The Detailed Design, Manufacture and Commissioning of a Prototype WOSP Wind/Wave Energy Plant

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2. The Project Partners

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3. Project Objectives

The objectives of the project were:

a) To install a VESTAS V39 500kW wind turbine to the prototype OSPREY Oscillating Water Column type wave energy converter at Dounreay in Caithness, Scotland for the purposes of validating, at the full scale the combined energy factor, structural loads and economic forecasts made on the basis of theoretical and model studies.

b) To obtain data on its operational performance taking advantage of near shore wind energy.

c) To provide a platform to permit the testing and monitoring of wind/wave pneumatic turbines and control systems and to obtain environmental and structural loading data under real sea conditions.

The prime project objective was the installation of a Vestas V39, 500kW wind turbine on a 2MW wave energy collector in order to amortise the structural cost between the two generation systems both to reduce overall cost and to improve the continuity of the output of the combined unit. It was anticipated that the successful commissioning of the prototype would demonstrate the economic production of this form of renewable electrical power leading to the installation of arrays of similar devices.

Installed WOSP units would have minimal detrimental impact on the environment and offer positive benefits to marine life by acting as artificial reefs promoting benthic growth.

The technology is suitable for local construction and since it would be applied in areas of high wind and wave activity, which are typically the more remote areas of the European Union, it would stimulate the local economy of such places. The technology also has a major export potential particularly in energy deficient island communities.

4. Technical Performance

4.1 Summary

The WOSP programme was designed as a development of the previous OSPREY project to install a near shore wave energy collector near to the coast off Dounreay in northern Scotland. The OSPREY collector was to provide a base on which to mount the VESTAS wind turbine. The OSPREY structure was however lost shortly after placement and was thus not available for the WOSP project. In order to provide an alternative, base studies have been performed into the design of a replacement unit but ultimately the available resources proved inadequate to permit manufacture and deployment. In consequence the OSPREY project was terminated and in respect of the non availability of the OSPREY wave energy device to act as a base structure for the wind turbine it was not possible to deploy the wind turbine as planned. Despite this a large body of preparatory work was performed which will be of great benefit to workers following in the field.

The major thrust of technical progress was directed towards developing a structure capable of adequately supporting the wind turbine whilst providing a suitable form to act as an oscillating water column wave energy generator. A design study was performed on behalf of the project team by Sir Robert McAlpine and the proposed form extensively tested in the wave tank to determine impressed loads, wave energy capture performance and flotation stability with particular reference to the acceleration loads suffered by the wind turbine assembly during transport. In addition to the work performed on the device structure a parallel study into the wind climate at the nominated site at Dounreay was performed and preparations were made to ready the site for the installation of the WOSP unit.

4.2 Review of Competing Technologies

The WOSP programme was unique in its objective of beneficially combining wind and wave power generation in a single near shore platform. Whilst land based wind turbines are becoming ever increasingly commercially attractive to the point where the demand for suitable sites is generating an environmental backlash, the development of offshore wind is inhibited by the high cost of the installation. The project participants remain firm in their belief that the amortisation of the cost of a near shore structure between a combined wind and wave generation facility will result in sufficient cost savings in relation to either stand alone offshore wind or wave to make the hybrid facility economically attractive in the medium term.

4.3 General Description

The WOSP collector consists of a concrete wave gathering structure supporting two twin wave powered air turbine and a Vestas V39, 0.5MW wind turbine. The combined nameplate capacity of the wave energy generators is 2 MW.

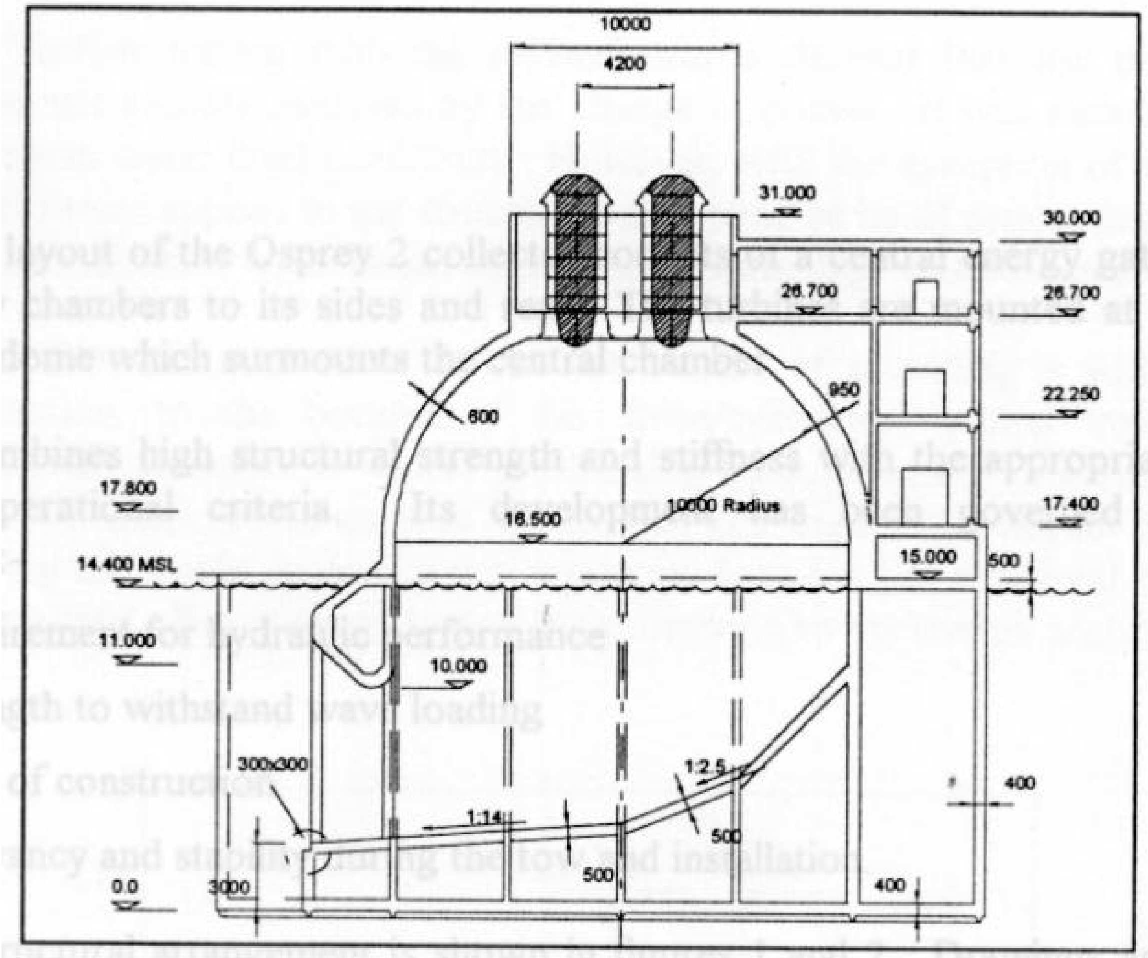

Figure 1. Section through WOSP Structure

The layout of the main Structure (excluding the wind turbine) is as shown in figures 1 and 2. The wave energy collector consists of a central energy gathering chamber with buoyancy chambers to its sides and rear. The wave energy turbines are mounted at the crown of a hemispherical dome which surmounts the central chamber. A secondary structure mounted at the rear of the dome provides accommodation for controls and transformers and also serves as a platform for the wind turbine unit.

The layout combines a high structural strength with the appropriate geometry to satisfy the operational criteria. Its development was governed by four main considerations:-

i) Requirements of Hydraulic performance.
ii) Strength to withstand wave loads.

iii) Ease of construction.
iv) Buoyancy and stability during tow and installation.

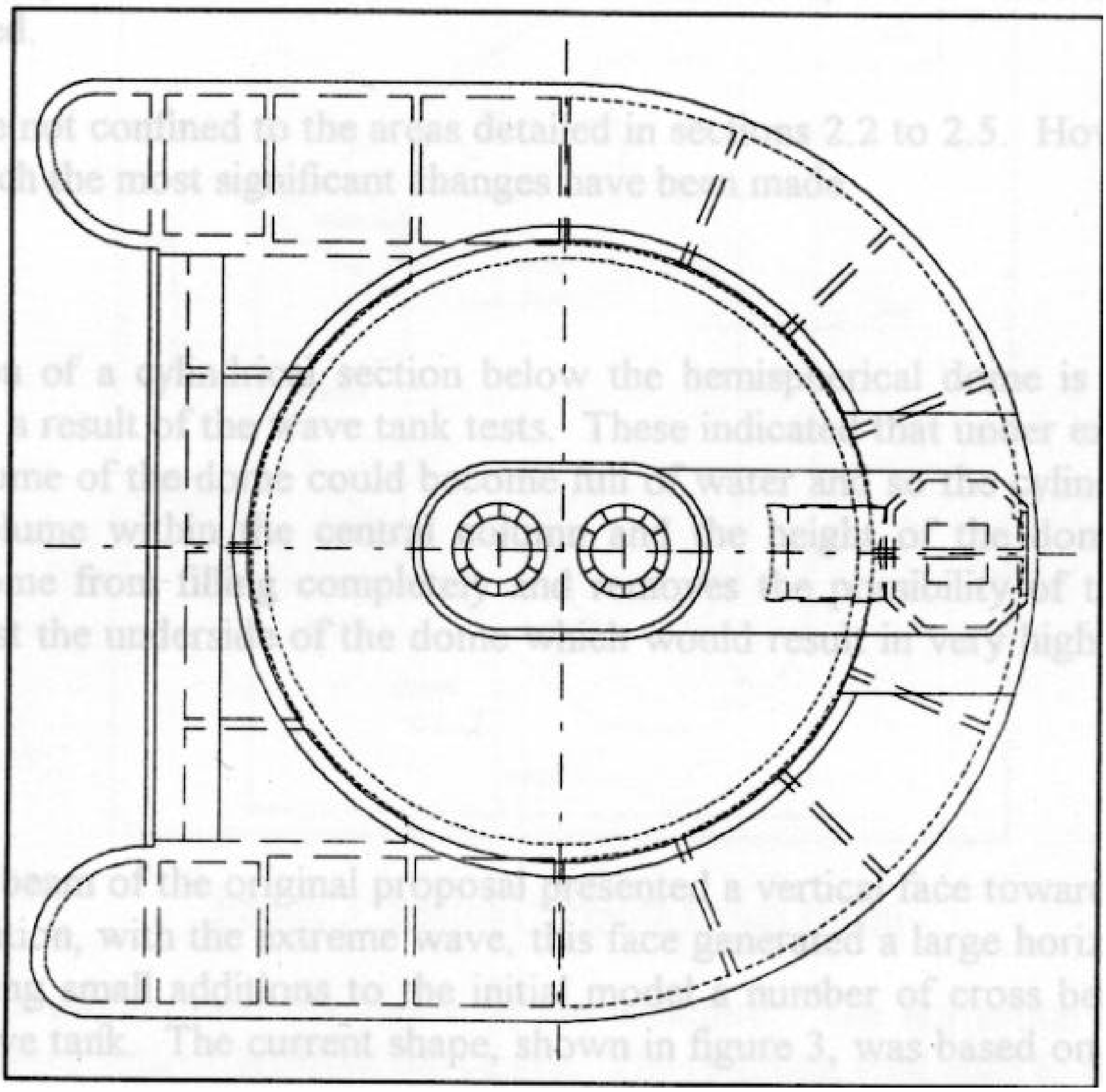

Figure 2. Plan view of WOSP Structure

The horseshoe form comprising the side and rear walls is compartmentalised to provide buoyancy during float out and ballast space when on station. The base of the unit is similarly compartmentalised and also ramped upwards towards the rear of the collector chamber to facilitate water entry and egress.

At the upper boundary of the front opening is a massive cross beam which is included to provide the necessary support for the dome. It was discovered during tank testing that the shape of this beam was highly influential is determining the propensity for wave slam on the beam and subsequently the peak impact loads on the structure.

A short cylindrical section is incorporated below the dome to ensure that even in the most severe of conditions the dome never fully fills with water. This is necessary to prevent a sudden arrestation of the rising water column with the associated water hammer on both the turbine structures and the internal surfaces of the water column.

The dome of the collector is pre-stressed to contain crack growth and to ensure stability of the concrete subjected to the fluctuating wave loads. A tower is provided at the rear of the horseshoe base to act as a support for the wind turbine. A secondary application for this tower is to provide on-board accommodation for equipment. A bridge between this tower and the turbine/generator units mounted on the top of the dome provides access to the turbine/generators and additional equipment space.

The design of the front entrance to the water column permits the fitment of a temporary gate which is required to give adequate buoyancy for float out.

4.4 Wind Turbine.

The wind induced loads on the wind turbine assembly are relatively small compared to the wave induced loads on the collector structure and as such the size of the wind turbine has a relatively small influence upon the strength requirements of the collector. In order to optimise the energy capture from the WOSP unit it is desirable that the largest possible wind turbine should be fitted. In the current state of development this would be a machine of around 63m diameter with a rating of 1.5MW (Vestas V63). Whilst such machines are operating satisfactorily on land they are relatively new and are not available in a marinised suitable form. By contrast Vestas are already using their 500kW, V39 design in marinised form in offshore wind farms in Denmark and the Netherlands and it was decided that whilst a preliminary investigation should be made to ensure that in future the base could accommodate the V63 wind turbine the basic project objectives should remain centred upon the 500kW machine. The specification of the V39 is attached as Appendix 1.

4.5 Evaluation of the Design

4.5.1 Wave Loading on the Structure.

Model testing was performed at a scale of 50:1 in the wave tank of Applied Research & Technology Ltd. in Inverness. The tank is 18.33m long x 2.4 m wide and is fitted with an active full width paddle able to produce both regular and irregular waves to a programmed spectral distribution. The wave environment at site had previously been assessed by ABP Research Ltd. as part of the JOULE contract JOU2-CT94-0283 who estimated the extreme wave heights at representative water depths to be as shown in Table 1.

The test model was mounted on a calibrated load table which recorded the vertical and horizontal loads on the unit together with the overturning moment. Figure 3 shows the model in the test tank during irregular wave testing. A flexible tube connects the top of the model to an air reservoir mounted on the tank cross walkway. This reservoir offers some compensation for the effects air compressibility on the test data.

A detailed examination of the effects of compressibility on scaling leads to the conclusion that it is not possible to simultaneously model both the chamber pressure and the water column excursion in that the required compensating volume to give correct modelling varies with the instantaneous volume of air within the collector chamber (see appendix 2). If however the water column excursions are small then the addition of a compensating volume equal to the scale factor times the mean chamber volume gives a good approximation; this is the procedure normally adopted when measuring energy capture in average seas. For large water column excursions as occur in extreme seas the errors become very significant and may underestimate positive chamber pressures by 10-20% whilst overestimating negative pressure by a large factor, sometimes exceeding 300%

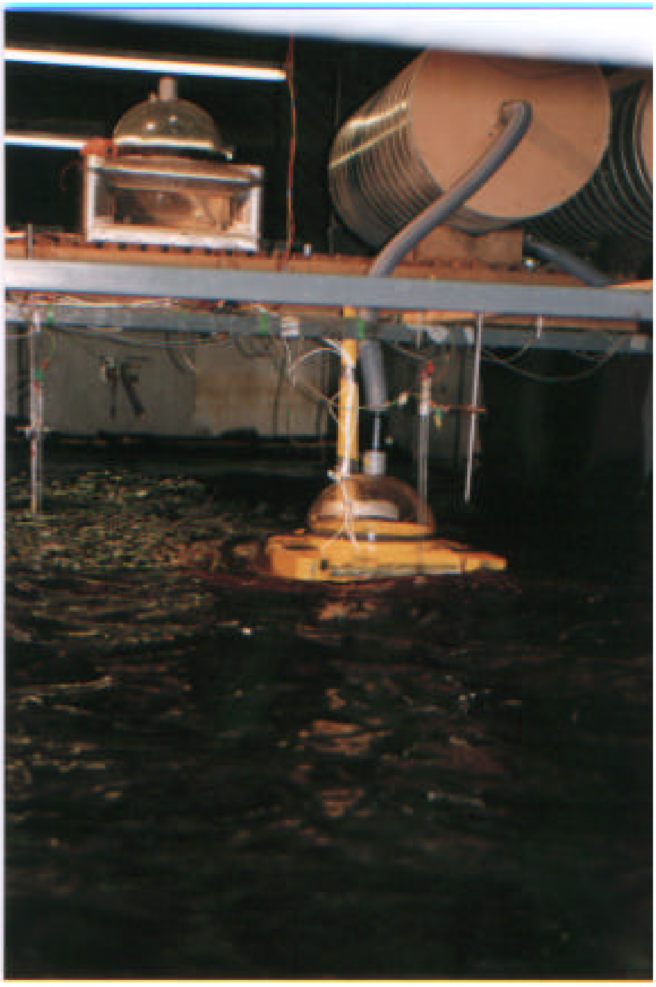

Figure 3. WOSP Model testing under Irregular Waves

The objective of the load measurements was to determine the aggregate wave forces on the structure both to permit a general assessment of the overall structural viability of the device and to establish the anchor loads. These are necessary to permit the foundation design to proceed. To permit a detailed examination of the structure the distribution of the overall wave forces must be assessed and to facilitate this a series of surface pressure measurements will be made at a later stage in the programme. The load data was captured at a scanning rate of 32Hz using LabTech Notebook software via a CIO-DAS 16 A/D board and stored on computer for subsequent analysis. This scanning rate was less than ideal in that the peak wave slam events are typically represented by no more than three data points and as such are not well described. Noting that the load table output derives from strain gauges and that it is thus more strictly a displacement sensing device rather than a direct load sensor the load sensed is also a function of the dynamic response of the load table and as such care must be taken in interpreting the results . Considerations in respect of load table dynamics are discussed in Appendix 3.

Tidal Status		**Water Depth at Site (m)**	**Extreme Value of Hs (m)**
Description	**Abbreviation**		
50 Year High Tide & Positive Storm Surge	MAX	17.8	9.19
Mean High Water Springs	MHWS	16.5	8.98
Mean High Water Neaps	MHWN	15.2	8.75
Mean Sea Level	MSL	14.4	8.6
Mean Low Water Neaps	MLWN	13.6	8.44
Mean Low Water Springs	MLWS	12.3	8.18
Lowest Astronomical Tide	LAT	11.5	8.00
50 Year Low Tide & Negative Storm Surge	MIN	11.0	7.89

Table 1. Extreme Significant Wave Height at Dounreay site for Representative Water Depths

Frequency	**Spectral Density**
0.0348	1.2589
0.0374	4.9367
0.0400	12.3845
0.0426	23.8416
0.0461	53.9857
0.087	114.8925
0.0504	179.6721
0.0522	226.8080
0.0550	196.7600
0.0580	123.9041
0.0610	83.5844
0.0636	69.6520
0.0751	51.9500
0.0980	28.2954
0.1324	8.9330
0.1668	3.0977
0.2928	0.2062

Table 2. Power Spectral Density of Extreme Spectrum at Dounreay Site at Maximum Water Depth.

The power spectral density of the extreme spectrum at MAX water depth was estimated as per Table 2 and this was used to reproduce the extreme spectrum in the test wave tank.

To give an indication of the model performance in irregular sea states other than the extreme the wave maker was also calibrated to provide Bretschneider Spectra of T_p 17 seconds and H_s 6m and 3m.

The wave induced loading on the structure is dependent upon the hydraulic flow in and around the unit and this flow is in turn modified by the operational state of the WOSP device, in particular the resistance to air flow from the oscillating water column offered by the wave energy turbines. Three turbine conditions were considered:

a) Normal running condition.
b) Turbines stopped but airflow continuing through the turbine.
c) Inlet valves to turbines closed to prevent air escape from water column chamber.

Tests on a one fifth scale turbine model at ART showed that because of turbulence generated by the flow of air over a stalled blade the pressure flow characteristic through the turbine assembly was remarkably similar whether or not the turbine was running. For this reason only two wave turbine exhaust conditions were used during testing, referenced as WORKING and CLOSED. The tests were initially performed at three different water depths corresponding to MAX, MSL and LAT. Additional tests were subsequently made at MLWN and MLWS.

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Figure 4 Peak Horizontal Force of WOSP Collector in Wave Direction

In addition to the irregular wave loading measurements a series of tests was performed using regular wave of 2.5m and 5m height with periods ranging from 7-19 seconds in two second intervals. A further series of tests were performed with the water column entry blocked to simulated the condition which will occur immediately after touch down at site when the temporary gate for buoyancy purposes will be in position. The results are detailed in Appendix 4.

Summary results for the extreme wave condition are shown in figures 4-9.

Figure 4 shows the influence of water depth and wave turbine exhaust condition upon the peak wave induced load. In the closed condition where there is a relatively small flow of water inside the oscillating water column chamber, a major proportion of the wave induced loading derives from the flow of water around the structure. Under these conditions the wave induced load is relatively insensitive to water depth but decreases slightly as depth limitation reduces the maximum wave height. Visual observations during testing indicate that at maximum water depth breaking waves broke around the dome of the structure rather than on the structure. As the water depth decreased however the wave increasingly broke at the entrance to the water column and created large impact forces either on the front cross beam or on the inside rear wall of the collector chamber. In consequence the peak recorded force increased as the water depth fell. The closing of the wave turbine valves to inhibit the flow of air into or out of the collector chamber also restricts the flow of water through the mouth of the collector therefore reducing the water velocities at the entrance to the collector and limiting the severity of wave impacts. In consequence the CLOSED condition leads to lower wave induced forces than the WORKING state.

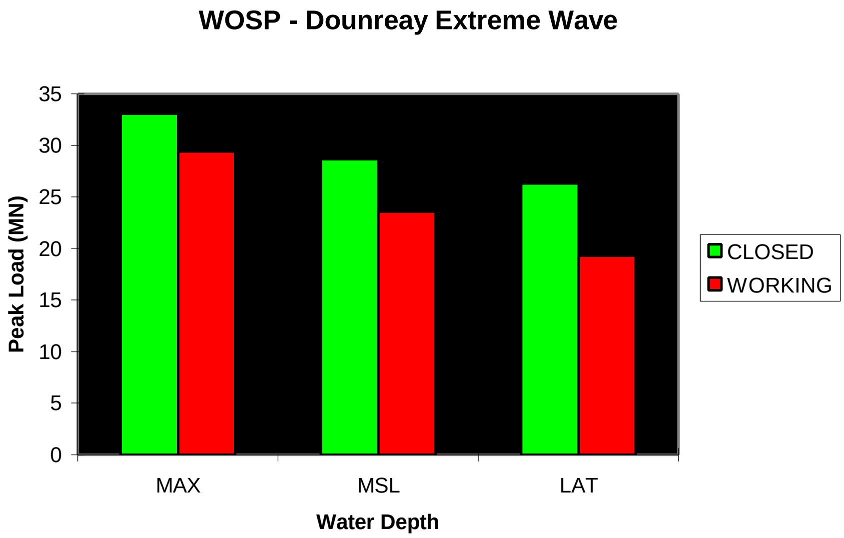

Figure 5. Peak Horizontal Force on WOSP Collector in reverse wave direction

The wave induced forces on the collector structure and anchor are not unidirectional and very significant forces are imposed upon the collector by the reversed flow of the wave (figure 5) In this instance there is no wave slam and as such the major proportion of the loads derived directly from the bulk flow of water around the structure. In consequence since the maximum wave height falls with water depth there is a corresponding fall in the peak horizontal force. Whilst the WORKING turbines encourage flow into the collector chamber during the forward part of the wave cycle thereby increasing the wave induced load, this same action takes energy from the wave, reducing the height of the back wave with a consequential fall in the level of wave induced force during the back flow. This effect is clearly seen in figure 5.

The relative magnitudes of the peak loads suffered during the forward and reverse sections of the wave cycle highlight the function of the structure as a wave energy collector. This comparison is made in figure 6.

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Figure 6. Relative Peak Horizontal Loads in Forward and Reverse Wave Directions

Save where the forward loads are dominated by the effects of wave slam the reverse load is typically between 30% and 45% of the forward load. This observation is very much in line with other observations, notably those of Jayakumar et al (1) who recorded that the wave force on a rectangular oscillating water column structure attracted more than twice the wave force as a rectangular caisson of the same size (represented in this instance by the reverse face of the WOSP structure).

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Figure 7. Peak Vertical Down force during Wave Passage

In addition to the horizontal loading the WOSP structure also suffers vertical forces and overturning moments. The peak down force and uplift are plotted in figures 7 and 8. Whilst down force is beneficial to gravity anchoring, uplift is highly detrimental both to a gravity anchor (loss of friction) or to a grouted anchor (placing grout in tension) and care has to be taken to ensure than the uplift does not exceed the buoyant weight of the structure.

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Figure 8. Peak Vertical Uplift during the Wave Passage

Examination of the time traces of the load signals shows that in the absence of slam the vertical force trace typically lags the horizontal force by 90^{o} so that at the time of the peak horizontal force resulting from the body flow of water the vertical forces are broadly neutral. In the case of wave slam the relative phasing of the vertical and horizontal forces is somewhat chaotic and whilst at times the down force might be beneficial in helping the anchor system it is not possible to guarantee this situation. The traces of overturning moment followed closely the form of the horizontal force.

It is not only important to establish the service loads but also the forces which the structure is likely to suffer during transport and installation. To that end measurements of the aggregate load were made with a front entry gate in place representing the situation just after placement of the device but before the completion of the installation. A

Bretschneider Spectrum with Hs=3m and Tp =17seconds was taken as representative and the measured peak horizontal force was 22MN (figure 9).

The load measurements proved satisfactory in that they provided an initial basis of design for the structure and anchor but it was concluded that additional measurements were required at intermediate water depths to ensure that the worst case slam conditions had been encountered. It was also confirmed that surface pressure measurements were necessary to provide an indication of the distribution of the aggregate load on the structure and to give a corroboration of the magnitude of the extreme wave forces.

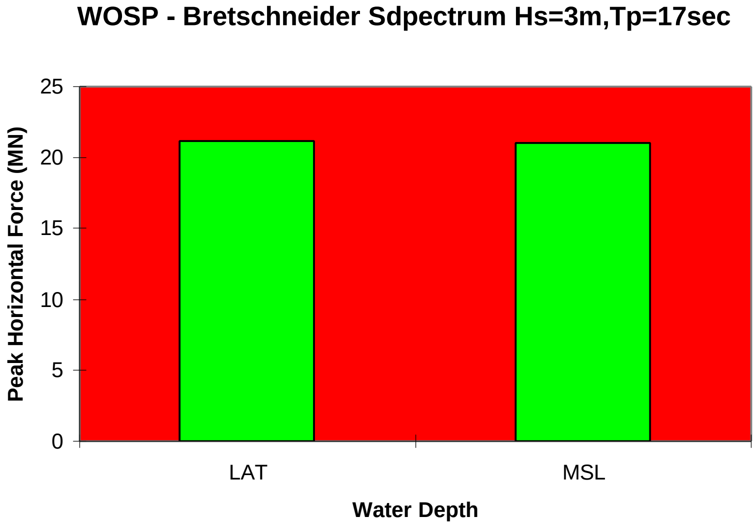

Figure 9. Peak Horizontal Loads with Front Gate in Position

4.6 Surface Pressure Measurements

To compliment the measurements of aggregate load and to facilitate an understanding of the distribution of the load on the structure a series of pressure measurements were made at the same time as the load recording. A total of ten miniature pressure transducers were available and these were used in a total of twelve positions as shown in figure 10. In the absence of wave slam the pressure sensed at each of the transducers rises and falls with the local water elevation, broadly in time with the change in hydrostatic pressure. An example is given in figure 11 with the correspondence between transducer number and measurement position as table 3

Transducer	P1	P2	P3	P4	P5	P6	P7	PT
Position	8	6	7	9	5	2	3	1

Table 3. Pressure Transducer Positions

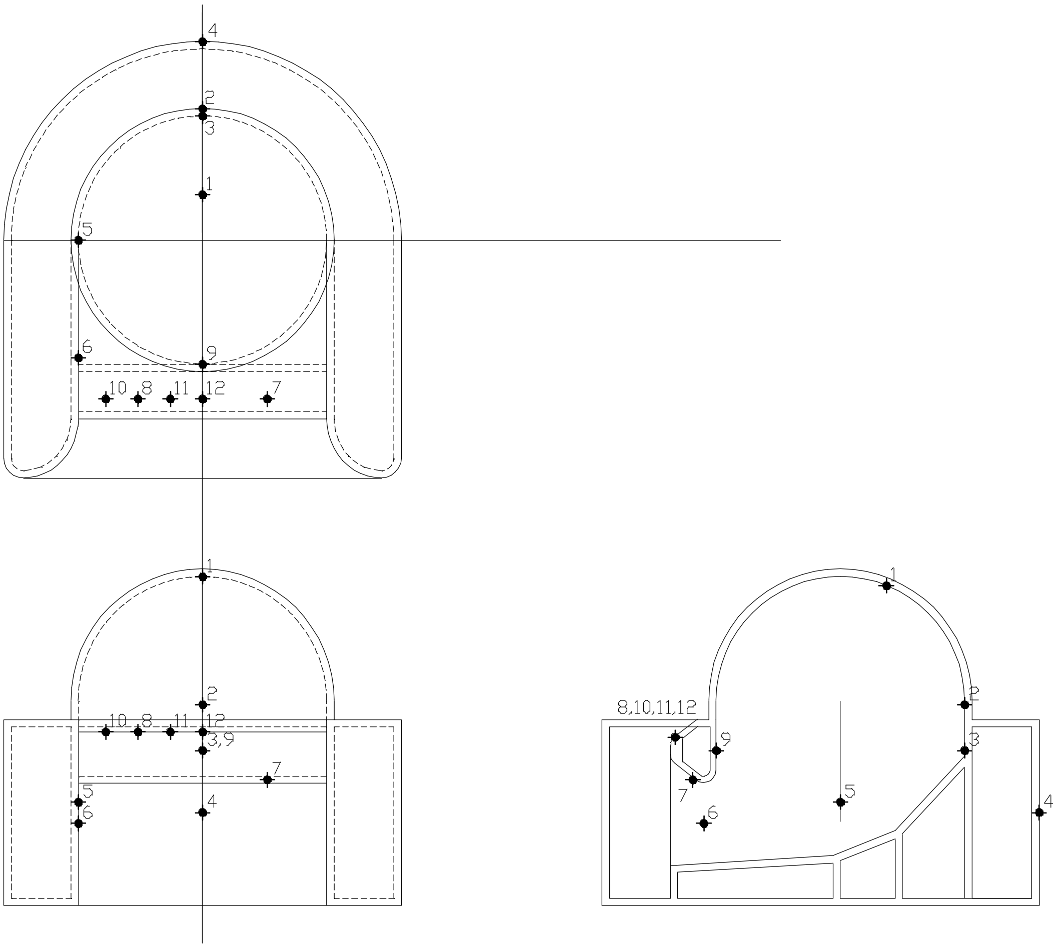

Figure 10. Pressure Transducer Locations

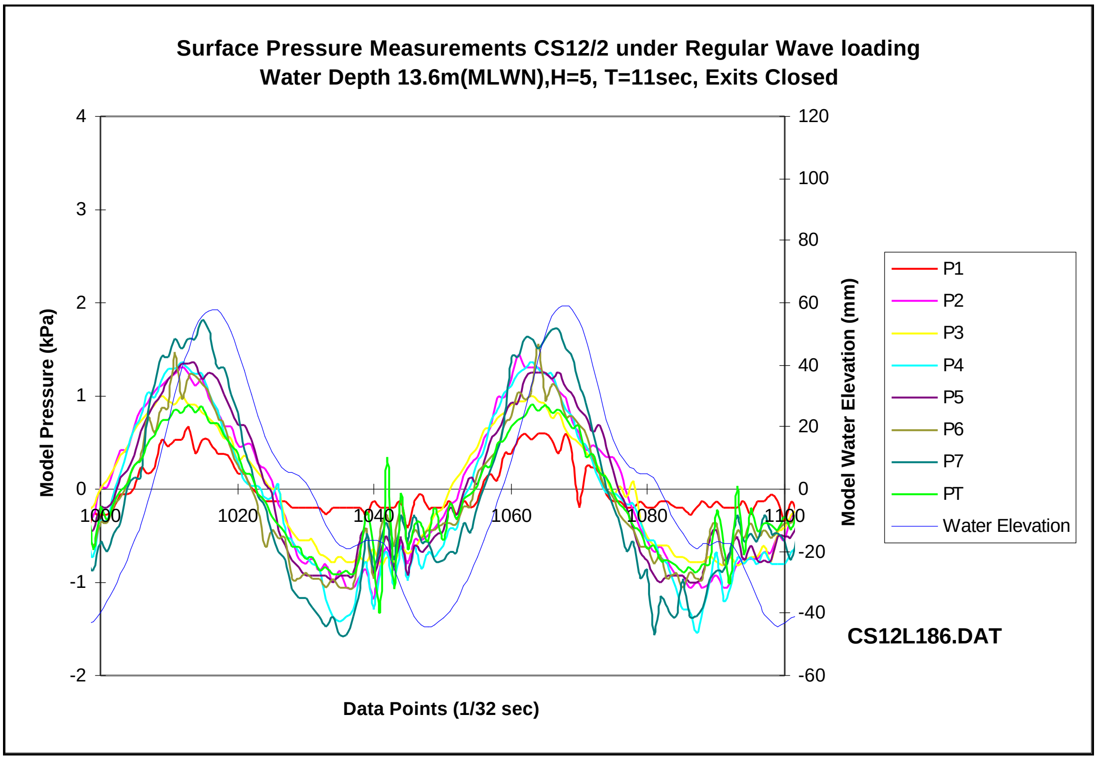

Figure 11. Pressure Readings with no Wave Slam

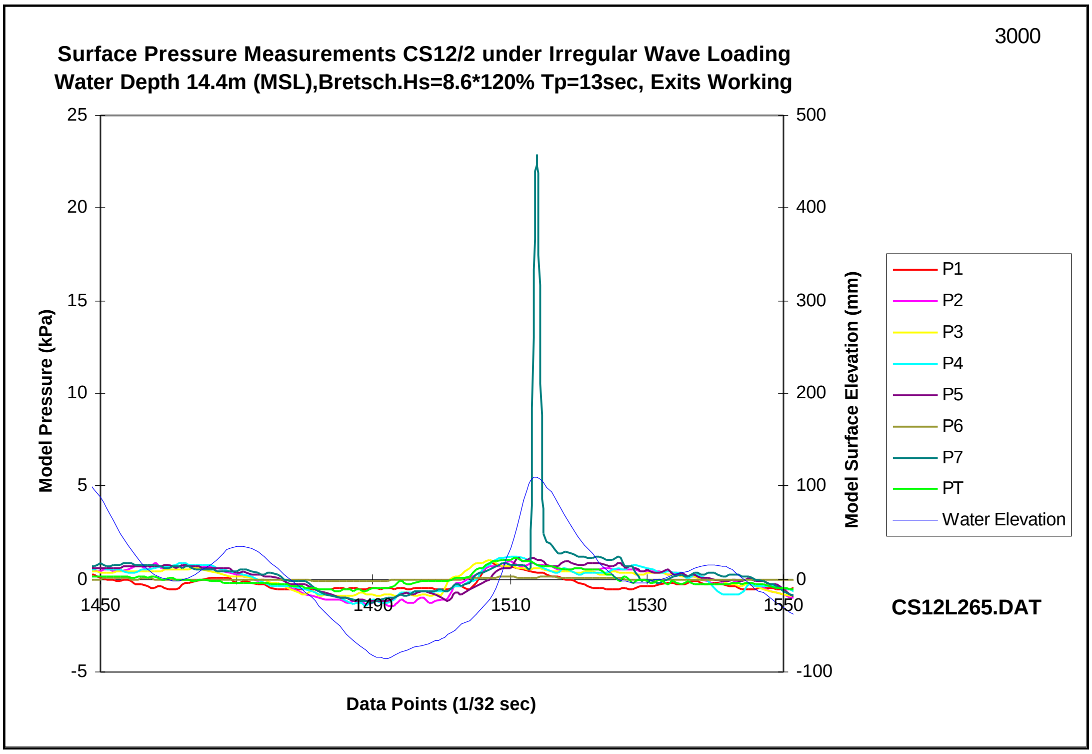

Figure 12. Extreme Surface Pressure under Wave Slam

In the event of wave slam against the structure surface there may however be a very high impulsive pressure (figure 12). The high local pressure is not necessarily indicative of high pressures elsewhere on the surface or indeed of a high force on the structure. The pressure spike on figure 12 is one of the highest values recorded during the test sequence and occurs at point 3 at the base of the cylindrical wall inside the collector. It has occurred as a result of the sudden arrestation of part of the in rushing flow stream impinging on the rear wall but it is noteworthy that no pressure peak is discernible on any other transducer at that time. The aggregate load trace (figure 13) similarly shows a minimal load peak again indicating that the pressure peak is highly localised. Given that the pressure peak is caused by the in rushing of water impacting the back wall the earlier peak on the load trace may be interpreted as the same in rush impacting on the collector lip. The observation of the extreme localisation of peak surface pressures is supported by other records and is highly significant in its implications on the design strength of the collector.

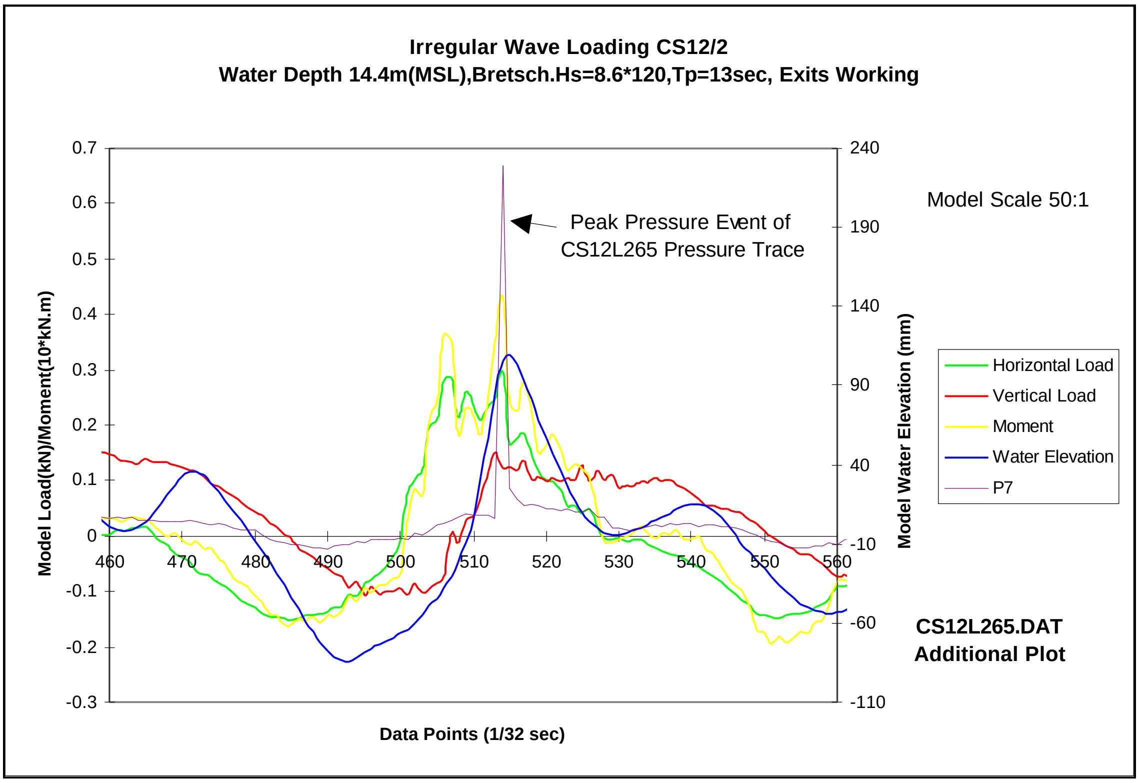

Figure 13. Global Loading at Time of Peak Pressure

4.7 Structural design

Based upon the loads measured during testing a preliminary analysis of the wall thicknesses required for structural stability was made by Sir Robert McAlpine Ltd. and the sizes shown in figure 1 determined. With these sizes it was possible to consider other important aspects of the design such as floating stability, build ability, anchor form etc.

4.7.1 Consideration of Anchoring of the Base Structure.

A diver survey was performed of the area where it is proposed to locate the base structure and a bore hole was drilled at the centre of the site. The areas of exposed rock adjacent to the intended location have a very uneven surface and the bore hole information indicates that there is very little weathering of the top surface of the rock indicating that and weak material has been removed by abrasion of the sand.. The area appears to be free of vegetation which is further evidence of the severity of the environment and of the mobility of the surface sand.

The wave tank measurements of aggregate loads indicated that under extreme conditions the structure and foundations would have to withstand the aggregate loads as indicated in table 4:

Water Depth	Turbine Condition	Maximum Moment (MN.m)	Maximum Horizontal Force (MN)
17.8m	CLOSED	410	78
17.8m	WORKING	348	66
14.4m	CLOSED	434*	76*
14.4m	WORKING	487*	81*
11.5m	CLOSED	571*	73*
11.5m	WORKING	532*	120*
*Denotes result due to a “spike” load			

Table 4 Global Loads under Extreme Wave Action

The loads that the structure imposes on the sea bed are well below the capacity of the supporting rock and as such the design of the foundations concentrated on sliding and overturning resistance.

It is assumed that in the absence of a prepared bed the structure would, on initial installation, rest on the sea bed sand which typically covers the underlying rock to a depth of 1.5m. An analysis of the frictional resistance of a fully ballasted structure shows that it is inadequate to resist the peak horizontal loads and as such it I concluded that additional security must be provided. It has however been determined that the weight of the un-ballasted structure has sufficient self weight to resist the loads from 2.5m high waves at touchdown and that the addition of ballast will of itself provide security against summer storms. It has also been determined that the weight of the structure provides resistance against overturning. This resistance to overturning makes the post installation grouting of the structure to the sea floor a practical proposition.

In consideration of the above three basic options for the foundations were considered.

a) Installing directly on to the existing sand; enclosing the sub base area with a containment skirt, removing the sub base sand and replacing with grout to provide a direct key to the underlying rock.
b) Founding on a prepared rock blanket whereby the sea bed sand would be cleared prior to placement and replaced with grouted stone. The Place collector would then itself be grouted to the prepared base.
c) Founding on a prepared structural foundation whereby the structure would be keyed to the sea floor using pre installed piles.

Whilst a direct installation appears to offer the lowest cost option it is also longest to complete before complete security is offered and thus carries most risk of weather breaking before completion. The other methods will allow faster completion and carry less risk Initial estimates suggest a similar cost for the other options and further studies are required to enable a firm selection.

4.7.2 Floating Conditions

Analyses of floating stability have been performed, considering all structural and buoyancy compartments, for both the intact device and with the main chamber flooded. Details of the analyses are shown in Appendix 5.

4.7.2.1 Draft

In the absence of trimming the structure will float nose down with a significant increase in the maximum draft in relation to the average. Trimming to the level will require either approximately 300 tonnes of ballast at the rear or the equivalent buoyancy at the front of the unit. Estimates of average draft, deepest draft and trim are given in table 3 for the intact and damaged conditions.

The drafts quoted in table 5 include a 300mm deep concrete down stand which is provided under the structure as an aid to the anchor installation.

Condition	Mean Draft (m)	Trim(^{o})	Deepest Draft (m)
Intact	9.9	13	12.5
Intact & Ballasted	10.4	0	10.4
Intact & Buoyancy	9.7	0	9.7
Damaged	13.7	36.	20.4
Damaged & Ballast	14.1	27	19.8
Damaged & Buoyancy	12.4	3	12.9

Table 5. Floating Draft and Trim

Flooding of the central void will cause a significant increase in the maximum draft of the structure, as shown by results in Table 5 for the ‘damaged’ condition. This is because it causes the structure to tip forward due to the eccentricity of the void, the area under the cross beam providing the eccentricity. The ‘damaged & ballasted’ results in Table 3 assume that the initial trim is achieved by adding 300 tonnes of ballast to the rear of the structure. It can be seen that flooding of the central void still produces a significant forward angle of trim to the structure.

The final condition shown in Table 5 for the damaged case assumes that temporary buoyancy of 900 tonnes is provided at the front of the structure. It can be seen that under this condition the trim is returned nearly level with a maximum draft of just under 13.0 m. The provision of this temporary buoyancy is considered prudent in view of the loss of trim, caused by the flooding.

The calculation for the damaged draft assumes that air is trapped within the dome. This will require the turbine stacks to be sealed by temporary plates which will require removal prior to opening the air valves at the bottom of the turbine stacks.

4.7.2.2 Intact Stability

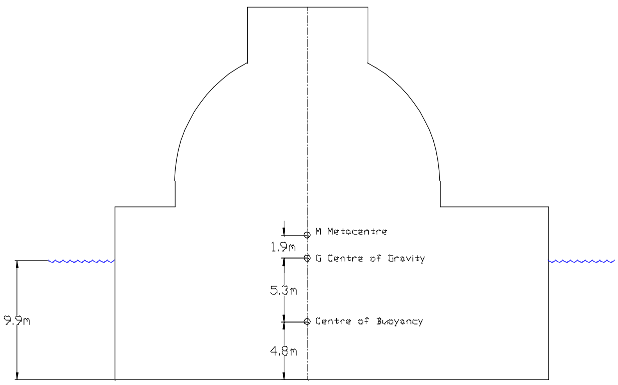

Figure 14. Intact Floating Parameters

Intact hydrostatic stability has been analysed to determine the metacentric height of the structure. This is currently 1.9m which can be considered satisfactory.

Hydrodynamic stability has also been analysed and the undamped natural periods of oscillation have been estimated and are shown in table 6 for the three basic degrees of freedom of the structure. These are Heave (up & down), Pitch (fore & aft), and Roll (side to side). The effect of damping and also added mass effect will lengthen these natural periods.

Heave (s)	Pitch(s)	Roll(s)
6.2	14.7	16.4

Table 6. Undamped natural periods for floating WOSP unit

Hydrodynamic stability has also been studied to see the effect of the free water plane. The internal water has a natural period of significantly less than the structure and so will not reinforce the oscillations. As with the previous periods damping and added mass will increase these periods.

The metacentric height of a body is only a good measure of floating stability for relatively small angles of inclination. To understand the stability of a body under large angles a different approach is required. To enable subsequent rapid analyses at differing drafts and centres of gravity a series of curves, called Cross Curves of Stability, are built up showing the righting moment about a fixed point on the centreline for varying draft and angle of heel. Once the positions of the draft and centre of gravity are known a curve of righting moment against angle of heel can be generated. The resulting curves are shown in figures 11 and 12. The stability curves show that in the intact case the righting moment increases with the angle of tilt up to at least 30° of heel. The damaged cases, however, show that the righting moments do not increase beyond 5° or 10°.

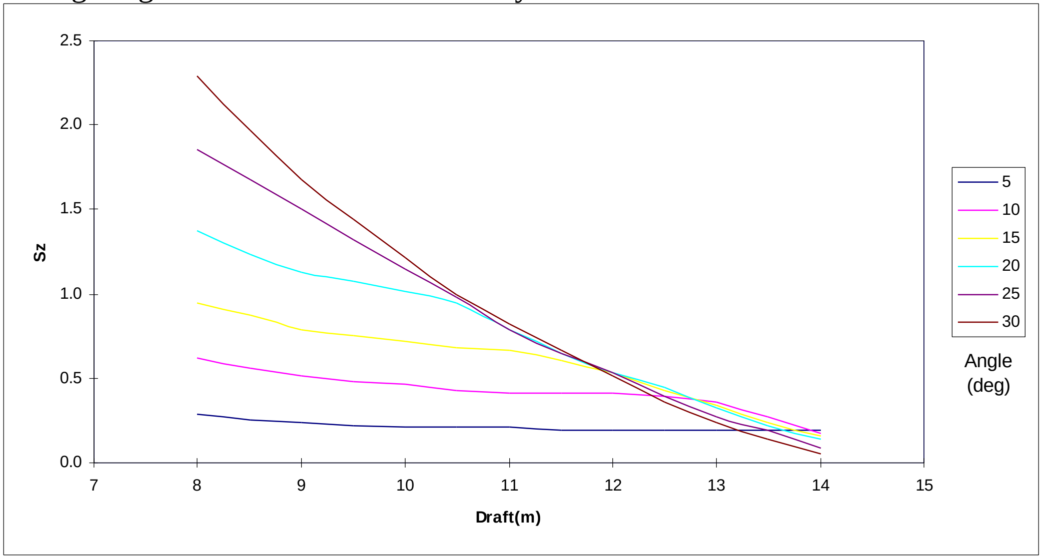

Figure 15. Cross Curves of Stability

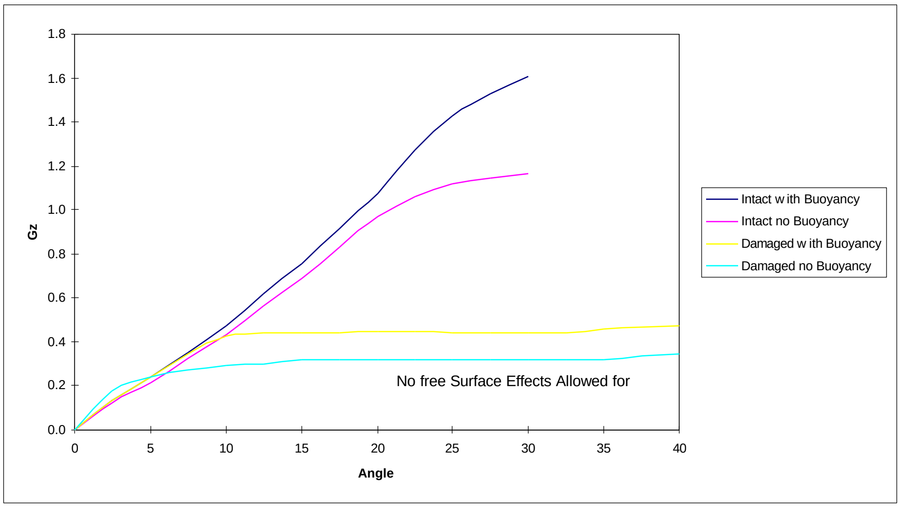

Figure 16. Stability Curves

4.7.2.3 Damaged Stability

Hydrostatic stability in the damaged condition including the effect of additional free water surface has been studied. With the collector chamber flooded the metacentric height of the structure decreases to 1.5m.

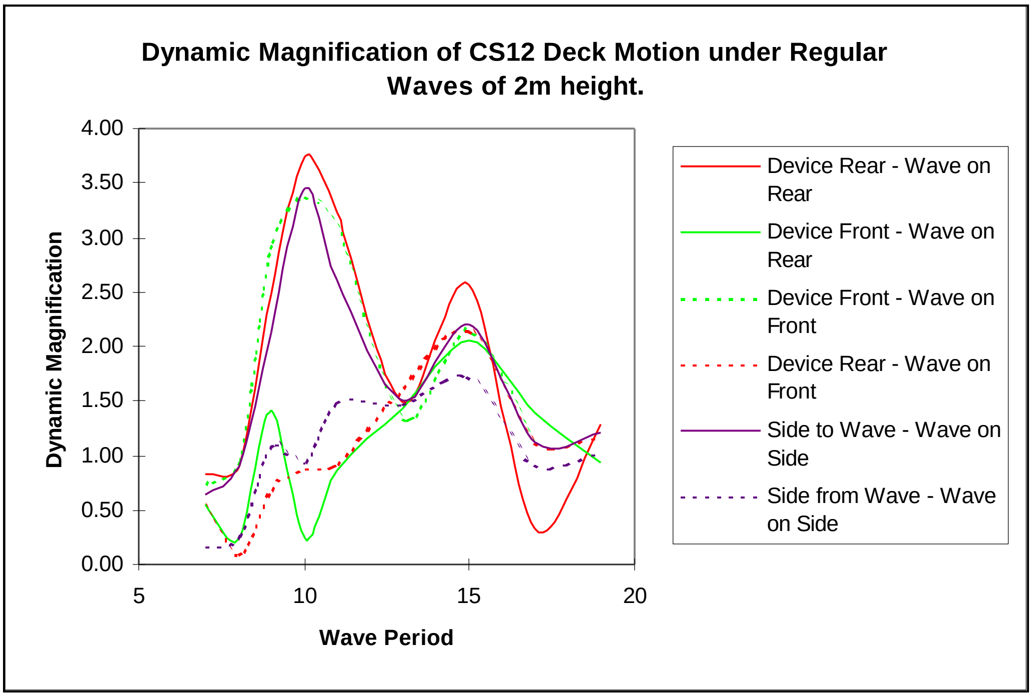

Figure 17. WOSP deck motion under regular wave action.

4.7.3 Model Tests of Floatation and Wind Turbine Accelerations

To support the theoretical studies a model was constructed with the correct weight distribution including a lumped mass to represent to wind turbine. The model was lightly tethered using “bungee” rubber and subjected to regular waves with a height of 2m. The vertical motion at the front and rear of the deck was recorded for various wave directions. The motions were measured by connecting light strings to the measuring points and running the strings over potentiometers mounted on a rigid frame so that the movement of the measuring points was converted directly into an electrical signal. The results are summarised in figure 17.

The peak in the measured response curves at 15 seconds corresponds to the predicted resonances in pitch and roll. The peak response at 10 seconds does not correspond however to the predicted heave resonance. The form of motion observed around 10 seconds was of a large motion on the side facing the incident wave with a relatively small motion on the opposite side. This response requires further investigation.

To investigate the likely loads on the wind turbine tower and turbine bearings during transportation a direct measurement of the acceleration of the top of the tower was made. The results of this measurement under 2m regular wave excitation are shown in figure 18. Wind turbine accelerations were also measured with a Bretschneider spectrum of H_s=1.5m and Tp=13 seconds. Under these conditions the maximum observed acceleration was 2.5m/sec^2. The ability of the wind tower to support such accelerations is being investigated by Vestas.

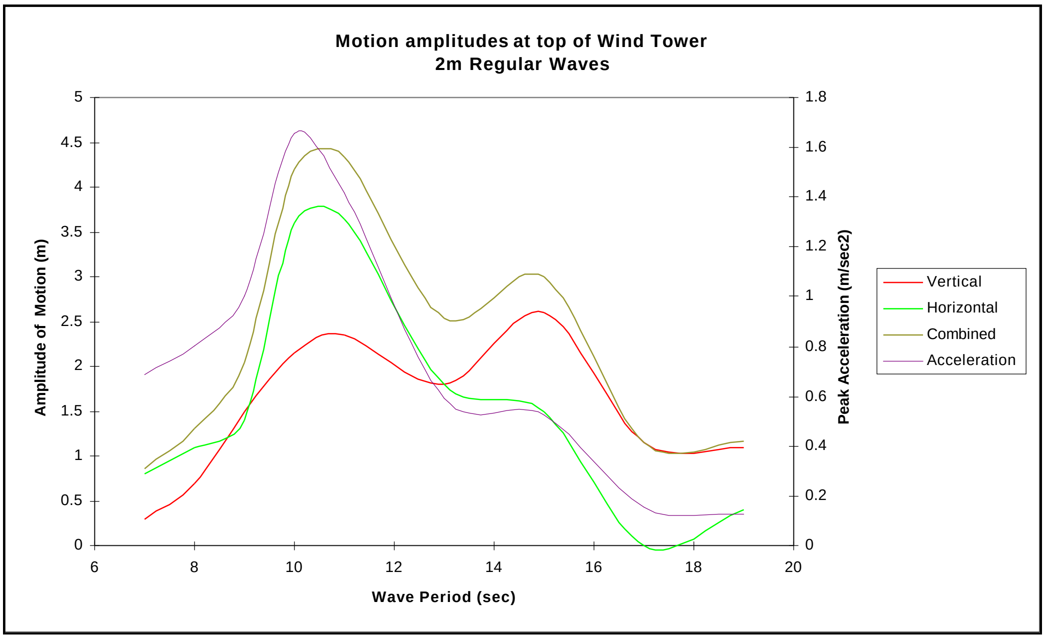

Figure 18. Motion of Top of Wind Tower under Regular Waves

4.8 Wind Climate at Dounreay

A study commissioned from the Advisory Services Branch of the UK Meteorological Office gave the summary data in respect of the wind climate at Dounreay as per Table 7.

Year	Maximum Gust (kts)	Maximum Hourly Mean (kts)	Annual Average (kts)
1970	83	60	19.1
1971	66	50	16.6
1972	80	60	17.6
1973	53	45	18.1
1974	76	56	16.4
1975	67	54	16.1
1976	92	68	17.4
1977	73	53	17.0
1978	76	56	16.3
1979	72	50	15.4
1980	74	52	17.1
1981	78	66	18.0
1982	72	60	17.5
		Overall Average	17.12

Table 7. Summary Wind Climate at Dounreay

4.9 Electrical Layout

It was agreed that in respect of the differing requirements of the control systems for the wind and wave energy generators that two independent control systems would be maintained on the WOSP structure but that these should be linked with a ring main device as shown in figure 19.

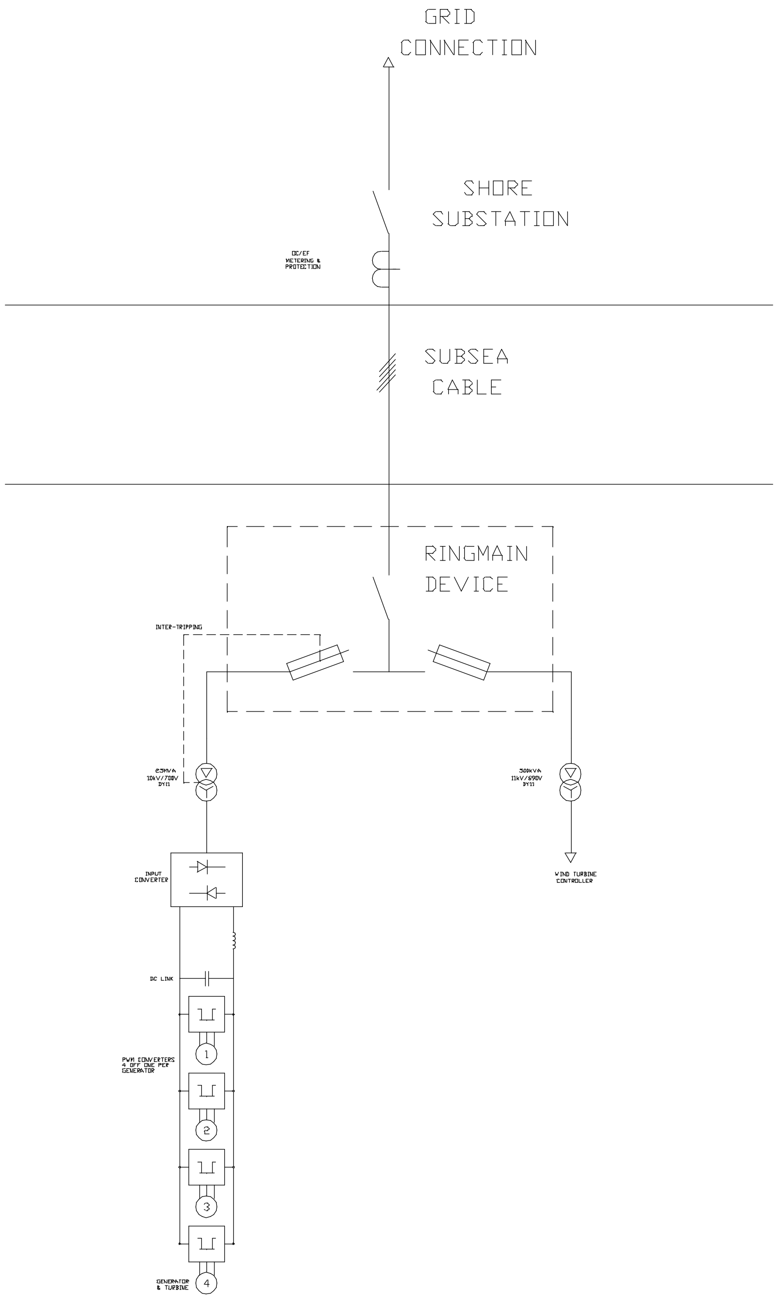

Figure 19. Single Line Diagram of WOSP Connections

5. Results and Conclusions.

Whilst the primary objective of combining a wind turbine with a near shore wave energy collector was not achieved due to circumstances beyond the control of the project team great benefit will be derived from the research and development work performed during the fulfilment of the project. In particular the data gathered on the structural loads induced by a range of wave environments will benefit structural design teams and the information gathered on local slam pressures has great increased our knowledge in this difficult area. The original justification for the project, namely the economic benefits of amortising structural costs between wind and wave energy generating technologies remains valid and there is no doubt that the performance of the various works during the project will further the future development of these combined technologies.

6. Reference

1. An Experimental Investigation of Wave Forces on an oscillating water column type wave energy caisson. Jayakumar, S.Neelamani and V.S.Raju. Proceedings of 1993 Edinburgh European Wave Energy Symposium. ISBN 0-903640-84-8