the grounds of cost although it results in permanent attachment of the segment to the end-cap. Option 2 must bear the cost of a post-tensioning system that must sustain the maximum bending moment, but unlike the equivalent for the concrete segment, is not actually needed to provide tensile capability in the segment wall, and therefore represents a partially redundant cost.

5.3 Life cycle issues

Innumerable examples of GRP boats and small ships point to the suitability of this material in the marine environment. Fibaflo advise that GRP has also been used for stressed pressure pipes for water services for over 30 years, and that samples recovered at various times for evaluation have shown no resin degradation in service conditions. Nonetheless care is required at both the design and manufacturing stages to ensure the long-term environmental stability of GRP and maintenance of initial performance.
Figure 20. Concept designs for end-cap attachment to GRP segment: (top) direct bonding by overlaminating; (bottom) post-tensioned attachment.
The key issues to be addressed are summarised by Shenoi [19] and include:

- Water absorption
- Stress corrosion and fatigue
- Gel coat blistering

The first two of these are generally associated with a loss of strength. Water absorption of 0.5-1.0% by weight under saturated conditions can lead to 20% strength reduction in polyesters after just 6 months immersion. The combination of seawater immersion and a continuous dead load, or fatigue loading, may also cause a reduction in strength in addition to that associated with the normal ‘dry’ fatigue curve. Gel coat blistering is often cosmetic, but this form of surface degradation can lead to increased water absorption by the laminate, and ultimately to stress corrosion.

To take account of the above factors the reference GRP segment is designed in isophthalic polyester resin, which is highly corrosion and seawater resistant, with good mechanical properties. The proposed laminate system incorporates resin-rich layers of C-glass tissue and chopped strand mat on the inner and outer skins to act as a moisture barrier and provide corrosion resistance. The outer surface is pigmented to provide UV and weathering protection. The lack of a gel coat, with continuous layup of the skin laminates, ensures compatibility and minimises the risk of blistering.

In addition the reference design is conservative in regard to fatigue, with a partial safety factor of 1.49 in addition to the endurance curve factor assumed in fatigue calculations, which was based on Germanischer Lloyd recommendations for GRP composites in wind turbine blades [4]. The calculated design fatigue factor (DFF) for the segment equates to a lifetime of 170 years. This is much higher than those for the steel or concrete alternatives, albeit based on the use of a different design code (GL rather than DNV).

The design is therefore robust in regard to fatigue, and remains acceptable even if a higher partial safety factor of 1.79 is assumed to allow for seawater degradation of the laminate. This would result in a predicted lifetime of 27 years, and is equivalent to the primary material losing 20% of its initial strength due to seawater immersion. Results from a historic comparison of GRP samples from the submarine USS Halfbeak suggest this is a severe assumption [19].

The Halfbeak was equipped with a GRP superstructure during a 1953 refit, and remained in service until 1972. Laminate samples were statically tested during construction then again after 11 years’ service. The results indicated essentially negligible change in flexural strength and stiffness, although Barcol hardness had reduced by 5%. In addition the fatigue design of the Pelamis segment is somewhat more conservative than for the submarine, for which the GRP fatigue limit was assumed to be 20-25% of static strength. The maximum skin stress on the Pelamis segment under moment-limiting operation is only 11% of static strength.

Further data from tests on a US Coastguard patrol boat, where GRP samples were analysed after 10 and 20 years in service, also give reassurance [19]. Although in this case no data were available from the initial time of manufacture the laminate showed no degradation between the two sampling intervals. In general it is concluded that the GRP reference design offers a viable solution for the Pelamis segment. Satisfactory long-term performance is expected on the basis of:

- Adequate design strength margins, including fatigue.
- Use of high quality isophthalic polyester resin.
- Attention to laminate design, particularly at the inner and outer surfaces.
- Modern environmental and process control during production.
6 Comparative design issues for preferred options

6.1 Rigidity and deflection

Rigidity in bending was assessed against (a) the design maximum bending moment, and (b) a possibly hypothetical case where the fully-ballasted segment is simply supported at its two ends with its weight uniformly distributed as shown in Figure 21 (it is likely that in practice the segment will only be lifted in the unballasted condition). In any case, the maximum operational bending moment is the more severe design case. The calculated maximum deflection of the segment for the three materials options is given in Table 5.

The reference steel and concrete designs have very similar rigidity and their deflection under the worst case load is almost negligible in regard to the segment length of 25m; the deflection in the end-supported case can safely be disregarded. Under the worst-case load the GRP tube deflection is significantly greater; the deflection corresponds to a strain level below the GL-recommended cracking limit of 0.42% for laminates of this kind, however, and should therefore be acceptable.

The lower rigidity of GRP does not necessarily equate to lower strength, and the calculated deflection of 162mm is still small in the context of the segment length. A GRP wind turbine blade of comparable length would routinely experience a tip deflection several times greater than the above. The greater flexibility of the GRP segment may have implications for the dynamic response of the device, but such an analysis is outside the scope of the present report.

6.2 Resistance to external pressure

The specification for maximum external pressure currently remains under discussion. A worst case, however, assumes immersion to 30m in extreme waves with resulting hydrostatic pressure of 3 bar (300kPa), with re-entry slamming giving a similar figure. The following formula from Roark [11] gives the critical external pressure q causing elastic buckling of a cylinder of length L:

\[
q = 0.807 \frac{E t^2}{L r} \left\{ \left[ \frac{1}{(1 - \delta)^2} \right]^3 (t/r)^2 \right\}^{0.25}
\]

Evaluated for the three options, neglecting internal bulkheads and setting L = 25m, the critical pressure q in each case is given as:

- Steel (25mm): 295 kPa
- Steel (20mm): 169 kPa
- GRP: 110 kPa
- Concrete: 4133 kPa
Table 5. Maximum deflection estimates for different segment constructions.

<table>
<thead>
<tr>
<th>Load case</th>
<th>Maximum deflection</th>
<th>Deflection in mm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Steel</td>
</tr>
<tr>
<td>(a) Design maximum bending moment</td>
<td>$ML^2/8EI$</td>
<td>13</td>
</tr>
<tr>
<td>(b) Fully-ballasted, simply supported at both ends</td>
<td>$5WL^3/384EI$</td>
<td>3.0</td>
</tr>
</tbody>
</table>

The above results imply that internal bulkhead reinforcements would be mandatory for steel and GRP: the 25mm steel wall is marginal in regard to buckling, while the 20mm steel wall and GRP options would have, respectively, only 56% and 37% of the required rigidity without internal reinforcement. Concrete in contrast has inherently high stiffness due to its thicker wall, and shows a high margin of safety against this load case without internal bulkhead reinforcement.

6.3 Weight and ballasting

The displaced weight of the segment is independent of its construction, and the implications for ballasting vary widely depending on the choice of material. This is illustrated by the comparison in Table 6, in which the ballast requirement is shown for two cases assuming 50% and 70% displacement by volume. The latter applies to the prototype, while production versions will potentially float higher in the water. The cost figures are somewhat provisional.

The calculated ballast requirements are regarded as maxima, neglecting allowance for the additional weight of bulkheads, plus any buoyancy adjustment to accommodate end caps or power modules. The results for concrete cover two cases with different wall thickness, both including 9 tonnes to account for reinforcing steel and 4 tonnes for the post-tensioning system. The end-caps and other internal structure are again neglected. The highest ballast requirement, not surprisingly, is for the GRP segment whose unloaded weight is only 10 tonnes.

The issue of ballasting the GRP segment is non-trivial, and the structural implications of such a large distributed weight on the lightweight shell may require to be investigated further. The use of an optimised lightweight structure that requires ballasting to 15 times its dry weight seems somewhat questionable. If other advantages of GRP warrant its use an alternative ballasting design might be considered. One possibility is to subdivide the segment hull into two chambers, the lower of which is essentially open to the sea, with the upper supplying buoyancy. At present, however, this remains only a concept design.

The lowest ballast requirement is for the concrete segment, which could in principle be designed to avoid the need for any ballast save trim. Such a solution might facilitate the design of the post-tensioning system by avoiding space conflicts inside the cylinder, although some degree of positive ballast may always be needed to ensure roll stability. An option here may be to incorporate a weight offset in the concrete section, ie by thickening the wall selectively below the waterline.
Table 6. Estimated ballast weight and cost (based on £50/tonne installed).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Steel (125mm)</th>
<th>Concrete (125mm)</th>
<th>Concrete (160mm)</th>
<th>GRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Segment weight unballasted</td>
<td>54t</td>
<td>99t</td>
<td>121t</td>
<td>10t</td>
</tr>
<tr>
<td>Ballast (50% displacement = 120t)</td>
<td>66t</td>
<td>21t</td>
<td>nil</td>
<td>110t</td>
</tr>
<tr>
<td>Ballast (70% displacement = 168t)</td>
<td>114t</td>
<td>67t</td>
<td>47t</td>
<td>158t</td>
</tr>
<tr>
<td>Ballast cost @ 50% displacement</td>
<td>£3.3k</td>
<td>£1.0k</td>
<td>nil</td>
<td>£5.5k</td>
</tr>
<tr>
<td>Ballast cost @ 70% displacement</td>
<td>£5.7k</td>
<td>£3.4k</td>
<td>£2.4k</td>
<td>£7.9k</td>
</tr>
</tbody>
</table>

6.4 Damage resistance and reparability

The reference steel segment is inherently robust, although thick welds may be susceptible to crack formation under locally high impact. Dents may not greatly affect strength, with the limiting case probably loss of buckling resistance, and Pelamis has a significant factor of safety in this regard. Most damage to the steel structure is reparable, albeit not on-site: repairs must be made to the same standard as the initial fabrication. Suspected minor damage to the steel structure can be surveyed using a variety of NDT methods prior to any repair being deemed necessary.

The GRP segment is probably the least impact-tolerant of the three options. The reference design has good margins on bending and shear, with the equivalent of a 16mm wall of laminate, but the critical issue is local impact resistance. The limiting dimension is the 7mm outer skin thickness, and GRP has relatively little through-thickness strength. Local impact damage is therefore a significant possibility. GRP shows little plastic deformation prior to failure and the condition of ‘barely-visible impact damage’ (BVID) may arise, with interlaminar cracking, resulting in a loss of compressive and fatigue strength [20].

Conversely, however, GRP is the easiest and cheapest material to repair, even when damage is relatively major. Cracked panels can be cut or ground out, and replacement material blended in using the same primary materials. The use of polyester resin as matrix means that external heating is unnecessary to cure the laminate on-site, and repairs can be carried out with fairly minimal site infrastructure. Tooling and materials are inexpensive.

Damage to post-tensioned concrete may comprise minor spalling, through more serious deformation or fracture of the reinforcement mesh, to failure of one or more post-tensioning tendons. The issue is complicated in that the bulk concrete is in compression, and major repairs must be done with the post-tensioning system relaxed, or using materials capable of sustaining tensile loads. Composites offer a good solution, and repair or strengthening of concrete structures using glass or carbon fibre is now relatively commonplace [21]. This would in principle allow close matching of the moduli of original structure and the repair material.

Damage to the post-tensioning strand would be a more major problem and depending on the design might result in rejection of the entire concrete segment. Repairs may then range from relatively straightforward external composite work, similar to that required for the GRP segment, through to major replacement of post-tensioning components. While the likelihood of damage is difficult to assess, however, the concept of a reinforced concrete tube with thick wall seems inherently robust. Table 7 is an attempt to summarise the issues for the three options in a qualitative way.
Table 7. Comparison of damage tolerance and reparability.

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Steel</th>
<th>Post-tensioned concrete</th>
<th>GRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tolerance of minor damage</td>
<td>Good</td>
<td>Good</td>
<td>Poor</td>
</tr>
<tr>
<td>Tolerance of major damage</td>
<td>Fair</td>
<td>Fair</td>
<td>Poor</td>
</tr>
<tr>
<td>Ease of minor repair</td>
<td>Poor</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td>Ease of major repair</td>
<td>Fair</td>
<td>Poor</td>
<td>Good</td>
</tr>
</tbody>
</table>

6.5 Corrosion and marine fouling

Of the materials under consideration steel is the most affected by the seawater environment in regard to corrosion, and a protective surface coating is essential. The steel plate should also include a thickness allowance, and according to DNV guidelines for offshore wind installations the assumed rate of corrosion in the splash zone is 0.3–0.5mm/per year [22]. DNV further recommend that the beneficial effect of an appropriate protection system (such as epoxy paint) may be taken into account on the fatigue life, when selecting the relevant SN-curve.

Based on the above assumptions and a 20-year life the prototype segment wall would have effective (load bearing) thickness of 15mm. Relatively large factors of safety exist on the ULS load case in regard to bending or buckling, and this residual value would still meet the DNV design criteria for ULS failure. For fatigue, compliance with DNV guidelines is assumed by adopting S-N curves based on seawater exposure with cathodic protection [6], assuming an epoxy coating confers a similar level of protection, but calculating on the basis of the reference 25mm wall.

The above approach is believed to be realistic without being too onerous. Modern epoxy coatings can be near-impermeable to seawater, and dramatically reduce the onset of corrosion. Conversely, however, if the paint is damaged an accelerated rate of corrosion may result at the site due to more concentrated electrochemical activity. Any coating therefore needs to be tough, and several epoxy-based systems, some containing glass or ceramic additives, are now on the market.

The coating chosen for the steel prototype is Interzone 954, a 2-component ‘modified’ epoxy designed to give long-term protection in seawater; it is suitable for single coat application, and will continue to cure when immersed in water. Interzone 954 has been used extensively in the North Sea offshore sector since at least 1984, including under deck and splash-zone applications. It does not contain glass flake, which improves abrasion resistance, but at a cost deemed unjustified in the present application.

Neither the GRP nor concrete segments in principle require any external coating for corrosion protection. In the case of GRP a pigmented resin-rich outer layer (not a gel coat) is incorporated in the laminate, and is designed to give lifetime protection against water ingress and its effects (see above). With concrete the outer layer cover is designed to protect the reinforcement against corrosion for the lifetime of the device; as noted earlier the option of cathodic protection of the reinforcing bar has been rejected as unnecessary.
Pelamis will be susceptible to marine growth, but the operational effects are not expected to be significant, or to warrant the widespread use of antifouling paint. The two main effects of fouling would be to increase skin friction and decrease buoyancy. The first of these results in increased viscous drag, but due to the low surface velocity of Pelamis – whose power extraction is largely driven by hydrostatic rather than viscous forces – no significant loss in performance is expected. The results of numerical modelling and 20th-scale tank tests indicate that power losses would be relatively low.

The change in buoyancy due to marine growth could potentially affect performance, but again the effect is not expected to be significant. Soft plant growth (seaweed) has a specific gravity similar to water, so in principle will not affect buoyancy even when present in large quantities. Hard-shelled organisms such as mussels and barnacles have higher density, however, and the weight of mussels has in some cases been known to sink buoys and navigational aids [23].

Assuming a 50mm layer of shelled marine growth over the entire lower surface, the change in displaced weight is unlikely to exceed around 5% of the nominal value. This should not greatly affect performance. It is also anticipated that marine growth on the main surfaces will build up in quiet months and be stripped off during storms, usually the case in the splash zone. Currently it is assumed that the segment shell may require to be cleaned every 2-4 years.

Should antifouling protection prove to be needed, there are numerous commercial solutions. Broadly speaking they can be divided into those containing biocidal agents that leach into the water, and more recent biocide-free products that utilise smooth surface texture to deter attachment. The biocidal agents are unattractive for environmental reasons, but also have a finite life of eg. 6-8 years, necessitating either a marine growth removal programme or scheduled re-coating.

Biocide free solutions such as Intersleek from Akzo Nobel remain effective for the design life of the segment, and would therefore minimise offshore maintenance costs. These coatings are, however, expensive: once surface preparation and application are accounted for the all-up cost of antifouling is expected to exceed £10k per segment, based on industrial sources [24]. It is clear that antifouling protection should be specified only if there is a compelling economic case, ie if the cost of lifetime protection is competitive with that of periodic fouling removal.

**6.6 Environmental impact of manufacture**

Generally speaking steel and fibre-reinforced composites have relatively high manufacturing and energy costs while those of concrete are low [25]. In the case of the Pelamis segment this difference is reduced by the need for steel reinforcement and post-tensioning strand in the concrete tube. An approximate estimate of energy content for the three candidate materials was made on the basis of data from the literature, taking energy and CO$_2$ content from O’Callaghan [26] and Ashby [25].

The materials data and overall energy content are summarised in Table 8. Note that the CO$_2$ figures account only for energy use in manufacture, and do not include any CO$_2$ released in chemical processes. The resulting totals indicate that steel is the greatest ‘polluter’ and that use of either of the alternative materials would at least halve the energy content of the segment structure. The best material from this viewpoint is GRP, with only 38% of the energy content of steel.
Table 8. Energy cost of segment materials options (excluding end caps).

<table>
<thead>
<tr>
<th>Construction material</th>
<th>Energy cost MJ/kg</th>
<th>CO₂ released (kg/kg)</th>
<th>DESIGN OPTION: material content of each option (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Steel</td>
</tr>
<tr>
<td>Steel</td>
<td>50</td>
<td>4.3</td>
<td>54000</td>
</tr>
<tr>
<td>Concrete</td>
<td>6</td>
<td>0.52</td>
<td>0</td>
</tr>
<tr>
<td>Glass fibre</td>
<td>51</td>
<td>4.4</td>
<td>0</td>
</tr>
<tr>
<td>Polymer resin</td>
<td>200</td>
<td>17.2</td>
<td>0</td>
</tr>
<tr>
<td>Total weight of segment (kg)</td>
<td>54000</td>
<td></td>
<td>99200</td>
</tr>
<tr>
<td>Total energy cost of segment (MWh)</td>
<td>750</td>
<td></td>
<td>322</td>
</tr>
<tr>
<td>CO₂ released (tonnes)</td>
<td>232</td>
<td></td>
<td>100</td>
</tr>
</tbody>
</table>

The energy payback of Pelamis is also important. Assuming the total energy content of the machine is around 6 times that of a single segment, the reference 750kW steel design incurs a usage of 4500 MWh in manufacture. Operating at 40% capacity factor, the energy payback period is then 1.7 years. If concrete or GRP were to be used instead, in conjunction with design improvements, a 50% reduction of energy content is conceivable, bringing the return period down to 10 months. This figure is comparable with the energy payback for onshore wind turbines, which (including foundations etc) is estimated to be in the range 3-10 months [27,28].

6.7 Disposal and recycling

In principle all three of the main materials considered may be recycled. Their relative order of merit in terms of economic usefulness is steel > concrete > GRP. Steel can be fully recycled and the technology and market for recycled product are both well established. The current average value for US scrap steel is around $85/ton [29], which equates to approximately 15% of its value new. This is a significant incentive to recycle, and if anything likely to increase with future trends in environmental legislation.

The thermosetting resins used in GRP manufacture make the finished composite more difficult to recycle, although unwanted GRP is now being used in ground form as a filler for new composites based on both thermoplastic and thermoset resin systems [30,31], in the latter case including pultrusions. While these applications are in their infancy compared with the steel recycling industry, they offer promising long-term solutions to the economic disposal of GRP.

High grade concrete cannot generally be recycled into an equivalent load-bearing application, and the energy used to make it in the first place is effectively lost. There are, however, a large and growing number of applications for reclaimed concrete in a crushed state, including base material for roads and foundations, cycle paths, landscaping material, drainage material for underground pipes, or aggregate in new asphalt or concrete.

All three materials are essentially non-polluting. Steel is the most biodegradable, however, while concrete and GRP may remain intact for very long periods. The fact that none is toxic would allow for disposal at sea which, although seemingly unwelcome, could in fact have potentially positive environmental benefits. Steel or concrete segments may be used to form breakwaters, and all three
materials potentially used to form artificial reefs to create fish breeding grounds; in the case of GRP ballasting may be necessary.

This last application is now becoming widespread and a good precedent is the reef at Pokai Bay, Oahu, formed from disused concrete pipes (Figure 22). Similar examples include the major artificial reef in Loch Linnhe off the west coast of Scotland, which uses concrete blocks [32]. There are numerous accounts of increased marine life and biodiversity in the vicinity of sunken ships. Structures of this kind may become increasingly important to regenerate depleted fish stocks, and disposal of used Pelamis segments at sea may well be the most viable option.

Figure 22. Artificial reef made using concrete pipe, Oahu; note the shoal of fish (photograph: NOAA Central Library).
7 Overall technical and cost comparison

7.1 Comparative costs

In the initial materials screening process described above budget costs were assumed for the different options, based on assumptions of the likely achievable costs in volume production. These figures were used in conjunction with the calculated minimum material requirement to indicate relative viability. The cost figures assumed for finished product were as follows:

- Steel (uncoated): £0.80/kg
- GRP: £5.0/kg
- Wood-laminate: £5.0/kg

These figures are relatively competitive. The cost of steel is based on budget figures available from the volume fabrication industry for products including wind turbine towers, and information gleaned from the prototype design. The GRP cost is around 66% of the current selling price for large wind turbine blades and takes account of the simpler shape of Pelamis, and use of lower-cost resins (polyester rather than epoxy).

The cost of wood-laminate was assumed to be similar to that of GRP, again broadly based on wind industry data, and the figure of £5/kg is believed to be very competitive for this technology. It was on this basis, however, that wood laminate was ultimately rejected. The minimum wall thickness calculated to achieve necessary fatigue resistance is 49mm, leading to a segment weight of approximately 9.4 tonnes and budget cost of £47k. It is difficult to see how this technology would approach the economics of GRP or concrete.

A simple budget cost was not attributed to post-tensioned concrete due to the difficulty in apportioning costs to the individual components. Instead a budget cost was obtained for the complete segment under the detailed subcontract study by Arup. Cost comparisons for this and the other remaining candidate materials were then made on a more detailed basis, taking account of specific design details and additional factors such as surface coating, and the knock-on implications of the segment design on that of the steel end caps.

Arup concluded that in volume production (100+ units) a concrete segment cost of £30k should be achieved, while a one-off prototype would cost £47k. The mass-production cost assumes 5-time re-use of formwork, which is a dominant cost in the equation. In addition Arup did not include the cost of providing and equipping a facility for production, and this assumption is continued here. Using similar assumptions the short consultancy by McAlpine arrived at a very similar result.

The cost of the GRP segment is based on the subcontract work of Fibaflo, with some adjustments in regard to optimal design. Fibaflo concluded that in mass production a GRP sandwich construction of 12.3 tonne weight would cost £40k, assuming manufacture in a suitably equipped facility. The above weight appears to be conservative, and a 10-tonne tube should have sufficient margins on loading and fatigue; assuming the same finished material price the cost would then be £32.5k.

The comparative cost of the reference steel design is not trivial to predict, despite the existence now of the full-scale prototype. It was always accepted that significant cost savings would be made from the prototype, and that issues such as the number and orientation of welds can have a high cumulative impact on the total cost. The cost of the prototype segment is therefore not used here for comparison, and instead a budget price of £800/tonne is adhered to, giving a cost of £42.9k for a 25mm-wall segment, or £34.3k for a 20mm-wall design (in both cases before surface coating). The latter is used below.
Although detailed design of the end caps is not covered by the present study, the choice of material used for the segment has structural and cost implications for the end caps. One finding was that the use of a concrete tube with inherently thick wall facilitates a simpler end cap design than with the reference steel segment: in the former case there is no need for a long transition from the heavily point-loaded area to the thin steel wall. The weight saving in the end cap may then be up to 50%, although again it is to be stressed that the prototype is not representative of an optimal steel design.

### 7.2 Overall technical comparison

The overall merits of the steel, GRP, and post-tensioned concrete segments are compared in the summary table shown below (Table 9, also reproduced in the Executive Summary). The comparison is semi-qualitative with no weighting applied to any of the listed criteria. In this regard it may seem questionable to equate the relative costs of the three options and eg their ease of final disposal at sea, or cost to provide corrosion protection. It may be noted, however, that the segment first cost is relatively similar in the three cases, and bracketed by a figure of £32.2k ± 7%. The relative cost difference is of a low order, and possibly less important than some of the other criteria considered.

It is also to be stressed that the cost figures assume long-term volume production, and do not include the cost of end-caps, bulkheads (if necessary), ballasting, or attachment details. The figures for the steel design do not reflect the costs of the prototype, and remain based on a budget figure of £800/tonne with a 20mm wall. This is believed to be appropriate for long-term production. Assuming the above figure is achieved in practice, the cost of the segment would be approximately halved in comparison with the prototype.

Other items in the table may be open to debate, eg the comparative reparability of the steel and concrete options, which was judged to be on balance equal. This was based on the premise that minor damage to concrete, not affecting the post-tensioning system, should be easier and cheaper to repair than equivalent damage to the thin-walled steel tube, whereas the reverse would be true for major repairs (see above Section 6.4). Again the semi-qualitative nature of the table should be stressed.

As noted in more detail in the following section, this study found that post-tensioned concrete offers a superior solution to steel or GRP for the Pelamis segment, as exemplified by its higher overall score in the table. The other two options are adjudged to have relatively equal merit.
Table 9. Summary table: qualitative ranking of the three preferred design options.

<table>
<thead>
<tr>
<th>CRITERION</th>
<th>Thin-walled steel tube (20mm)</th>
<th>GRP sandwich construction</th>
<th>Post-tensioned concrete tube</th>
</tr>
</thead>
<tbody>
<tr>
<td>Segment cost in volume manufacture</td>
<td>£34.3k 1</td>
<td>£32.5k 2</td>
<td>£30k 3</td>
</tr>
<tr>
<td>Surface coating cost</td>
<td>£13.7k 1</td>
<td>Included 2</td>
<td>Included 2</td>
</tr>
<tr>
<td>Fatigue capability</td>
<td>Worst 1</td>
<td>Best 3</td>
<td>Middle 2</td>
</tr>
<tr>
<td>End-cap design</td>
<td>Constrained 2</td>
<td>Uncertain 1</td>
<td>Best 3</td>
</tr>
<tr>
<td>Antifouling protection</td>
<td>No 1</td>
<td>No 1</td>
<td>No 1</td>
</tr>
<tr>
<td>Bending rigidity</td>
<td>Excellent 2</td>
<td>Good 1</td>
<td>Excellent 2</td>
</tr>
<tr>
<td>Immersion buckling rigidity</td>
<td>Poor 1</td>
<td>Poor 1</td>
<td>Excellent 3</td>
</tr>
<tr>
<td>Ballast requirement as ratio of dry weight</td>
<td>1.8–3.0 2</td>
<td>11–16 1</td>
<td>0.2–0.7 3</td>
</tr>
<tr>
<td>Handling eg. by road</td>
<td>Difficult 2</td>
<td>Easy 3</td>
<td>v. Difficult 1</td>
</tr>
<tr>
<td>Damage resistance</td>
<td>Good 2</td>
<td>Poor 1</td>
<td>Good 2</td>
</tr>
<tr>
<td>Reparability</td>
<td>Fair 1</td>
<td>Good 2</td>
<td>Fair 1</td>
</tr>
<tr>
<td>Energy &amp; CO₂ content</td>
<td>Worst 1</td>
<td>Best 3</td>
<td>Middle 2</td>
</tr>
<tr>
<td>Recycling capability</td>
<td>Good 3</td>
<td>Limited 1</td>
<td>Fair 2</td>
</tr>
<tr>
<td>Disposal at sea</td>
<td>Good 2</td>
<td>Fair 1</td>
<td>Good 2</td>
</tr>
<tr>
<td>Choice of manufacturing location (existing)</td>
<td>Many 2</td>
<td>Limited 1</td>
<td>Many 2</td>
</tr>
<tr>
<td>TOTAL</td>
<td>24</td>
<td>24</td>
<td>31</td>
</tr>
</tbody>
</table>
8 Conclusions and recommendations

The study considered a number of potential materials options for the Pelamis segment, with a view to cost reduction in future commercial production. The most promising options were found to be:

- Optimal steel design with 20mm wall
- GRP sandwich construction
- Post-tensioned concrete

All three of the above were found to be potentially economic, with production costs (without surface coating) estimated in the range £30-34k, approximately half that of the prototype steel segment. Of the three, however, post-tensioned concrete offers the greatest number of advantages. The most significant of these are:

(i) Lowest production cost.
(ii) High rigidity, in particular under extreme hydrostatic loading.
(iii) Minimal (and potentially zero) ballasting requirement.
(iv) Facilitating a simpler end cap design.
(v) No structural requirement for surface coating.

Compared with steel, concrete promises to yield secondary cost reductions because (a) internal bulkhead reinforcement is no longer necessary for purposes of rigidity, and may be dispensed with pending re-assessment of its function for flooding protection, and (b) the cost of the end-caps may be significantly reduced due to the inherently better interface between the thick concrete wall and the steel parts. In addition concrete has superior long-term performance in the marine environment.

Although the GRP segment has a number of attractive features, including good long-term performance, ease of manufacture, and high fatigue resistance, there is more uncertainty regarding its use. GRP would represent a higher risk in regard to the extreme immersion and accidental impact cases. There also remain unknowns regarding the ease with which an end-cap interface may be designed, and the effect of the lower rigidity of GRP on the structural dynamics of the overall device.

An optimal steel design based on 20mm wall thickness would be economically superior to the prototype, but would retain some of its disadvantages. Internal bulkheads would be mandatory to achieve rigidity in the extreme immersion case, and the importance of corrosion protection would become even greater than for the prototype. The design of a 20mm wall segment would also be non-trivial in regard to limiting the fatigue stress levels in critical welds, and the use of circumferential welds may have to be minimised or eliminated.

On balance, the post-tensioned concrete segment is found to be the best solution for future commercial Pelamis. It is recommended that further work be directed towards this end, and the present study has highlighted a number of key steps required to verify the concept. In particular the following are recommended:

- Further research and verification of fatigue rules for post-tensioning strand under high preload.
- Full design exercise for end-caps suitable to interface to a concrete segment.
- Re-assessment of the requirement for internal bulkheads.
- A full-scale fatigue test if and when a prototype concrete segment is built.
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[29] Recycler’s World website (www.recycle.net); online market prices for bulk recycled materials.


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