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Foundation Risk & Geotechnical Uncertainty Mapping for future Offshore Wind Farm Developments

"D3: Final Report"

INFRASTRUCTURE★RENEWABLES★OFFSHORE★STRUCTURES

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1. INTRODUCTION

In early 2012, INFOMAR commissioned Gavin and Doherty Geosolutions (GDG) to undertake a study to investigate the geotechnical uncertainties and risk to the foundations of future offshore wind farm developments around the Irish coast. This study proposed to review the existing INFOMAR data and to assess the potential usefulness of this data in de-risking the capital cost of offshore wind farm construction.

The offshore wind energy sector across Europe has undergone rapid expansion in recent years as wind has been identified as a viable source of clean, renewable energy. Recent technological developments, coupled with societal and political pressures to reduce carbon dioxide emissions and our dependence on fossil fuels, has led to a rapid development of the wind energy industry across Europe. Offshore wind energy offers a number of advantages compared to onshore wind, such as limited aesthetic impact due to their location far from land, higher and more consistent wind speeds, and greater power generation through the use of large-capacity turbines. There are currently 1371 offshore wind turbines installed and grid connected in ten European countries which produce approximately 3813 MW of electricity. Offshore wind turbines with an additional generating capacity of 5285 MW are currently under construction or in the planning stage (EWEA 2012). There are also plans to develop hundreds of GW of offshore wind energy globally over the next 20 years, with approximately 50 GW consented at sites around the UK alone.

Despite this global trend, to date there has only been one partially completed offshore wind farm in Ireland, at Arklow bank, where seven turbines are currently generating electricity. In order for Ireland to meet its renewable energy targets by 2020, there is a need to rapidly exploit offshore wind energy. For the past 10 years, Ireland has focused on wind energy generation from onshore resources. However, it is becoming increasingly more difficult to (i) secure onshore grid connections and (ii) obtain planning consent due to environmental factors. As a result, offshore wind energy is the most obvious source for future development, and it is made even more attractive by the potential to export energy to the UK and further afield by means of offshore substations and grid interconnectors. After an open tendering process in late 2009, the UK crown estate announced the results of the Round 3 offshore wind release, which gave consent for the development of nine sites around the British coast. This was part of a strategic development initiative by the UK government to promote offshore wind energy as the primary means to meet their 2020 renewables targets. The Round 3 release provides a driver for other countries to follow the same framework and exploit this underdeveloped resource.

As Ireland begins to develop offshore wind energy, there will be a need to consider appropriate site selection and strategic development of the supporting infrastructure. One of the key factors controlling the cost of offshore wind farm construction is the type of foundation, and therefore, this is often a primary concern when considering whether or not to develop a site. However, there is often insufficient geological and marine knowledge at a given location for developers to make an informed decision about the most appropriate foundations and what the associated risks are with each concept for that site.

Significant volumes of data have been gathered by the INFOMAR research programme in recent years that could better inform offshore development activities. This report reviews the available data and assesses the potential to exploit the data to reduce the risk to offshore wind farm foundations.

2. OFFSHORE WIND DESIGN

2.1 Development Process

Offshore wind farm projects adopt a staged approach to their development. The initial stage of this process involves site selection and application for a foreshore development lease, which starts the consenting process. This begins by conducting a detailed geophysical survey, and the collection of met-ocean data and preliminary geotechnical investigations including cone penetration testing (CPT) and drilling/sampling. All preliminary data collected informs the initial design stages including foundation engineering. At this stage in the project timeline, identification of foundation risks can be a direct impediment to development and could pose serious economic concerns. Alternatively, where risks are not identified, remedial solutions may be required at the later construction stage, which can result in an even more costly outcome. As the project progresses toward completion, more information is obtained which reduces the uncertainties and therefore reduces the unknowns surrounding budget estimates. This allows the project to proceed to the next stages with reasonable financing. At the project outset if there is insufficient data and an elevated risk profile, then this can become a technical barrier to development. The impact of improving the amount of data available on the project risk is illustrated in Figure 2.1.

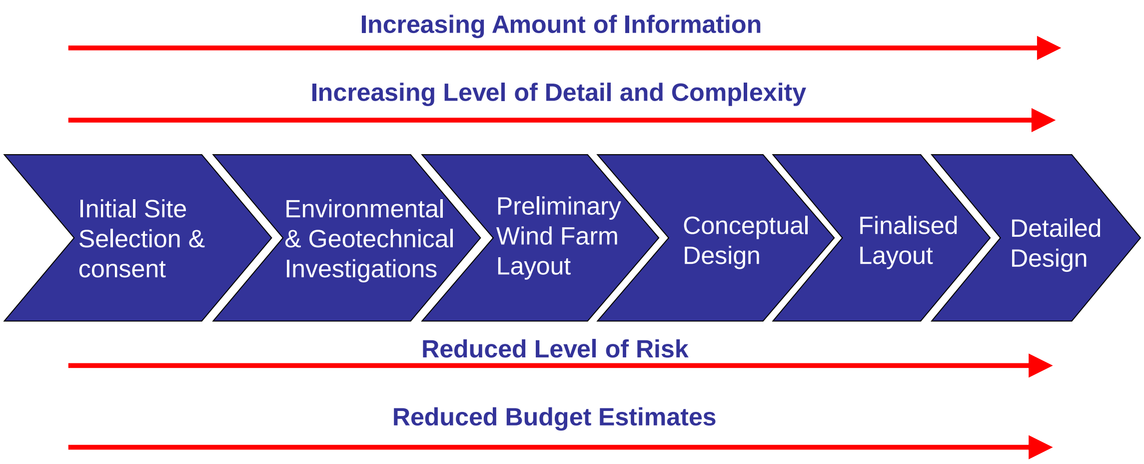


Figure 2.1: Impact of increasing data on project risk

2.2 Current Policy

The EU directive on renewable energy sets an ambitious target, i.e. to yield a 20% share of total energy consumption from renewable sources by 2020 (EU 2009). This is to be achieved through a series of equally ambitious national targets for individual Member States. In the long term, the renewable target for 2030 is set at 33% of total energy. The importance of reaching these targets was stressed in the 2007 Renewable Energy Road Map document, *Renewable energies in the 21st century: building a more sustainable future* (EC 2007), which described Europe's current societal and economic vulnerability due to the following factors; it's increasing dependence on oil and gas, climate change, growing imports, and rising energy costs. An aggressive strategy to increase renewable energy production will combat these issues and wind energy will play a critical role in transforming the long-term European energy market into a sustainable and competitive environment. The pivotal role of wind power was reinforced in the SET-Plan communication, *Investing in the Development of Low Carbon Technologies (SET-Plan)*, published by the European Commission (EC 2009).This stated that wind power would be "capable of contributing up to 20% of EU electricity by 2020, and as much as 33% by 2030".

Two key indicators of quality of life are a clean environment and economic self-sufficiency. Ireland currently relies on imported fossil fuels to provide 90% of its energy needs. It is clear that there is significant potential to reduce fossil fuel reliance and therefore environmental effects and avoid other alternative solutions (including nuclear power) if we can utilise our natural wind resource. The Forfas and Intertrade Ireland (2008) report identified the potential for Ireland to develop a green economy to lead economic growth and allow huge growth potential in Irish jobs. In the last five years, worldwide employment in the total renewable energy sector (incl. Wave, tidal etc.) has grown from 230,000 to 550,000. The European Wind Energy Association (EWEA) estimates that 462,000 people will be employed in the wind energy sector (both onshore and offshore) by 2020, with the figures working in offshore wind sector alone predicted to reach 300,000 by 2030. Despite being identified in the report as a major global developer and innovator in technical aspects related to wind farm developments, Ireland has to date accounted for less than 1.5% of EU employment in wind energy. From its resource potential it is obvious that Ireland could become a major exporter of wind energy. The European offshore industry is predicted to produce up to 150 GW of power (enough to supply 13-20% of the EU requirements) by 2030 (Greenpeace and EREC 2008). The capability for exporting wind energy will be increased significantly with the development of high capacity offshore grids. In December 2010, Environment ministers from nine EU countries (including Ireland) signed a political declaration in Brussels on cooperation for the development of infrastructure necessary to develop hundreds of Gigawatts of offshore wind energy in the Irish and North Sea. Plans for this EU Supergrid which are being championed by the Joint Research Centre, are rapidly gaining traction within the EU.

The Irish Offshore Renewable Energy Development Plan which was launched in November 2010 stated that through a combination of offshore wind, wave and tidal energy, Ireland has the capacity to generate up to 10 times its own electricity requirements. The European Wind Energy Association (2006) notes that although Europe has a mature offshore oil and gas industry, the demands of offshore wind farm developments are quite specific and therefore research and development in the area of foundation behaviour and the risks associated with these is required. A more developed understanding of the geotechnical risks associated with these projects is particularly urgent since foundation systems represent up to 45% of the capital expenditure (CAPEX) costs of offshore wind developments. A range of foundation systems which are widely used in the oil and gas sector have been used in order to support offshore wind turbines built to date. The trend to move to deeper water sites with higher environmental loading and larger turbines has led to the development of new innovative foundation systems. Each system has a number of constraints governing whether its use if feasible (See risk assessment in Appendix A). As a first estimate, the primary constraints are the water depth and geological conditions. The INFOMAR dataset provides key information regarding both of these important quantities. In this report, the foundation systems likely to be used in the medium-term to support offshore wind turbines are considered and general items which might cause risk (e.g. variable geological conditions) and specific risk elements associated with each system (e.g. difficulty in driving large diameter monopiles) are considered.

2.3 Foundation Options

The specific foundation concept adopted for a given offshore development site is affected by the local geology amongst other factors, the most important of which is water depth. In water depths of up to 35 m, single large steel tubular piles of up to 6 m diameter, referred to as monopiles, are driven into the sea bed to support the turbine structure (See Figure 2.2). Over the past ten years, the vast majority of offshore wind farms have targeted near-shore and shallow water sites, and at these locations over 70% of the turbines have been supported on monopiles. The cost efficient and elegant monopile design coupled with the straightforward installation process has resulted in monopiles becoming the preferred industry concept. The DNV (2007) offshore guidelines recommend that monopiles are suitable for water depths ranging from 0 to 25 metres. However, recent installations have pushed the upper limit of these water depths to over 30 meters.

Gravity bases account for 20% of the currently installed turbine foundations, where they resist the applied loads through the deadweight of the ballasted structure and the bearing capacity of the underlying soils. The first offshore wind farm installed off the Danish coast in 1991 was installed on a gravity foundation in shallow water less than 10 meters. The DNV (2007) guidelines recommend that these foundations are suitable up to 25 meters water depth, however systems currently in development (e.g. Gravitas system) suggest that gravity bases may be suitable for water depths in excess of 50 meters.

In water depths of 20 to 50 m, which are typical of many areas designated for development in the current UK Round 3 developments in the North Sea, tripod (three-legged) or jacket (four-legged) structural frames can transfer the platform loads to steel piles driven into the sea-bed, located under each leg. Whilst it is believed that these foundation types are appropriate for much of the geology found in the North and Irish Sea, traditional jacket structures typically involve a step increase in costs compared to monopiles.

Research into this area highlights the dynamic nature of the industry where foundation concepts are continuously being developed to suit the conditions that are encountered offshore. For example, suction bucket foundations are recommended by DNV (2007) as being suitable up to 25 m water depth, whereas recent research (Universal Foundations, 2012) suggests that this concept is suitable up to 60 meters water depth.

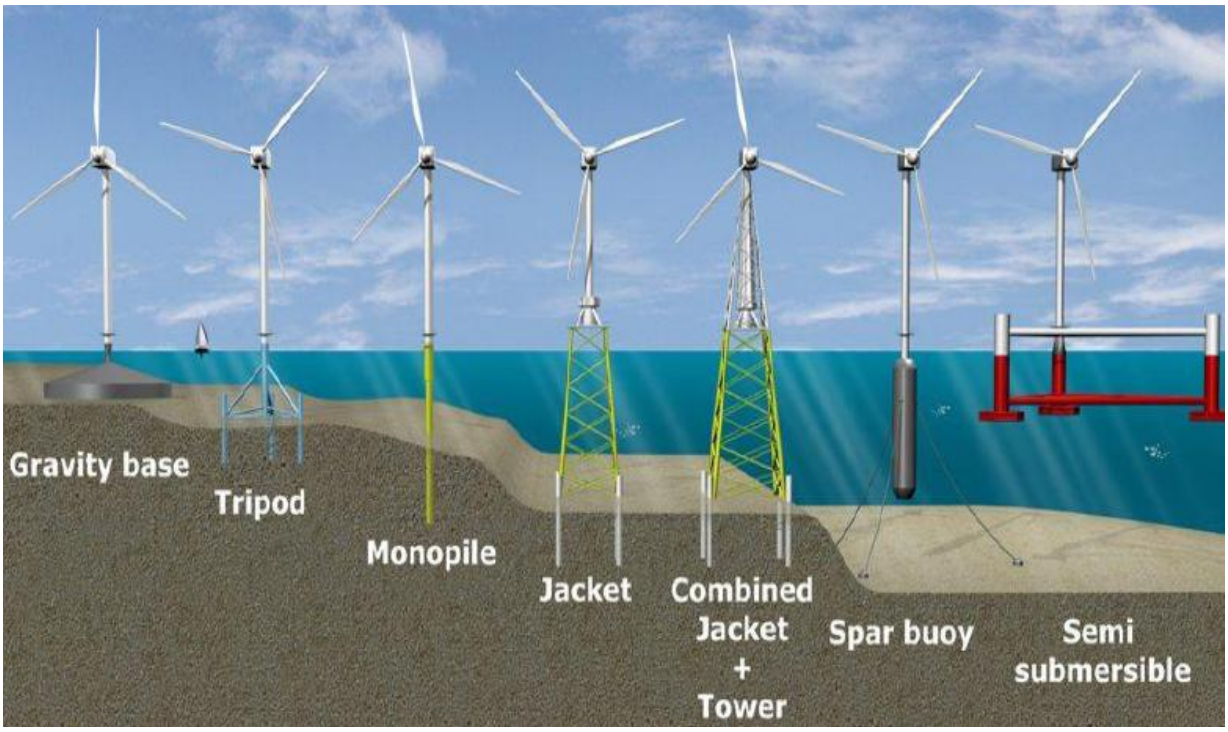


Figure 2.2: Foundation Options for Offshore Wind Turbines (After Plocan, 2012)

For water depths greater than 60 meters, floating structures start to become viable and are more competitive than fixed bottom structures. A number of floating concepts can be considered including semi-submersible, tension leg platforms, and spar buoys.

2.4 Background on foundations systems

The foundations for conventional offshore structures such as oil and gas platforms carry a significant vertical load due to the large self-weight of the platform. This load increases the structure's stability against overturning moments generated by environmental loads caused by wind and wave action. In contrast, offshore wind turbines have a relatively low self-weight, and with the hub of a typical 5 MW turbine being 150 m above sea bed level, they generate extremely large over-turning moments (See Doherty and Gavin, 2011). The low dead weight of an offshore wind structure increases the compliance and this enhances the impact of cyclic wave loading on the structural response. Despite the difference in loading conditions and offshore constraints, a number of foundation systems were developed for the offshore oil and gas sector are now being used to support wind turbines.

In the short to medium term, offshore wind developments in Ireland are likely to be undertaken at sites with high consistent wind speeds, water depths below 20-30 m, which are located at a minimum distance of 5 km from shore, with low wave exposure and deep sea bed sediment. Given that water depths increase rapidly with distance from the shoreline along the West coast, offshore wind turbines in this region are likely to be founded on floating structures and would at present be viewed as long-term opportunities. The Irish south-coast has deep water combined with shallow rock. The ideal location for the development of offshore wind turbines, using existing concepts for fixed-bottom structures is the East coast. This is reflected in the number of projects under development in the Irish Sea (See Figure 2.3). This area boasts a less severe wave climate, a large number of sandbanks and proximity to large domestic and export markets.

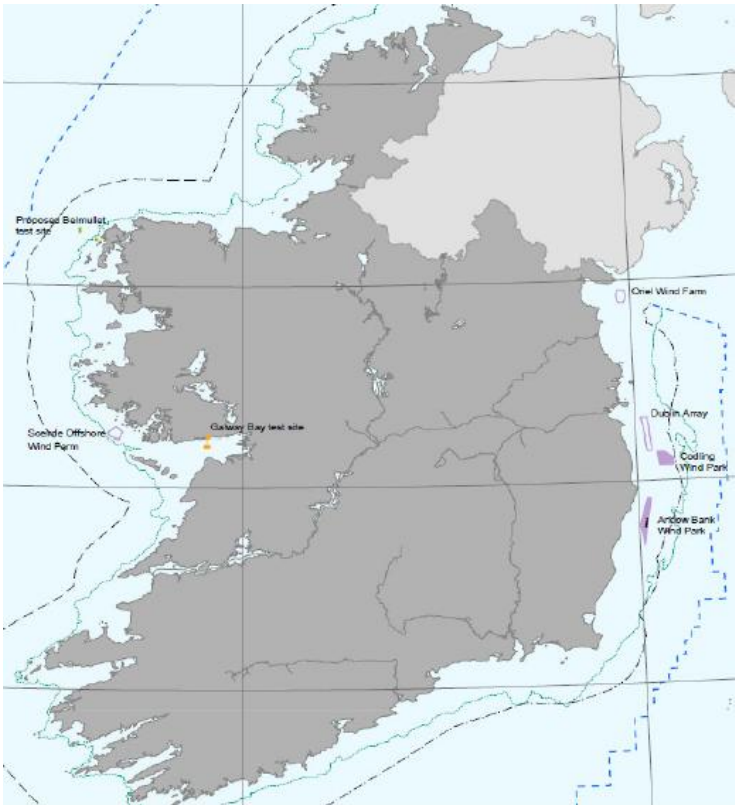


Figure 2.3: Offshore Wind Farm developments proposed along the Irish Coast (SEAI 2010)

2.5 Foundation Systems for Fixed Offshore Structures

In water depths of up to 35 m, gravity bases or single large steel tubular (mono) piles of up to 5 m diameter are driven into sea bed to support the structure (See Figure 2.4). The trend to develop relatively near shore and shallow water sites to date has resulted in 95% of offshore developments being founded on either gravity bases (20%) of Monopiles (75%).

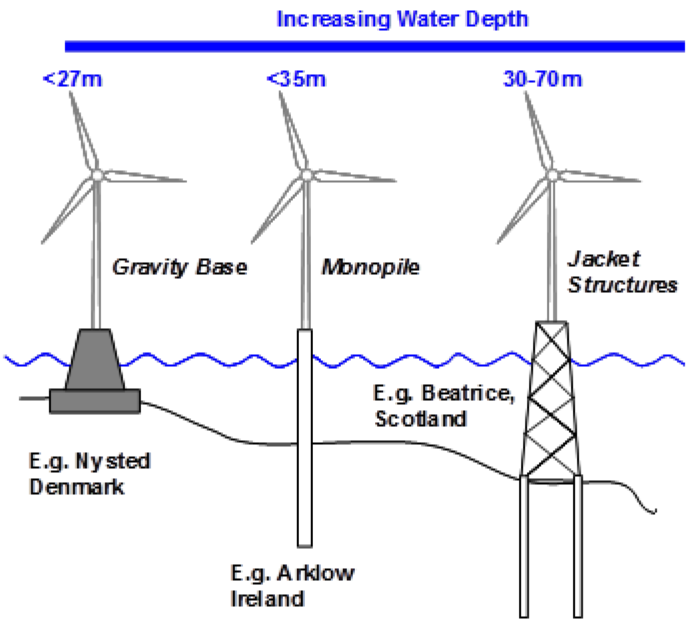


Figure 2.4: Effect of water depth on foundation choice

2.5.1 Gravity Base

Gravity Bases are suitable at sites with relatively shallow water depths and where the near surface soils are both competent and laterally consistent. High seabed gradients and soil heterogeneity can be a significant risk. Where shallow soft deposits exist near surface or where scour is an issue, shallow skirts can be used to transmit loads to deeper, more competent strata. Gravity bases are used for the majority of onshore wind turbine projects and their principle of operation is to use the dead weight of the structure (gravity loads) to resist the overturning moments generated by environmental loading. Where skirts are used, the soil plug inside the skirt can provide additional resistance through the deadweight of the constrained soil mass and the tensile capacity generated from transient suctions which resist wind and wave loads. Gravity foundations are used extensively in the offshore oil and gas sectors where heavy topsides generally provide very large vertical reaction to resist horizontal and moment loads. The structures are typically constructed of reinforced or pre-stressed concrete (See Figure 2.5). One major advantage of this type of foundation is the ability to float out to the turbine location and ballast the structure in-situ. However, it should be noted that all gravity bases constructed to date have required heavy lift vessels as there was insufficient hydrodynamic stability to facilitate floating transit and deployment. One key disadvantage is the requirement to provide sea bed preparation prior to placing the foundation.

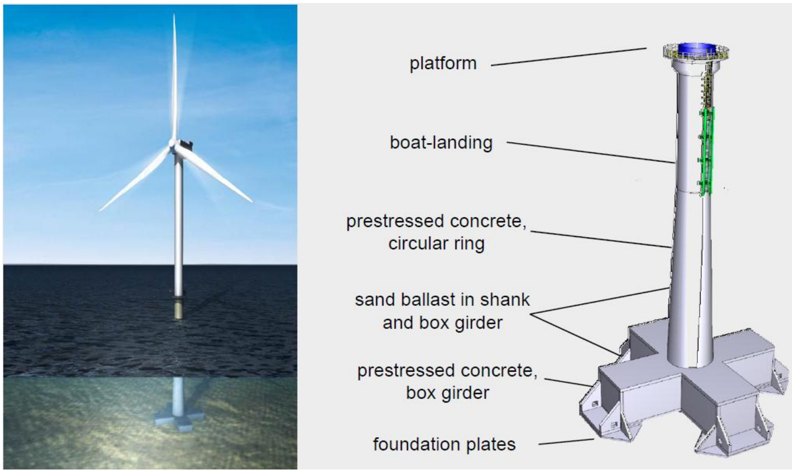


Figure 2.5: Strabag gravity based foundation system (source Weber 2010)

2.5.2 Suction Caissons

Suction Caissons were developed as an alternative to the use of skirted foundations for offshore oil and gas platforms (Randolph and Gourvenec 2011). Concrete caissons were first employed in 1991 to support the Snorre A tension leg platform in the North Sea. Steel buckets or suction caissons have been used successfully on a number of jacket platforms. The suction bucket consists of a skirted section and a lid, which is reinforced by a series of stiffeners. The bucket is installed by applying a suction pressure to the top of the lid. This generates a differential pressure between the inside and outside of the bucket, sucking the foundation into the seabed. The structure is simple and requires a minimal number of installation steps. This can considerably reduce both the offshore installation time and the associated CAPEX cost. In the offshore wind sector, single suction caissons (See Figure 2.6) have has been developed as a flexible solution that is technically and economically feasible across a range of water depths, from 5 m to 55 m.

Figure 2.6: Universal Foundation suction bucket concept (after Universal Foundations A/S)

Suction caissons have been used in a demonstration project to support a 3 MW Vestas turbine. This concept has also been used to support offshore MET MASTs in shallow water at Horns REV 2. The environmental benefits of suction installation include the elimination of pile driving (meaning a reduction of noise and vibration during installation), and for decommissioning purposes at the end of life the caissons can be easily removed by reverse suction. Caissons can also be used in a tripod or quadripod arrangement. For example, the SPT self-installing platform (shown in Figure 2.7) is a concept where the turbine is transported to site pre-assembled and is supported on a frame founded on three suction caissons.

Figure 2.7: SPT Offshore Self-Installing Foundation Concept (image from sptoffshore.com)

2.5.3 Monopiles

Monopiles are single, large diameter (4 to 6 m) steel tubes which are driven into the sea bed and provide lateral restraint to resist the applied environmental loading by mobilising horizontal earth pressures in the near surface soils. The Monopile foundation system consists of two pieces (See Figure 2.8), the pile that extends out of the sea bed, and a transition piece that is placed over the pile. The transition piece has a larger diameter to allow it to slide over the pile and provide a vertical overlap of approximately 10m-12m. The joint is then grouted. The purpose of the transition piece is to facilitate the turbine connection and to correct the vertical tolerance of the monopile so that the turbine tower can be installed within an agreed offset (for example, 0.5 degrees). Monopiles used in the offshore wind sector are typically driven 20 to 30 m into the sea bed resulting in relatively low pile slenderness ratios (ratio of pile length to diameter) of 5 to 6. Piles are designed using semi-empirical design methods developed for long slender piles. At present, there is some concern regarding the use of these design methods which were developed for specific conditions experienced in the offshore oil and gas industry (long flexible piles with high ratios of vertical to lateral forces) and are now being extrapolated to a considerably different situation in the offshore wind sector (large diameter rigid monopiles with high ratios of lateral to vertical forces) (See Doherty and Gavin 2012). Given strict rotation tolerances (deflections less than 0.5°) used for offshore wind turbines standard monopiles become inefficient in water depths greater than ≈ 30m.

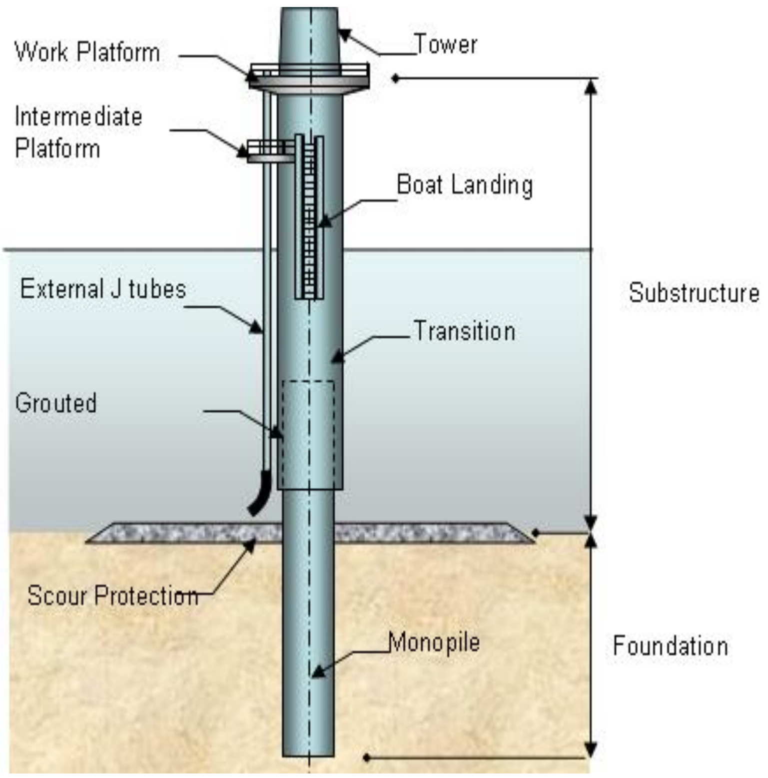


Figure 2.8: Details of a Monopile Foundation (source: Garrad Hassan)

Various methods to increase the resistance of monopiles have been investigated. One of the most promising methods of improving the structural rigidity is to include some form of bracing. This type of approach has resulted in hybrid concepts being developed (combined monopile/jacket concepts). For example, the Keystone novel "Twisted Jacket" Structure was chosen by the UK Carbon Trust as one of the four winners in an international competition to promote emerging foundation concepts for the offshore wind energy sector. The concept uses a relatively small diameter central monopile which is braced by three raking piles arranged within a light-weight jacket (See Figure 2.9). The concept has recently been used to support the first Met Mast structure installed as part of the UK Round 3 developments, which was deployed in the Hornsea zone in late 2011.

Figure 2.9: Hornsea Met Mast supported on Keystone "twisted jacket" structure (www.smartwind.co.uk)

2.5.4 Jacket Structures

In water depths of 30 to 70 m which are typical of many areas designated for development in the current UK Round 3 developments in the North Sea, tripod (three-legged) or jacket (four-legged) structural frames transfer the platform loads to steel piles driven into the sea-bed. Whilst it is believed that these foundation types are appropriate for much of the geology found in the North and Irish Sea, the relative costs of traditional jacket structures are considerably higher than that of a monopile at water depths of between 30 m and 40 m. Jackets are essentially steel structural frames which transfer the platform loads to piles located either under each leg (See Figure 2.9) or possibly to skirt piles around the jacket base perimeter. The overturning moments and self-weight forces are translated by the jacket structure into vertical reactions at the sea bed level, with the foundation reacting in a push-pull manner, with opposing piles acting in compression and tension (uplift). Piles loaded in compression develop a resistance to axial loads through a combination of shear resistance developed between the pile shaft and soil and end bearing resistance at the pile base. When loaded in tension, the resistance at the pile base is zero, and the pile length is controlled by the shear resistance which can be developed along the shaft of the pile. Since all piles beneath the structure can be loaded in either tension or compression (depending on the direction of the wind and/or wave loads) the critical design pile length is that which is required to resist the tension load. Traditional design methods for jacket structure foundations were developed for the offshore oil and gas sectors. Gavin et al. (2011) show that existing design methods provide poor estimates of the pull-out capacity of jacket structure foundations.

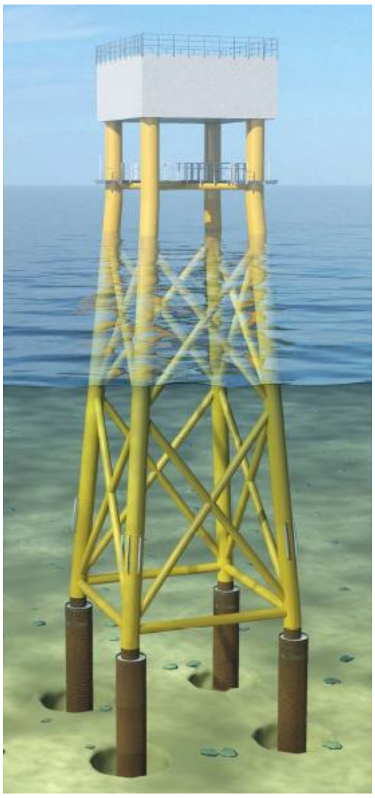
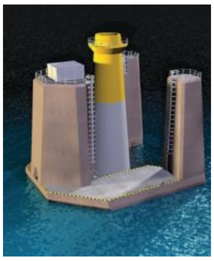
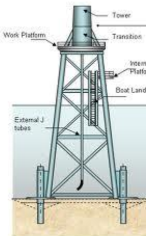
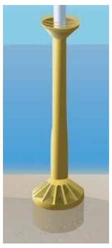


Figure 2.10: Jacket Structure (source Jensen 2010)

Whilst the choice of foundation will be very much a site specific decision, the relative advantages and disadvantages of various systems are summarised in
Table 1.

Table 1: Summary of foundation systems used for offshore wind

Foundation Concept	Advantage	Disadvantage
Monopile 	1. Design Simplicity 2. One Element, so quick fabrication 3. Simple/Quick offshore installation process 4. Vast offshore experience: hundreds of monopiles installed	1. High Steel Costs 2. Variable Steel Costs=Budget Risk 3. Limited to shallow waters 4. Poor design tools available 5. Very limited number of suppliers. 5. Environmental noise from piling 6 Large vessels for installation 7. Requires high capacity offshore hammers (At limits of technology)
Gravity Base 	1. Uses concrete, which is readily available & comparatively cheap 2. Can be ballasted & tugged to site to avoid costly installation vessels 3. No pile driving (reduces noise)	1. Reliant on high bearing capacity of seabed 2. Technical concerns: Differential Settlement & Rotations 3. Poor design tools available 4. Needs seabed preparation: increased installation time 5. Requires massive quayside space & onshore crane 6. Lack of experience in deep water
Jacket/Tripod 	1. Smaller elements so more suppliers 2. More flexible geometries (can be spliced) 3. Takes advantage of deep bearing strata 4. Oil & Gas experience	1. Expensive - High Steel Costs 2. Variable Steel Cost 3. Heavier & More Expensive than Monopiles 4. High Fabrication costs 5. Requires piling, which has environmental noise concerns 6. Large vessels are required
Suction Caisson 	1. Less steel required than monopile 2. Heavy cranes not required 3. Small Installation Vessels 4. No transition piece required 5. No driving 6. Stiffer structure	1. Limited experience 2. Not suited to non-homogeneous soils 3. Not suited to sites with shallow/variable rock levels. 4. More complicated fabrication process 5. Requires grouting beneath lid 6. Only suitable in some soil types 7. Float-out requires deep port

The Offshore Wind Accelerator (OWA) programme is the Carbon Trust's flagship collaborative research programme, which was established in 2008 and brings together eight of the largest offshore wind developers to work towards reducing the cost of offshore wind by 10%. One of the cornerstone initiatives of the OWA is to promote innovative and efficient sub-structures, which have the potential to reduce the cost of offshore wind energy. These concepts receive the support needed to bring these new technologies to market. The Carbon Trust ran a major international design competition to determine the most promising foundation concepts that could be used at the next generation of offshore wind farm sites. The four successful concepts to emerge from this competition consisted of a light weight twisted jacket, a large integral gravity base, the suction bucket concept, and a tri-suction pile braced sub-structure. These concepts can be considered the state of the art in foundation technology for wind farm applications, and it is likely that one or more of these concepts will advance to market for serial production in the very near future and be used to support the next generation turbine concepts. It is worth noting that these novel foundation solutions have all been developed with current turbine technology in mind and have not been developed to consider the loading applied by turbine machines greater than 10MW. Therefore, there is considerable scope to identify the most efficient substructure concepts for future high capacity turbines.

All of the foundation concepts discussed above have specific design challenges that need to be addressed and each of these design issues requires the input of different seabed parameters. The INFOMAR dataset has the potential to de-risk certain foundation solutions and facilitate early decision making regarding the most suitable foundation options, which in turn will improve the project feasibility and reduce the cost of finance. The specific design issues are discussed below.

2.6 Design Issues

The offshore wind sector is developing relatively quickly. Many of the design methods used in the industry were developed for the offshore oil and gas sector where loading conditions and serviceability considerations are quite different. In order to understand where enhanced geotechnical and geological data fits within the risks associated with developments it is important to highlight some of the areas of greatest uncertainty in design.

2.6.1 Limit States

Although foundation design codes focus on avoiding the occurrence of the ultimate limit state (ultimate resistance), in practice, serviceability and fatigue limit states control the design of offshore wind turbine foundations. The serviceability limit state assessment should consider (i) the accumulated rotation/displacements over the lifetime of the structure and (ii) the foundation stiffness required for dynamic calculations and the impact of cyclic loading on this stiffness. The serviceability and fatigue limit states can vary depending on the nature of the project and the compliance of the structure, but in general, strict tolerances are required. For example, Peire et al. (2009) described the design of the gravity based foundations used to support turbines at the Thornton Bank wind farm, where the design rotation under cyclic loading was limited to 0.25°. Stricter tolerances are required in Chinese projects where the design codes which specifies that the accumulated angular rotation due to cyclic loading must remain below 0.17°. In general, the vast majority of design codes specify a tolerance for cyclic induced movements. However, the codes do not include a prescriptive methodology to undertake the required analysis. This knowledge gap is forcing engineers to employ cyclic loading and fatigue models which were developed for the oil and gas sector (Seidel and Coronel 2011). A quasi-static approach is often adopted in practice where the pile length is increased until a situation where toe-kick (pile toe movement under lateral loading) of zero is achieved (Germanischer Lloyd 2005, Faber and Klose 2006). The intention here is that by maintaining no toe movements under the extreme static case, the result should be a design that will not undergo any permanent movements under lower magnitude cyclic loading. The DNV code proposes a less stringent methodology whereby the pile length is increased until further increases in length have minimal impact on the displacement response under ultimate static loading. The pile is then deemed sufficiently rigid to resist cyclic loading. Clearly, these quasi-static load approaches are not a robust means for considering cyclic loading, and could potentially lead to unsafe designs in some circumstances and uneconomic designs in others.

2.6.2 Dynamic Response

Another significant difference between offshore oil and gas platforms and renewable energy structures is the relative flexibility and dynamic constraints. In order to assess the dynamic sensitivity of a structure it is critically important to accurately predict the foundation stiffness to ensure that the natural response frequencies of the structure do not interact with the excitation frequencies from the turbine and the forcing ocean waves, see Houlsby et al. (2005). The turbine's first natural frequency has to avoid low frequency ocean waves, the frequency band corresponding to the rotor rotation (1P) and the frequency band corresponding to each blade shadowing the tower (3P). For soft-stiff compliant structures such as monopiles, the 1st natural frequency is designed to fall between the 1P and 3P range, which means that slight changes in the foundation response with the number of cycles (either increasing or decreasing stiffness) could lead to dynamic interaction effects that result in catastrophic failure of the structure. For stiffer structures that are designed with natural frequencies higher than the 3P, it is very important to ensure that cyclic stiffness degradation does not shift the natural response frequency toward the 3P excitation band.

2.6.3 Axial Pile Capacity

The evolution of offshore foundation design practice has been driven by the oil and gas industries and is reflected directly in periodic updates to the industry standard American Petroleum Institute (API) RP2A (2007) design code. Although the API approach for pile design adopts a theoretical framework based on effective stress analysis, a number of empirical parameters are introduced to modify this theory for practical application. Reviews of the API design approach by Chow (1997) and Gavin (1998) suggest that these empirical design methods are amongst the least reliable available. In some circumstances, the methods are overly conservative; in others they are un-conservative and result in unsafe designs (See Gavin et al. 2011). More reliable methods to estimate the ultimate resistance of foundations proposed by Lehane et al. (2005) and Jardine et al. (2005) were developed largely from experiments performed by Lehane (1992), Chow (1997), Lehane and Gavin (2001) and Gavin and Lehane (2003). The ultimate capacity of a geotechnical structure occurs at very large displacements. In practice, what is of real concern to wind turbine designers is the response at low-strains, particularly to cyclic loading.

Little if any attention is given to this matter in the design codes. More advanced models which can capture (i) the initial load-displacement response and (ii) the unloading behaviour are urgently required. Gavin and Lehane (2007) and Gavin et al. (2009) present a design framework to predict the load-displacement response of shallow and deep foundations to static loading. This offers a framework for analyses which could be extended to consider the response of offshore foundation structures.

2.6.4 Cyclic Loading

The Offshore Geotechnics Technical Committee TC 209 of the International Society for Soil Mechanics and Geotechnical Engineering (along with the corresponding ISO and API groups) has identified the urgent need for a better understanding of, and guidance on, cyclic loading for foundation design. This has also been recognised by the French National SOLCYP project described by Puech et al (2012). A review of the current state of the art in this area presented by Jardine, Andersen and Puech (2012) showed that foundation cyclic effects are more important than had been appreciated and must be considered very carefully for dynamically sensitive wind turbine structures.

As oil and gas exploration moved offshore, the impact of cyclic wave loading stimulated considerable amounts of worldwide research into cyclic soil behaviour. The Norwegian Geotechnical Institute (NGI) initiated a programme of experimental testing that has spanned the past 30-40 years and has concentrated on the impact of waves on oil/gas structures such as gravity platforms and tension leg platforms. These experimental tests were supplemented by practical foundation designs of fixed platforms, See Bjerrum (1973). More recently Andersen (2009) provided an overview of the issues associated with cyclic loading of offshore structures, where a framework for considering cyclic soil response in response to particular stress paths was presented.

However, because of the particular regional geology, the vast majority of work focused on cohesive soils, with a limited body of work available for granular soils. The NGI design methodology proposes the use of cyclic interaction diagrams to assess the cyclic design life of various offshore structures. However, these diagrams tend to consider simple loading cases (constant stress cycles), which rarely extend to a sufficient numbers of cycles (10^7) or consider appropriate stress conditions for wind turbine applications.

Despite a large body of experimental work undertaken to assess cyclic response and the corresponding cyclic failure diagrams for shear strength degradation of clay, silt, and sand that have been established from direct simple shear (DSS) and triaxial laboratory tests, specific tests are needed to consider stress paths adjacent to wind turbine support structures subjected to fatigue loading.

Laboratory scale testing that assesses the soil-structure response of model offshore suction caissons have been presented by Villalobos (2006) and Senders (2009). This work focussed on the cyclic response of caisson foundations subjected to storm loading conditions, with applied loads ranging from 10 to 1000 cycles. LeBlanc et al. (2010a, 2010b) performed a series of cyclic load tests on stiff model monopiles (at approximately 1:100 scale) with up to 65,370 cycles applied. Similar work was undertaken by Zhu et al. (2012) who presented a study on model caissons in loose sand where approximately 10,000 cycles with different loading characteristics were applied to the foundation to assess the accumulated angular rotation. A generalised framework was proposed to describe the observed response, where the main findings concluded that: (i) the accumulated settlement increased with the number of cycles, (ii) the accumulated settlement increased with the cyclic amplitude, (iii) the most significant cyclic settlements were observed during intermediate loading conditions that involved some stress reversals but not complete 2-way cyclic conditions, (iv) the nature of the foundation movements was primarily through a deep seated rotation and lateral translation of the caisson body and (v) the cyclic unloading stiffness was unaffected by the number of cycles. Interestingly, the final observation regarding the unloading caisson stiffness is in sharp contrast to the observations made by LeBlanc for stiff pile behaviour, where up to a 60% increase in stiffness was observed over the lifetime of a monopile supported wind turbine.

2.7 Foundation Detailed Design Process

The foundation design process involves (i) concept design; (ii) preliminary design and (iii) detailed design. The site specific environmental and ocean conditions effect all three of these design phases and the level of foundation risk.

As described in the previous section, the bathymetry is a significant factor in determining the concept design and the water depth can often result in several concepts being excluded from an early stage. The foundation concept design is also controlled by the seabed geology and seabed features, with specific features often suggesting that one foundation may be more suitable than an alternative. For example, at sites with significant seabed gradients (e.g. slopes greater than 5 degrees), shallow foundations such as gravity bases may not be suitable. At sites with significant boulder content, suction caissons may not be suitable. These site specific geological and geotechnical constraints are often a development risk, due to the lack of suitable geotechnical information at the project conception. However, as the project develops and more information is acquired, the magnitude of these risks decrease and a preferred (reduced risk) solution often becomes apparent. The preliminary and detailed design processes will involve taking specific foundation concepts through to engineering and analysis, where the foundation elements will be sized and fully costed. This will often involve a detailed soil-structure interaction design and will require location specific sampling at each turbine location and in-situ field testing, coupled with high quality lab testing on undisturbed soil samples.

2.8 Additional Factors to Consider

When considering whether a site is suitable for various wind farm foundation concepts, it is important to consider the method of installing the foundations and turbine components. To date, all offshore turbines have been installed using heavy lift jack-up vessels, which requires a suitable site where the jack-up rig can safely lower the legs into the seabed and achieve a stable working platform. There are many areas of the North Sea that are not suitable for jack-up vessels due to local variability and the presence of deep deposits of soft soil. At these locations, it is significantly more difficult to assess the foundation risk, because the entire site is very sensitive to the choice of installation vessel.

Figure 2.11: Installation using Jack-up vessel

3. INFOMAR DATA REVIEW

3.1 Industry Data Requirements

At a detailed design level, high quality sampling and laboratory testing is required to provide input into different design limit states, which include; consideration of ultimate limit state (failure), settlement and rotation (serviceability) and for cyclic dynamic loading, consideration of fatigue design. However, at a strategic planning and site selection level, information on water depths and soil classification would provide sufficient information to de-risk most foundation selections. This information will be assessed separately below.

3.2 Bathymetry

A significant amount of Bathymetry data is available for potential sites around Ireland. The data extends to hundreds of kilometres off the West coast into the deep Atlantic Ocean, where water depths approaching 5 km are observed (See Figure 3.1). Information for the Irish Sea is less comprehensive, with the data focusing on the near shore environment along the East coast.

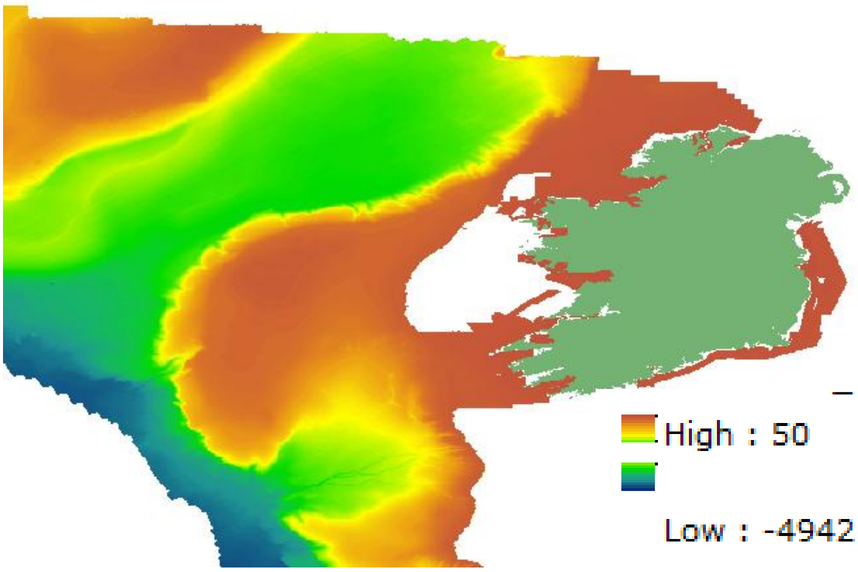


Figure 3.1: Water depths around Irish Coast

3.3 Soil Classification

Over the past number of years, the INFOMAR surveying programme has obtained a vast amount of seabed information, including physical sediment samples. These may be from the surface where grabs are used or may penetrate through the seabed and retain the vertical structure of the sediment by using various coring methods. The various techniques that have been employed by the INFOMAR programme are summarised in Table 2 below.

Table 2: Ground Truthing Methods

	Grab Sampler	Box Corer	Vibrocore
Principle	A bucket is used to grab a sample from the immediate surface sediments on the sea floor.	This comprises of a cylinder or box which has a weight attached to it within a frame. This is left to free fall through the water column and under the force of gravity is driven down through the seabed. A hinged grab then pivots to trap the sediment within the box which is then recovered onboard.	The vibrocore is made up a base which sits on the seabed, a motor then creates vibrations that allows a metal cylinder (into which the plastic liner is inserted) to penetrate the sediments collecting a soil sample in the cylinder as it goes. When the core is recovered from the seabed, the plastic liner is removed with the undisturbed sediments safe inside. These are later cut in half, photographed, scanned and analysed.
Depth Range	approx. 200mm	300-400mm (Dependent on soil)	3-6 m
Equipment			
Sample Quality	Fully disturbed bulk sample	Relatively undisturbed shallow sample	Relatively undisturbed shallow samples

A large number of surveys have been undertaken to collect a variety of samples, which have all been logged and recorded on the INFOMAR database. The soil samples have been classified to determine the primary soil type, whether course or fine grained, etc. and this information is plotted on Figure 3.2 below. There is clearly a significant spread of data collated from around the Irish coast.

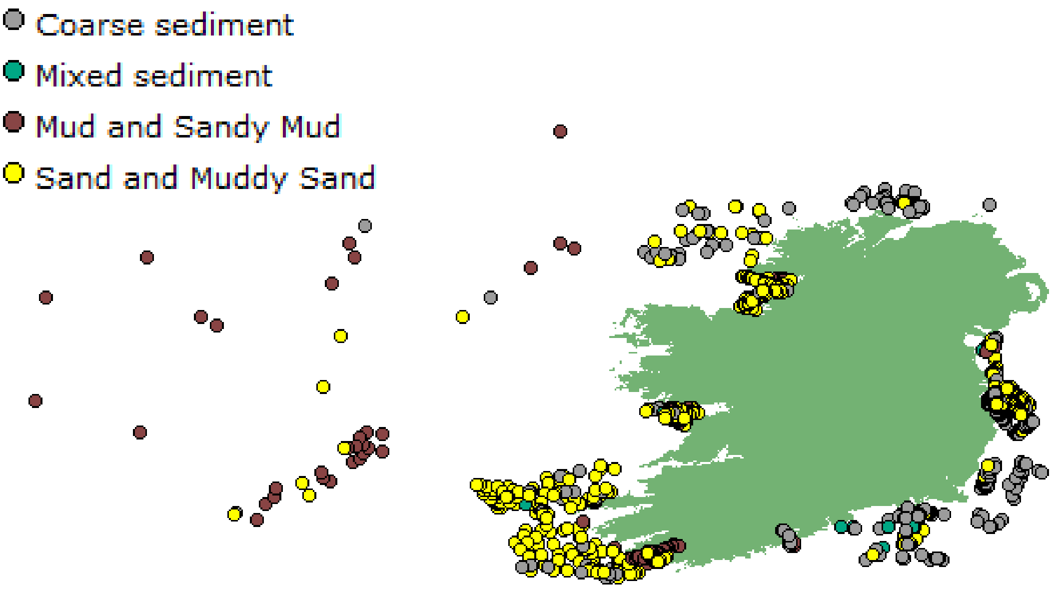


Figure 3.2: Seabed Classification

It is worth noting that Figure 3.2 classifies the soil into different categories depending on the relative coarseness of the material. However, an explicit methodology for defining the material type has not been presented in the INFOMAR material. A standard method for classifying the material would be very useful to confirm that all material is compared on the same scale.

3.4 Data Gaps

3.4.1 Spatial Extents

The spatial extents of the bathymetric coverage observed in Figure 3.2 are shown to cover a high proportion of the Irish Atlantic ocean area. This Figure also indicates significantly shallower water depths off the Eastern coast than in the near shore Atlantic Ocean. As a result of the shallower water and the ability to construct wind farms in these conditions using conventional technology, it is most likely that the first significant offshore wind farms will be developed in the Irish Sea. However, in this area of the Irish Sea, which is recognised in the OREDP (2010) document as being the most suitable for immediate development, the bathymetric coverage is not as extensive with most of the surveys terminating quite close to shore. As sites greater than 5 km from shore are most likely to be the critical development areas, a data gap exists. This could be remedied by undertaking further bathymetric surveys further from shore.

3.4.2 Sufficient Depth Information

One of the major deficiencies in the available information is the lack of geotechnical data present at significant depths. The zone of influence for wind farm foundations refers to the depth of soil that impacts on the foundation performance and for a monopile this zone is controlled by the penetration depth. Monopiles are commonly driven to 30 meters below the seabed and therefore geotechnical information is required for at least 30 meters. Jackets and Tripods piles could be driven to 80 or 90 meters depth, whereas gravity bases will have zones of influence of approximately 40 meters. So regardless of the foundation concept, significant geotechnical information is required over at least the upper 30 meters of soil in order to determine the most suitable foundation concept. The available soil information has been determined from a mix of grab sampling, box coring and vibrocore samples, and therefore extends to a maximum of 6 meters depth.

3.4.3 Quantitative Information

The information available to date has focused on identifying the type of soil conditions at offshore locations and has largely ignored the geotechnical properties of these soils. For example, soil is classified as fine grained based on visual inspections but does not always undergo sieve testing to determine the percentage clay and/or silt content. Because natural soils are often comprised of a variety of particles, the engineering behaviour depends critically on knowing the relative proportions of coarse and fine grained particles present. There is also a lack of strength data available for the existing soil samples, and the accuracy of any lab testing conducted on these samples would also be called into question due to the sample quality. Whilst obtaining high quality soil samples from depth and testing these in the laboratory would be prohibitively expensive, it is recommended that Cone Penetration Testing could be used as a simple and efficient means of obtaining strength profiles with depth of the underlying materials. The CPT test measures three different parameters, the tip resistance, the sleeve friction and the pore pressure response, and depending on the relative ratios of these measurements, the soil type can also be determined. This is a very useful tool for informing strategic locations of wind farms and can be carried forward into the detailed design process.

3.4.4 Time Dependent Information

The information available to date does not consider the time dependent behaviour of the seabed sediments. A critical issue for designing wind farm foundations in granular deposits is the dynamic sediment characteristics and the potential for the seabed to change due to processes such as moving sand waves. As the turbines need to survive a 30 year design life, it is important that these dynamic characteristics are considered from the outset of the project. However, as the data coverage to date has focussed on assessing different areas of seabed, little attention has been given to repeat surveys in the same areas to assess the seabed mobility over time. These dynamic processes, known to be prevalent along the East coast, will have particular implications for the design of scour protection systems.

4. ANALYSIS OF INFOMAR DATASET FOR OFFSHORE WIND

4.1 Introduction

The study area has been split into the six assessment zones around the Irish coastline outlined in the Irish Department of Communications, Energy and Natural Resources Offshore Renewable Energy Development Plan (OREDP 2010). The division of the coastline into these assessment zones allowed for an in-depth study on the suitability of offshore wind development up to a distance of 100km from shore. This buffer distance was chosen as it is the upper length limit of conventional AC cable technology that is currently in use. As the Shannon assessment zone has primarily been identified for the development of tidal energy converters it has been excluded from this study. The accumulated maps are presented in Appendix B.

The Bathymetry, soil property data and ship wreck locations were downloaded from the INFOMAR Interactive Web Data Delivery System and processed using ESRI ARCGIS. Information on constraints and any other relative information were gathered from the following resources.

- Offshore Renewable Energy Development Plan (2010)
- Offshore Renewable Energy Site Suitability Mapping Report (ORESSuM)
- Irish Nation Parks And Wildlife Service (www.npws.ie)
- Irish Marine Institute (www.marine.ie)
- Sustainable Energy Authority Ireland (www.seai.ie)

4.2 Data Coverage

The available offshore data was mapped and analysed to determine its suitability in the design of foundation structures for offshore wind turbines. This study found that good quality, high resolution data (5-10m grids) is available for large areas of the Irish coastline. Large quantities of Ground Truthing (GT) data points are also available in certain areas. However, there are large portions of the assessment zones with little Bathymetry and almost no soil classification data available. The available GIS bathymetry and soil classification data was then subjected to a statistical analysis to quantify the coverage of the existing data. The results of which are presented in Table 3.

Table 3: Summary of INFOMAR recorded data coverage

Assessment Zone	Zone Area (km^2)	Soil Classification (km^2)	Primary soil type	No. of GT Data Points*	Bathymetry Coverage (km^2)	Bathymetry Statistics	Distance containing data** (km)
East Coast - North	4181	907 (21.7%)	Fine to medium sands	230	2020 (48.3%)	Min = 2.9m Max = 135m Mean =43.6m	36
East Coast - South	6402	470 (7.3%)	Coarse sand and gravel	80	2456 (38.4%)	Min = 3m Max = 135m Mean = 44m	35
South Coast	25439	1279 (5.0%)	Gravel and course sand	12	1802 (7.1%)	Min = 4m Max = 79m Mean = 38m	10 – 23
West coast - South	14615	528 (3.6%)	Medium dense sand	245	7605 (52.0%)	Min = 5m Max = 374m Mean = 93m	40 - 70
West Coast	21557	800 (3.7%)	Fine to medium sands	489	5538 (25.7%)	Min = 5m Max = 192m Mean = 90m	70 – Kerry 30 - Mayo
West Coast – North	24047	637 (2.6%)	Medium dense sand	310	21898 (91.1%)	Min = 4m Max = 374m Mean = 93m	100
SUMMARY	96241	4621 (4.8%)		Tot.= 1366 Ave. = 228	41319 (42.9%)		
Notes:	* GT = Ground Truthing and Grab Samples ** Distance from shore to end of survey data						

The table shows that soil classification data is only available for approximately 4.8% of the total combined assessment zone area. The assessment zone with the greatest coverage is the East Coast – North with 21.7% coverage. The zone with the smallest percentage coverage is the West Coast – North with only 2.6% of the assessment zone classified. However, a large number of ground truthing samples have been taken from this zone which can be used in the initial assessment of potential development sites. There is a dearth of soil classification information for the East Coast – South which has been identified as a primary area for the further development of offshore wind farms (see Table 4).

There is significantly more bathymetric data available for each assessment zone with coverage ranging from 25.7% to 90.1%. However, there are significant gaps in data in both the West Coast and South Coast assessment zones.

4.3 Assessment Zones

The accumulated data was then assessed for each zone to determine the design potential of each zone. The existing and proposed offshore wind farms were also considered in the investigation. The proposed offshore wind developments in each assessment zone are presented in Table 4.

Table 4: Proposed offshore wind farm developments

Assessment Zone	Existing Developments	Proposed Developments	Capacity	SEAI Total Capacity For Zone
East Coast - North	-	Dublin Bay Oriel Wind	364MW 330MW	1200 -1500MW
East Coast - South	Arklow Bank I	Codling Bank Arklow Bank II	1100MW 520MW	3000-3300MW
South Coast	-	-	-	1500-1800MW 6000MW*
West coast - South	-	-	-	600-900MW 5000-6000MW*
West Coast	-	Sceirde Wind	100MW	500MW 7000MW*
West Coast - North	-	-	-	3000-4500MW 7000-8000MW*
Notes:	* Floating Wind Energy Potential (SEAI OREDP2010)			

An initial investigation into existing constraints in each assessment zone was also conducted to identify possible constraints that may affect the choice of foundation utilised. A brief summary of some of the relevant constraints for each zone are presented in Table 5.

Table 5: Potential development constraints

Assessment Zone	Shipwrecks	Navigation Channels[1]	SPA [2]	SAC [2]	Exclusion Zones	Other[3]
East Coast – North	≈39	Dundalk Carlingford Dublin	Dundalk Bay Rockabil		DoM - Bettystown DoD - Drogheda	Fishing Pipelines Cables
East Coast – South	≈29	Wicklow Arklow Rosslare		Carnsore Point	DoM Exclusion Zone	Fishing Cables
South Coast	≈32	Waterford Cork Bantry	Saltee Islands Hook Head		DoM - Baltimore DoD– Clonakilty and Cork	Fishing
West coast – South	≈17	Fenit Shannon Dingle	Blasket Islands	Blasket Islands Bantry Bay	DoM – Shannon Estuary	Fishing Cables
West Coast	0	Galway		Kerry Head Clew Bay	-	Fishing
West Coast - North	≈110	Ballina Sligo Killybegs Derry	Inishtrahull Tory Island	Tory Island	-	Fishing Cable
Notes:	1 = SEAI Wind Atlas 2 = National Parks and Wildlife Services 3 = Offshore Renewable Energy Site Suitability Mapping			DoM = Department Of the Marine – See Wind Atlas DoD = Department of Defence		

For each of the assessment zones, an individual data coverage map is presented in Appendix B. The individual notes about each assessment zone that are presented below should be read in conjunction with the visual information contained in the Appendix B maps.

4.3.1 Assessment Zone 1 – East Coast North

There is Bathymetry data available for approximately 48.3% of the assessment zone up to a maximum water depth of 135 m. It can be observed that there is a steep increase in water depth approximately 18 km from the Dublin coast. Within 18 km of the coastline the water depth varies from 0-30 m and the ground conditions are predominantly fine to medium sands making the area suitable for gravity base and monopile foundation systems. However, some rock was detected approximately 5 km northeast of Skerries that may make piling difficult and will require further investigation. The deposits of rock outcrops also make the use of suction caissons unlikely. However, the lateral extent of these features needs to be quantified by more detailed surveys. At distances of greater than 18 km from shore, the water depth increases and jacket or tripod foundation systems may be required. At distances of greater than 45 km east of Dublin bay, floating foundation systems may be required. As the ground conditions suggest, with soft sands in this area, a torpedo anchor or gravity anchor foundation system may be utilised to anchor the floating wind turbines.

Important points to note about this area include;

- Average Wind Speeds are 8-10 m/s at 100m above sea level (SEAI Wind Atlas).
- Dublin bay and Drogheda bay are regarded as priority bay areas.
- A large number of soil grab samples have been taken from the entire area – indicating course and sandy sediments.

Important constraints which apply include;

- There are approximately 39 ship wrecks in the area.
- Navigation channels exist from Greenore, Dundalk, Dublin Port and Dun Laoghaire.
- There is a Department of the Defence exclusion zone east off Bettystown
- A Department of the Marine Exclusion zone exists east off Drogheda.

4.4 Assessment Zone 2 – East Coast South

The bathymetric data shows large areas with water depth of less than 45 m off the Wicklow coast. A substantial amount of the Wicklow coastline has water depths of less than 30 m, within a distance of approximately 20 km from shore. These conditions make the area suitable for monopiles, suction caissons or gravity base foundations. However, little or no soil classification data is available for this region making an initial assessment of foundation solutions difficult. Some grab sample results suggest varied deposits of course sediments throughout the assessment zone. For a more detailed initial assessment of potential foundation solutions further soil classification data is required. It is worth noting that the existing offshore wind farm in Arklow was constructed on monopile foundations. As the region is particularly susceptible to coastal erosion and sand transport, further research will be required to investigate the erosion and deposition processes in the area which may cause movement of the various sand banks located along the East coast and the effect of scouring on foundation performance.

Important points to note about this area include;

- Average Wind Speeds are 8-10 m/s (increasing to 10.5 in southern areas) at 100 m above sea level (SEAI Wind Atlas).
- Wexford Harbour is regarded as a priority bay area and is within the newly designated south priority area.
- Very few soil grab samples have been taken from the area.
- There are approximately 29 known ship wrecks in the area, although no major concentrations have been identified.

Important constraints which apply include;

- There are fishing restrictions due to Cod spawning areas.
- Navigation channels exist from Rosslare Harbour and Dunmore East.
- There is a Department of the Marine exclusion zone at Rosslare Harbour.

4.5 Assessment Zone 3 – South Coast

The recorded bathymetric data shows that within this zone water depths can reach up to 50 meters at distances of 10 km from shore. There has been a trend in recent years for offshore wind farms to be constructed further from shore to minimise the visual impact and are therefore generally consented at distances of 10-20 km from the coast. However, there are many examples of wind farms (past and current) that are planned very near the coastline. Considering the most likely distances from shore, most potential developments in this zone will require either jackets/tripod constructions or floating foundation concepts in the particularly deep water sections. The available geological data classifies most of the sea bed as gravel to course sands which may be suited to either gravity or torpedo anchors. As rock outcrops are also scattered intermittently among the sand and gravel strata, the use of rock-socketed anchors and gravity anchors may also need to be investigated. The installation of anchors into rock will require a quantitative investigation of the geological fracture patterns and geotechnical strength properties of the rock.

Important points to note about this area include;

- Average Wind Speeds are 8.5-10.5 m/s (increasing to 11 in southern areas) at 100m above sea level (SEAI Wind Atlas).
- There are several bays and harbours along the coastline.
- There is a deep water dock at Cork City.
- Grab samples were only available within Cork Harbour.

Important constraints which apply include;

- All waters west of Waterford Harbour have been designated as biologically sensitive areas as they are important breeding and spawning grounds for several species of fish (Department of Agriculture fisheries and food - 2009)
- There are approximately 32 known ship wrecks in the area, with a small concentration south of Waterford Harbour

4.6 Assessment Zone 4 – West Coast South

The water depth increases rapidly with distance from shore where it can reach depths of 50 m within 2 km from the coastline. The average depth in the assessment zone is 93 m and the maximum depth is 374 m. Therefore, floating wind turbines would have to be utilised for the majority of the zone. As very little soil classification data (3.6% of zone) is available, little discussion can be made on a suitable method of anchoring the floating turbines. The Dingle Peninsula and the area around Bantry Bay are two locations where the water is shallow enough for conventional foundation methods such as monopiles, gravity bases and jacket structures to be used. However, it is highly unlikely that the area will be developed due to fishing traffic to Dingle, Castletownbere and Bantry.

Important points to note about this area include;

- Average Wind Speeds are 6.5-8 m/s between peninsulas, offshore wind speeds vary from 9 – 11 m/s (increasing with distance from land) at 100m above sea level (SEAI Wind Atlas).
- There are several bays and harbours along the coastline.
- There is a deep sea port at Fenit Harbour.
- A large number of soil grab samples have been taken from the northern portions of the assessment zone indicating sands and muddy sediments.

Important constraints which apply include the following;

- All waters in this assessment zone have been designated as biologically sensitive areas as they are important breeding and spawning grounds for several species of fish (Department of Agriculture fisheries and food - 2009).
- There are over 17 known ship wrecks in the area, with no concentrations identified.

4.7 Assessment Zone 5 – West Coast

Bathymetric data is limited in this assessment zone and tends to be concentrated in the south of the zone and the bays and estuaries along the Mayo and Galway coast. Water depths at 10km off the north Kerry and Galway coast line are approximately 50 m. Bathymetric data in the location of the proposed Sceirde Wind farm is not available. However, it can be estimated to be in the region of 30-50m, making suction caissons, jacket structures and gravity base foundations possible foundation solutions in the future. Without accurate soil classification and bathymetric data it is difficult to predict suitable foundations. Detailed bathymetric data is only currently available for Galway Bay and the Aran Islands. This area, east of Inishmore, has 100m wind speeds of between 9-10 m/s and water depths ranging from 20-50m. The ground conditions in the area are primarily fine to medium sands with intermittent rock outcrops suggesting that gravity foundations and jacket structures may be used. However, in rocky locations, the jacket foundations may need to be "rock-socketed" to the sea bed and detailed geological information will be required. As regular ferries run to the Aran Islands and there is fishing and recreational boats berthing in Galway Harbour, the location and potential of this site may be limited. As soil classification data is only available for 3.7% of the assessment zone area it is difficult to assess near shore foundation solutions. It is known that water depths increase rapidly with distance from shore in this zone; water depths of 60 m are found within 20 km of the Mayo coast. Therefore, floating wind technology may need to be utilised in order to harness the full potential energy of the zone.

Important points to note about this area include;

- Average Wind Speeds are 9-11m/s (increasing with distance from land and in the north west of the assessment zone) at 100m above sea level (SEAI Wind Atlas).

- Several bays and harbours are located along the coastline.
- Large number of soil grab samples have been taken in the Galway Bay area and a small concentration have been taken along the border with assessment zone 6 which indicate varying sands and course sediments.

Important constraints which apply include;

- The Southern region of the assessment zone has been designated as a biologically sensitive area as it is an important breeding and spawning grounds for several species of fish (Department of Agriculture fisheries and food - 2009)
- No shipwrecks have been identified in the assessment Zone (INFOMAR web access).
- Navigation channels exist to the Aran Islands and fishing and recreational traffic is important to Galway bay.

4.8 Assessment Zone 6 – West Coast North

Bathymetric data in this assessment zone shows water depths of up to 374 m. The average water depth in the region is 93 m. Therefore, for large scale exploitation of the wind resources in this assessment zone, floating substructure technology will need to be explored. Soil classification data is only available for approximately 2.6% of the assessment zone and is concentrated in the Donegal Bay area and indicates the presence of varying sand deposits with some rock surrounding the islands. The bathymetric data for this location (Figure 4.1) shows a large area with water depths ranging from 30 m to 80 m off the Donegal and Sligo coasts that may be exploited using a combination of monopile, gravity base and jacket foundations. Suitable water depths for these foundations can also be found off the northern coast of Donegal. Initial investigations of the ground samples suggest that course and mixed sand sediments are also found off the northern Donegal Coast but more detailed soil classification data is required for further investigation into the most suitable foundation solution.

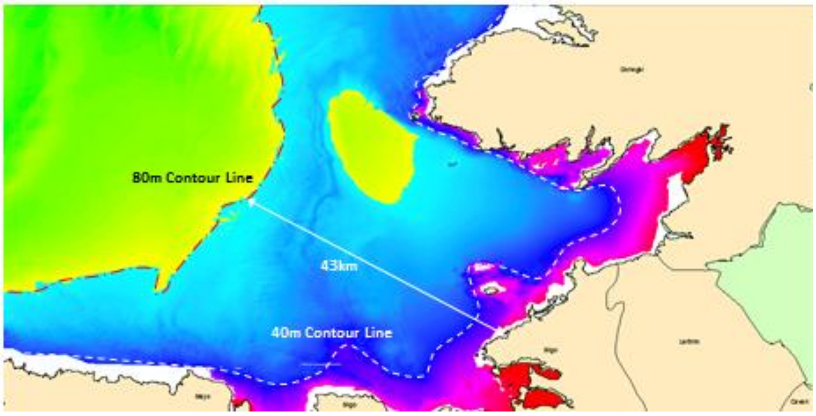


Figure 4.1 Donegal Bay bathymetric contours

Important points to note about this area include;

- Average Wind Speeds are 8-10 m/s in Donegal Bay, increasing to 11m/s further out to sea at 100m above sea level (SEAI Wind Atlas).
- Several bays and harbours are located along the coastline, with major fishing ports and a deep water harbour at Killybegs.
- A large number of soil grab samples have been taken from the entire area suggesting sands and course sediments.

Important constraints which apply include;

- There are over 40 known ship wrecks in the area – with a large concentration identified in the north of the assessment zone.
- There is significant commercial fishing traffic from Killybegs and Sligo Harbours.
- There are underwater cables crossing north of Donegal.

5. RECOMMENDATIONS – CPT SURVEYING

5.1 Gap Analysis

The analysis of the INFOMAR dataset in the previous section highlighted a number of limitations of the existing survey measurements. In particular, (i) there was limited soil information over the depth range that is relevant to wind farm foundations; (ii) the information that is available does not contain quantitative parameters that can be used in design and (iii) where soil classification information is available, there is no clear framework defining the process to classify the seabed material. Considering these limitations it is recommended that Cone Penetration Testing (CPT) be undertaken to address the data gaps in the existing INFOMAR dataset.

5.2 The CPT Test

The CPT test consists of an instrumented cone connected to the end of a series of rods, which are pushed into the seabed at a constant rate. Measurements of the cone tip resistance (q_c), and the frictional resistance (f_s) along the outer surface of a cone sleeve located behind the cone tip are measured during installation. In the piezocone penetrometer, the pore pressure is often measured at up to three locations. For the purpose of this report, all references to the CPT test will refer to the piezocone test that includes pore pressure measured at the u_2 location behind the cone shoulder.

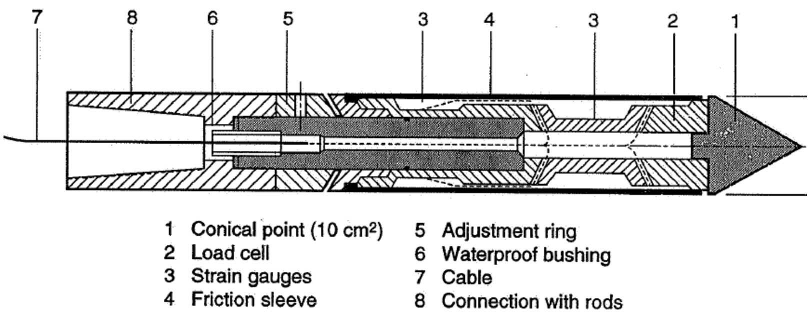


Figure 5.1: Sample CPT Piezocone Penetrometer (Source: Robertson 1997)

In the offshore sector CPT tests are performed in either the seabed mode, where the cone is pushed from the seabed until refusal, or the drilling mode, where the cone is pushed from the base of pre-drilled borehole.

The first offshore CPT using the Seabed mode was developed by Fugro in 1966 and early designs included the *Seabull* rig (shown in Figure 5.2). Since then many lighter and more advanced units have been developed by Fugro and several other geotechnical investigation companies. Coiled CPT rigs have since been developed by IFREMER that have been used at water depths of 6000m with penetration depths of up to 30m (Lunne 2010). Robotic "mini-CPT rigs" have also been developed and have allowed for the use of smaller less expensive ships and easier transportation.

Figure 5.2: Fugro's Seabull rig (Source: Lunne 2010)

Drilling Mode or "down-hole" CPT tests can allow for much deeper penetration depths as the drill assists the cone to the desired depths and also allows for hard layers to be drilled through. Once the desired depth is reached the CPT cone is pushed through the drill bit into the subsoil. A new system called the CPT-while drilling (CPTWD) was developed by SPG and ENVI where the cone penetrometer protrudes in front of the drill bit during drilling. The CPT data as well as the drilling parameters (drill bit load, torque load, fluid pressure and penetration rate) are recorded in real time and interpreted to determine the soil properties.

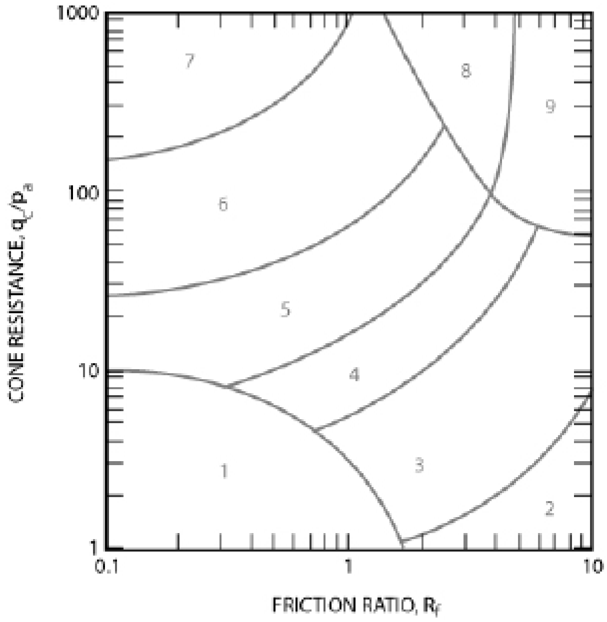
5.3 General CPT Framework

A CPT testing regime should be established whereby cone tests are undertaken at locations with the highest potential to develop offshore wind farms in the near future. These wind farms will all be constructed on fixed bottom structures, as floating technology is unlikely to be commercially deployed before 2020. A number of areas around the Irish coast would benefit from CPT testing and these locations are summarised in the maps presented in Appendix C. In particular, CPT tests should be targeted at the areas located between the 5 km and 20 km contour lines in water depths up to 50 meters. Appendix C indicates that the East coast is most suitable for a CPT survey, with a couple of bays near Galway and Donegal also proving suitable for testing. To establish a holistic geotechnical and geological interpretation of the underlying soil deposits and to develop a baseline understanding of the offshore deposits, it would be useful if the first few CPTs deployed as part of this framework were undertaken at well characterised areas with significant amounts of geophysical and ground truthing information. Ideally, a number of CPTs would be conducted adjacent to existing borehole records where a complete vertical stratification profile has already been established to a significant depth. This information could then be used as a calibration and validation exercise to confirm the reliability of the CPT test in the Irish deposits. The subsequent CPT tests could then be used with confidence to both characterise the soil deposits and to confirm their detailed engineering properties.

5.4 CPTs for use in classifying soil types

A major application of the CPT is for soil profiling, classification and geostratigraphy. This is possible through a holistic interpretation of the discrete measured parameters which were introduced above (i.e. the tip resistance, q_c, the friction ratio, fs and the pore pressure, u_2). For the most commonly encountered soil types, the tip resistance, (q_c) is high in sands and low in clays, and the friction ratio ($R_f = f_s/q_c$) is low in sands and high in clays. The pore pressures are also usually higher in clays than sands. The CPT provides a consistent and repeatable guide to the mechanical characteristics (strength and stiffness) of the soil in the immediate vicinity of the advancing probe; this is commonly termed the soil behaviour type (SBT). This is the fundamental basis for developing a classification system around the CPT measurements.

One of the earliest classification systems based on CPT data was formalised by Robertson et. al in 1986 and was based on comparing the cone resistance, qc, directly against the friction ratio, Rf. This classification system was proposed in the form of the chart presented in Figure 5.3 below.



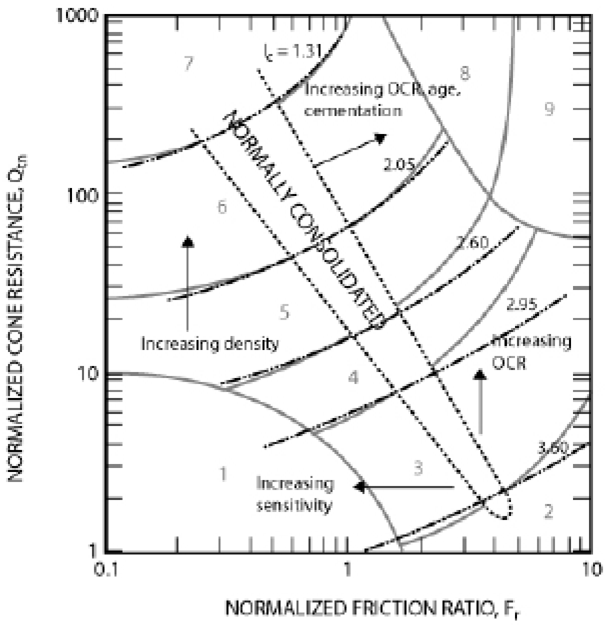
Zone	*Soil Behavior Type*
1	*Sensitive, fine grained*
2	*Organic soils - clay*
3	*Clay – silty clay to clay*
4	*Silt mixtures – clayey silt to silty clay*
5	*Sand mixtures – silty sand to sandy silt*
6	*Sands – clean sand to silty sand*
7	*Gravelly sand to dense sand*
8	*Very stiff sand to clayey sand**
9	*Very stiff fine grained**

** Heavily overconsolidated or cemented*

P_a = atmospheric pressure = 100 kPa = 1 tsf

Figure 5.3: CPT Soil Classification Chart proposed by Robertson et. al (1986)

The classification system proposed by Robertson et al. in 1986 provides reasonable estimates of the soil behaviour type for CPT soundings up to 20 m depth. However, the above chart did suffer from one major limitation in that the vertical stress dependence was not considered. As the cone tip resistance and sleeve friction both increased with depth due to the increasing overburden pressure (vertical overburden stress, σ_{v0}. The CPT profile needs to be normalised with respect to σ_{v0} so that both shallow and deep soundings can be compared within the same framework. To account for the vertical stress dependence Robertson proposed a normalised chart in 1990, which is presented in Figure 5.4.



Zone	Soil Behavior Type	I_c
1	*Sensitive, fine grained*	N/A
2	*Organic soils – clay*	> 3.6
3	*Clays – silty clay to clay*	2.95 – 3.6
4	*Silt mixtures – clayey silt to silty clay*	2.60 – 2.95
5	*Sand mixtures – silty sand to sandy silt*	2.05 – 2.6
6	*Sands – clean sand to silty sand*	1.31 – 2.05
7	*Gravelly sand to dense sand*	< 1.31
8	*Very stiff sand to clayey sand**	N/A
9	*Very stiff, fine grained**	N/A

** Heavily overconsolidated or cemented*

Figure 5.4: CPT Soil Classification Chart proposed by Robertson et. al (1990)

An alternative chart (see Figure 5.5) was also presented by Robertson et al. (1990), which utilised a normalised pore pressure parameter, B_q rather than the friction ratio. B_q was determined from the excess pore pressure value, Δu, and the net cone resistance, q_{net}.

$\Delta u = u_2 - u_0$

$q_{net} = q_c - \sigma_{v0}$

u_0 = hydrostatic pore pressure

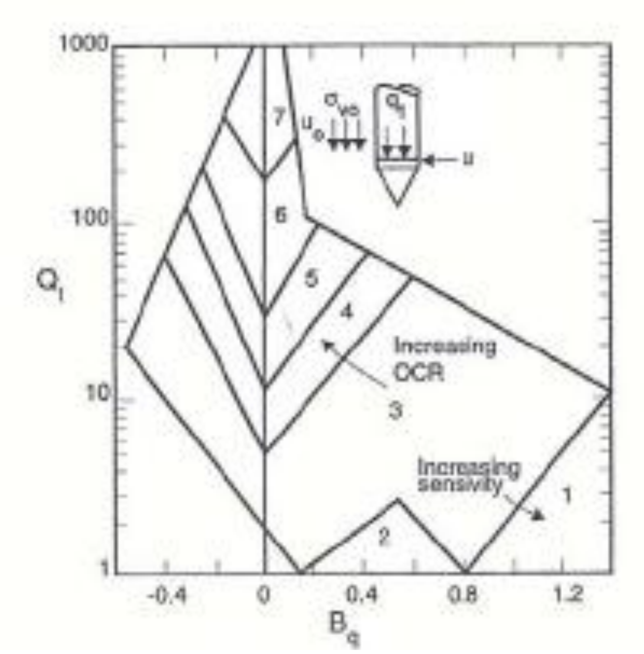


Figure 5.5: CPT Soil Classification Chart proposed by Robertson et. al (1990)

Where accurate pore pressure measurements are attained during the CPT test, as should be the case in an offshore sounding, the combined use of the charts presented in Figure 5.4 and Figure 5.5 will provide a more comprehensive interpretation of the soil classification.

It is proposed that CPT tests are conducted at discrete locations around the Irish coast and validated using drill records at a couple of locations. Once the reliability of the soil classification has been confirmed through physical testing and visual inspection, it is proposed to use the CPT test as a primary tool to assess the soil types present at other locations. By adopting the CPT based classification system and correlating the results to available geophysical measurements it may be possible to develop a secondary classification system that is based on geophysical correlations.

5.5 CPTs for use in detailed foundation design

The CPT tip resistance parameter (q_c) can be used directly and/or indirectly in the design of most foundation concepts. The specific approaches to CPT based design of different foundation concepts are described below.

5.5.1 Gravity Foundations

Due to their relatively low cost and proven capabilities, gravity foundations are widely used to support both onshore and offshore structures. Most textbooks and design codes including the American Petroleum Institute (API) and the Det Norske Veritas (DNV) recommend conventional bearing capacity approaches to calculate the ultimate bearing capacity (q_{ult}) of gravity or shallow footings on sand:

[1] $$q_{ult} = 0.5\, B\, \gamma\, N_\gamma\, s_\gamma\, d_\gamma + c' N_c\, s_c + \gamma\, D\, N_q\, s_q\, d_q$$

in which B is the footing width; γ is the unit weight of the ground, c′ is the effective cohesion, D is the embedment depth; N_γ, and N_q are bearing capacity factors that depend on the footing shape and the effective friction angle (ϕ') of the soil while factors s_γ, s_c, s_q, d_γ ,d_c and d_q take account of the footing shape and embedment depth. Having calculated the soil resistance, the footing settlement is often estimated using approaches which assume some elastic secant stiffness for the sand. It is worth noting that even when applying conventional bearing capacity formulae directly, the input parameters such as the friction angle can be determined from correlations with the CPT test parameters (this is referred to an indirect CPT approach). For example, Kulhawy and Mayne (1990) proposed the relationship given in Equation 2 to determine the friction angle for clean round uncemented quartz sands based on correlations between high quality triaxial test results and CPT field measurements.

[2] $$\phi' = 17.6^\circ + 11.\log[(q_c - \sigma_{v0})/\sqrt{(\sigma'_{v0.}\ \sigma_{atm})}]$$

Briaud (2007) argued that whilst conventional bearing capacity theory would produce good estimates of q_{ult} of footings in normally consolidated soil. In over-consolidated deposits the soils strength is often relatively constant with depth and the assumption that q_{ult} increases with footing width, B or footing depth, D is not valid. By compiling data from a number of full-scale footing tests he demonstrated that when the mobilised bearing pressure *q*, was normalised by an in-situ measurement of soil strength such as the Standard Penetration Test (SPT) *N* value or the Cone Penetration Test (CPT) end resistance (q_c) value averaged over the zone of influence of the footing, a unique normalised load-settlement response was obtained for a given site. This approach is illustrated in Figure 5.6, which shows the pressure-settlement response measured during load tests performed at Texas A&M University reported by Briaud and Gibbens (1999). The tests were performed on square footings where the width varied from 1 m to 3 m, and which were founded 0.75 m below the ground surface. The recently deposited medium-dense sand was in a lightly over-consolidated state (*OCR* ≈ 2) following the removal of about 1.0 m overburden depth. The mean CPT q_c resistance ranged from 5 to 7.25 MPa in the zone of influence of the footing.

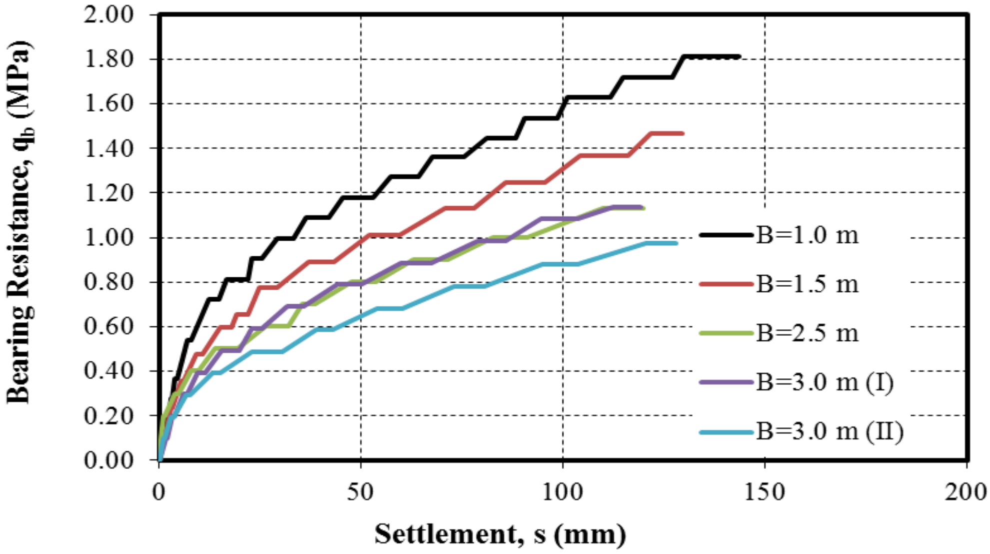


Figure 5.6: Bearing pressure mobilised during shallow footing load tests performed at Texas A&M (after Briaud 2007)

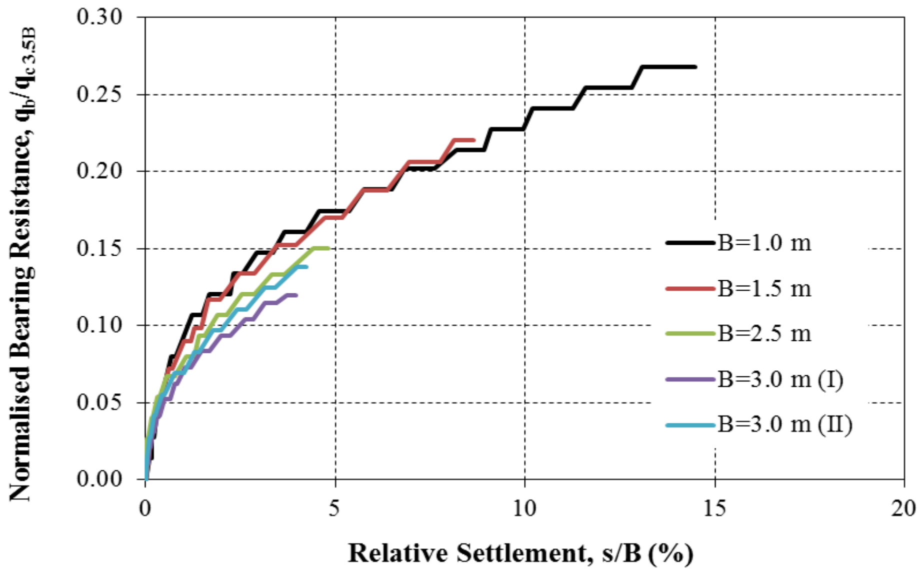


Figure 5.7: Unique normalised bearing pressure-settlement curve for shallow footing load tests performed at Texas A&M (after Gavin et al 2009)

When the bearing pressure was normalised by the average q_c value and the settlement s, was normalised by the footing width, B, a direct relationship was suggested for the normalised pressure-settlement response, which was independent of footing width or relative embedment (D/B) was obtained (See Figure 5.7). The footing resistance mobilised when the settlement reached 10% of the footing width, $q_{b0,1}$ could be given as:

[2] $q_{b0.1} = \alpha\, q_c$

An α value of 0.25 was suggested by Briaud and Gibbens (1999) to provide a good fit to the measured data.

Whilst the Texas A&M footing tests were performed on medium dense sand at a constant depth of embedment, Eslaamizaad and Robertson (1996) compiled a database of footing tests and found that the back-figured α values varied with soil density, relative embedment and footing shape. Randolph et al. (2005) summarised the results of laboratory and field tests and numerical analyses performed on shallow footings and buried piles. Although a relatively wide range of α values were reported, with α varying from 0.13–0.21, there was no evidence that α varied with footing width or sand state. To investigate this trend Gavin and Tolooiyan (2012) performed finite element analyses to investigate the effect of varying footing width for shallow footings placed in dense Blessington sand. Their mobilised pressure-settlement response for footings at a depth, D of 1 m below ground level is shown in Figure 5.8. The data suggested that a shallow foundation placed in dense sand mobilised very high resistance and that at a given displacement, the resistance reduces as the footing width, B increases. By performing sensitivity analyses for a range of footing sizes and sand densities (strengths) they found that α values formed a relatively narrow range if the zone of influence over which the soil strength considered (in this case the q_c value) was large enough. They found that it was necessary to calculate the mean q_c value over a zone of influence extending 3.5 footing widths below the foundation (See Figure 5.6). However, the most important finding of this work is the usefulness of the CPT test as a flexible input parameter that can facilitate the direct design of gravity base foundations.

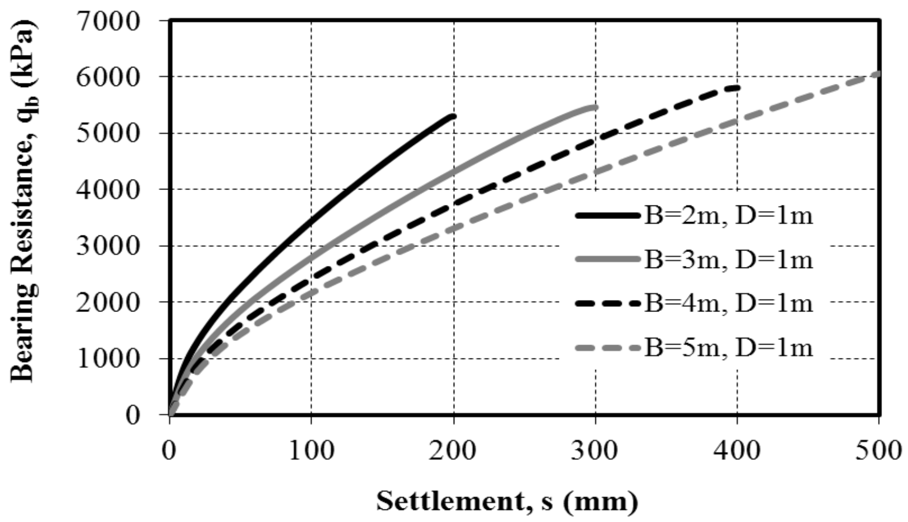


Figure 5.8: Finite element predictions of the effect of footing width on the bearing pressure mobilised on dense sand (after Gavin and Tolooiyan 2012)

For foundations on clay, direct CPT methods are not as widely used as for footings on cohesionless deposits. Undrained analyses can be performed using Equation 1, with the undrained soil strength, s_u in place of the apparent cohesion and bearing capacity factors which account for variable soil strength profiles and footing base roughness (Houlsby and Martin 2003). The soil strength properties for these analyses are usually derived from correlations between the soil strength and CPT resistance using site specific correlations between the CPT data and high quality soil strength tests. Theoretical solutions have provided a direct relationship between the undrained shear strength (s_u) and the tip resistance, allowing the s_u value to be determined from: $s_u=(q_c-\sigma_{v0})/N_{kt}$. This indirect CPT approach is a useful means of developing detailed vertical profiles of the undrained shear strength. However, it should be noted that the cone factor, N_{kt}, varies from 10 to 18, with 14 as an average. N_{kt} tends to increase with increasing plasticity and decrease with increasing soil sensitivity and generally some site specific calibration is needed using high quality triaxial test data.

Under normal working loads, wind turbine foundations are subjected to a combination of vertical loads and overturning moments from the tower structure. As the conventional bearing capacity approach was derived for the problem of central vertical loading, modification factors have been introduced to account for overturning and approaches such as Meyerhoff's (1953) effective width approach are commonly adopted.

Recently, there has been a gradual departure from the use of conventional bearing capacity type approaches to the use of plasticity models in the estimation of footing capacity under combined loading, See Nova and Montrasio (1991), Gottardi et al. (1999) and others. For foundations subjected to significant horizontal and/or moment loading these allow for rational consideration of the interaction effects of vertical, horizontal and moment loading. To date there has been limited work undertaken to link the plasticity based methods with in-situ test results.

5.5.2 Design of Monopiles

The design of offshore monopoles is undertaken using the API (2007) and DNV (2007) design codes which are essentially identical. The analysis of laterally loaded piles is based on a Winkler model and is commonly referred to as the p-y (lateral force versus displacement) approach. This method of analysis assumes that the pile-soil interaction problem is equivalent to a beam supported by a series of uncoupled springs. The principle of using soil springs to represent the soil reaction is illustrated in Figure 5.9. The p-y response can be characterised by a linear or non-linear curve, which describes the soil reaction, p, at a given depth as a function of the lateral movement, y. The spring stiffness, E_{py}, is defined as the secant modulus of the p-y curve (see Figure 5.10).

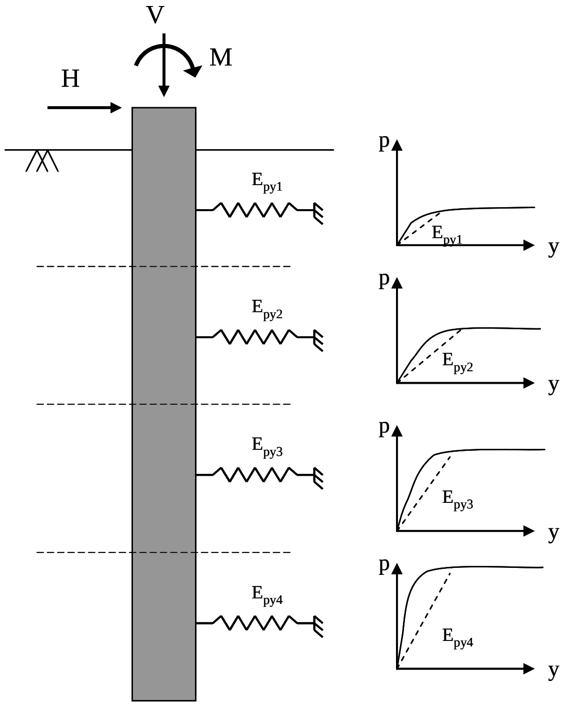


Figure 5.9:Basis of the p-y methods for Monopile design (after Doherty and Gavin 2012)

The application of the Winkler approach for laterally loaded pies was first suggested by Reese and Matlock in 1956. The original p-y curves for piles in cohesionless deposits developed by Reese et al (1974) were based on empirically derived curve-fitting parameters from the results of lateral load tests performed on two identical instrumented test piles installed at Mustang Island in Texas, described by Cox et al (1974). The test piles were 610 mm outside diameter steel tubes, with a wall thickness of 9.5 mm and were driven open-ended to a penetration depth of 21 m into saturated sand. The slenderness ratio (length, L over diameter, D) L/D ratio of 34 of the piles is much larger than the slenderness ratios typical of the wind industry (L/D=5-6). Pile with high slenderness ratio will tend to bend under the application of lateral loads, whilst, those with low slenderness will be more rigid, and tend to rotate under loading.

The Mustang Island test piles were subjected to static and cyclic load tests. Strain gauges were used to measure the bending moment profile with depth in the pile.

Experimental p-y curves were derived through integration and differentiation of the bending moment profile, see Reese et al (1974). Following assumptions on the failure modes controlling pile failure, a semi-empirical p-y curves which consisted of four discrete parts was assembled into a continuous piecewise curve (see Figure 5.10). The initial portion of the curve is a straight line, which is followed by a parabola adjoined to another linear portion and finally a constant ultimate strength.

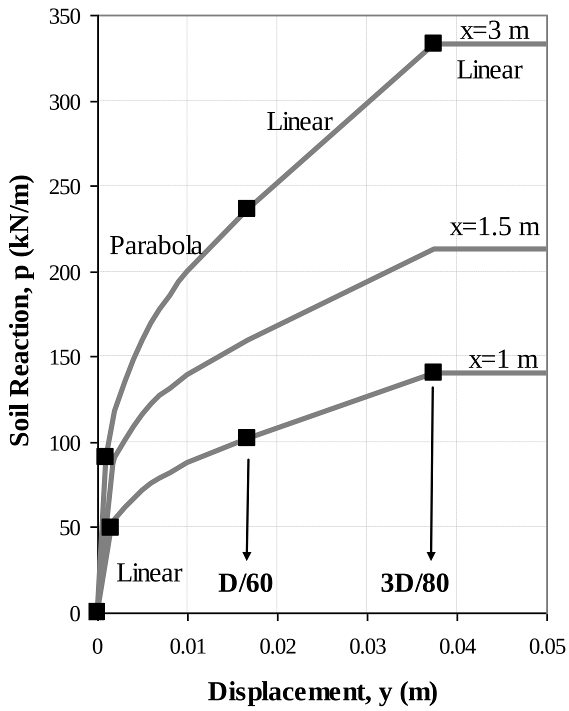


Figure 5.10: Construction of a p-y curve using the Reese et al (1974) approach

In order to construct the curve the designer needs to estimate the theoretical ultimate resistance, p_c. This value depends on the failure mechanisms (deep or shallow) for piles with low slenderness ratios a shallow (Rankine wedge type failure) usually governs the design. The ultimate resistance, p_u, is deemed to be fully mobilised when the pile displacement reaches 3D/80 (See Figure 5.10). An empirical parameter, B (which depends on the normalised depth) is used to fit the theoretical resistance, p_c, with the measured resistance, p_m, at a deformation of D/60. A linear increase in pile resistance is assumed between p_m and p_u. The initial portion of the curve was obtained using a linear resistance relationship, where $p=E^0_{py}.y$. The initial stiffness, E^0_{py} increases linearly with depth, x ($E^0_{py}=k.x$). The increase is defined by the initial modulus of sub-grade reaction, k, which depends on the relative density, with Reese et al (1974) suggesting values of 5.4, 16.3 and 34MN/m^3 for loose, medium and dense sands respectively. The intermediate section of p-y curve is described by a parabola which adjoins the straight line portions of the curves, as illustrated in Figure 5.10. The results from cyclic load tests performed on the Mustang Island piles were used to propose a set of empirical reduction factor, which acts as reduction factor on the ultimate pile resistance.

Murchinson and O'Neill (1984) compiled a database of lateral load tests and compared the reliability of the Reese et al (1974) design approach against three alternative p-y formulas. A hyperbolic model, given by Equation 3, was shown to provide better predictions of the lateral deflections and the maximum moments than the traditional 1974 approach and this model has been incorporated into the current design methods (API, 2007 and DNV, 2007). The ultimate resistance for this model is determined using the same methodology (based on Rankine earth pressures) as previously established. However, estimating p_u is simplified by introducing the dimensionless coefficients, C_1, C_2 and C_3 which are functions of the friction angle. The ultimate soil resistance can then be determined without the need to calculate the Rankine pressures acting on the pile.

[3] $$p = Ap_u \tanh\left(\frac{kx}{Ap_u} y\right)$$

[3a] $$p_u = \min(p_{us}, p_{ud})$$

[3b] $$p_{us} = (C_1 x + C_2 D)\gamma x$$

[3c] $$p_{ud} = C_3 D \gamma x$$

[3d] $$A = \left(3 - 0.8\frac{x}{D}\right) \geq 0.9$$

The pile-soil stiffness can be obtained by differentiating Equation 3:

[4]
$$E_{py} = \frac{d}{dy}\left[Ap_u \tanh\left(\frac{kx}{Ap_u}y\right)\right] = Ap_u \frac{\frac{kx}{Ap_u}}{\cosh^2\left(\frac{kxy}{Ap_u}\right)}$$

Values of k which depend on the friction angle are given in the API design codes, although k is assumed to be constant for relative densities above 80%.

It is worth noting that the p-y approach outlined above is fundamentally reliant on the soil friction angle to determine C1, C2, C3 and k. As ϕ' can be accurately determined from CPT based correlations such as the Kulhawy and Mayne approach described previously, the standard lateral pile design methodology can be modified into an indirect CPT based approach through the use of CPT based estimates of the soil strength.

Limited work has been undertaken to link the lateral resistance of piles directly with the CPT parameters. Lee et al. (2010) used synthetic CPT profiles developed using the CONPOINT computer program to examine correlations between q_c and the ultimate lateral resistance developed on a pile calculated using three popular design methods. They performed a statistical regression analysis to propose a correlation between the ultimate lateral stress q_u and q_c. However, given that neither the CPT profile or pile response was actually measured, the applicability of the method as a general design tool is questionable.

5.5.3 Jacket Structures

The design of axially loaded piles to support jacket structures has evolved directly from offshore pile design practice and has been driven by the oil and gas industries and is reflected directly in updates to the American Petroleum Institute (API) RP2A design code. The local shaft resistance (τ_f) of a pile is estimated using a conventional earth pressure approach:

[5] $\tau_f = \beta\, \sigma'_v$

Where: σ'_v is the vertical effective stress, β is an empiric factor which depends on K, the earth pressure coefficient linking σ'_v to the radial effective stress (σ'_r) and $\delta_{f,}$ the interface friction angle. The β values, which are given in the code, depend on the soils relative density together with limiting maximum shaft resistance values.

A number of researchers have commented on the poor reliability of the API method when estimating the shaft capacity of piles. In addition, due to the natural variability of sand deposits and the well proven concept of friction fatigue on driven piles, techniques which average the shaft resistance and or soil properties along the pile shaft are somewhat questionable. For this reason there has been a significant move towards developing correlations between τ_f and in-situ test parameters. Because of the similarities between the penetrometer installation in the Cone Penetration Test (CPT) and pile installation, and also the widespread use of CPT testing in the offshore environment, correlations between τ_f and the CPT end resistance (q_c) have been extensively explored. Based on experiments using instrumented closed-ended piles Jardine et al. (2005) show that the local shaft resistance developed during loading of the pile at both tests sites could be described using the Mohr-Coulomb failure criteria:

$$\tau_f = (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan \delta_f$$

where: σ'_{rc}, was the radial effective stress acting on the pile shaft prior to the pile load test and $\Delta\sigma'_{rd}$ is a component derived by dilation during loading. Equation 6 forms the basis of the well-known ICP-05 design method for displacement piles. Jardine et al, 2005, proposed the following equation to estimate σ'_{rc}:

[7] $$\sigma'_{rc} = \frac{q_c}{34}\left(\frac{h}{R}\right)^{-0.38}\left(\frac{\sigma'_{v0}}{P_{atm}}\right)^{0.13}$$

where: R is the pile radius, and P_{atm} is the atmospheric pressure (which can be taken as 100 kPa). In this expression, the constants in the first term describes the constant ratio of radial effective stress to the q_c value mobilised near the toe of the pile, the second accounts for the effects of friction fatigue (a minimum h/R value of

8 should be used), whilst the third term suggests a weak stress dependence in the correlation. Lehane (1992) used simple elastic cavity expansion theory to estimate the dilatational component of radial stress, $\Delta\sigma'_{rd}$ developed during pile loading:

[8] $$\Delta\sigma'_{rd} = \frac{4\,G}{D}\Delta y$$

where: G is the operational shear modulus of the soil and Δy is the radial displacement during pile loading. Since $\Delta\sigma'_{rd}$ is inversely proportional to the pile diameter, its effects are likely to be relatively small for offshore piles. Jardine et al. (2005) recommend that the δ_f value used in Equation 6 should be obtained from simple laboratory interface ring shear tests.

Since most offshore piles are open-ended steel tubes, Chow (1997) considered methods to correct the σ'_{rc} derived from Equation 7 to account for the reduced levels of soil displacement and lower stress level changes caused during the installation of an open-ended pile. Her approach, based largely on observations from tests performed on 324 mm open-ended piles installed in Dunkirk and reported by Brucy et al. (1991), was to assume that the σ'_{rc} value developed near the base of open and closed-ended piles were equal, and that the rate of reduction of shear stress with h/D (friction fatigue term) was increased for open-ended piles. This was achieved by substituting R in Equation 7 with R^*;

[9] $$R^* = \sqrt{R^2 - R_i^{\,2}}$$

where: R_i is the internal diameter of the pile. This reduction technique, which implies that no plugging takes place during pile installation, was adopted in the ICP-05 design method.

An alternative CPT design, method known as Fugro-05 was developed specifically for offshore open-ended piles by Kolk et al. (2005). The authors compiled a database of large scale instrumented load tests and reasoned that since offshore piles usually have large diameters, the effects of dilation could be ignored. They assumed that the interface friction angle was constant at 29° and that the local shear stress could be calculated using the following expressions:

[10a] $\tau_f = \left[0.08 \cdot q_c \left(\frac{\sigma'_{v0}}{p_{atm}} \right)^{0.05} \left(\frac{h}{R^*} \right)^{-0.9} \right]$ *for comp. loading where* $h/R^* > 4$

[10b] $\tau_f = \left[0.08 \cdot q_c \left(\frac{\sigma'_{v0}}{p_{atm}} \right)^{0.05} (4)^{-0.9} \left(\frac{h}{4R^*} \right) \right]$ for comp. loading where $h/R^* < 4$

[10c] $\tau_f = \left[0.045 \cdot q_c \cdot \left(\frac{\sigma'_{v0}}{p_{atm}} \right)^{0.15} \cdot \max\left(\frac{h}{R^*}, 4 \right)^{-0.85} \right]$ *for tension loading*

Although Equation 10 maintained the same basic form as Equation 6, wherein the constants were adjusted to fit the author's database, the distribution of shear stress predicted using the Fugro-05 expressions differ significantly from those predicted using ICP-05.

A study by the Norwegian Geotechnical Institute (NGI) by Clausen et al. (2005) resulted in the formulation of a design method known as NGI-05. Although ostensibly a CPT method, the cone resistance is used in a correlation to obtain the sands relative density. Friction fatigue is incorporated using a sliding triangle approach where the shaft resistance mobilised by open-ended piles is assumed to be ≈ 38% lower than closed-ended piles:

[11a] $\tau_f = z / L \cdot p_{ref} \cdot F_{Dr} \cdot F_{sig} \cdot F_{tip} \cdot F_{load} \cdot F_{mat} \geq \tau_{\min}$

[11b] $F_{Dr} = 2.1(D_r - 0.1)^{1.7}$

[11c] $D_r = 0.4\ln(q_{c1N}/22)$

[11d] $q_{c1N} = (q_c/p_{atm}) / (\sigma'_{v0}/p_{atm})^{0.5}$

where:

F_{sig} = $(\sigma'_{v0} / p_{ref})^{-0.25}$

F_{tip} = 1.0 for driven open-ended and 1.6 for driven closed-ended

F_{load} = 1.0 for tension and 1.3 for compression

F_{mat} = 1.0 for steel and 1.2 for concrete

τ_{min} = 0.1 σ'_{v0}

Lehane et al. (2005) performed a review of a proposed updated version of the 21st edition of the API code (API-00) and the three CPT methods: ICP-05, Fugro-05 and NGI-05. They compiled a database of static load tests performed on instrumented piles in sand with which to check the reliability of the existing design approaches (See Schneider et al. 2008). As a result of this review, the authors proposed an alternative CPT design method known as UWA-05 where the equalised radial effective radial effective stress developed by a displacement pile in sand was given by:

[12] $$\sigma'_{rc} = \frac{q_c}{33} \left(\frac{h}{D}\right)^{-0.5}$$

The UWA-05 design is similar in many respects to ICP-05. The local shear stress is calculated using Equation 2, which includes a dilatational component for radial effective stress and a reduction factor for tension loading. The principal difference between the UWA-05 and ICP-05 design guidelines is in their treatment of the effect of plugging on the shaft resistance developed by open-ended piles.

The end bearing resistance of piles is estimated in both the API and DNV codes using a modified form of the Terzaghi's bearing capacity theory, and assume the ultimate end bearing resistance (q_b) value developed by a deep foundation to be directly proportional to the vertical effective stress acting at the pile base through a relationship of the form:

[13] $$q_b = N_q \sigma'_v$$

where: N_q is a bearing capacity factor which depends on ϕ' (the soils friction angle) and the embedment depth, and σ'_v is the vertical effective stress at the pile base.

Although both N_q and σ'_v can be assessed with reasonable accuracy, a major drawback of this approach is the sensitivity of N_q to small changes in ϕ'. Sampling difficulties for cohesionless soils are such that empirical correlations between ϕ' and in-situ tests data are generally used in design, introducing additional uncertainty.

Recent design methods linking the ultimate base resistance of driven closed-ended or full-displacement piles in sand and the cone penetration test (CPT) end resistance (q_c) have been shown to have a relatively high reliability (Lehane et al. 2005) and have been widely accepted in industry. These techniques generally estimate the base resistance at relatively large pile base settlement (s_b), typically at 10% of the pile diameter, $q_{b0.1}$, using an empirical reduction factor α:

[14] $q_{b0.1} = \alpha \, q_c$

Jardine et al. (2005) performed a database study that suggested that a diameter dependent α value reduced from 0.63 to 0.43 as the pile diameter increased from 200 mm to 500 mm. Randolph (2003) and White and Bolton (2005) argued that once appropriate averaging techniques were adopted to derive design q_c values and the effects of residual loads were accounted for, a constant α factor can be adopted which is independent of pile diameter. Lehane et al. (2005) found that a α value of 0.6 gave the best-fit to a database of instrumented pile load tests with diameters ranging from 0.2 m to 0.68 m. For partial displacement (open-ended) pipe piles, model and full-scale pile tests reported by Lehane and Gavin (2001) and Foye et al. (2009) show that direct correlations between α (based on the average pressure mobilised over the entire pile base area) and q_c which are independent of pile diameter or sand state can be determined once the effect of sand displacement at the pile base during pile installation are included. Sand displacement is best quantified through the incremental filling ration (IFR), which compares the rate of soil intrusion during pile installation with IFR = 1 for a fully coring pile (which causes minimal disruption) and IFR = 0 for a pile with a fully formed plug, which prevents soil intrusion and results in what is effectively a closed-ended pile. Minimum α factors in the range 0.15 to 0.2 have been suggested by Lehane and Randolph (2002) and Gavin and Lehane (2003) for fully-coring piles.

6. SUMMARY and CONCLUSIONS

The INFOMAR surveying completed over recent years has provided widespread information around the Irish coast that will be useful in planning suitable sites for wind farm developments. However, two areas where additional information is needed were highlighted in this report:

1. Bathymetric data in deep water sections of the Irish Sea
2. Data on soils in the region of 30to 50 m below the sea bed

In order to de-risk specific foundation concepts and capitalize on the abundant offshore wind resource, additional information will be required including increased spatial coverage off the East coast, data from deeper soil strata, and more quantitative information regarding geotechnical properties. In addition, when soil samples are taken, a recognised classification system should be adopted to describe the engineering behaviour of the soil. Many of these issues could be addressed in a relatively cost effective manner by a programme of targeted CPT testing in Irish Coastal waters. A CPT framework has been described in this report that could be implemented as a data gathering exercise within the INFOMAR programme.

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Appendix A

Risk Register

Irish Wind Farm Developments
Technical Risk Assessment

CLIENT: INFOMAR

Objective: The aim of this risk register is to identify the technical risks that would impact on the feasability of construction offshore wind farm developments around the Irish coast.

Risk Impact Table:

Probability (P)	Severity (S): **1. Negligible :** Minimal impact on project schedule or budget	**2. Slight:** Minor impact on budget or schedule	**3. Moderate:** Significant impact on budget or schedule	**4.Critical:** Serious delays and/or detrimintel cost implications	**5. Catastrophic:** Threatens the project completion
1. Rare: A freak combination of factors would be required for an incident to occur	1	2	3	4	5
2. Unlikely: A rare combination of factors required for incident to occur	2	4	6	8	10
3. Possible: Could happen with additional factors but otherwise unlikely to occur	3	6	9	12	15
4. Likely: Not certain to happen but additional factors could cause to happen	4	8	12	16	20
5. Highly Likely: Expected to happen	5	10	15	20	25

RISK FACTOR	ACTION:
1-6: LOW	May be an acceptable risk level, however efforts should be taken to reduced risk where possible
7-14: MED	May be tolerable, however risk mitigation and control measures are definitely required
15-20: HIGH	Unacceptable, mitigation measures must be introduced to reduce both the severity and probability of the risk.

ACTIVITY	EVENT	CONSEQUENCE & DETAILS	RISK			MITIGATION / CONTROL MEASURES	RESIDUAL RISK		
			P	S	CAT		P	S	CAT
All	Insufficient wave data available to develop safe, economical design	Poor understanding of the wave data can lead to over-conservatism in design (affecting the financial viability of the project) whilst under-estimation of wave loading effects can lead to poor serviceability and failure.	3	4	12	Site specific wave measurements should be undertaken over a prolonged period to include; wave height, wave period, surface elevation, crest elevation and wave direction.	1	4	4
All	Deep deposits of soft (recent) clay/silt at the sea bed level (e.g. Dundalk)	Even in relatively shallow water depths can preclude the use of gravity and suction caisson foundation solutions. Where soft deposits exceed 10 m the use of jack-up vessels may be precluded.	3	4	12	Identify the near surface stratigraphy accurately.	1	4	4
All	Scour	Erosion of soil leading to loss of support, additional rotation and potential failure	3	5	15	Site specific scour assessment and remedial measures including scour protection mats, rock armour etc.	2	3	6
All	Presence of very high strength glacial soils, with boulders or rock near sea bed	Premature refusal of in-situ site investigation techniques such as Cone Penetration Tests	3	2	6		3	2	6

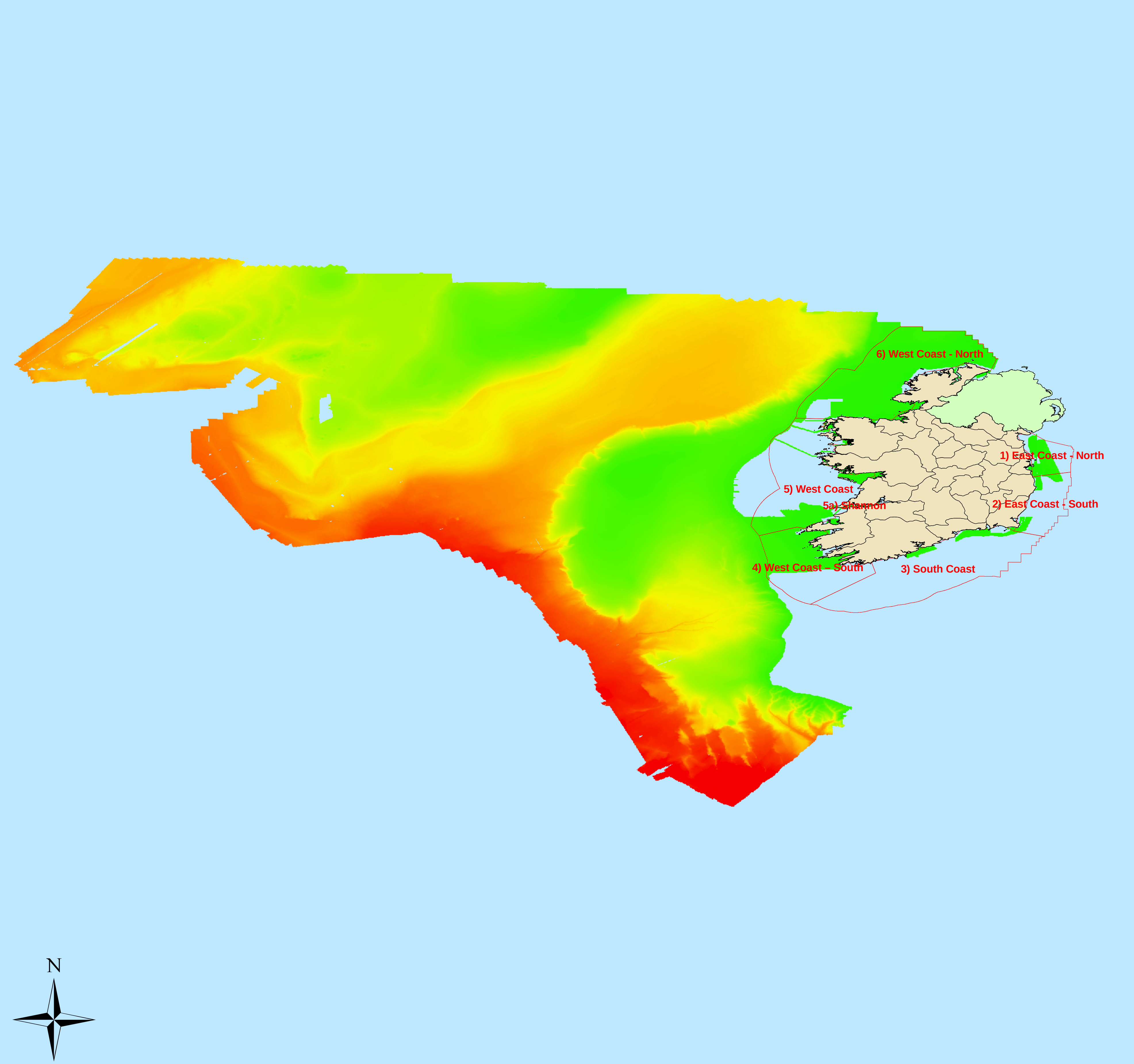
All	Presence of very soft Clays (eg. Macamore) possible in area along Southern extents of Irish Sea	Material susceptible to severe strength loss under cyclic loading	3	2	6	Careful site investigation and determination of clay mineral content and identification of residual shear planes	3	1	3
Gravity Base Foundation	Insufficient soil data leading to delays in construction	Poor understanding of the near surface soils (e.g. Presence or rocks/boulders, soft spots, local depressions) require additional sea bed preparation.	3	3	9	Comprehensive site investigation including geophysical testing and in-situ drill holes, Cone Penetration Testing and laboratory testing	2	3	6
Gravity Base Foundation	Seabed gradient variable or greater than construction tolerance (eg. > 5 degrees)	Construction not possible due to both installation and serviceability requirements	3	5	15	Accurate geophysical surveying to determine the seabed profile as undertaken in the INFOMAR surveys	3	1	3
Gravity Bases	Insufficient soil data leading to poor serviceability behaviour	Poor understanding of the in-situ soil stratigraphy may lead to long-term differential settlement/rotation of the foundation.	4	3	12	Comprehensive site investigation including geophysical testing and in-situ drill holes, Cone Penetration Testing and laboratory testing	3	2	6
Gravity Base Foundation	Grouting	Requirement to grout beneath base to maintain level, provide uniform stress distribution and avoid piping	3	3	9	Reduce the need for grouting through seabed preparation	2	3	6

Monopile Foundations	Scour	Excessive scour of mobile sea bed or high velocity currents leading to increased effective length and excessive rotations (eg. Arklow Bank)	2	5	10	Identification of mobility of sand banks using historical data. Hydrodynamic analysis to be undertaken	2	3	6
Monopile & Jacket Foundations	Environmental	Concerns over environmental noise caused by pile driving leads to restriction	3	4	12	Use of noise reduction measures, drilled pile construction or alternative systems.	2	2	4
Monopile & Jacket Foundation	Driving Difficulties	Presence of near surface rock causes premature refusal and inability of pile to sustain design moment/tension loads	3	4	12	Undertake site specific driveability studies	2	2	4
Monopile Foundation	Driving Difficulties	Any delay is driving may allow considerable set-up in stiff glacial soils and sand	2	3	6	Undertake site specific driveability studies	1	3	3
Monopile Foundation	Grouting	Requirement to grout between foundation and transition piece	3	4	12	QA procedures to be adopted during grouting	3	3	9
Jacket Structures	Grouting	Issues grouting between pile sections and leader piles/structure	3	4	12	QA procedures to be adopted during grouting	3	3	9
Jacket Structures	Variable Ground Conditions	Highly variable glacial soils might prevent difficulties in achieving allowable tolerances when installing piles within a jacket template.	2	2	4	Consider using multiple piling gates /sleeves on your jacket structure	1	1	1

Suction Caisson	Insufficient soil data leading to installation problems	Thin walled suction caissons will experience difficulties during installation into deposits with high boulder content or where near surface rock is present.	4	4	16	Direct establishment of rock head and the extent of boulder using intrusive rock coring is necessary	2	4	8
Suction Caisson	Natural variability in strength and stiffness of soil deposits	Difficult to achieve verticaility tolerances	2	4	8	Undertake comprehensive site investigation	2	2	4
Suction Caisson	Presence of clastic dykes	Local vertical zone of high permeability may affect ability to sustain transient suctions.	1	4	4		1	4	4
Suction Caisson	Gas Deposits	Risk of blow-out causing failure	2	3	6	Undertake a deskstudy at the outset to assess the likely presence of shallow gas. Geophysical surveying to identify gas	1	3	3

Appendix B
Assessment Zones & INFOMAR Data

Irish Designated Zone



Legend

- Ireland
- Assessment Zones
- Northern Ireland

Bathymetry

Value

High : 0m

Low : 5000m

0 60 120 240 360 480 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:

1: Not to be used for navigation

Rev:		
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Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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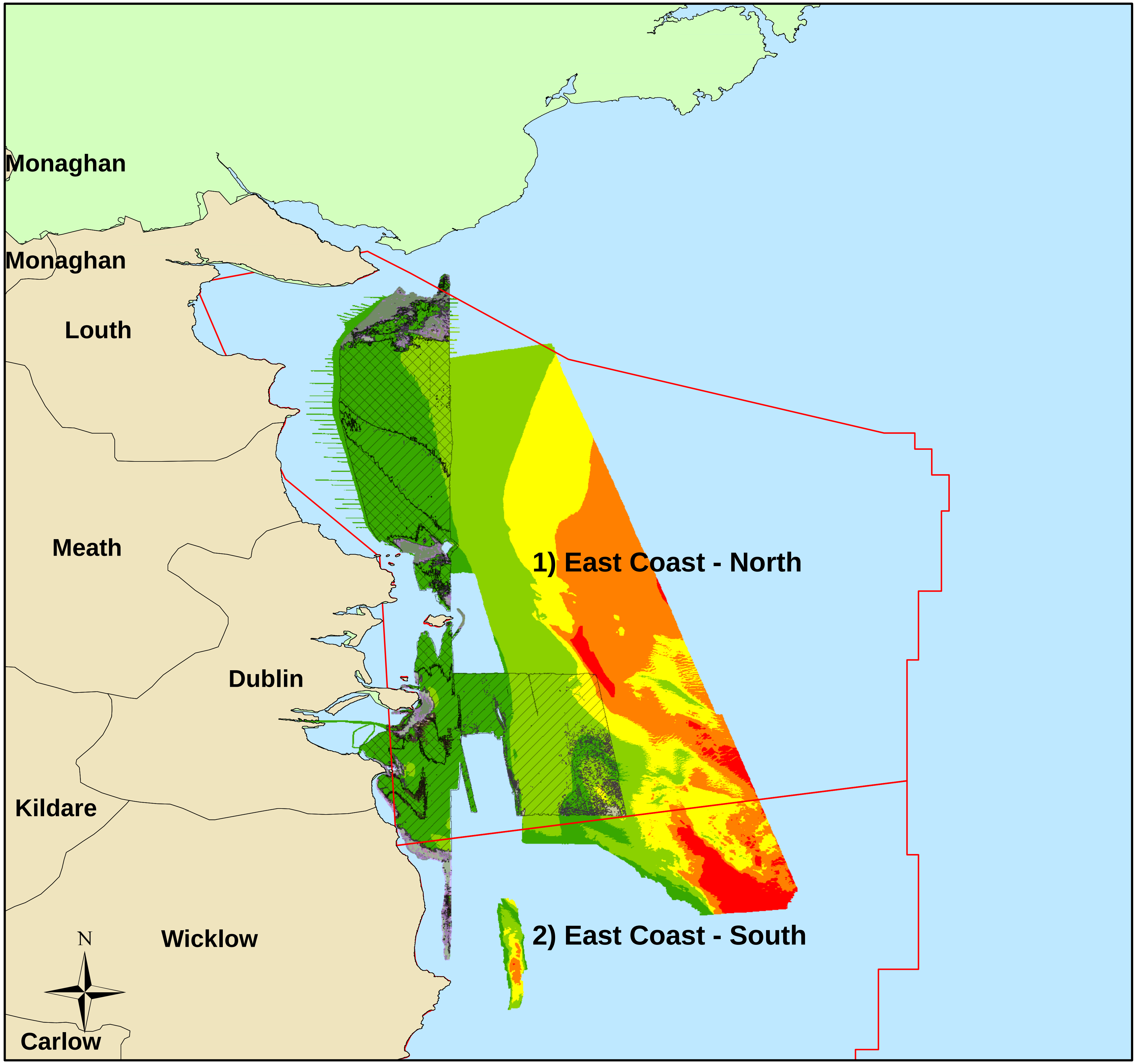
Assessment Zone 1 - East Coast - North



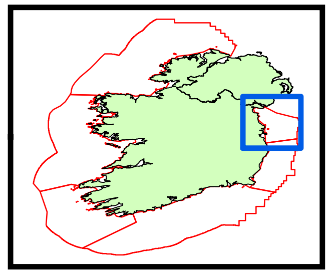
Legend

- Ireland
- Assessment Zones
- Mud to fine sand
- Fine to medium sand
- Course sand and gravel
- Rock and gravel
- Coarse sand and gravel (High biogenic content)

Bathymetry

Depth

- 135m - 80m
- 80m - 60m
- 60m - 45m
- 45m - 30m
- 30m - 0m



0 5 10 20 30 40 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

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Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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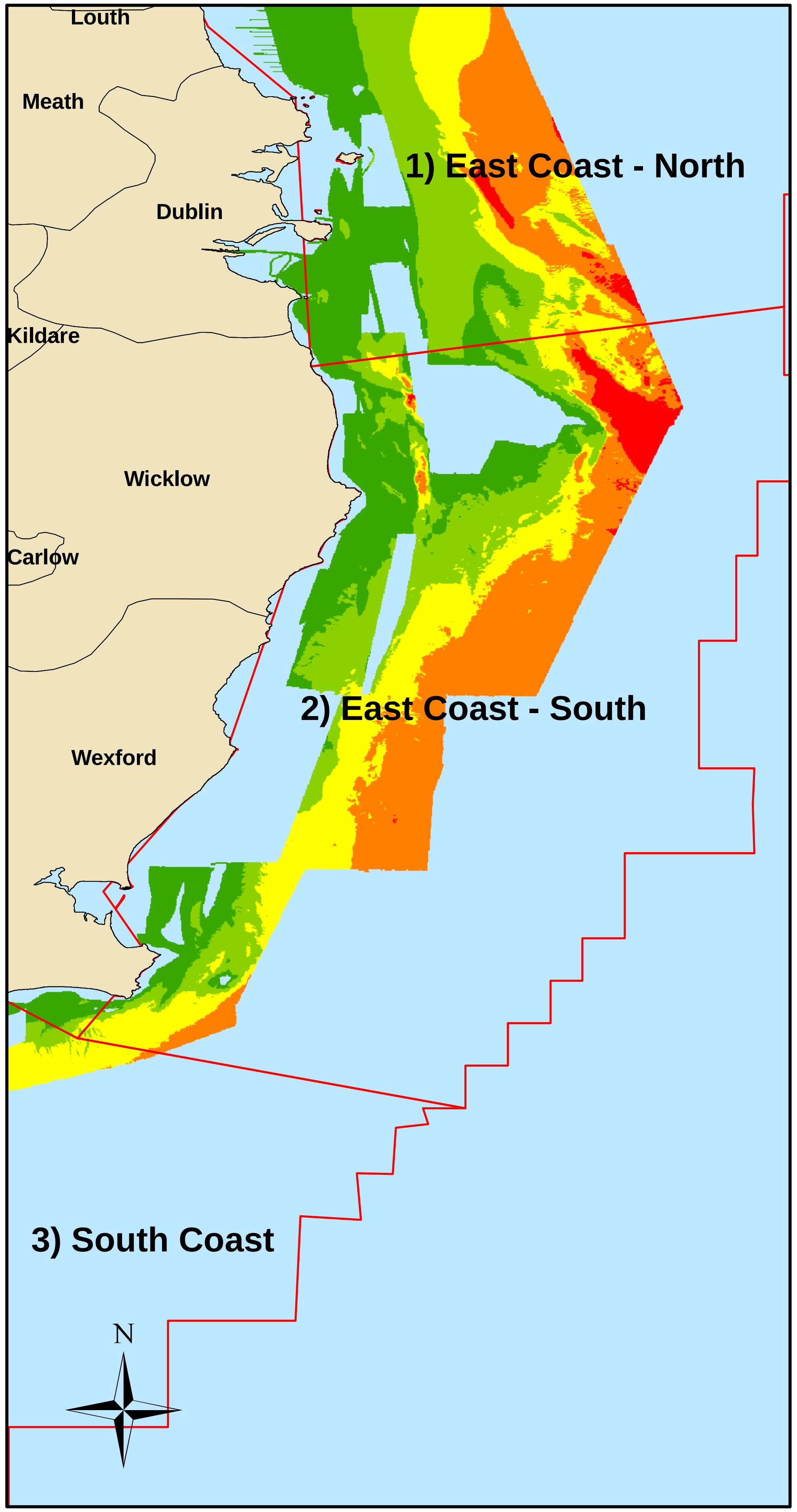
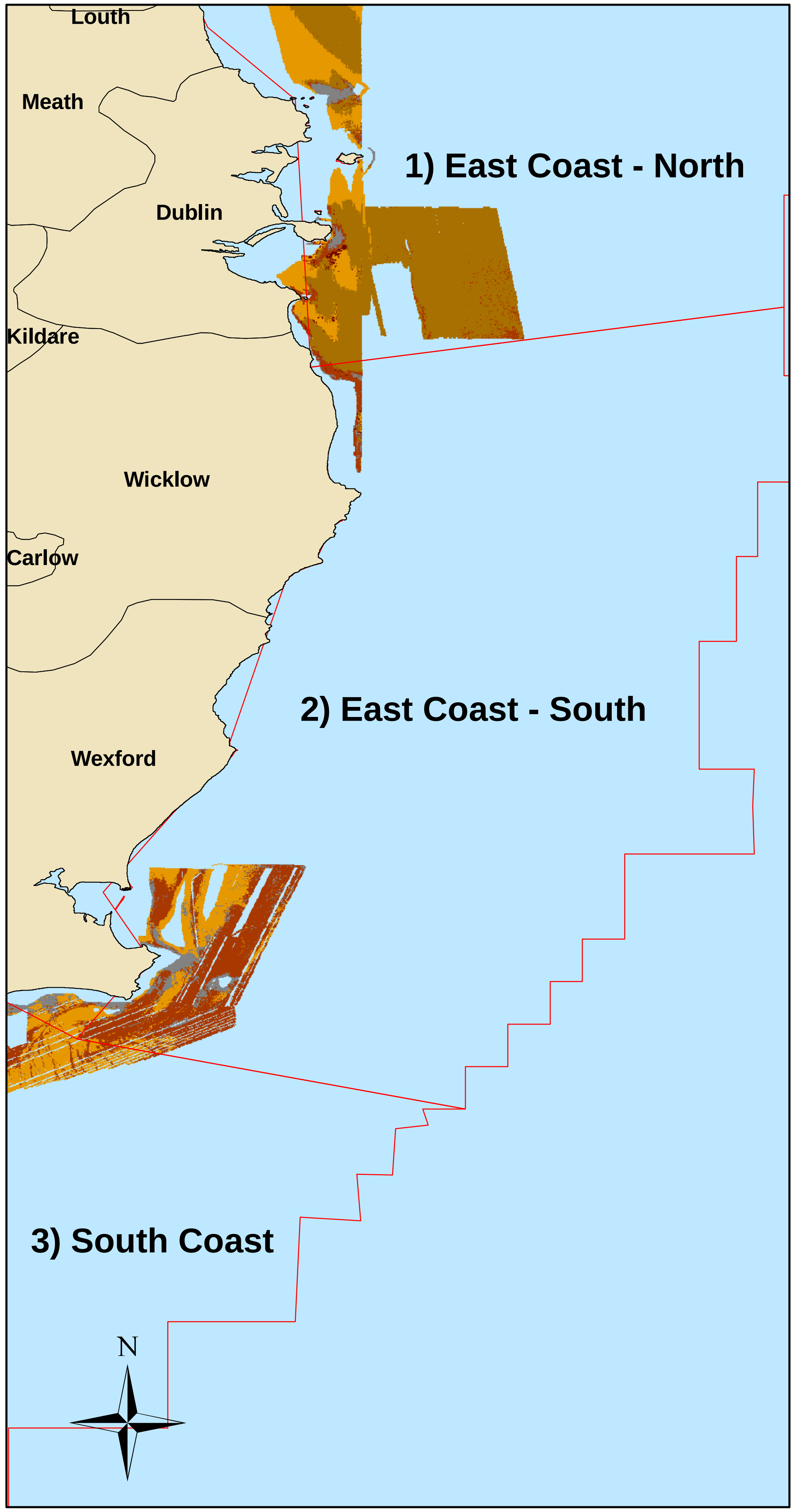
Assessment Zone 2 - East Coast - South





Legend

- Ireland
- Assessment Zones

Seabed Classification

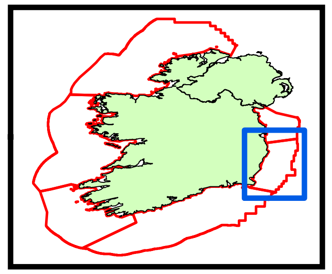
Soil Type

- Mud to fine sand
- Fine to medium sand
- Coarse sand and gravel
- Rock and gravel
- Coarse sand and gravel (High biogenic content)

Bathymetry

Depth

- 135m - 80m
- 80m - 60m
- 60m - 45m
- 45m - 30m
- 30m - 0m



0 5 10 20 30 40 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

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Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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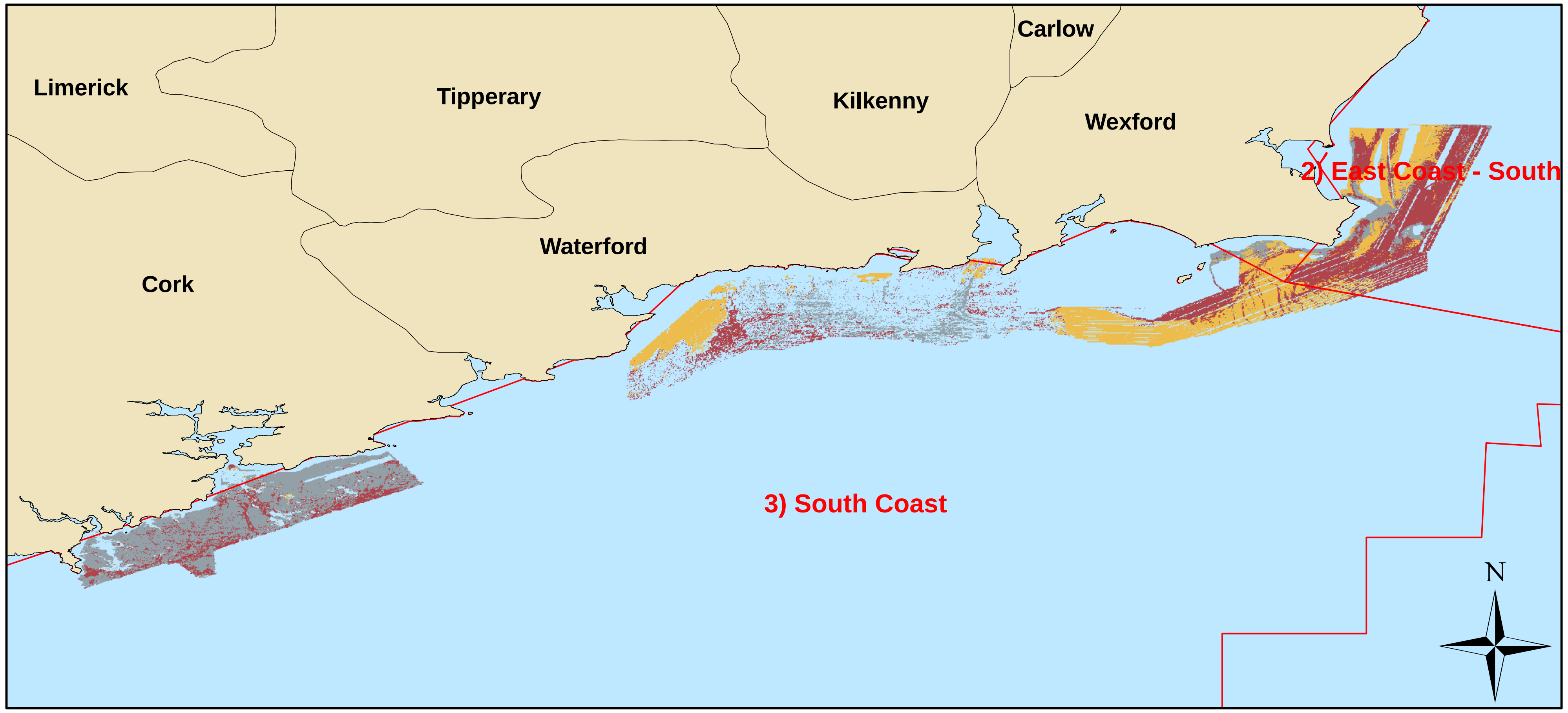
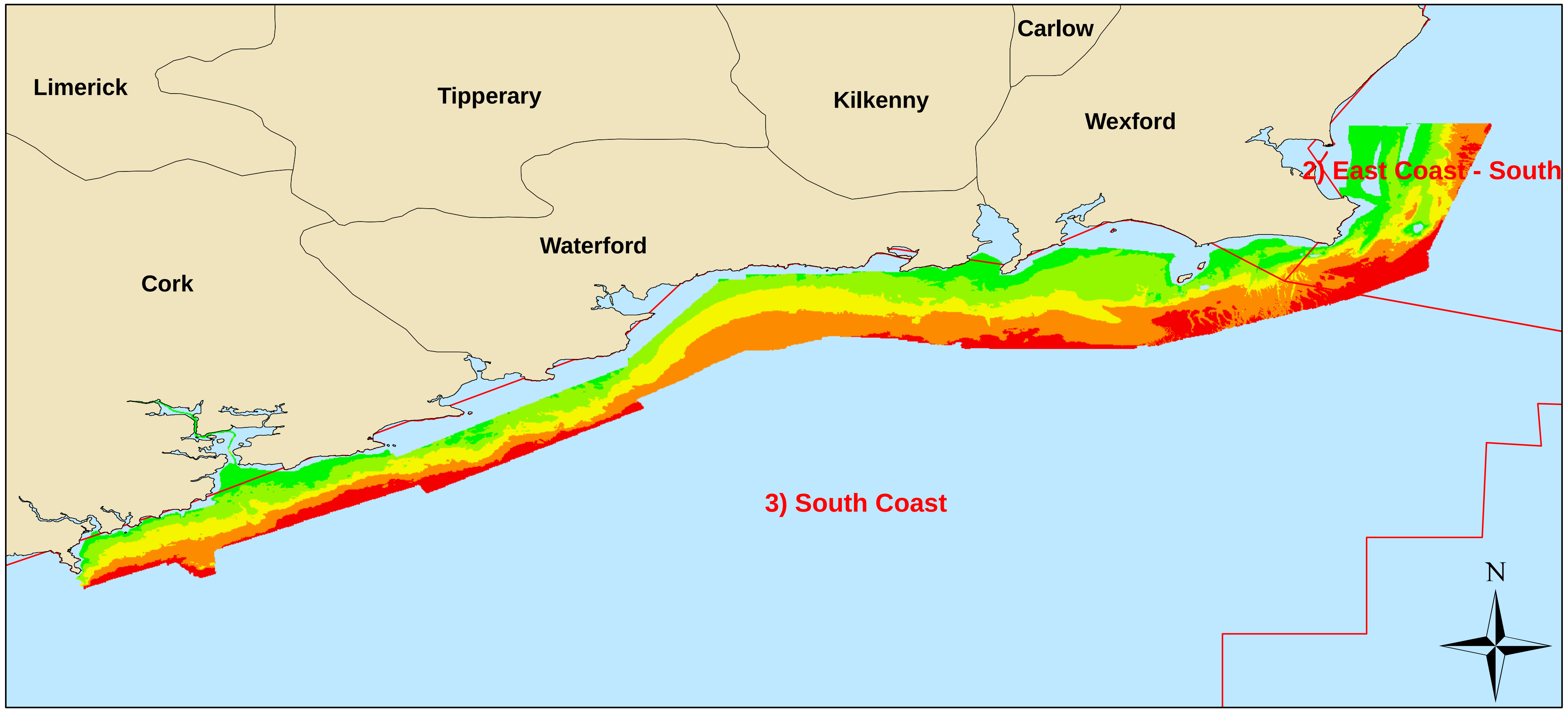
Assessment Zone 3 - South Coast





Legend

- Ireland
- Assessment Zones

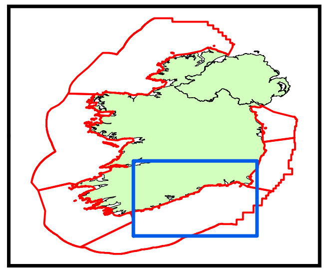
Seabed Classification

- Fine Sand to Mud
- Gravel to Coarse Sand
- Rock

Bathymetry

Depth

- 80m - 50m
- 50m - 40m
- 40m - 30m
- 30m - 20m
- 20m - 5m



0 5 10 20 30 40
Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

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Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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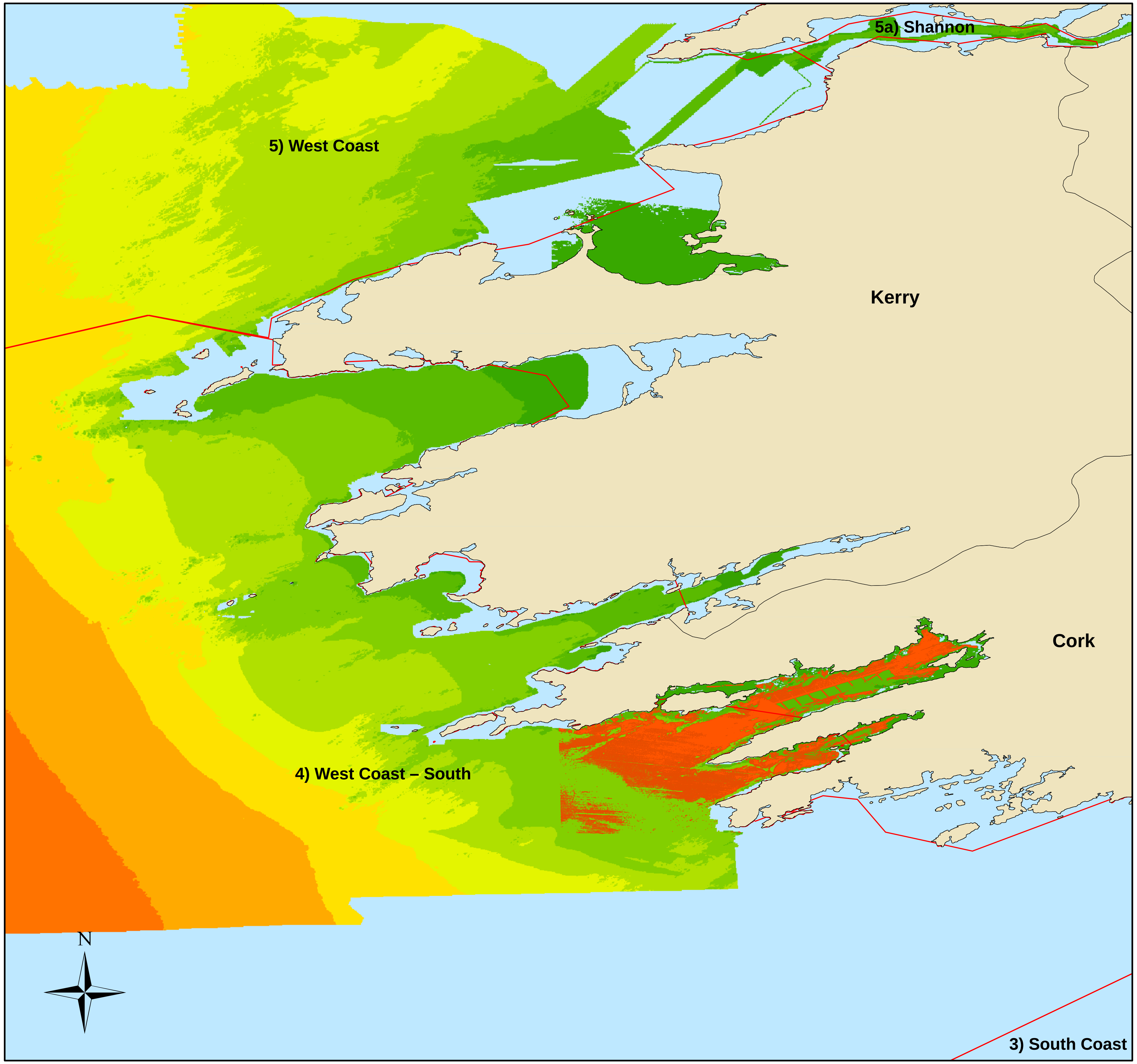
Assessment Zone 4 - West Coast - South



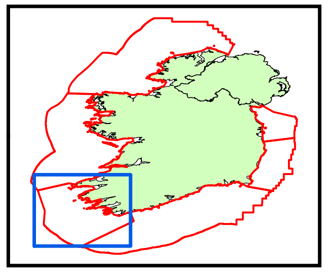
Legend

- Ireland
- Assessment Zones
- Fine Fine - Medium Dense Sand

Bathymetry

Depth

- 375m - 250m
- 250m - 180m
- 180m - 160m
- 160m - 135m
- 135m - 115m
- 115m - 95m
- 95m - 75m
- 75m - 50m
- 50m - 20m
- 20m - 5m



0 5 10 20 30 40 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

Rev:		
Title:	Bathymetry - Showing Sand Deposits	
Dwr No:	11011-03-DWR05	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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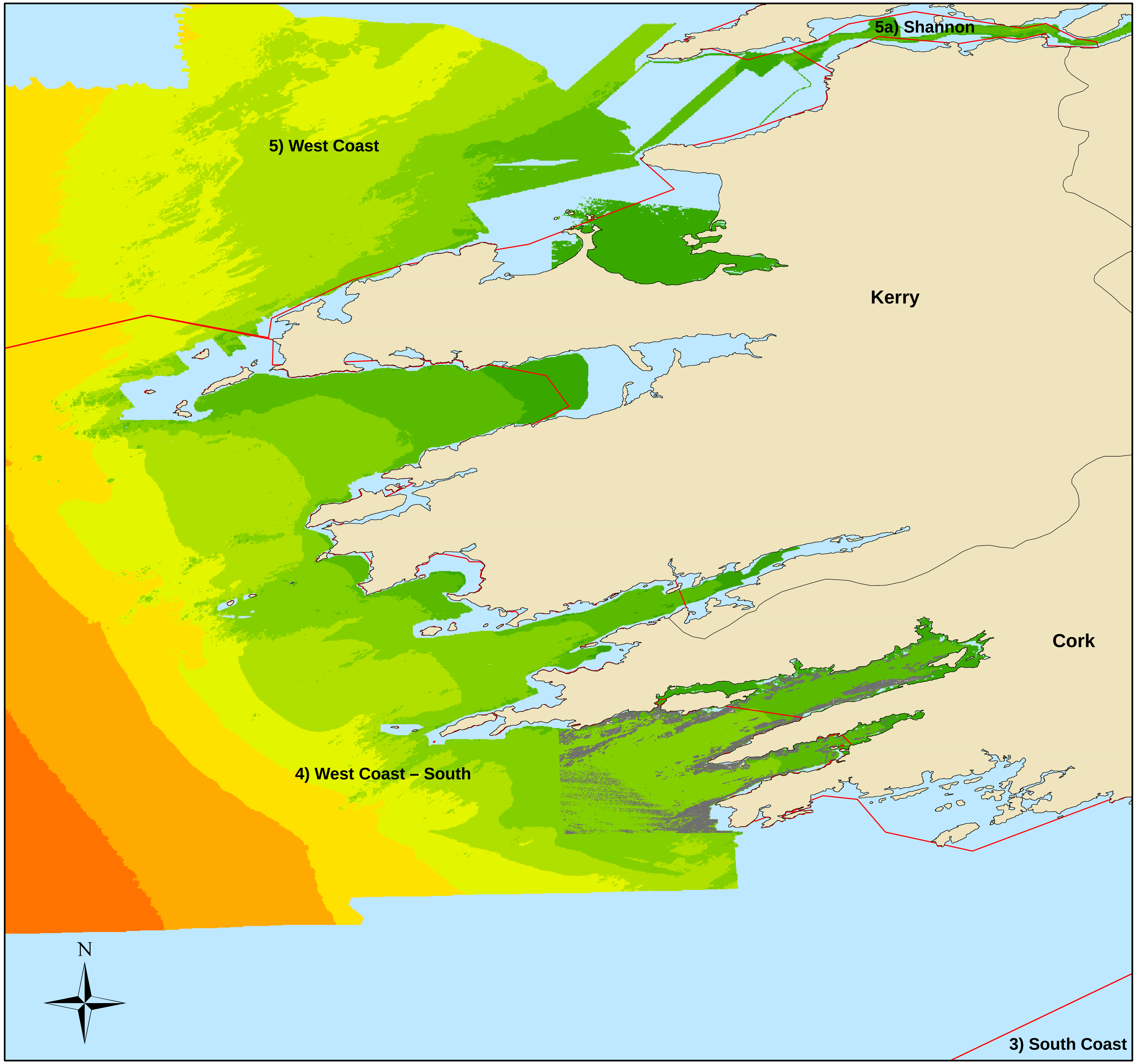
Assessment Zone 4 - West Coast - South



Legend
Ireland
Assessment Zones
Rock
Bathymetry
Depth
375m - 250m
250m - 180m
180m - 160m
160m - 135m
135m - 115m
115m - 95m
95m - 75m
75m - 50m
50m - 20m
20m - 5m

0 5 10 20 30 40 Kilometers

INFOMAR
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Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

Rev:		
Title:	Bathymetry - Showing rock outcrop's	
Dwr No:	11011-03-DWR06	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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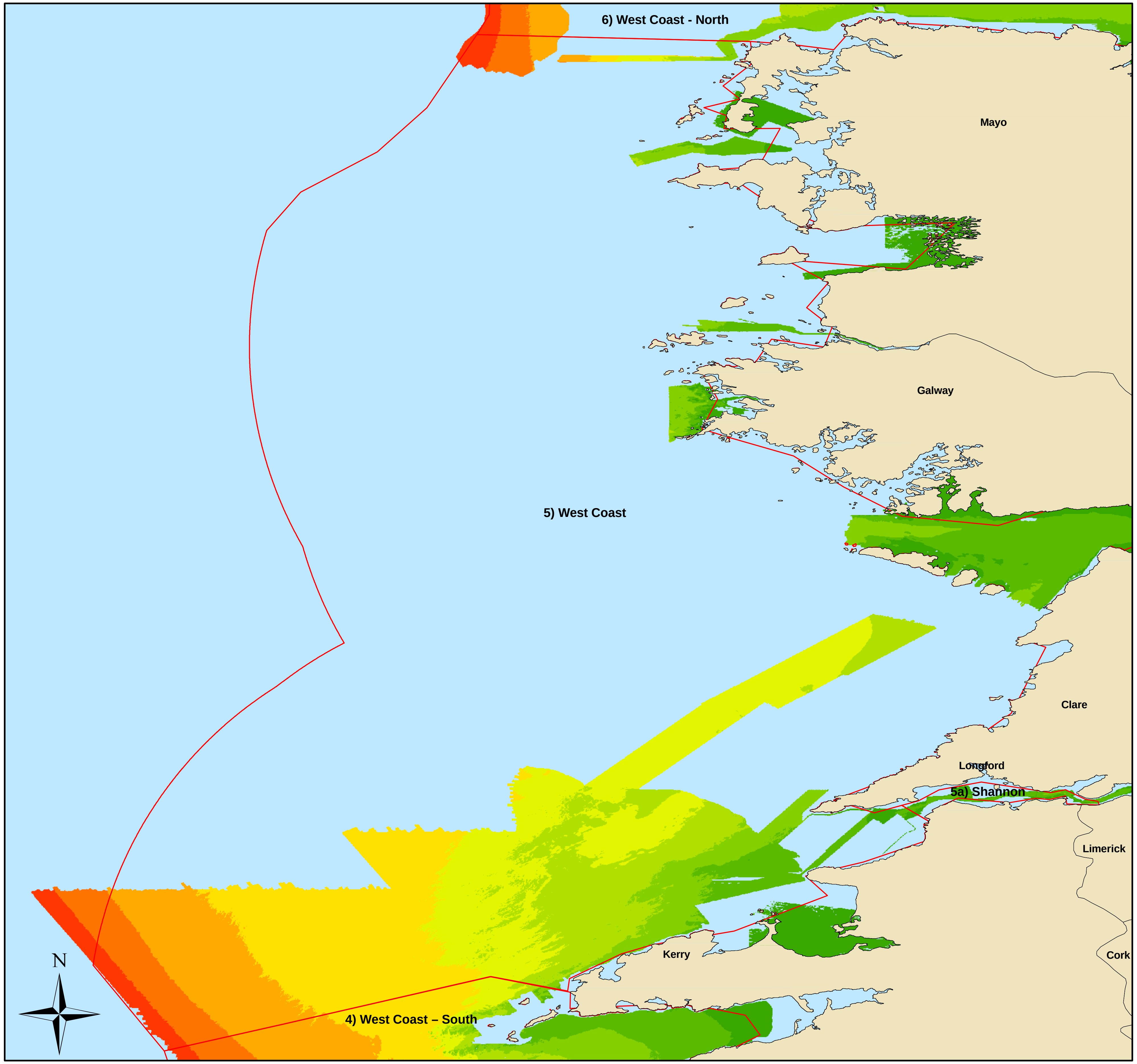
Assessment Zone 5 - West Coast



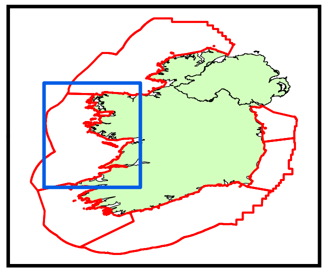
Legend

- Ireland
- Assessment Zones

Bathymetry

Depth

- 375m - 250m
- 250m - 180m
- 180m - 160m
- 160m - 135m
- 135m - 115m
- 115m - 95m
- 95m - 75m
- 75m - 50m
- 50m - 20m
- 20m - 5m



0 5 10 20 30 40 Kilometers 1 cm = 7 km

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Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation

Rev:			
Title:	Bathymetry Whole Region		
Dwr No:	11011-03-DWR05		
Date:	5/12/2012		
Ref:	INF-11-24-DOH		
Projection:	WGS_1984_UMT_Zone_29N		
Source:	Infomar Interactive Web Data Delivery System		
Projection:	WGS_1984_UMT_Zone_29N		
Checked:	Produced by:	G Murphy	
	Checked by:	P Doherty	

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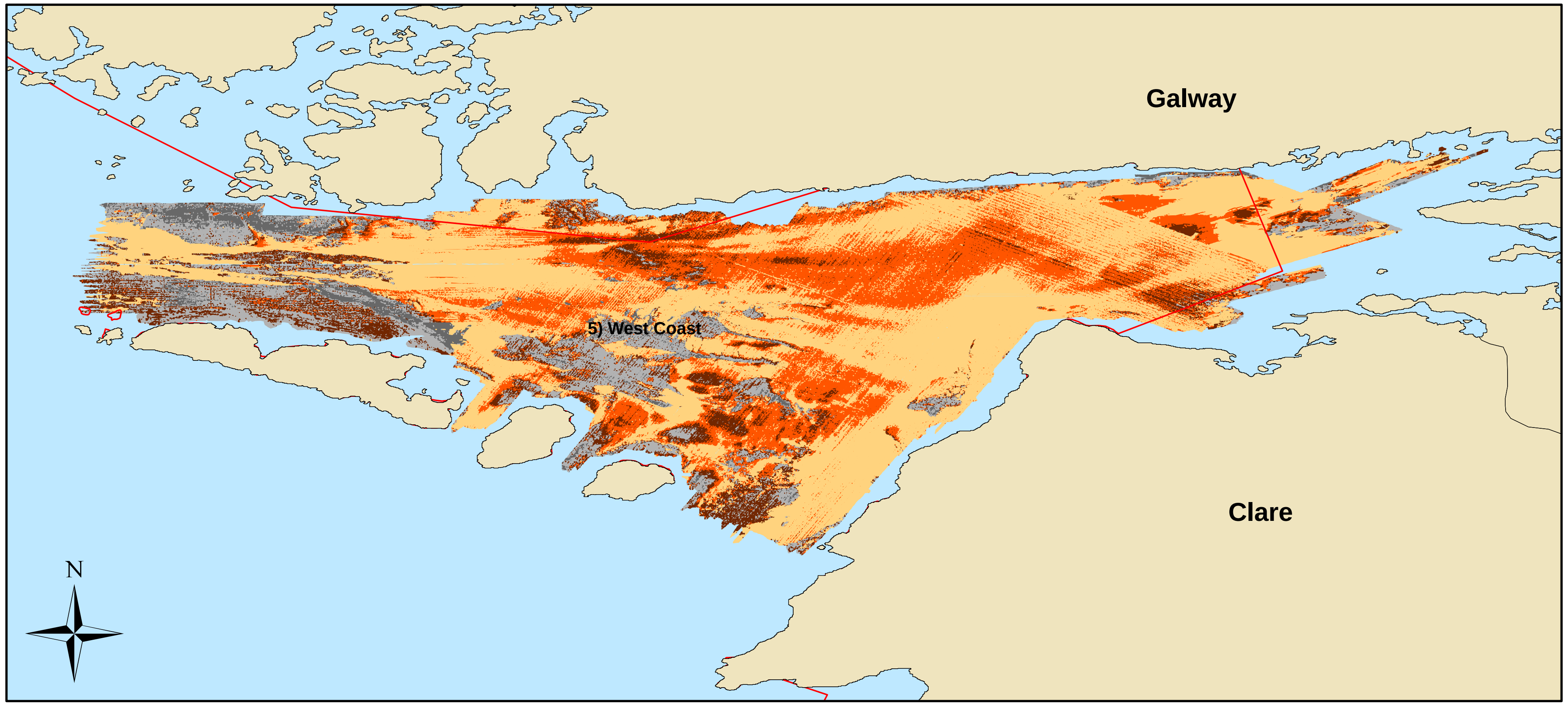
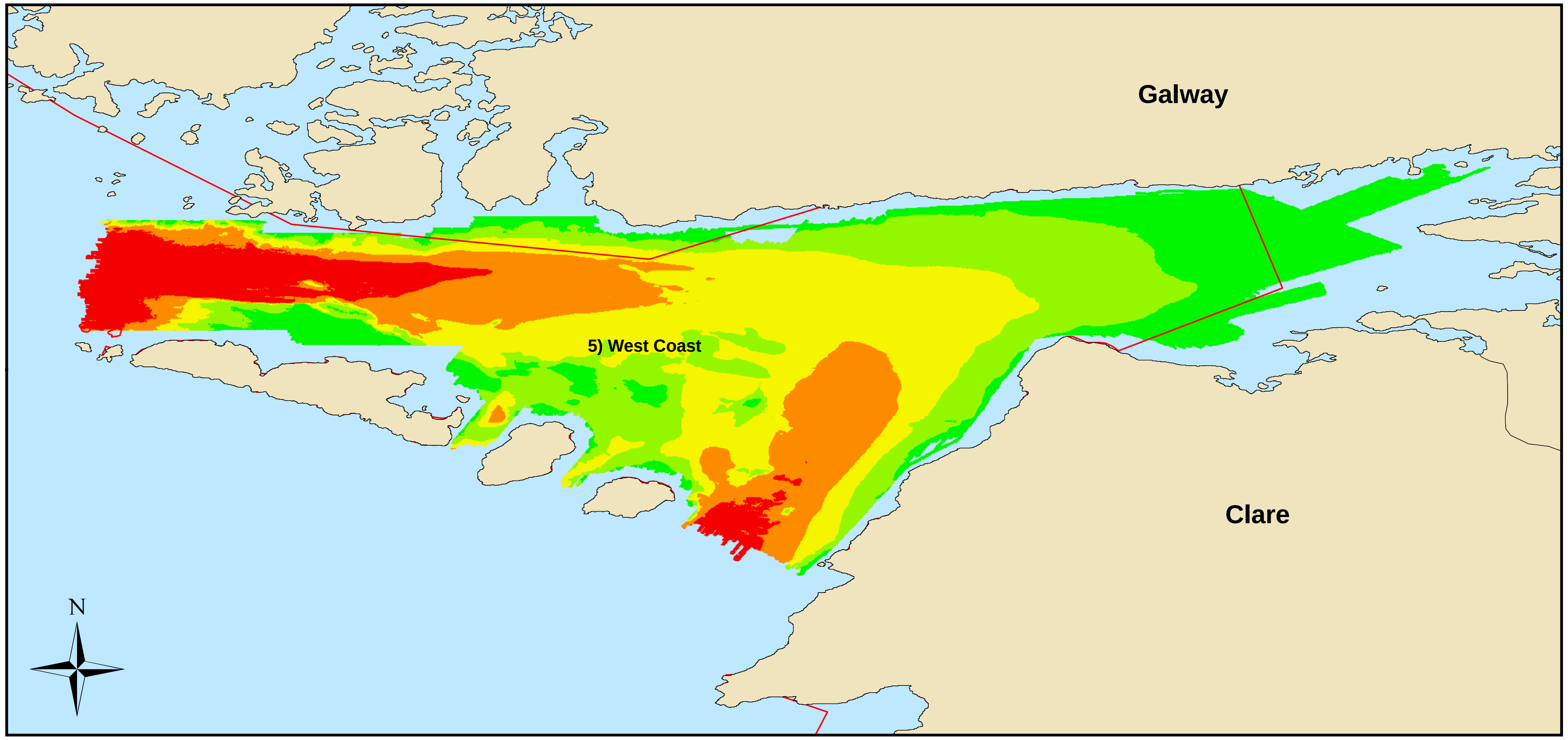
Assessment Zone 5 - West Coast





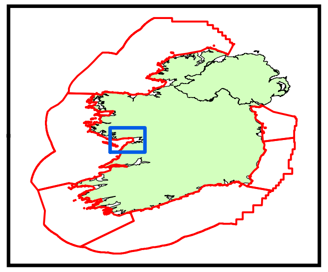
Legend

- Ireland
- Assessment Zones
- Fine Sands to Mud
- Coarse to Medium Sand
- Gravels to Coarse Sands
- Rock (High Texture)
- Rock

Batthymetry

Depth

- 70m - 50m
- 50m - 40
- 40m - 30m
- 30m - 20m
- 20m - 5m



0 2.5 5 10 15 20 Kilometers

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Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

Rev:		
Title:	Zone 5Bathymetry and Soil Properties	
Dwr No:	11011-03-DWR08	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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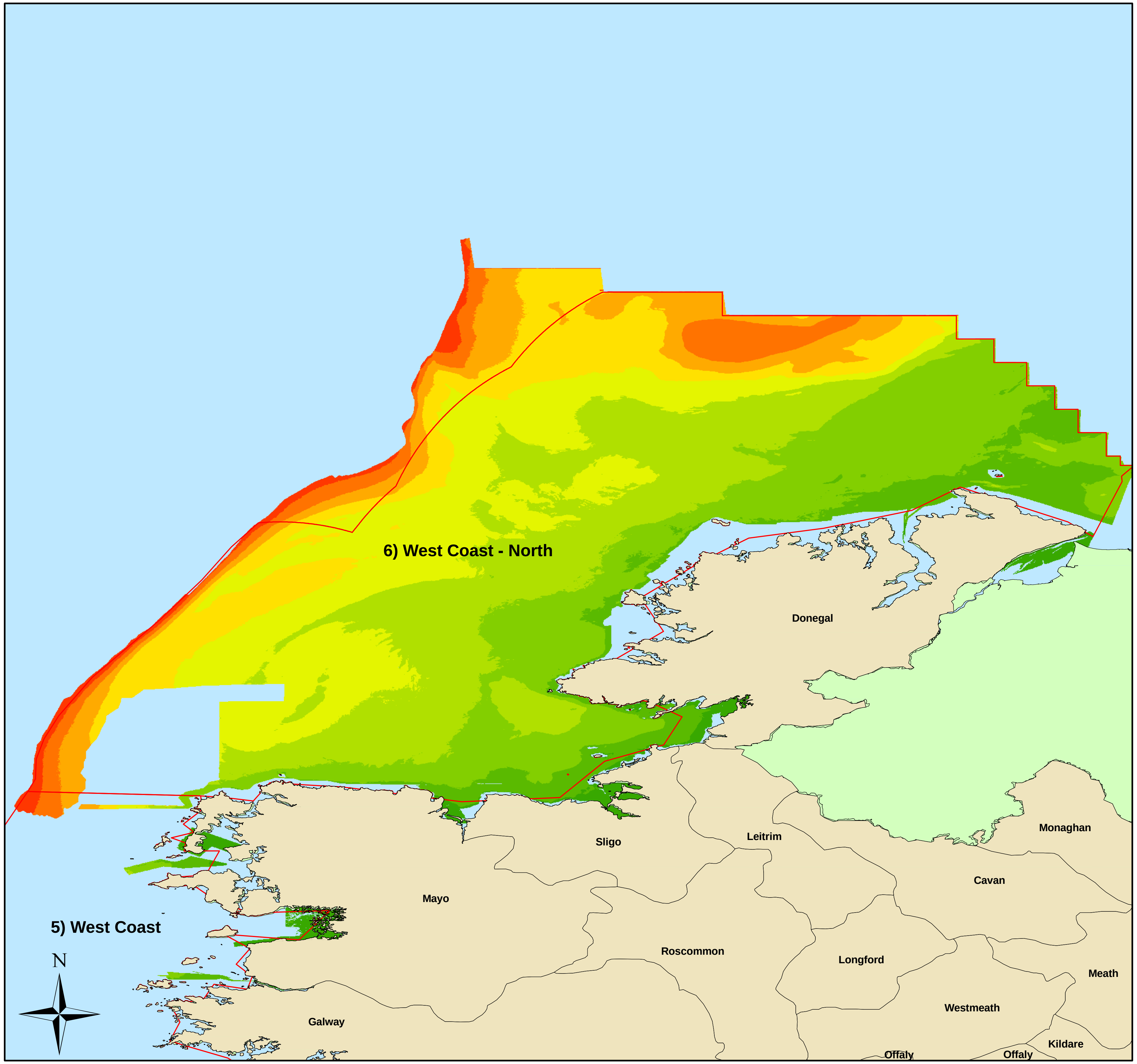
Assessment Zone 6 - West Coast - North



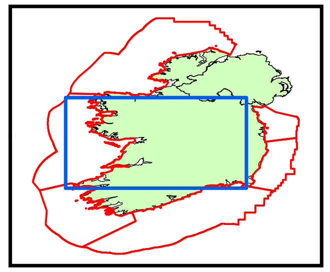
Legend

- Northern Ireland
- Ireland
- Assessment Zones

Bathymetry

Depth

- 375m - 250m
- 250m - 180m
- 180m - 160m
- 160m - 135m
- 135m - 115m
- 115m - 95m
- 95m - 75m
- 75m - 50m
- 50m - 20m
- 20m - 5m



0 10 20 40 60 80 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:

1: Not to be used for navigation

Rev:		
Title:	West Coast North - Bathymetry Overview	
Dwr No:	11011-03-DWR09	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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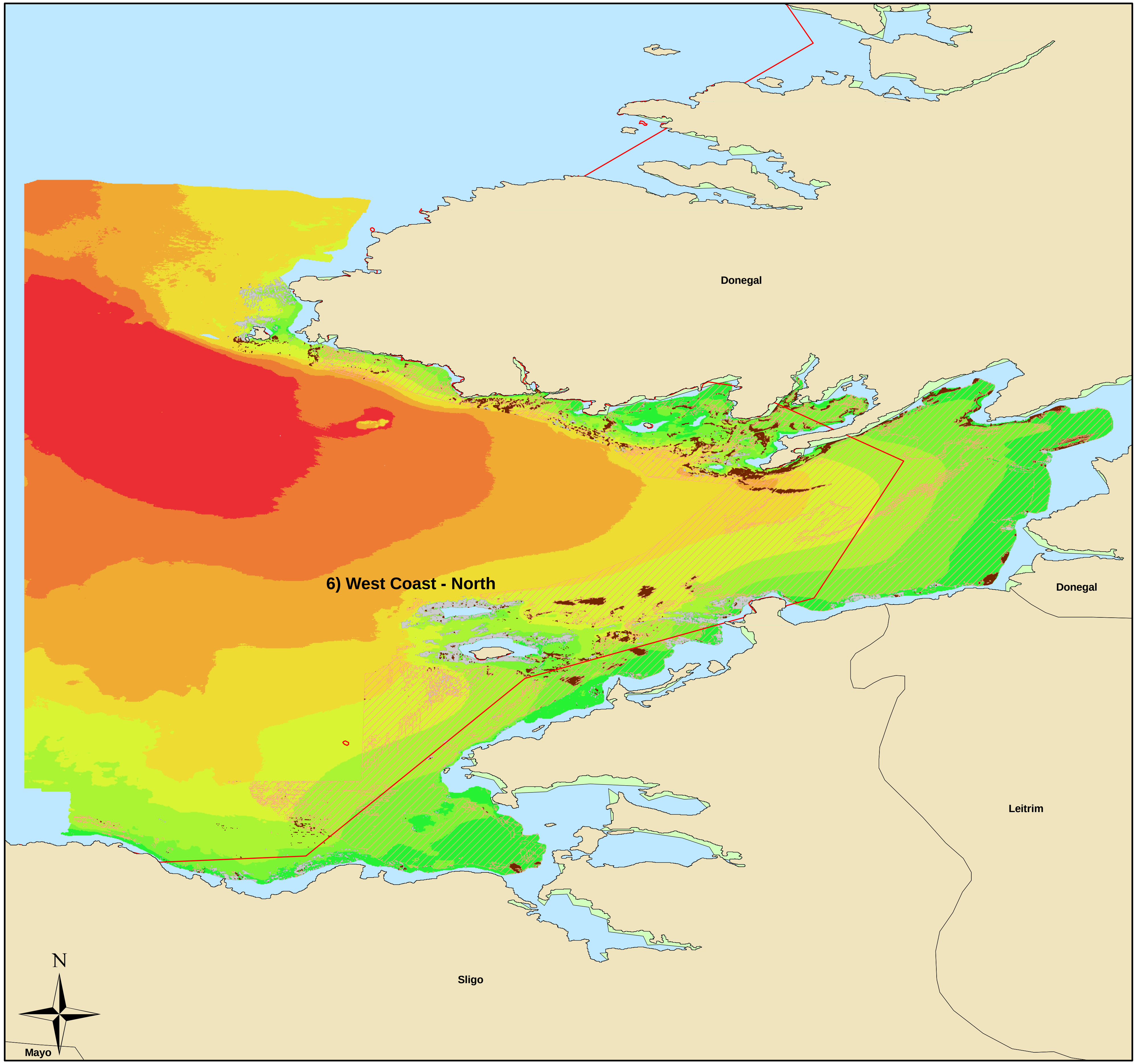
Assessment Zone 6 - West Coast - North



Legend

- Ireland
- Assessment Zones
- Northern Ireland
- Sand
- Sand veneer over Rock
- Class 4 Rock (Less texture)
- Rock

Bathymetry

Depth

- 90m - 80m
- 80m - 70m
- 70m - 60m
- 60m - 50m
- 50m - 40m
- 40m - 30m
- 30m - 20m
- 20m - 4m

0 2 4 8 12 16 Kilometers

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Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

Rev:		
Title:	Bathymetry - Soil Properties	
Drw No:	11011-03-DWR10	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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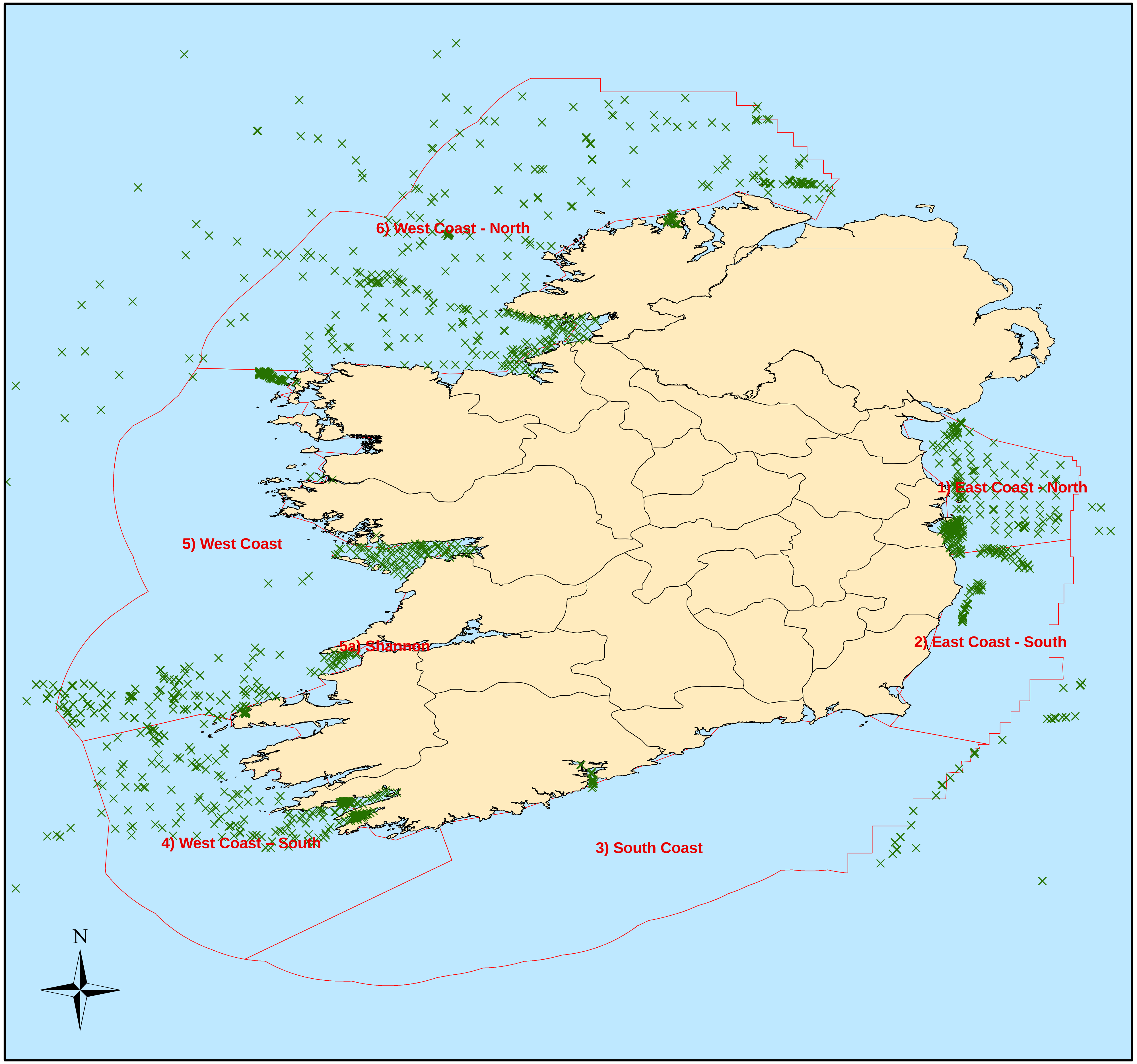
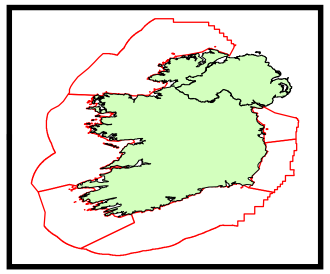
INSS Ground Truthing Locations



Legend

- INSS Groundtruthing Sample Location
- Ireland
- Assessment Zones



0 20 40 80 120 160 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Notes on each sample taken available through INFOMAR

Rev:		
Title:	Ground Truthing Locations	
Dwr: No:	11011-03-DWR11	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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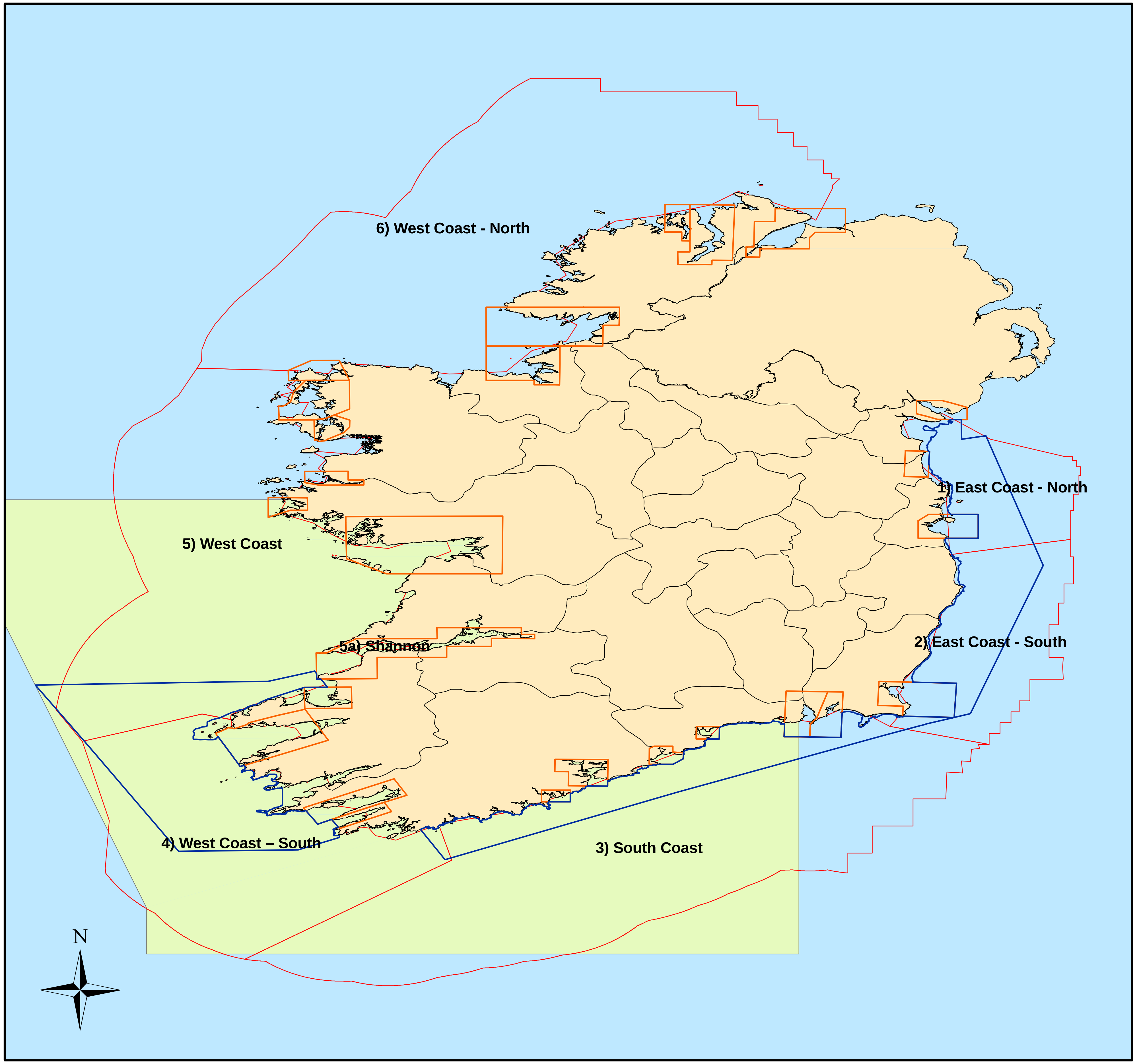
Priority Bays and Areas



Legend

- New_Priority_Areas_12April2011
- New_Priority_Bays_12April2011
- Ireland
- Assessment Zones
- BIOLOGICAL_SEN

0 20 40 80 120 160 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Priority Areas and Bays as given in INFOMAR Web Map Service

Rev:		
Title:	Priority Bays and Areas	
Drw No:	11011-03-DWR12	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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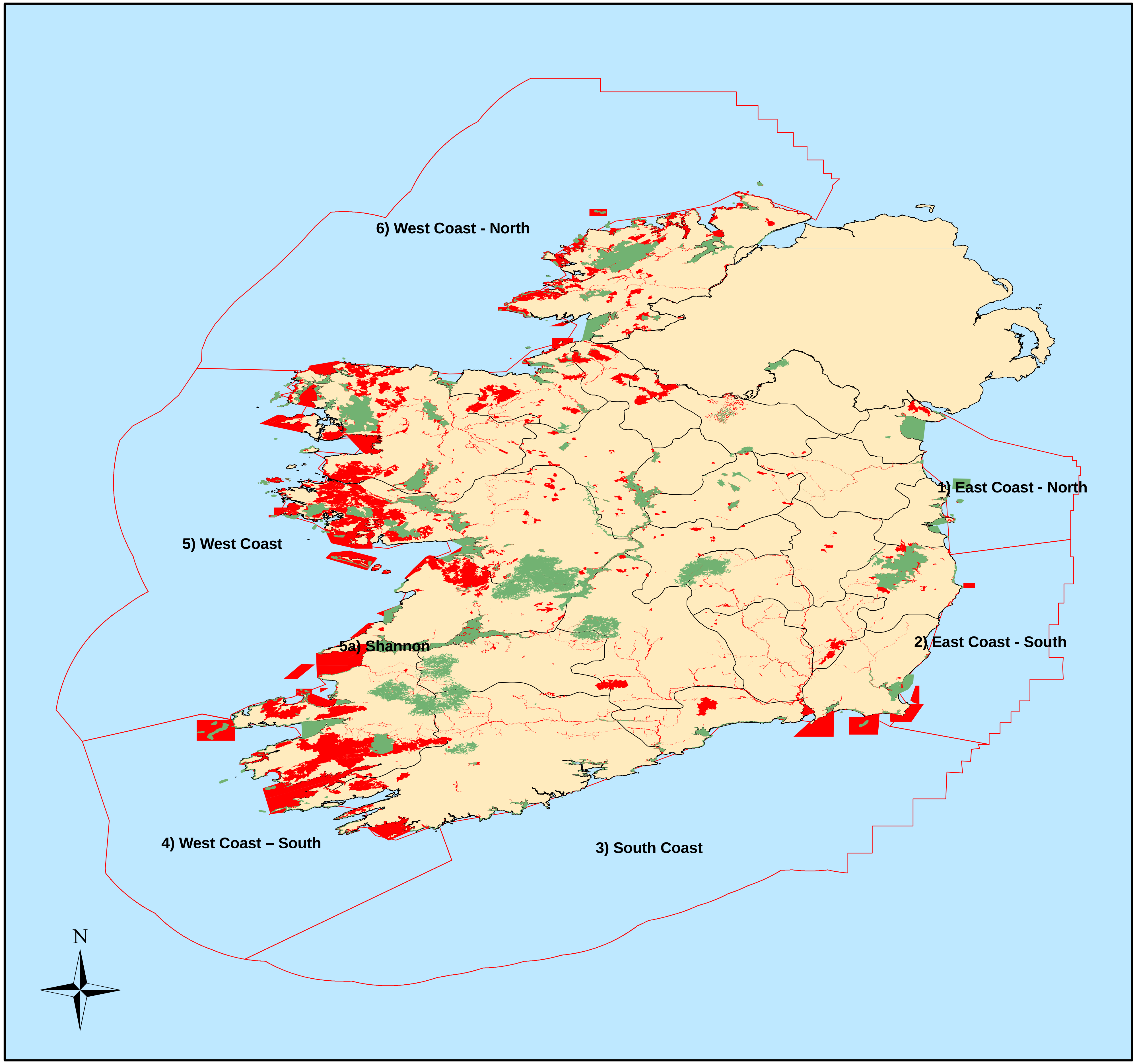
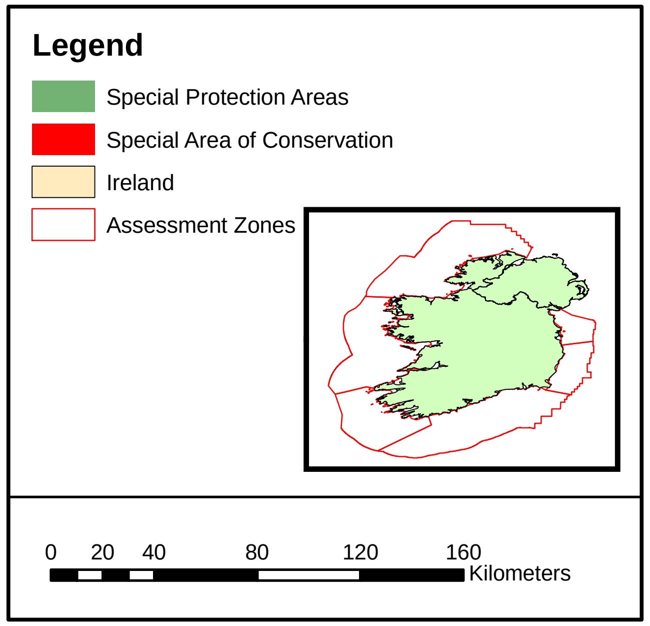
Conservation and Protection Areas





INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: see www.npws.ie for designation parameters

Rev:		
Title:	Conservation and Protection Areas	
Drw No:	11011-03-DWR13	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	National Parks and Wildlife Services Map Viewer	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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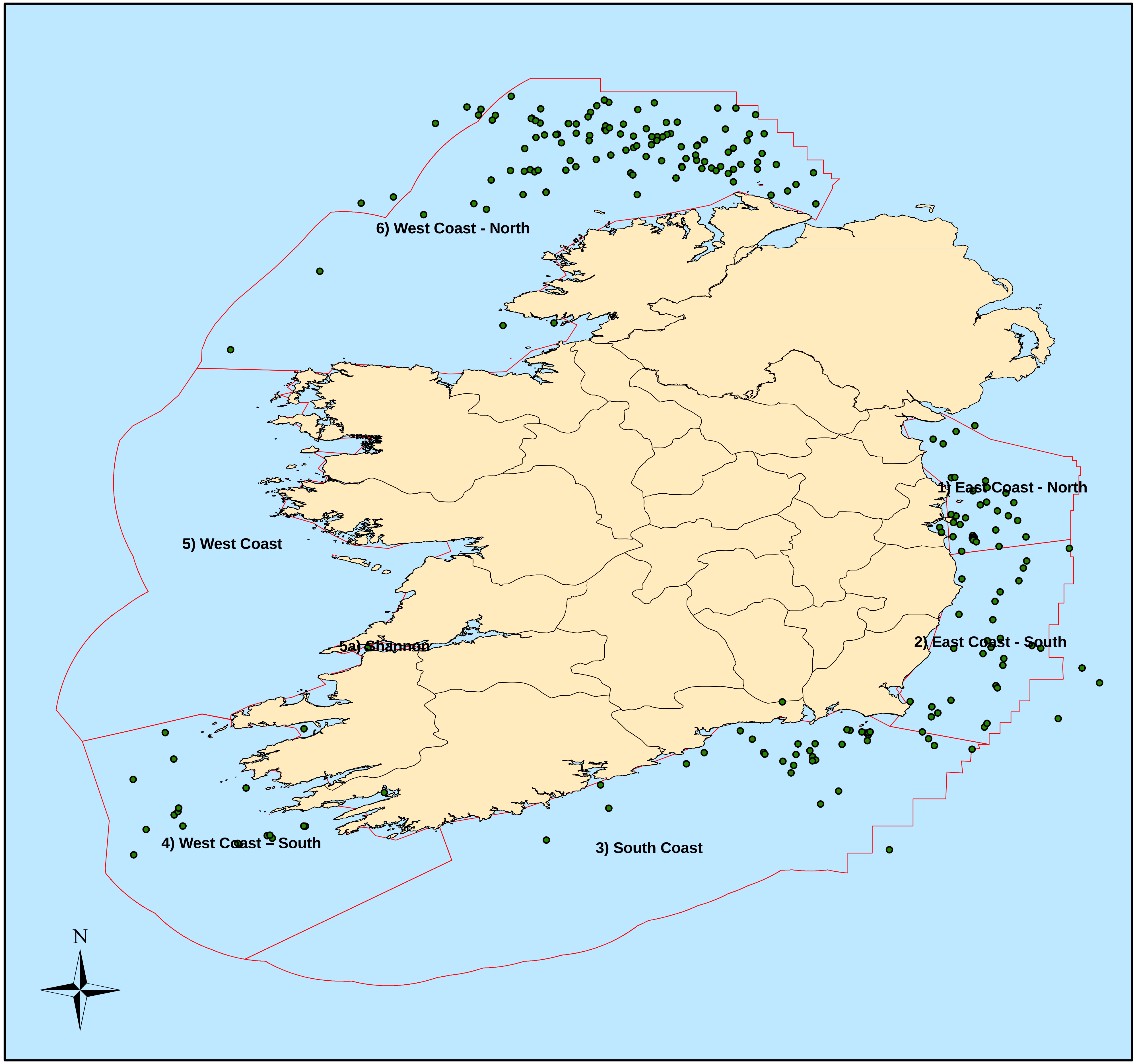
Ship Wreck Locations



Legend

- Shipwreck Locations
- Ireland
- Assessment Zones

0 20 40 80 120 160 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Priority Areas and Bays as given in INFOMAR Web Map Service

Rev:		
Title:	Constraints - Ship Wrecks	
Dwr No:	11011-03-DWR14	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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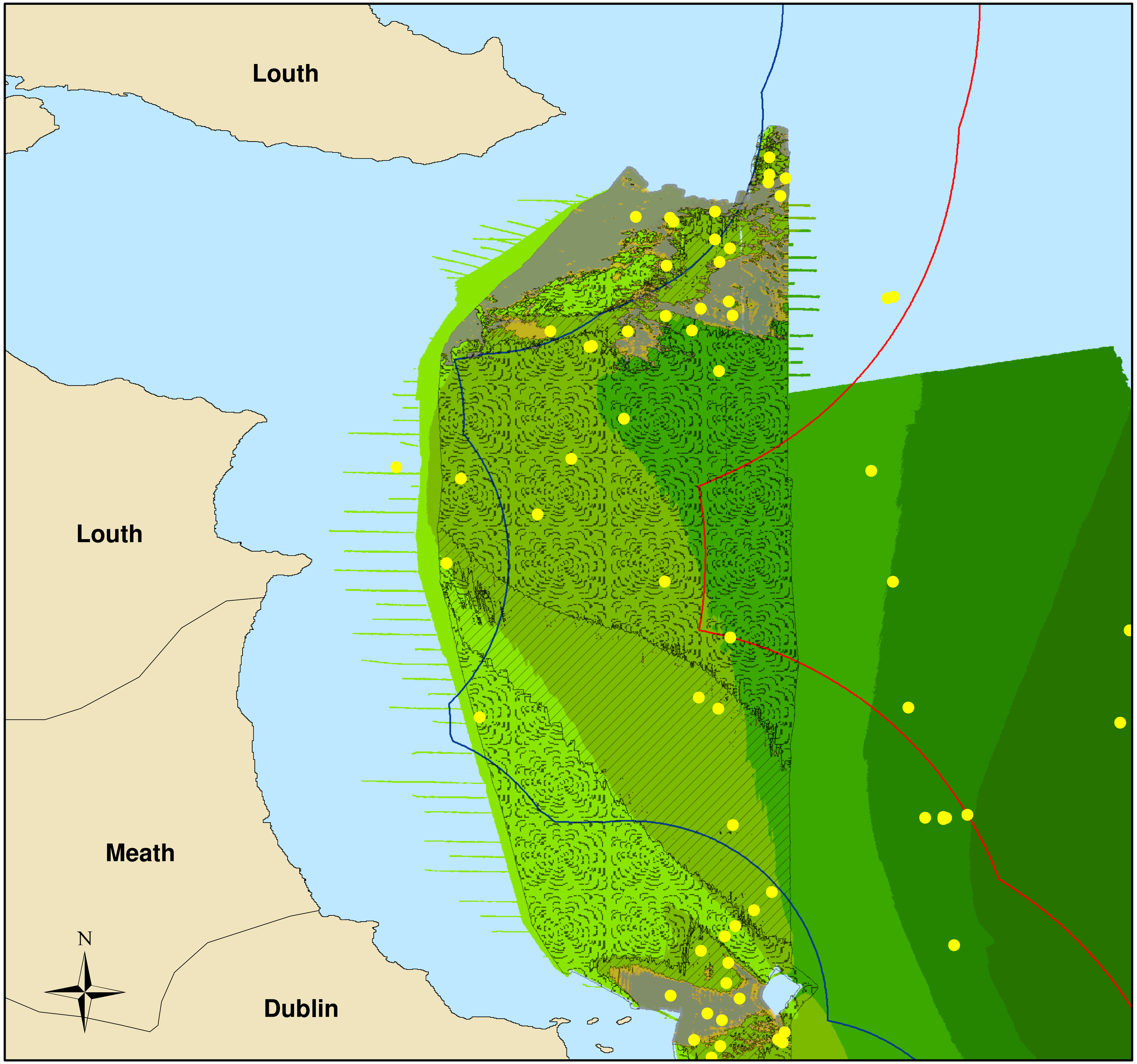
Appendix C

Proposed Locations for CPT Data

Assessment Zone 1 - East Coast - North



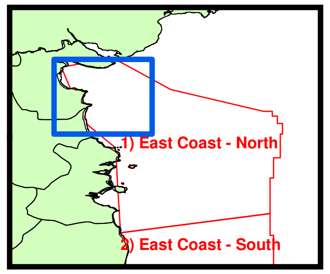
Legend

- Groundtruthing Data Point
- Ireland
- Mud to fine sand
- Fine to medium sand
- Course sand and gravel
- Rock and gravel
- 20km Contour
- 10km Contour

Bathymetry

Depth

- 60m - 50m
- 50m - 40m
- 40m - 30m
- 30m - 20m
- 20m - 0m



0 1.5 3 6 9 12 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

Rev:		
Title:	Zone 1 Site Study- Dundalk Bay	
Dwr No:	11011-03-DWR15	
Date:	05-12-2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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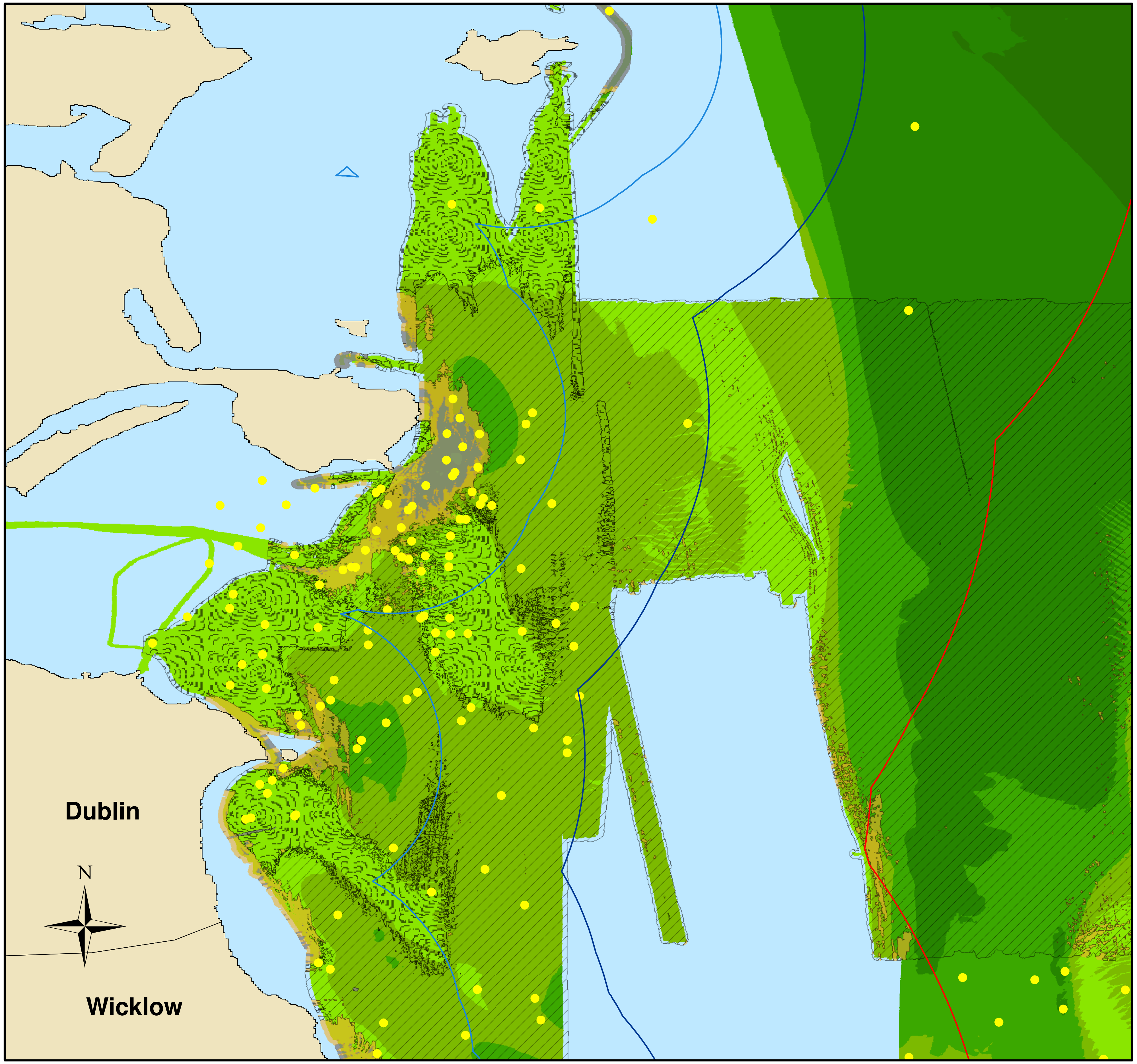
Assessment Zone 1 - East Coast - North



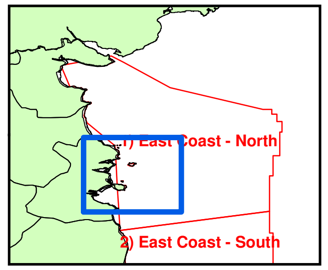
Legend

- Ireland
- 20km Contour
- 10km Contour
- 5km Contour
- Groundtruthing Data Point
- Mud to fine sand
- Fine to medium sand
- Course sand and gravel
- Rock and gravel

Bathymetry

Depth

- 60m - 50m
- 50m - 40m
- 40m - 30m
- 30m - 20m
- 20m - 0m



0 1.25 2.5 5 7.5 10
Kilometers

INFOMAR
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Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

Rev:		
Title:	Zone 1 Site Study - Dublin Bay	
Dwr No:	11011-03-DWR16	
Date:	05-12-2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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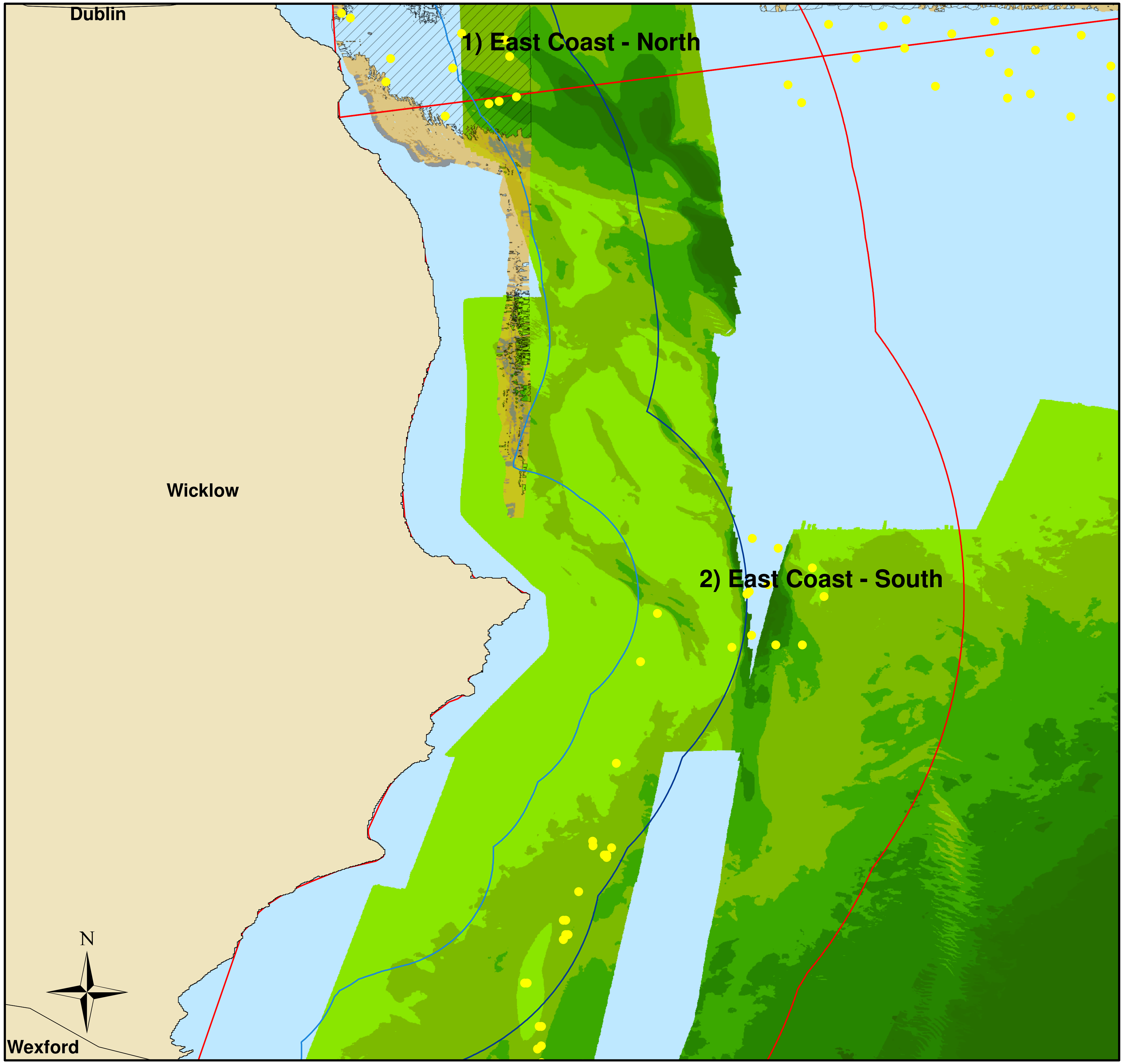
Assessment Zone 2 - East Coast - South



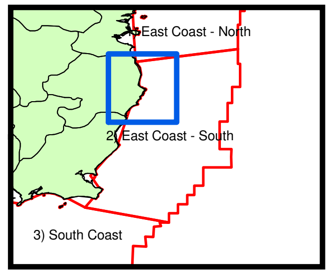
Legend

- Ireland
- 20km Contour
- 10km Contour
- 5km Contour
- Assessment Zones
- Groundtruthing Data Point
- Mud to fine sand
- Fine to medium sand
- Course sand and gravel
- Rock and gravel

Bathymetry

Depth

- 50m+
- 50m - 40m
- 40m - 30m
- 30m - 20m
- 20m - 0m



0 0.5 1 2 3 4 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

Rev:		
Title:	Zone 2 - Wicklow Coast	
Drw No:	11011-03-DWR17	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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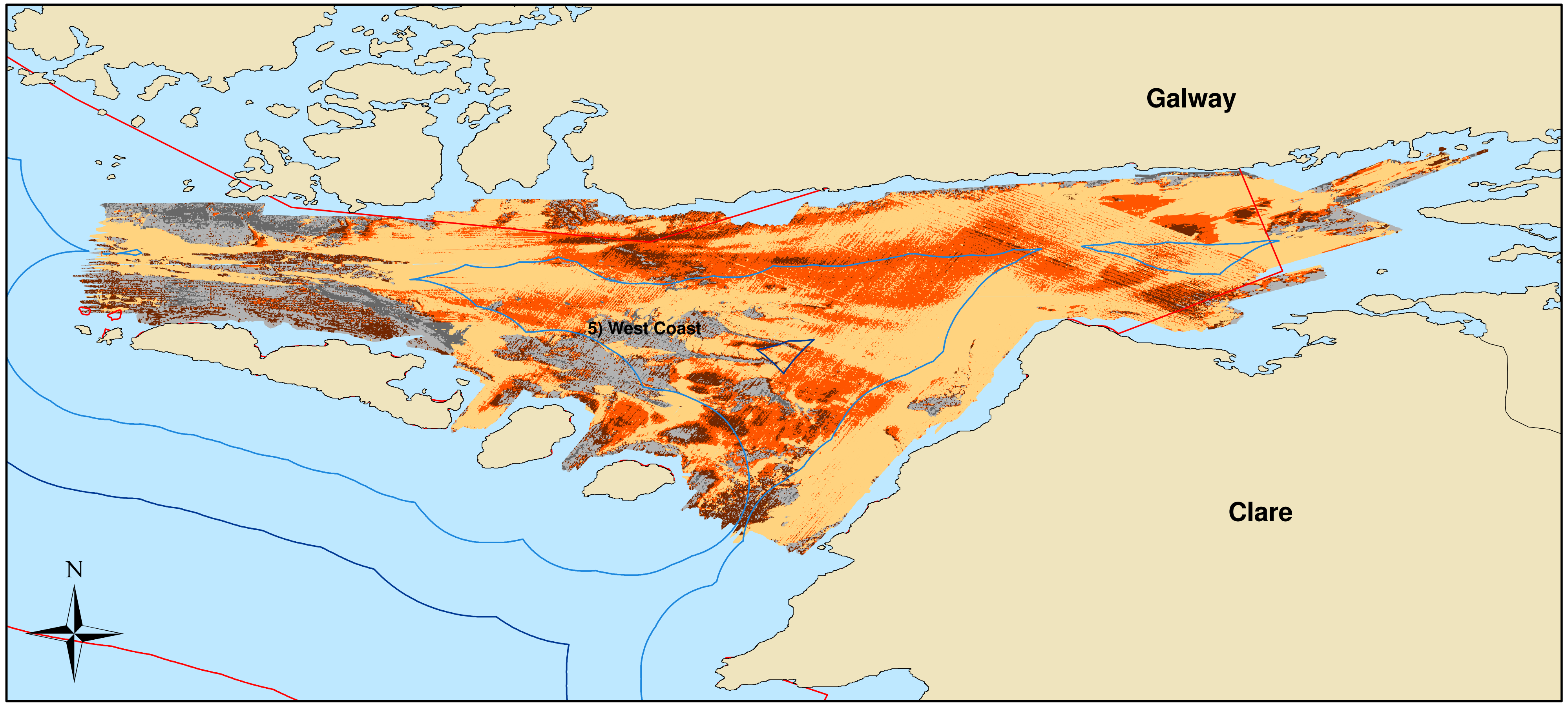
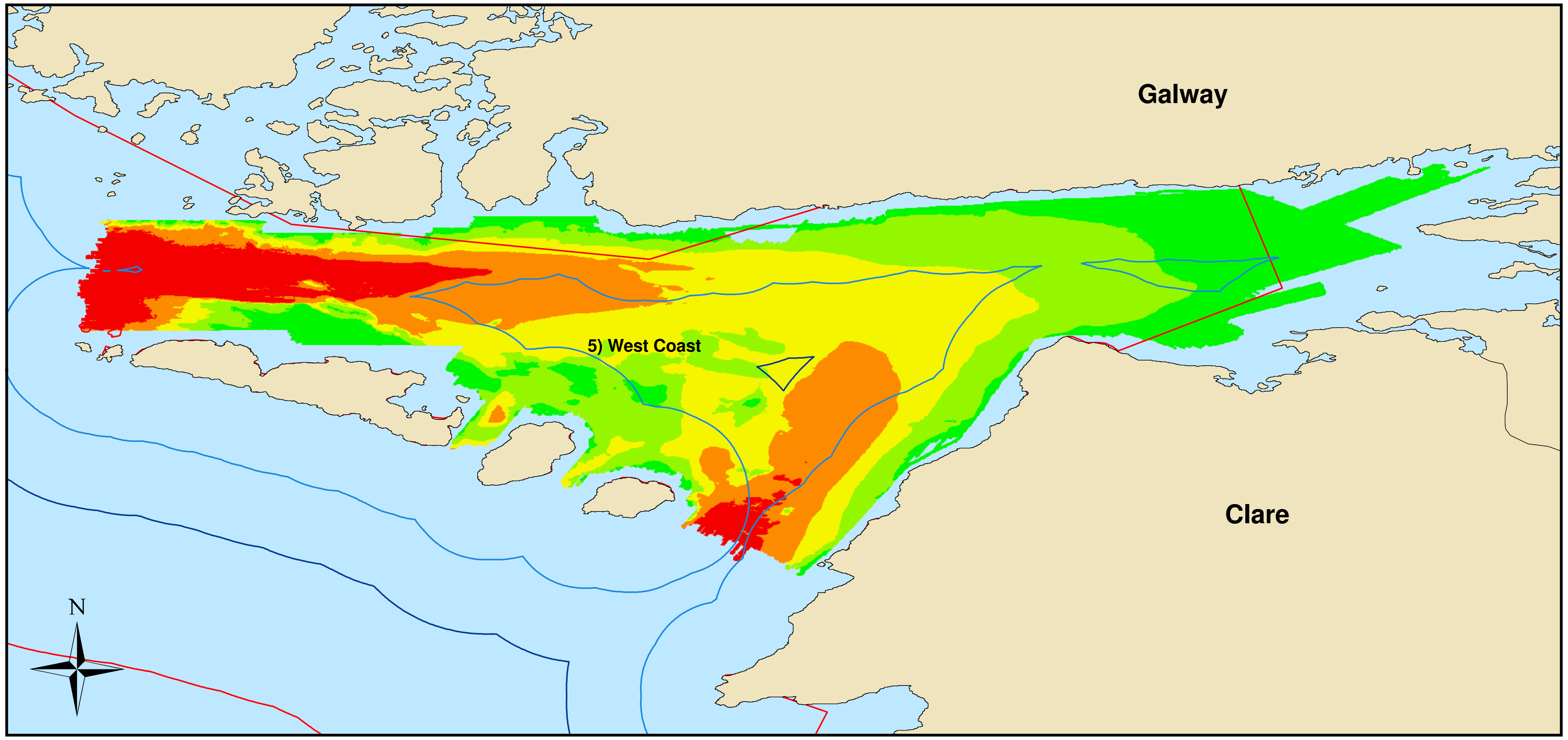
Assessment Zone 5 - West Coast





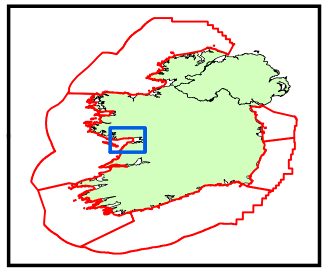
Legend

- Ireland
- Assessment Zones
- Fine Sands to Mud
- Coarse to Medium Sand
- Gravels to Coarse Sands
- Rock (High Texture)
- Rock
- 20km Contour
- 10km Contour
- 5km Contour

Bathymetry

Depth

- 70m - 50m
- 50m - 40
- 40m - 30m
- 30m - 20m
- 20m - 5m



0 2.5 5 10 15 20 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

Rev:		
Title:	Zone 5Bathymetry and Soil Properties	
Dwr No:	11011-03-DWR08	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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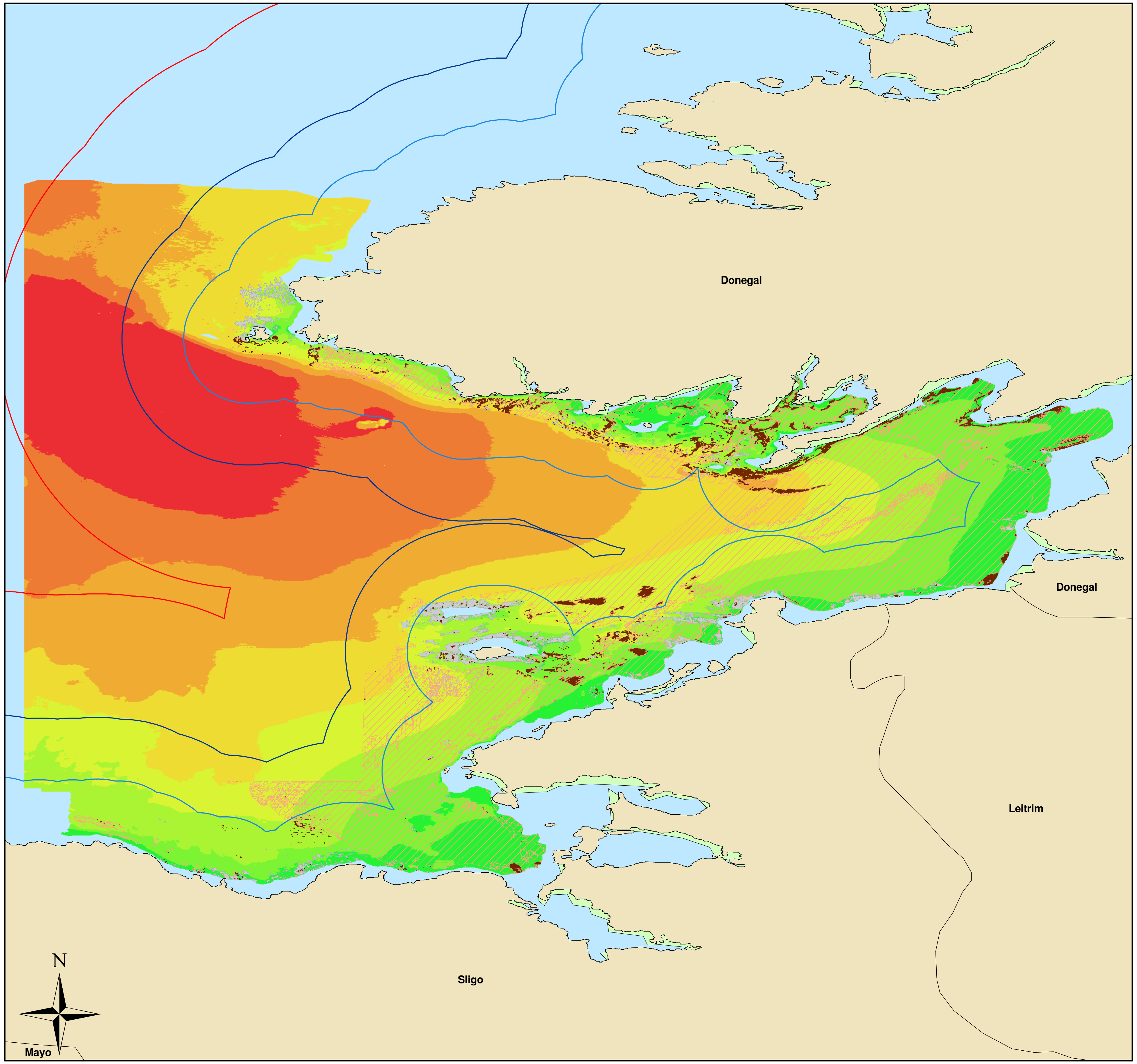
Assessment Zone 6 - West Coast - North



Legend

- Ireland
- Northern Ireland
- Sand
- Sand veneer over Rock
- Class 4 Rock (Less texture)
- Rock
- 20km Contour
- 10km Contour
- 5km Contour

Bathymetry

Depth

- 90m - 80m
- 80m - 70m
- 70m - 60m
- 60m - 50m
- 50m - 40m
- 40m - 30m
- 30m - 20m
- 20m - 4m

0 2 4 8 12 16 Kilometers

INFOMAR
Integrated Mapping for the Sustainable Development of Ireland's Marine Resource

Notes:
1: Not to be used for navigation
2: Soil type determined using visual based classification as presented in INFOMAR data - soil not classified by GDG

Rev:		
Title:	Bathymetry - Soil Properties	
Drw No:	11011-03-DWR10	
Date:	5/12/2012	
Ref:	INF-11-24-DOH	
Projection:	WGS_1984_UMT_Zone_29N	
Source:	Infomar Interactive Web Data Delivery System	
Projection:	WGS_1984_UMT_Zone_29N	
Checked:	Produced by:	G Murphy
	Checked by:	P Doherty

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