香港國際機場 | HONG KONG INTERNATIONAL AIRPORT

Important Note

Airport Authority Hong Kong (AAHK) is responsible for preparing the Hong Kong International Airport (HKIA) Master Plan 2030 and commissioning the associated consultancies. At different stages of these consultancies, the consultants produced various documents for AAHK's consideration, culminating in the production of final reports. Where a final report was not produced, the consultants' work was consolidated into the HKIA Master Plan 2030 Technical Report. As the reports were produced at different times, they may contain outdated or inconsistent contents.

The HKIA Master Plan 2030 was not drawn up solely on the basis of the various consultancies commissioned by AAHK, but also has incorporated input from relevant airport stakeholders as well as AAHK's own input on the basis of its solid experience in airport operations. Hence, for any differences between the consultancy reports and the HKIA Master Plan 2030, the latter and the Technical Report should always be referred to.

Airport Authority Hong Kong

July 2011

香港國際機場
HONG KONG INTERNATIONAL AIRPORT

Hong Kong International Airport

Contract P131 - Initial Land Formation Engineering Study for Airport Master Plan 2030

Final Construction Options Report Rev. 1

August 2009

MEINHARDT
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91134

Hong Kong International Airport

Contract P131

Initial Land Formation Engineering Study for Airport Master Plan 2030

Final Construction Options Report Rev1

Meinhardt (Hong Kong) Ltd
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Airport Master Plan 2030
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Final Construction Options Report

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1 INTRODUCTION

1.1 Details of Appointment

1.1.1 On 25th July 2008, Airport Authority Hong Kong appointed Meinhardt (Hong Kong) Ltd. (MHKL), under Contract No. P131, to provide consultancy services in respect of the initial land formation engineering study associated with the New Airport Master Plan 2030, referred to as the Assignment. Meinhardt have engaged the services of GCG (Asia) Ltd, to provide supporting consultancy services in relation to specialist geotechnical issues. Further, specialist advice has been sought from a number of internationally accredited experts in the subject of reclamation over soft marine clays.

1.1.2 The assignment is required to investigate options associated with site formation or reclamation to form new land in an area positioned over the top of the contaminated mud pits, located to the north of the existing airport platform. The site created through the reclamation or site formation techniques assessed in the study will be used to support the runways and associated outfield infrastructure necessary for the expansion of the airport to meet the requirements of proposals defined in the Airport Master Plan 2030.

1.1.3 The material in the mud pits is likely to be very variable, of very low strength and of much greater thickness than has generally been addressed in past similar projects. Initial investigations also indicate that the CRPC pit was filled with trailer dredger won marine sediments and therefore contains predominantly slurrified material. Other pits retain material derived from grab dredging which more closely resembles clay lumps, interspersed with slurry, water filled voids and disturbed materials. This study therefore acknowledges that these materials will respond in differing ways and the assignment will be required to investigate this particular aspect.

1.1.4 Recommendations for preliminary site investigation works have been developed with the objective of investigating the range of strength and consolidation parameters likely to be experienced during construction. The study is also required to assess the age of the materials in the pits and their consolidation history since they were dumped.

1.1.5 The preliminary site investigation work was undertaken during the course of this study and the results from this work have been included in the assessment of the materials lying within the pits. It is proposed that a comprehensive assessment of the findings of the preliminary SI should be undertaken as an annexe to this study, the findings of the comprehensive assessment will assist in developing a definitive series of geotechnical parameters upon which more detailed design work can be based.

1.1.6 It is important to emphasise that a fully comprehensive site investigation should be undertaken prior to detailed design and construction, with the objective of not just determining the applicability of the preferred options but also to identify specific areas where difficulties might be encountered in the application of the techniques being proposed.

1.2 Notional Study Area

1.2.1 The proposed study work comprising this Assignment and covered by this report can be summarised as follows:-

(a) Investigation, identification and determination of all feasible construction options for land formation over or near the Contaminated Mud Pits requiring no or minimum disturbance to the contaminated material and having minimum environmental impact;

(b) Preliminary assessment of the environmental impact of each identified option;

(c) Review of programme impacts for each identified option; and

(d) Estimation of likely costs associated with each option;

(e) Completion of a preliminary risk assessment in relation to the engineering, environmental impact and costs associated with each option.

(f) Engineering constraints are to be ascertained to determine the feasibility of the respective land formation options but will be restricted to being based upon currently available SI data for the site. The study will however incorporate the findings of any additional SI that might become available within the timescale of the assignment.

(g) The study is not required to investigate reclamation outside areas occupied by the contaminated mud pits where traditional reclamation and dredging methods may be used but is required to address issues associated with construction that will have an interface with the contaminated mud pits.

(h) Review of literature to identify and acquire relevant publications and data necessary to complete the assignment;

1.2.2 The extent of the assignment investigation area is detailed on Drawing No. PSU/P131/C/001 Rev A reproduced in *Figure 1.1.* The drawing depicts a notional study area which includes general filling areas, areas proposed to support a potential runway, taxiways and supporting infrastructure as well as sea walls. The assessment of the various options from the point of view of their feasibility has been based upon the development of the site in line with the plan arrangement depicted in *Figure 1*.

1.2.3 As built records of the dredged profiles of the various pits have been retrieved from CEDD's archives. These records show the dredged profile of each pit immediately prior to filling. These records have been included in the accompanying volume to this document (Site Investigation Summary Report) and were prepared under this assignment.

1.2.4 Following the completion of the initial assessment options report a review of the assignment concluded that it would be more appropriate to undertake cost estimating for comparison of the various development options by assuming a plan arrangement for the site in line with the outcome from other planning studies being undertaken as part of the Airport Master Plan 2030 development. This study has therefore been completed in accordance with the requirements of the study brief by assessing the development issues for various techniques assuming the study area as detailed in *Figure 1.1.* However, the cost estimates for the various options in this report have been developed based upon a more appropriate site layout as defined in *Figure 1.2.* This arrangement is described as Option R (C+Y) and makes provision for runways and taxiways of comparable dimensions to the existing facilities and an apron area (ref. Area Y in *Figure 1.2*) comparable with the western apron area on the existing airport platform.

1.2.5 The notional reclamation as depicted in *Figure 1.1* occupies an area approximately 3.35km by 1.80km, having a plan area bounded by the seawall copeline of 582 Ha. The proposed reclamation area will be bounded by 6.35km of new seawall and will abut the existing airport platform reclamation along one side over a length of 3.35km.

1.2.6 The reclamation arrangement assumed for cost estimating and for preliminary evaluation of environmental impacts as detailed in *Figure 1.2* and Section 8 of this report occupies an area having a plan area bounded by the seawall copeline of 589 Ha. The proposed reclamation area will be bounded by 11.9km of new seawall and will abut the existing airport platform reclamation along one side over a length of 4.9km.

1.2.7 In undertaking the cost estimating exercises it has been acknowledged that the terminal buildings will be common to all the options. These elements have therefore been excluded from the cost estimates although their presence has been acknowledged in terms of the environmental impacts that the piling associated with their construction will have.

1.2.8 The notional investigation area has been selected with the intention of including reclamation as follows:-

- Reclamation over shallow (15m nom depth) mud pits I, II a, b, c, d and III a, b, c and d where dumped material was won through grab dredging;

- Reclamation over the shallow (15m nom depth) CRPC mud pit where dumped material was won through trailer dredging operations; and

- Reclamation over the deep (25m nom) mud pit IV a containing grab dredged contaminated materials.

1.2.9 Similarly, the notional formation area has been selected such that seawall construction will be required over the following mud pits:-

- Seawalls over shallow (15m nom depth) mud pits II a and III a containing grab dredged materials;

- Seawalls over the shallow (15m nom) CRPC mud pit containing trailer dredged materials; and

- Seawalls over the deep (25m nom) mud pit IV a containing grab dredged materials.

2 EXISTING GEOTECHNICAL CONDITIONS

2.1 Existing Site Layout

2.1.1 The proposed site for the notional expansion of the airport platform lies to the north of the existing airport platform and interfaces with the seawall which lies parallel with the northern runway. Details of the arrangement are illustrated in *Figure 1.1.* The site area occupied by the mud pits is currently gazetted for contaminated mud disposal and is managed by CEDD on behalf of the HKSAR Government. Filling of the mud pits has been completed for some time and the seabed profile in the area has generally been restored to levels close to those which originally existed, with the exception of Mud pit Iva, IVb and IVc. These mud pits have still not been fully capped off and seabed levels in this area are lower than surrounding areas by 3 – 5m.

2.2 Existing Reclamation and Seawall

2.2.1 The existing reclamation against which any new works must lie has been formed as a naturally armoured (rock armoured) seawall. The existing structure was formed by dredging the soft marine deposits from beneath the new formation, before bottom dumping and end tipping a rockfill core with subsequent armouring. A typical cross section of the seawall in the study area is illustrated in *Figure 2.1*. The arrangement is typical of seawalls constructed around major reclamations in Hong Kong, relying for stability on the fact that the soft marine deposits are removed and replaced with sandfill before construction of the main rockfill bund. Sandfilling in this type of construction is generally undertaken up to the original seabed level before deposition of the rockfill mound.

2.2.2 The seaward face of the existing wall has been constructed at a slope of 1:2 and is armoured with 2 layers of nominally 5 Tonne armour stones over two 1m thick underlayer courses of typically 1-2 Tonne stone. At the time the seawall was designed, Greiner Maunsell undertook an assessment of the most appropriate armouring size, concluding that the seawall face slope should be constructed with 5 Te armour stone placed at 1:2 in preference to 7 Te armour stone placed on a 1:1.5 slope. This decision was made based upon the availability of suitable armouring stone. It is noted that a similar assessment undertaken at the present time would probably reach the same conclusion as the supply of rock armour stone, if anything, has become more restricted in recent years.

2.2.3 Scour protection along the toe of the seaward facing embankment has been achieved by using a buried scour apron. This approach is most efficient in terms of providing the required degree of protection with the minimum quantity of armour. Generally, the toe scour apron has been constructed to be compatible with a seabed level of -6.0 m PD.

2.2.4 The coping level of the existing sea wall has been set at +6.5 m PD with the adjacent formation level which supports the perimeter access road being set nominally at + 6.0 m PD level.

2.2.5 Greiner Maunsell (1991b) quote that the existing seawall was designed to accommodate static water levels at 1:100 year return periods of between - 0.55 m PD and + 4.05 m PD. Details are presented in *Table 2.1***.**

Table 2.1 Water Levels for Seawall Design

Design Sea State	Return Period (Years)	Water Level (mPD)
Normal – low	5	-0.15
Normal – High	5	+2.95
Extreme – Low	100	-0.55
Extreme – High	100	+4.05

(after Greiner Maunsell, 1991b)

2.2.6 The seawall armouring was designed to accommodate attack from waves as detailed in *Table 2.2.*

Table 2.2 Significant Wave Heights and Mean Periods used in Seawall Design

Return Period Years	Significant Wave Height Hs (in m) and Period Tz (in s)					
	Western Walls		Northern Walls		Eastern Walls	
	Hs	Tz	Hs	Tz	Hs	Tz
2	2.2	4.3	2.0	4.1	1.5	3.3
5	2.5	4.6	2.3	4.4	2.0	3.8
10	2.9	4.9	2.6	4.7	2.3	4.1
20	3.2	5.1	2.9	4.8	2.6	4.4
50	3.5	5.2	3.2	5.0	2.9	4.5
100	3.9	5.5	3.5	5.2	3.0	4.6
500	4.6	5.8	4.2	5.6	3.2	4.7
1000	5.0	6.0	4.5	5.7	3.5	4.8

(after Greiner Maunsell, 1991b)

2.2.7 The reclamation supporting the existing airport infrastructure has generally been formed to a baseline level of + 6.0 m PD with the site being graded for reasons of surface drainage. Finished levels therefore range between about +6.5 m PD and +6.0 m PD. The site formation around the eastern end of the site and supporting the terminal buildings has generally been constructed at a higher general level with the site grading down to a perimeter road set at about +6.5 m PD level. Formation levels in this area generally range between +6.5 m PD and +7.0 m PD.

2.2.8 The runways at the existing platform have been constructed at slightly higher levels than the surrounding reclamation and taxiways. They have been completed to curved longitudinal profiles to assist with aircraft take off and landing requirements. The South runway varies in level between +9.5 m PD level at each end whilst the alignment sags in its central portion down to a level of about 6.5 m PD level at its mid point. The profile rises over the end 100 to 150m in a “ski jump” profile terminating at +9.5 m PD level at each end.

2.2.9 The Northern runway has been constructed to a slightly different profile, ranging from about +7.75 m PD at its mid point to +8.25 m PD at each end.

2.3 Bathymetry

2.3.1 Natural water depths to the north of the existing airport platform in the area of the proposed reclamation are typically of the order of – 4 m PD to - 6m PD with a deeper area (around -14 m PD) at the north west corner of the airport where the fuel supply pipeline currently makes landfall. Sea bed levels are illustrated on *Figure 2.2*. The contaminated mud pits I, IIa, IIb, IIc, IId, IIIa, IIIb, IIIc, IIId, and the CRPC pit which are influenced by the proposed reclamation, were dredged to the base of the soft post-glacial deposits typically around 15m below the seabed (-18 to -20m PD) and were constructed with typical side slopes of 1:6 (Ng, 1997; Whiteside et al, 1996). The existing Contaminated Mud Pit IV, to the east of the proposed 3rd runway was excavated in order to provide sand for the existing airport reclamation and was excavated to a depth of around 35m below the seabed (-40 m PD). CMP IV was divided into three disposal cells of which CMP IVa and IVb have been backfilled with contaminated material and partially capped while CMP IVc is still in operation. The proposed reclamation will only affect pit IVa.

2.4 Geotechnical Conditions – Prior to Chek Lap Kok Development

2.4.1 *Marine Mud Deposits*

2.4.1.1 Historical site investigation data which has been made available for this study, primarily from GEO and the Airport Authority archives, indicates that the thickness of the marine deposits comprising the Chek Lap Kok formation materials varies between 12 and 14 m in thickness over much of the area to be affected by the proposed reclamation and seawall works. There is however one area within the study boundary where the marine mud thickness becomes as much as 20m in thickness. This particular area lies just to the south of mud pit I and is centred about 20-25m north of the existing seawall. Isopachs depicting the deduced thickness of the marine mud prior to the formation of the mud pits have been plotted and are presented in *Figure 2.3*. Data used in the preparation of *Figure 2.3* has been derived from both pre and post mud pit construction information available to the general public through the GIU. Application has been made to CEDD for the release of as built survey details for the mud pits, completed immediately prior to their filling. This data has not been made available within the timescale of this study but is being pursued.

2.4.1.2 Details of the location of site investigation works identified during this study and derived from the AA geotechnical data library and from the CEDD Geotechnical information unit are presented in *Figures 2.4 and 2.5*.

2.4.1.3 Design parameters applicable to the undisturbed in-situ marine clays adopted for design purposes by Greiner Maunsell are summarised below in *Table 2.3*

Table 2.3 Summary of geotechnical parameters for soft marine clays adopted for the design of the existing airport platform

Typical Index Properties and Recommended Design Parameters	**Unit**	**Values applicable to the upper soft marine clays**
Unit Weight	Mg / m^3	1.45
Void Ratio e_0	-	2.0
Maximum Past pressure P_p	-	4.5 + 7x Depth below seabed
Compression Index C_c	-	1.20
Recompression Index C_α	-	0.10
Coeff of Consolidation C_v	m^2/yr	1.3
Coeff of Reconsolidation C_{vr}	m^2/yr	20
Undrained Shear Strength ($N_k = Q_{nett} / S_u$) including 0.8 material loading factor	kN/m^2	$N_k = 23.5$

(After Greiner Maunsell (1991c))

2.4.2 *Underlying Materials*

2.4.2.1 The marine clays overlie a series of stiffer and much less compressible alluvial deposits comprising the Hang Hau formation. These materials include inter-bedded sands and gravels and are interspersed with clayey layers. The bearing capacity and compressibility of these materials is such that they are quite capable of supporting the proposed new development. They will, however, contribute to part of the settlement of the new reclamation works. These materials are quite varied in their thickness and geotechnical parameters and consequently the magnitude of these settlements would need to be examined in additional site investigation works. However, it is anticipated that the likely settlements arising from the consolidation of the clayey bands within the alluvial deposits are likely to be tolerable in the same manner as they have been in areas affected by the existing reclamation. Geotechnical performance parameters for these materials to be adopted for design purposes have been assessed and summarised previously by Greiner Maunsell. Details are included in *Table 2.4.*

Table 2.4 Summary of geotechnical parameters for alluvial marine clays and sands adopted for the design of the existing airport platform

Typical Index Properties and Recommended Design Parameters	Unit	Stiff Alluvial Clays	Firm to Stiff Alluvial Clays	Alluvial Sands
Unit Weight	Mg / m^3	1.90	1.85	2.0
Void Ratio e_0	-	0.9	1.03	0.65
Maximum Past pressure P_p	-	N/A	55+15x Depth below seabed	4.5 + 7x Depth below seabed
Compression Index C_c	-	0.2	0.42	N/A
Recompression Index C_α	-	0.06	0.085	0.03
Coeff of Consolidation C_v	m^2/yr	3.0	2.2	N/A
Coeff of Reconsolidation C_{vr}	m^2/yr	20	15	N/A
Undrained Shear Strength ($N_k = Q_{nett} / S_u$) including 0.8 material loading factor	kN/m^2	N_k =21.25	N_k =17	N/A
Secondary Compression C_α	-	0.3%	1.5%	N/A

After Greiner Maunsell (1991c)

2.4.2.2 Beneath the alluvial sands and clays, more competent materials are exhibited. These have generally not been explored to any great degree in previous site investigation works ie the archived site investigation work in general does not include records of investigations of materials lying below the alluvial deposits in the area of interest. Indeed, the vast majority of the site investigation work undertaken to date has been directed to investigation of the alluvial and soft marine clays at the site and have not penetrated through these to the underlying weathered rock and bedrock.

2.4.2.3 This study is aimed at investigating the feasibility of developing reclamations over the mud pits. Settlements and strength related issues associated with the proposed reclamation will not be significantly influenced by the materials underlying the alluvial deposits which generally only occur at depths below -45 m PD level.

2.4.2.4 *Figure 2.6* indicates the location of all site investigation which has been identified as being directly relevant to the study area from within the archive data. *Figures 2.7 to 2.14* inclusive depict sections annotated with the borehole data and in particular records of the SPT values derived from boreholes which penetrated to levels beneath the soft marine clays. These generally indicate that the decomposed rock lying beneath the study area occurs at about -45 m PD level or in some cases a few metres lower.

2.5 Formation of the Contaminated Mud Pits

2.5.1 *Disposition of Contaminated Mud Pits*

2.5.1.1 The contaminated mud pits have been constructed progressively in response to the demand for disposal space since the early to mid 1990's. Disposal commenced with the excavation of Pit I in early 1993. Subsequently, disposal has been undertaken in each pit sequentially. Once filled, each pit has been capped off with environmental monitoring being undertaken continuously.

2.5.1.2 Shortly after the completion of pit IIa disposal was undertaken under a private arrangement in the CRPC pit. The CRPC pit construction commenced in November 1993 and capping was completed in April 1994. The CRPC pit contains trailer dredged material which is likely to have been deposited with a very low solids content by comparison with the other pits where materials have been constrained to be won by means of grab dredging.

2.5.1.3 Subsequently, filling has progressed and at the present time, filling is still under way in Pit IVc. The Pit IV group of pits at present have not been fully capped off and are awaiting completion of filling.

2.5.1.4 The plan arrangement of the pits is indicated on *Figures 1.1 and 2.2*. Generally, with the exception of pits IV, the pits have been constructed to a depth of between 15 and 20m with excavation being undertaken down to the top of the alluvial deposits at each site. ie the softer materials have been removed and the pits have been curtailed once the excavation work extends into the underlying alluvial deposits.

2.5.1.5 The pits have been constructed with side slopes of about 1:6 but steeper side slopes of up to 1:2 have been reported by Whiteside of CEDD. Experience from maintenance dredging works in the Urmston Road navigation channel for CLP reveals that side slopes of about 1:5 remain stable for periods in excess of 5 years. On this basis it can be assumed that side slopes of 1:6 should be stable at the mud pits site where tidal flow velocities are in fact of lower magnitude than in the main navigation channel. It is however considered unlikely that the side slopes of the pits remain stable at angles steeper than 1:4 for any extended period of time. This assessment has therefore assumed that the pit side walls slump to a notional side angle of 1:6.

2.5.1.6 A summary of the arrangement of the filling pits is detailed in *Table 2.5*.

Table 2.5 Fundamental Mud Pit Parameters

Pit No	Approx Seabed Plan Area Ha	Estimated Design Capacity Million m^3	Deduced base level m PD
I	49	3.3	-20
IIa	21	0.7	-18
IIb	21	1.2	-20
IIc	21	1.5	-20.5
IId	21	1.4	-19
IIIa	24	1.8	-18
IIIb	18.5	1.2	-19
IIIc	3.25	1.5	-16
IIId	2.5	1.3	-16
IVa	13.2	7.2	-35
IVb	n/a	-	-
IVc	n/a	-	-
CRPC	15.3	(1.5-assumed)	-20

The estimated design capacity quoted in *Table 2.5* above has been assessed based upon the plan area of the mud pits and their nominal depth making due allowance for the side slopes at 1:6 and the capping.

2.5.2 *Filling Arrangements*

2.5.2.1 With the exception of the CRPC pit contaminated materials have generally been excavated using closed grab techniques. Some material, although estimated as a small percentage may have been excavated using airlift techniques. The material generally arrives at the mud pits in split hopper barges and is bottom dumped. Smaller projects involving smaller quantities of material can arrive at the dumping grounds in derrick lighters. This material is grabbed from the hold of the vessel and is deposited one grab load at a time.

2.5.2.2 Material arriving from selected parts of the Central and Wanchai reclamation project has arrived in fabricated geotextile bags, each bag containing up to about 250 m^3 of material. The integrity of these bags once deposited within the mud pits is unknown.

2.5.2.3 As noted above material held in the CRPC pit was won by means of trailer dredging techniques. This material would have been reslurrified in the vessel hold in order to discharge it into the pit. The nature of these materials is further discussed below.

2.5.3 *Chronology*

2.5.3.1 The materials held within the mud pits have been progressively placed in the various pits. The placement history and capping details for each pit are detailed below in *Table 2.6.*

Table 2.6 Placement History and Capping Details

Pit Reference	Design Capacity	Received Volume	Pit Excavation	Dumping	Capping	Recapping
I	3.3	3.3	Sep - Dec 92	Dec – Jul 93	Oct 93-Jan 94	N/A
IIa	0.7	0.7	May – Jun 93	Jul – Nov 93	Mar – Jul 94	Jul – Nov 94
IIb	1.2	1.2	Jul – Oct 93	Nov 93 – May 94	May – Jul 94	Jul 94
IIc	1.5	1.6	Oct 93 –Dec 93	May –Feb 95	Mar – Oct 95	N/A
IId	1.4	1.5	Dec 93 – Mar 94	Feb – Sep 95	Sep - Dec 95	N/A
CRPC	*	*	Nov – Dec 93	Jan – Mar 94	Mar – May 94	N/A
IIIa	1.8	1.9	*	Sep 95 – Jul 96	Aug – Nov 96	N/A
IIIb	1.2	1.17	*	Jul – Dec 96	Jan – May 97	N/A
IIIc	1.5	1.45	*	Dec 96 – Mar 97	Mar – Sep 97	N/A
IIId	1.3	1.3	*	Mar – Nov 97	Nov 97 – Mar 98	N/A
IVa	7.2	8.5	*	Nov 97-Mar 00	-	-
IVb	*	15.8	*	Mar 00 – Apr 02	-	-

Pit Reference	Design Capacity	Received Volume	Pit Excavation	Dumping	Capping	Recapping
IVc	*	23.1	*	Apr 02 – Ongoing	-	-

- Data not available in archives

The design capacity quoted in *Table 2.6* above has been assessed based upon the plan area of the mud pits and their nominal depth making due allowance for the side slopes at 1:6 and the capping as in *Table 2.5*. The received volume is that volume recorded by CEDD from records of the delivery of materials. There are clearly discrepancies between the two figures which can be accounted for in terms of bulking, losses and general accounting errors relating to over estimated dumping allowances not having been fully utilized etc.

2.5.3.2 Details of the source of the filling materials have been retrieved from CEDD fill Management Database. Full details of the material sources are presented in *Appendix D*. We observe however, that data being gleaned from the latest SI reveals that the material within the pits can be very variable in nature, even within small depth increments in the pits. We conclude that because of the apparent mixing and re-compaction which has occurred within each pit little can be deduced about where specific material might now reside.

2.5.4 *Capping Arrangements*

2.5.4.1 Capping of the mud pits, with the exception of the much larges pits IV has occurred soon after filling has been completed. The capping comprises a nominal 1m of sand followed by 2m of uncontaminated marine mud.

2.5.4.2 The borehole records indicate that the 1m thick sand blanketing layer was not placed uniformly across the top surface of the filling. The borehole records indicate that in some areas the sand was not present, suggesting that the placement technique was not particularly effective at distributing the sand uniformly across the surface of the underlying filling. Alternatively, the filling top surface of the marine mud might have been irregular leading to sand which is known to have been placed by bottom dumping to be present only in the valley areas of the top surface of the dumped mud.

2.6 Properties of the CMP Clay

2.6.1 Knowledge of the properties of the material in the CMPs is fundamental to the design of land formation methods. In particular, it is important to make an assessment of the current undrained strength profile of the dredged clay in the CMPs, its consolidation characteristics (stiffness and permeability) and its rate of gain of strength during consolidation.

2.6.2 This section presents a review of the properties of the clay in the CMPs based on the properties of the marine clay in Hong Kong prior to dredging, a limited amount of investigation of the capping layers, the properties of similar dredged marine clay in Singapore and the history of the CMPs.

2.6.3 ***Overview of the Pits Relevant to Assessment of Properties***

2.6.3.1 The overall ground conditions prior to the construction of the mudpits was 15 m of lightly overconsolidated (1.2 to 2) marine clay, overlying around 20 m of mixed alluvial sands/gravels and overconsolidated marine clay (OCR 2 to 4). Although slightly outdated, the paper by Fung et al. (1984) (Appendix A) provides a good background to the typical ground conditions at the adjacent airport site prior to construction of the airport and also notes that the properties presented in the paper are generally appropriate for the "upper marine clay" in Hong Kong. The upper marine clay from various sites in Hong Kong forms the majority of the contaminated fill which has been subsequently placed in the CMPs and as such the properties of the clay presented in Fung et al. (1984) are a reasonable starting point to deduce the properties of the material in the CMPs prior to their being dredged and placed in the pits.

2.6.3.2 *Section 2.5* gives an overall description of the construction, operation and capping of the mudpits. Mudpits 1, IIa to IId and IIIa to IIIe have a similar design and operation and these cover the majority of the study area. A summary of the details of these pits are as follows:

- The pits were dredged to the base of the upper marine clay using a grab dredger with side slopes of about 1:6. The pits are typically 15 m deep below the seabed;
- The pits have been backfilled by "contaminated mud" which is predominantly the upper marine clay taken from other sites in Hong Kong excavated by grab

dredgers and transported and placed in the mudpit using hopper barges;

- The contaminated mud initially had a thickness of approximately 12 m with its upper surface 3 to 4 m below the seabed;
- A nominally 1 m thick layer of clean sand was then placed over the contaminated mud by carefully moving a split hopper barge over the surface of the clay; and
- A further 2 m thick layer of dredged, clean upper marine clay was then placed in two layers over the top of the sand. Subsequently, a further 1 or 2 m of dredged, clean upper marine clay has been placed over the pits to make up for any settlement which has occurred.

2.6.3.3 The location of a private pit on the northern boundary of the study area is shown on *Figure 1.1*. This is similar in principle to Pits I to III, the only difference being that the pit has been backfilled by contaminated mud dredged using a trailer suction dredger. The capping clay layer was also obtained and placed by the trailer suction dredger.

2.6.3.4 Pit IVa, which encroaches into the north eastern corner of the study area differs from the other pits in as much as it is significantly deeper, up to 35 m from the seabed, because it was originally dredged to recover the underlying marine and alluvial sand. It is also the last of the pits within the study area to have been backfilled and capped.

2.6.3.5 The remainder of this section concentrates on Pits I to III, which cover the majority of the study area.

2.6.4 *Filling and Capping and Investigation of the Pits*

2.6.4.1 Based on the information that is currently available, *Table 2.7* summarises the dates on which backfilling and capping of each of the pits were carried out. The table also includes the dates of when ground investigations were conducted over the completed CMPs. The ground investigations to date have been limited to vibrocoring of the top 6 m of the pits, with a limited amount of laboratory index testing.

Table 2.7 Summary of Dates of Filling of CMPs

Pit	Start	Finish	Capping Completed	Ground Investigation
I	Nov 92	July 93	Jan 94	July 94 TN 1/95
IIa	Aug 93	Oct 93	Apr 94 – Jun 94	Feb 95 TN 6/97
IIb	Nov 93	Apr 94	May 94 – Jul 94	Feb 95 TN 6/97
IIc	May 94	Jan 95	Feb 95 – Sep 95	Feb 98 GIU28198 VA Series
IId	Feb 95	Sept 95	Sept 95 – Feb 96	Feb 98 GIU28198 VB Series
IIIa	Sept 95	Jul 96	Jul 96 – Nov 96	Feb 98 GIU28198 VC Series
IIIb	Jul 96	Dec 96	Jan 97 – May 97	Feb 98 GIU28198 VD Series
IIIc	Dec 96	Mar 97	Apr 97 – Sept 97	Sept 99 GIU30723 VE Series
IIId	Mar 97	Nov 97		
IIIe	Nov 97	Dec 97		
Private				
IVa	Dec 97[*1]	Mar 2000		September 03 GIU 39203

Note *1 - Yip (2001)

2.6.4.2 At the time of preparing this report not all the filling and capping dates were available and could be ascertained. However, based on the records, the majority of the CMPs in the study area were completed and capped at least 10 years ago and the remainder were completed and capped between 6 and 10 years ago. All the records of additional placement of capping material on the top of the mudpits were not available for this report. However, it is understood that additional capping was placed on Pits IIa and IIb between July 1994 and November 1994.

2.6.5 *Ground Investigations carried out in the CMPs*

<u>July 94 TN 1/95 Ground Investigation of CMP I</u>

2.6.5.1 A summary of the investigation and its findings is as follows:

a) Investigation comprised 5 vibrocores and piston sampling at 2 locations;
b) Visual description of vibrocores – upper capping layer comprised very soft blocks 0.2 to 2 m in size, with torvane shear strengths of 5 to 10 kPa, similar to the original clay strength. Contacts between blocks had softened;
c) The sand capping layer was only identified in 2 of the 5 cores;
d) The material below the capping was characterized by low strengths (<2 kPa); and
e) There were no laboratory index tests carried out on the samples.

February 95 TN 6/97 Ground Investigation of CMP IIa and IIb

2.6.5.2 A summary of the investigation and its findings is as follows:

a) Investigation comprised 10 No. 6 m deep and 10 No. 2 m deep vibrocores in each of CMPIIa and CMPIIb;

b) Laboratory tests included natural moisture content, Atterberg Limits and gradings;

c) *Table 2.8* summarises the laboratory test results;

d) A distinct layer of sand could usually but not always be identified. There was evidence of the sand sinking into the underlying CMP fill at some locations.

e) Using a torvane the undrained strength of the clean capping mud varied from 2 to 10 kPa.

f) Features of the grab dredged CMP material commonly remains as large blocks or lumps.

g) Undrained shear strength (presumably using the torvane) ranges from <1 to 10 kPa.

Table 2.8 Summary of Laboratory Tests on Vibrocore Samples - CMPIIa and IIb

Pit		MC w%	Sat Dens	Dry Dens	LL	PL
IIa Capping	Minimum	72.1	1.25	0.56	64	24
	Maximum	123.5	1.51	0.84	91	36
	St Dev	10.2	0.05	0.06	6	2
	Mean	89.5	1.40	0.74	74	30
IIb Capping	Minimum	74.4	1.37	0.66	64	23
	Maximum	112.2	1.46	0.82	86	33
	St Dev	10.0	0.02	0.04	5	2
	Mean	94.1	1.42	0.73	74	29
IIa CMP	Minimum	62.7	1.30	0.69	60	22
	Maximum	89.4	1.52	0.93	83	31
	St Dev	8.6	0.07	0.07	6	3
	Mean	79.4	1.38	0.77	72	28
IIb CMP	Minimum	64.6	1.23	0.67	64	24
	Maximum	105.7	1.49	0.87	81	33
	St Dev	11.2	0.07	0.07	6	2
	Mean	83.6	1.39	0.76	70	29

2.6.5.3 Although the material within the pits has variable properties (see variation in minimum, maximum and standard deviation), it is worth noting that the mean liquid limit (72%) and plastic limit (29%) is almost constant between the two pits and in the material in the pits. For the purposes of this study, the average liquid limit, plastic limit and Plasticity Index of the capping mud and lumpy clay in the CMPs can be assumed to be 70%, 30% and 40% respectively.

2.6.5.4 The attached *Figure 2.6.1* presents a summary of the average values of natural water content versus depth for each of the two pits (5 test results at each level) and also an average between the two pits. Also shown on *Figure 2.6.1* is a plot of the natural water content versus depth for a hypothetical soil which follow the modified Cam Clay concept and has N=3.6 and Lambda=0.231 for Pits IIa and IIb and N=3.1 and Lambda=0.182 for Pits IIc, IId, IIIa and IIIb. These values are consistent with the concept of the "Omega Point" at e=0.25 at Sig V' = 15 MPa (The Omega point concept is that all critical state e v p' curves pass through this point). The attached *Figure 2.6.2* presents moisture content and density profile for the full 16 m depth of the CMP assuming that the clay is normally consolidated using these parameters. The presence of the sand layer has conservatively been ignored in these plots as many of the vibrocores indicate that the layer is missing.

2.6.5.5 The grading of the material in CMP IIa and IIb and the capping material is summarised in *Table 2.9*.

Table 2.9 Summary of Material Grading of CMP IIa and IIb

	Material Type by %			
	Gravel	**Sand**	**Silt**	**Clay**
Mean	1.3	12.5	42.1	44.0
Standard Dev	1.7	6.2	4.9	4.9

February 98 Investigation of Capping Layer

2.6.5.6 A vibrocore investigation was carried out in February 1998 in CMPIIc, CMPIId, CMPIIIa and CMPIIIb. The investigation was primarily concerned about the capping layers and consisted of 6 m deep vibrocores with associated index testing in the laboratory. At the time of preparing this report only the factual investigation report was available and we have not obtained an interpretative report of this investigation carried out at the time (For pits CMPIIa and IIb both the site investigation contractor's factual investigation report and an interpretative report prepared by the GEO were available). Based on our review of the earlier reports, it is possible that the interpretative investigation would include more detailed logs than provided in the ground investigation contractor's factual report.

2.6.5.7 Our review of the vibrocore logs indicates that an identifiable sand layer at the base of the capping was only encountered in approximately 20% of the vibrocores and very few of the vibrocores give any indication of a sand capping layer having been placed. This aspect needs further investigation to see if a sand capping layer continued to be used.

2.6.5.8 *Tables 2.10* and *2.11* present a summary of the various index test results for all the material in the pits and capping layers combined (i.e. within the top 6 m of the various CMPs). For comparison purposes the equivalent results for Pits IIa and IIb taken from the February 1995 investigation are included in the table.

Table 2.10 Summary of Moisture Content, Density and Atterberg Tests

Pit		Nat Moisture	Sat Density	Dry Density	Liquid Limit	Plastic Limit	Plasticity Index
		%	t/m3	t/m3	%	%	
IIc, IId, IIIa, IIIb, IIIc	Average	74^ 69*	1.60^ 1.62	0.93^ 0.96*	58^ 59*	28^ 30*	30^ 30*
	Min	43^ 51*	1.43^ 1.44*	0.77^ 0.77*	44^ 54*	21^ 25*	22^ 24*
	Max	109^ 96*	1.83^ 1.84*	1.28^ 1.18*	68^ 69*	31^ 59*	40^ 41*
	StDev	14^ 12*	0.08^ 0.10*	0.11^ 0.12*	5^ 4*	2^ 5*	4^ 4*
IIa and IIb	Average	92^ 80*	1.41^ 1.38*	0.74^ 0.77*	74^ 72*	29^ 28*	45^ 44*
	Min	72^ 63*	1.25^ 1.30*	0.56^ 0.69*	64^ 60*	23^ 22*	33^ 29*
	Max	124^ 89*	1.51^ 1.52*	0.84^ 0.93*	91^ 83*	36^ 31*	59^ 52*
	StDev	10^ 9*	0.04^ 0.07*	0.05^ 0.07*	6^ 7*	2^ 3*	5^ 8*

*Note: ^ capping clay *CMP clay*

Table 2.11 Summary of Grading Tests

Pit		Gravel	Sand	Silt	Clay
Pits IIc, IId, IIIa, IIIb, IIIc	Average	2^ 4*	19^ 16*	40^ 42*	39^ 39*
	Min	0^ 0*	3^ 2*	30^ 33*	28^ 25*
	Max	9^ 7*	40^ 33*	52^ 52*	52^ 55*
	StDev	2^ 2*	9^ 8*	5^ 5*	6^ 7*
Pits IIa and IIb	Average	1^ 3*	12^ 14*	42^ 41*	45^ 42*
	Min	0^ 0*	1^ 2*	31^ 31*	35^ 28*
	Max	5^ 9*	28^ 36*	51^ 58*	54^ 51*
	StDev	1^ 3*	5^ 10*	4^ 8*	4^ 8*

*Note: ^ capping clay *CMP clay*

2.6.5.9 *Figure 2.6.1* presents the average moisture content profile for each of the 4 CMPs investigated in the February 1998 investigation (average of 5 vibrocores at each pit) and an average line for all 4 pits. Also shown on *Figure 2.6.1* is a plot of the natural water content versus depth for a hypothetical soil which follows the modified Cam Clay concept and has N=3.1 and Lambda=0.182. *Figure 2.6.3* presents a moisture content and density profile for the full 16 m depth of the CMP assuming that the clay is normally consolidated using these parameters. The presence of the sand layer has conservatively been ignored in these plots as many of the vibrocores indicate that the layer is missing.

2.6.5.10 Based on a review of the laboratory test data, the following points are noted:

a) At the time of the investigation, Pit IIc had been filled and capped for approximately 2 years and Pit IIIb had been filled and capped for approximately 1 year. If the material placed in the two pits were similar it would be reasonable to assume that the clay in Pit IIc would have a lower moisture content (i.e. be more consolidated) than the material in pit IIIb. However, reference to the average moisture content profile in *Figure 2.6.1* indicates the opposite to be the case.

b) The average moisture content of the samples from the 4 pits for capping clay and CMP clay in the February 1998 investigation is 18 and 11% less than that measured for the two pits in the February 1995 investigation respectively. The average density is approximately 13% and 17% greater for capping clay and CMP clay respectively.

c) The average liquid limit and plasticity index for the samples from the 4 pits in the February 1998 investigation is approximately 15% less than the equivalent values for the February 1995 investigation, although the plastic limit is almost the same.

d) The average fines content (silt and clay) of the samples in the February 1998 investigation (80%) is less than the equivalent value for the February 1995 investigation (85%).

2.6.5.11 Based on the above, it appears that the dredged clay used for both the capping and placed towards the top of the contaminated mud in the pits investigated in February 1998 is denser than the equivalent material in the pits investigated in February 1995. Four possible reasons for this are as follows:

a) The elapsed time between filling the pits and carrying out the investigation was greater for the pits investigated in February 1998 and as such more consolidation had taken place.

b) The sand capping material had become heavily mixed with the clay material investigated in the February 1998 investigation during placement of both the sand capping and clay capping.

c) The nature of the marine clay being placed in the pits has changed.

d) The material came from a different source/depth and the difference in densities reflects the natural variability of the marine clay.

2.6.5.12 The actual reason is likely to be a combination of all if the above three effects.

<u>September 2003 Investigation of Pit CMPIVa (GIU 39203)</u>

2.6.5.13 Vane tests were carried out in the top 5 m of pits CMPIVa and CMPIVb in September 2003. At the time of preparing this report the dates of filling and capping these pits had not been determined. The reported seabed level for the 5 test locations in pit CMPIVa varied between -8.29 and -9.13 mPD. The reported seabed level for the 5 tests in pit CMPIVb varied between -10.85 and -10.65 mPD. As Pit CMPIVa was completed before Pit CMPIVb, this suggests that the capping had been placed on Pit CMPIVa but not on Pit CMPIVb.

2.6.5.14 *Figure 2.6.4* presents a summary of peak vane strengths measured in the tests. The results are discussed in *Section 2.6.4*. When considering the results it should be noted that the very low strengths recorded by the vane are at the limit of the accuracy of the test and that the ground is very prone to disturbance during drilling and installation of the vane. The vane size was 130 by 65 mm and there is no information as to whether this was a standard vane or a penetration vane.

GEO Report 18 Mud Anchor Trials

2.6.5.15 Geo Report 18 titled "Backfilled Mud Anchor Trials Feasibility Study" describes the results of a field trial to determine the capacity of ship's anchors when dropped over seabed pits backfilled with dredged clay. The pits used for the tests were located adjacent to CT8 near Tsing Yi Island.

2.6.6 *Geotechnical Design Properties to be used for Preliminary Design Purposes*

Present State of CMP Lumpy Clay

2.6.6.1 This section relates primarily to the state of the mudpits in the study area with the exception of the private pit which encroaches into the northern boundary of the site and which was backfilled with clay obtained using a trailer suction dredger and CMPIVa on which encroaches into the north eastern corner of the study area.

2.6.6.2 The following comments are made with respect to the present state of the CMP lumpy clay fill.

a) The majority of the mudpits were completed and capped more than 10 years ago and the remainder between 6 and 10 years ago.
b) The clay material in the capping and in the body of the pits can be considered to be similar material with respect to the basic engineering properties.
c) The majority of the material placed in the pits is understood to have been obtained by grab dredgers and placed by split hopper barges.
d) The investigations carried out within one to two years of capping already noted that the clay lumps were breaking down and the soft material had begun to reconsolidate. (see *Section 4* of GEO Reports TN 1/95 and TN 6/97)
e) For the purposes of the preliminary design of reclaimed land over the top of the CMPs, it would be reasonable to assume that the clay in the mudpits is

formed predominantly of normally consolidated becoming under-consolidated with depth, destructured marine clay interspersed with the remnants of marine clay lumps.

f) Although a sand layer was placed as part of the capping process, the layer was missing from many of the vibrocores taken through the capping layer and there was no evidence of sand in many of the vibrocores. It would be conservative to ignore the presence of the sand layer for preliminary design purposes.

Typical Consolidation Properties for Magnitude of Settlement

2.6.6.3 *Table 2.12* compares the parameters determined for the normal consolidation lines based on the vibrocore samples with the values determined for the Singapore Lumpy Clay fill determined from an experimental small scale model of lumpy clay (Robinson et al., 2005) and the intrinsic consolidation line (ICL) for Singapore lower marine clay (Karthikeyan et al. 2004). Also shown in *Table 2.6.6* is an estimate for the Lambda (the slope of the normal consolidation line) based on the relationship between Lambda and PI given in Atkinson and Bransby (1978), λ = PI/171.

Table 2.12 Comparison of NCL Lines for HK CMP and Singapore Lumpy Clay

	Singapore LMC ICL	HK CMP Feb 98 GI	HK CMP Feb 95 GI	Singapore Experimental Clay lumps
N	2.95	3.1	3.6	4.0
Cc	0.39	0.42	0.53	0.682
Lambda	0.17	0.182	0.231	0.296
Omega Lambda	0.167	0.182	0.231	0.27
PI relationship		0.175	0.233	

2.6.6.4 An alternative approach to that described above is to establish the Intrinsic Compression Line for the Hong Kong upper marine clay using the method proposed by Burland (1990). Burland (1990) presents a relationship between the intrinsic compression index Cc* and the void ratio at the liquid limit (e_L) expressed as:

$$Cc^* = 0.256\, e_L - 0.04$$

2.6.6.5 *Table 2.13* presents a comparison of the intrinsic compression index based on the liquid limit void ratio and the compression index calculated from the moisture content profiles as set out in *Table 2.12*.

Table 2.13 Intrinsic Compression Line for HK Marine Clay Burland (1990)

Test Series	Liquid Limit %	Void Ratio	Cc*	Cc from Table 2.12	Ratio
HK CMP Feb 98 GI	58	1.54	0.354	0.42	1.19
HK CMP Feb 95 GI	73	1.93	0.454	0.53	1.17

2.6.6.6 It can be seen from *Table 2.13* that the intrinsic compression index based only on the average values of the liquid limit of the Hong Kong marine clay samples is approximately 15% to 20% less than the value determined from measured moisture contents and the concept of the Omega point. The intrinsic compression line would be expected to form a lower bound to the actual test data because the intrinsic compression line assumes that all of the clay is de-structured.

2.6.6.7 In addition to the above, by reference to the results presented in the test reports, it is noted that the natural water content of the samples taken from the CMPs is higher than the liquid limit, which suggests that the clay is either structured or has a small organic content. It is possible that the samples were dried before determining the liquid limit and this drying can result in a significant reduction in the measured liquid limit.

2.6.6.8 The NCL parameters determined for the Hong Kong CMP material from the February 98 investigation are similar to the intrinsic compression line for the Singapore Lower Marine Clay. The NCL parameters for the Hong Kong CMP material from the February 95 investigation, which were determined within 6 months of placing the capping layer, are slightly softer than for the later investigation which was carried out between one to two years after placing the capping layer.

2.6.6.9 In the initial Options Report it was recommended that for the purposes of the preliminary design of reclamations over the HK CMP material the NCL parameters for the February 95 investigation should be adopted as these would provide a slightly more conservative design than the February 98 investigation. Using these parameters *Table 2.14* presents approximate estimates of the magnitude of settlement that would be expected for various fill levels. The calculation is based on an initial seabed level of -6 mPD, makes an allowance for the seabed settlement but not the increase in the density of the marine clay and estimates the additional settlement of the ground below the CMP based on experience of the original airport construction. For the purposes of the calculation the mean sea level is taken to be 1.3 mPD. It is noted that the results obtained more recently from the Contract P398 ground investigation has resulted in a review of the consolidation characteristics of the CMP clay and a re-assessment of the magnitude of the ground settlement, see *Section 2.7* for the revised estimate.

Table 2.14 Preliminary Estimate of Magnitude of Reclamation Settlement

Situation	Fill Level	Increase in Stress kPa	CMP compression m	Additional Settlement m	Total Settlement m
Drainage Blanket	-3 mPD	30	1.0	0.0	1.0
Fill above water	+3 mPD	115	2.0	0.3	2.3
Final Formation	+6.5 mPD	195	2.6	0.7	3.3
Allow for Surcharge	+11 mPD	290	3.0	1.2	4.2

Consolidation Properties for Rate of Settlement

2.6.6.10 The Singapore experimental data on the behaviour of lumpy clay (Robinson et al. (2005)) demonstrated that prior to full degradation of the clay lumps, when the inter lump voids are still a significant factor in the behaviour, the permeability of the overall lumpy clay material is relatively high (e.g. 1×10^{-5} m/s) but as the inter lump voids close the permeability rapidly drops and becomes similar to the values for remoulded clay (e.g. $<1 \times 10^{-9}$ m/s). The Singapore experiments demonstrated that the change in permeability behaviour occurred when the vertical effective stress exceeded 50 to 100 kPa.

2.6.6.11 The majority of the lumpy clay in the Hong Kong CMPs has been in place for over 10 years and it is likely that the majority of the inter lump voids have now disappeared over most of the depth of the clay. It is likely that the overall behaviour of the clay is now controlled by the permeability of the normally consolidated clay which forms the majority of the material.

2.6.6.12 GEO Report No 63 reviews the performance of "drained" reclamations in Hong Kong constructed over the in situ upper marine clay using vertical drains and surcharging. The report includes a summary of the properties of the marine clay, which is included in *Appendix A* of this report. The table indicates a back-figured range for the coefficient of vertical consolidation of 0.3 to 3.2 m^2/year, with the majority of results being in the range 1 to 2 m^2/year and an average value of 1.6 m^2/year.

2.6.6.13 Yip (2001) presents an assessment of the mud consolidation in the Hong Kong CMPs. The analysis is based on the results of oedometer tests carried out on samples of clay recovered from the CMPs. The oedometer tests were carried out by the GEO. At the time of writing this report the factual report on the testing was not available and there is no information available on how the samples were obtained. The paper states that "bag samples of contaminated mud were obtained for the oedometer tests". The reliability of the test results is therefore not certain, although it is likely that the samples were disturbed and results at low stress values are likely to be unreliable and potentially softer than the in situ material.

2.6.6.14 *Figure 2.6.5* summarises the results of the oedometer tests and includes back analysed values of the vertical coefficient of consolidation as a function of the stress increment magnitude. It is of interest to note that the Cv value increases as the vertical stress increases, presumably because the rate of increase in the stiffness of the sample is more significant than the decrease in the permeability of the sample. At relatively low stresses an average Cv value of approximately 0.4 m2/year was determined and this increased to approximately 1.0 m2/year at a vertical stress of 100 kPa.

2.6.6.15 As the clay will be left in place in the CMPs it is almost a certainty that some form of vertical drain will be adopted to accelerate the rate of consolidation. Although the structure of marine clay often results in the coefficient of horizontal consolidation being higher than the vertical value, much of the lumpy clay fill in the CMPs is a de-structured mass and the permeability is more likely to be close to the vertical value.

2.6.6.16 Based on the back analysed values for Cv from drained reclamations and the Cv values presented in Yip (2001) the following range of Cv values were recommended for preliminary design:

a) Lower Bound value 0.5 m^2/year
b) Average value 1.0 m^2/year
c) Upper Bound value 2.0 m^2/year

Based on the results of consolidation tests carried out under Contract P398 which were available at the time of writing the Final Options report it is appears that the clay in the body of the CMPs exhibits lower Cv values than presented in Yip (2001) and a more appropriate range of values would lie in the range 0.5 to 1.0 m2/year, see *Section 2.7*.

Relationship Between Undrained Strength and Consolidation State

2.6.6.17 Fung et al. (1984) presents the results of a series of consolidated undrained triaxial tests carried out on samples of the in situ upper marine clay following the Shanshep procedure of consolidating and swelling back to known OCR values. The results obtained for samples of the upper marine clay are shown in *Figure 2.6.6*. Fung et al. (1984) determined that for normally consolidated samples of the marine clay the relationship between the undrained strength (Su) and vertical effective stress (σv') for varying consolidation states (isotropic or Ko) and loading path directions (compression, extension or simple shear) range between 0.26 and 0.31.

2.6.6.18 Previous investigations of the ratio of the undrained strength of the marine clay to the in situ vertical effective stress have determined a lower bound value of 0.22. In the Initial Options Report a preliminary design a ratio of 0.22 was recommended to take account of the destructured nature of the clay within the CMP.

2.6.6.19 *Figure 2.6.6* presents an assessment of the undrained strength of the clay in the CMP based on the assumption that the clay is normally consolidated and a ratio of 0.25. For reference purposes, the figure also shows the strength gain associated with the placement of a 3 m thick drainage blanket on the surface of the CMP capping, assuming primary consolidation is complete. The placement of a sand blanket on the surface of the CMPs, installation of a vertical drainage system through the CMP clay and allowing for consolidation of the clay is likely to be the first stage of any land formation method carried out over the CMPs.

2.6.6.20 The assumption that the clay profile in the CMP has reached a normally consolidated state may be overly optimistic and requires consideration of the likely rate of consolidation of the dredged material. There are two distinct manners in which the dredged clay placed in the pits could have undergone consolidation. The first assumes that the clay is placed as relatively large lumps in the CMPs and then consolidates in a manner similar to that determined for the lumpy clay in small scale experiments in Singapore. The second assumes that the dredged lumps of Hong Kong marine clay softens and disperses relatively quickly when placed into the CMPs and forms a slurry. The consolidation process would then be controlled by self weight sedimentation and consolidation. The two processes could end up with the clay being in distinctly different states.

Consolidation State following Lumpy Clay Behaviour

2.6.6.21 Robinson et al. (2005) carried out a series of experiments looking at the consolidation behaviour of lumps of clay deposited in water with lump sizes varying from 25 to 100 mm. The clay lumps were placed underwater in a large cylindrical consolidometer to model the behaviour of lumpy clay. Various tests were carried out investigating parameters such as the initial state of packing, the effect of lump size and varying the time allowed for the lumps to swell (or disintegrate) before loading the consolidometer in a similar manner to a standard oedometer test. The results were analysed to determine the variation of void ratio as a function of vertical effective stress and to determine the mass permeability of the lumpy clay at different vertical pressures.

2.6.6.22 The test results indicated, amongst other things, the following:

a) The mass permeability of the lumpy clay changed significantly as the vertical stress was increased and this effect was apparent even for samples which were allowed to fully swell prior to the commencement of loading, see *Figure 2.6.7*.
b) The mass permeability at vertical effective stress less than approximately 30 kPa was 1E-5 m/s, decreasing to 1E-7 m/s at an effective stress of 50 kPa and 1E-9 m/s at approximately 100 kPa. This very significant change in the mass permeability was assumed to be associated with the inter-lump voids being closed.
c) At stresses above 100 kPa the permeability was similar to a normal marine clay and continued to decrease as the vertical stress increased, albeit more slowly.
d) This variation of permeability was independent of lump size, initial packing and time allowed for swelling.

2.6.6.23 In order to investigate the influence of the change in permeability on the consolidation of the full thickness of clay in the CMPs, an assessment has been made of the time required for a 16 m thick layer of lumpy clay to dissipate 95% of its excess pore pressure. The calculation has been carried out for various permeability values assuming that the clay has a Cc value of 0.53 (i.e. the design value) and a void ratio using N = 3.6 at a vertical effective stress of 1 kPa. *Figure 2.6.7* presents the time required for 95% consolidation for both single (i.e. dissipating only through the upper surface) and double drainage (i.e. dissipating at the top and bottom assuming the CMP to be underlain by an extensive sand layer).

2.6.6.24 *Table 2.15* summarises the time at various permeability values and the effective stress associated with each permeability value.

Table 2.15 Time for 95% Consolidation of a 16 m Thick Clay Layer (N=3.6, λ=0.23)

Permeability	**Effective Stress**	**Time for 95% (years) Single Drainage**	**Time for 95% (years) Double Drainage**
$1x10^{-5}$	<30 kPa	0.02	0.005
$1x10^{-7}$	50 kPa	2	0.5
$1x10^{-8}$	70 kPa	20	5
$1x10^{-9}$	100 kPa	200	50

2.6.6.25 The CMPs have been filled for approximately 10 years. Based on the results in *Table 2.15* it would be reasonable to assume that if the clay in the CMPs behaves in a similar manner to the clay in the Singapore model tests then there would be sufficient time for clay with a permeability of approximately 2E-8 m/s to have dissipated excess pore pressures and that this would be associated with a vertical effective stress of approximately 60 kPa. A simplistic and potentially conservative conceptual model for a 16 m thickness of lumpy clay after 10 years of consolidation would be to assume that it is normally consolidated to a depth where the in situ effective stress is approximately 50 kPa and that below this level the effective stress remains approximately constant.

Consolidation State Following Self Weight Consolidation Behaviour

2.6.6.26 An alternative mechanism to that represented by lumpy clay behaviour would be self weight consolidation of a slurry. This mechanism would assume that the rate of filling of the mudpits is sufficiently slow that the dredged clay lumps are fully softened prior to being covered by the next load of dredged clay. Mak (2005) includes test results which look at the rate of dispersion (i.e. softening and total degradation) of small lumps of Hong Kong marine clay when it is dropped through water. The tests indicated that small lumps of clay (<200mm) would completely disperse if they were to fall through a 15 m depth of water. Although dispersion when falling through water and softening in situ are not identical mechanisms the tests indicate that the clay has the potential to soften and disperse relatively rapidly.

In order to determine the rate at which a fully dispersed slurry would consolidate it is necessary to determine the in situ permeability of the clay at different stress levels. Yip (2001) presents the results of oedometer tests carried out on samples of Hong Kong marine clay obtained from the CMPs. The results are summarised in *Figure 2.6.5* in terms of both coefficient of volume compressibility (mv) and coefficient of consolidation (Cv) against vertical stress level. *Figure 2.6.8* presents the same information with equivalent mv values determined assuming the proposed CMP clay design consolidation parameters (N=3.6 and λ = 0.231). It can be seen that the proposed stiffness parameters plot in the middle of the range of oedometer test results. *Figure 2.6.8* also presents an average value for Cv for the test results presented in Yip (2001) as a function of vertical effective stress.

2.6.6.27 The permeability (k) of the dredged clay can be determined from the relationship:

$$k = m_v.C_v.\gamma_w$$

(γ_w is the unit weight of water, say 10 kN/m^3)

2.6.6.28 It is normally expected that the permeability of a clay decreases as the effective stress increases and void ratio decreases. Based on the design lines given in *Figure 2.6.8* the permeability of the marine clay when consolidated under self weight from a slurry is shown in *Table 2.16*.

Table 2.16 Variation of Permeability Under Self Weight Consolidation

Vertical Effective Stress	Void Ratio from N=3.6 λ = 0.231	Mv based on Design Line	Average Cv from Yip 2000 - Fig 8	Permeability	Time for 95% Consol	Ratio $\Delta e/\Delta \log(k)$
kPa	E	m^2/MN	m^2/year	m/s	Years	
5	2.228	15	0.4	1.90E-9	25	
20	1.907	4	0.5	6.30E-10	80	0.67
130	1.475	0.7	1.0	2.26E-10	200	0.97
5	2.228	30	0.4	3.8E-9	13	

2.6.6.29 The dredged clay has been in place for typically between 10 and 15 years (i.e. approximately 5000 days). *Figure 2.6.7* shows the time required to achieve 95% consolidation for a 16 m thick clay layer with either single or double drainage. For single drainage the minimum permeability required to achieve 95% consolidation within 5000 days would be approximately 1.2×10^{-8} m/s and for double drainage the equivalent permeability would be 4×10^{-9} m/s. Without carrying out detailed calculations it can be seen from the relationship between vertical effective stress and permeability in *Table 2.16* that if self weight consolidation were the dominant consolidation mechanism, the typical effective stress could be of the order of 5 kPa but would be unlikely to exceed approximately 25 kPa at any point in the profile if the consolidation was occurring as a slurry through self weight consolidation without the more rapid consolidation associated with lumpy clay behaviour.

Undrained Strength Profile

2.6.6.30 The current undrained strength of the clay in the CMPs will be controlled by the current consolidation state of the clay. In the Initial Options Report it was recommended that the ratio between undrained strength and vertical effective stress be conservatively assumed to be 0.22, and the report noted that the results of laboratory tests to be carried out on the clay may result in this being increased to approximately 0.25. As described in *Section 2.7*, the ground investigation data obtained for the Final Options Report indicates that the ratio of 0.25 is appropriate and 0.22 is unnecessarily conservative.

2.6.6.31 Based on the two postulated consolidation mechanisms, the undrained strength profiles shown in *Figure 2.6.9* were recommended in the Initial Options Report for preliminary design purposes. The upper bound line assumed that the full thickness of the clay is in a normally consolidated state and uses a ratio of 0.25. The design line assumes that the clay is consolidated to a depth where the vertical effective stress is 50 kPa and that the vertical effective stress remains constant below this level. The Initial Options Report recommended that the ratio of 0.22 be adopted for the design line but as noted above, this can be increased to 0.25 on the basis of more recent ground investigation data presented on *Section 2.7*. The lower bound line presented in the Initial Options Report, based on the assumption of self weight consolidation, assumed that the clay was consolidated to a vertical effective stress of 22 kPa (i.e. undrained strength of 5 kPa) and that the vertical effective stress remains constant below this level. The ground investigation results presented in *Section 2.7* have demonstrated this to be overly conservative.

2.6.6.32 The three profiles have been included on the Field Vane Shear Strength profiles determined for the mudpits used in the anchor trials reported in GEO Report 18 in *Figure 2.6.10*. It is noted that the dredged clay placed in the pits for the anchor trials was dredged and placed using suction hopper methods and had been in place for less than one year when the vane tests were carried out. It would be expected that the strength would be greater had the clay been in place for 10 years. The following points are noted:

a) The theoretically derived design profile determined for the CMPs is close to an upper bound to the data in Geo Report 18.
b) There is no reason to suppose that the strength would be significantly greater than the upper bound profile determined for the CMPs.
c) The lower bound profile is consistent with the strength measured in the anchor trials at boreholes 3 and 5. The data at BH12 is anomalous.

<u>Secondary Compression</u>

2.6.6.34 The work carried out on lumpy clay fill in Singapore has concluded that the ratio $C\alpha/Cc$ falls in the range 0.03 to 0.05 as found by Mesri and Godlewski (1977) for normally consolidated clays and this ratio can be used with the results of the current ground investigation to make estimates of the likely secondary compression. As discussed in *Section 2.7*, Cc values in the range 0.6 to 1.2 are recommended and as such $C\alpha$ values of the order of 0.025 to 0.05 per log cycle of time for normally consolidated clay are likely.

2.7 Ground Investigation for P131 Land Formation Consultancy

2.7.1 *Overview of the Ground Investigation*

2.7.1.1 At the time the initial options report was prepared there was no specific ground investigation data available for the P131 Land Formation Consultancy. A ground investigation commenced on site on 17 October 2008 specifically to investigate the current state of the clay in the mudpits and was still ongoing when the Final Options Report was prepared. At the time of preparing the Final Options Report the following additional ground investigation data was available:

a) Four boreholes advanced using piston sampling methods to a depth of 35 m below the seabed.

b) Four in situ vane test profiles using the penetration vane, carried out to approximately 20 m below seabed level.

c) Nine CPTs carried out from a seabed testing rig to depths of approximately 30 m below seabed level. The seabed rig weighs approximately 20 tonnes and is supported on the seabed on a 3 m by 3 m steel footing surrounded by a 0.5 m deep steel skirt. The seabed rig settles into the soft slurry at the seabed and as such affects both the initial seabed level and the state of the clay near the seabed. The CPT results at shallow depth need to be corrected to take account of the load from the seabed rig

d) Two shallow depth CPTs through the upper 3 m of the material at the seabed, referred to as slurry CPTs. These CPTs were supported from a steel casing pushed into the seabed and penetrated into the seabed at a controlled rate using the self weight of the rods, allowing the soft slurry at the seabed surface to be profiled without being affected by the weight of the seabed rig.

e) One T Bar profile carried out from the CPT seabed testing rig.

f) Two shallow depth T Bar profiles through the upper 3 m of the material at the seabed, referred to as slurry T Bar profiles.

2.7.1.2 The locations of the various tests are shown in *Figure 2.7.1*.

2.7.1.3 Laboratory tests have been carried out on the samples recovered from the boreholes. At the time of preparing the final options report the following laboratory test results were available:

a) 16 No. Index tests comprising natural water content, Plastic Limit, Liquid Limit, Plasticity Index and grading.

b) 18 No. Unconsolidated Undrained triaxial tests

c) 9 No. Consolidated Undrained triaxial tests. The samples were isotropically consolidated to a cell pressure equivalent to the estimated in situ vertical effective stress (At the time of writing the Final Options Report no anisotropically consolidated CU tests had been carried out)

d) 8 No. Oedometer tests

e) 2 No. Rowe cell tests with vertical drainage (At the time of writing the final options report no Rowe cell tests with radial drainage had been carried out).

f) 1 No. intrinsic consolidation test to measure the consolidation properties of the CMP clay after it has been de-structured.

2.7.1.4 At the time of writing the Final Options report the factual ground investigation report had not been prepared and only preliminary logs and test results were available. The Preliminary borehole logs and CPT profiles available for this report are included in *Appendix E***.** A review of the preliminary results of the laboratory testing, the vane test results and an interpretation of the CPT and T bar profiles is provided in this section. The discussion in this section concentrates on the properties of the material in the CMP.

2.7.2 *Index Test Results*

2.7.2.1 At the time of writing the Final Options Report index test results comprising Atterberg Limit Tests (Plastic and Liquid Limit) and grading results were only available for samples from Boreholes 1-2 and 2c-1 (i.e. CMP I and CMP IIc). *Figures 2.7.2 (a)* presents a summary of the Atterberg limit test results with the natural water content of the samples. *Figure 2.7.2 (b)* present a summary of the grading test results.

2.7.2.2 It can be seen from *Figure 2.7.2 (a)* that the Atterberg limits are relatively variable with depth, with the Plastic Limit typically in the range 25 to 35% (Average value 29%), Liquid Limit typically in the range 55 to 75% (Average value 61%) and PI typically in the range 25 to 40 with an average value of 32. These results are comparable with the range and average values determined for Pits IIc to IIIc in *Table 2.10*.

2.7.2.3 It can be seen from *Figure 2.7.2 (b)* that the clay content of the samples is typically in the range 25 to 50% with an average value of 40%, silt content typically in the range 35 to 50% (average value 43%), sand content typically in the range 5 to 25% (average value 15%) and gravel content typically in the range 0 to 5% (average value 2%). These results are comparable with the range and average values determined from the previous vibrocore investigations.

2.7.3 *Water Content, Bulk Density and Void Ratio*

2.7.3.1 The in situ bulk density and void ratio can be determined from the initial state of the samples in the oedometer, Rowe Cell and triaxial tests. *Figure 2.7.3* presents plots of the variation of bulk density and void ratio with depth below the seabed. As a result of the large variation both within and between pits all the data has been included on one plot. A review of the results for individual pits has been carried out and no particular trends can be identified for individual pits.

2.7.3.2 Because of the variability of the material in the pits it is not possible to determine a best fit line to the data. As would be expected, there is a trend for the bulk density to increase with depth and the void ratio to decrease with depth. For comparison purposes the average normally consolidation density and void ratio profiles determined from the previous investigations has been included on the plots.

2.7.3.3 The variation of water content with depth for all laboratory tests (oedometer, Rowe cell, triaxial and index test result) is presented in *Figure 2.7.2 (b)*. As above, because of the variability of the material it is not possible to put a best fit line on the data. Nevertheless, as would be expected there is a trend for the water content to decrease with depth. For comparison purposes the average normally consolidated water content profiles determined from the previous investigations has been included on the plots.

2.7.4 *Oedometer and Rowe Cell and Intrinsic Consolidation Test Results*

2.7.4.1 The oedometer, Rowe cell and intrinsic consolidation tests are all similar tests which measure the drained consolidation characteristics of the clay samples. The tests are used to determine the following parameters:

a) The compression index Cc which is the slope of the normal consolidation line in void ratio (e) versus $\log_{10}$ vertical effective stress (σ_v') space. (This is equivalent to 2.303 times the slope of the normal consolidation line (λ) in e v ln (σ_v') space). Along with the pre-consolidation pressure this is used to determine the magnitude of the primary consolidation settlement which is likely to occur under the weight of the reclamation.

b) The pre-consolidation pressure, which for the purposes of this report is the vertical effective stress at which a test sample becomes normally consolidated. By comparing the pre-consolidation pressure with the estimated in situ vertical effective stress prior to sampling it is possible to determine whether the sample is still undergoing consolidation in situ, is normally consolidated or is over-consolidated.

c) The coefficient of consolidation (Cv) which is a measure of how quickly the clay will consolidate under load and is related to both the stiffness and permeability of the material.

2.7.5 *The Compression Index*

2.7.5.1 *Table 2.7.1* summarises the results of the various consolidation tests with respect to the Compression Index and pre-consolidation pressure. The method adopted to determine these values is described in the following paragraphs.

Table 2.7.1 Summary of the results of the various consolidation tests

				"κ - λ" method		"log – log" method	
Borehole No.	Specimen Depth	Material Type	Cc	Pc (kPa)	OCR	Pc (kPa)	OCR
1-2*	3.9 m – 4.0 m	CMP Clay	0.81	32.1	1.25	27.5	1.07
1-2*	4.7 m – 4.8 m	CMP Clay	0.81	32.1	1.04	29.5	0.96
1-2*	7.9 m – 8.0 m	CMP Clay	0.30	43.4	0.84	32.1	0.62
1-2*	15.3 m – 15.4 m	CMP Clay	0.33	73.6	0.74	56.4	0.57
1-2*	18.9 m – 19.0 m	Mar. Clay	0.84	77.9	0.63	62.5	0.51
2c-1*	7.9 m – 8.0 m	CMP Clay	0.52	42.4	0.82	34.9	0.67
2c-1*	11.8 m – 11.9 m	CMP Clay	0.78	65.8	0.85	50.2	0.65
2c-1*	17.4 m – 17.5 m	CMP Clay	0.17	94.1	0.83	81.0	0.71
1-2^	5.9 m – 6.0 m	CMP Clay	0.48	31.5	0.81	56.7	1.47
1-2^	1.7 m – 1.85 m	CMP Clay	1.34	N.A.	N.A.	24.2	2.10

*Note: * Oedometer tests ^ Rowe cell tests*

2.7.5.2 *Figure 2.7.4* shows a typical oedometer test result plotted in void ratio v. log10 vertical effective stress space. The compression index is the slope of the normal consolidation line and is defined on the figure. *Figure 2.7.5* presents a summary of all the values for compression index determined from the various consolidation tests. It can be seen from the results that there is no particular trend with depth, borehole or test type.

2.7.5.3 *There* is presently insufficient data available to carry out a rigorous statistical analysis of the Compression Index results (i.e. too many variables and insufficient tests) and a pragmatic approach to reviewing the data must be adopted in order to determine values for preliminary design purposes. The following values are recommended:

a) At the time of preparing this report only one test result was available for material within 3 m of the seabed surface, which is likely to be the capping material. The one test gave a Cc value of 1.3, which is significantly higher than the other test results. Further tests have been scheduled on the upper capping material. For the purposes of the Final Options report it is recommended that a value of 1.3 be adopted for the upper 3 m of the CMP (i.e. the average capping layer thickness) for upper bound estimates of settlement and a value of 1.0 be adopted for best estimate purposes. The value of 1.0 is at the upper end of the range of values presented in Table 1 of GEO Report No. 63, which summarises the properties of marine clay from other sites in Hong Kong.

b) For the CMP material below 3 m depth, ignoring the one anomalously low result from Borehole 2c-1 at approximately 17 m depth, the average Cc value is approximately 0.6 and a reasonable value for upper bound estimates of settlement is 0.8.

2.7.5.4 The values determined from the ongoing ground investigation are larger than those determined for the initial options report and should be adopted for preliminary settlement calculations.

2.7.5.5 The ground investigation carried out in the CMPs to date has shown that the material in the CMP is extremely variable. Comparison of the cone load (qt) profiles for CPT 1-2, CPT 3b-1a, CPT 2b-1 and CPT 2d-2 gives a clear indication of the variability of the CMP material as described in the following paragraphs:

- CPT 1-2 shows relatively low cone loads from -6 to -26 mPD with evidence of the sand capping layer between approximately -9 to -10 mPD and the occasional thin silty or sandy seam below this level. There is approximately 18 m of normally consolidated clay at this location.

- CPT 3b-1a also shows relatively low cone loads but only from -6 to -20 mPD. The sand capping layer is missing although there is evidence of a sand layer from approximately -15 to -16 mPD. There is approximately 13 m of normally consolidated clay at this location.

- CPT 2b-1 encountered normally consolidated clay from approximately -6 to -13 m PD and from -19 to -24 m PD. However from approximately -13 to -19 m PD the majority of the ground comprised very stiff silt or medium dense sand. There is approximately 12 m of normally consolidated clay at this location.

- CPT 2d-2 encountered soft clay from approximately -6 to -14 m PD with evidence of the sand capping from approximately -11 to -12 m PD. Below approximately -14 m PD a very stiff clay layer was encountered. There is only approximately 7 m thickness of normally consolidated clay at this location.

2.7.5.6 It can be seen from the above that both the thickness of the soft normally consolidated clay and the presence and thickness of sand layers is variable and this will have a significant influence on the predicted magnitude of ground settlement.

2.7.5.7 Calculations have been carried out to assess the likely range of the reclamation settlement over the CMPs using both the best estimate and upper bound compression index values for the clay capping and the CMP clay in the body of the CMPs.

2.7.5.8 For the profile encountered at CPT 1-2, which is considered to be representative of a location where the largest settlement would be expected, the best estimate of the compression of the CMP clay is of the order of 4.5 m with an upper bound estimate of the order of 6 m.

2.7.5.9 For the profile encountered at CPT 3b-1a, which is considered to be representative of a location where average settlement would be expected, the best estimate of the compression of the CMP clay is of the order of 3.5 m and the upper bound estimate of the order of 4.5 m.

2.7.5.10 In addition to the compression of the CMP clay, there will be a contribution to the total reclamation settlement from the ground underlying the CMP and also creep compression of the reclamation fill materials. Based on experience during construction of the existing airport, where the soft marine clay was dredged prior to filling, an additional settlement of the order of 1 m should be allowed for preliminary design purposes.

2.7.6 *Pre-Consolidation Pressure*

2.7.6.1 The pre-consolidation pressure represents maximum previous vertical effective stress that a sample has experienced or alternatively is the vertical effective stress above which the incremental settlement magnitude rapidly increases. The pre-consolidation pressure for each consolidation test has been determined using both the "log-log" method and the "κ – λ" method as illustrated in *Figure 2.7.6*. *Table 2.7.1* summarises the pre-consolidation pressure for each test.

2.7.6.2 The approximate normally consolidated status of each sample has been determined by assuming that the CMP clay has a density of 16.5 kN/m^3. *Figure 2.7.7* presents a summary of the OCR versus depth for all one dimensional consolidation tests. It can be seen from *Figure 2.7.7* that above a depth of approximately 12 m below the existing seabed the CMP clay is close to being normally consolidated. Below this depth the results indicate that the CMP clay may still be undergoing primary consolidation.

2.7.7 *Coefficient of Consolidation*

2.7.7.1 The coefficient of consolidation is a measure of the rate at which primary consolidation occurs and is an important parameter for determining, for example, vertical drain spacing or time required for surcharging. A high coefficient of consolidation is associated with fast rates of consolidation. The coefficient of consolidation is a function of both the permeability and stiffness of a sample, neither of which is constant even during a single loading stage. Although the coefficient of consolidation is often considered to be a constant value, in reality it varies both between load steps and during individual load steps.

2.7.7.2 A number of methods are available for estimating the coefficient of consolidation from oedometer/Rowe cell test data. The most commonly adopted methods are referred to as the "square root time" and the "log time" methods, both of which fit mathematical functions to the test data. The square root time method is based on the time to achieve 90% consolidation and the log time method is based on the time to achieve 50% consolidation.

2.7.7.3 *Figure 2.7.8* presents a plot of the coefficient of consolidation determined for each loading stage of an oedometer test using both the square root method and the log time method. It can be seen that for this particular test the values obtained using the log time method are approximately half of those obtained using the square root time method. Cv values for the log time method are useful for considering the time required for the early stages of consolidation, for example when considering strength gain during a stage loading process. Cv values for the square root time method are useful when considering the latter stages of consolidation, for example when estimating the time required to achieve 90% consolidation beneath a surcharge.

2.7.7.4 *Figure 2.7.9* presents a summary of the Cv values determined using the square root time method only for loading stages above the pre-consolidation pressure. *Figure 2.7.10* presents a summary of the Cv values determined using the log time method for only for loading stages above the pre-consolidation pressure.

2.7.7.5 Also included on *Figures 2.7.11* and *2.7.12* are the results obtained from an intrinsic consolidation test carried out on a sample of clay from Borehole 1-2 at a depth of 4 to 5 m below the seabed. Prior to the test the clay from this piston sample was thoroughly mixed and its water content increased to approximately 1.5 times its liquid limit. The aim of the mixing was to destroy any structure in the sample.

2.7.7.6 The following points are noted:

a) Comparison of *Figures 2.7.9* and *2.7.10* with *Figure 2.6.8,* which shows the oedometer test results presented in Yip (2001), indicates that the Cv values determined from the recent testing are generally lower than estimated for the Initial Options report based on the data presented in Yip (2001). This means that the rate of consolidation will be lower, requiring both closer drain spacing and/or longer times required to achieve the necessary degree of consolidation.

b) Laboratory consolidation test results are often found to under-estimate the coefficient of consolidation in the field. This is usually assumed to be because the clay in the field is structured. An alternative method of determining the coefficient of consolidation using CPT dissipation tests has been included in the P398 CMP ground investigation but the results or the dissipation tests were not available at the time of writing this report.

c) Rowe cell tests with radial drainage are being carried out to investigate whether there is any significant difference between the coefficient of consolidation in the vertical and horizontal (i.e. radial) directions. At the time of writing this report the results of the Rowe cell tests with radial drainage were not available.

2.7.7.7 Based on the results of the testing to date a coefficient of consolidation for preliminary design purposes in the range 0.5 to 1.0 m^2/year is recommended when assessing parameters such as drain spacing, surcharge heights and time for consolidation. This value needs to be reviewed when all the consolidation and piezocone dissipation tests are available. Note added after preparation of the Final Options report - The lower bound value of 0.5 m^2/year for the CMP clay has been based on a preliminary review of all the consolidation tests carried out on samples of the CMP clay. One out of the twenty consolidation on the CMP clay tests gave a value consistently lower than 0.5 m^2/year. The Rowe cell test on a sample from BH1-2 at a depth of 5 to 6 m indicated a value of approximately 0.25 m^2/year. In addition, all the piezocone dissipation tests gave values higher than 0.5 m^2/year.

2.7.8 *Secondary Compression*

2.7.8.1 After the practical completion of primary consolidation the clay in the CMPs will continue to compress as a result of a process referred to as secondary compression. The secondary compression of clay is characterised by a parameter $C\alpha$ which refers to the change in void ratio per log cycle of time. For example, if the primary consolidation of a clay layer is practically complete after one year then $C\alpha$ is the ongoing decrease in the void ratio after a further 9 years (i.e. at 10 years).

2.7.8.2 As noted in *Section 2.6*, the work carried out on lumpy clay fill in Singapore has concluded that the ratio $C\alpha/Cc$ falls in the range 0.03 to 0.05 as found by Mesri and Godlewski (1977) for normally consolidated clays. This ratio can be used with the results of the current ground investigation to make estimates of the likely secondary compression. In order to estimate the settlement of a clay layer due to secondary compression it is easier to use the parameter $C\alpha\varepsilon$, which is defined as $C\alpha/(1+e)$, where e is the void ratio of the clay. The parameter $C\alpha\varepsilon$ can be used to calculate the strain in the clay layer per log cycle of time.

2.7.8.3 Based on the results of the consolidation testing carried out to date it is recommended that $C\alpha\varepsilon$ for the clay in the CMP should be assumed to be 1% (range

0.8% to 1.2%) and for the 3 m of clay in the capping should be assumed to be 1.5% (only one test result).

2.7.8.4 At the time of preparing the Final Options report preliminary results for the first intrinsic consolidation test were available. The test was carried out on a reconstituted sample of clay taken from a depth of 4 to 5 m below the seabed at Borehole 1-2. Each loading stage of the intrinsic consolidation test was maintained for sufficient time for the end of primary consolidation to be achieved and a measure of the secondary compression to be determined. The $C\alpha\varepsilon$ value for each loading stage from 20 kPa to 320 kPa fell in the range 0.95% to 1.08%, with an average value of approximately 1%. This result is consistent with the estimates made using based on the Cc values in oedometer and Rowe cell tests.

2.7.9 *Undrained Strength Profile*

2.7.9.1 The in situ undrained strength profile is a critical design parameter as this determines the maximum thickness and maximum slope of the leading edge of the initial fill placement on top of the clay in the mudpits and also has an influence on the feasibility of constructing sand compaction piles. The undrained strength profile has been determined using in situ vane tests, the CPT, the T Bar and laboratory unconsolidated undrained triaxial tests. In the following sections the strength profiles determined from the P398 ground investigation are presented with the design strength profile and upper bound strength profile from *Figure 2.6.9*.

Vane Tests

2.7.9.2 *Figure 2.7.13* presents the peak vane strength test results with depth. The test results have been corrected using the method proposed by Aas et al. 1986 for normally consolidated clay. This correction factor varies as a function of the ratio of vane strength to vertical effective stress and was adopted rather than using the Bjerrum correction factor based on Plasticity Index. The Aas method was selected because the PI is very variable and we do not have values associated with each of the vane test locations. It is noted that the average correction factor using the Aas method was approximately 0.8, which is similar to the Bjerrum correction factor for a PI of 45. Using the average PI determined from this investigation of 32, the Bjerrum factor would be 0.88, which would indicate strengths approximately 10% higher than obtained using the Aas method.

Laboratory Triaxial Tests

2.7.9.3 *Figure 2.7.14* presents a summary of the peak undrained strength obtained from the laboratory unconsolidated undrained triaxial tests. *Figure 2.7.15* presents the same data with the corrected vane test results. With the exception of the results of tests on samples from Borehole 4a-2, the undrained strength measured in the triaxial tests is similar to that measured in situ with the vane. The reason for the triaxial test results from Borehole 4a-2 (i.e. CMP IVa) being lower than from other boreholes is not known, although it is of interest to note that the shear vane profile obtained at this CMP is also lower than obtained at the other locations. CMP IVa is the most recently filled CMP, which partially explains the relatively lower strength. At the time of preparing the Final Options report the CPT profile at location 4a-2 was not available.

CPT Tests

2.7.9.4 The undrained strength profile can also be determined from the results of the CPT. The method adopted for this report is as follows:

$Su = (qt - \sigma_{vo})/N_{kt}$

Where qt is the cone load corrected for the pore pressure effect

σ_{vo} is the total vertical stress

N_{kt} is a cone factor

2.7.9.5 The majority of the CPT tests were carried out using a seabed testing rig which weighs approximately 20 tonnes and is founded on a steel bearing plate with an area of approximately 9 m^2. When calculating σ_{vo} an allowance was made for the weight of the CPT testing rig, assuming a Boussinesq stress distribution beneath the rig. A small number of shallow depth CPT (and T Bar) tests were carried out without the seabed rig. These tests, which are referred to as slurry CPTs or slurry T Bars, were supported from a steel casing driven into the clay and used the weight of the CPT rods to push the CPT into the clay. These results do not need correcting for the weight of the seabed rig.

2.7.9.6 Although the P398 ground investigation included 4 locations where vane profiles and CPT profiles were to be obtained adjacent to each other, at the time of preparing the Final Options report the only comparable full depth results had been obtained at GI location 1-2 and shallow depth results from the slurry CPT at GI location 2c-1. *Figure 2.7.16* presents a comparison of the corrected peak vane test and CPT derived undrained strength using a N_{kt} cone factor of 15, which appears to give a reasonable site specific correlation and is in line with the value determined on other sites.

2.7.9.7 *Figure 2.7.17* presents a summary of the undrained strength profiles obtained from all the CPTs available at the time of preparing the Final Options report.

T Bar Tests

2.7.9.8 At the time of preparing the Final Options report only one T Bar profile and two slurry T Bar profiles were available. The undrained strength has been estimated from the T Bar by applying a "Nkt" factor of 10.5 as recommended by Stewart and Randolf (1994) directly to the recorded T Bar load. (There is no requirement to correct for the in situ vertical stress when analysing T Bar test results). The results are presented on *Figure 2.7.18* along with the CPT test results. Based on this initial interpretation, the T Bar is apparently indicating higher undrained strengths than the CPT or vane. Further correlation data is required before any conclusions are reached from this test method.

2.7.9.9 *Figure 2.7.19* presents all the various assessments of the undrained strength profile on one plot. Based on the results of the available ground investigation data it is apparent that the best estimate profile determined in *Section 2.6 i*s a reasonable lower bound to the test data and that the upper bound strength profile determined in *Section 2.6* would actually be a reasonable design profile to be adopted for preliminary design purposes.

2.7.10 *Summary*

2.7.10.1 At the time of preparing the Initial Options Report no specific ground investigation data was available with respect to the current state of the clay in the CMPs. Accordingly an assessment was carried out based on a limited amount of test data obtained in the upper 6 m of the CMPs relatively soon after constructing the capping layers, the findings of work carried out on dredged clay fill (lumpy clay) in Singapore and from basic principles of soil behaviour. This assessment has now been supplemented by the results of a ground investigation carried out at the CMPs, which was still ongoing at the time of preparing the Final Options Report.

2.7.10.2 For preliminary design purposes the following points are noted:

a) The material in the CMPs is variable. Although the majority of the material is dredged marine clay, significant thicknesses of sand were encountered at some locations, for example see CPT profile 2d-1 where a 10 m thick sand layer was encountered from 4 to 14 m below the seabed level. A 1 m thick layer of angular coarse gravel sized fragments was identified within the dredged clay at Borehole 4a-2 at a depth of 10 m below the seabed.

b) Only a relatively limited ground investigation has been carried out to date. The ground investigation is sufficient for preliminary design purposes but is not sufficient for detailed design. For example, it would be a mistake to assume that a single CPT profile, for example at CMP IIIc, could be representative of all the material in a CMP.

c) Although a sand capping layer could be identified at most locations, it is not always present, see for example the CPT profile at 3b-1a.

d) The consolidation tests and the strength test results indicate that it is reasonable to assume that the clay in the CMPs is normally consolidated, although at some locations below a depth of approximately 12 m below the seabed the clay is still consolidating.

e) At the time of preparing the final options report only one consolidation test result was available from the upper three m of the CMPs, from within the clay capping layer. This one test indicates a compression index of 1.3, which is significantly higher than all the test results below this level. Further consolidation tests are being carried out on the capping material to confirm whether this is representative of the clay in the capping layer. For preliminary design purposes a compression index of 1.0 is recommended for the upper 3 m of the material in the CMP with a value of 1.3 for upper bound predictions.

f) The compression index of the clay within the main body of the CMP is variable. For preliminary design purposes it would be reasonable to assume an average value of 0.6 and an upper bound value of 0.8.

g) The coefficient of consolidation is variable. Based on the information presently available it would be difficult to justify a value higher than 0.5 m^2/year for preliminary design. Further consolidation tests and CPT dissipation tests are currently being carried out and the preliminary design value needs to be reviewed when this information is available.

h) Secondary compression can be estimated using values for $C\alpha\varepsilon$ or 1% for the clay in the CMP and 1.5% for the clay capping.

i) For preliminary design purposes it is recommended that the profiles shown in *Figure 2.7.20* be adopted for lower bound and design undrained strength profiles. It is noted that these profiles are similar to the design and upper bound profiles adopted in the Initial Options Report and shown in *Figure 2.6.9*.

j) For the determination of strength gain due to consolidation under the reclamation fill it is recommended that the ratio $Su/\sigma v' = 0.25$ be adopted.

k) The strength profile of the capping layer is a critical parameter for the design of the sand blanket. The most relevant data in this respect obtained from the CPT and T Bar. As noted in *Paragraph 2.7.9.5*, the standard CPT and T Bar tests are carried out using a seabed testing rig weighing approximately 20 tonnes. The seabed rig will settle on the seabed surface and also have an influence on the CPT recorded cone load at shallow depths. It is recommended that the strength profile in the top 3 m of the CMP be reviewed when the results of the slurry CPT and slurry T Bar tests are available.

2.8 References

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3 DESIGN CONSTRAINTS AND LOADING

3.1 Tidal Sea Levels

3.1.1 Tidal levels applicable to the proposed reclamation site can be derived from recommendations made in Volume 1 of the Port Works Design Manual. Reference is made to the tidal levels proposed for the Lok On Pai tide recording station. This is the nearest tide recording station to the proposed reclamation and is located just 8.5 km from the proposed reclamation site.

3.1.2 The proposed tide levels have been assessed based upon data collected between 1982 and 1999 and is therefore considered to be representative of the current tidal level regime at the site. Allowances for the long term predictable changes in sea level are discussed later. Tidal predictions are presented in *Table 3.1.*

Table 3.1 Predicted Tide Levels

Location	**Mean Sea Level m PD**	**Mean Higher High Water m PD**	**Mean Lower Low Water m PD**
Lok On Pai	+1.2	+2.1	+0.3
Chek Lap Kok	+1.35	+2.15	+0.55

3.1.3 Previously, Greiner Maunsell undertook a detailed assessment of the likely tidal and surge levels at Chek Lap Kok. The conclusion from their analysis was that a small correction is required to adjust the predicted water levels to make allowance for the tidal effects which prevail within Urmston Road, the main channel separating Lok On Pai from the airport platform. A correction of +150mm was deemed appropriate to correct mean sea level while corrections of +250mm and +50mm were deemed appropriate for mean low lower water and mean higher high water at 1 in 5 year return respectively. These corrections would appear to be reasonable although it is noted that no long term records are available for tidal levels at Chek Lap Kok to corroborate this.

3.2 Extreme Water Levels & Surges

3.2.1 Extreme surge and low water levels have been predicted for differing return periods at Lok On Pai, the details being reported in the Port Works Design Manual. The predictions are based upon data collected between 1981 and 1999. These are presented in *Table 3.2*.

Table 3.2 Predicted Extreme High Water Level

Return Period Years	Lok On Pai m PD (PWDM)	Chek Lap Kok m PD (Maunsell Greiner)	Tsim Bei Tsui m PD (PWDM)
2	2.8	2.8	3.0
5	2.9	2.95	3.3
10	3.1	3.25	3.4
20	3.2	3.45	3.6
50	3.3	3.85	3.8
100	3.4	4.05	3.9

3.2.2 Maunsell Greiner also undertook a detailed analysis of water levels for use in the design of the sea defences for the existing airport platform. The results are also included in *Table 3.2*. It can be seen that there is some divergence between the two series of predictions (Lok On Pai and Chek Lap Kok) particularly at the longer return periods. The divergence is attributed to the magnifying shoaling influence of Deep Bay where surge water piles up in the Deep Bay inlet. The effect is more pronounced at Chek Lap Kok due to its closer proximity to Deep Bay. For the purpose of this report and the assessment of the feasibility of site formation over the contaminated mud pits the previously assessed extreme water levels for Chek Lap Kok remain appropriate as the difference is ostensibly too small to be of significance in the design of the reclamation..

3.2.3 Extreme low water levels are not critical in the design of the proposed reclamation. However the Port Works Design Manual gives guidance on extreme low water levels although the results of a detailed assessment of extreme low water levels against return period is only proposed for Quarry Bay. Results are presented in *Table 3.3***.**

Table 3.3 Extreme low water levels at Quarry Bay

Return Period Years	Quarry Bay m PD (PWDM)
2	-0.15
5	-0.20
10	-0.25
20	-0.30
50	-0.40
100	-0.45

3.2.4 It is noted that the lowest recorded water level at Lok On Pai has been -0.43 mPD while that at Quarry Bay has only been -0.28 m PD. Greiner Maunsell in their assessment proposed a minimum level of -0.55 m PD at 1 in 100 years return. The predictions made by Greiner Maunsell would again appear to make provision for the amplifying effect of Deep Bay. For the purposes of this assignment where minimum sea levels are non critical in the assessment of the feasibility of forming an expansion to the airport platform over the contaminated mud pits it is proposed to adopt the previously proposed levels adopted in the design of the existing airport platform.

3.3 Effect of Global Warming

3.3.1 It is a requirement to complete the design of any works for the proposed expansion to ensure the platform will remain serviceable for a design life of 120 years. Accordingly, whilst the Port Works Design Manual makes provision for typical design lives of 50 years the predicted water levels etc are based upon the statistical analysis of past observations making provisions in line with historical trends. Given the concerns over possible sea level rise resulting from global warming effects it is considered appropriate to make appropriate provision in the design of new facilities at Chek Lap Kok.

3.3.2 The Hong Kong Observatory commissioned a study into sea level changes in 2003. They concluded that between 1953 and 2003 sea level rise at Quarry Bay was 2.3mm per year. The study also reviewed data collected at Lok On Pai over a shorter baseline period concluding that there was little change in sea level at Lok On Pai. This finding was however not in line with other research work undertaken within the South China Sea and in particular satellite based measurements. The State Oceanic Administration of China forecasts 1.0 to 2.0mm per year. Church et al (2003) predicted a rise of 2.5mm to 3.1mm per year while Bindoff et al (2007) predict a sea level rise of 2.4 to 3.8mm per year. It is apparent that the Sea level changes will not be uniform throughout the world and Clark et al (1987) suggest rises about 15% above the predicted norm along the China coast. By contrast Yim (1991) concluded that as a consequence of crustal uplift in the Pearl river delta the consequences of sea level rises were almost cancelled out at North Point and Quarry Bay. Works Branch Technical circular 6/90 recommends making no provision for projects with a design life not exceeding 50 years but indicates that a provision of up to 10mm per year might otherwise be made subject to review with regard to cost implications.

3.3.3 On balance it is considered appropriate to make provision for sea level rise of 2.0 to 3.0 mm per year in undertaking this assignment. Accordingly, maximum sea levels detailed in *Section 3.2* above will be increased by 250 to 350 mm to make provision for a design operational period of 120 years. For the purposes of this study we have made allowances based upon historical observations. These allowances have been used to set the height of the sea defences and are aimed at establishing the seawall and filling profile required at the time of construction. Cost estimates and stability analyses have therefore been completed on this basis.

3.3.4 We observe that there is adequate provision in the proposed platform level to maintain drainage provisions and accommodate extreme storm events even under the influence of 4mm / year sea level rise.

3.3.5 The top level of the storm defences may become compromised in the event that sea level rise predictions significantly exceed the proposed 2-3mm / year sea level rise. This shortfall could however be corrected if necessary in the future by adding a raised coping to the seawall. The need for such a correction would not affect the feasibility of the proposed reclamation proposals and therefore do not influence the findings of this report.

3.4 Waves

3.4.1 The assessment of waves, their impact on the seawall armouring and the selection of an appropriate wall height to limit overtopping to acceptable levels is critical to the design of the reclamation sea defences.

3.4.2 A full assessment of the wave climate during typhoons was previously undertaken by Greiner Maunsell during the design of the existing sea defences for the airport platform. The study concluded the following, as shown in *Table 3.4*:

Table 3.4 Significant Wave Heights and Mean Periods used in Seawall Design

Return Period Years	Significant Wave Height Hs (in m) and Period Tz (in s)					
	Western Walls		Northern Walls		Eastern Walls	
	Hs	Tz	Hs	Tz	Hs	Tz
2	2.2	4.3	2.0	4.1	1.5	3.3

5	2.5	4.6	2.3	4.4	2.0	3.8
10	2.9	4.9	2.6	4.7	2.3	4.1
20	3.2	5.1	2.9	4.8	2.6	4.4
50	3.5	5.2	3.2	5.0	2.9	4.5
100	3.9	5.5	3.5	5.2	3.0	4.6
500	4.6	5.8	4.2	5.6	3.2	4.7
1000	5.0	6.0	4.5	5.7	3.5	4.8

(after Greiner Maunsell, 1991b)

3.4.3 The wind data used in deriving the extreme wave climate forecasts detailed in *Table 3.4* has changed very little since the earlier analysis was undertaken. The Port Works Design manual has updated the wind data to include more recent information collected over the period 1970 to 1991 while Greiner Maunsell's assessment was based upon data collected over the period 1951 to 1982. Accordingly, it is considered appropriate for the purposes of this assessment to make use of the design wave data previously adopted for the design of the sea defences surrounding the existing airport platform.

3.5 Reclamation Loading

3.5.1 Loading derived from the reclamation itself and the confining seawalls will comprise the dominant loading to be applied to the underlying soft marine deposits contained within the mud pits and the surrounding areas. Considerable attention has been given to the evaluation of loading criteria in the design of Penny's Bay Stage 2 reclamation. This reclamation has included filling derived from a variety of sources including public fill material, dredged sand and dedicated borrow area production. It must be assumed that the use of stockpiled public filling material currently held in fill banks will be required to make up a proportion of any new reclamation proposed by HKIA. The remainder, because of the volumes involved must be sourced from dedicated marine borrow areas. Loading criteria adopted for Pennys Bay Stage 2 reclamation are detailed in *Table 3.5***.** These criteria have been adopted in the geotechnical assessments undertaken as part of this study.

Table 3.5 Fill Loading Criteria

Material Type	**Bulk Density kN/m^3**
In situ Marine Deposits	17
Contaminated Marine Mud	Varies – refer to *Section 2.6*
Alluvial Clays	18
Sand Filling (river or marine)	18
Public Fill	18.0 below HWM 19.5 above HWM
Rock Fill	20
Pavements for runways and apron areas	23.5

3.6 Loadings

3.6.1 *Superimposed Loading*

3.6.1.1 Superimposed loading will be derived from four main sources:-

- Loading derived from surcharge systems;
- Loading derived from general circulation;
- Loading applicable to runways; and
- Loading applicable to taxiways and parking aprons.

3.6.2 *Surcharge loading*

3.6.2.1 Loading derived from surcharge systems will be regulated to provide adequate surcharge pressure whilst minimising the volume of filling necessary to achieve the required results. Surcharge loading will be applied at the rate of 19.5 kN / m^2 for each metre height of surcharge applied.

3.6.3 *General Circulation*

3.6.3.1 General circulation loading will generally only be of consequence to pavement structures. The transient nature of this form of loading will be such that it will not remain in place for sufficient time to instigate settlement of the reclamation. In assessing stability of the platform surface however it is considered appropriate to impose a circulation load equivalent to 10.0 kPa. This intensity of loading will be adequate to address loading from all standard road going vehicles and specialist vehicles on the site including the large foam tenders.

3.6.3.2 The 747 400 freighter currently applies the highest intensity of loading at pavement level. From the point of view of load intensity at pavement level the A380 is not the most critical. However, the A380 has the highest overall weight and an undercarriage footprint only slightly larger than the 747 400 freighter. Load intensity at the interface between the fill blanket and the marine deposits will therefore be highest under the influence of the A380 Airbus

3.6.3.3 We have completed an assessment of surcharge loading based upon A380 airbus loading (590,000kg) in determining loading intensity within the reclamation at the interface between the fill blanket and the marine deposits ie at -5 m PD level. 10kPa UDL at pavement level remains the most onerous superimposed loading applicable to the vulnerable level within the reclamation.

3.6.4 *Runway Loading*

3.6.4.1 Runway loading will be of a transient nature and as with circulation loading will not be applied continually. This type of loading will not contribute to surcharging contributing to consolidation of the reclamation. Locally, the static loading apparent at the underside of the runway paving slab is approximately 100 kPa arising from 4 x 200 kN wheel loads comprising a main landing gear bogie with axle centres at 1500mm and wheels spaced at 1400mm. Stability assessments in relation to the stability of the filling will be required to accommodate localised 100 Kpa loading with an appropriate margin for impact load application applied at approximately +5.5 m PD level. This is equivalent to a loading of less than 10 Kpa at the current seabed level of -5.0 m PD.

3.6.5 *Taxiways and Parking Aprons*

3.6.5.1 As with runways, taxiways and parking aprons will be required to accommodate loading from aircraft. Loading applicable to the reclamation will be of a transient nature but stability of the filling immediately beneath the pavement will be required to tolerate the equivalent of 100 Kpa. For the purposes of this assessment of reclamation feasibility, the blanket loading equivalent to 10 Kpa will remain applicable.

3.6.6 *Building Structures*

3.6.6.1 On the basis that terminal and other building structures will be common to all options under assessment and will be piled, the loading from building structures has not been considered in the stability and settlement assessment of reclamation. Piled structures will always be independent of the reclamation and as such are common to all development schemes. Items common to all schemes will not influence the feasibility of a development option and are therefore excluded from the assessment of feasibility. Similarly, localized loading from bridge structures and tunnel/drainage culvert structures will be addressed in a case by case basis in detailed design. Provisions for such localized phenomena are not considered likely to influence the feasibility of any development option.

3.7 Development Programme

3.7.1 The brief for this study requires the feasibility assessment of site formation options to be undertaken in respect of a notional reclamation arrangement. Similarly, this study is required to assess the feasibility of the site formation options against a notional construction programme. The assessment will be undertaken assuming construction will commence in 2011 and all construction will be required to be complete by the end of 2018. The 7 year time frame for construction and completion of the proposed new reclamation area must include for the following:-

- Construction of the seawalls and reclamation filling;
- Consolidation to effectively draw out all primary settlement; and
- Construction of paving works including runway and taxiways.

3.8 Rainfall

3.8.1 The surface profile of the new reclamation will be required to be graded to falls to facilitate drainage. Establishment of site levels and falls is described further in *Sections 4 and 5* below.

3.8.2 Rainfall intensity to be adopted in development of the drainage will be in accordance with parameters established by the Drainage Services Department. These will need to be supplemented to make provision for the large scale of the reclamation plan area.

3.8.3 A statistical assessment of the rainfall intensity, duration and depth has been undertaken by the Hong Kong Observatory. Design parameters are established in *Table 3.6*.

Table 3.6 Rainfall Parameters

Rainfall Intensity mm/hr							
Duration	**Return Period Years**						
	2	5	10	20	50	100	200
5 mins	134	168	190	212	240	260	281
10 mins	117	145	164	183	206	224	242
30 mins	85	107	122	136	154	167	181
1 hr	61	79	92	103	118	130	141
Rainfall Depth mm							
Duration	**Return Period Years**						
	2	5	10	20	50	100	200

2.0 hr	83	113	133	153	178	197	216
4 hr	109	155	185	214	252	280	308
12 hr	162	236	285	331	392	438	483
1 day	205	302	366	427	506	566	625
2 days	248	361	435	506	599	668	737
5 days	319	455	545	631	743	827	910
15 days	471	633	741	844	977	1077	1177
31 days	655	831	948	1059	1204	1313	1421

3.8.4 The existing airport platform has been constructed to falls with +6.0 m PD being the baseline level applicable to the northern seawall. Levels grade from runway level typically at +8 m PD on the northern runway. It is considered appropriate for a similar policy to be adopted in this study.

3.8.5 We have considered the possible effects of global weather pattern changes on the likely maximum rainfall to be handled by future drainage systems. We note that it is difficult to justify the need and expense for enhanced drainage works where historical records do not indicate such provisions are required. It has been concluded therefore that if such provisions were to be included within this study they would be applicable to all options and will not materially influence the feasibility of any one option.

3.9 References

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4 ACCEPTANCE CRITERIA

4.1 Settlement

4.1.1 The normally accepted residual settlement parameters adopted for general reclamations in Hong Kong are as follows:-

- Maximum Residual Settlement at 50 years after completion of the defects and maintenance liability period - **500mm**; and
- Maximum Differential Settlement at 50 years after completion of the defects and maintenance liability period - **1:300**

4.1.2 These settlement criteria have been adopted recently in Penny's Bay Stage 2 reclamation and represent the currently accepted criteria for Hong Kong reclamation projects.

4.1.3 Surcharging of the currently proposed reclamation must be undertaken with the objective of meeting these criteria for all areas of the new platform with the exception of areas to be occupied by the new runway. Runway settlement criteria are more onerous and performance criteria for this critically sensitive area have been specified by ICAO (1990). These criteria relate to differential settlement rather than absolute settlement and were previously adopted for the design of the existing runways at Chek Lap Kok.

4.2 Differential settlement

4.2.1 Differential settlement criteria adopted for the existing airport platform were addressed through the generally applied settlement criteria that residual settlement should be limited to 200mm following removal of surcharging. This is an onerous requirement which is required for selected areas of the apron and the runways. Other areas of the site need not however meet this requirement as they are not required to support settlement sensitive structures or infrastructure.

4.2.2 Differential settlement criteria for the most sensitive runway strip are defined by the ICAO as follows:-

- "The transition from one slope to another should be accomplished by a curved surface with a rate of change not exceeding 0.1% per 30m (Minimum radius of curvature 30,000m"; and

- Undulations or appreciable changes in slopes located close together along a runway should be avoided. The distance between the points of intersection of two successive curves shall be not less than the sum of the absolute numerical values of the corresponding slope changes multiplied by 30,000m or 45m whichever is greater.

4.2.3 In general these criteria can be interpreted as a differential settlement of 30mm over 45 m length of runway will be the limiting criteria.

4.3 Reclamation Stability

4.3.1 Reclamation stability is critical during the initial formation if mud-waves and local instability issues are to be avoided. Stability will essentially be maintained only if differential loading of the exposed weakest marine deposits is controlled such that differential stresses are maintained at levels below those which can be sustained by the marine clays as their strength accumulates during the consolidation process. *Section 6* of this report addresses the issues of maintaining the stability of the weakest strata in the earliest stages of filling. It is generally considered appropriate that during the initial filling stages no newly deposited strata shall exceed depths which might exceed the strength of the immediate underlying deposits by a factor of 3. This will result in the maximum side slope on any sand blanketing layer being restricted to 1:10.

4.4 Seawall Stability

4.4.1 Seawall stability criteria are set in the Port Works Design Manual. These are generally accepted criteria and will be applicable to any new designs as detailed in *Table 4.1* as follows:-

Table 4.1 Factor of Safety for Seawall Stability

Failure Scenario	Extreme Case	Normal Case
Sliding	1.5	1.75
Overturning	1.5	2.0
Deep Slip Failure	1.2	1.3

4.4.2 The above detailed criteria are normally taken to apply to conditions with a 1:5 year return for normal conditions and a 1:100 year return for structures designed with a 50 year design life. It is proposed that these criteria should be adopted for the proposed designs with appropriate adjustment being made to compensate for 1:100

year return period events but 1:120 year design life. The relationship between design life and return period of environmental conditions is defined in *Figure 11* of the Port Works Design Manual. Accordingly the return period for wave loading should be incremented to suit.

4.5 Overtopping

4.5.1 Seawall crest levels at the existing platform have been set at +6.5m PD level allowing for 500mm residual settlement and a small margin for rises in sea level due to global warming. These levels were established based upon overtopping criteria of 0.2 l / s / m of sea wall during credible normal storm events and 500 l / s / m during extreme events. Full details are indicated in *Table 4.2*.

Table 4.2 Seawall crest levels required to limit overtopping at existing airport platform

Location	Discharge (l/s/m)	Return Period Years	Seawall crest level (m PD)
North facing Seawalls	500	500	5.55
	0.2	20	4.75
	0.2	70	5.55
West Facing Seawalls (1v : 2 h)	500	500	5.55
	0.2	20	4.85
	0.2	40	5.55
East facing Seawalls	500	500	5.65
	0.2	20	5.05
	0.2	60	5.55

4.5.2 The Port Works Design Manual makes recommendations in respect of limits to overtopping following limits as defined by CIRIA 1991 for safety. These are defined in *Table 4.3.*

Table 4.3 Limits to overtopping for safety (PWDM vol 4) for safety

Risk	Overtopping rate l / s / m
Danger to Personnel	0.03
Unsafe for Vehicles	0.02
Damage to unpaved surfaces	50
Damage to paved surfaces	200

4.5.3 It can be seen that the initial design of the seawall crest and coping level for the existing platform has been set with a top level compatible with limiting the overtopping to levels well below those which might cause damage to unpaved surfaces at a return period of about 1:100 years. However, it noted that safe passage of vehicles and pedestrians would not be permissible with storms in excess of about 1:10 years. This is clearly an accepted and known risk associated with the existing seawall. It must be concluded therefore that the design overtopping rate of 0.2 l/s/m was previously set by virtue of the need to discharge the overtopping back to the sea efficiently. Drainage provisions compatible with 0.2 l/s have therefore previously been deemed to be appropriate and economical. It is proposed that this approach might be appropriate for the design of the newly proposed reclamation as it makes allowance for 500mm residual settlement and a margin of about 250mm to allow for long terms ea level rise. It is recommended however that the level of overtopping should be reassessed in line with more recent proposals in respect of long term sea level changes arising due to global warming.

4.6 Flood Resistance

4.6.1 As detailed in *Section 4.5* above a long term seawall overtopping rate of 0.2 l / s / m was deemed appropriate and economical previously. It is therefore proposed that, subject to verification, this rate of overtopping might be adopted once again, making due provision for the residual settlement and the proposed allowance for long term sea level rise.

4.6.2 Flood resistance to rainfall events should be developed in accordance with the provisions defined by the Drainage Services Department which make provision for discharge of rainfall arising during 1:200 year events.

4.7 Allowance for Changing Climate

4.7.1 We have concluded that this study must be undertaken within an established reference system for environmental parameters if bias between the options is to be avoided. The assessment of the options against known historical records is therefore considered to be most appropriate in terms of the requirement to assess option feasibility. This approach avoids possible crystal ball gazing which may well place bias particularly against systems which are environmentally sensitive i.e. those which corrode or deteriorate and in particular the piled and floating options.

5 SITE DEVELOPMENT CRITERIA

5.1 General

5.1.1 The following section summarises the basic design criteria adopted in the analyses which have been completed as part of the Assignment. The design criteria have been established based upon the need to comply with the Acceptance Criteria detailed in *Section 4*.

5.2 Site Formation Levels

5.2.1 Site formation proposals have been developed based upon forming the site nominally to a level of +6.0 m PD at the peripheral areas rising to the runway platform levels which will be established at levels ranging from +7.5 to +8.5 m PD. This is in line with the requirements for drainage as defined in *Section 4.6*. Site formation levels during construction will include a provision for residual settlement of 500mm occurring at the end of the defects and maintenance liability period.

5.3 Seawall

5.3.1 The site formation will be required to raise the development area to the levels specified in *Section 5.2* above whilst providing adequate capacity to accommodate the proposed superimposed loading equivalent to 10 kPa at sea bed level. It has been concluded that provision for 500mm residual settlement across the site plus an allowance of 250mm to allow for long term sea level rises will be appropriate. In critically settlement sensitive area including the runway it will be appropriate to design for a residual settlement of the order of 200mm following removal of any surcharge by the end of the maintenance period.

5.4 Filling Rates and Restrictions

5.4.1 In line with the criteria established in *Sections 3* and *4* of this report the seawall will be designed with a nominal crest level of 6.5m PD after residual settlement. Accordingly the crest profile will be established at +7.0 m PD level to comply with wave runup and overtopping design criteria.

5.4.2 Seawall armouring will be based upon the adoption of 1:2 slopes armoured with 5 – 7 Tonne primary armour. This armour can be recycled from the existing sea defences along the existing north shore of the airport platform.

5.4.3 The development of the proposed new platform will require the importation of over 80 million m^3 of fill material. An assessment of where this material will be sourced from lies beyond the scope of this report. However, it has been acknowledged that acquiring such large volumes of material will demand specific provisions to be made. In preparing the comparative cost estimates it has been assumed that it will not be possible to source such large volumes of material from borrow areas in China lying within close proximity to Hong Kong. Recent policy changes have also made the issue of sourcing the material from China unlikely. The cost estimates therefore assume that a marine sand borrow area will be developed within Hong Kong waters. Such reserves of sand fill have previously been identified and remain viable. These could be exploited to meet the needs of the proposed development.

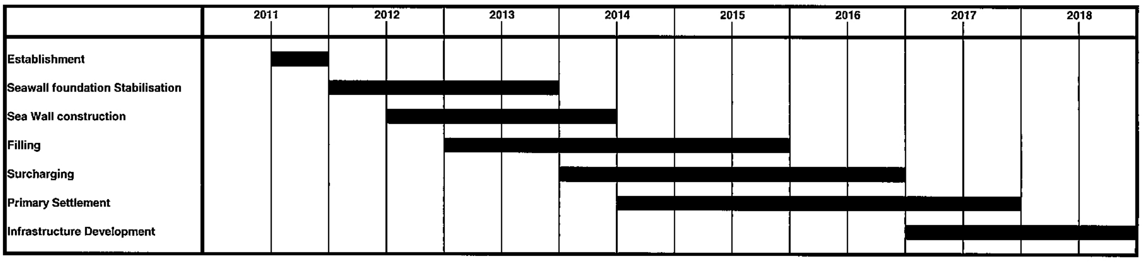
5.4.4 The current restrictions imposed by the flight envelope exclusion zones have been reviewed. At present, height restrictions and constraints on working would apply to reclamation operations within about 450m of the existing seawall copeline. This assumes a maximum equipment height of 15m above high tide. However it is observed that if the exclusion envelope were to be modified in line with provisions applicable adjacent to the south side of the southern runway then the restricted area could be reduced to as little as 50m from the existing seawall copeline. It can be concluded therefore that restrictions on working next to the operational runway could probably be limited to a narrow strip occupying only about 24Ha or less than 5% of the proposed 582Ha reclamation area. Provided therefore that the general height restrictions applicable over the sea at the present time can be amended in line with those applicable elsewhere on the airport platform, then the influence of the restrictions imposed by the operation of the existing runway on the project programme are likely to be limited.

5.5 Programme Requirements

5.5.1 It is proposed that the construction of the new reclamation works including implementation of the infrastructure works will be undertake between mid 2011 and the end of 2018.

5.5.2 Accordingly, the proposed filling and construction programme will generally follow the following outline programme as detailed in *Figure 5.1*. Specific details of the development programme for each option have been developed based upon this fundamental baseline programme making elaborations to suit each potential development scenario.

Figure 5.1 Outline programme



6 CONSTRUCTION TECHNIQUE OPTIONS

6.1 Background to Land Formation over CMPs

6.1.1 *Basic Principles of Reclamation Construction Over Soft Ground*

6.1.1.1 Before discussing the various methods available for constructing reclaimed land over the CMPs it is worth describing the basic issues which control this type of work.

6.1.1.2 The seabed in Hong Kong is blanketed by soft marine clay. In recent years, when constructing very large reclamations (for example the airport, CT9 and Penny's Bay) the soft clay has been dredged from the seabed prior to filling. The main reason for dredging the soft marine clay has been to limit the risk associated with ground settlement on these major infrastructure projects. However, a number of reclamations have been constructed over the top of the soft in situ marine clay, for example the reclamations for Cyberport, at Pak Shek Kok for the Science Park, the reclamation for the airport express depot at Siu Ho Wan and more recently the second phase reclamation at Penny's Bay. Endicott (2001) presents a useful summary of the construction of these "drained" reclamations in Hong Kong.

6.1.1.3 The major problems associated with constructing reclaimed land over soft marine clay relate to stability of the sea wall, stability of the leading edge of the reclamation fill during construction, the time taken for the soft clay to consolidate beneath the fill and achieving acceptable long term total and differential residual settlement criteria. Even where the soft clay has been left in place beneath the general reclamation areas, it has been normal practice to dredge the clay from beneath the alignment of the seawall, primarily to address stability issues but also to control long term settlement of the seawalls. The only recent exception to the practice of dredging for construction of the seawall is the construction of the seawall around Peng Chau Island (Dou and Swann, 2001).

6.1.1.4 The typical method adopted in Hong Kong for constructing reclamations over the in situ marine clay usually proceeds following some or all of the sequence of operations listed below:

a) Construction of the seawall over a trench dredged to the base of the soft marine clay
b) Placement of a geotextile filter cloth on the sea bed (not always included).

c) Placement of a sand drainage blanket in one or more layers, sometimes with a control on the slope of the leading edge of the blanket.

d) In deep water, installation of vertical drains through the full thickness of the sand blanket and the soft deposit. In shallow water, which does not allow marine installation, the vertical band drains can only be installed after the first stage filling has raised the reclamation level above the high tide level.

e) Placement of the first stage of the reclamation fill. The thickness of the first stage fill and slope of the leading edge being determined from stability analyses.

f) A delay period whilst the clay beneath the first stage reclamation fill consolidates and gains in strength.

g) Placement of the second stage reclamation fill, with thickness and slope being determined from stability analyses which adopt the increased strength of the marine clay.

h) A delay period whilst the clay beneath the second stage reclamation fill consolidates and gains in strength.

i) Placement of the final reclamation fill and construction of surcharge mounds.

j) A delay period whilst the clay beneath the reclamation and surcharge fill consolidates.

k) Removal of the surcharge fill after a pre-determined degree of consolidation has been achieved, typically 90%. This is confirmed on site using monitoring data. Endicott (2001) has proposed removal of the surcharge when the underlying marine clay has achieved an over-consolidation ratio of the order of 1.2 under the future working load. By achieving an over-consolidated state, the magnitude of ongoing secondary compression settlement can be significantly reduced.

6.1.1.5 The specific construction sequence depends on a number of factors including the initial thickness and undrained strength of the soft marine clay layer, the water depth, fill type available, time for construction etc. Additional steps such as vibro-compaction of sand fill or variations such as the use of stone columns to improve the characteristics of the marine clay have been adopted. However the general principles are always the same and include consideration of stability during construction, controlling/accelerating settlement during construction and achieving acceptable total and differential residual settlement characteristics after completion of construction.

6.1.2 *Difficulties Encountered During Construction*

6.1.2.1 Although there are a number of examples of reclaimed land having been constructed over the soft marine clay there have also been a number of both published and unpublished problems with these reclamations, examples of which are as follows:

a) There are known to have been stability related failures during construction of the reclaimed land at Cyberport. Mudwaves caused by these failures are known to be trapped under the reclamation. In order to deal with this problem, a supplementary contract for additional ground treatment by PVD installation and surcharging has been carried out. Despite this, ongoing settlement of the ground has caused damage to some of the buildings and associated facilities constructed on the reclamation.

b) A number of failures occurred during the construction of reclaimed land at Pak Shek Kok and mudwaves are known to be trapped under the reclamation. Although ground treatment by PVD installation and surcharging was included in the original construction work, a supplementary contract for additional ground treatment by surcharging has been carried out in order to improve the settlement characteristics sufficiently for roads and services to be constructed at the site.

c) There are known to have been stability related failures during construction of the reclaimed land for the MTR's Siu Ho Wan Depot. Mudwaves caused by these failures are known to be trapped under the reclamation. A separate contract for additional ground treatment by surcharging has been carried out. Even at locations without mudwaves, ongoing settlement of the ground has caused damage to some of the buildings constructed on the reclamation and required remedial structural work to be carried out.

6.1.2.2 Many of the problems associated with mudwaves have been caused by failures which occurred during construction, primarily because the leading edge of the reclamation was too steep for the weak ground which underlies these reclamations. The long term settlement problems which have resulted in damage to structures have been caused by incomplete consolidation of the soft clay deposit left in place beneath these reclamations. The settlement related problems have occurred despite the installation of vertical drains and the use of surcharges to accelerate consolidation during construction.

6.1.2.3 Even where reclamations have been successfully constructed over the soft clay deposits in Hong Kong, settlement related problems have occurred caused by either the water table being drawn down below the site (for example at Tsung Kwan O as a result of the SSDS tunnel construction) or overloading of the reclamation subsequent to construction (Cyberport).

6.1.2.4 The problems relating to stability, large settlements and the time required for settlements to occur are caused respectively by the weak and soft nature of the soft marine clay and its relatively low permeability.

6.1.2.5 The strength and stiffness of the clay is related to its past history and age. The Hong Kong soft marine clay is only relatively lightly over-consolidated having typical undrained strength of the order of 15 to 30 kPa and an over-consolidation ratio (OCR) of 1.5 to 2. The OCR of clay is defined as the ratio between the maximum previous (or apparent) effective stress acting on the clay and its current effective stress. In essence, it is a measure of the additional stress (or load) which the clay can sustain above which its stiffness decreases by a large factor (approximately 10) and large settlements occur. Both the strength and stiffness of clay are related to its OCR value, clay with a high OCR is both stronger and stiffer than the same clay with a low OCR.

6.1.2.6 The low permeability is caused by the fine grained nature of marine clay. However, as a result of the way in which the marine clay is deposited, its horizontal permeability is typically 2 to 10 times higher than its vertical permeability and this is an important factor when determining the speed at which vertical drains installed through the clay layer are capable of accelerating consolidation.

6.1.3 *Difference Between CMP Clay and Typical Hong Kong Marine Clay*

6.1.3.1 Within the study area the typical thickness of the CMP clay, including the capping layer, is approximately 15 m and increases up to 25 m in CMPIVa. The material in the CMPs was formerly the soft clay which blankets the seabed in Hong Kong, which has been dredged from the seabed and dumped in the CMPs. This process has resulted in very severe disturbance to the clay, initially causing an increase in the water content of the clay, a reduction in its mass strength, a loss in the apparent OCR value and a decrease in its stiffness. The CMP clay is weaker than the marine clay from which it was derived and because much of the clay is likely to be normally consolidated, it will be initially softer even though the Cc value may be lower than the in situ clay.

6.1.3.2 As determined by the recent ground investigation, most of the CMP clay is normally consolidated (i.e. OCR of 1) and there is evidence that at some locations the clay is still undergoing self weight consolidation at depth (i.e. still undergoing settlement due to primary consolidation). The self weight consolidation is associated with excess pore pressures being present in the clay. The severe disturbance to the clay will have caused it to lose much of its original structure. This in turn will have reduced the ratio between the peak undrained strength of the clay and its effective consolidation stress and will reduce the horizontal permeability of the clay in comparison to the normal in situ marine clay in Hong Kong.

6.1.3.3 The overall effect of these factors will be to exacerbate the stability problems and increase both the magnitude of settlement and time required to achieve consolidation. In addition, as a result of the variation in the sources of the material in the CMPs and the methods of dredging and placement, the material in the CMPs is more variable than normally encountered, resulting in a greater potential for differential settlement over relatively short distances.

6.1.3.4 Perhaps the most significant difference between the CMP clay and the typical seabed marine clay in Hong Kong is its very low initial undrained strength profile and associated low stiffness. This will have very significant implications for the placement of the first layer of fill on the seabed. The capping placed over the CMPs comprised a 1 m thick sand layer (not always present) overlain by between 2 to 4 m of dredged clay. Although uncontaminated the capping clay has similar physical characteristics to the underlying contaminated clay albeit that the ground investigation available to date indicates that it is more compressible. The upper meter has an extremely low strength of 0.5 to 2 kPa and may even comprise a slurry like material in places. This aspect is currently being investigated by “Slurry CPTs” and “Slurry T Bar” profiles, although the results were not available in time for this report.

6.1.3.5 For a variety or reasons, the most likely method of land formation over the CMPs will commence with the very careful placement of a sand capping layer on the surface of the CMP. Following this, a variety of ground treatment methods will be required. The following sections consider first the construction of the sand capping layer followed by consideration of the various possible methods of ground treatment required to ensure stability during construction and to control the rate and magnitude of settlement. Once the available ground treatment methods have been described, an assessment of the most appropriate methods of land formation will be presented for different situations ranging from general reclamation in non sensitive settlement areas (e.g. landscaped areas), through land formation for settlement sensitive areas (e.g. at the runway) and for construction of the permanent seawalls.

6.2 Overview of Construction Technique Options

6.2.1 The basic approach to construction of land over the CMPS and the associated ground improvement techniques are summarized as follows:

a) Place a sand capping/drainage blanket on the top of the CMP. The allowable thickness of the sand capping and the slope of the leading edge of the fill is controlled by the current undrained strength profile of the clay in the CMP.
b) Install vertical drains through the soft compressible clay to accelerate the primary consolidation settlement of the clay. Consolidation of the clay also leads to an increase in its undrained strength, allowing the thickness of fill to be increased and the slope of the leading edge to be steepened if required.
c) Consideration will be given at this stage to the potential for applying underwater vacuum consolidation to the clay layer in order to increase the rate of consolidation of the clay.
d) The use of deep well dewatering will also be considered at this stage to increase the rate of consolidation and to reduce the requirement for surcharging by fill placement.
e) Install additional ground improvement methods such as sand compaction piles or deep cement mix columns where these are required to either improve the strength of the ground, for example beneath the seawalls, or to control settlement, for example beneath the runway alignment.
f) Construct the reclamation in two or more stages making use of the increase in strength of the clay where necessary to improve stability.
g) Place surcharge to accelerate consolidation of the clay and to reduce long term settlement.

6.2.2 The following sections describe the various construction technique options following the overall construction sequence described above. Alternative methods of land formation using piles, semi buoyant and buoyant structures will then be described.

6.3 Construction of a Sand Capping Layer

6.3.1 As will be described in the various sections on appropriate ground treatment methods, the construction of a sand capping layer will be required as the first stage of the construction sequence for the following reasons:

a) To consolidate the upper very soft clay at and close to the surface of the CMP;
b) To provide a drainage layer for vertical drains which will be installed through the capping layer;
c) To limit the potential for shallow slip failures during subsequent filling operations;
d) To provide confining stress above the clay for the construction of sand drains, sand piles, sand compaction piles or stone columns;
e) To control ground heave during installation of all forms of ground treatment which require displacement of the clay in the CMP, including sand piles and deep cement mix columns;
f) To contain the clay which would be displaced at the surface of the CMP during installation and removal of construction tools used to install ground treatment elements; and
g) To assist in cleaning the tools used to install ground treatment elements as they are extracted through the sand blanket from the underlying clay.

6.3.2 As described in *Section 2.5*, the first stage of construction of the existing capping layer above the CMPs involved placement of a one meter thick layer of sand by "controlled" bottom dumping from barges. A review of the vibrocores taken through the CMP capping layers (see *Section 2.6*) indicates that, although the capping layer had successfully contained the contaminated clay, a continuous 1 m thick sand layer was not consistently achieved. Even at CMPII and III, where the capping placement was supervised more closely, the sand layer was not identified in many of the vibrocores and CPT profiles.

6.3.3 It is apparent from the above that placement of a capping layer by "controlled" bottom dumping is not appropriate for large scale reclamation works where a consistent and continuous sand capping layer is required over the clay surface. In view of the very weak and variable nature of the upper one to two meters of the clay capping, this is perhaps not surprising. Accordingly it is proposed that the sand capping layer be placed using a sand spreading techniques. This type of equipment can be controlled to place sand layers of a much more consistent thickness. The thickness can be controlled and, if required, can be as thin as 0.2 m or less. Various methods for sand spreading are available for example speed controlled spreader pontoons which place hydraulic sandfill into the water using weir or drum methods or direct placement of the sand layer on the seabed using sand spreading barges equipped with seabed spraying equipment or tremie pipes.

6.3.4 In order to control stability the layers will need to be placed in a manner such that each successive layer commences a distance behind the previous layer in order to generate an overall shallow slope profile. The thickness of each layer and overall slope angle is controlled by the current undrained strength profile in the ground.

6.3.5 The allowable slope angle and fill thickness is very sensitive to the undrained strength profile and this will need to be checked when the results of the preliminary ground investigation are available. *Table 6.1* presents a summary of the Factor of Safety against slip circle development as a function of slope angle and fill thickness for the lower bound and design strength profiles shown in *Figure 6.1*.

Table 6.1 Preliminary Stability Analyses for Sand Blanket Placement

Fill Thickness m	CMP Clay Strength Profile	Side Slope Gradient			
		1:8	1:10	1:12	1:15
2	Lower Bound	1.00	1.20	1.40	>1.6
3	Lower Bound	1.00	1.20	1.39	>1.6
4	Lower Bound	1.00	1.20	1.39	>1.6
2	Design	1.09	1.28	1.47	>1.6
3	Design	1.03	1.22	1.41	>1.6
4	Design	1.01	1.20	1.40	>1.6

6.3.6 Based on the design undrained strength profile set out in *Section 2.7* it will be necessary to adopt an initial slope profile of 1 Vertical to 10 Horizontal. More detailed stability analyses will be required when the initial undrained strength profile in the CMPs has been established by the ground investigation required for detailed design.

6.3.7 At the present time it is proposed that the speed controlled spreader pontoon or similar be used to place the sand capping in layers not exceeding 0.5 m in thickness and that the location for the start of each subsequent layer be controlled such that an overall slope angle of 1:10 is maintained. Based on an estimate of the undrained strength of the clay in the CMPs, this slope should maintain a factor of safety against local failure in excess of 1.2.

6.3.8 If the detailed ground investigation demonstrates that the strength is locally less than the assumed design line then the fill thickness and slope angle will have to be limited to prevent failure of the ground until sufficient strength has been gained due to consolidation following fill placement.

6.3.9 Irrespective of the follow on reclamation construction method, a target sand capping thickness of 2 m is proposed along with a minimum sand capping thickness of 1.5 m to allow for fluctuations in the seabed level. Provided the overall slope of the front of the capping layer is controlled, a thicker capping sand capping layer has many advantages and a capping layer thickness of 3 m or more would be beneficial if this could be achieved. The actual thickness of the capping layer will need to take account of the initial seabed level and the ground treatment method to be adopted, in particular the draft of vessels needed for the continuing reclamation work.

6.3.10 In order to control the fill placement extensive marine survey work will be required to check on each layer of fill placement.

6.3.11 It is noted that a geotextile is sometimes placed on the seabed prior to construction of the sand capping/drainage layer. The geotextile serves two purposes. First, it stops clay from penetrating into the sand layer thereby reducing the permeability and the drainage efficiency of the sand blanket. Second, it provides some additional strength on the clay surface to reduce the potential for localized failures.

6.3.12 Assuming that the sand blanket is placed using a speed controlled spreader pontoon or similar then it is not proposed to place a geotextile on the CMPs. When using a spreader pontoon the layer thickness is controlled such that local failure should not be a problem. Where the surface of the CMP is of slurry like consistency then a thinner layer of sand (say 0.2 m) can be placed as the initial layer and, in the absence of a geotextile layer, the sand can penetrate into the slurry. The resulting sand clay mixture will increase the density of the slurry and assist in more rapid consolidation and strength gain in this upper layer.

6.3.13 In addition to the above, most of the ground improvement techniques which will be adopted in the CMPs require some form of penetration through the sand blanket and underlying clay and the presence of a geotextile layer could interfere with the penetration.

6.3.14 For the above reasons it is presently proposed that no geotextile layer be placed on the top of the CMPs prior to placement of the sand blanket.

6.4 Accelerating Consolidation Using Pre-fabricated Vertical Drains

6.4.1 *Background*

6.4.1.1 The clay in the CMPs has a typical thickness of the order of 15 m and can be up to 25 m in CMPIVa. Without some form of ground improvement, this thickness of clay would take many years to consolidate under the load from a reclamation. Even assuming two way drainage of the clay in the CMP (i.e. into a sand blanket at the surface and into an alluvial sand layer beneath the CMP), without some form of vertical drainage installed through the clay layer the time required to achieve any reasonable degree of primary consolidation would be excessive. For example, it is estimated that it would take over 10 years to achieve 50% consolidation and over 40 years to achieve 90% consolidation without additional vertical drainage being installed.

6.4.1.2 The installation of vertical drains serves two purposes. First it will accelerate primary consolidation so that the reclamation can be constructed more rapidly. Second, the consolidation process is associated with an increase in the strength of the underlying clay, allowing the thickness of fill placed over the CMPs to be increased and the slope angle of the working face of the filling operation to be steepened

6.4.2 *Process Description*

6.4.2.1 The most commonly adopted vertical drain type in Hong Kong is the Prefabricated Vertical Drain (PVD). A PVD typically comprises a semi rigid core with in built drainage channels, which has a width of around 100 mm and thickness of approximately 3 mm, surrounded by a geotextile filter. There are a number of proprietary types of PVD available. Sand wicks are similar to PVDs and comprise a cylindrical geotextile sock filled with sand typically with 50 mm diameter. The function of a sand wick and PVD is similar.

6.4.2.2 The PVD is installed by driving a long hollow mandrel through the sand capping and underlying clay to refusal (or a defined depth) into underlying stiffer ground. During driving the PVD is within the mandrel. When the mandrel is extracted a steel plate attached to the base of the PVD remains stuck in the underlying stiff deposit and the PVD remains in the ground as the mandrel is removed. The PVD is cut off above the drainage blanket.

6.4.2.3 During driving the base of the mandrel is blocked by the steel plate so no contaminated clay should enter the mandrel. On extraction, a small amount of clay can adhere to the outside of the mandrel.

6.4.2.4 The PVDs can be installed from either marine barges or from land based equipment. In order to maximize the effectiveness of the drains it would be preferable to install them at an early stage in the construction process from marine barges. Specially fabricated barges have been built which are capable of installing 12 PVDs at a time at spacing down to 1 m and to depths of 40 m. During ground improvement work associated with the construction of a breakwater at Tianjin Port, Yan et al. (2008) report that a similar barge had installed 1200 drains per day to a depth of approximately 20 m.

6.4.3 *Fundamental Parameters and Requirements*

6.4.3.1 PVDs are installed in either a square or triangular grid pattern. The required spacing between the PVDs is determined by the time that is required to achieve a pre-determined degree of primary consolidation, typically a criterion such as 90% consolidation in 6 months. The fundamental parameter which controls the time for consolidation is the horizontal coefficient of consolidation (C_h). This parameter is itself a function of both the stiffness and permeability of the soil, both of which vary with stress level. Accordingly, ch is not a constant. As described in *Section 2.7*, ch is likely to vary in the range 0.5 to 1.0 m^2/year and is also likely to increase slightly as the stress level increases.

6.4.3.2 *Figure 6.2(a)* to *(c)* present the time required to achieve varying degrees of consolidation for PVD spacing of 1.0, 1.2 and1.5 m for C_h values of 0.5, 1.0 and 2.0 m^2/year.

6.4.3.3 Based on an assessment of the consolidation characteristics of the CMP clay it is likely that a PVD drain spacing in the range 1 to 1.2 m will be required to achieve an acceptable degree of consolidation (80 to 90%) within a reasonable time frame (less than 12 months). For preliminary design purposes a spacing of 1.0 m is recommended. The relatively close drain spacing is required because the clay in the CMP is heavily disturbed and the horizontal permeability will be similar to the vertical permeability. In normal design practice an allowance is made for a "smear" zone around the drain caused by the process of installation. Because the clay in the CMP is heavily disturbed there should be no requirement to allow for a smear zone around the drain in the CMP.

6.4.4 *Benefits, Disadvantages, Limitations and Risks*

6.4.4.1 The purpose of installing vertical drains is to accelerate consolidation settlement of the soft clay in the CMPs and to make use of the gain in strength of the clay to enhance the stability of the edge of the reclamation fill or seawall.

6.4.4.2 Very significant settlements will occur within the CMP clay typically of the order of 3 to 4 m, equivalent to 20% to 25% of the layer thickness and in excess of 5 m at some locations. There is a potential concern about loss of drain efficiency where such large strains take place due to a combination of both drain clogging from fines which permeate the geotextile membrane and kinking of the drain. In this respect it is noted that Bo et al. (2005) adopted two phases of PVD drain installation to accelerate the consolidation of a slurry pond in Singapore. A first phase of drains was installed at 2 m centres and a second phase was installed at a later stage to close the spacing down to 1 m. Research carried out by the Port and Airport Research Institute in Japan has found that the discharge capacity of PVDs decreases by up to approximately 50% at these large strains.

6.4.4.3 The presence of obstructions such as boulders in the CMP could prevent installation of a PVD. An isolated obstruction which prevented the installation of a single drain would not be a particular problem. However, if a barge load of rubble had been placed in a CMP then this could prevent the installation of a group of PVDs, which would result in a long term localized settlement problem. A 1 m thick layer of gravel sized fragments was encountered in Borehole 4a-2, which demonstrates that such obstructions might be present, and more detailed investigation is required to determine the extent of such obstructions. Similarly, significant thicknesses of sand were encountered by some of the CPTs which would also pose difficulties to drain installation. When the Contract P398 ground investigation work is completed the results should be reviewed with relevant contractors to confirm that PVDs can be installed.

6.5 Accelerating Consolidation using Sand Drains

6.5.1 *Process Description*

6.5.1.1 The majority of large scale reclamation work over thick deposits of soft clay in Japan use sand drains rather than PVDs. A sand drain is a column of sand constructed through the full thickness of the soft clay. A sand drain diameter of 400 mm is typically adopted in Japan with spacing varying from around 1.5 to 2.5m. Smaller diameter sand drains have been adopted on other projects.

6.5.1.2 The sand drain is installed by pushing a steel tube through the sand capping and soft clay and driving it into the underlying stiffer deposits below the base of the CMPs. The steel tube usually penetrates using self weight but can also be assisted by either vibration at the top of the tube or water/air jetting at the base of the tube. The steel tube is initially blocked by a hinged plate at the base and the tube is installed by displacement methods and does not require boring. As the tube is extracted sand is placed through the base of the tube to form a continuous sand column up through the CMP clay and into the sand capping. On extraction a small amount of clay can adhere to the outside of the steel tube.

6.5.1.3 The sand drains can be installed from either marine barges or from land based equipment. In order to maximize the effectiveness of the sand drains it would be preferable to install them at an early stage in the construction process from marine barges. Specially fabricated barges have been built which are capable of installing 12 sand drains at a time to depths in excess of 40 m.

6.5.2 *Fundamental Parameters and Requirements*

6.5.2.1 The design of sand drains is based on the same parameters as for PVDs with the spacing determined from an assessment of the C_h value for the ground. *Figures 6.3(a) to (c)* present the time required to achieve varying degrees of consolidation for sand drain spacing of 1.2, 1.5 and 2.0 m for C_h values of 0.5, 1.0 and 2.0 m^2/year assuming a sand drain diameter of 400 mm.

6.5.2.2 Based on an assessment of the consolidation characteristics of the CMP clay it is likely that a PVD drain spacing of approximately 1.5 m will be required to achieve an acceptable degree of consolidation (90%) within a reasonable time frame (less than 12 months).

6.5.3 *Benefits, Disadvantages, Limitations and Risks*

6.5.3.1 The main advantage of sand drains over PVDs is that they have a higher flow capacity and do not become blocked. They also provide a small amount of additional strength to the soil and will reduce settlement and provide a small additional factor of safety for slope stability. Based on a sand drain diameter of 400 mm and a sand drain spacing of 1.5 m, the area replacement ratio is approximately 7%.

6.5.3.2 Because installation is by a displacement method there will be some ground heave during installation. Assuming a 16 m thick clay layer the maximum ground heave based on 100% undrained conditions would be approximately 1.1 m. However, there will be some consolidation taking place during installation and, as a result, the ground heave will be less and probably of the order to 0.8 to 1 m.

6.5.3.3 The presence of obstructions such as boulders in the CMP could prevent installation of a sand drain. An isolated obstruction which prevented the installation of a single drain would not be a particular problem. However, if a barge load of rubble had been placed in a CMP, then this could prevent the installation of a group of sand drains, which would result in a long term localized settlement problem. The CPTs carried out in the CMPs have identified thick sand layers at some locations which will require water jetting to penetrate.

6.5.3.4 It is also understood that the sand drain equipment has difficulty penetrating through thick layers of sand with SPT N in excess of 15. The capping layer placed on the CMPs includes a sand blanket. If this is locally thick then it may be necessary to use water of air jets to penetrate through the sand and this could potentially result in contaminated clay from below the sand capping to be blown back up the outside of the steel installation pipe. When the Contract P398 ground investigation work is completed the results should be reviewed with relevant contractors to confirm that sand drains can be installed.

6.5.3.5 When a PVD is subjected to a large compressive deformation due to settlement, this can cause buckling and/or kinking deformation of the PVD and this could result in reduction of discharge capacity. Pradhan et al. (1991) report that the discharge capacity decreases by approximately 85 to 80% of the initial value when subjected to the axial compressive strain of 50%. Ali (1993) carried out laboratory model tests and the results showed that all the drains show considerably decreases (approximately 50 to 95%) in the discharge capacity when subject to compressive strain of the order of 20 to 30%. CROW (1993) reports that the discharge capacity of buckled drains may reduce to about 25 to 50% in comparison to a straight drain.

6.6 Underwater Vacuum Consolidation

6.6.1 *Process Description*

6.6.1.1 When fill is placed over clay the water pressure in the clay initially rises to a value above the normal "hydrostatic" value. As this "excess" pore pressure dissipates the clay consolidates resulting in ground settlement and an increase in the strength of the clay. The same end result can be achieved by reducing the hydrostatic pressure to generate an excess pore pressure in the clay. This is referred to as vacuum consolidation. The process has been adopted in China and Japan to treat soft ground where the soft material is at the ground surface.

6.6.1.2 The standard process is carried out on land as follows:

a) Vertical drains are installed through the clay layer (PVD or sand drains). The drains are stopped approximately 1 m above the base of the layer to be treated.
b) A granular drainage blanket is placed over the top of the drains with a grid of horizontal drains placed adjacent to the rows of vertical drains
c) An impermeable membrane is placed over the top of the drainage blanket and sealed around the edges of the area to be treated by burying the edge of the impermeable membrane in a clay filled trench
d) Vacuum pumps are used to lower the water pressure in the drainage blanket, in essence sucking the water out of the clay.
e) The vacuum pressure reduction is maintained for a number of months in order to consolidate the underlying soft clay.

6.6.1.3 Theoretically a maximum vacuum of approximately 100 kPa is possible. However in practice a reduction of the water pressure of the order of 70 to 80 kPa is actually achieved. The application of a vacuum of 80 kPa is equivalent to the placement of a 4 m high surcharge fill, assuming the fill is placed above the water table.

6.6.1.4 The main advantages of vacuum consolidation over the placement of 4 m of fill are that there are no stability related problems and there is no requirement to import the surcharge fill material. The technique is particularly suitable for treating very soft deposits which have not attained sufficient strength for the placement of fill of a reasonable thickness. Chu et al. (2000) report that the vacuum pre-loading is adopted in China when clay slurry dredged from the seabed is used as fill for land reclamation projects. Thousands of hectares of land have been reclaimed in Tianjin in this manner. Details of a typical application of vacuum consolidation used to consolidate dredged clay fill at Tianjin is given in Yan and Chu (2005).

6.6.1.5 For land formation at the CMPs, the use of vacuum consolidation would be particularly useful for a situation where the clay in the CMPs is still in a slurry like condition. In this situation there would be very onerous stability considerations.

6.6.1.6 As noted above, the standard vacuum consolidation process is carried out on land whereas the ground to be treated for land formation above the CMPs is typically 6 or more meters below the mean sea level. Although an underwater trial of vacuum consolidation has been carried out in Japan and in China, it is understood that the there were very significant technical difficulties fixing the drainage layer and impermeable membrane. Based on discussions with experts in China and Japan it is understood that there has been no experience of using an underwater construction technique on a real construction project. Research has been carried out recently on the potential for underwater vacuum consolidation for development work in Tianjin, including carrying out model tests but it does not appear that a full scale test has been carried out.

6.6.1.7 A hybrid land/underwater system has been used for port development work in Shenzhen. At this location the water depth was relatively shallow and a bund was constructed around the site to be treated allowing the site to be dewatered and the surface of the clay deposit to be exposed. The vacuum drainage system, including horizontal drains, drainage blanket and impermeable membrane was installed over the dewatered site area. Once the vacuum drainage system was installed the site could be flooded and the actual vacuum consolidation work was carried out underwater. *Figure 6.4* shows a schematic view of this hybrid land/underwater system.

6.6.1.8 One advantage of the hybrid land/underwater system is that when the site has been flooded the water increases the hydrostatic water pressure and the vacuum system can then be used to achieve a greater consolidation stress.

6.6.1.9 The water depth at the CMPs is too deep to allow a simple bund system to be adopted to enable the site to be drained and a vacuum consolidation system to be installed using the hybrid system. However, although difficult it would not be impossible to construct a seawall around the whole site and then construct a vertical cut-off structure within the seawall. The vertical cut-off structure could then be connected to an impermeable membrane which would then allow the site to be drained and a land based vacuum consolidation system to be installed. A simpler version of this, which uses dewatering rather than vacuum consolidation, is discussed in *Section 6.7*.

6.6.1.10 Nooy Van derr Kolff and Mathijssen (2003) present details of a modified PVD system for vacuum consolidation referred to as the "BeauDrain" manufactured by Royal Boskalis Westminster nv. The BeauDrain is in essence a pre-fabricated PVD which is driven to the within approximately 1 m of the base of the layer to be treated and the top of the drain is buried approximately 1 m below the top of the clay layer. A vacuum tube is attached to the top of the drain and can be connected through horizontal pipes to vacuum pumps. The system works by using the top meter of the clay layer to act as the impermeable membrane. This does away with the need to place a drainage blanket and impermeable membrane.

6.6.1.11 A modification to the installation method has been developed using a combined plough and drain installation rig. The system installs the pre-fabricated BeauDrains at a predetermined depth below ground level and at the same time places a horizontal collector drain covered by an impermeable membrane below the ground connected to the BeauDrains. Although this system has been used using land based equipment to manipulate the plough it has not been used below water.

6.6.1.12 Van Impe (2001) describes a project where horizontal drain pipes (as opposed to vertical drains) were installed in a very weak silty clay layer (strength of about 1 kPa) with a thickness of 6.5 m. The drains were installed using a specially developed plough. It is likely that both the depth of the mudpits and the presence of denser sand layers within the CMP clay will render this installation method unsuitable.

6.6.2 ***Fundamental Parameters and Requirements***

6.6.2.1 The investigation carried out to date has demonstrated that standard methods using staged fill placement can be adopted over the CMPs. Nevertheless, if the detailed ground investigation identifies that the undrained strength profile of the clay in the CMPs is locally very weak then vacuum consolidation with associated installation of vertical drains could be considered to assist in consolidating the clay. The fundamental design parameter is the coefficient of horizontal consolidation, which is the same as for the design of vertical drainage systems. As before it is believed that the coefficient of horizontal consolidation will vary from 0.5 to 1.0 m^2/year. *Figure 6.5* presents an assessment of the time required to achieve 90% consolidation for various different drain spacings, assuming the use of PVDs for the vertical drains. Based on these results it can be seen that a drain spacing of approximately 1 m will be required to achieve a reasonable degree of consolidation over a typical construction period.

6.6.3 ***Benefits, Disadvantages, Limitations and Risks***

6.6.3.1 The major benefit of a vacuum consolidation system is that it enables very weak ground to be consolidated in conditions where normal fill placement would cause significant stability problems. A second advantage is that the system can be combined with a normal filling procedure so as to avoid surcharge fills being required.

6.6.3.2 The main disadvantage of the method is that it has never been attempted underwater on the scale that would be required at the CMPs.

6.6.3.3 The CMPs have been capped by the placement of a 1 meter thick layer of sand and a 2 meter thick layer of clay. If this had been constructed exactly as planned and the sand layer was present continuously over the full extent of the top of the CMPs then it would be possible to use the sand layer as the top drainage layer and the upper 2 m of clay capping as the impermeable membrane. This would be an ideal situation for the application of vacuum consolidation. Unfortunately the available ground investigation indicates that the sand capping is not continuous. However this option could be considered if more detailed ground investigation can identify large areas where the continuity of the sand layer is assured.

6.7 Deep Well Dewatering

6.7.1 ***Process Description***

6.7.1.1 Deep well dewatering is similar to vacuum dewatering in as much as it relies on the reduction of the hydrostatic pressure to create excess pore water pressures in the clay. In this method individual pumping wells are used to draw the water pressure down in permeable horizons above or below the clay in the CMPs. The deep well dewatering would be used as a method of surcharging the CMP clay in conjunction with vertical drains through the clay layers. Deep well dewatering could be used in two situations as follows:

a) After placement of fill over the CMPs has reached above the maximum tide level, an impermeable cut-off wall is constructed through the fill and into the top of the CMP clay around the perimeter of an area to be treated. Dewatering wells with pumps would then be used in the fill to reduce the water level down to the base of the fill, which is typically below -5 mPD. The ground water level in a sand fill reclamation is usually at or above the mean sea level (+1.3 mPD). Reducing the ground water level to -5 mPD would be equivalent to the effect of applying approximately 60 kPa of surcharge loading. As the clay consolidates and the base of fill to top of clay interface settles, the water level can be reduced further. As settlements of 3 or more meters are expected, deep well dewatering could eventually be equivalent to a 90 kPa surcharge, which is equivalent to placing a 4.5 m thick fill surcharge layer. Because the impermeable cut-off structure is constructed in the fill and there is no requirement for excavation, the impermeable cut-off can be non structural, for example a thin bentonite cement slurry wall of the type used as containment structures around landfill sites. An alternative method to an impermeable cut-off wall is to install additional pumped wells around the perimeter of the site to be dewatered. The additional pumped wells on the periphery introduce nodes of low pore pressure and prevent water from outside the site flowing in, acting in a similar manner to a cut-off wall.

b) The CMPs are underlain by alluvial deposits of the Chek Lap Kok formation. These range from firm to stiff clay through medium dense to dense sand to a basal sand and gravel layer. In areas where the CMP is underlain by a sandy stratum it would be possible to install dewatering wells below the CMPs in order to underdrain the clay. This would have the effect of accelerating the consolidation of the clay in the CMP.

6.7.2 *Fundamental Parameters and Requirements*

6.7.2.1 The design of a deep well dewatering system would rely on a system of vertical drains in the soft clay and dewatering wells in the overlying fill or underlying alluvial sand. The vertical drain system would be designed in the same manner as would be adopted for a normal stage construction with surcharging. PVDs would be required at 1 m spacing or sand drains at approximately 1.5 m spacing.

6.7.2.2 If the deep well dewatering is to be carried out only from the fill then the vertical drains should be terminated approximately 1 m above the base of the clay in the CMPs or underlying alluvial clay. However if dewatering is envisaged from beneath the clay then the drains should be driven into the underlying alluvial sand layer where present.

6.7.2.3 The design of the pumping system for dewatering the fill above the CMP would need to be capable of:

a) Removing the water from the fill, assume 30% porosity and 7 m depth which is approximately equivalent to a 2 m depth of water.
b) Removing the water produced due to settlement of the ground, assume equivalent to a 3 m depth of water.
c) Removing rainfall which seeps into the ground (typical yearly rainfall is 2.5 m and monthly rainfall in summer is of the order of 0.5 m per month).
d) Water which penetrates through or under the cut-off wall.

6.7.2.4 As a preliminary design, wells at 50 m centres with a capacity of 10 l/second would be suitable for reducing the water level in the fill. Once the water level is drawn down the pumps would not be required to work continuously.

6.7.2.5 The design of a pumping system for dewatering granular layers below the CMP would need to consider both the water produced due to consolidation of the overlying clay and the water which will flow laterally through the ground from outside the site area. In order to reduce the lateral flow of water from outside the site it would be possible to construct a grout curtain in the alluvial deposits around the site.

6.7.2.6 The duration of pumping would be controlled by the design of the vertical drains system. It is envisaged that the dewatering system would be required to operate for 6 months to one year after the reclamation had reached formation level. Assuming that such a dewatering scheme is designed to replace surcharging by fill placement, then it will be necessary to construct the reclamation to a high level to allow for the settlement which would be expected.

6.7.3 *Benefits, Disadvantages, Limitations and Risks*

6.7.3.1 For large scale reclamations in situations where time for construction is not a controlling factor, the use of surcharge fills is cost effective because the surcharge can be moved around, allowing the fill to be used more than once. The major benefit of using dewatering to surcharge the ground is that there is no requirement for surcharge material to be placed on the reclaimed land. In addition, stability concerns associated with surcharge loading, particularly adjacent to the seawalls, are not an issue for deep well dewatering. A deep well system which dewaters the fill above the CMP would require impermeable cut-off walls around the area to be dewatered.

6.7.3.2 A deep well system which dewaters the alluvial deposits beneath the CMPs would require a very detailed understanding of the site hydrogeology. Deep well dewatering from beneath a site would not be appropriate for small reclamations because the lateral flow from outside the site would dominate the ground water flow. However, when considering large scale reclamation work, the granular layers beneath the site are relatively thin in comparison to the width of the reclamation and the assumption that these layers act as a free draining boundary may not be not appropriate. During construction of the existing airport reclamation it was assumed that the basal sand and gravel layer at the base of the CLK formation would act as a free draining boundary and that this would assist consolidation of the clay layers within the CLK formation. However, groundwater pressure monitoring carried out in the basal sand and gravel layer indicated that excess groundwater pressures were building up in this layer and that the layer was not actually behaving as a free draining drainage layer (Plant et al. (1998)). This was because of the long distances required to drain the water from beneath the reclamation. This excess ground water pressure which was generated in the underlying basal sand and gravel layer reduced the rate at which the ground was settling.

6.7.3.3 Deep well dewatering from within the basal sand and gravel would have increased the rate at which the existing reclamation had settled and reduced the magnitude of residual settlement. The same considerations will apply for the proposed land formation works, particularly if a wide reclamation option is adopted.

6.7.3.4 It is unlikely that deep well dewatering alone could be used to replace surcharging by fill to achieve acceptable settlement characteristics over the CMPs. This is because variations in the underlying geology at the site mean that an alluvial sand layer may not always by present below the CMPs and because this system tends to treat the bottom of the clay in the CMPs rather than the clay near ground surface. Nevertheless serious consideration should be given to including this option where surcharging is being considered as it will assist in the consolidation process.

6.7.3.5 Another potential difficulty with deep well dewatering would be driving the PVDs into the underlying permeable strata, particularly if the top of the underlying alluvial material is stiff clay.

6.8 Sand Compaction Piles

6.8.1 All of the ground treatment methods described in the previous sections have been associated with accelerating the rate of consolidation of the very soft clay in the CMPs and making use of the strength gain as the clay consolidates to achieve a stable slope to the reclamation face. The following section describes the use of sand compaction piles which are installed to improve the clay in the CMP, increasing the overall mass strength of the material in the CMP, decreasing the magnitude of settlement and increasing the rate of consolidation. Sand compaction piles are used extensively in Japan and Korea to improve the ground below vertical and sloping seawalls, primarily to improve stability and also to control the settlement.

6.8.2 *Process Description*

6.8.2.1 A sand compaction pile refers to the construction of a column of dense sand through the full thickness of the clay layer. The majority of sand compaction piles are installed from marine barges which have been specially constructed for this purpose. The pile is formed in the following manner:

a) A 1 to 2 meter thick sand blanket is often, but not always, placed on the seabed prior to installation of the SCPs.

b) A steel tube, typically varying in diameter from 0.4 to 0.8 m, is pushed from the ground surface to below the base of the layer to be treated. Insertion of the tube can be assisted by vibration at the top of the tube and air/water injection at the base of the tube. The tube is blocked during insertion and there is no boring or removal of spoil as the tube is inserted.

c) When the steel tube has reached the required level it is withdrawn a short distance and at the same time sand is forced out of the base of the tube by compressed air.

d) The level of the sand in the tube is monitored to ensure that the tube always contains sand and that the clay around the tube is not allowed to collapse below the tube.

e) The tube is then pushed back into the sand and vibrated back on top of the sand that has been deposited in the ground below the tube in order to increase the diameter of the sand column by pushing it out against the clay and densifying the sand at the same time.

f) When the desired diameter has been achieved (determined from the known volume of sand placed in the column) more sand is added to the tube and the process of sand placement and compaction is repeated.

g) The process is continued until the sand compaction pile has reached the desired level.

6.8.2.2 The sand compaction piles are installed at relatively close centres and the extent of the treatment is usually referred to as the replacement ratio, which is the ratio of the area of the sand compaction piles to the area of the ground being treated. The replacement ratios are often very high. For example, a design replacement ratio of 70% has been adopted when used to support the caisson for the construction of the new port facilities at Busan and in practice a higher replacement ratio is often achieved. The practical advantage of this system is that the majority of the clay beneath the caisson structure is replaced by compacted sand without requiring a deep trench to be dredged into the soft clay. Because of the relatively shallow side slopes that would be required in the soft clay the cost savings associated with both dredging and filling can be substantial.

6.8.2.3 The sloping rockfill seawalls which are being constructed around the perimeter of the reclamation for the fourth runway at Haneda airport are being placed in 20 m depth of water over approximately 20 m thickness of soft clay. The soft clay is close to being normally consolidated. The soft clay is being improved using sand compaction piles with a replacement ratio of 30%, which penetrate approximately 4 m below the base of the clay.

6.8.2.4 The construction of a sand compaction pile is a displacement process, with the clay adjacent to the pile being pushed aside. There will be an increase in the lateral stress acting on the clay, which will raise the pore pressure in the clay and also shearing of the clay will result in an increase in the pore pressure in the clay. Because of the high replacement ratio used and the size of the sand columns there will be fairly rapid consolidation of the clay, which assists in increasing the strength of the clay and reducing the potential for settlement of the clay. Nevertheless, the installation sequence must be controlled so as to minimize disturbance to SCPs which have already been completed. This requires an installation sequence which works away from SCPs which have already been installed. One of the most significant issues which needs to be considered for use of SCPs in the CMPs is that of ground heave associated with SCP installation and this is discussed in *Section 6.8.3*.

6.8.3 *Fundamental Parameters and Requirements*

6.8.3.1 Sand compaction piles are usually used to enhance the strength of the ground beneath a seawall, allowing more rapid construction than having to wait for the clay beneath the wall to consolidate and gain strength. For this purpose, the fundamental design parameters relate to the design friction angle that can be achieved in the sand compaction pile, the area replacement ratio, the slope of the seawall and the undrained strength profile in the clay.

6.8.3.2 Stability analyses are carried out to determine the required slope geometry and SCP replacement ratio. Because the sand compaction pile is significantly stiffer than the clay between the piles, the SCP tends to carry more of the load from the fill placed over the area than the clay. This increases the vertical stress in the SCP in comparison to the clay and, because the strength of an SCP is frictional in nature, the increase in the vertical stress also increases the frictional resistance of the SCP. Standard stability analyses cannot take account of this strength enhancement and as such either empirical enhancement factors are adopted or the strength improvement is modelled using finite element programs which take account of load re-distribution.

6.8.3.3 Preliminary stability analyses have been carried out to assess the typical replacement ratio that is likely to be required to stabilize sloping seawalls constructed over the CMPs. The analyses have assumed the following combination of conditions:

a) Two different initial undrained strength (Su) profiles for the clay in the CMP. The first is similar to the “initial design profile” shown in *Figure 6.1* (Su = 1 + 1Z, where Z is the depth below the top of the clay). The second is an “enhanced strength profile” which assumes that a 3 m thick sand blanket has been placed on the CMP and that sufficient time has been allowed for the clay to consolidate under the weight of the fill (Su = 6 + 1.2Z).

b) Three different SCP replacement ratios. 33% is typical of the lower bound replacement ratio that is commonly adopted, 50% is typical of the replacement ratio that is adopted below sloping seawalls, 66% is typical of the replacement ratio adopted below the rock fill bunds supporting vertical caisson walls.

c) Two different slope angles for the seawall ranging from 1V to 2H, a relatively steep seawall to 1V to 3H, a relatively shallow angle seawall.

d) The body of the seawall is granular and a friction angle of 35 degrees has been assumed. This is a relatively conservative value.

e) The maximum height of fill placement is +10 mPD to allow for surcharging of the ground.

f) The SCPs extend out to the toe of the seawall slope.

6.8.3.4 The factor of safety against rotational slips is summarised in *Table 6.2*.

Table 6.2 Seawall Stability Analyses for SCP Ground Improvement

SCP Replacement Ratio	Undrained Strength Profile	Factor of Safety	
		Seawall slope 1:2	Seawall slope 1:3
33	Design Profile	0.77	1.06
50	Design Profile	0.99	1.27
66	Design Profile	1.12	1.36
33	Enhanced Profile	0.99	1.25
50	Enhanced Profile	1.17	1.46
66	Enhanced Profile	1.30	1.54

6.8.3.5 The factors of safety given in *Table 6.2* are indicative only and are sensitive to the assumptions made in the analyses. Based on the assumptions presented above it is apparent that if a seawall is to be constructed over the CMPs without allowing time for a sand blanket to increase the strength of the clay then both a relatively shallow seawall slope and a relatively high SCP replacement ratio will be required.

6.8.4 *Benefits, Disadvantages, Limitations and Risks*

6.8.4.1 The primary benefit of using sand compaction piles relates to the increase in the mass strength and stiffness of the ground. This allows sloping seawalls to be constructed without requiring long waiting periods whilst the soft clay in the CMP consolidates and increases in strength. In addition, the installation of SCPs also leads to a reduction in the settlement of the ground. A secondary benefit is that the SCPs assist in relatively rapidly consolidating the clay remaining between the SCPs and help to make the ground more uniform.

6.8.4.2 For land formation over the CMPs there are two interlinked technical issues which need to be addressed. The first relates to the existing strength and consolidation state of the ground and whether it is possible to construct SCPs in such weak soil. The second relates to the ground heave caused by SCP installation and how this will affect the plant which can be used to install the piles.

6.8.4.3 Before discussing these two issues it is worth considering the construction methods used for construction of SCPs at Busan new port and Haneda Airport fourth runway. Both sites are underlain by thick deposits of soft clay, albeit that the clay is not as weak as that in the CMPs. At Busan new port the SCPs are installed through up to 40 m of soft clay with a design strength of approximately 15 to 50 kPa. The completed SCPs are 2 m diameter and the replacement ratios adopted vary between approximately 40% and 70%. The SCPs were installed from the existing seabed surface but were terminated approximately 5 m below the seabed surface and then the upper 5 m of clay was dredged prior to fill placement.

6.8.4.4 At Haneda Airport, the soft clay is approximately 20 m thick and the sand compaction piles were used beneath the sloping seawalls with a replacement ratio of approximately 30%. The SCPs were installed after a sand blanket had been placed over the soft clay. During installation of the SCPs the ground heaved by 2 to 3 m and the heaved ground was dredged from above the SCPs before further fill was placed for the seawall.

6.8.4.5 In preparing this report discussions have been held with various experts in Singapore, Japan and Korea on the design and construction of SCPs, with particular reference to the expected ground conditions in the CMPs and the overriding concern to minimize environmental impact and to avoid any requirement for dredging.

6.8.4.6 None of the experts were aware of any case where sand compaction piles have been installed in soft clay deposits with strength as low as that expected in the CMPs and also in clay layers which were still undergoing consolidation. Nevertheless, all considered that it would be possible to construct SCPs in this material provided it had attained the undrained strength profile represented by the design line in *Figure 6.1*.

6.8.4.7 The experts on SCP installation at Toa Corporation were very concerned about whether it would be possible to construct SCPs in the CMP if the strength profile was closer to the lower bound profile in *Figure 6.1*. Their concern with respect to a low strength profile is whether the clay could provide sufficient confining stress to allow the sand in the SCP to be compacted in a controlled manner. The Toa experts recommended that for the case of the clay in the CMP relatively small diameter SCPs should be adopted, for example starting with an installation diameter of 40 cm and enlarging to 70 cm. For a 30% replacement ratio, this would require the SCPs to be installed at approximately 1.1 m centers.

6.8.4.8 All experts consulted on the use of sand compaction piles recommended that a sand blanket be placed over the CMPs before sand compaction piles were installed. The consensus view was a minimum thickness of 1.5 m but that a thicker blanket would be preferable. The sand blanket is required to provide a confining stress particularly for construction of the SCP near the existing seabed surface.

6.8.4.9 It can be seen from the Busan and Haneda examples above that there is ground heave associated with SCP installation and dredging is usually carried out to remove the ground which has heaved. Based on the discussions with various experts it has been established that the total ground heave varies from approximately 60% to 80% of the installed volume of the SCP (one expert considered that an allowance for 100% ground heave should be assumed, albeit that the heave spreads out beyond the direct area over the SCPs.

6.8.4.10 Based on records taken during construction of the SCPs at Haneda it is understood that the total volume of SCPs installed was 6 Mm^3 and that the volume of ground heave directly above the SCPs was 2.5 Mm^3, although the total heave volume would have been greater than this. This is equivalent to a 40% ground heave directly above the SCPs. For preliminary design purposes a ground heave directly above the SCPs of 50% of the installed volume will be assumed. Based on a replacement ratio of 30% and a typical SCP length of 16 m the ground heave directly over the top of the SCPs is likely to be of the order of 2.5 m (i.e. 0.5 x 0.3 x 16).

6.8.4.11 Assuming that the typical seabed level in the CMPs is -5.5 mPD and allowing for a sand blanket with a minimum thickness of 1.5 m, the level of the sand blanket at the commencement of SCP installation would be – 4 mPD. A ground heave of 2.5 m associated with the installation of SCPs at a 30% replacement ratio would result in the sand blanket heaving to a level of approximately -1.5 mPD. Although the minimum tide level is 0.0 mPD (lowest astronomical tide) for approximately 95% of the time the sea level is above 0.5 mPD, only dropping below this level for a few hours a day during spring tides.

6.8.4.12 When considering the type of marine plant that is available to install sand compaction piles the draft of the vessel must be taken into account. In addition, a minimum water depth of approximately 1 m below the bottom of the barge should be allowed as a safety margin and for operational purposes. Based on discussions with Toa Corporation it is understood that the smallest barges have a draft of approximately 1.2 m but are only capable of installing two SCPs at a time. The bigger barges have a draft of approximately 3 m and are capable of installing 6 SCPs at a time.

6.8.4.13 Based on the above it would appear that if marine based plant is to be used to construct the SCPs then the smaller vessels will have to be used. However the available water depth will increase relatively rapidly if the ground is allowed to consolidate before constructing SCPs. This could be achieved if time was allowed after placing the sand blanket for vertical drains to accelerate the ground consolidation. Alternatively, land based plant could be adopted once the reclamation level has been raised above the high tide level.

6.8.4.14 When the Contract P398 ground investigation is completed further discussions should be held with the contractors who specialise in SCP installation.

6.9 Soil Mix Columns

6.9.1 *Process Description*

6.9.1.1 Soil mix columns are constructed by passing a rotating auger through the full thickness of a soil deposit as it injects and mixes a cementing agent into the soil. There are many variations on the process with respect to factors such as whether cement slurry or dry cement is injected, whether the mixing and injection is carried out as the auger is inserted or withdrawn. In addition, alternative additives such as lime or pfa can be used in place of the cement. The overall aim of the process is to form a column of treated material which has a significant cohesive strength and is stiffer than the surrounding soil.

6.9.1.2 For the purposes of this report the term Soil Mix Columns (SMC) will be used in a generic nature to refer to any form of construction of a cemented column in the soil by in situ mixing. For the particular ground conditions and constraints associated with land formation over the CMPs, the most likely form of SMC would be a column formed using injection and mixing of a cement or pfa slurry as the auger is withdrawn from the base of the layer to be treated to the ground surface.

6.9.1.3 With respect to land formation over the CMPs, Soil Mix Columns have two functions. First they can be used as load bearing columns, similar to piles, to carry loads through the full thickness of the soft clay in the CMP. Second they can be used to enhance the overall shear strength of the CMP clay to prevent deep seated slips through the soft clay.

6.9.1.4 The main difference between sand compaction piles and SMCs is that the SMC is typically stronger and stiffer than SCPs but SMCs do not assist in accelerating vertical drainage. In reclamation projects over soft ground SMCs are usually adopted where vertical or steeply sloping seawalls have to be constructed over thick soft clay deposits. Another difference is that sand compaction piles are usually installed as individual columns with soft soil between the SCPs whereas most applications using SMCs comprise either treated blocks or walls constructed using overlapping SMC columns.

6.9.2 *Fundamental Parameters and Requirements*

6.9.2.1 The strength and stiffness of SMC piles is controlled by a number of factors including the quantity and type of the stabilizing agent (i.e. cement, lime, pfa) the cement water ratio, the cement to soil ratio and the nature of the in situ soil. The design strength of the SMC column is usually specified in terms of the unconfined compressive strength (UCS) of the treated material and this can be varied by controlling the mix design. The design strength can range from as low as 200 kPa to 2 MPa or more. Design UCS values of 600 to 1000 kPa are commonly adopted.

6.9.2.2 The mix design is controlled by the in situ properties of the soil to be treated. Based on discussions with experts in Japan who regularly use SMC it is understood that typically the slurry cement mix has a water to cement ratio in the range 60% to 100% by weight. Where the SMC columns are required to support vertical caisson structures the weight of cement per m^3 of treated soil is usually in the range 150 to 250 kg/m^3. Lower cement contents associated with weaker SMC material can be used below sloping seawalls subject to a lower bound value of approximately 70 kg/m^3. Cement contents below this level yield no significant strength improvement. A recent design of a sloping sea wall in Singapore in marine clay which is simlar to the Hong Kong marine clay has adopted a water cement ratio of 80% and a cement content of 110 kg/m^3 of treated soil.

6.9.2.3 Yin (2001) presents strength and stiffness data for samples of Hong Kong marine clay mixed under laboratory conditions with various quantities of cement. The tests were carried out with cement contents of 5%, 10%, 15% and 20% defined as mass of dry cement divided by mass of soil. Yin (2001) concluded that a cement content of only 5% had only a very small effect on the soil strength but that a cement content of 10% showed a large gain in strength. The results of UCS tests on samples of marine clay with an initial water content of 80%, which is likely to be similar to the clay in the CMP, with a various cement contents is presented in *Figure 4* of Yin (2001) and at various water contents in *Figure 11* of Yin (2001). These figures are reproduced in *Figure 6.6* of this report. The UCS strength with 10% cement content was found to be approximately 1200 kPa (note UCS = 2 x deviator stress (q) in the figure). The density of clay with an initial moisture content of 80% is approximately 1500 kg/m^3 and a 10% cement ratio is equivalent to 150 kg/m^3 of treated soil.

6.9.2.4 Van Impe et al. (2006) report that cement mixed soil in the field using in situ mixing techniques is stronger than that mixed in the laboratory because of the potential for air entrainment in the laboratory. Based on the results presented in Yin (2001), it would be reasonable to adopt a cement content of 130 kg/m^3 for a preliminary assessment of the quantity of cement for treating soil and that a design UCS strength of 800 kPa, which is equivalent to an undrained strength of 400 kPa, could be adopted for the SMC material.

6.9.2.5 When designing ground improvement schemes below seawalls using SMC material it is normal practice to design with interlocking columns in the form of either contiguous walls or as soil blocks. The use of individual piles is not common because lateral loads acting on the SMC columns results in bending and tension and the SMC material has a relatively low tensile strength.

6.9.2.6 Based on discussions with Japanese experts it is understood that a commonly adopted layout for the support of seawalls is to use contiguous walls across the width of the base of the seawall. The walls are spaced at 3 SMC diameters apart and typically penetrate 4 metres into the underlying firm material. Short columns, between 3 to 5 m deep are used to treat the upper part of the soft layer between the contiguous walls to provide continuity of support to the overlying structure. Alternatively blocks of treated ground are used where each block is comprised of 4 or more interlocking SMC columns. Sketches showing typical layouts for SMC reinforcement of soft soil layers below seawalls are shown in *Figure 6.7*.

6.9.3 *Benefits, Disadvantages, Limitations and Risks*

6.9.3.1 The benefit of using SMC is to reinforce the material in the CMP to carry both the vertical load from the seawall core and to prevent deep seated slip movement. The technology for designing and constructing this type of reinforcement is well established. Discussions have been held with experts in the design and construction of reclamations over soft ground using this technique. The general consensus was that despite the potentially very low strength of the clay in the CMP the technique could be successfully carried out.

6.9.3.2 In this respect it is noted that Van Impe et al. (2006) report on the use of this method of ground improvement for a "*soft deposit of fine-grained material, the result of prolonged sedimentation and self weight consolidation process of dregs removal from the waterways within the harbour of Antwerp*". The paper includes field vane test results for the Antwerp harbour case which determined an undrained strength in a range from approximately 0.5 to 4 kPa in the upper few meters of the clay. These low values are likely to be similar to the material in the CMPs. At the time the paper was prepared, SMC columns were being used to improve the foundation for construction of a 27 m high sand embankment over an 8 m thickness of the very soft dredged material.

6.9.3.3 According to Van Impe et al. (2006), at the Antwerp site the clay was improved using only blast furnace cement with a water cement ratio of 0.8 and a binder dosage of 275 kg/m^3 (equivalent to approximately 150 kg of cement per m^3 of soil). UCS test results in the upper 6 m of the treated clay were in the range 0.8 to 4 MPa.

6.9.3.4 As noted above, the construction of a soil mix column involves injection and mixing of cement slurry into a soil by rotating a mixing tool as it is extracted from the base of the clay to the surface. As the mixing tool nears the surface the ground will become very disturbed and sea water will be drawn into the mix. The experts in this technique in Japan considered that for the likely ground conditions in the CMPs if the SMC column was constructed from the existing seabed then the top 0.5 m would have to be cut off by dredging.

6.9.3.5 All experts consulted strongly recommended that a sand blanket be placed on the seabed prior to the installation of the SMC columns. The primary purpose of the sand blanket was to allow the SMC improvement to be stopped within the sand blanket prior to reaching the surface of the fill so as to ensure a competent top to each column. In addition, a sand blanket would minimize the potential for cement slurry to be lost into the sea, would assist in cleaning the mixing tool as it was extracted from the ground and would provide a confining stress to the upper surface of the clay to assist in consolidating the upper surface which may be close to a slurry consistency.

6.9.3.6 A second consideration is the potential variability of the material in the CMP, particularly if the clay is still undergoing self weight consolidation. Variability of the ground will affect the properties of the SMC material. In this respect the construction of a sand blanket with vertical drains and allowing time for consolidation of the CMP material prior to improvement with SMC will assist in improving the overall consistency of the SMC as weaker zones should have had an opportunity to consolidate.

6.9.3.7 It is noted that because the material in the CMP is still undergoing self weight consolidation it will be necessary to take account of negative skin friction in the design of the improved ground.

6.9.3.8 Construction of SMC columns involves injecting cement slurry into the clay, which will cause displacement of the ground and some heave of the ground surface. Assuming that the treatment adopts 130 kg/m^3 of cement (density 3120 kg/m^3) at a water cement ratio of 80% then the volume of the injected slurry per m^3 of treated ground will be approximately 0.15 m^3.

6.9.3.9 For a preliminary assessment of the ground heave resulting from the use of SMC ground improvement assume that the treatment depth is from -6 mPD to -22 mPD for one third of the ground and from -6 mPD to -10 mPD for the remaining two thirds of the ground below the sea wall core. Total average volume injected per m^2 of ground area is 1.2 m^3 [i.e. (16x0.15 + 2x4x0.15)/3]. Allowing for some consolidation of the clay during construction and some lateral displacement of the ground, a ground heave of the order of 1 m would be a reasonable value to adopt for preliminary design purposes. Assuming that the SMC method is to be adopted for stabilizing the ground below the seawalls then the installation of the treated columns will be carried out using marine barges. Details of marine barges in Japan which can carry out this work are summarised in *Table 6.3*.

Table 6.3 Summary of Japanese SMC Marine Barge Capabilities

Nominal Treatment Area	Diameter of treated column	No of Interlocking columns	Layout of columns	Max depth of treatment	Draft of Vessels
m^2	M			m	m
5.7	1 or 1.6	8 or 4	4x2 or 2x2	60 to 70	2.7 to 3.3
4.6	1.3	4	2x2	40 to 50	1.5 to 2.9
2.2	1.2	2	1x2	30 to 40	1.3 to 2.3

6.9.3.10 The marine barges require a minimum clear water depth of 1.0 m for operational purposes. Assuming that the top level of the sand blanket at the commencement of treatment is -4 mPD, a maximum heave during construction of 1 m will raise the seabed level to -3 mPD. Assuming a minimum sea level of +0.5 m, barges with a draft of 2.5 m or less will be able to carry out the SMC treatment. Based on the details summarised in Table 7.3, the majority of the small and medium sized barges currently operating in Japan could be used over the CMPs.

6.9.3.11 When the Contract P398 ground investigation is completed further discussions should be held with the contractors who specialise in SMC installation.

6.10 Summary of Non Structural Land Formation Options

6.10.1 *Background*

6.10.1.1 The major issues to be considered for land formation over a soft clay deposit are associated with stability of the seawall and internal reclamation front during construction and accelerating the time required for consolidation of the clay in order to achieve acceptable residual total and differential settlement characteristics. Land formation over the CMPs is further complicated by two additional factors. First the requirement to ensure minimum environmental impact, which in practical terms means no dredging of the clay, and minimizing risk of slope stability failures. Second the material in the mud pits is dredged marine clay which will be extremely weak and may still be undergoing consolidation.

6.10.1.2 At the time of preparing the Initial Options report, other than a limited amount of testing of the capping layer carried out shortly its placement, there had been no investigation into the properties of the material dumped in the CMPs. Accordingly it was necessary to make an assessment of the likely properties based on the limited information from the capping investigation, knowledge of the undisturbed properties of the marine clay prior to dredging and experience in Singapore of the properties of similar dredged materials. Subsequently a ground investigation has been carried out in the CMPs which, although not complete when the Final Options report was prepared, had obtained sufficient information to confirm the likely properties of the material in the CMPs

6.10.1.3 The critical geotechnical design parameters are the initial undrained strength profile for the clay, the consolidation parameters controlling magnitude and rate of settlement and the relationship between consolidation stress, void ratio and undrained strength. Likely ranges for these parameters are discussed in *Sections 2.6 and 2.7* of this report.

6.10.1.4 Various techniques for constructing reclamations over soft ground are discussed in *Sections 6.1 to 6.9* along with a preliminary assessment of the applicability of each technique with reference to the likely ground conditions in the CMPs. This section presents a recommendation for the most appropriate combination of techniques for land formation over the CMPs based on information available to date. When constructing reclamations it is normal practice to construct the seawalls in advance of the general reclamation and this aspect is discussed before considering general land formation methods and land formation methods for settlement sensitive areas (i.e. the runway).

6.10.2 *Construction of Seawalls over the CMPs*

6.10.2.1 This section considers the construction of sloping seawalls over the CMPs. It is noted that the general principles described for construction of a sloping seawall are also appropriate for vertical seawalls, albeit that the geometry of ground treatment work would have to be adjusted to take account of the differing loading from a vertical seawall.

6.10.2.2 As a result of stability and settlement related problems, sloping seawalls would normally be constructed over trenches dredged to the base of the soft in situ marine deposits. However, in the present situation as the soft material must be left in place at the CMPs it will be necessary to strengthen the CMP clay instead of dredging it. Although theoretically it would be possible to adopt a slow stage construction technique, making use of the gain in strength of the clay as it consolidates to construct the seawall, in practice the time required to gain the necessary strength would be excessive, particularly as the seawalls must be constructed in advance of the main reclamation. Perhaps more importantly, there have been a number of failures during construction of reclamations constructed over soft clay using the stage construction method and this approach is considered to be unsuitable for construction of the seawalls over the CMPs in the light of the need to minimise environmental impact.

6.10.2.3 The seawalls will be constructed in open water and the construction will be affected by tidal currents and waves associated with storms. It is assumed that the seawall core will be rockfill and that the seawall slope will require rock armouring. The stability of the seawall during construction is affected by the slope of the fill as it is placed. The water depth is relatively shallow and it is likely that the seawall core material will be placed by end tipping which will form a temporary slope at an angle of approximately 1V:1H. The permanent slopes and rock armouring are then placed and shaped using grabs. Alternatively it would be possible to place all the fill by grab or using a conveyor placement method from a barge. These methods would allow more control on the temporary slopes of the fill during placement. For the purposes of this report the more onerous end tipping method has been considered.

6.10.2.4 *Figure 6.8* presents a section through a sloping seawall constructed over a CMP using the methods proposed in the following sections.

6.10.3 *Sand Blanket and Vertical Drainage System*

6.10.3.1 Irrespective of whether the land formation is associated with seawall construction or general reclamation, the first stage of any land formation over the CMPs is recommended to be construction of a sand blanket and installation of a vertical drainage system through the full thickness of the material in the CMP. There are a number of reasons for proposing the construction of a sand blanket and vertical drainage system including containing the underlying clay during follow on construction work, consolidating soft spots in the surface of the capping layer and within the underlying material in the CMP and providing a confining stress over the CMP to assist follow on ground treatment work especially close to the surface. Further details are given in *Section 6.3*.

6.10.3.2 The sand blanket will need to be placed using a speed controlled spreader pontoon or similar, which can accurately place thin (less than 0.2 m) layers of sand over the seabed. The blanket will have to be constructed in thin layers and in such a manner that the overall slope of the leading edge is relatively shallow, typically less than 1V to 10H, see *Section 6.3.*

6.10.3.3 The overall aim of this work is to place a sand blanket in layers with a minimum thickness of 1.5 m and preferably thicker. The thickness of the layer will be controlled by the water depth and the draft of the vessels which will be used to carry out ground improvement work. As described in the previous sections, the calculation of minimum water depth needs to consider the following factors:

a) An allowance for the ground settlement which will occur as a result of the placement of the sand blanket and installation of a vertical drainage system (0.5 to 1 m).
b) The draft of the vessels used for follow on ground treatment (ranging from approximately 1.2 to 3.5 m, see *Sections 6.8.6.7* and 6*.8.6.13*).
c) A minimum water depth below the vessel draft, which for operational reasons should be at least 1 m.
d) The ground heave during installation of the follow on ground treatment work, see *Sections 6.9.3.9* and *6.9.3.10*.

6.10.3.4 The requirement for a vertical drainage system beneath the seawalls will need to be re-assessed after the ground investigation associated with the detailed design has been carried out in the CMPs. If, as found in the investigation carried out to date, the clay material is shown to be normally consolidated with no excess pore pressures and the capping clay has a reasonable strength, then vertical drains may not be required at locations where other forms of ground treatment (SMC or SCP) are proposed. If required, the vertical drainage system is likely to comprise either PVDs at a spacing of approximately 1 m or sand drains at a spacing of approximately 1.5 m. The selection of vertical drain type will be controlled by a number of factors including cost, availability of plant to install the drains, time allowed in the program to install the drains and the nature of any follow on ground treatment work.

6.10.3.5 As discussed in this section, it is proposed that the seawall core be supported over the CMP by improving the CMP material using the soil mix columns stabilized with cement or pfa. The construction of SMC columns is achieved using a rotating mixing tool and as such PVDs are not recommended in the areas where the SMC ground treatment technique is to be used because the PVD will tangle with the mixing tool.

6.10.4 *Ground Treatment Beneath a Seawall with Soil Mix Columns*

6.10.4.1 Stability analyses have been carried out to assess the stability of fill slopes placed on the sand blanket assuming that a vertical drainage system has achieved at least 90% consolidation under the load from the sand blanket. These analyses indicate that the maximum fill slope that can be placed on the sand blanket is of the order of 1V:6H. As the final seawall profile is likely to be between 1V:3H to 1V:2H and temporary fill slopes may be as steep as 1V:1H it is clear that some additional ground treatment of the CMP clay is required to ensure stability.

6.10.4.2 *Sections 6.8 and 6.9* describe the use of sand compaction piles (SCP) and Soil Mix Columns (SMC) which are two well established methods that are used to treat thick deposits of soft clay. Both SMC ground treatment and installation of SCPs for the seawall would have to be carried out from marine barges working over the top of the sand blanket.

6.10.4.3 Ground treatment using soil mix columns (often referred to as deep cement mixing) has been widely used around the world and particularly in Japan to treat soft ground below sloping seawalls and caisson structures. As described in *Section 6.9*, the overall process involves the in situ construction of columns of the CMP clay mixed with cement or PFA slurry using a rotating mixing tool which simultaneously injects and mixes the slurry as it is raised through the clay. The top of the SMC columns would terminate in the sand blanket. The SMC would be extended some distance below the base of the CMP to transfer the load from the seawall into the underlying more competent material.

6.10.4.4 *Section 6.9* presents details of the typical design mixes in terms of cement content (130 kg/m3) and water cement ratio of the slurry (80%) which is likely to be appropriate for the CMP material. *Figure 6.7* presents typical details of the layout of the SMC ground treatment. Although the final layout of the SMC treatment will depend on aspects such as the depth and thickness of the CMP material, the strength of the ground underlying the CMP and the seawall slope, the overall layout presented as type 3.6.6 in *Figure 6.7* would be appropriate for supporting the seawall core.

6.10.4.5 This configuration adopts full depth rows of contiguous (i.e. interlocking) SMC columns across the width of the embankment to transfer the load from the seawall to the underlying more competent material. The contiguous walls are formed with a clear spacing between the walls of two times the thickness of the wall. The contiguous walls would penetrate approximately 3 m into the underlying more competent ground and approximately 0.5 m into the underside of the sand blanket. Shorter SMC columns, approximately 4 m long, would be used to fill the gap between the contiguous walls so as to provide a continuous strengthened zone directly below the core of the seawall. The purpose of the continuous strengthened zone is to minimize the potential for localized failure of the capping layer during rock fill placement and to span between the contiguous walls to allow for the potential for continuing self weight consolidation of the CMP material.

6.10.4.6 The proposed extent of the SMC treatment across the width of the seawall is shown in *Figure 6.8*. For preliminary design purposes it is has been assumed that the seawall core will be placed by end tipping from a platform level of +3 mPD and with a width at this level of approximately 8 m. The SMC treatment has been extended approximately 5 m beyond the toe of the inner seawall core placed by end tipping at an assumed slope of 1V to 1H. The 5 m additional width is to allow for load spread though the sand blanket and the potential for shallower side slopes during end tipping of the seawall core material.

6.10.4.7 Stability analyses have been carried out using a rock fill friction angle of 40 degrees and a design UCS strength for the treated clay of 600 kPa and 400 kPa. No allowance was made for ground treatment outside the zone to be improved by SMC. The analyses indicate the following:

a) The FoS during placement of the inner seawall core will be in excess of 1.6 for the assumed UCS strength of both 600 kPa and 400 kPa.
b) The FoS for the completed seawall prior to filling for the main reclamation will be controlled by shallow slip circles through the embankment slope which pass outside the SMC zone and for which a FoS of approximately 1.35 is predicted.
c) The FoS for the completed seawall after construction of the reclaimed land behind the wall will be approximately 1.35 for deep slip circles which pass through the reclamation and the SMC treated zone for the case where the SMC has a UCS design strength of only 400 kPa. This assumes no improvement of the CMP clay beneath the reclamation.

6.10.4.8 As discussed in *Section 6.9*, the water depth after placement of a sand blanket is considered to be sufficient for marine barges to install the proposed SMC treatment. As also discussed in *Section 6.9*, this form of treatment has been adopted for improving similar very weak dredged clay for reclamation work in Antwerp (Van Impe et al., 2006). On the basis of the above, it is considered that the use of SMC ground treatment is a feasible method of improving the CMP material to allow the seawall to be constructed over the CMPs without any requirement for dredging the CMP material.

6.10.4.9 The CMP treatment provides a relatively hard zone beneath the seawalls and the seawall settlement will be controlled by the compression of the more competent ground beneath the base of the CMPs, which might be of the order of 1 m. This is relatively small in comparison to the predicted total ground settlement of the order of 4 m beneath the remainder of the reclamation. There is therefore a potential for both differential settlement at the edge of the zone treated with SMC and also for the SMC area attracting additional load as the adjacent area settles. Additional treatment of the CMP material directly adjacent to the zone treated using SMC to smoothen the transition is therefore required and this is discussed in the following section.

6.10.5 *Ground Treatment Beneath a Seawall with Sand Compaction Piles*

6.10.5.1 Sand Compaction Piles have been adopted widely in Japan and Korea to treat thick deposits of soft clay prior to construction of both sloping seawalls and vertical caisson structures. The use of SCPs therefore needs to be considered as an alternative to SMC treatment.

6.10.5.2 As discussed in *Section 6.8,* the construction of SCPs causes significant heave of the ground surface. A sand replacement ratio of approximately 30% could result in a ground heave of the order of 2.5 m. Unless the heaved surface was dredged to maintain a reasonable water depth, only relatively shallow draft barges could be used to install the SCPs. Without dredging, a replacement ratio in excess of approximately 30% would not be possible using marine based plant.

6.10.5.3 A 30% SCP replacement ratio could be used in combination with a relatively shallow angle seawall of the order of 1V:3H. However there would be difficulties during construction if the temporary fill slopes became too steep. In addition, experts in Japan have expressed some concern about the potential difficulties of constructing SCPs in the very weak CMP clay and have suggested using small diameter SCPs. There does not appear to be any experience of having constructed SCPs in clay with an undrained strength profile as weak as that which is likely in the CMPs. Further review of this aspect should be carried out when all the results of the Contract P398 ground investigation are available.

6.10.5.4 On the basis of the ground heave associated with SCP installation, the requirement for dredging and constraint on temporary construction slopes, the use of SCPs to treat the CMP clay directly beneath the seawall core is not recommended, unless subsequent ground investigations show large zones of stronger and or thinner clay to be present.

6.10.5.5 Nevertheless, as discussed in *Section 6.10.1* there is a potential for significant differential settlement between the general reclamation and a seawall supported on SMC treated ground and also of load transfer from the general reclamation to the SMC treated ground as the general reclamation settles. The use of SCPs over the transition zone between the seawall and general reclamation is therefore proposed, both to control differential settlement and to prevent overloading the SMC treated zone.

6.10.5.6 *Figure 6.8* includes a zone of ground on both sides of the seawall which includes treatment with sand compaction piles. At the present time the a treatment width of approximately 15 m has been adopted, which is similar to the thickness of the fill overlying the SCP and should be sufficient to prevent significant load transfer onto the SMC zone and to enhance the stability of the ground during construction of the seawall. It is envisaged that the replacement ratio can be varied across the zone. More detailed analysis of the extent of the transition zone can be carried out when the detailed ground investigation in the CMPs has been carried out.

6.10.6 *Completion of the Seawall*

6.10.6.1 Assuming that the seawall core is supported on a zone of SMC treated ground and that sand compaction piles have been used in the transition zones on both sides of the seawall then construction of a sloping seawall can proceed in the usual manner. On the seaward side of the wall this would involve placement of rock armour using grabs and a scour protection mattress on the seabed. The inclusion of a zone of sand compaction piles in front of the SMC treated zone on the seaward side of the seawall will also assist in preventing scour development in front of the seawall.

6.10.6.2 On the landward side of the wall, filter zones or a geotextile filter cloth may be required if the reclamation fill material is finer than the rockfill core material. The ground on the landward side of the wall will need to be surcharged to accelerate settlement of the ground. The adoption of a transition zone treated with sand compaction piles will assist in both spreading the load due to this surcharge and compensating for differential settlement. Assuming that SMC treatment has been carried out under the core of the seawall, there would be no requirement for direct surcharging of the seawall.

6.10.7 *Land Formation at General Reclamation Areas*

6.10.7.1 As discussed in *Section 6.10.1.1*, the first stage of any land formation over the CMPs is recommended to be construction of a sand blanket and installation of a vertical drainage system through the full thickness of the material in the CMP. Following placement of a sand blanket, the two controlling factors are the slope of the reclamation front required to maintain stability and the time required to achieve acceptable settlement characteristics under surcharge loading. In this respect, the following comments are made:

a) During the initial stage of fill placement, the slope of the reclamation front must be maintained at less than approximately 1V to 10H, see *Section 6.3*.
b) Fill placement across the full width of the reclamation using speed controlled spreader pontoons or sand placement barges with spreader arms could be used combined with careful hydrographic survey techniques to achieve the required slope gradient.
c) As consolidation takes place under the weight of the fill that has been placed, the slope of the reclamation front can be increased. For example, after the consolidation associated with the placement of 3 m of fill on the seabed surface is 90% complete, the slope face gradient can be increased to 1V to 6H. However, the rate of filling to achieve a steeper slope face gradient would be relatively slow and would affect the programme of the works.
d) Estimates of the time taken to achieve consolidation of the CMP material can be made using *Figure 6.2* and *Figure 6.3* assuming vertical drainage using PVD or sand drains respectively.

6.10.7.2 The CMP parameters presented in *Section 2.7* are suitable only for preliminary assessment of land formation techniques and should not be used for detailed design calculations. As such calculations relating the slope of the reclamation front to the degree of consolidation achieved at any filling stage in order to develop a detailed stage construction sequence are not worthwhile. It is suffice to note that a stage construction sequence could be developed which would ensure stability during fill placement. The most important aspect of a staged construction process would be the requirement for strict on site control of the filling process and detailed instrumentation to monitor ground settlement and pore pressure dissipation.

6.10.7.3 It is worth noting that reclamations being constructed for the airport extensions at both Kansai airport and Haneda airport in Japan have been constructed over thick deposits of soft clay and that the general reclamation areas have had no special ground treatment other than the installation of sand drains to accelerate consolidation. The only major difference between land formation for these airports and land formation over the CMPs is that the starting strength and consolidation state of the clay in the CMPs is weaker and softer. However after the ground consolidation associated with the first few meters of fill over the CMPs has taken place, the overall behaviour will be similar.

6.10.7.4 Accordingly, provided that the first few meters of fill can be placed and time is allowed for some consolidation, there should be no reason why areas of general reclamation cannot be constructed using standard drained stage construction methods. A caveat to this statement is if the detailed ground investigation identifies that the material in the CMPs is still locally significantly under-consolidated, in which case stage construction using fill placement which relies on shallow slope angles and time for the underlying clay to consolidate and gain strength may not be possible in the time available time scale. The most likely viable alternative to gain sufficient early strength for standard drained construction methods to be adopted would be to use vacuum consolidation. The difficulties associated with vacuum consolidation underwater are described in *Section 6.6* and a potential method using "BeauDrains" is also discussed.

6.10.7.5 Assuming that the reclamation fill can be placed in a controlled manner so as to ensure stability during fill placement then the next issue to consider is achieving adequate consolidation of the clay such that long term total and differential settlement is acceptable. In view of the settlement related problems which have been experienced on various "drained" reclamations in Hong Kong, it is recommended that for the present assessment of land formation options, the recommendation made in Endicott (2001) to achieve an OCR after surcharging of 1.2 under working load conditions be adopted for design of surcharging work on general areas of the reclamation.

6.10.7.6 To put this in perspective, the typical level of the top of the CMPs is presently -5.5 mPD and after completion of the land formation the level will have settled by approximately 4 m to -9.5 mPD. The base of the CMP material is initially at a level of approximately -20 mPD and the final level will have settled by approximately one meter to -21 mPD. Assuming a final reclamation level of +7 mPD, an allowance of 10 kPa for live loading, a ground water table of +1.3 mPD (mean sea level), an average fill density of 20 kN/m3 and an average density for the CMP material after consolidation of 17 kN/m3, then the effective stress at the top and base of the clay will be approximately 230 kPa (10x(9.5+1.3) + 20x(7-1.3)) and 310 kPa respectively. An OCR of 1.2 would require an overstress due to surcharging of approximately 45 kPa to 60 kPa. For preliminary design purposes an overstress of 60 kPa should be adopted.

6.10.7.7 This level of overstress by surcharging could be achieved by, for example, the placement of a 4 m high surcharge and achievement of 80% consolidation under the surcharge or the use of deep well dewatering within the fill, as described in *Section 6.7*, to draw the water table down by 8 m and also maintaining the drawdown until 80% consolidation had been achieved.

6.10.7.8 Although not specifically related directly to land formation over the CMPs it is worth noting that if sand fill is used as the reclamation material then, in line with the practice at the existing airport, the sand fill will need to be vibro-compacted in areas where structures are to be constructed. Because the general reclamation will be constructed over a thick deposit of soft clay, all but the smallest structures will need to be founded on piles and there are likely to be excavations required for basement structures, pumping stations etc., which might require sheet pile walls to be installed. The vibrations associated with the installation of piles for buildings and sheet piles for excavations can result in shakedown settlements of sand fill placed in a loose condition underwater.

6.10.8 *Land Formation at Settlement Sensitive Areas*

6.10.8.1 The previous section describes land formation methods which are suitable for general areas of the reclamation. In areas which are particularly sensitive to long term total and, more importantly, differential settlement, additional ground treatment is likely to be required. Settlement sensitive areas would include the area of the runway and rapid exit taxiways and the alignment of main stormwater drainage pipes. It is noted that the differential settlement criterion for a runway is significantly more onerous than for normal facilities constructed on reclaimed land. Accordingly, there are two main reasons why additional ground treatment is proposed.

a) Construction of a runway over a thick deposit of dredged clay has not been attempted before and as such there is no experience of the likely nature of the variability and long term settlement characteristics of the ground. Long term total and differential settlement related problems caused by incomplete consolidation of the CMP material would represent a significant risk and it would be extremely difficult to correct the problem without closing the runway.

b) Experience of the settlement characteristics of the northern runway obtained from close grid surveys (10 m spacing on 100 by 100 m grids) demonstrated significant differential settlements were being caused by the natural variability of both the fill and the underlying stratum and this required additional ground treatment using a rolling surcharge prior to construction of the runway. At the time of construction of the existing airport reclamation the fill and underlying Chek Lap Kok deposits were considered to be reasonably uniform and the differential settlements over relatively short distances were not expected. In comparison, it is known that the material placed in the CMPs was derived from various dredging contracts in Hong Kong and the ground investigation carried out to date has already demonstrated that the CMP clay is variable and as such it is likely that an engineered solution will be required to limit this variability.

6.10.8.2 In addition to the surcharging required for land formation at general areas of the reclamation it is proposed that additional ground treatment using either SMC treatment or sand compaction piles be carried out beneath settlement sensitive areas. The three most appropriate measures would be as follows:

<u>Ground Treatment Using Additional Surcharge</u>

6.10.8.3 Achieving an OCR of 1.2 in the CMP material is unlikely to represent a sufficient safety margin to reduce differential settlement to within the tolerance required for a runway. However, when a detailed ground investigation has been carried out to investigate the variation in the properties of the CMP material then it should be possible to determine a degree of surcharging which will satisfy the required differential settlement criteria. For preliminary design purposes it would be appropriate to adopt an OCR of the order of 1.5 after surcharge removal. The surcharging could comprise a combination of fill dewatering and surcharge mounds.

<u>Ground Treatment Using SMC</u>

6.10.8.4 Once a sand blanket has been constructed then stabilization of the CMP material using the SMC technique described in *Section 6.10.1* for construction of the seawalls could be adopted for construction of the settlement sensitive reclamation areas over the CMPs. For example, the use of SMC stabilization with walls to transfer the reclamation load down to the more competent ground below the CMP material combined with shallow SMC treatment of the remainder of the CMP to provide a platform on which to place fill material would allow land formation over the CMPs to proceed in a similar manner to a normal dredged reclamation. This would allow great flexibility in the choice of land formation but it is likely to be costly and there would be significant differential settlement problems at the interface between treated areas and non treated areas.

<u>Ground Treatment Using SCP and Surcharging</u>

6.10.8.5 Sand compaction piles assist in both reducing total settlement and accelerating the rate of settlement. Both of these characteristics are of benefit in settlement sensitive areas. As discussed in *Section 6.8,* after placement of a sand blanket, it would be possible to install sand compaction piles up to a maximum replacement ratio of between approximately 30% to 40% using the smaller marine barges. Surcharging would still be proposed to be carried out to minimize the magnitude of total and differential settlement.

6.10.8.6 If sand compaction piles are to be adopted then for preliminary design purposes the recommendations of the Japanese experts to use relatively small diameter sand compaction piles at close centres would be recommended, see *Section 6.8*. One of the advantages of using sand compaction piles is that the replacement ratio can be varied to control differential settlements between areas with and without treatment.

6.10.9 *Land Formation over the CMPs and In Situ Marine Clay*

6.10.9.1 It is noted that the proposed land formation methods described in *Sections 6.10.1 to 6.10.3* are based on the construction over the full depth of the CMPs and it is recognized that the CMPs were constructed in pits which had been dredged primarily in the soft in situ upper marine clay and that the reclaimed land will extend over both the CMPs and the adjacent undredged in situ marine clay. In this respect it is noted that the methods which are proposed for land formation over the

CMPs are equally applicable to construction over the in situ marine clay and across the pit boundaries. The primary differences between the CMP material and the in situ marine clay are as follows:

a) Although soft, the in situ marine clay is stronger and initially stiffer than the material in the CMPs. The undrained strength is typically of the order of 15 to 30 kPa and the pre-consolidation pressure is typically in the range 50 to 100 kPa.

b) The higher initial undrained strength allows the thickness and slope of the leading edge of the initial sand blanket to be greater over the in situ marine clay than over the CMP. Nevertheless the thickness and height still needs to be controlled to prevent failure.

c) During the initial filling (i.e. loading) stages when the applied load results in a ground stress which is less than the pre-consolidation pressure, consolidation related settlements will be significantly less for land constructed over the in situ clay than over the CMP. However for the increment of load which exceeds the pre-consolidation stress, the associated incremental magnitude of consolidation settlement will be of a similar order of magnitude.

d) In practice the difference in pre-consolidation pressure between the in situ marine clay and the clay in the CMPs will result in the reclamation over the CMPs settling by 1 to 2 m more than the reclamation over the in situ marine clay. Nevertheless total settlements in the range 2 to 3 m can be expected for land formation over the in situ marine clay.

e) Both the in situ marine clay and the material in the CMP will require vertical drains to accelerate consolidation settlements. The in situ marine clay is usually structured such that its horizontal permeability is higher than its vertical permeability but this structure will have been largely destroyed in the CMP material and the horizontal permeability will be similar to the vertical permeability. As a result the vertical drain spacing can be greater for treating the in situ marine clay than adopted for treating the CMP material.

f) Despite the fact that the initial undrained strength of the in situ marine clay is greater than material in the CMPs, it will still be necessary to provide additional ground improvement methods such as sand compaction piles or SMC treatment for construction of seawalls over the in situ clay. The methods proposed for constructing seawalls over the CMPs can be adopted for construction of the seawalls over the in situ marine clay adjacent to the CMPs.

g) If a fully dredged seawall construction is more cost effective for the seawalls over the in situ marine clay then a transition zone will be required but this will have to be sufficiently far from the CMPs so that there is no risk of failure back into the CMPs.

h) For the stresses associated with the full thickness of the reclamation (i.e. up to +7 mPD) the consolidation stiffness of the in situ marine clay and the CMP material will be similar, as will their secondary compression characteristics. The ground treatment techniques proposed for land formation over the CMPs at settlement sensitive areas of the reclamation, such as the runway, are equally applicable to similar land formation over the in situ marine clay. Assuming that all areas are treated in a similar manner, then differential settlement problems between these different areas should not be a problem.

i) The secondary compression characteristics of the in situ marine clay under the final reclamation load will be similar to the material in the CMPs.

6.10.9.2 Accordingly, assuming that the overall construction techniques recommended for land formation over the CMPs are also applied for land formation over the in situ marine clay and that the same performance criteria are applied, there should not be any further need to consider long term total and differential problems between these different areas. In particular surcharging to achieve an OCR of 1.2 after surcharge removal should be adopted everywhere on the reclamation. The OCR of 1.2 is based on the design working load for the reclamation surface and must take account of the proposed final formation level.

6.11 Piled Structures

6.11.1 *Process Description*

6.11.1.1 Piled structures have been adopted previously to support runways, taxiways and apron areas. However, the high construction cost associated with this type of structures has generally prohibited their use. Of particular relevance are the airports in Macau and Haneda and Japan.

6.11.1.2 In Macau the taxiways linking the main runway to the apron areas comprise piled structures constructed in relatively shallow water. The taxiways and parts of the apron areas comprise reinforced concrete deck structures supported by driven concrete piles founded in the alluvial deposits beneath relatively shallow water. Such techniques, adopting driven concrete piles would not be appropriate at Chek Lap Kok where the seabed lies some 12m below finished deck level. Driven piles of the prestressed precast type adopted in Macau would not have an adequate stiffness necessary to span between the seabed and underside of the deck structure.

6.11.1.3 Of more relevance to this project is the construction of a new piled runway at Haneda in Japan where construction is currently under way on the 4^{th} runway. At Haneda the piled runway deck structure is supported on 1.6 m diameter driven steel tubular piles. Piles of this diameter have sufficient sectional strength to support the loading from the deck structure without buckling or the need for complex bracing arrangements.

6.11.1.4 The mild steel piles have been driven through the soft clay deposits at the site and are founded by end bearing in firmer sands beneath the silty clays comprising the seabed.

6.11.1.5 The structure is being completed using prefabricated deck structures comprising structural steel frames fabricated from an array of primary pre-painted steel beams connected to pre-fabricated sleeves which are lowered on to the pre-driven piles. Each unit has a plan area of 63 m by 45 m and comprises the main steel frame constructed over 6 stainless steel sleeves which slide over the supporting driven piles. The deck units comprise pre-cast concrete panels some 450mm in thickness with infill stitching concrete between deck panels which key with the steel beams by means of shear studs. Reinforcement intensities in the precast panels are set at about 0.9% per face. Stainless steel pile sleeves bridging between the permanently submerged mild steel piles and the underside of the deck structure have been adopted to achieve the required design life of 120 years. ie the vulnerable area of the piles located within the splash zone has been protected through the use of stainless steel. The prefabricated steel beam network is completed with shear stud connectors to ensure structural composite action and its inherent structural efficiency. High strength concrete incorporating microsilica has been adopted to enhance durability as well as strength.

6.11.2 *Fundamental Parameters and Requirements*

6.11.2.1 Similar structures to those adopted in Haneda could be constructed at the proposed reclamation site to support the runways and taxiways, although the high construction cost associated with piled structures makes this technique questionable for all but the most settlement sensitive elements of the construction ie the runway and taxiways.

6.11.2.2 Driven steel piles are probably to be favoured at the proposed Chek Lap Kok site for reasons of expediency and cost. Driven steel tubular piles of 1600 mm diameter or 1800 mm diameter would be adopted with pile wall thickness of either 16 or 20mm. Piles would be driven to effective refusal in the alluvial sands and clays lying beneath the contaminated mud pits. Similar piles have been driven previously for the support of CLP Castle Peak Power Station Jetty and more recently for the PAFF jetty at area 38 in Tuen Mun. The adoption of the stainless steel sleeve will be reliable in terms of achieving the required 120 year design life but may be prohibitively expensive. It may be more appropriate to adopt polyester powder coating systems and a cathodic protection system in Hong Kong.

6.11.2.3 Concrete piles if adopted would be of the pre-bored cast in situ type. These piles would be cast inside pre-driven steel casings. Permanent steel liners would be required to contain the concrete as the temporary casings are withdrawn, this being necessary to contain the concrete through the very soft marine deposits within the contaminated mud pits. The adoption of steel piles, provided these were fitted with closed steel shoes, would avoid the need for excavation and disposal of material which might otherwise remain trapped in the pile shafts as they were driven. The adoption of concrete piles would require the removal and disposal of contaminated mud from within the temporary casings. Concrete piles would also need to be founded at a deeper level than driven steel piles to meet the exacting and conservative standards for the founding of such piles by the Buildings Department. Concrete piles would necessarily be founded approximately 10 – 15m deeper than the equivalent driven steel piles for the same design capacity. It must be concluded that driven steel piles would be a superior alternative for the relatively light loading conditions applicable to the runway when considered in relation to the equivalent concrete piles.

6.11.2.4 The deck arrangement in Hong Kong will probably be most cost effective if constructed in a very similar manner to that at Haneda. It will be necessary to make provision to make use of high performance concrete and it will be necessary to make provision for the installation of cathodic protection systems if the design life of 120 years is to be achieved. This is however standard practice on modern marine structures and for all submerged MTR facilities at the present time.

6.11.3 ***Benefits***

6.11.3.1 The adoption of a steel piled solution will be beneficial from a number of points of view. These can be summarised as follows :-

- No dredging of marine deposits will be required;
- The disposal of excavated marine deposits can be avoided;
- Settlement related issues can be avoided;
- Prefabrication techniques can be adopted to reduce the overall constriction time;
- Most environmental issues can be avoided with the exception of piling noise; and
- The effective are of the site to be treated can be reduced with only settlement sensitive areas of the site being treated.

6.11.4 ***Disadvantages***

6.11.4.1 Disadvantages associated with the adoption of a piled solution can be summarised as follows:-

- Piled structures are inherently more costly and less robust than simple reclamation;
- Pile driving will generate noise impacts which may be considered unacceptable without significant mitigation;
- Corrosion and durability issues are significant and will require the adoption of either stainless steel as at Haneda or the adoption of cathodic protection systems;
- The adoption of a precast decking system will only be able to treat a selected and critical areas of the site typically that occupied by the runway and taxiways; and
- Differential settlement issues will become predominant in areas where the pile structures interface with areas of the site such as apron areas which are far more cost effectively addressed through reclamation.

6.11.5 *Limitations*

6.11.5.1 Piling systems have a number of inherent limitations associated with the pile driving or generation technology adopted. These can be summarised as follows:-

- Driven piles have a limited capacity associated with the ability for the pile to be driven to an acceptable set. The proposed hollow tubular piles will probably be restricted to penetration only a few metres into the alluvial sands and clays at the site. This will limit the ability of the piles to resist uplift and lateral loading;
- Current technology places constraints on the durability of the structure. Achieving the required design life of 120 years may be difficult to achieve in practice;
- Maintenance will be a continual issue particularly once the piling and structural steelwork components of any system reach an age compatible with 50 or more years; and
- The extent of any piled structure will be limited as a consequence of high construction cost. Limited coverage of the site will restrict emergency vehicle access and flexibility to make additions and alterations to the mid field infrastructure in future years.
- Long term but modest settlement must be considered if piles are driven only a few meters into alluvial strata.

6.11.6 *Environmental Implications*

6.11.6.1 The adoption of a piled solution, provided driven steel piles are adopted, can avoid many of the environmental issues associated with reclamation works and the need to stabilise or treat the contaminated sediments. Piling noise during driving will require mitigation while the adoption of concrete piles will necessarily require the removal and disposal of contaminated materials which will need to be removed from the pile casings before the concrete can be cast. In conclusion, many of the environmental issues associated with addressing the contaminated mud can be avoided.

6.11.7 *Implementation Issues*

6.11.7.1 There are few implementation issues associated with the adoption of driven tubular steel piles and the construction of conventional composite steel and concrete deck construction. The technology is well tried and tested and the requirement for sophisticated and specialist ground treatment equipment can be avoided. Strict quality control will be required over the construction to ensure the construction meets exacting durability requirements. Again, this is well tried and tested technology which should prove to be feasible in practice. The construction of such a large number of piles and the necessary prefabrication yards will place considerable demands on local resource. However the equipment necessary, with the exception of high capacity marine cranes to lift precast units into position should not be difficult to mobilise.

6.12 Semi Buoyant Construction

6.12.1 *Process Description*

6.12.1.1 It should be emphasized that semi buoyant construction is not a technique in itself for the construction over the mud pits. The techniques, involving very light weight void filling, can however be used in construction along with standard reclamation and filling techniques to reduce the loading induced by filling and, as a consequence the amount of settlement anticipated.

6.12.1.2 There are predominantly two types of tried and tested very lightweight filling which can be incorporated into the reclamation fill. These include pulverized fuel ash (PFA) and expanded polystyrene. The availability of PFA as a lightweight fill is common for the construction of embankments where poor founding conditions and settlement is an issue. PFA in itself is not a semi buoyant material but has an in situ density some 30% less than the equivalent sand filling.

6.12.1.3 Unfortunately, the volume of the reclamation above sea bed level for the notional reclamation will be in excess of 65 million cubic metres. This far exceeds the market supply for PFA in Hong Kong and the surrounding areas of southern china. At present PFA production in Hong Kong is limited to less than about 1million cum per annum. This supply rate is not significantly greater than the demand for the material for use in concrete production. There is also a shortfall of PFA for use ion concrete production in southern china with the result that the price for PFA is considerably (more than 5x) the price of sand and, as a consequence the viability of PFA use rules the use of the material us non viable in all bust specialist applications.

6.12.1.4 The use of expanded polystyrene to fill the reclamation at very low weight is however more feasible. Polystyrene filling for very lightweight embankments has been implemented since 1972 and in the UK since 1985. The main use has been for road embankments where the technology is well established although of limited application. The material arrives at site in standard 2440 x 1220 x 610 mm blocks. The density of the material is only 100 kg / m3 although water absorption at a rate of between 8 and 10 % can be anticipated for blocks permanently submerged below the water table.

6.12.1.5 The material, because of its high buoyancy makes it very difficult to handle when placed below the water table. In practice it is only practical to use the material above mean sea level. Nevertheless, significant savings amounting to 35% of the loading attributable to the reclamation can be achieved by placing the material between +1.3 and +5.5 mPD levels within the reclamation. This will generate significant reduction in the surcharge requirements for the site. The material can be substituted into the reclamation by rotating it with the surcharging, allowing surcharged to be removed at an earlier time. The use of the foam will also reduce the volume of surcharge required thereby reducing the disposal problem at the end of surcharging.

6.12.1.6 Specific provisions need to be made to protect the material from degradation through exposure to ultraviolet light, leaching of plasticizers, attack by solvents. This is however not difficult to achieve by simply wrapping the foam blocks in polythene sheeting and by burying the blocks. The potential for fire is more difficult to mitigate against, requiring the material to be placed in compartments of restricted size.

6.12.2 *Fundamental Parameters and Requirements*

6.12.2.1 The adoption of the material in large quantities places demands on the supply of the raw materials necessary to fabricate the blocks. For an efficient production the blocks would be fabricated from raw styrene monomer on the site in a dedicated production facility. Whilst this will be very efficient, the necessary precautions will need to be taken for handling the raw materials safely as these are toxic and highly flammable. The construction of a major fabrication plant will involve major environmental, health and safety requirements in their own right. Nevertheless these issues could be overcome albeit requiring time consuming protocols.

6.12.2.2 Handling the material is relatively easy due to its light weight. However, the issues associated with retaining the material during typhoons before it is placed and issues associated with control over its buoyancy during placing below water will need to be addressed.

6.12.2.3 The block fabrication and handling costs are very low and the benefits associated with avoidance of disposal of surcharge are significant.

6.12.3 *Benefits*

6.12.3.1 The benefits of the adoption of lightweight foam filling in place of sand filling can be summarised as follows:-

- Light weight reduces overburden stresses and the need for surcharging;
- Reduced volume of filling materials and borrow provisions required;
- Rapid filling rates can be achieved;
- Allows cycling of surcharge and potentially shorter surcharge periods; and
- Established technology and track record of over 30 years.

6.12.4 *Disadvantages*

6.12.4.1 There are a few disadvantages associated with the adoption of foam filling within the reclamation. These can be summarised as follows:

- Unknown performance characteristics in long term – over 50 years. This might result in disintegration and collapse of the foam structure;
- Potential environmental risks associated with the production plant involving large quantities of potentially toxic and highly flammable ingredients;
- Risk from attack if the foam is attacked by a possible fuel or solvent spill; and
- Risk associated with the potential for fire.

6.12.5 *Limitations and Environmental Risks*

6.12.5.1 As noted, the potential risks are associated with possible failure of the foam in the longer term. To date this has not been proved to be an issue provided the material is not exposed to sunlight once it is permanently installed. Nevertheless there remains an unknown limitation in this respect. Of more concern are the vulnerabilities of the material to the effects of fire of disruption as a result of fuel or solvent spills. All of these aspects can be mitigated against through burying the material and ensuring it is divided into manageable packages such that in the event of a fire or fuel spill only limited volumes would be affected. Filling areas would need to be compartmented and separated with inert filling materials. The potential limitations associated with the emission of toxic fumes in the event of a fire would need to be addressed.

6.12.6 *Implementation Risks*

6.12.6.1 The greatest risks in adopting the use of styrene foams are related to their fabrication and installation when environmental and fire risks are at their greatest. Nevertheless the proven track record of the use of these materials is such that it has been demonstrated that these risks can be mitigated effectively.

6.13 Floating Structures

6.13.1 *Process Description*

6.13.1.1 Floating runways have been proposed for a number of years for adoption particularly in Japan where offshore and artificial island development for the construction of airports has become common place. Airports at Kansai, Kitakyushu, Kobe and Chubu are all built offshore because of the difficulties associated with planning such facilities on land.

6.13.1.2 Consideration has been given to the adoption of floating structures at each of the above detailed sites but to date practical experience has been limited to the construction and testing of a large scale model termed the Mega Float project in Tokyo Bay. This particular project involved the construction of a large scale mock up comprising the construction of a series of floating units which were assembled offshore into a runway structure some 1000m in length. In phase 2 of the experimental work a structure was assembled from pre-fabricated units, each 300m x 60m x 3m. The total cost of the experiment had a budget of $US 103.6 million.

6.13.1.3 It is proposed that a floating development at Chek Lap Kok might be constructed from similar sized units although the height of the units might be increased to about 4m to make provision for improved resistance to overtopping during typhoons. Draught would need to be set at about 1.5 – 2m. Units would need to be assembled on the site to form a 4000m long strip some 350m wide to accommodate the runway and taxiways in a similar arrangement to that of the existing north runway. Previously, in the Mega Float project the individual box units were fabricated into 1000m long x 60m wide units having a fabricated weight of approximately 40,000 tonnes each. The deck surfacing comprised precast concrete construction developed as a composite construction in a similar manner to the piled structure at Haneda.

6.13.2 *Fundamental Parameters and Requirements*

6.13.2.1 The construction of some 24 prefabricated units, each of 40,000 tonnes would place a major demand on the shipbuilding industries of South East Asia. Nevertheless construction on this scale would be achievable, occupying about 10% of the world capacity.

6.13.2.2 Whilst the shipbuilding technology associated with the construction of the units lies well within the capabilities of the modern shipyards in China and Korea there remain considerable obstacles to the advancement of the technology necessary to fabricate the units into a sufficiently robust final structure. This would require welding of the units on site.

6.13.2.3 Significant consideration will need to be given to the most appropriate corrosion technology to be adopted in order to achieve the required design life of 120 years. Specific consideration will need to be given to the advancement of cathodic protection technology designed to address the issues associated with such large structures.

6.13.2.4 Fundamental to the design of this structure will be the need to adequately anchor it in position. It will be necessary to construct a series of large scale anchorage points to effectively anchor the structure whilst permitting it to rise and fall with the tide. Consideration will need to be given to the concept of allowing the structure to settle on to the seabed by partially flooding the structure. Although the surcharge arising from this process can be carefully controlled it will necessarily result in an imposed loading of about 55 kPa if the structure is to remain firmly placed on the seabed during typhoon surge conditions. Long term settlement will therefore be inherent in the adoption of such a system.

6.13.2.5 As an alternative it might be appropriate to dynamically adjust the buoyancy of the structure with every tide in order to control the loading on the seabed. Such a system would entail the movement of up to 2 million cubic metres of water in and out of the ballasting arrangement with the rise and fall of each typical 1.5m tide. This is equivalent to almost 100m3/s and the pumps necessary to achieve this will be comparable with those of a major coal fired power station. (cf Castle Peak B Power Station – 88 m3/s) This is not a preferred option because of the very high operating and maintenance costs.

6.13.3 *Benefits*

6.13.3.1 The benefits associated with the adoption of a floating structure can be summarised as follows :-

- With the exception of anchorage points the development of the platform can be achieved without disturbance to the underlying marine deposits. There would therefore be no significant environmental impacts;
- Through the use of a series of fabrication yards the construction programme could be reduced considerably; and
- No provision needs to be made for settlement of filled areas.

6.13.4 *Disadvantages*

6.13.4.1 The disadvantages of this type of construction mainly relate to the fact that this will be a ground breaking technological advance over any previous methodology used for airport construction, despite the fact that this option has been considered in many previous cases. Whilst the shipbuilding technology is well tried and tested there remain many uncertainties in respect of the performance of such a large structure under the influence of longitudinal bending of the vessel hull. Specific disadvantages can be summarised as follows:-

- Technology not previously adopted and long term fatigue and bending of the main hull need to be addressed;
- Control of corrosion in such a large scale structure will introduce new challenges which may lead to difficulties in meeting the required 120 year design life;
- Large scale anchorages to be constructed through the seabed to arrest lateral movements of the structure;
- The runway will change levels with the state of the tide unless the whole structure is ballasted to rest on the seabed;
- Ramp structures will be required where the new runway structure interfaces with the surrounding areas. Aircraft will not be able to negotiate such ramps;
- Settlement issues remain although the magnitude of settlement can be limited by controlling the ballasting forces; and

- Reliability, maintenance and operation costs of any dynamic ballasting system will be key issues.

6.13.5 *Limitations*

6.13.5.1 The limitations associated with the adoption of a floating scheme relate to the ability to assemble such a large structure from prefabricated units on site. Sinking of the structure into the correct location may also prove difficult unless the seabed is firstly pre-levelled. This can however be achieved by placement of a sand bedding on the existing sea bed. Concerns are however related to the fact that consolidation in the mud pit areas will continue at a different rate to other areas. The end result will be a runway which will be constrained to span between harder less settlement prone founding areas. Whilst compartmentalisation of the structure and differential ballasting can be adopted there still remains significant concern over the bending of such a long structure.

6.13.5.2 Dynamic ballasting of the structure in real time represents a major step forward in civil engineering technology. The size of the pumps and the magnitude of the ballasting operation will create a major logistical issue. It will be necessary for reasons of the need for redundancy in such systems to have multiple pumps and power supplies to ensure backup at down times and during maintenance interventions. Such systems will require replacement possibly more than once during the 120 year design life.

6.13.6 *Environmental Implications*

6.13.6.1 The adoption of a floating runway represents a major advance in avoiding the environmental issues associated with developing the proposed site. With the exception of the anchorage locations very little intervention with the contaminated mud pits is foreseen.

6.13.6.2 While the size of the floating platform is large, it is relatively small by comparison with the seabed area in the north western waters and it is not considered that this would significantly affect water quality by restricting normal gaseous exchange. Additionally, many species may benefit from the shade and as such, this could enhance the habitat of the area.

6.13.7 Furthermore, the Japanese Mega-Float project has actually build a large floating platform and the survey results confirms no significant ecological and fisheries impacts. Thus, the shading is not considered as habitat loss as would be in a reclamation. The relevant reference is cited in 8.6.10.5.

6.13.8 *Implementation Risks*

6.13.7.1 There are opportunities in this development scenario for fast tracking the project. There is a high degree of potential for fabricating the modules in different shipyards permitting parallel operations over the initial prefabrication stages of the project. Assembling the units on site however presents major logistical issues, both from the point of view of ensuring fit and the vulnerability of the structure to typhoons whilst in a semi complete state.

6.13.7.2 The evaluation of fatigue and extreme longitudinal bending brought about by differential settlements if the supporting marine muds will require an in depth and potentially time consuming development period.

6.13.7.3 There have been a number of proposals for the floating structure solution to airport construction. In the past this development option has always been rejected for a variety of reasons but primarily the perceived risks associated with the adoption of an alien technology to solve a reclamation project which arguably resolves the majority of the problems through the simple use of filling material. The adoption of the floating scheme introduces many uncertainties into what would otherwise be a simple and robust filling job.

6.14 Consolidation Using Electro Osmosis

6.14.1 *Process Description*

6.14.1.1 The concept and processes associated with forced consolidation by means of electro osmosis within soft clays have been established since the 1930’s. However, the technique has only really been adopted for use in a very limited number of specialist applications. Never has this approach been adopted to achieve forced consolidation of clays on such a large scale.

6.14.1.2 The process involves the installation of a network of vertical electrodes which penetrate through the clay fill stratum requiring consolidation. The electrodes are spaced in a regular grid and a DC or more recently a pulsed electrical DC potential is applied between the electrodes. The mechanism of consolidation relates to the attraction of water and ions to the cathode (negative electrode). Processes of electro osmosis occur at the cathode which result in the repulsion of pore water from the electrode. Oxygen gas is generated at the anode and Hydrogen is generated at the cathode. These gasses must be efficiently and safely discharged to the atmosphere. As a consequence of the forced expulsion of the pore water from the cathode consolidation occurs in the area adjacent to the cathode. Commensurate increases in density and strength occur as the pore water is discharged in much the same manner as when consolidation occurs through the application of stress.

6.14.1.3 Typically an electro-osmosis system might be arranged with electrodes at 1 to 3m spacings and potentials in the rage 10 to 120 volts might be applied depending upon the soil resistivity. Currents in individual circuits in an electrode array system might be as large as 200 – 350 A and will lead to consolidation of the extent required for this project within about 100 days. Again the duration of the applied potential will be very dependent upon the particular resistivity of the soil in question.

6.14.1.4 The power consumed by a typcial electro osmosis might be of the order of about 20 kWh/m^3 of material treated. This would suggest that the cost of electricity consumed would be of the order of $HK 15 – 20 per m^3 Power would be derived from large numbers of transformer and rectifier units while electrodes would be manufactured from carbon coated steel rods or tubes.

6.14.2 *Fundamental Parameters and Requirements*

6.14.2.1 An electro osmosis system suitable for forced consolidation of the proposed notional study area would require the installation of electrodes into the mud pits at about 2-3m centres. Some 1 to 1.5 million coated steel electrodes would need to be installed to achieve the required consolidation over about a 120 day period. On this basis, and assuming about 25% of the site would be treated at any one time then the power consumption might be of the order of 130 MW. .

6.14.3 *Benefits*

6.14.3.1 The benefits of adoption of this system relate to the speed with which the consolidation process can be achieved. Specific benefits include

- Rapid consolidation of affected materials
- Possibility of applying differing amounts of power to selected areas to achieve just the performance required.
- If the hydrogen generated by the process can be collected it could be used to generate some of the power used for the process
- There is the opportunity to collect the pore water being discharged through the cathodes. Any contamination could be arrested and treated in this manner

6.14.4 *Disadvantages*

6.14.4.1 Many of the disadvantages associated with the adoption of this type of system relate to how little experience there is in using such a system and the difficulties associated with operating such a system in the sea. Specific disadvantages include:-

- Lack of previous experience or proof of robustness on any type of large scale project
- Evolution of hydrogen is potentially a health and safety issue and must be controlled.
- A Large power demand must be addressed
- The system has never been implemented under water and insulation of the electrodes will not easily be achieved
- The risks associated with short circuits are considerable
- The electrodes corrode severely as a result of changes in the pH adjacent to the electrodes

6.14.5 *Limitations*

6.14.5.1 The limitations of the system relate to the effectiveness of the electrodes. Placing the electrodes closer together potentially makes the system more reliable however the number of electrodes then potentially becomes unmanageable.

6.14.5.2 The clays at the site have a very low density initially. This makes the overall resistivity of the soil mass very low. The consequence lies in the fact that very high currents will flow as a result of the low overall resistivity of the soil and the fact that the system will effectively be stifled by the sea water shorting out the system.

6.14.6 *Environmental Implications*

6.14.6.1 Whilst it is potentially possible to abstract the pore water from the cathode and there is potential to make the cathodes from perforated or porous pipes to enable the pore water to be efficiently extracted and treated there are many issues associated with the fact that the technology has not been tried and tested on any large scale projects.

The severe changes in pH at the electrodes and the corrosion of the electrodes potentially make many issues of metal contamination far worse than at present.

6.14.6.2 The evolution of hydrogen and oxygen from around the electrodes makes the system potentially dangerous although the flaring of the hydrogen will result in harmless generation of water.

6.14.7 *Implementation Risks*

6.14.7.1 By far the greatest risks associated with the adoption of an electro osmosis scheme relate to whether or not the scheme can be made to operate in the very low density mud within the pits. Insulation of the electrodes and the difficulties associated with working in sea water may prove to be insurmountable.

6.14.7.2 In general, it has been concluded that there is insufficient experience in operating a system of the type proposed and the risk of the system not functioning as required is deemed to be high. The consultants, having explored the potential for the use of the system have concluded that it is insufficiently robust to be considered further in the study.

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7 CONSULTATION WITH INTERNATIONAL EXPERTS

7.1 Overview

7.1.1 During preparation of the Initial Options Report a number of international experts have been consulted on appropriate methods of land formation over the contaminated mud pits.

7.1.2 At the time of preparing the Initial Options Report there is only very limited ground investigation data available on the properties of the material in the mudpits. The information which is available is limited to the upper 6 m of the clay. It became clear at an early stage in the consultation process that a set of preliminary design parameters was required to assist the various experts in assessing appropriate land formation options. Accordingly the first priority was to discuss the likely properties of dredged clay fill with experts who had dealt with this material in the construction of reclaimed land.

7.1.3 In this respect the first meetings were held in Singapore with Professor Tan Thiam Soon, who has investigated the properties of dredged clay fill and been involved in the construction of reclamations in Singapore using this material. Following these first meetings a briefing note on the likely properties of the dredged clay fill was produced and this note was provided to the various international experts who have been consulted on appropriate methods of land formation over the CMPs. The briefing note formed the basis for *Section 2.7* of this report.

7.1.4 The consultation has included two trips to Singapore and a trip to Japan. The first trip to Singapore was primarily to discuss the properties of dredged clay fill with Professor Tan and to develop an overview of the most appropriate land formation methods. During this trip two meetings were also held with engineers working for Penta Ocean. Penta Ocean is currently using cement stabilized dredged clay as filling material for a new reclamation in Singapore. Penta Ocean is one of the two largest Japanese contractors who specialize in marine construction and has particular experience of ground treatment work.

7.1.5 The second trip to Singapore was to discuss the proposed CMP properties and to meet with engineers working for Surbana who are responsible for the design and construction supervision of reclamations using dredged clay as filling material as well as reclamations which have used sand compaction piles and deep cement mixing.

7.1.6 The visit to Japan was to discuss land formation options with engineers who have been responsible for the design and construction of airport reclamations over soft ground. Professor Tan also attended to trip to Japan to provide additional advice, particularly when discussing the applicability of the various land formation techniques to the dredged clay fill in the CMPs. The trip included a visit to the reclamation that is currently being constructed for the Kansai airport extension and to the reclamation that is currently being constructed for the Haneda airport extension. These two reclamations are being constructed over thick deposits of weak and soft clay and are using a combination of sand spreading techniques and sand drains to accelerate consolidation and cement mix piles and sand compaction piles to reinforce the soft ground below the seawalls.

7.1.7 Whilst in Japan meetings were also held with experts from the Ports and Airport Research Institute who have been responsible for the development of land formation methods in Japan, engineers from Toa Corporation and individual experts on land formation methods. Toa Corporation is the second of the two largest Japanese contractors who specialize in marine construction work and ground treatment work in particular.

7.1.8 In addition to the above, experts involved in reclamation construction work in Korea, China (including Hong Kong) and Europe have been contacted by phone and email. The contact with European experts has been largely through Professor David Hight, who is an expert on the properties and characterization of soft ground and who has wide experience of reclamation work around the world. The experts in Korea have been contacted through Mr Olivier Hayes who has recently been involved in the construction of the Busan New Port, which is a reclamation constructed over very thick deposits of soft clay primarily using sand compaction piles to support the vertical seawalls.

7.1.9 Professor's Tan and Hight were also consulted on the methods of investigating the contaminated mud pits and assisted in the preparation and review of specifications of this work.

7.1.10 *Table 7.1* presents a list of the various experts who have been consulted during preparation of this report together with a brief summary of their particular areas of expertise. The recommendations made by the experts have been incorporated into the discussion of feasible land formation techniques described in *Section 6* of this report.

Table 7.1 Summary of Discussions with International Experts During Preparation of Initial Options Report

Name – Organisation	Method of Contact	Particular Area of Expertise Relevant to the P131 Contract
Professor Tan Thiam Soon – National University of Singapore	Meetings in Singapore and Japan, email and phone and briefing note on CMP	The properties of grab dredged clay in situ and under reclamation loading. Reclamation design and construction over soft clay. Responsible for Peer review.
Er. Loh Yan Hui –Surbana International Consultants.	Meetings in Singapore, email and phone and briefing note on CMP	Er. Loh is the Vice President Reclamation and Infrastructure Division and has very extensive experience of reclamation work over soft ground in Singapore using a variety of ground treatment methods
Er. Chia Way Seng – Surbana International Consultants.	Meetings in Singapore, email and phone and briefing note on CMP. Also attended trip to Japan	Er. Chia has wide experience of construction of reclamations over soft ground and is currently responsible for design and construction supervision of reclamation work in Singapore using sand compaction piles and in situ cement mixed piles.
Er Seah Kim Huah – Surbana International Consultants	Meeting in Singapore, and briefing note on CMP.	Er Seah has wide experience of construction of reclamations over soft ground and is currently responsible for design and construction supervision of reclamation work over dredged clay deposits
Er Tee Choon Peng	Meeting in Singapore, and briefing note on CMP.	Er Peng has been responsible for the construction supervision of reclamation over dredged clay deposits for the past 7 years.
Professor David Hight – Geotechnical Consulting Group	Email and phone discussions and briefing note on CMP	Professor Hight is an expert in the characterization of soft clay deposits and has wide experience of reclamation construction around the world.
Ir Olivier Hayes – Bouygues Construction	Meeting in Hong Kong, email, phone and briefing note on CMP	Ir Hayes was involved in the design and construction of reclamation and seawalls for the Busan New Port which has been constructed over a 40 m depth of very soft clay. The vertical caisson walls are supported on sand compaction piles and the reclamation has used PVDs to accelerate consoldation in excess of 5m.
Professor Tham L G – Hong Kong University	Phone	Use of vacuum consolidation to improve ground in China
Professor Chu Jian – Nanyang Technological University Singapore	Phone, email and briefing note on CMP	Use of vacuum consolidation in to improve ground in China

Name – Organisation	Method of Contact	Particular Area of Expertise Relevant to the P131 Contract
Dr Masaki Kitazume – Port and Airport Research Institute Japan	Meeting in Japan, email and briefing note on CMP.	Director of Geotechnical and Structural Department of PARI. Responsible for research into land formation techniques over soft ground. Particular experience in deep cement mixing and sand compaction piles for treatment of soft ground
Dr Masaaki Terahi – Nikken Sekkei Consultants	Meeting in Japan, email and briefing note on CMP	Former Director of Geotechnical and Structural Department of PARI. Responsible for research into land formation techniques over soft ground. Particular experience in deep cement mixing and sand compaction piles for treatment of soft ground
Associate Professor Mamoru Mimura – DPRI Kyoto University	Meeting in Japan, email and briefing note on CMP	Design of reclamation and ground treatment work for the Kansai International Airport.
Osamu Nakagome – Toa Corporation	Meeting in Japan	Mr Nakagome is the senior executive officer of Toa Corporation, a Japanese contracting company with wide experience of land formation and ground treatment works. Mr. Nakagome organized a meeting with 7 other Toa Corporation engineers to discuss the practical aspects of sand compaction pile and deep cement mixing.
Tsuyoshi Emura – Kansai International Airport Co. Ltd.	Meeting in Japan	Ir Emura is the engineering division manager responsible for the construction of the reclamation for the extension to Kansai airport
Brian Sephton – Penta Ocean Koon Hyundai Van Oord JV	Meetings in Singapore	Ir Brian Sephton is the Project Coordinator for the JV responsible for the construction of the Pasir Panjang reclamation in Singapore. This includes the use of cement mixing with dredged marine clay and use as backfill material. Mr Sephton organized a meeting with 5 engineers from the JV to discuss cement stabilized dredged backfill.
Mr Klaus Ooms – Demas Dredging	Email, phone and briefing note on CMP	Mr Ooms is a specialist dredging consultant with very wide international experience of dredging and fill placement methods. Mr Ooms is familiar with Hong Kong conditions, the airport site and the filling of the CMPs.
Nooy van derr Kolff A.H – Boskalis	Email, phone and briefing note on CMP	Mr Van Derr Kolff is an expert on dredging technology. In particular he has provided advice on the potential application of the BeauDrain for vacuum consolidation
Dr A Au – Lecturer Hong Kong University	Meeting in Hong Kong, email and briefing note on CMP	Dr Au has been involved in research work in Hong Kong studying the dispersion of dredged Hong Kong marine clay when placed in water and the properties of the Hong Kong marine clay when mixed with cement.

Name – Organisation	Method of Contact	Particular Area of Expertise Relevant to the P131 Contract
Dr. Y M Na – Hyundai Engineering and Construction	Email and briefing note on CMP	Dr Na is an expert in the construction of reclamations over soft ground in Korea. Response outstanding at time of preparation of Final Options Report
Ir J B Kwon – Ensung Construction Company Ltd	Email and briefing note on CMP	Ir Kwon is the vice president of Ensung Construction and is responsible for the construction of sand compaction piles at Busan New Port Phase 2 and 3. Response outstanding at time of preparation of Final Options Report.
Dr Nick Bray – Dredging Research International	Email, phone and briefing note on CMP	Mr Bray is an expert on dredging and reclamation construction in Europe. Response outstanding at time of preparation of Final Options Report.

8 PRELIMINARY ENVIRONMENTAL REVIEW

8.1 Review of Water Quality Baseline Conditions

8.1.1 *North-Western Water Control Zone*

8.1.1.1 Under the provisions of The Water Pollution Control Ordinance 1980 (WPCO) (Cap. 358) Hong Kong's waters have been divided into a series of Water Control Zones (WCZs) and each WCZ has a set of Water Quality Objectives (WQOs) which have been declared to protect the specific beneficial uses and conservation goals of each of the zones.

8.1.1.2 The study area lies within the North Western Waters Water Control Zone (NWWCZ). Marine waters within the NWWCZ are identified as having the following beneficial uses:

- a source of food for human consumption;
- a commercial fisheries resource;
- a habitat for marine organisms;
- recreational bathing beach;
- secondary contact recreation including diving, sailing and windsurfing;
- domestic and industrial supply;
- navigation and shipping; and
- aesthetic enjoyment.

8.1.1.3 Relevant marine WQOs applicable to the NWWCZ are summarised in *Table 8.1* below.

Table 8.1 Water Quality Objectives for the North Western Waters Water Control Zone

Aesthetic Appearance
➢ There should be no objectionable odours or discolouration of the water.
➢ Tarry residues, floating wood, articles made of glass, plastic, rubber or any other substances should be absent.
➢ Mineral oil should not be visible on the surface.
➢ There should be no recognisable sewage derived debris.
➢ Floating, submerged and semi-submerged objects of a size likely to interfere with the free movement of vessels, or cause damage to vessels, should be absent.
Bacteria
➢ The levels of *Escherichia coli* should not exceed 180 counts per 100 ml at bathing beaches,

calculated as the geometric mean of the 5 most recent samples collected by EPD.

- The levels of *Escherichia coli* should not exceed 610 counts per 100 ml at secondary contact recreation sub-zones, calculated as the geometric annual mean.

Dissolved Oxygen

- The depth averaged concentration of dissolved oxygen should not fall below 4 mg/l for 90% of the sampling occasions during the whole year
- The concentration of dissolved oxygen should not be less than 2 mg/l within 2m of the seabed for 90% of the sampling occasions during the whole year.

pH

- The pH of the water should be within the range 6.5 – 8.5 units.
- Human activity should not cause the natural pH range to be extended by more than 0.2 units.

Temperature

- Waste discharges shall not cause the natural daily temperature range to change by more than 2.0°C.

Salinity

- Waste Discharges shall not cause the natural ambient salinity to change by more than 10%.

Suspended Solids

- Human activity should neither cause the natural ambient level to be raised by more than 30% nor give rise to accumulation of suspended solids which may adversely affect aquatic communities.

Ammonia

- The un-ionised ammoniacal nitrogen level should not be more than 0.021 mg/l calculated as the annual average (arithmetic mean).

Nutrients

- Nutrients should not be present in quantities sufficient to cause excessive or nuisance growth of algae or other aquatic plants
- Without limiting the generality of the above point, the level of inorganic nitrogen should not exceed 0.5 mg/l, or 0.3 mg/l within Castle Peak sub-zone, expressed as the annual water column average.

Toxins

- Waste discharges shall not cause the toxins in water to attain such a level as to produce significant toxic, carcinogenic, mutagenic or teratogenic effects in humans, fish or other aquatic organisms, with due regard to biologically cumulative effects in food chains and to interactions of toxic substances with each other.
- Waste discharges shall not cause a risk to any beneficial use of the aquatic environment.

8.1.1.4 There are currently no defined marine water quality objectives (WQO) for metals in Hong Kong but the European Union Environmental Quality Standards (EQS; annual average concentrations; Cole *et al.*, 1999), prescribed for the protection of marine life for metals and metalloid arsenic are often referred for marine projects in NWWCZ. The EQS values to protect marine life are presented below in *Table 8.2*.

Table 8.2 The European Union Environmental Quality Standard (EQS) Values to Protect Marine Life

Metals and Metalloid	**Water Quality Standard (as dissolved metal) (µg/L)**
Arsenic	25
Cadmium	2.5
Chromium	15
Copper	5
Lead	25
Mercury	0.3
Nickel	30
Silver	N/A
Zinc	40

Note: N/A = Not available.

8.1.2 *Existing Pollution Sources and Activities*

8.1.2.1 The NWWCZ contains several significant sewage outfalls (Pillar Point, Northwest New Territories and Siu Ho Wan) and cooling water discharges from a number of users including Castle Peak Power Station, Hong Kong International Airport (HKIA) and Shiu Wing Steelworks.

8.1.2.2 Disposal of contaminated dredged material began in 1992 at the Contaminated Mud Pits (CMP) at East of Sha Chau (*Figure 8.1)* The capacity of existing pits was predicted to be exhausted by early 2009 but two potential sites for future CMPs near the HKIA have been identified (CEDD, 2005). The disposal of category M material that passes biological screening / uncontaminated dredged material also continues intermittently at the North Brothers Marine Borrow Area, shown in *Figure 8.1*. The operation of the open sea disposal ground at North Lantau Borrow pit, however, has been suspended since 2000 and there is currently no schedule for the reopening of the facility.

8.1.2.3 Commercial trawling is undertaken over much of the North Western Waters and the Urmston Road is an active shipping channel for river trade vessels, high speed ferries, large coal vessels servicing Castle Peak Power Station and the existing temporary Aviation Fuel Receiving Facility delivery vessels to Sha Chau.

8.1.3 *Baseline Water Quality Conditions*

8.1.3.1 Existing water quality in the North-western waters has been monitored for many years as part of the EPD Routine Monitoring Programme. Water quality is monitored monthly at six stations within the NWWCZ and the location of the monitoring stations are shown in *Figure 8.1*. Of these, station NM6 lies close to the study area. However, stations NM3, NM5 and NM8 are also positioned in close proximity to the study area and can be considered to represent conditions within the likely area of influence of the study. A summary of the data published by EPD for the NWWCZ during 2004 - 2006 is presented in *Table 8.3* below.

Table 8.3 A Summary of the Data Published by EPD for the NWWCZ during 2004 - 2006

Station	Temp (℃)	Salinity (psu)	Depth Average DO (mg/L)	Bottom DO (mg/L)	pH	NH_3 (mg/L)	TIN (mg/L)	*E. coli* (cfu/ 100ml)	TSS (mg/L)
2004 Dry Season									
NM 3	19.62	32.55	6.29	6.42	8.01	0.0049	0.34	288.67	9.64
NM 5	19.64	32.26	6.20	6.26	7.99	0.0056	0.42	625.60	14.50
NM 6	19.21	32.62	6.74	6.66	8.06	0.0035	0.27	10.89	15.26
NM 8	19.51	33.13	6.60	6.62	8.09	0.0025	0.18	20.43	11.80
2004 Wet Season									
NM 3	26.21	28.53	5.42	5.14	8.04	0.0067	0.53	960.00	9.30
NM 5	26.44	26.84	5.39	5.11	8.01	0.0085	0.68	763.81	10.33
NM 6	26.64	27.11	5.85	5.94	8.09	0.0059	0.56	26.33	7.66
NM 8	26.24	28.86	6.41	6.49	8.14	0.0034	0.34	2.50	11.49
2005 Dry Season									
NM 3	18.51	32.59	6.91	6.94	8.04	0.0046	0.28	364.67	11.44
NM 5	18.51	32.26	6.85	6.86	8.03	0.0070	0.40	800.00	19.79
NM 6	18.25	32.74	6.98	7.16	8.09	0.0038	0.25	193.40	14.80
NM 8	18.05	32.92	7.35	7.38	8.12	0.0016	0.15	1.30	13.76
2005 Wet Season									
NM 3	26.20	27.43	5.73	5.19	8.04	0.0084	0.60	1142.71	8.82
NM 5	26.43	25.71	5.56	4.97	8.01	0.0103	0.77	1471.90	13.46
NM 6	26.88	23.92	6.44	5.96	8.07	0.0096	0.82	59.14	8.78
NM 8	26.69	25.61	6.50	6.01	8.11	0.0064	0.62	8.12	10.30
2006 Dry Season									
NM 3	20.34	31.91	7.23	7.34	7.97	0.0046	0.40	408.33	7.59
NM 5	20.43	31.43	7.03	7.18	7.95	0.0059	0.51	1576.00	9.96
NM 6	20.04	31.81	7.27	7.36	7.98	0.0031	0.36	26.07	12.50
NM 8	19.83	32.46	7.43	7.52	8.01	0.0014	0.19	1.77	20.65

Station	Temp (℃)	Salinity (psu)	Depth Average DO (mg/L)	Bottom DO (mg/L)	pH	NH_3 (mg/L)	TIN (mg/L)	*E. coli* (cfu/ 100ml)	TSS (mg/L)
2006 Wet Season									
NM 3	26.10	27.63	5.63	5.21	7.86	0.0055	0.57	1309.05	8.52
NM 5	26.52	24.14	5.68	5.04	7.81	0.0089	0.79	1752.38	19.80
NM 6	26.64	22.20	6.18	6.10	7.86	0.0088	0.86	451.85	12.94
NM 8	26.54	24.08	6.28	6.19	7.90	0.0060	0.63	146.43	12.41

Note: Results are mean values.

8.1.3.2 According to the EPD annual report 2006, the water quality in the NWWCZ was largely stable, attaining an 89% compliance with the WQOs, the same as for the previous two years. Except for the two north-western most stations (NM 5 and NM 6) which did not meet the WQO for total inorganic nitrogen, all other stations achieved full compliance with the key WQOs for dissolved oxygen and unionised ammonia. Although high nitrogen levels were detected, orthophosphate phosphorus and total phosphorus were largely low and stable in recent years.

8.1.3.3 As the NWWCZ is situated at the month of the Pearl River Estuary (PRE) and heavily influenced by the massive freshwater flows from the hinterland, the water quality of the area exhibit distinct seasonality. During summer (April – October), the estuarine influence is especially pronounced when the freshwater flows are great and salinity and temperature stratifications are prominent. During winter (November – March), with the reduced input from the PRE, water conditions are more typically marine and salinity and other parameters vary less with depth. Water temperature ranges between 18.1°C and 26.9°C over an annual cycle with a mean of 22.0°C to 23.5°C. Salinity typically varies within the range of 3.8 to 33.6 ppt with a mean of 27 to 31 ppt.

8.1.3.4 The Pearl River carries heavy loads of nitrogenous compounds and suspended sediment. As a consequence concentrations of these parameters within North Western Waters are variable. EPD monitoring data indicate that mean unionised ammonia concentrations range from 0.00295 to 0.00865 mg/l over an annual cycle with the highest value of 0.047 mg/l recorded at the station NM5 during 2006. Mean total inorganic nitrogen concentrations lie in the range of 0.26 to 0.65 mg/l with the highest value of 2.3 mg/l at the station NM5 during 2005. Mean suspended solid concentrations typically lie in the range of 8.06 to 16.63 mg/l over an annual cycle with the highest recorded value of 150 mg/l at station NM5 during 2006.

8.1.3.5 EPD's routine water quality monitoring programme does not included analysis of heavy metals. However, the major heavy metals of environmental concerns are monitored under the EM&A programme for the East of Sha Chau CMPs and the results can be considered as typical of the waters in the area. In general, many of the inorganic contaminants were below the levels of reporting limit (MIEL, 2007) and those detected were below the European Union Environmental Quality Standards. A summary of metals concentration recorded in 2005 monitoring is presented below in *Table 8.4*.

Table 8.4 Example of Metal Concentrations Recorded at the North of Airport (November 2005)

Area	**As**	**Cd**	**Cr**	**Cu**	**Pb**	**Hg**	**Ni**	**Ag**	**Zn**
RL	2	0.2	1	1	1	0.1	1	1	4
Near Field	1	0.1	0.5	1.6	0.5	0.1	0.8	0.5	5.1
Mid Field	1	0.1	0.6	2.8	0.8	0.1	0.5	0.5	6.8
Far Field	1	0.1	0.5	1.5	0.5	0.1	1.2	0.5	12

Note: RL = reporting limit; All values are area mean concentrations (µg/L). Values below RL are substitutedwith 1/2RL for mean concentration calculation; Source: Meinhardt (2007b). Near Field was located down-current of the active pit, Mid Field was further down-current of the Near Field while Far Field was located up-current of the active pit.

8.2 Review of Sediment Quality Baseline Conditions

8.2.1 *Sediment Quality of North-Western Control Zone*

8.2.1.1 The criteria used for the audit of sediment samples in Hong Kong were previously based on the Criteria for Marine Sediment Quality Classification, specified in EPD TC 1-1-92 (*Table 8.5*). This Technical Circular was later superseded by improved contaminant screening criteria namely the Management of Dredged/Excavated Sediment WBTC 3/2000 (applicable to all projects or portions of projects commencing after 31 December 2001) and more recently the ETWB No. 34/2002 (*Table 8.6)* that was promulgated on the 15 August 2002. In terms of sediment classification criteria, both WBTC 3/2000 and ETWB 34/2002 are the same and WBTC 3/2000 is, thus, not discussed further.

8.2.1.2 The previous (EPD TC 1-1-92) sediment classification system was based on the concentration of seven metals present in the sediment was classified into three classes depending on the level of any one criterion contaminant present:

- Class A: uncontaminated;
- Class B: moderately contaminated; and
- Class C: seriously contaminated.

Table 8.5 Sediment classification system under EPD TC 1-1-92

Metal	Class A (mg/kg)	Class B (mg/kg)	Class C (mg/kg)
Cd	<0.9	1-1.4	>1.5
Cr	<49	50-79	>80
Cu	<54	55-64	>65
Hg	<0.7	0.8-0.9	>1.0
Ni	<34	35-39	>40
Pb	<64	65-74	>75
Zn	<140	150-199	>200

8.2.1.3 The new sediment management and classification scheme (ETWB 34/2002), in addition to the previously used seven metals, incorporate arsenic, silver and a range of organic contaminants. Based on the level of any one criterion contaminant present, the sediments are categorized into three categories, The Lower Chemical Exceedance Level (LCEL) represents a contaminant concentration below which toxic impacts in marine organisms is considered unlikely. Conversely, the Upper Chemical Exceedance Level (UCEL) is a contaminant concentration above which toxic responses in marine organisms might be induced (Dawes, 2001; Long et al., 1995; Nicholson, 2001; Nicholson and Kennish, 2000). The three contamination categories are:

- *Category L:* Sediment with all contaminant levels lower than or equal to the LCEL. The material must be dredged, transported and disposed of in a manner which minimises the loss of contaminants either into solution or resuspension. This material will not be placed in the CMPs but disposed in open spoil grounds.

- *Category M:* Sediment with any one or more contaminant levels exceeding the LCEL and none exceeding the UCEL. The material must be dredged and transported with care, and must be effectively isolated (in the CMPs) from the environment upon final disposal unless appropriate bioassay (biological) tests demonstrate that the material will not adversely affect the marine environment.

- *Category H:* Sediment with any one or more contaminant levels exceeding the UCEL. The material must be dredged and transported with great care, and must be effectively isolated from the environment upon final disposal (in the CMPs).

Table 8.6 Criteria for Marine Sediment Quality Classification (ETWB 34/2002)

Contaminant	Lower Chemical Exceedance Level (LCEL)	Upper Chemical Exceedance Level (UCEL)
Metals (mg kg^{-1} dry wt.)		
Cadmium	1.5	4
Chromium	80	160
Copper	65	110
Mercury	0.5	1
Nickel[1]	40	40
Lead	75	110
Silver	1	2
Zinc	200	270
Metalloid (mg kg^{-1} dry wt.)		
Arsenic	12	42
Organics-PAHs (µg kg^{-1} dry wt.)		
Low Molecular Weight PAHs	550	3160
High Molecular Weight PAHs	1700	9600
Organics-non-PAHs (µg kg^{-1} dry wt.)		
Total PCBs	23	180
Organometallics (µg TBT L^{-1} in interstitial water)		
Tributyltin[1]	0.15	0.15

Note 1: The contaminant level is considered to have exceeded the UCEL if it is greater than the value shown.

8.2.1.4 In addition to the chemical screening values, ETWB TCW 34/2002 also incorporate biological testing requirement in order to determine the treatment/disposal requirement. The process is summarised in *Figure 8.2*, and in essence, four treatment/disposal options are defined:

- *Type 1 Open Sea Disposal.* Category L material will be disposed in open sea. Biological screening tests are not required before disposal.

- *Type 1* Open Sea Disposal in dedicated sites.* Category M material is required to conduct biological screening tests before disposal. If material passes in the tests, it will be disposed in dedicated sites of open sea.

- *Type 2 Confined Marine Disposal.* Category M material that fails in the biological screening tests and Category H material with all contaminant levels lower than or equal to ten times of the LCEL require confined marine disposal. Category H material with any one or more contaminant levels exceeding ten times of the LCEL is required to conduct biological screening dilution tests before disposal. Material that passes in the tests needs confined marine disposal.

- *Type 3 Special Treatment / Disposal.* If Category H material with any one or more contaminant levels exceeding ten times of the LCEL fails in the biological screening dilution tests, special treatment and disposal is required.

8.2.1.5 While the ETWB TCW 34/2002 chemical screening values (i.e., LCEL and UCEL) are defined for a wide range of contaminants found in the marine environment, several organic contaminants are not included (a range of individual PAHs, Total DDT, 4,4'-DDE and TBT in sediment). For these contaminants, Interim Sediment Quality Values (ISQVs) are often referred to. Although not formally applied in Hong Kong at present, ISQVs have been used elsewhere in the assessment of dredged material (Long et al., 1995; EVS, 1996). There are examples where these values have been used in Hong Kong and as such there is a precedent for their acceptability. Indeed, because of the lack of formal criteria, the ISQVs are extensively referenced and followed in local studies including the EIAs for the CMPs. As such, it is considered that they should be acceptable as a reference in this case.

8.2.1.6 The ISQVs are the same in principle to the LCEL and UCEL criteria and the ISQV-Low represents the lower value below which sediments are considered to be uncontaminated whereas the ISQV-High represents a value above which the material is considered highly contaminated and likely to induce toxic responses in marine benthic organisms. The set of ISQV values adopted by the East of Sha Chau EM&A programme are presented below in *Table 8.7* for reference.

Table 8.7 ISQV Criteria for Marine Sediment Quality Assessment

Contaminant	ISQV-Low	ISQV-High
Organics- PAHs (μg kg^{-1} dry wt.)		
Acenaphthene	16	500
Acenaphthylene	44	640
Anthracene	85.3	1100
Fluorene	19	540

Contaminant	ISQV-Low	ISQV-High
Naphthalene	160	2100
Phenanthrene	240	1500
Benzo(a)anthracene	261	1600
Benzo(a)pyrene	430	1600
Chrysene	384	2800
Dibenzo(a,h)anthracene	63.4	260
Fluoranthene	600	5100
Pyrene	665	2600
Pesticides (μg kg^{-1} dry wt.)		
4,4'-DDE	2.2	N/A
Total DDT	1.58	N/A
Organometallics (μg kg^{-1} dry wt.)		
Tributyltin (in sediment)	12.2[1]	N/A

Notes: N/A = Not applicable as presently no value is available; 1There is no formal assessment criteria for TBT in sediments and this value is the 25th percentile based on data from studies conducted in both uncontaminated and contaminated sites in Hong Kong (Aspinwall, 1998).

8.2.2 *Baseline Sediment Quality Conditions*

8.2.2.1 Existing sediment quality in the North-western waters has been monitored for many years as part of the EPD Routine Monitoring Programme. Sediment quality is monitored every six months at four stations in the North Western Waters, namely NS2, NS3, NS4 and NS6 (*Figure 8.1*). Of these, station NS6 lies close to the study area. The stations NM3 and NM4 lie in convenient close proximity to the study area and can represent conditions within the likely area of influence of the study. Maximum concentrations of key potential toxicants reported by EPD over the 3-year period 2004 – 2006 for these sites are presented in *Table 8.8*. The corresponding LCEL criteria are presented in the same tables for reference.

Table 8.8 EPD Routine Sediment Quality Data for North Western Waters (2004-2006)

Contaminant	Maximum Concentration	LCEL
Metals	**(mg/kg dry weight)**	**(mg/kg dry weight)**
Cd	0.1	1.5
Cr	42	80
Cu	48	65
Hg	0.23	0.5
Ni	24	40
Pb	46	75
Ag	0	1
Zn	120	200
Metalloid	**(mg/kg dry weight)**	**(mg/kg dry weight)**
As	16	12

Contaminant	**Maximum Concentration**	**LCEL**
Organics	**(ug/kg dry weight)**	**(ug/kg dry weight)**
Total PCBs	18	23
Low MW PAHs	68.5	550
High MW PAH	116.5	1700

8.2.2.2 It is clear that the contaminants in the surface sediment comply comfortably with the LCEL except Arsenic. Upper range arsenic concentrations were observed to exceed the LCEL. EPD noted in the annual water quality report 2000 that these arsenic concentrations might be related to the high natural arsenic levels in the soil of some areas of the Northern New Territories which could then be transported to the marine environment through river discharges and storm run off.

8.2.3 *Contaminated Sediments in CMPs*

8.2.3.1 To this Study, knowing the contaminants and their level inside the capped pits is essential to predict and assess the potential environmental impacts and the mitigation regime. However, there is only limited data about this and even if present they are generally not available to the public. EPD should have the most comprehensive log of the sediment chemistry of the dumped material as they are the permits granting authority. For various reasons, however, EPD was unwilling to provide the data collected from other projects as per our previous enquiry under another project.

8.2.3.2 Since the contaminated sediments (Class C) contained in capped pits (CMP I – IVa) were classified based on an old classification scheme (EPD TC 1-1-92; prior to 2002) obtaining the historical data while useful is not essential as the contamination level measurement were conducted before dredging. The subsequent dredging and disposal operations could have homogenise the sediment affecting the contaminants concentrations. Further, as it only assessed seven heavy metals, the information is incomplete by today's standards (c.f. ETWB TCW 34/2002). Nonetheless, knowing the concentrations of the seven heavy metals would provide useful insight to the possible scale of problem.

8.2.3.3 During the investigation of the structure of the caps, Ng (1997) also included viborcore sampling and chemical testing of seven heavy metals contained inside CMP IIa and IIb. A summary of the recorded heavy metal concentration is presented in *Tables 8.9* and *8.10* below. By applying the current LCEL/UCEL criteria to the data set, the results indicated that 84.3% of the sediment samples can be considered as uncontaminated *Category L* material. The rest of the samples, however, are contaminated by certain degree. The low percentage of contaminated samples is not surprising most of the samples analysed were collected at the clean capping layer and the results indicated that cross contamination by uncontrolled mixing of contaminated material with the capping material.

8.2.3.4 It is noted that of the 20 samples taken at 4.95 – 5.05m level, supposed to be the contaminated material, only two samples were category H (i.e., 10% of the 20 samples), four were category M (i.e., 20% of the 20 samples) and the rest were category L (i.e., 70% of the 20 samples). Unfortunately, the dataset have only included one layer of material supposed to be the disposed contaminated material and as this layer interfaces with the capping layer, it could be "diluted" by the clean capping material rending the results less representative of the bulk of contained contaminated material inside the CMPs. The information is also incomplete by today's standards as the tests have not included arsenic, silver, organics and biological testing need under ETWB TCW 34/2002.

Table 8.9 Heavy metals concentrations of CMP IIa sediments

Vibrocore No.	Depth (m)	Heavy Metal Concentration (mg/kg dry wt.)							Category[1]
		Cr	Cu	Pb	Zn	Cd	Ni	Hg	
A1b	0 - 0.2	24	12	39	81	0.6	22	0.2	L
A1b	0.4 - 0.6	23	13	36	80	0.6	21	0.2	L
A1b	1 - 1.2	23	13	32	74	0.6	21	0.2	L
A1b	1.4 - 1.6	22	13	33	75	0.6	20	0.2	L
A1b	1.8 - 2	23	12	38	80	0.6	21	0.2	L
A1a	2.95 - 3.05	25	36	50	40	**2.4**	10	0.1	M
A1a	4.95 - 5.05	22	11	31	76	0.6	19	0.2	L
A2b	0 - 0.2	22	13	48	80	0.6	21	0.2	L
A2b	0.4 - 0.6	21	12	46	78	0.6	19	0.2	L
A2b	1 - 1.2	22	12	45	84	0.6	21	0.2	L
A2b	1.4 - 1.6	21	12	47	84	0.6	21	0.2	L
A2b	1.8 - 2	26	13	**88**	90	0.6	20	0.2	M
A2a	2.95 - 3.05	22	12	38	90	0.6	22	0.2	L
A2a	4.95 - 5.05	**200**	**800**	**110**	**350**	0.6	**110**	0.2	H
A3b	0 - 0.2	23	12	36	74	0.6	21	0.2	L
A3b	0.4 - 0.6	26	12	33	76	0.6	24	0.2	L
A3b	1 - 1.2	26	12	43	76	0.6	23	0.2	L
A3b	1.4 - 1.6	24	12	31	74	0.6	22	0.2	L
A3b	1.8 - 2	22	13	33	72	0.6	21	0.2	L
A3a	2.95 - 3.05	25	15	40	89	0.6	23	0.2	L
A3a	4.95 - 5.05	26	36	50	100	0.6	22	0.4	L
A4b	0 - 0.2	20	17	46	92	0.6	22	0.2	L
A4b	0.4 - 0.6	22	11	38	87	0.6	22	0.2	L
A4b	1 - 1.2	21	11	39	86	0.6	21	0.2	L
A4b	1.4 - 1.6	22	12	35	88	0.6	22	0.2	L
A4b	1.8 - 2	23	12	39	87	0.6	21	0.2	L

Vibrocore No.	Depth (m)	Heavy Metal Concentration (mg/kg dry wt.)							Category[1]
		Cr	Cu	Pb	Zn	Cd	Ni	Hg	
A4a	2.95 - 3.05	22	13	33	86	0.6	22	0.2	L
A4a	4.95 - 5.05	24	13	36	86	0.6	22	0.2	L
A5b	0 - 0.2	22	11	33	130	0.6	20	0.2	L
A5b	0.4 - 0.6	23	13	35	76	0.6	21	0.2	L
A5b	1 - 1.2	22	12	32	73	0.6	19	0.2	L
A5b	1.4 - 1.6	22	13	31	74	0.6	20	0.2	L
A5b	1.8 - 2	20	20	36	78	0.6	18	0.2	L
A5a	2.95 - 3.05	36	**91**	56	150	0.6	23	0.3	M
A5a	4.95 - 5.05	21	**89**	**130**	**290**	0.6	13	0.5	H
A6b	0 - 0.2	22	20	40	90	0.6	20	0.2	L
A6b	0.4 - 0.6	22	16	47	81	0.6	21	0.2	L
A6b	1 - 1.2	24	22	**89**	97	0.6	21	0.2	M
A6b	1.4 - 1.6	23	23	**86**	98	0.6	21	0.2	M
A6b	1.8 - 2	21	12	36	79	0.6	20	0.2	L
A6a	2.95 - 3.05	26	13	38	84	0.6	24	0.2	L
A6a	4.95 - 5.05	9.1	2	18	20	0.6	4.5	0.1	L
A7b	0 - 0.2	24	27	50	98	0.6	21	0.2	L
A7b	0.4 - 0.6	25	28	54	100	0.6	22	0.2	L
A7b	1 - 1.2	22	14	42	81	0.6	21	0.2	L
A7b	1.4 - 1.6	22	13	33	76	0.6	21	0.2	L
A7b	1.8 - 2	21	11	31	72	0.6	19	0.2	L
A7a	2.95 - 3.05	34	59	57	120	0.6	20	0.4	L
A7a	4.95 - 5.05	31	47	48	100	0.6	21	0.2	L
A8b	0 - 0.2	25	13	40	81	0.6	22	0.2	L
A8b	0.4 - 0.6	26	13	43	84	0.6	24	0.2	L
A8b	1 - 1.2	24	13	52	86	0.6	22	0.2	L
A8b	1.4 - 1.6	24	13	40	82	0.6	22	0.2	L
A8b	1.8 - 2	26	13	51	90	0.6	24	0.2	L
A8a	2.95 - 3.05	29	15	37	88	0.6	27	0.2	L
A8a	4.95 - 5.05	3.8	2	12	5.1	0.6	2.5	0.1	L
A9b	0 - 0.2	22	12	**86**	82	0.6	20	0.2	M
A9b	0.4 - 0.6	23	13	58	75	0.6	22	0.2	L
A9b	1 - 1.2	22	12	41	76	0.6	20	0.2	L
A9b	1.4 - 1.6	24	12	36	75	0.6	22	0.2	L
A9b	1.8 - 2	21	12	35	76	0.6	20	0.2	L
A9a	2.95 - 3.05	30	14	40	92	0.6	27	0.2	L
A9a	4.95 - 5.05	13	11	27	53	0.6	12	0.1	L
A10b	0 - 0.2	26	25	47	100	0.6	22	0.2	L
A10b	0.4 - 0.6	21	11	31	70	0.6	19	0.2	L
A10b	1 - 1.2	22	19	47	120	0.6	17	0.2	L

Vibrocore No.	Depth (m)	Heavy Metal Concentration (mg/kg dry wt.)							Category[1]
		Cr	Cu	Pb	Zn	Cd	Ni	Hg	
A10b	1.4 - 1.6	21	12	34	77	0.6	20	0.2	L
A10b	1.8 - 2	25	15	34	81	0.6	22	0.2	L
A10a	2.95 - 3.05	33	43	64	130	0.6	25	0.3	L
A10a	4.95 - 5.05	22	13	40	86	0.6	21	0.2	L

Note: Shaded cell = value exceed UCEL; bold = value exceed LCEL; Data Source: Ng (1997); [1]The categorisation is provide for illustration only and shall not be considered as complete as some metals and organic pollutants are not tested.

Table 8.10 Heavy metals concentrations of CMP IIb sediments

Vibrocore No.	Depth (m)	Heavy Metal Concentration (mg/kg dry wt.)							Category[1]
		Cr	Cu	Pb	Zn	Cd	Ni	Hg	
B1b	0 - 0.2	26	17	55	89	0.6	22	0.2	L
B1b	0.4 - 0.6	25	22	43	95	0.6	22	0.2	L
B1b	1 - 1.2	31	52	64	130	0.6	23	0.2	L
B1b	1.4 - 1.6	24	17	42	90	0.6	22	0.2	L
B1b	1.8 - 2	**82**	50	**140**	**290**	**1.9**	**73**	0.2	H
A1a	2.95 - 3.05	56	**100**	**140**	180	0.69	33	0.2	H
B1a	4.95 - 5.05	55	**100**	**90**	180	1	21	0.4	M
B2b	0 - 0.2	22	13	40	86	0.6	22	0.2	L
B2b	0.4 - 0.6	22	12	39	79	0.6	20	0.2	L
B2b	1 - 1.2	21	12	43	83	0.6	21	0.2	L
B2b	1.4 - 1.6	23	11	42	70	0.6	20	0.2	L
B2b	1.8 - 2	20	13	40	76	0.6	19	0.2	L
B2a	2.95 - 3.05	20	19	51	72	0.6	15	0.1	L
B2a	4.95 - 5.05	38	**86**	**85**	180	0.6	21	0.7	M
B3b	0 - 0.2	25	39	56	100	0.6	22	0.2	L
B3b	0.4 - 0.6	25	34	57	100	0.6	21	0.2	L
B3b	1 - 1.2	24	33	51	89	0.6	19	0.2	L
B3b	1.4 - 1.6	23	34	50	91	0.6	18	0.2	L
B3b	1.8 - 2	22	31	56	89	0.6	18	0.2	L
B3a	2.95 - 3.05	32	49	70	110	0.62	22	0.2	L
B3a	4.95 - 5.05	29	41	52	100	0.6	18	0.2	L
B4b	0 - 0.2	21	11	30	76	0.6	20	0.2	L
B4b	0.4 - 0.6	24	12	**80**	77	0.6	20	0.2	M
B4b	1 - 1.2	21	12	30	72	0.6	19	0.2	L
B4b	1.4 - 1.6	21	12	51	73	0.6	20	0.2	L
B4b	1.8 - 2	15	20	30	54	0.6	11	0.1	L
B4a	2.95 - 3.05	21	14	27	46	0.75	9.4	0.1	L
B4a	4.95 - 5.05	31	18	37	72	0.6	20	0.2	L
B5b	0 - 0.2	19	9.8	46	81	0.6	20	0.2	L
B5b	0.4 - 0.6	24	13	40	88	0.6	22	0.2	L

Vibrocore No.	Depth (m)	Heavy Metal Concentration (mg/kg dry wt.)							Category[1]
		Cr	Cu	Pb	Zn	Cd	Ni	Hg	
B5b	1 - 1.2	22	12	35	82	0.6	22	0.2	L
B5b	1.4 - 1.6	24	14	50	82	0.6	22	0.2	L
B5b	1.8 - 2	23	16	**110**	100	0.6	21	0.2	M
B5a	2.95 - 3.05	43	**76**	71	160	0.6	29	0.3	M
B5a	4.95 - 5.05	30	35	62	110	0.6	26	0.2	L
B6b	0 - 0.2	25	13	51	84	0.6	22	0.2	L
B6b	0.4 - 0.6	21	12	38	84	0.6	21	0.2	L
B6b	1 - 1.2	25	13	51	81	0.6	23	0.2	L
B6b	1.4 - 1.6	22	13	50	79	0.6	20	0.2	L
B6b	1.8 - 2	23	14	39	83	0.6	22	0.2	L
B6a	2.95 - 3.05	31	58	57	120	0.6	22	0.2	L
B6a	4.95 - 5.05	25	26	26	57	0.6	16	0.4	L
B7b	0 - 0.2	31	32	56	120	0.6	27	0.2	L
B7b	0.4 - 0.6	23	18	40	87	0.6	22	0.2	L
B7b	1 - 1.2	24	14	36	81	0.6	22	0.2	L
B7b	1.4 - 1.6	19	11	34	80	0.6	20	0.2	L
B7b	1.8 - 2	24	13	39	84	0.6	22	0.2	L
B7a	2.95 - 3.05	29	29	51	100	0.6	26	0.2	L
B7a	4.95 - 5.05	20	31	**97**	160	0.6	15	0.2	M
B8b	0 - 0.2	22	12	42	81	0.6	20	0.2	L
B8b	0.4 - 0.6	19	12	71	75	0.6	20	0.2	L
B8b	1 - 1.2	19	12	52	74	0.6	19	0.2	L
B8b	1.4 - 1.6	22	13	53	77	0.6	21	0.2	L
B8b	1.8 - 2	22	16	39	80	0.6	20	0.2	L
B8a	2.95 - 3.05	28	16	42	92	0.6	25	0.2	L
B8a	4.95 - 5.05	40	**84**	**92**	170	0.6	28	0.2	M
B9b	0 - 0.2	35	**72**	52	120	0.6	26	0.2	M
B9b	0.4 - 0.6	49	**130**	58	140	0.6	35	0.2	H
B9b	1 - 1.2	42	**110**	55	120	0.6	31	0.2	M
B9b	1.4 - 1.6	27	34	49	89	0.6	22	0.2	L
B9b	1.8 - 2	31	47	46	100	0.6	26	0.2	L
B9a	2.95 - 3.05	41	**91**	55	130	0.6	31	0.2	M
B9a	4.95 - 5.05	23	14	35	88	0.6	22	0.2	L
B10b	0 - 0.2	32	61	56	110	0.6	24	0.2	L
B10b	0.4 - 0.6	35	**77**	64	120	0.6	26	0.2	M
B10b	1 - 1.2	25	29	68	89	0.6	22	0.2	L
B10b	1.4 - 1.6	26	38	47	96	0.6	22	0.2	L
B10b	1.8 - 2	23	28	46	84	0.6	21	0.2	L
B10a	2.95 - 3.05	36	51	53	120	0.6	31	0.2	L
B10a	4.95 - 5.05	27	14	43	91	0.6	24	0.2	L

Note: Shaded cell = value exceed UCEL; bold = value exceed LCEL; Data Source: Ng (1997); [1]The categorisation is provide for illustration only and shall not be considered as complete as some metals and organic pollutants are not tested.

8.3 Review of Ecological Baseline Conditions

8.3.1 The major sensitive receiver present within the study area is the Chinese White Dolphin (CWD). CWD is a CITES-listed species and is internationally protected. The project site is located in the northwestern waters (and the National Nature Reserve for Chinese White Dolphin in the Pearl River Estuary (PRE)), which is the prime habitat for the CWD in PRE. This area of Hong Kong will be subject to considerable pressure from development (e.g., the Hong Kong Zhuhai Macao Bridge package including the Boundary Crossing Facilities and the Tuen Mun Chek Lap Kok Link) and high volumes of contaminated mud disposal (at East Sha Chau Contaminated Mud Pits). The dolphins are under considerable environmental pressure both locally and within their geographic ranges. The local dolphin population is, therefore, considered to be at risk from a wide range of sources.

8.3.2 The dolphin predominantly frequents the shallow, less saline brackish waters around the PRE and loss of habitat to numerous developments, fishing, shipping activity and pollution from various sources have placed increasing pressure on the dolphin population. In Hong Kong, the dolphin population is centred in the northwestern waters but higher numbers are present throughout the PRE. The total size of the PRE CWD population is difficult to estimate accurately although has been predicted to comprise about 1300 – 1500 individuals (Jefferson, 2005). The CWD population exhibit strong asymmetry in the distribution, but the encounter rate at West Lantau and Northwest Lantau is always high.

8.3.3 The other major ecological and fisheries receivers most likely to be impacted are coastal/ marine habitats and species and comprise the following:

- The Sha Chau and Lung Kwu Chau Marine Park;
- Horseshoe crabs (*Tachypleus tridentatus* and *Carcinoscorpius rotundicauda*);
- Mangroves, seagrass at the San Tau SSSI and Tai Ho Wan;
- Soft and hard corals;
- Benthic macrofauna;
- Intertidal and rocky shores;
- Artificial Reef at the Airport Exclusion Zones and also the Sha Chau and Lung Kwu Cha Marine Park;
- Fisheries spawning ground; and
- Marine fisheries.

8.3.4 The distribution of key ecology sensitive receivers are presented in *Figure 8.1* and the EIA Report for PAFF (Meinhardt, 2007a) includes a comprehensive review of the available ecological information.

8.4 Potential Environmental Impacts

8.4.1 *Introduction*

8.4.1.1 One of the major concerns for this proposed development is how to deal with the contaminated mud pits (CMPs) north of the Airport Island. The study area covers four major capped CMPs (including 11 sub-pits) containing at least 20 Mm^3 of contaminated material (*Figure 1.1*). With due consideration of the precision of the dredging process, the *Marine Mud Study for the Third Runway* (Meinhardt, 2006) estimated that disturbance to seven sub-pits (I, IIb, IIc, IId, IIIb, IIIc and IIId) alone may require handling (removal, handling, treatment and subsequent disposal) of 17.1 Mm^3 of contaminated material including any over-dredging required. Disturbance to the 11 sub-pits would, thus, involve a significantly higher quantity of contaminated material, if a traditional complete dredge and fill reclamation technique is assumed. This would be considered as the worse case scenario and would not represent the most environmentally acceptable option due to the predicted large amounts of dredged spoil involved and the need for subsequent offsite treatment/disposal. As such, this option is not considered further in this report

8.4.1.2 Twelve construction options that minimise the need to remove the contaminated material inside the CMPs and, hence, subsequent disposal, have been identified and discussed in *Section* 6. All the construction options would cause certain degrees of disturbance to the CMPs. It is possible that the floating structure may be able to avoid the CMPs as only a small footprint for the anchorage piles are is required but this will depend upon the final layout. For *in-situ* consolidation options not requiring dredging, the potential for environmental impacts would arise from the release / extraction of pore (interstitial) water contained inside the CMPs which is highly likely to be contaminated with heavy metals and/or organic contaminants, desorbed from the sediment. For other construction options requiring some dredging/excavation, release of plumes of contaminated sediment is another key issue. Backfilling during the reclamation process could also cause sediment plumes, but the impacts are likely to be less serve as the backfill material will be uncontaminated and comprise relatively coarse particles which settle rapidly by gravity.

8.4.1.3 The major potential environmental impacts associated with disturbance to the CMPs are further discussed in detail below and their relevance to the specific construction options are presented in *Section 8.6* below. It should be noted that, apart from the impacts identified below which are related to construction impacts, the formation of the reclamation will also cause loss of habitats to the Chinese White Dolphin and benthos, loss of fishing grounds (primary impact) and also indirectly affect offsite sensitive receivers (see *Section 8.1*) via potential water quality / hydrodynamic changes, etc. These impacts, however, are common to all the development methods and, as the objective of this preliminary assessment is to identify and establish the significance of any environmental impacts that may be expected as a result of the interface of the various construction options with the CMPs, these potential "operational" impacts are not discussed further in this report.

8.4.2 *Contaminated Pore Water*

8.4.2.1 The initial removal of contaminated sediment from the donator site and subsequent disposal to the CMPs will increase the bulk volume of the disturbed sediment and a nominal bulking factor of 1.3 is applied in Hong Kong. This means the disposal volume is estimated to increase by about 30%. As the dredging and disposal processes are in open water, this means that the expanded void is mostly filled with water and, thus, the pore (interstitial) water of sediment placed inside the CMPs would be higher than in the surrounding undisturbed seabed environment. With time, the chemicals / contaminants adsorbed to the sediment will equilibrate with the pore water and, hence, the sediment bound contaminants will also enter the water phase and contaminate the pore water.

8.4.2.2 When a load is placed on the sediment, as is the case during backfilling or reclamation, the pore pressures increase. Then, under site conditions, the excess pore pressure dissipates and water leaves the sediment, resulting in consolidation settlement. This process takes time and the rate of settlement also decreases over time. Because contaminated sediment is usually fine-grained and has a relatively high water content, it is often susceptible to large amounts of consolidation, a phenomenon that entails the squeezing/pressing together of sediment particles as pore water is expelled under sustained loading.

8.4.2.3 The capped and sealed CMPs have already undergone the initial consolidation by self-weight and the capping and a proportion of the pore water will already have been dissipated over time. However, for the construction options that involve the further consolidation of the contaminated materials after placement of fill and loading material, further expulsion of pore water would occur. This is a potential concern as, while the physical sediment particles are not being removed, the contaminants that are in the dissolved form can still be released into the water column and could be more easily dispersed by the current and affect the water quality of a wider area than the sediment bounded form which would settle relatively quick by gravity. The dissolved contaminants are also more readily absorbed by biota and, hence, represent a potential risk to the marine ecology. The potential toxicological impacts to the higher consumer like humans and Chinese White Dolphins need to be considered as many of the heavy metals and persistent organic contaminants can bio-magnify and accumulate through the food-web.

8.4.2.4 The amount of a contaminant that is dissolvable in the pore water is characterised by its partition coefficient, $K_{d,}$ which can be defined as the ratio of contaminant concentration associated with the solid to the contaminant concentration in the surrounding aqueous solution. The reciprocal of K_d represents the amount of contaminant leachable from the solid under equilibrium conditions. K_d is contaminant and site specific and also depends on the type of solid (i.e., sediment) and environment (e.g., temperature and pH), but typical values applied for the CMPs (CED, 1997 and CED 2005) are summarised in *Table 8.11* below. As indicated in *Table 8.11*, the K_d values of most contaminants relevant to the CMPs and, hence this study, are fairy high indicating that the pore water inside the CMPs is less likely to be contaminated with metals and organic PCBs, but more likely to be contaminated with organic PAHs which has relatively low K_d.

Table 8.11 Typical Partition Coefficients of Contaminants

Contaminant	Partition Coefficients, K_d (l/g)
Metals	
Cadmium	100
Chromium	290
Copper	122
Mercury	700
Nickel	40
Lead	130
Silver[1]	200
Zinc	100
Metalloid	
Arsenic	130
Organics-PAHs	
Low Molecular Weight PAHs	0.075

Contaminant	Partition Coefficients, K_d (l/g)
High Molecular Weight PAHs	1.14
Organics-non-PAHs	
Total PCBs	1585 (l/gOC)

Note: OC (organic carbon) = 0.012g OC/g; Source: CEDD (2005a).

8.4.2.5 As mentioned above, the CMPs, by design, cause the expulsion of pore water to occur naturally without containment. In order to assess the potential impact of the pore water dissipation as a result of the mud disposal and capping activities, the *Environmental Impact Assessment Study for Disposal of Contaminated Mud in the East Sha Chau Marine Borrow Pit* (i.e., CMP IV; CED, 1997) conducted a consolidation analysis based on calculation. Based on the design of 6m thick capping material (1m sand cap, 2 m initial mud cap and 3m additional mud cap), and with a 21m thick layer of contaminated mud for CMP IVa-b and a 13m thick layer of contaminated mud for CMP IVc, the estimated total primary consolidation depth due to self-weight, including the cap layer, was 3.8m and 2.5m for CMPs IVa-b and IVc, respectively. The estimated associated peak pore water expulsion rate was 0.6 $m^3/m^2/yr$ for all the three sub-pits and this was estimated to occur during the first year, but with a rapid rate of decay thereafter (see *Table 8.12*).

Table 8.12 Mud Thickness and Prediction of Pore Water Expulsion at CMP IV

Parameters	CMP IVa & IVb	CMP IVc
Additional Mud Cap (m)	3	3
Initial Mud Cap (m)	2	2
Sand Cap (m)	1	1
Contaminated Mud Layer (m)	21	13
Total Fill Thickness (m)	27	19
Primary Consolidation (m)	3.8	2.5
Peak Pore Water Expulsion Rate ($m^3/m^2/yr$)	0.60	0.60
Pore Water Expulsion Rate After 50 years ($m^3/m^2/yr$)	0.01	0.00

Source: CED (1997)

8.4.2.6 The estimated elevations of contamination concentrations in the water column as a result of the pore water expulsion are summarised in *Table 8.13* below and the percentage increases were predicted to range between 0.0022% (for mercury) to 0.87% (for chromium). These small percentage increases are not significant and within the natural background range of fluctuation. The 1997 EIA report assumed full thickness of the capping material to be placed before any consolidation of the contaminated fill material would take place. It, also, assumed natural tidal exchange of the overlaying water column to be daily instead of approximately hourly based on the hydrodynamic of the site. Thus, the estimates were conservative and gave higher rates of consolidation, quantities of pore water expulsion and elevation of contaminants in the water column.

Table 8.13 Mean Background Water Column Metals Concentrations and Percentage Increase Per Day in Concentration Due To Peak Fluxes

	Cd	Cr	Cu	Hg	Ni	Pb	Zn
Background Concentration (ug /L)	0.37	1.6	7.0	1.8	4.4	11.0	17.0
Increased in Water Column Concentration (ug /L)	0.00026	0.0014	0.014	0.00004	0.0098	0.018	0.12
Percent Increase (%)	0.07	0.087	0.23	0.0022	0.22	0.16	0.7
Updated background (ug/L)[1]	<2	0.5 – 0.6	1.5 – 2.8	<0.1	0.5 – 1.2	0.5-0.8	5.1-12

Source: CED (1997) and Meinhardt (2007b); [1]See Table 8.4 for updated background concentrations.

8.4.2.7 Since most of the interfacing CMPs under this Study have been capped and sealed for years (the latest being CMP IVa in which disposal operation was completed in 2000), it is predicted that the contaminants in the pore water would have equilibrated with the sediment and contain the contaminants previously bound to the sediment. As predicted in the 1997 EIA report for CMP IV, a portion of the contaminated pore water is predicted to have already have been expelled as a result of the natural consolidation of the mud in the pits due to its own weight. However, it is important to note that the release of this contaminated pore water has not resulted in any significant elevation in contaminants in the water column (see last row in Table 8.12), as demonstrated by the on-going EM&A for the CMP IVs (e.g., Meinhardt, 2007b). The anticipated low expulsion rate by self weight consolidation, possible cleaning effects of the clean capping (the contaminants in the pore water can be adsorbed to the clean capping mud which is exactly the reverse process as the dissociation from the contaminated mud), the availability of three dimensional escape pathways (the pore water expulsion path is not limited to the cap layer), and notably, the large dilution effect of the overlaying water column could all have contributed to the low level of impacts that have occurred to date from the pore water expulsion from the self-weight consolidation.

8.4.2.8 However, in terms of the construction methods being assessed, it should be noted that, while the contaminant concentrations in the pore water should be the same, as discussed in *Section 6*, some of the construction options involve accelerated consolidation through the use of drains and loading that would expedite the pore water expulsion rate ("**dispersion**") and, hence, potentially increase the amount of contaminants released over a period of time (i.e., increase the percentage of background elevations). There are also options (e.g., vacuum consolidation, electro-osmosis, deep well dewatering) that accelerate the process of pore water removal by mechanical extraction the pore-water out ("**extraction**") that may require a separate consideration and the distinction between "dispersion" and "extraction" are further discussed below.

8.4.2.9 Notwithstanding, the accelerated nature of the consolidation and associated and pore water expulsion, the dispersion of the pore water will also be mostly unidirectional through the caps due to the introduction of specific pathways (i.e., the drains), rather than three dimensional as the case under natural consolidation. However, it is expected that the potential effects of the expedited contaminated pore water dispersion under these accelerated processes would be subject to significant reduction through dilution. Based upon an average water speed over the pit of around 0.2m/s, and an average water depth of around 7m, then, on average, 1.4$m^3/s/m^2$ of water flows over of the pit. In a year, this is equivalent to 44,000,000m^3/m^2/year. Based upon the CED 1997 EIA results, the pore water loss rate was expected to peak at 0.6m^3/m^2/year under the natural consolidation process, and the loss of pore water in this case would, therefore, be predicted to be roughly 1/73,000,000th of the tidal flow which will dilute it. Assuming the accelerated consolidation processes increase the pore water rate by say 10 times, the loss of pore water is still around 7,300,000 smaller than the tidal flows over the pits. Thus, it would be reasonable to assume the accelerated release ("dispersion") of contaminated pore water would not be a major environmental concern given this significant level of dilution. We note that that the filling and release of pore water is a dynamic processes and the significance of the impacts would depend on the construction sequence. For example, when the site is almost filled, the release path would tend to follow the periphery of a permeable seawall where high natural dilution will rapidly disperse the contaminants. It is recommended that further site investigation, laboratory test and detailed hydrodynamic / water quality modelling study should be undertaken to investigate particular localized effects to ascertain whether these will generate significant localized environmental impacts. Meaningful modelling, however, would require full details of the likely detailed construction scenario.

8.4.2.10 However, when referring back to the predicted percentage of contaminants elevation presented in Table *8.13*, it is worth noting that the background contaminants level recorded in recent times are lower than the assessment level in the 1997 EIA. For example, the lead concentrations have reduced from 11.0 ug/l to around 0.8 ug/L. Thus, even with the same rate of release, the percentage of elevation would increase from 0.16% to 2.3% and with the accelerated methods, a higher percentage of elevation could be anticipated especially during stagnant conditions when tidal flushing is low. The localised effect, while not necessary environmental detrimental, could be a key issue for the Authorities and Green Groups. It could also be an issue in terms of the Water Quality Objectives (WQOs) as while these have no specified acceptable levels for most of the contaminants of potential concern under this Study, they do state that "*waste discharges shall not cause a risk to any beneficial use of the aquatic environment*" (*Table 8.1*), and there is no strict guideline on the interpretation clauses of similar type.

8.4.2.11 Notwithstanding, it is considered that the dilution effects are a notable factor and, as such, in order to further quantify the significance of the contaminated pore-water issue, a preliminary estimation of the likely contaminant concentrations in the pore water as well as released concentrations has been undertaken. The calculation has involved several assumptions that are set forth below:

- the metals and metalloid in the CMPs have equilibrated between the solid and solution phases (a reasonable assumption given the pits have been inactive for 8-15 years);
- the partition coefficients of the contaminants (Table 8.11) are representative of the site condition (an acceptable assumption for CMP V EIA);
- the highest concentrations of all the metals reported (Tables 8.9 – 8.10 of the report) in the capped pit are used in the estimation (a reasonable worse case assumption as about 30% of the samples were considered as Category M or H);
- the estimation has excluded organics as there is insufficient information for a reasonable estimation; and
- the average flow is assumed to be 0.2 m/s and the average water depth is 7m

deep water giving a dilution factor of about 3.6 million fold. The calculation of this dilution factor is presented in *Appendix F*.

8.4.2.12 Based upon the above assumptions, the estimated concentrations of metal and metalloid contaminants are presented in Table 8.14 below both, for before and after dilution. The calculation indicates that the pre-diluted pore water could be highly contaminated with heavy metals (e.g., copper) that would exceed the European Union Environmental Quality Standard (EQS) Values to Protect Marine Life (Table 8.2). Compared to the ambient environment, the contaminant concentrations in the pore water would be expected to be much higher. However, with even just with a one million fold natural dilution (as opposed to the over 3 million fold dilution predicted), the contaminated pore water, if released, would be expected to be barely traceable and, therefore, unlikely to induce measurable environmental change. Table 8.14 has been prepared by assuming 1 million : 1 dilution as a lower bound average likely scenario. For comparison 50 :1 dilution has been presented as an absolute worst case scenario assuming pore water could be come trapped before being released into the tidal flow.

Table 8.14 Estimation of Metals and Metalloid in Pore-water and Release

	Sediment	**Pore water**			**Release with 50 fold dilution**		**Release with million fold dilution[1]**	
Contaminant	**mg/kg dry wt.**	**ug/l**	**% of EQS**	**% of Ambient**	**% EQS**	**% Ambient**	**% EQS**	**% Ambient**
Scenario 1. ESC Pit IIa Max value								
Cd	2.4	0.024	1.0%	Na	0.019%	na	0.000%	na
Cr	200	0.690	4.6%	**115%**	0.092%	2.299%	0.000%	0.000%
Cu	800	6.557	**131.1%**	**234%**	2.623%	4.684%	0.000%	0.000%
Hg	0.5	0.001	0.2%	na	0.005%	na	0.000%	na
Ni	110	2.750	9.2%	**229%**	0.183%	4.583%	0.000%	0.000%
Pb	130	1.000	4.0%	**125%**	0.080%	2.500%	0.000%	0.000%
Ag	no data							
Zn	350	3.500	8.8%	29%	0.175%	0.583%	0.000%	0.000%
As	no data							
Scenario 2. ESC Pit IIb Max value								
Cd	1.9	0.019	0.8%	na	0.015%	na	0.000%	na
Cr	82	0.283	1.9%	47%	0.038%	0.943%	0.000%	0.000%
Cu	130	1.066	21.3%	38%	0.426%	0.761%	0.000%	0.000%
Hg	0.7	0.001	0.3%	na	0.007%	na	0.000%	na
Ni	73	1.825	6.1%	**152%**	0.122%	3.042%	0.000%	0.000%
Pb	140	1.077	4.3%	**135%**	0.086%	2.692%	0.000%	0.000%
Ag	no data							
Zn	290	2.900	7.3%	24%	0.145%	0.483%	0.000%	0.000%
As	no data							

Note: Please refer to Table 8.2 for EQS standard, Table 8.4 for typical contaminants concentrations in ambient water, and Tables 8.9 and 8.10 for contaminants concentrations in Pit IIa and IIb sediments; 1For illustration purpose, the calculation is used a million fold dilution factor but 3.6 million fold may be expected at site (see also Appendix F).

8.4.2.13 As demonstrated above, with the predicted high level of natural dilution, the environmental risk to water quality and marine ecology associated with the release of the potential contaminants inside the pore-water should be relatively low in general. Nonetheless, local hotspots could develop if flows are restricted by large structures such as peripheral seawalls which would lower the scale of the natural dilution. If a peripheral seawall is constructed before the main reclamation, the natural dilution could be restricted to the tidal change. However, the seawall will not be watertight and there will probably be a gap for access (100 m wide perhaps) so the tidal level behind the seawall will rise and fall more or less as normal. During each tide, any built-up of contaminated water caused by pore water expulsion will be flushed. Based on the typical average tidal range of around 1.5m in Hong Kong, about 1,500 mm of the water column within the seawall will be exchanged each tide. With the assumed settlement rate of 3m in 3 months (that is about 30 mm/day), this means a layer of pore water 30 mm thick might be expelled. Hence, about 50 (i.e., 30mm pore water / 1500mm water column exchanged) fold dilution may still be expected in this case. The anticipated release concentrations under this reduced dilution has also been presented in Table 8.14 above and the results also indicate insignificant environmental effects under the reduced dilution situation.

8.4.2.14 It should be noted, that while the assumptions adopted in the above calculation are reasonable, they have been derived from limited field data or from laboratory experiment. The high level of dilution would also be applicable to the pore-water **dispersion** process only. While this dilution factor may also apply to the pore-water **extraction** process as both would occur under the same ambient environment, with the extraction process, the pore-water will be mechanically removed from the sediment through highly specific pathways and likely with very little dilution. As indicated in Table 8.14, the undiluted pore water can be highly contaminated and, as such, direct discharge of the extracted pore-water into the ambient environment would unlikely be acceptable from the perspective of regulatory authority (ie once collected, the subsequent release of the pore water would be considered as a discharge, this is as opposed to the “natural” dispersion of the pore water albeit at an accelerated rate). Any analogy to exemplify this case is the discharge of work force related domestic sewage. With the high natural dilution in the area, a direct discharge of the sewage will unlikely cause any noticeable impacts to the environment. However, this will unlikely be allowed without any treatment.

8.4.2.15 For the purpose of comparing the construction options, the key issue is whether the pore-water is to be **dispersed or extracted**. In summary, in terms of the dispersion process, there is a probability that detectable amounts of contaminants in the pore water could be released during the accelerated consolidation, which would most likely to be in the form of heavy metals as exemplified by the limited site data from CMP IIa and IIb (Tables 8.9 – 8.10). While dilution into the larger water body and tidal flushing would reduce the concentrations of contaminants, in theory, biota can still bio-magnify and accumulate contaminants under chronic exposure. However, based on the dilution calculations above, the environmental risk is more likely perceived rather than realistic. The high levels of natural dilution predicted shows that the issue of contaminated pore-water release through dispersion should not present a key factor in the comparison of various construction options proposed and assessed. It is recommended, however, that some on-site testing data are collected and further quantitative assessment be conducted as this could remain a sensitive issue to EPD and Green Groups and be the focus of comments during public consultation. In terms of the extraction process, however, subsequent discharge will unlikely be allowed and further treatment would, therefore, be required.

8.4.3 *Contaminated Sediment Plumes*

8.4.3.1 For options causing removal (or allowing escape) of some of the contaminated materials placed inside the CMPs, there are two major environmental issues; sediment plumes and the subsequent treatment/disposal of any extracted material. This section focuses on the sediment plumes and the treatment/disposal issue is discussed in below.

8.4.3.2 Physical disturbances to the CMPs during the construction process can cause re-suspension of the contaminated sediments, that is, dislodgement and dispersal of sediment into the water column and the finer particles and flocs are subject to transport and dispersion by currents. The contaminated sediment plume would have the usual properties of any sediment plume and the potential environmental impacts of dredged sediment plumes are well understood (e.g., Clarke & Wilber, 2000; Meinhardt, 2007a). The major issues are:

- reduction of dissolved oxygen in the water column;
- reduced light penetration and affect the photosynthetic process;
- clogging of gills and other organs causing suffocation;

- mechanical damage to the gills and other filtering structures;
- reduction of feeding efficiency leading to reduced growth;
- deposition and smother benthic organisms;
- reduction of underwater visibility and impairment to organisms depending on visual cue; and
- direct mortality (e.g., corals, benthos and fish) in extreme cases.

8.4.3.3 The biological effect of suspended solids, however, should not be exaggerated, however, as studies of local fish, shrimp, amphipod, barnacle and diatom species have suggested that organisms can sustain high level of suspended solids (>5,000 mg/L) over 2 days without an observable mortality response (CCPC, 2001).

8.4.3.4 For plumes of contaminated sediment, the additional environmental concern is the release of contaminants associated with the sediments. There are two phases of contaminant release from the sediment plume. The first phase involves the release of contaminants already in the dissolved form (i.e. the pore water) and this would happen rapidly during the initial sediment release (dredging or rupture of capping layer). The second phase comprises the slow release of sediment bound contaminants, which can be either adsorbed, absorbed or chemically bound. This second phase takes time and the rate and amount of release will depends on both the contaminants involved and the ambient environment. However, the second phase could have a longer lasting effect as chemical dissolution will continue after the dispersed sediment has settled on receiver sites.

8.4.3.5 The potential impacts of contaminants release from the sediments will be similar to that of contaminated pore water. As the release process is likely to be slow, acute toxicological effects (e.g., mortality and physical impairment) to the ecological sensitive receiver would be less likely. However, sub-acute chronic effects (e.g., reduction of growth, reduction of fertility, cancer, etc.) to the ecological sensitive receivers could be a long term issue.

8.4.3.6 The significance of environmental impacts of the contaminated sediment plumes will largely depend on both the quantity of sediments involved and the concentration of contaminants contained. It may be argued that, as the dredging release and sediment plumes can be largely controlled (e.g., by reduction of dredging rate, by use of closed grab dredger, application of silt curtain/screen, etc.), it would be ultimately possible to minimise any release to an acceptable level or else the initial dredging at donator sites would not have been allowed in the first place.

However, while control(s) to minimise this release to an acceptable may exist, it may not be reasonably practical as it may significantly lengthen the overall construction programme rending it not desirable and this would have to be explored.

8.4.3.7 For the purpose of comparing the construction options, options not disturbing the contained contaminated material are environmentally more favorable. For options where disturbance to the contaminated material is unavoidable, disturbance under a controlled environment (e.g., behind seawall) will be more favourable over those in the uncontrolled environment.

8.4.4 *Contaminated Sediment Disposal/Treatment*

8.4.4.1 The second major issue associated the removal of the contaminated materials is subsequent disposal/treatment. As mentioned in Section 8.1, the removed sediments would need to be properly managed. In Hong Kong, the required level of management for dredged spoils ranges from open disposal to special treatment depending on the contamination, and recently also on the results of bioassay under ETWB TCW 34/2002. If strict compliance to ETWB TCW 34/2002 is considered necessary by the Government, then any removed sediments would need to be re-tested and classified. However, as highlighted in Meinhardt (2006), the sediments placed inside the CMPs were previously classified as contaminated requiring confined marine disposal (i.e., CMPs) and therefore, whether re-classification would be acceptable would need to be addressed. It may be appropriate to address the issue of reclassification of the sediments with CEDD however such an approach lies beyond the scope of this assignment and could not be achieved within the timescale of this study.

8.4.4.2 Regardless of the acceptability/need of reclassification, it will be expected that a reasonable proportion of any disturbed sediments inside the CMPs would be contaminated (about 20% based on the limited data available from CMP IIa-b, (Tables 8.9-8.10) requiring disposal at another CMP. It could be possible that, in the extreme case where highly contaminated material is encountered, special treatment would be required. The currently accepted practice (applied in the Wan Chai Development Phase II in 2003) is to contain the highly contaminated material within geotextile containers before disposal into a CMP. This practice, however, may not be acceptable based on recent experience, with CEDD commenting that the possibility to reuse of the dredged material shall be explored whenever possible.

8.4.4.3 Unfortunately, the available CMP disposal space is fairly limited and requests for allocation of disposal space is closely scrutinised by the Marine Fill Committee (MFC). The currently in use CMP IVc is due to be exhausted by 2009. The planned South Brothers Facility has a predicted total design capacity of 8 Mm^3 (CEDD, 2005a) and is expected to be in use by 2009, with a predicted usage time of about 4.5 years, based on the forecast at the time of the EIA study). As the prediction was made in 2005, it would unlikely have taken into consideration the possible disposal need for the 3rd Runway project. Assuming the alternative site next to the CMP IVc can be put into use after the South Brothers Facility has been exhausted, there could be another 8 Mm^3 capacity available for another 4.5 years. However, the acceptability of this could be highly uncertain as it is known that the local community has a strong objection to the continued development of CMPs along the North Lantau coast.

8.4.4.4 It is important to note the 10 years CMP demand statistics published in June 2005 (MFC, 2005) only anticipated requests for 6.4 Mm^3 of disposal space with virtually no disposal demand in 2011. The latest available 10 years CMP demand statistics published in June 2006 (MFC, 2006), however, has adjusted this figure upwards to about 8.7 Mm^3, but it is likely that the demands could increase substantially with the implementation of the Hong Kong–Zhuhai–Macao Bridged (HZMB) related infrastructure such as the Hong Kong Boundary Crossing Facility (HKBCF), Hong Kong Link Road and also the Tuen Mun Chek Lap Kok Link (TMCLKL) where some disposal demands can be anticipated. An extract of the relevant statistics form Marine Fill Committee's annual fill requirement and surplus statistics is presented in *Table 8.15* below.

Table 8.15 Annual Demands of Dredged Mud for Type 2 Confined Marine Disposal

	Publication Time	
Year	**June 2005 (Mm^3)**	**June 2006 (Mm^3)**
2005	1.334	-
2006	1.577	0.992
2007	1.608	1.721
2008	0.937	0.953
2009	0.325	0.955
2010	0.621	2.249
2011	0	1.114
2012	0	0.728
2013	0	0.021
2014	0	0.016
2015	0	0
2016	-	0

	Publication Time	
10 Years Total	**6.402**	**8.749**

Source: MFC (2005, 2006)

8.4.4.5 For the purpose of comparing the construction options, options not requiring removal of the contaminated materials are environmentally more favorable as they would exerts less pressure on the waste management issues.

8.4.5 *Reclamation*

8.4.5.1 Apart form the dredging works, sediment plumes could also arise during the reclamation process affecting the water quality of the surrounding area. Also as discussed previously, reclamation could change the coastline configuration and affect the hydrodynamic and water quality of the area. Furthermore, the marine habitats of the reclaimed area will be permanently lost. However, these are common issues to all reclamation projects and not peculiar to reclamation works over CMPs. The study area for this Assignment is the same for all construction options and thus, it is assumed that the reclamation footprint is also the same. Thus, reclamation shall not be a comparative environmental criterion. Nonetheless, as there are also options requiring substantially smaller reclamations or no reclamation at all, it is considered necessary to include this for comparison.

8.5 Mitigation Measures

8.5.1 *Background*

8.5.1.1 This section briefly outlines the mitigation measures available to abate some of the environmental issues discussed above. It is, however, important to note that the applicability and effectiveness will depends on the construction options being considered, the contaminants to be controlled as well as the hydrodynamic of the site. Since most of this site specific and design information is not available at this stage, the appropriateness of the measures cannot be confirmed at this stage. Furthermore, while these measures are to abate the environmental issues associated with disturbance to the CMPs, although many of these mitigation measures can also be considered for the other parts of the project area where marine works are required. Palermo et al. (2008) has recently published the *Technical Guideline for Environmental Dredging of Contaminated Sediment* which has in-depth discussions of the experience learnt in the USA.

8.5.1.2 It is also likely that no single solution will work for the whole project and a combination of mitigation methods will be required. Detailed hydrodynamic and water quality modelling will help to determine the best combination and schedule of the works to ensure the environmental impacts are controlled to an acceptable level.

8.5.2 *Minimisation of Disturbance to CMPs*

8.5.2.1 From the perspective of environmental protection, especially for this project which is located at the core habitat for the internationally protected endangered Chinese White Dolphin and also with the presence of other locally rare species (e.g., such as seagrass and horseshoe crabs) nearby, avoiding disturbance to the CMPs shall the first consideration and it is noted that this is the key objective of this Assignment. When it is not practicable to completely avoid disturbing the CMPs, minimization of disturbance should be the second priority. This could be by means of *in-situ* treatment or dredging under a contained environment. However, there is a need to consider the balance between minimisation the quantity of pollutants being released and the overall duration of impacts.

8.5.3 *Controlling the Release of Contaminated Sediment Plumes*

8.5.3.1 The common approaches in Hong Kong to controlling the release of sediment plumes include:

- *Restriction of dredging rate*. This is commonly required for projects in the North Lantau, including the dredging and disposal operation at the East Sha Chau CMPs;

- *Restriction on the number of dredgers being used.* This is to control the number of simultaneous plume generating sources, but it effectively also control the dredging rate; and

- *Restriction of dredging equipment.* The instantaneous sediment loss rates associated with grab dredging are likely to be less than that for trailer dredging. Thus, the intensity and extent of a sediment plume emanating downstream of a working dredge is likely to be less for a grab than a trailer and thus trailer suction dredgers are generally regarded as being more environmentally damaging than grab dredgers. However, the plume from a trailer dredger will predominantly be formed at depth near the seabed, close to the drag head whereas grab-dredging plumes, though less concentrated, are likely to originate throughout the water column as the grab is pulled to the surface. The loss during the pull up can be controlled with the use of a water tight close grab. Trailing suction hopper dredger (TSHD) is mostly allowed for dredging works at navigation channel to minimise impacts on navigation safety.

8.5.3.2 The above approach is applicable when dredging is required although the construction options proposed in this report have minimal dredging requirements. Nevertheless, it is also applicable during the reclamation filling process.

8.5.4 *Controlling the Spread of Contaminated Sediment Plumes*

8.5.4.1 The most recognised control to the spread of sediment plumes is the use of silt curtains. These are generally used at the project boundary to ensure the plumes are contained inside the site. Apart from enclosing the entire project site, they are also commonly deployed around the dredgers (e.g., frame type) to minimise the initial release. Silt curtains can also be deployed at sensitive receivers to protect them from accidental plumes. For areas within deep water or strong current/flow, however, effective deployment of silt curtain may be difficult. Favouring parameters like small flows and shallow water are among the reasons for East Sha Chau being the preferred disposal grounds for contaminated material. These environmental conditions would also allow the effective application of silt curtains to be used to contain sediment plumes during this project. Prior to the adoption of alternative disposal procedures for CMP IIId, silt curtains encroaching the pits were deployed. Overall, it can be expected that silt curtain can be deployed effectively, if required, for the 3rd runway project.

8.5.4.2 Reclamation projects usually involve seawall construction. There is a recent increase of projects proposing to construct the peripheral seawall first before the main reclamation. For example, this has been applied in the Pak Shek Kok reclamation at Shatin and the Reclamation for the Disney Theme Parks at Penny’s Bay and it is also being proposed for the Lantau Logistic Park near Tai Ho. The peripheral seawall would effectively contain the sediment plumes within the project site, except the approach channel which is often gated with silt curtain. Palermo et al (2008) recommend the use of structural barriers be considered where there is a need to contain sediment plumes that contain highly mobile, highly toxic, or bioaccumulative contaminants, and when there is uncertainty that a silt curtain will be effective. It is engineering feasible to construct the peripheral seawall first before the main reclamation for the 3rd runway and the engineering details are described in *Section 6*.

8.5.4.3 Other structure barriers, such as sheet-pile walls, have also been used in some cases, but this is normally considered in shallow works area like rivers. For example, a sheet-pile enclosure was used for the pile construction of the Hong Kong Shenzhen Western Corridor as an additional measure to control sediment plumes. Another common form of structural enclosure is metal casing. A typical application is for bored piling where a metal case is inserted before the material inside is extracted. Their application, however, often is related to structural need of the construction rather than primarily for environmental control.

8.5.4.4 In addition, by restricting the dredging operation to the tidal state when the flow is minimal (usually during high and low water), the plume would not disperse significantly and would be expected to settle rapidly around the site. This is not a common practice, but a similar principle is adopted for the disposal operation at CMPs which effectively negates the need for a silt curtain. Silt curtain has been a standard mitigation method disposal/backfilling at CMP I-III. However, Mouchel developed an alternative backfilling procedure for CMP IIId in which the dumping always takes places on the up-current side of the pit. This allows the plumes sediment to settle as it is carried across (down-current) the pit by the tidal current and this process is now adopted for disposal activities at CMP IV as well.

8.5.5 *Controlling Contaminated Pore Water*

8.5.5.1 For regular dredging works, there are no specific controls usually applied to control the release of contaminated pore water. However, mitigation measures to control the release/spread of sediment plumes will also help to reduce the release of contaminated pore water when dredging / removing contaminated sediments is required.

8.5.5.2 For construction options in which the contaminated pore-water is **dispersed**, the high natural dilution effect should render the water quality impact non-detectable and negligible and, hence, no further treatment or mitigation would be required. However, as these options do not allow collection of pore water under controlled conditions, it is possible that the Authorities adopt a precautionary stance and may still insist to implement certain forms of treatment to minimise the amount of contaminants released into the environment as much as possible as a preventative mitigation measure. Under such a scenario on-site treatment with adsorbents may be a solution but it should be noted that these methods are largely untested. Dissolved contaminants may be removed by dispersing adsorbents which will strip the contaminants from the water column. However, this approach would need to contain the pore water for effective application and also avoid releasing the adsorbents themselves into the environment.

8.5.5.3 Dissolved contaminants may also be removed by designing the containment enclosures to be adsorbent. Filtering geotextiles with adsorbents have been developed for application with silt curtains to prove permeable sections that act as layers of filters and adsorbents to treat water passing through the dredging site. Palermo et al (2008) suggest these innovations might be viable controls when the operation poses unacceptable risk. Filtering geotextiles with adsorbents, however, can be considered as experimental and there is very limited information about their large scale application. Significant research and development effort would likely be required to adopt this mitigation measure to the local setting and demonstrate its effectiveness to the Government.

8.5.5.4 Structural barriers discussed above to contain sediment plumes can also be considered to contain the expelled contaminated pore water. From an environmental perspective, structural containment is preferable over adsorbents as there is then no risk of secondary impacts due to the adsorbents. In order to control the pore water dispersion, however, the structural barriers would have to be impermeable rending large scale application difficult in the marine environment.

8.5.5.5 For construction options (e.g., vacuum consolidation, electro-osmosis and deep well dewatering) in which the contaminated pore-water is **extracted**, discharge back to the ambient without treatment will unlikely be acceptable to the government as discussed in Section 8.4. A possible constraint on the treatment and disposal of the collected contaminated pore water is the Chemical Waste Treatment Centre (CWTC) at Tsing Yi. The CWTC has a design capacity of 100,000 tonnes per year. Dedicated treatment works for the project may also be considered although this may not be cost-effective as based on the engineering prediction, the bulk of the pore water would be expelled in the first three years of works implementation. In addition, the construction of a project specific chemical treatment/disposal facility may also be another designated project entailing additional statutory processes.

8.5.6 *Treatment of Contaminated Sediment*

8.5.6.1 Under the ETWB TCW 34/2002, disposal of contaminated materials in a CMP is considered as an acceptable treatment. However, as discussed above, the available disposal space at the time of the project implementation is uncertain and early discussion with the FMC to seek space allocation would be important. In the case where significant quantities of contaminated material is involved or highly contaminated material is involved, a project specific treatment facility may be a viable option. A tentatively feasible site for project specific confined disposal facility/confined aquatic disposal (e.g., CMP) has been detailed in the Marine Mud Study for the Third Runway (Meinahrdt, 2006).

8.5.6.2 Should special treatment be considered necessary, there also exist numerous treatment technologies for consideration. Many of these technologies have been developed for treating contaminated soils with hazardous waste, especially those developed in the USA. Some of the better-established technologies, particularly those that have been demonstrated on contaminated sediments, have been reviewed in USEPA (1994). While the USEPA (1994) can be considered as slightly outdated, the technologies considered in the review have been bench-scale tested and can be considered as more reliable and available. An up to date report of the use and status of treatment technology is presented in USEPA (2007).

8.5.6.3 Experience of off-site treatment in Hong Kong, however, is fairly limited and the only large scale application is the temporary thermal treatment works at To Kau Wan for the reclamation works at Penny's Bay. Similar to a project specific off-site treatment facility for collected pore water, an off-site project specific sediment treatment facility could entail another EIAO Designated Project. Identification of an appropriate site would also be a constraint.

8.5.6.4 It is important to note that many of the construction options considered in this Report comprise or incorporate elements of *in-situ* treatment technologies, specifically to reduce the issues related to the removal and treatment of contaminated sediment.

8.6 Preliminary Environmental Assessment of Construction Options

8.6.1 *Introduction*

8.6.1.1 *Section 6* of the report described the various construction options and has concluded that all options are feasible with the exception of the electro-osmosis. However, for completeness, all options including the electro-osmosis have been considered from an environmental perspective in this section. The major environmental compartments likely to be directly affected in relation to interface with CMPs and the possible mitigation measures are discussed in *Sections 8.2 – 8.4* above. This section provides a preliminary assessment the potential direct impacts associated with each construction options and also an evaluation of the likelihood of successful mitigation, should the field sampling and testing confirm mitigation measures are required.

8.6.1.2 For the purpose of comparative environmental assessment, the environmental performance of the construction options considered are ranked on the following scale: Very Poor, Poor, Fair, Good and Very Good. It must be noted that this is descriptive only for the purposes of comparative ranking and is not an indication of the environmental acceptability of the construction option or the project as a whole. The actual environmental acceptability of the project as a whole will depend upon the construction methodology finally selected. This will be the subject of a more detailed study at such time as the project becomes better defined.

8.6.2 *Sand Blanket*

8.6.2.1 For the various options considered, often, a sand capping layer (sand blanket) of at least 1.5m thick will be required, covering the whole project area, as a pre-treatment in order to provide sufficient ground stability for the construction equipment. One important function of the sand blanket is to provide a drainage layer for the pore water dissipation. During the placement of this layer, there will be some disturbance to the seabed at the interface, which could possibly give rise to a small sediment plumes. The sand fill itself, however, would unlikely be the source the sediment plumes as it will consist of coarse material. This process is similar to capping of the CMPs to seal it off and significant impacts to the water column from the capping activities have not been recorded in the on-going EM&A works for the CMPs (e.g., Meinhardt, 2007b). However, unlike the capping of the CMPs using bottom dumping, the layering of the sand blanket would be even more controlled using a spreader pontoon which would be expected to have a much reduced disturbance effect. Furthermore, as the interfacing material will be generally uncontaminated capping material or seabed, the anticipated environmental impacts would be expected to be very low in comparison with the capping of CMPs in which the interfacing material is contaminated mud.

8.6.2.2 The layering of the sand blanket would also initiate further consolidation of the CMPs and lead to further expulsion of contaminated pore water. Since the thickness of the blanket (about 1.5 – 3 m) is about comparable to the caps of the CMPs (3-6m of sand/mud), the concentrations of contaminants released would likely be similar to prediction of the EIA for CMP IV (Table 8.13) and be considered acceptable.

8.6.2.3 With the application of additional ground treatments, the sand blanket would act as a medium for the dissipation of pore water. Most of the accelerated consolidation processes will target to achieve 90% consolidation (i.e., about 3m) within a short time frame of 3 to 6 months. This is considerably much faster than the scenario considered in the EIA for CMP IV and as such, the total amount of pore water expelled and the rate of expulsion could be significantly higher as discussed in Section 8.4.2. While, the 1997 EIA estimation did not consider additional processes to accelerate the CMP consolidation and, thus, expedite pore water expulsion, the dilution effects detailed in Section 8.4.2.10 demonstrate that impacts from pore water dispersion are unlikely to be significant. The environmental effect of the sand blanket formation is considered as “neutral”, for the purposes of this assessment, as the sand substrate is a natural seabed material in Hong Kong and significant sediment plumes are not anticipated for its formation.

8.6.2.4 The designation of “neutral”, however, assumes that the required sand fill can be imported from places outside Hong Kong. For a sand blanket of 2m thick in the study area (~ 5,580,000 m^2), around 11 Mm^3 of sand fill would be required. If this is to be sourced from a local borrow area, the potential environmental impacts associated with the extraction could be an issue especially if a near shore borrow ground is explored. Furthermore, a marine borrow area of any size or a land borrow area of more than 200,000 m^3 or more than 50,000 m^3 with specified sensitive receivers within 500m is a designated project on its own and would required a separate EIA study.

8.6.3 *Accelerated Consolidation - Prefabricated Vertical Drains*

8.6.3.1 Sediment Plume: The details of this option are described in *Section 6* and would be used in combination to the sand blanket. Subsequent to the formation of the sand blanket, prefabricated vertical drains (PVDs) are inserted into the CMPs to accelerate the mud consolidation. PVDs will penetrate through the sand blanket and capping layers of the CMPs into the main body of the CMPs. The insertion of PVDs would unlikely cause any sediment plumes as the interfacing material is coarse sand blanket. The extraction of the penetrating device (mandral), however, has the potential to cause minor sediment plumes due to the possible adherence of fine clay material to the outside of the device. It can be expected, however, that the sand blanket would filter most of the adhered clay and as such the sediment plumes should be minimal.

8.6.3.2 Sediment Removal and Treatment: There is no requirement for sediment removal for the installation of PVDs. Hence, there is also no need of sediment disposal or treatment, nor impacts associated with its removal.

8.6.3.3 Pore Water: The primary purpose of PVDs is to accelerate the sediment consolidation and, hence, pore water expulsion. Typically, the design of PVDs (size and quantities) will aim at achieving 90% of the required consolidation in the first 3 to 6 months. The pore water will be dispersed through the sand blanket but is expected to be highly diluted by the natural tidal currents and significant impacts are not anticipated.

8.6.3.4 Mitigation Measures: In general, other than good site management, no specific environmental mitigation measures are recommended as there is minimal removal of contaminated sediments. The primary consideration for mitigation measures will be to minimise the generation of sediment plumes during the main reclamation filling. This could be achieved by the use of silt-curtain or constructing the peripheral seawall first before the main reclamation works or a combination of silt-curtain and seawall containment.

8.6.3.5 Key Environmental Concerns: This construction method is not likely to cause significant environmental impacts. The main concern is likely to be the sediment plumes generated during the main reclamation filling but these should be able to be controlled.

8.6.3.6 Comparative Environmental Ranking: The environmental performance of PVD is rated as "Fair". The process itself is not likely to induce significant environmental impacts. The ranking for PVD is also considered as the base case for comparison and the environmental performance of other options are, thus, largely relative to PVD.

8.6.4 *Accelerated Consolidation - Sand Drains*

8.6.4.1 Sediment Plume: The details of this option are described in *Section 6.* A sand drain is essentially the same as PVD, but a sand column is used to provide the drainage path instead of geo-textile material. Since, the placement of sand column is through a steel tube (casing) in a controlled manner and sand is generally consists of heavier particle, the use of this material is not considered of environmental concern. Similar to the PVD, the insertion of metal tubing and its final withdraw could lead to a minor sediment plumes as described in the PVD section. Thus, the environmental performance and environmental concerns in terms of sediment plumes and pore water dispersion are the same as PVDs described above.

8.6.4.2 Sediment Removal and Treatment: There is no requirement for sediment removal for the installation of sand drains similar to PVD. Hence, there is also no need of sediment disposal or treatment, nor impacts associated with its removal.

8.6.4.3 Pore Water: Similar to the PVD, pore water will be dispersed through the sand blanket but is expected to be highly diluted by the natural tidal currents and significant impacts are not anticipated.

8.6.4.4 Mitigation Measures: In general, other than good site management, no specific environmental mitigation measures are recommended as there is minimal removal of contaminated sediments. The primary consideration for mitigation measures is similar to PVD.

8.6.4.5 Key Environmental Concerns: This ground improvement method is not likely to cause significant environmental impacts. The main concern is likely to be the sediment plumes generated during the main reclamation filling but these should be able to be controlled.

8.6.4.6 Comparative Environmental Ranking: The environmental performance of is rated as "Fair" similar to PVD.

8.6.5 *Sand Compaction Piles*

8.6.5.1 Sediment Plume: The details of this option are described in *Section 6* and will be used in association with the sand blanket. Subsequent to the formation of the sand blanket, steel tubes would be inserted through the sand blanket and caps of the CMPs into the main body of the CMPs. Sand material is then injected in the steel tubes under high pressure. The steel tubes are gradually extracted with the formation of sand compaction piles (SCPs) underneath. The insertion of the steel tubes themselves would be similar to the PVDs and would unlikely cause any sediment plumes as the interfacing material is the coarse sand blanket. The extraction of the penetrating device (steel tube), however, has the possibility of cause minor sediment plumes due to possible adherence of fine clay material to the outside of the device. It would be expected that the sand blanket would filter most of the adhered clay, however, and as such the sediment plumes should be minimal. Ground heave (see *Section 6)* may be formed causing engineering difficulties, but as the injected material is clean sand, impacts from any sediment plumes are not expected should ground heave develop.

8.6.5.2 Sediment Removal and Treatment: There is no requirement for sediment removal for the installation of SCPs. Hence, there is also no need for sediment disposal and treatment, nor impacts associated with its removal.

8.6.5.3 Pore Water: SCPs will accelerate the sediment consolidation and, hence, pore water expulsion. The accelerated consolidation rate is similar to PVD and, hence, due to large dilution effects, significant impacts are not expected.

8.6.5.4 Mitigation Measures: In general, other than good site management, no specific environmental mitigation measures are recommended as there is minimal removal of contaminated sediments. The applicable mitigation measures for the control of sediment plumes and their effectiveness are the same as PVDs. As this technique does not need surcharge material to be applied, the SCPs have some advantage over those methods that do and can be considered as a mitigation measure to the fill / waste management.

8.6.5.5 Key Environmental Concerns: This construction method is not likely to cause significant environmental impacts. The major concern is likely to be the potential for sediment plumes generated during the main reclamation filling. Compared to ground improvement technique like PVD, this technique has the advantage of not requiring additional surcharge after the reclamation. This minimises the need to import and transport additional fill material, and its subsequent removal.

8.6.5.6 Comparative Environmental Ranking: The environmental performance of SCP is rated as "Good" in recognition of the fact that it can significantly reduce the surcharge requirement. The development of ground heave could lead to a slightly higher probability of spillage of contaminated sediments, but it is not considered significant enough to affect it ranking of environmental performance give the advantage of reducing the surcharge requirement.

8.6.6 *Deep Cement Mixing*

8.6.6.1 Sediment Plume: The details of this option are described in *Section 6.* The deep cement mixing (DCM) option is similar to SCP in terms of construction, but cement slurry is mixed with the sediment by a rotating paddle inside the metal casing instead of sand fill injection. During the final stages of the construction of a cement stabilised column When the paddle approaches the seabed and is no longer contained the injected cement can be expelled on to the seabed and potentially can escape into the environment. Hence, the potential environmental disturbance associated with the installation and formation of the very last section of the column in DCM is similar to SCPs. Ground heave is not a key concern to DCM as the required injection force is not high. However, leakage of cement slurry from the last section of the column formation is common (typically 5% of the application) and could be an environmental concern as fine cement particles (typically 5-30 microns), once released into the water columns, are relatively difficult to deal with. Accidental leakage of cement stored on site is also a potential issue.

8.6.6.2 The fine cement particles plumes will have similar physical effects on the environment as the sediment plumes, albeit with a higher dispersive power. However, the high dispersive power would also mean the plume can be rapidly diluted by the current rendering it less detrimental to the environment. Cement does not appear to pose significant environmental toxicity and the LC50 aquatic toxicity rating is often not determined in the material safety datasheet (e.g., CEMEX, 2006). Cement is also a strong alkaline (i.e., high pH in the typical range of 11-14) and could also affect the pH of the water column leading to an increase of ammonia toxicity to sensitive species such as juvenile fish. On the other hand, an alkaline medium accelerates the precipitation of metals from the water columns and could be considered as beneficial in some respects.

8.6.6.3 The fraction of the toxic free ammonia is very sensitive to change to the pH, for example with the typical salinity and temperature of NWWCZ, slight change of pH from 8 to 8.5 increases the percentage of free ammonia from 5% to about 15%. As waters of NWWCZ is often nitrogen enriched and the free ammonia concentration recorded (max. 0.0103 mg/l, Table 8.3) could be fairly close to the WQO of 0.021 mg/l, the WQO can be breached easily by the leaked cement even though any impact would be localised to the leakage area. Thus, it can be anticipated that the negative effect on the toxic ammonia concentrations will dominate the potential benefit of heavy metal sequestration. However, since only a relatively small amount of material at most would be likely to leak out (5%, assuming a typical column of 0.6m diameter and 20m long, the amount of loss will be about $0.3m^3$ only per cement column), the impacts on the water quality are likely to be small and localised.

8.6.6.4 Sediment Removal and Treatment: There is no requirement for sediment removal for the installation of DCM. Hence, there is also no need for sediment disposal or treatment, nor impacts associated with its removal.

8.6.6.5 Pore Water: Unlike the options considered before, DCM does not accelerate sediment consolidation and, thus, the accelerate release of pore water will be at a lower rate. Cement columns have low permeability and, hence, dissipation of pore water via the cement columns is not predicted. Indeed, the alkaline nature of the DCMs will reduce the heavy metal content of the interfacing pore waters. Direct injection of cement into sediments is actually an established *in-situ* remediation measure, commonly known as solidification/stabilization (S/S) to contaminated sites (USEPA, 2007). The main difference between DCM and S/S is that the cement is injected via a confined casing in DCM to form a structural column whereas in the S/S process, cement is directly injected without confinement.

8.6.6.6 Mitigation Measures: The primary consideration for mitigation measures will be to control the spillage of cement slurry during application and on-site storage. A comprehensive waste management plan and good site practices would help to reduce accidental cement spillage during on-site storage. Sediment plumes generated during the main reclamation filling remains a concern to DCM and would require mitigation. The applicable mitigation measures and their effectiveness to control spilled cement slurry and sediment plumes are similar to PVDs. As this technique does not need surcharge material to be applied, the DCMs have some advantage over those methods that do and can be considered as a mitigation measure to the fill / waste management.

8.6.6.7 Key Environmental Concerns: In general, other than good site management, no specific environmental mitigation measures are recommended as there is minimal removal of contaminated sediments. Leakage of fine cement slurry is possible and cement plumes can physically affect sensitive receivers and indirectly affect the pH balance of the water column. This technique, however, has the advantage of not requiring additional surcharge after the reclamation. This minimises the need to import and transport additional fill material, and its subsequent removal.

8.6.6.8 Comparative Environmental Ranking: The environmental performance of DCM is rated as "Fair to Good" as there is also no need for dredging. Leakage of fine cement particles is a potential concern, but its environmental effects should not be be significant, as the quantities involved would be very much smaller. Compared to ground improvement technique like PVD, this technique has the advantage of not requiring additional surcharge after the reclamation. Thus, it is rated higher than Fair, but is not rated as Good because of the potential of the leaked cement to affect the pH balance and increase ammonia toxicity affecting the sensitive ecosystem of the site area.

8.6.7 *Piled Structures*

8.6.7.1 Sediment Plume: The details of this option are described in *Section 6*. The piled structures would involve the percussive piling of a steel tube similar to the metal casing in the options discussed above (e.g., SCP) and, thus, has a small potential for some plume generation during casing installation. However, casing extraction is not required for these steel piles as the casing itself forms part of the main structural element of the pile. Unlike the other options, however, sediments inside the casing (i.e., steel pile) then have to be excavated for the formation of a pile foundation. Thus, sediment plumes may be generated if spillage occurs while transferring the sediment from the extraction head to the holding facilities (typically a barge). This loss, however, should be relatively easy to control with good management procedures.

8.6.7.2 Sediment Removal and Treatment: Based on the site configuration, it is likely about 0.5 Mm^3 of sediment inside the piles will have to be excavated as described above. This is a small amount given the size of the reclamation. The disposal / treatment of this quantity should also be relatively easy although some of this can be highly contaminated requiring special treatment.

8.6.7.3 Pore Water: Similar to DCM, the installation of piles structure does not assist in accelerating sediment consolidation and, thus, the release of pore water is not an issue.

8.6.7.4 Mitigation Measures: The primary consideration for mitigation measures for the piled structure option will be the noise impacts to the endangered Chinese White dolphins (CWD) inhabiting the area. Based on the assumed site configuration, it is estimated that about 6,540 steel piles will be required. A percussive hammer head of around 100 tones would be required to drive the steel piles and while effective noise mitigation measures to protect the CWD exists and has been acceptable before, its applicability for such a large scale application over almost 5 years has not been demonstrated. Treatment/disposal of contaminated sediment removed as a result of the pile installation would be another issue that would need to be addressed. If the Government continues to provide disposal space for infrastructure projects in Hong Kong, compliance with ETWB TCW 34/2002 should be sufficient. If not, project specific treatment facility and site has to be identified and this has been discussed in *Section 8.5.6*.

8.6.7.5 Key Environmental Concerns: The large scale application of percussive steel piling over about 5 years in the heart of important CWD habitats is likely to induce significant impacts to the CWD community. Although percussive piling has been accepted for works around important CWD habitats with mitigation, none was on this scale and even then, there were highly restricted working conditions and their use was often for a short period of time only. Demonstration of the effectiveness of the accepted mitigation over a prolonged period will be required. Given that the core CWD habitats at the northwestern waters are under continual threat of development and habitat loss mainly through reclamation, the resilience of the CWD population to such additional prolonged sub-acute impacts would be questionable even if the noise can be effectively mitigated to non-lethal levels. The Government has enforced strict measures to protect this endangered species and frequently demand the use of alternative piling methods such as bored piling. However, the 5 year programme for percussive piling has not been allowed for non-working periods due to the CWD calving seasons and bored piling would be even more onerous. Section 11 of this report has considered the potential impact on the reference programme which might be anticipated in the event that piling is restricted not to occur during the calving season for the CWD or at night.

8.6.7.6 Apart form the impacts to CWD, however, the piled structure has some environmental benefits. It would remove the need of reclamation and, hence, the potential associated long-term changes in the hydrodynamic regime as well as short-term reclamation related sediment plumes. Furthermore, most of the marine habitats would be preserved as the footprint of the 6,540 1.8m diameter piles is only about 1.7 ha. Hence, the impacts to the marine ecology in term of habitat loss could be significantly reduced. While the noise impacts would also affect fisheries and other species that are sensitive to noise and pressure, the pile could ultimately produce a "reef effect" and enhance habitat diversity of the area over the operation life the pile platform. Thus, it can be considered as having some benefits in the long-term.

8.6.7.7 A minor issue with respect to the piled structure is the management of about $0.5Mm^3$ contaminated spoils excavated from the CMPs. Unlike regular dredging operations, it may not be efficient to segregate the contaminated stratum from uncontaminated stratum, rendering all the excavated sediment to be treated as contaminated material. This will, thus, increase the amount of "contaminated" material to be treated/disposal of. The balance between project progress and the waste management would need further assessment. However, should it be determined that minimising waste shall be the priority, then the project waste management plan would need to specify the need to segregate the "more contaminated stratum" from the "less contaminated one".

8.6.7.8 <u>Comparative Environmental Ranking</u>: The major environmental benefit of this option is the elimination of reclamation. As the piled structure will allow the use of stilted platform to form the land, the overall reclaimed area can be significantly reduced to the footprint of the piles itself and the associated impacts such as habitat loss and changes in the hydrodynamic regime could also be significantly reduced. The available marine area between piles would also be expected to provide opportunities for ecological enhancement such as artificial reefs, if deemed necessary. However, on the negative side, other than the small quantities of dredged spoils, the piling works has the potential to induce significant noise impacts on the highly endangered CWD present in the area. The noise impacts associated with the large scale pilling would be difficult to mitigate effectively given its duration. Based upon the importance of this species, with only about 1200 individuals in the area, their protection would be considered a high priority. Thus, overall, the environmental performance of the piled structure option is comparatively rated as "Poor" mainly for the interest of this precious species.

8.6.8 ***Vacuum Consolidation – At Reclamation Level***

8.6.8.1 Sediment Plume: The details of this option are described in *Section 6*. This is actually an enhancement to the PVD/SCP by inclusion of an impermeable membrane over the top the drainage blanket (e.g., the sand blanket) either underwater or above sea level. The sediment plume issues are, similar, to the PVD /SCP and not predicted to be significant.

8.6.8.2 Sediment Removal and Treatment: Similar to PVD / SCP, there is no requirement for sediment removal and treatment.

8.6.8.3 Pore Water: As an active process, the vacuum consolidation process extracts the contaminated pore water via a set of vacuum pipes connected to the drainage layer. While the high natural dilution will likely render the environmental impacts of the subsequent release of this collected contaminated pore-water negligible, direct discharge will unlikely be allowed by the Government. Hence, the contaminated pore water will need to be treated on-site or off-site. The treatment options have been explored in *Section 8.5* above.

8.6.8.4 Mitigation Measures: The need to treat the extracted pore-water will be the first consideration as it is likely to be contaminated. Apart from the contaminated pore-water issue, this technique does not need surcharge material to be applied which gives this some advantage over those methods that do and can be considered as a mitigation measure to the fill / waste management.

8.6.8.5 Key Environmental Concerns: This ground improvement method is not likely to cause significant environmental impacts. The management of the extracted contaminated pore-water would be a key concern although treatment methods are readily available. Another main concern is likely to be the sediment plumes generated during the main reclamation filling. Compared to the method like PVD, this technique has the advantage of not requiring additional surcharge after the reclamation which minimises the need to import and transport of additional fill material, and its subsequent removal.

8.6.8.6 Comparative Environmental Ranking: The environmental performance of Vacuum Consolidation is rated as “Good” in recognition of the fact that it can significantly reduce the additional fill requirement. While the need to treat the contaminated pore-water would further increase the development cost of the project, this is not considered as a limiting factor as it ultimately reduce the amount of the pollutants entering the environment. In the worse case where there is a spillage of the extracted pore water, the effect is likely to be similar to the other construction options involving the “dispersion” of the pore-water.

8.6.9 *Semi- Buoyant Construction*

8.6.9.1 Sediment Plume: The details of this option are described in *Section 6*. Under the Semi Buoyant Construction (SBC) option, the normal reclamation filling material is replaced with light weight material such as polystyrene. The platform is essentially build on top of PVD/SCP and, thus, the potential sediment plume issues are similar to those options and not predicted to be significant.

8.6.9.2 Sediment Removal and Treatment: Similar to PVD / SCP, there is no requirement for sediment removal and treatment.

8.6.9.3 Pore Water: This is similar to PVD/ SCPs in which contaminated pore water will be released. However, with the reduced weight, the loading on the CMPs will be lower and this will reduce both the rate and ultimate amount of contaminants released as pore water. As such, based on this and the level of dilution expected, significant impacts are not expected.

8.6.9.4 Mitigation Measures: In general, other than good site management, no specific environmental mitigation measures are recommended as there is minimal removal of contaminated sediments. The applicable mitigation measures for the control of sediment plumes and their effectiveness are the same as PVDs. Also, this option has the advantage of reducing the surcharge. Notwithstanding the above, it must be noted that as significant amount of polystyrene material will be required for the site configuration (~21 Mm^3), a dedicated production plant within Hong Kong would likely be required. A chemical plant with a storage capacity of more than 500 tonnes and in which substances are processed or produced is a designated project and will, thus, need a separate EIA study.

8.6.9.5 Key Environmental Concerns: This construction method is not likely to cause significant environmental impacts. The main issue is likely to be the sediment plumes generated during the main reclamation filling. Compared to ground improvement technique like PVD, this technique has the advantage of both reducing the filling material for the main reclamation and reducing surcharge requirement after the reclamation. This would minimise the need to import and transport of fill material and subsequent disposal.

8.6.9.6 Comparative Environmental Ranking: The environmental performance of Semi Buoyant Construction is rated as "Good". The SBC has the same environmental issues as PVD/SCP, but as both fill material for the main reclamation and surcharge can be reduced in a significant way, it is rated higher as "Good". Although there could be risk associated with the handing of the raw material for the polystyrene and also operational risk associated with fire, it does not that appear that these risks are imminent as discussed in the *Section 6*. Other than the potential need of a separate EIA study for the dedicated production plant, it is considered not necessary to down-weight the raking on these grounds.

8.6.10 *Floating Structure*

8.6.10.1 Sediment Plume: The details of this option are described in *Section 6*. The very large floating structure (VLFS) is an engineering innovation, whereby, instead of traditional dredging and reclamation, large floating platforms are constructed off-site (e.g., at ship-yard) and welded together on-site to form a platform. Typically an anchoring piled platform is required to hold the structure in position. The required anchorage piled platform, however, would only be a fraction (in the range of 10-20%) of the size of the VLFS. As discussed, in the Piled Structure section above, the anchorage piled platform would not be expected to cause significant sediment plumes. The issue of sediment plumes, if any, is further diminished by the much reduced reclamation size. In addition, with careful positioning of the piled platform, it is may be possible to totally avoid interfacing with the CMPs completely.

8.6.10.2 Sediment Removal and Treatment: Similar to Piled Structure, but with a substantially reduced quantity.

8.6.10.3 Pore Water: The installation of piles for the anchorage does not result in accelerating sediment consolidation and, thus, the dispersion of pore water is not an issue.

8.6.10.4 Mitigation Measures: The primary consideration for mitigation measures is to minimise the number of piles required for the anchorage piled platform and avoid percussive piling in order to reduce noise impacts to CWD. Another consideration is to position the anchorage piled platform in such locations as to totally avoid the CMPs (and associated removal of contaminated material from the piles) while also reducing possible impacts on the local hydrodynamics.

8.6.10.5 Key Environmental Concerns: By totally avoiding interfacing with the CMPs, concerns associated with disturbance to CMPs are avoided. The common environmental concerns associated with marine projects in the area would still remain. It shall, however, be noted that the Japanese experience on the Mega-Float airport project has demonstrated that the impacts on the flow, water quality, benthos and fish are relatively small and insignificant (SCR, 2008). However, as about 84 steel piles would still be needed, the associated noise impacts to the CWD would be an issue. Section 11 of this report has considered the potential impact on the reference programme which might be anticipated in the event that piling is restricted not to occur during the calving season for the CWD or at night.

8.6.10.6 Comparative Environmental Ranking: The environmental performance of VLFS could have been rated as "Very Good" given the small footprint of the anchorage pile required and the possibility to totally avoiding interfacing with CMPs. The elimination of reclamation would also mean almost no impacts to the marine ecosystem, water quality and hydrology as exemplified by the Mega-Float project. However, it is considered that construction of 84 steel piles in around CWD habitats could induce significant impacts to this important species. Thus, it is down-rated as "Good" only.

8.6.11 *Electro-osmosis*

8.6.11.1 Sediment Plume: The details of this option are described in *Section 6* and this is the only option that was not considered feasible on engineering and cost grounds. However, its environmental performance is considered here for completeness. The environmental performance and environmental concerns of electro-osmosis in terms of sediment plumes are the same as PVD described above.

8.6.11.2 Sediment Removal and Treatment: There is no requirement for sediment removal for the installation of the electrodes. Hence, there is also no need for sediment disposal or treatment, nor impacts associated with its removal.

8.6.11.3 Pore Water: As an active process, the electro-osmosis process extracts the contaminated pore water via a set of pipes. While the high natural dilution will likely render the environmental impacts of the subsequent release of the contaminated pore-water negligible, direct discharge will unlikely be allowed by the Government. Hence, the contaminated pore water will need to be treated on-site or off-site. The treatment options have been explored in *Section 8.5* above.

8.6.11.4 Mitigation Measures: The need to treat the extracted pore-water will be the first consideration as it is likely to be contaminated. Furthermore, the electric process has a potential to generate and release reactive chemicals and also affect the pH balance of the pore water and ambient. Further studies will be required to explore the possible mitigation measures to manage the generation and release of reactive chemicals.

8.6.11.5 Key Environmental Concerns: The electro-osmosis requires the use of a direct electric current (DC) to be applied to drive the pore water out. Reactive chemicals can be generated though the electric process. The compounds can be highly reactive, if it they are not destroyed during the electrolysis process and will likely be toxic to the ecosystem. Since, as this has not been explored further in the engineering section, not quantification of impacts are possible at this stage. However, since the electric process involves drastic process like electrolysis, it can be expected the highly toxic intermediate may be generated since various toxic chemicals may be present in the CMPs.

8.6.11.6 Comparative Environmental Ranking: The environmental performance of Electro-osmosis is rated as "Poor" for the potentially high risk of generation of reactively environmental contaminants. Unlike the vacuum consolidation extraction process, however, this technique still requires surcharge material to be applied and it, thus, does not have a major environmental benefit for this process.

8.6.12 *Deep Well Dewatering*

8.6.12.1 Sediment Plume: The details of this option are described in *Section 6*. The environmental performance and environmental concerns of deep well dewatering (DWD) in terms of sediment plumes, fill and surcharge requirements are the same as PVD described above.

8.6.12.2 Sediment Removal and Treatment: Similar to PVD / SCP, there is no requirement for sediment removal and treatment.

8.6.12.3 Pore Water: As an active process, the DWD process extracts the contaminated pore water via a set of pumps. While the high natural dilution will likely render the environmental impacts of the contaminated pore-water negligible, direct discharge will unlikely be allowed by the Government. Hence, the contaminated pore water will need to be treated on-site or off-site. The treatment options have been explored in *Section 8.5* above.

8.6.12.4 Mitigation Measures: The need to treat the extracted pore-water will be the first consideration as it is likely to be contaminated. The applicable mitigation measures for the control of sediment plumes and their effectiveness are the same as PVDs.

8.6.12.5 Key Environmental Concerns: This ground improvement method is not likely to cause significant environmental impacts. Similar to the Vacuum Consolidation option, the management of the extracted contaminated pore water would be key although treatment methods are readily available. Another main issue is likely to be the sediment plumes generated during the main reclamation filling although this is controllable.

8.6.12.6 Comparative Environmental Ranking: The environmental performance of DWD is rated as "Fair" similar to PVD. While the need to treat the contaminated pore-water would further increase the development cost of the project, this is not considered as a limiting factor as discussed in the Vacuum Consolidation section.

8.6.13 *Rock Armoured Seawalls*

8.6.13.1 Sediment Plume: The details of this option are described in *Section 6*. This option would be used in combination with one of the options discussed above as the Rock Armoured Seawalls would form part of the reclamation element rather than as an option it its own right. A peripheral seawall is required for all the reclamation options, except the Piled Structure and Floating Structure. However, unlike the traditional construction approach in which the foundation for a seawall is formed by preparing a dredged trench, the seawall over the CMPs is proposed to be sited on the sand blanket. As no dredging is required, there will be no direct disturbance to the contaminated mud. Sediment plumes will be minor during the formation of the sand blanket as discussed in *Section 8.6.2*. Similarly, sediment plumes associated with the seawall placement will be minor as the interfacing material will be the coarse material of the sand blanket. As discussed in the Mitigation Measures Section (*Section 8.5*), by forming the peripheral seawall prior to the main reclamation, it is an effective mitigation measure to control the release and spread of sediment plumes in the subsequent backfilling works.

8.6.13.2 Sediment Removal and Treatment: Similar to PVD / SCP, there is no requirement for sediment removal and treatment.

8.6.13.3 Pore Water: This is similar to PVD/ SCPs in which pore water will be released. Normally, a Rock Armoured Seawall has to be permeable in order to avoid the hydraulic loading effects of water level variation inside and outside the seawall. Thus, it would not act as an effective containment measure for pore water although this should not be an concern based on the high dilution of any release. Indeed, if the peripheral Rock Armoured Seawall is constructed prior to the main reclamation, it could reduce the natural flushing of area behind the seawall to only the tidal change and, hence, reduce the dilution power as discussed in Section 8.4 above. If the pore water is highly contaminated, the reduced dilution would promote the development of local hot-spots, albeit within the reclamation area only.

8.6.13.4 Mitigation Measures: Constructing the peripheral seawall prior to the main reclamation will reduce possible sediment plumes in the subsequent backfilling works and is an effective mitigation measure to control sediment plumes during the main reclamation works.

8.6.13.5 Key Environmental Concerns: This method is not likely to cause significant environmental impacts. A potential concern is the built-up of local hot-spots within the reclamation area if the pore water is highly contaminated.

8.6.13.6 Comparative Environmental Ranking: The environmental performance of Rock Armoured Seawalls is rated as "Good" based on the fact that it is an effective mitigation measures against sediment plumes. However, given the various construction options proposed that substantially negate the need to relocate contaminated sediments in the CMPs, plumes of contaminated sediment are not necessary the key concern and hence it is not rated higher.

8.6.14 *Summary of Environmental Assessment*

8.6.14.1 The main CMPs interfacing environmental issues associated with the construction options proposed and their relative environmental performance are summarised in *Table 8.16* below.

Table 8.16: Summary of CMPs Interfacing Environmental Issues and Environmental Ranking of Construction Technique

Process	Sand Blanket	Surcharges	Pore Water Extraction	Contaminated Sediment Plumes	Sediment Removal & Treatment	Reclamation	Environmental Ranking	Main Consideration
Sand Capping Layer (Sand Blanket) **	Yes	NA	NA	Minor	No	NA	NA	NA
Prefabricated Vertical Drains (PVD)	Yes	Yes	No	Minor	No	Yes	Fair	▪ Sediment plumes related to the main reclamation
Sand Drains	Yes	Yes	No	Minor	No	Yes	Fair	▪ Sediment plumes related to the main reclamation
Sand Compaction Piles (SCP)	Yes	No	No	Minor	No	Yes	Good	▪ Eliminate surcharge (extra fills) requirement; ▪ Sediment plumes related to the main reclamation
Deep Cement Mixing (DCM)	Yes	No	No	Minor	No	Yes	Fair to Good	▪ Risk of cement leakage affecting the water quality; ▪ Eliminate surcharge (extra fills) requirement; ▪ Sediment plumes related to the main reclamation
Piled Structures	No	No	No	Minor	Yes, about 0.5 Mm^3 of spoils inside the pile columns which could be contaminated. .	No	Poor	▪ High noise impacts to CWD during piling (6,540 piles); ▪ Management of dredged spoil; ▪ No reclamation and minimal impacts to water quality and ecosystem
Vacuum Consolidation	Yes	No	Yes	Minor	No	Yes	Good	▪ Sediment plumes related to the main reclamation; ▪ Eliminate surcharge (fill) requirement
Semi Buoyant Construction	Yes	Yes (reduced)	No	Minor	No	Yes	Good	▪ Reduction of fill material requirement (~35%) and thus sediment plumes related to the main reclamation ▪ Reduce surcharge (fill) requirement
Floating Structure (VLFS)	No	No	No	Minor	Yes, a small amount of spoils inside the anchorage pile columns.	No.	Good	▪ Some noise impacts to CWD during piling; ▪ No reclamation and minimal impacts to water quality and ecosystem
Electro-osmosis	Yes	Yes	Yes	Minor	No	Yes	Poor	▪ Generation of reactive chemicals that can potentially highly toxic; ▪ Sediment plumes related to the main reclamation; ▪ Potentially added concern of DC field effects on the ecosystem
Deep Well Dewatering (DWD)	Yes	Yes	Yes	Minor	No	Yes	Fair	▪ Sediment plumes related to the main reclamation
Rock Armoured Seawalls **	Yes	NA	No	Minor	No	Yes	Good	▪ Minimise the impacts of sediment plumes during main reclamation;

**to be applied in combination with another option. All the environmental ranking are relative to the PVD.

8.7 Statutory Issues and Time

8.7.1 *Environmental Impact Assessment Ordinance*

8.7.1.1 The principal environmental legislation that governs this Project is Chapter 499, the Environmental Impact Assessment Ordinance (EIAO) which specifies a list of projects as designated projects (DP) which require an environmental permit (EP) before they can proceed. Items B1 “an airport (including its runway and the development and activities related to aircraft maintenance, repair, fueling and fuel storage, engine testing or air cargo handling)”, C1 and C2 of Schedule 2 of EIAO are particularly relevant. If a project specific contaminated sediment and/or pore water treatment facilities were considered necessary, then Items C10 (a marine dumping ground) and G4 (waste disposal facility or waste disposal activity) may also be relevant. It is understood that the Airport is an exempted DP under the EIAO and the relevance and applicability of the exempted status being extend to cover the anticipated expansion will be assessed under the Engineering Feasibility and Environmental Assessment Study (Contract P 132).

8.7.1.2 If the exempted status can be extend to the 3rd runway, than general environmental legislation will also be relevant and applicable, including the following:

- Air Pollution Control Ordinance;
- Noise Control Ordinance;
- Waste Disposal Ordinance;
- Water Pollution Control Ordinance; and
- Dumping at Sea Ordinance.

8.7.1.3 An environmental impact assessment report (EIA report) will only be required if the EIAO is applicable. If the EIAO is applicable, then given the scale of the project, it will be unlikely that and direct EP application will be approved. The first step of the normal process under the EIAO is then the submission of a Project Profile (PP) for application of an EIA Study Brief by the project proponent. Based on the PP, and any comments received during a 14 days public consultation period, EPD issue a Study Brief within 45 days of receiving the application.

8.7.1.4 Upon submission of the EIA report, EPD will review and comments on the report within 60 days. Upon acceptance of the EIA report by EPD, it then has to be exhibited for public inspection / consultation for a period of 60 days. EPD will then, in 30 days of the expiry of the public inspection period, or the receipt of comments from the Advisory Council on the Environment, or the receipt of further information, approve or reject the EIA report.

8.7.1.5 Based on an approved EIA report, the client can then apply for an EP which shall be processed by EPD within 30 days. However, the application for approval of EIA report and EP can be submitted at the same time and thus saving 30 days for the EP.

8.7.1.6 The overall statutory time is thus 45 days for the EIA Study Brief and 150 days for the EIA report, or about 6.5 months. A period of time, however, has to be allowed for the conducting the EIA study. Given the project nature, the EIA Study Brief might required 9 - 12 months full ecological survey, 3-6 months may also be required to collect the necessary site-specific water quality and sediment quality data although these can be conduct in parallel to the ecology survey. Assuming 3-6 months study period is allowed, it will take about 1 to 1.5 year to complete the EIA report.

8.7.1.7 Based upon this and the possibility of extended consultation time to obtain approval of the report, more time from the current 9 moths as allocated under Contract P132 may be required.

8.7.2 *Dumping at Sea Ordinance*

8.7.2.1 As discussed previously, the Dumping at Sea Ordinance (DASO) regulate the marine dumping activities in Hong Kong via a permit control system. Anyone who intends to dump dredged marine mud must first obtain a permit from EPD. Spoil grounds have been designed for the disposal of dredged mud. All marine dumping activities must be carried out at these specified areas in accordance with the marine dumping permit.

8.7.2.2 The technical circular ETWB TCW 34/2002, provides guideline on the information need to be provide the project proponents when applying for a marine dumping permit. The circular also specifies the sediment sampling and testing requirements as well as the sediment classification frameworks and disposal/treatment requirements (see *Section 8.2*). Given that fact that the sediment contained inside the CMPs are contaminated, a sampling scheme of either 100m x 100m or 50m x

50m grids with vertical profiles would likely be required. This wound amount to about 340 (100m grid) to 1350 (50m grid) vibrocore samples for the 11 sub-pits. It is reasonable to assume that a sampling barge can collect 4 viborcore samples per day and it will take 2 barges about 42 working days (~ 1.7 months) to complete the sampling 340 vibrocores or 170 working days (~ 7 months) to complete 1350 vibrocores. If further assuming the available testing capacity in Hong Kong can be secured, the sediment quality testing results can be provided in about 2 months after the sample collection.

8.7.2.3 Provided that advanced disposal allocation space have been granted by the Marine Fill Committee (MFC) and full sediment quality testing results are provided to EPD, the dumping permit application process should be relatively short (say a month). Thus, the critical step is the sediment sampling and testing process rather than the statutory procedures. Given the estimates above, it is likely to take between 3 months to 10 months for the DASO permit process. As the sediment quality testing results are considered valid for a period of 3 years, the DASO permit related process is normally undertaken after the EIA report is approved.

8.7.3 *Management of CMPs and Disposal Space*

8.7.3.1 It shall be noted that, so far, all the active or completed CMPs in Hong Kong are pre-EIA and thus not under the EIAO. Hence, disturbance to the capped and sealed CMPs by the 3rd runway is not restricted by the EIAO. However, the area has been gazetted for use as marine disposal grounds under DASO. Changing the "land use" would need to seek approval by the Government Departments which would include the Civil Engineering and Development Department who is managing all the disposal grounds in Hong Kong. Being the designated authority under DASO, EPD would also be involved for any change to the capped and sealed CMPs. The requirement time for seeking an agreement to covert the disused dumping grounds to another use will be largely a policy and administrative matter and there does not appear to have a statutory timeframe for this although it can be anticipated that this will somehow depends on the findings of the EIA study. It is noted that triggering the EIAO is possible if there is to be significant dredging, then item C.12 of the EIAO schedule may become relevant although disturbing a previous mud pit may not be considered as a designated project per se.

8.7.3.2 Under the current policy, the Government (mainly EPD and CEDD) is responsible for the designation, management and provision of appropriate disposal space in

Hong Kong. While there has been case where a third party have been required to be responsible for the operation a dumping ground, for example the CPRC pit interfacing with this Study and also the intermittent use of the MBA at North of the Brothers, the space is provided by the Government. It is, thus, reasonable to expect the Government will continue to provide the appropriate disposal space to meet the need of the 3rd Runway Project.

8.7.4 *Mitigation Measures*

8.7.4.1 Any innovative or untested mitigation measures may be required to undergo trials to demonstrate their applicability and effectiveness. Such trials could be post EIA (i.e., as a permit condition) or possibly EPD may insist on some proof prior to accepting the EIA report. Clearly, avoiding untested methods where possible could be the easiest course of action but if such mitigation was need, a period of 3 – 6 months can be required depending on the scale of such trials.

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9 RISK ASSESSMENT

9.1 Risk Assessment Procedure

9.1.1 The risk assessment methodology adopted in this assessment has been established to be consistent with the policy requirements and guidelines set out under the following Environment, Transport and Works Bureau (ETWB) documents:

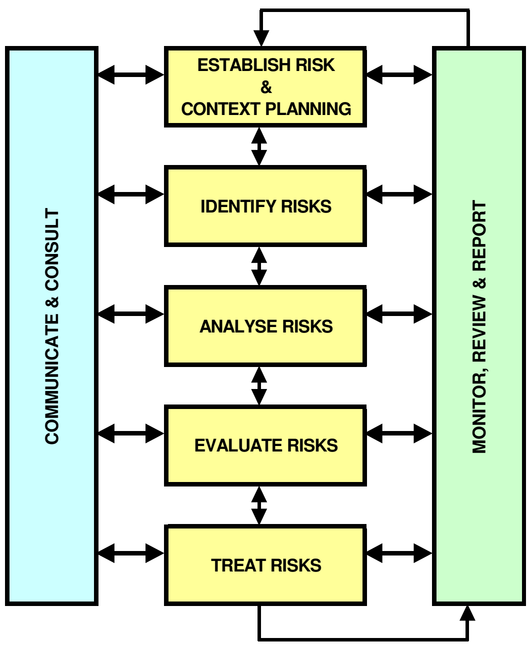
- TCW 6/2005: Implementation of Systematic Risk Management in Public Works Projects; and
- ETWB Risk Management User Manual.

9.2 Risk Identification & Register and Methodology

9.2.1 *Methodology*

9.2.1.1 The methodology used for the management of risk on this project will be consistent with the risk management process described in the ETWB Risk Management User Manual. This is summarised in the flowchart below:

THE RISK MANAGEMENT PROCESS



9.2.2 ***Risk Identification***

9.2.2.1 The basic intent in this stage is to generate a comprehensive list of possible events and consider scenarios and causes.

9.2.2.2 The key identifiable risks have been recorded in the risk register, details of which are included in *Appendix B*. The Register documents all currently identified risks. It is the intention that this document can be enhanced as the project evolves and as new stakeholders become involved. It is a necessary part of the process of agreeing a set of the most important risk areas to be addressed in the next stages of the process. Highlighting and documenting previous lessons learnt will go some way to managing their potential re-occurrence on this project.

9.2.3 ***Risk Categories (Heads of Risk)***

9.2.3.1 The following categories of risk are considered to be possible in the light of the scope of this assignment.

(a) Design Risks;
(b) Construction Risks;
(c) Performance Risks (in relation to the function of the reclamation);
(d) Environmental Risks;
(e) Programme Risks; and
(f) Financial Risks.

9.2.4 ***Risk Analysis***

9.2.4.1 Risks have been analysed against the identified risk analysis criteria using qualitative techniques. This form of risk analysis involves assessing the probability and consequences of each risk occurring to determine the relative level of risk.

9.2.4.2 Where present, existing controls will be identified. Their strengths and weaknesses will then be taken into account in the analysis.

9.2.4.3 The *Tables 9.1, 9.2 and 9.3* below present the proposed draft analysis criteria.

Table 9.1 Proposed Consequence Criteria Table

Consequence Level ID	Consequence Descriptor	Health & Safety Impact Off Site	Environmental Impact Off Site	Level of Delay	Level of Financial Loss
Cl	Insignificant	No injuries or health effects	Insignificant environmental impact	Insignificant Delay (< l month)	Insignificant financial loss (< HK$1,000, 000)
C2	Minor	First aid treatment	Immediately contained environmental impact	Slight project delay (1 month to 3 months)	Minor financial loss (HK$1 M to HK$10 M)
C3	Moderate	Medical treatment required	Contained environmental impact	Moderate project delay (3 months to 6 months)	Moderate financial loss (HK$10 M to HK$100 M)
C4	Major	Extensive injuries (on site or off site)	Localised uncontained environmental impact	Major project delay (6 - 12 months)	Major financial loss (HK$100 M to HK$1000 M)
C5	Catastrophic	Fatality and/or severe injuries	Regional uncontained environmental impact	Project halted (> 1 yr to project halted)	Huge financial loss (> HK$1000 M)

Table 9.2 Proposed Likelihood (Probability) Criteria Table

Probability Level ID	Probability Descriptor	Description	Probability / Frequency
PI	Rare	This event may occur in exceptional circumstances -perhaps once in 100 years - Examples of this have occurred historically, but it is not anticipated during the project	Once in every 100 projects (< 100 years)
P2	Unlikely	This event could occur at some time – perhaps once in 10 years - Unlikely to occur during the project	Once in every 10 projects (< 100 years)
P3	Possible	This event might occur at some time – perhaps once per year - Might occur during the project	Once during projects (> 1 years)
P4	Likely	This event will probably occur in most circumstances - perhaps several times per year - Expected to occur once or more during the	Several times during project (> 1/year)
P5	Almost Certain/ Frequent	This event is expected to occur in most circumstances – perhaps daily or continously - Expected to occur many times during the project	Many times during project (> 11 month)

Table 9.3 Proposed Risk Analysis Matrix

		Consequence				
		Insignificant (C1)	**Minor (C2)**	**Moderate (C3)**	**Major (C4)**	**Catastrophic (C5)**
Probability (Likelihood)	**Rare (P1)**	Low	Low	Low	Medium	High
	(Unlikely (P2)	Low	Low	Medium	High	High
	Possible (P3)	Low	Medium	Medium	High	Very High
	Likely (P4)	Medium	Medium	High	Very High	Extreme
	Frequent (P5)	Medium	High	Very High	Extreme	Extreme

9.2.5 *Risk Evaluation*

9.2.5.1 The purpose of the risk analysis phase is to gain an understanding of level of risk to make decisions about future actions and define priorities. The following risk evaluation criteria in *Table 9.4* are proposed to provide a means by which decisions can be made, and resources allocated.

Table 9.4 Risk Evaluation Criteria

Level of Risk	Recommended Level of management Attention
Extreme	IMMEDIATE senior management attention needed, action plans must be developed, with clear assignment of individual responsibilities and time frames
Very High	Senior management attention needed, action plans must be developed, with clear assignment of individual responsibilities and time frames.
High	
Medium	Risk requires specific ongoing monitoring and review, to ensure level of risk does not increase. Otherwise manage by routine procedures.
Low	Risk can be accepted or ignored. Manage by routine procedures, however unlikely to need specific application of resources.

9.2.6 *Risk Treatment*

9.2.6.1 For this project the following treatment hierarchy will be adopted for those risks that require treatment:

- Avoiding the risk;
- Reducing the likelihood of occurrence;
- Reducing the consequences;
- Transferring or sharing the risk; and
- Retaining the risk.

9.2.6.2 Selection of preferred risk treatments will typically be a cost-benefit decision, with preference given to treatments that provide the best all round benefit to the project. In the majority of cases, the identification of which treatment provides the greatest benefit will be straightforward and will not require an in-depth analysis. Additionally, options for risk treatment will be assessed on the basis of the extent of any additional benefits or opportunities created.

9.2.6.3 For any particular risk, a number of treatment options may be considered, and applied either individually or in combination. These have been identified in the register.

9.3 Risk Analysis

9.3.1 The risk assessment exercises which have been completed and as detailed in *Appendix B* identify a number of significant risks associated with each option. Whilst for every option high risk items have been identified in no cases have very high risks been deduced. Accordingly, mitigation proposals are proposed for each option.

9.3.2 The relevant risks have been identified by the consultants' team through discussion internally and with GCG during the report preparation process. In particular, the risks have been identified through the discussion sessions with the specialists. The risk register presented in *Appendix B* therefore requires review by the client other key stakeholder and their comments must be added back through the risk assessment process. In this manner the feedback on the first round of the risk identification and assessment process can be closed out. The register can then be updated in line with the client and stakeholder review and commenting.

9.3.3 The risk register details the assessment outcomes identifying those risks which particularly require mitigation. Mitigation proposals have been put forward. The conclusion to be drawn from the process is that there are no obviously insurmountable risks associated with the proposals which have been carried forward with the exception of that identified for the polystyrene foam semi buoyant construction and the reliability of the electro osmosis proposal.

9.3.4 For the semi buoyant filling techniques the very high risk is associated with the fact that there will be 21 million cubic metres of the material to be fabricated and handled on the site, the risks associated with handling the styrene monomer in such large quantities are exceptional and, as correctly noted, the potential fire risk is considerable, particularly as the material will be so widely distributed around the site.

9.3.5 The findings of the initial risk assessment are summarised in Table 9.5

Table 9.5 Summary of Risk Assessment Findings

Option	Probability Descriptor	Very High Risk	High Risk	Medium Risk	Low Risk
1	PVD Band Drains	0	9	9	0
2	Sand Drains	0	9	9	0
3	Sand Compaction Piles	0	10	8	0

Option	Probability Descriptor	Very High Risk	High Risk	Medium Risk	Low Risk
4	Deep Cement Mixing	0	10	8	0
5	Piled Platform Structure	0	10	10	0
6	Floating Structure	0	10	10	0
7	Vacuum Consolidation	0	8	10	0
8	Semi Buoyant Construction	1	9	9	0
9	Deep Well Dewatering	0	10	8	0
10	Electro Osmosis	1	0	0	0

10 COST ESTIMATES

10.1 Estimation methodology

10.1.1 For each option identified as being viable and as listed in *Table 10.1*, cost estimates have been developed.

Table 10 1 Development Scenarios Adopted for Cost Estimations

Development Option	Attributes
1	Reclamation within Optimised Seawall using PVD drains and surcharge
2	Reclamation within Optimised Seawall using Sand Drains and surcharge
3	Reclamation within Optimised Seawall using PVD drains and vacuum consolidation
4	Reclamation within Optimised Seawall using Sand drains and vacuum consolidation
5	Reclamation within Optimised Seawall using PVD drains and deep well dewatering
6	Reclamation within Optimised Seawall using Sand drains and deep well dewatering
7	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and restricted surcharge
8	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and restricted surcharge
9	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and vacuum consolidation
10	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and vacuum consolidation
11	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and deep well dewatering
12	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and deep well dewatering
13	Reclamation within Optimised Seawall using sand compaction piles throughout
14	Reclamation within Optimised Seawall using deep cement mixed piling throughout
15	Piled structure
16	Floating Structure

10.1.2 The estimates have been prepared on the basis that all common elements present in each scheme have been taken out of the costing exercise. Whilst this method does not yield a true cost estimate for the whole construction it does identify the costs associated with the differences between the schemes. As a result, the cost estimates evaluate the costs associated with forming the site platform and protective seawalls only. This approach is considered appropriate for the purposes of evaluating the relative cost differential between options only.

10.1.3 In undertaking the evaluation exercise it has been necessary to make a number of assumptions the details of which are set out below:-

- The piled and floating options necessarily include for the provision of a paved top surface equivalent to the runway, taxiways and apron area paving. Accordingly, all other options have also included for these paving elements.

- In each case it has been assumed that the reclamation work can be completed in accordance with the proposed programme and that normally anticipated production rates will be achieved for each element of the construction. ie no enhancements for accelerated production have been included. Similarly, the rates assume there to be no restriction to 24 hour working or works during the CWD calving season.

- It has been assumed that sand and sandy fill material will be sourced from a marine fill abstraction area located in relatively close proximity to the Chek Lap Kok site. Such areas have previously been identified by FMC. Winning the fill will require dredging and disposal of marine clays sufficient to expose the underlying sandy alluvial deposits applicable to the needs of the reclamation. The overall cost of acquiring sand in this manner amounts to an overall cost in excess of $HK113/m^3 on the basis that a dedicated borrow pit will need to be established and the sand extracted from beneath the marine mud. It is possible that sand could be sourced directly from China although the establishment of an agreement to provide in excess of 80 million m^3 will require high level negotiation. If it were possible to source sand in this manner and to avoid the costs associated with removing marine mud and exposing a marine borrow pit then the costs might be considerably lower and nearer to $HK50/m^3. The higher cost has been assumed in these estimates.

- It has been assumed that a rolling surcharge programme will be adopted and that the equivalent of surcharge to a 4.5 m height over 33% of the site area will need to be imported and disposed of after completion of its use. For the purposes of this estimation it has been assumed that the surcharge material will be recovered from and returned to the marine borrow area.

- No differentiation has been made between the options in respect of environmental monitoring.

- Provision has been made for the deployment of silt curtains during seawall construction and for piling works. No provision for a silt curtain has been made for the floating option.

- It has been assumed that no significant premium will need to be paid in respect of using the specialist floating plant and dredging equipment to meet the proposed programme. ie the plant will all be available to meet the proposed programme.

- It has been assumed that there will be adequate plant and specialist equipment available to meet the programme requirements and that specialist plant already exists in the market to suit the project requirements. ie sufficient plant already exists and can be made available. Whilst the options of sand compaction piles across the whole site and deep cement mixing across the whole site have been rejected on the basis that it would not be possible to mobilise sufficient plant to achieve these scenarios within the specified timescale, the cost estimates have assumed that sufficient plant could be made available.

- The estimates make no provision for preliminaries or contingencies on the basis that these are common to all schemes and that they should therefore be excluded from the evaluation of cost differences.

- Cost estimates for the floating and piled options assume that a runway/taxiway strip 360m wide and 4000m long (144 Ha) and an Apron Area (160Ha) will be constructed on structure. All other areas will remain open to the sea. Options involving reclamation assume that the full notional area (582 Ha) will be occupied by the reclamation.

- A risk analysis in relation to the estimates has been undertaken as a separate exercise to this assessment.

- Recurrent costs (maintenance and operational costs) have not been included except for the floating structure.

- Environmental mitigation works outside the construction mitigation costs for silt curtains have not been included.

- Provision has been made in the estimates for making good the anticipated settlement across the site

- Allowance has been made in each reclamation option for the installation of sand compaction piles along the runway alignment over a zone 60m wide x 4000m long.

- The estimates do not make provision for land premiums or for fees associated with mineral extraction rights ie there is no provision for the land premium associated with the development of the proposed marine sand borrow area.

10.2 Option Cost Estimates

10.2.1 The results to the cost estimation analysis are summarized below in *Table 10.2*. Estimates relate to the option evaluation scenario and assumptions detailed in *Section 10.1* above and assume a baseline case whereby work can proceed 24 hours a day and without any disruption for the Chinese White Dolphin calving season:-

Table 10.2 Summary of Cost Differentials

Option	Attributes	Estimated Construction Cost HK$ Billion	Whole life Average risk Cost HK$ Billion	Whole life Maximum Risk Cost HK$ Billion	Rank
1	Reclamation within Optimised Seawall using PVD drains and surcharge	28.64	64.52	68.72	4
2	Reclamation within Optimised Seawall using Sand Drains and surcharge	28.49	64.35	68.53	2
3	Reclamation within Optimised Seawall using PVD drains and vacuum consolidation	29.87	65.91	70.28	12
4	Reclamation within Optimised Seawall using Sand drains and vacuum consolidation	29.57	65.57	69.90	10
5	Reclamation within Optimised Seawall using PVD drains and deep well dewatering	29.04	64.97	69.22	8

Option	Attributes	Estimated Construction Cost HK$ Billion	Whole life Average risk Cost HK$ Billion	Whole life Maximum Risk Cost HK$ Billion	Rank
6	Reclamation within Optimised Seawall using Sand drains and deep well dewatering	28.74	64.64	68.85	6
7	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and restricted surcharge	28.56	64.43	68.62	3
8	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and restricted surcharge	28.41	64.27	68.43	1
9	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and vacuum consolidation	29.79	65.82	70.18	11
10	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and vacuum consolidation	29.50	65.48	69.81	9
11	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and deep well dewatering	28.96	64.88	69.12	7
12	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and deep well dewatering	28.67	64.55	68.75	5
13	Reclamation within Optimised Seawall using sand compaction piles throughout	29.90	65.94	70.32	13
14	Reclamation within Optimised Seawall using deep cement mixed piling throughout	50.33	88.96	96.33	15
15	Piled structure	42.20	69.54	75.72	14
16	Floating Structure	91.37	131.88	145.27	16

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10.2.2 Many of the cost estimates appear to have similar magnitudes. This is a consequence of the fact that the construction costs are dominated by the costs associated with forming the borrow pits, excavating, importing and placing the sand fill. These costs are necessarily similar for all of the reclamation schemes but are not incurred in the floating solution. Had the floating option been set aside from the reclamation options then the differential costs associated with the differing reclamation scenarios would have become much more apparent.

10.2.3 Summaries of the cost estimates for each option are included in *Appendix C*. The summaries include spreadsheet details of all the assumptions made in undertaking the estimates including quantities and rates of working together with cost rates for each component part of the estimates.

10.2.4 The estimates clearly indicate that the options involving reclamation will be considerably less expensive to implement than piled or floating options.

10.2.5 The cost differential between sand drains and prefabricated wick drains is small. Given the benefits of sand drains over wick drains under conditions whereby large settlements are anticipated then sand drains are to be favoured.

10.2.6 Vacuum consolidation represents a costly way to provide the necessary surcharging stresses and is probably only justified in cases where the availability of fill is restricted or the disposal of the fill after surcharging is prohibitive.

10.2.7 There are distinct advantages in the use of lightweight semi buoyant forms of construction as these reduce settlements and the need for large volumes of surcharge materials. There are however potential issues associated with environmental acceptability and robustness with the use of such materials. If sand can be sourced more cheaply from China than from a dedicated borrow pit in Hong Kong then the semi buoyant construction method looses its potential but risky advantage.

10.2.8 Supplemental estimates have been prepared assuming that the site formation work will be constrained by two factors. Prevention of working on a 24 hour basis and prevention of works during the 6 months of the year when the CWD calving season prevails. It can be seen that there are heavy penalties associated with such constraints not only in terms of the programme but also the costs associated with having major plant idling for the time where work is restricted.

10.2.9 A supplemental reference estimate has been prepared for selected options, assuming that the new platform were to be constructed over the mudpits but that the mudpits were filled with normally consolidated marine mud i.e the mudpits did not exist and that the seabed remained in its normal undisturbed state prior to construction of the proposed reclamation.

10.2.10 In assessing recurrent and maintenance costs the estimates assume the following:-

Replacement of runways – 50 year intervals

Replacement of runways - 50 year intervals
Replacement of apron slabs – 75 year intervals
Signage and line marking replacement - 10 year intervals
Lighting systems replacement – 20 year intervals
Major Pump replacement - 50 year intervals

General maintenance and cleaning of runways
General maintenance and cleaning of apron areas
Grass maintenance
Lamp replacement

Power costs

All other items have been deemed to be common to all scenarios under consideration. These items do not therefore feature in the analysis.

10.3 Option Cost Estimates – Restricted Working Times

10.3.1 The baseline cost estimation analysis has been repeated making the following assumptions in respect of environmental restrictions which might be applied:-

- Working hours will be restricted, with no marine earth works including dredging and filling operations or pile driving operations between March and August during the Chinese White Dolphin calving season.

- Working hours will be restricted, with no marine earth works including dredging and filling operations or pile driving operations between March and August during the Chinese White Dolphin calving season. Additionally working hours will be restricted to daylight hours only (assuming 12 hour days).

10.3.2 The results to these two supplemental cost analyses are summarized below in *Tables 10.3 and 10.4* for the 50% and 25% productivity scenarios respectively. Estimates relate to the same option evaluation scenario and assumptions as detailed for the baseline case and as set out in Section 10.1 above but make provision for the extended programmes and the need for additional resource mobilization to meet the construction objectives despite the 50% and 25% productivity restrictions imposed by environmental impact mitigation.

Table 10.3 Summary of Cost Differentials – No working during CWD calving period (50% productivity)

Option	Attributes	Estimated Construction Cost HK$ Billion	Whole life Average risk Cost HK$ Billion	Whole life Maximum Risk Cost HK$ Billion	Rank
1	Reclamation within Optimised Seawall using PVD drains and surcharge	34.16	68.98	73.98	1
2	Reclamation within Optimised Seawall using Sand Drains and surcharge	34.61	69.48	74.55	3
3	Reclamation within Optimised Seawall using PVD drains and vacuum consolidation	35.06	69.99	75.12	5
4	Reclamation within Optimised Seawall using Sand drains and vacuum consolidation	35.50	70.49	75.69	6
5	Reclamation within Optimised Seawall using PVD drains and deep well dewatering	34051	69.37	74.43	2
6	Reclamation within Optimised Seawall using Sand drains and deep well dewatering	34.96	69.87	74.99	4
7	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and restricted surcharge	37.79	73.06	78.60	7
8	Reclamation within	38.23	73.56	79.16	8

Option	Attributes	Estimated Construction Cost HK$ Billion	Whole life Average risk Cost HK$ Billion	Whole life Maximum Risk Cost HK$ Billion	Rank
	Optimised Seawall using Sand Drains, semi buoyant filling and restricted surcharge				
9	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and vacuum consolidation	38.97	74.40	80.11	11
10	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and vacuum consolidation	39.42	74.90	80.67	12
11	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and deep well dewatering	38.43	73.79	79.42	9
12	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and deep well dewatering	38.87	74.29	79.98	10
13	Reclamation within Optimised Seawall using sand compaction piles throughout	46.75	83.16	90.00	14
14	Reclamation within Optimised Seawall using deep cement mixed piling throughout	85.67	127.02	139.57	15
15	Piled structure	45.41	71.40	78.05	13
16	Floating Structure	90.92	129.61	142.93	16

Table 10.4 Summary of Cost Differentials – No work during CWD calving period or at night time (25% productivity)

Option	Attributes	Estimated Construction Cost HK$ Billion	Whole life Average risk Cost HK$ Billion	Whole life Maximum Risk Cost HK$ Billion	Rank
1	Reclamation within Optimised Seawall using PVD drains and surcharge	44.43	81.31	87.81	1
2	Reclamation within Optimised Seawall using Sand Drains and surcharge	46.16	83.25	90.01	3
3	Reclamation within Optimised Seawall using PVD drains and vacuum consolidation	45.33	82.31	88.95	5
4	Reclamation within Optimised Seawall using Sand drains and vacuum consolidation	47.05	84.26	91.15	6
5	Reclamation within Optimised Seawall using PVD drains and deep well dewatering	44.78	81.70	88.26	2
6	Reclamation within Optimised Seawall using Sand drains and deep well dewatering	46.51	83.64	90.57	4
7	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and restricted surcharge	47.06	84.26	91.16	7
8	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and restricted surcharge	48.78	86.21	93.37	8
9	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and vacuum consolidation	48.24	85.60	92.67	11
10	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and vacuum consolidation	49.97	87.55	94.87	12

Option	Attributes	Estimated Construction Cost HK$ Billion	Whole life Average risk Cost HK$ Billion	Whole life Maximum Risk Cost HK$ Billion	Rank
11	Reclamation within Optimised Seawall using PVD drains, semi buoyant filling and deep well dewatering	47.70	84.99	91.98	9
12	Reclamation within Optimised Seawall using Sand Drains, semi buoyant filling and deep well dewatering	49.43	86.93	94.17	10
13	Reclamation within Optimised Seawall using sand compaction piles throughout	70.32	110.48	120.78	14
14	Reclamation within Optimised Seawall using deep cement mixed piling throughout	146.43	196.24	217.69	15
15	Piled structure	49.01	76.20	83.38	13
16	Floating Structure	90.92	130.37	143.69	16

11 PROGRAMME

11.1 Work Load

11.1.1 For each option a baseline construction programme has been developed with the objective of completing the construction and achieving the primary settlement of the site within the required timeframe of mid 2011 to the end of 2018. The construction programmes for each baseline case are presented in *Figures 11.1, 11.2….11.6* incl. Supplemental programmes have been developed for each option by assuming that dredging will not be permitted and that earthworks operations including both dredging and filling as well as piling works will not be permitted during the CWD calving season between March and August each year. Accordingly, production rates have been assumed to reduce from 96.7% (assuming 12 days of T3 per year average) to 50 % to make provision for the six month CWD calving season each year. The programmes depicting restriction of work during the CWD calving period are presented in *Figures 11.1a, 11.2a….11.6a* incl. Further programmes have also been developed for each option assuming that dredging, marine earthworks and piling work will neither be permitted during the CWD calving period or at night time. Under this scenario productivity has been assumed to reduce to as little as 25%. The programmes depicting restriction of work during the CWD calving period and at night are presented in *Figures 11.1b, 11.2b….11.6b* incl. To maintain the construction periods within sensible timeframes it has been assumed that the amount of plant mobilised under the most onerous 25% productivity scenario will be increased to the maximum considered feasible. The maximum mobilisation deemed appropriate has been devised by considering the limitations imposed by the maximum number of fronts from which construction could progress or in cases where specialist plant is to be mobilised, including the cement mixing, sand compaction pile and sand drains equipment, then the limitation on the amount of plant which can be mobilised is governed by the likely amount of plant available to the project.

11.1.2 In general, the programme requirements for the baseline cases have been set and, as a consequence the necessary production rates have been evaluated for comparison with those achievable using reasonable amounts of currently available plant.

11.1.3 As a baseline the following fundamental key site formation parameters have been assumed. These relate to the development of the notional reclamation area:-

Total Plan area of reclamation	=	589 Ha

<u>Seawall</u>

Total Length of new seawall	=	11.9 km
Volume of seawall rock core	=	185 m^3 /m
Volume of Armour stone	=	90 m^3 /m
Ancillary armouring / scour apron	=	29 m^3 /m
Sand Blanket (seawall)	=	160 m^3 /m
Sand compaction piles	=	224 m^3 /m
Soil Mix column	=	304 m^3 /m

<u>Reclamation</u>

General filling	=	63 $x10^6$ m^3
Surcharge	=	5.8 $x10^6$ m^3 / m height
Compensatory filling to Make good settlement	=	17.7 $x10^6$ m^3

<u>Runway /Taxiway areas</u>	=	140 Ha
<u>Apron Areas</u>	=	160 Ha

11.2 Achievable Production Rates

11.2.1 The following production rates have been described in section 7 and have been adopted as being achievable:-

- Installation of PVD wick drains — 1200 units / day / 6 rig barge
- Installation of sand compaction piles — 150 units / day / 3 rig barge
- Installation of Sand Drains — 150 units / day / 3 rig barge
- Installation of Soil Mix column units — 90 units /day / 6 rig barge

• Placing of rockfill		2000 m^3 day / barge / front
• Fill placement	10 km haul	35,000 m^3 / 5000 Te dredger / day
• Floating Structures	Fabrication	120 Te / day / yard
	Assembly	20 days / unit
• Piled Construction	Bored piles	20 days / unit / rig
	Driven piles	10 days / unit / rig
• Semi buoyant construction		
	Block production	20,000 m^3 day

These rates are comparable with the following rates adopted in the assessment study for the Boundary crossing facilities for the Hong Kong-Zhuhai-Macao Bridge.

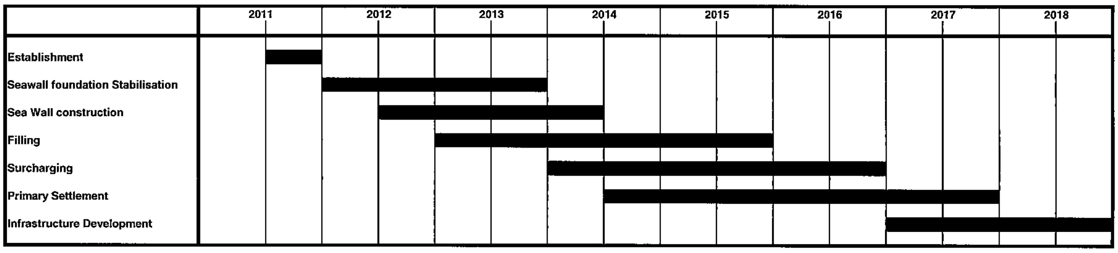
• Dredging	200,000 m^3 / barge / month
• Sandfilling	900,000 m^3 / Trailer / month
• Seawall Construction	50m / front / month
• Installation of PVD drains	60,000m / rig / month
• Installation of Sand compaction piles	15,000 m / barge / month

11.2.2 These rates have been derived by reference to the initial airport platform filling rates, filling rates achieved during the construction of Penny's Bay Stage 1 reclamation and from the design assumptions as reported in the working papers for the Tuen Mun Chek Lap Kok link boundary crossing reclamation. Discussions have also been held with the panel of experts consulted in respect of each reclamation technique to derive the maximum rates achievable in projects within the scope of their experience. By way of reference the following rates are currently being adopted in the preliminary design of the Tuen Mun Chek Lap Kok link boundary crossing reclamation.

- Dredging rates (11m3 grab) 4,200 m3 / barge / day
- Filling rates – pelican barge 2000 m3 / barge / day
- Seawall construction 1.7m / barge / day @ 16hrs /day
- Installation of band drains 8m / rig / min,
  5 min between drain locations
  and 19hr / day

11.3 Construction Programmes

11.3.1 Detailed construction programmes for each development scenario and restrictions regime are included in *Figures 11.1 – 11.6b* inclusive. The programmes have been developed by assuming the production rates detailed in *Section 11.2* above to derive the quantity of plant required to be mobilised and complete the works in line with the overall programme detailed below.



11.3.2 Programmes relating to the development of the site using deep cement mixing and sand compaction piles across the whole reclamation site, *Figures 11.3 and 11.4* have been prepared on the basis that sufficient plant could be mobilised to meet the programme constraints. In practice, it is unlikely that such large quantities of plant could be mobilised for these two particular options. It must be concluded therefore that whilst the programmes indicate that the required production rates could be achieved this is unlikely in practice. More realistic programmes for these two options would result in extension of the programme by approximately 3 years.

11.3.3 The baseline programmes have all been developed on the basis that work will be free to progress unhindered by any environmental impact related constraints. It has therefore been assumed that major equipment can operate on a 24 hour per day basis and that seawall construction can progress unhindered during both daylight hours and at night. At present it is uncertain whether restrictions applicable to not working during the dolphin calving season or whether fixed rates of dredging and handling of sand filling might be imposed as a requirement of the acceptance of any EIA. Programmes reflecting the impact of these restrictions are included as *Figures 11.1 – 11.6 revisions a.*

11.3.4 By way of assessment of the likely worst case programme scenarios applicable in the case of restrictions being applied in respect of 12 hour daytime working only and no piling or sediment generating operations between March and August during the CWD calving period, we have prepared adjusted and more lengthy programmes applicable to each construction method scenario. See *Figures 11.1 – 11.6 rev b* The programmes assume that the same plant configuration will be adopted in each case but that working efficiency will be reduced by 50% through the limiting the working day to a typical 12 hour period. A further 50% reduction (75% total) has also been taken into consideration as a result of the ban on piling works during the CWD calving period. It is proposed that a further 30% reduction (66% total) will be applicable in the case of dredging and filling works where an embargo will be applicable for the 4 most significant months of the CWD calving season. In both cases the restrictions have been discounted as they will be concurrent with the typhoon season when there will typically be 12 days of outage arising from No. 3 typhoon signals.

11.3.5 The conclusions derived from the programming assessment are presented below in *Table 11.1*

Table 11.1 ***Summary of proposed construction durations***

Option	Programme Reference	Scheme	Duration Months Note 1
Wick Drains	11.1	Seawall with Deep cement mixed core, sand compaction pile shoulders and rock armoured revetment Reclamation using wick drains	88 a 120 b 125 c
Sand Drains	11.2	Seawall with Deep cement mixed core, sand compaction pile shoulders and rock armoured revetment Reclamation using sand drains	86 a 123 b 122 c

Option	Programme Reference	Scheme	Duration Months Note 1
Sand Compaction Piles	11.3	Seawall with Deep cement mixed core, sand compaction pile shoulders and rock armoured revetment Reclamation using sand compaction piles	88 a 122 b 133 c
Deep Cement Mixing	11.4	Seawall with Deep cement mixed core, sand compaction pile shoulders and rock armoured revetment Reclamation using deep cement mixing	88 a 123 b 141 c
Piled Structure	11.5	Fully piled structure	86 a 141 b 141 c
Floating Structure	11.6	Floating Structure	81 a 81 b 81 c

Note 1 Duration (XX) a Assuming 24 hour working and no restrictions

(YY) b Assuming no summer working (March – August)

(ZZ) c Assuming No summer working or night work but maximum plant mobilised

12 CONCLUSION

12.1 This report has assessed the feasibility of construction of a reclamation of some 589Ha plan area together with protective seawalls over the top of the contaminated mud pits lying to the north of the existing airport platform at Chek Lap Kok. The study has made reference to a notional platform arrangement and to a more detailed arrangement as detailed on *Figure 1.2.* The arrangement, referred to as Option R (C+Y) has been taken as a baseline arrangement used in the cost estimating for the project.

12.2 Detailed consideration has been given to minimising the environmental impacts associated with the proposed construction works. Cost estimates and Construction Programmes have been developed by initially assuming a baseline case whereby 24 hour working throughout the year can be undertaken. Supplemental construction programmes and cost estimates have been devised assuming that construction would be restricted to avoid dredging, filling and piling works during the Chinese White Dolphin Calving season between March and August each year, limiting productivity by 50%. Similarly, programmes and cost estimates have been devised assuming that construction would be restricted to avoid dredging, filling and piling works during the Chinese White Dolphin Calving season between March and August each year and, additionally no night working would be permitted, representing a 25% productivity scenario.

12.3 The report has reviewed the particular geotechnical conditions at the site and has identified the particular constraints applicable to construction of the new reclamation. The particular site properties are described and discussed in *Sections 2* and *3* of the report.

12.4 A series of proposed design criteria for the platform development have been established and compared with those adopted in the development of the existing reclamation and seawalls at the site.

12.5 *Sections 4* and *5* of the report have taken the details of the site and the specific performance requirements for the development of a new reclamation and derived the specific site development criteria against which the development options have been assessed.

12.6 A series of possible construction techniques have been investigated with the objective of creating the required reclamation within a 7.5 year baseline construction programme. Details of the construction methodologies, their benefits and disadvantages have been reviewed in Section 6 of the report arriving at an overall favoured solution encompassing a combination of the assessed technologies. It has been concluded that electro-osmosis will probably not be feasible as a consequence of its lack of robustness and track record. Similarly, a number of questions remain in respect of the practicality of the use of semi buoyant construction techniques despite the fact that their adoption might be cost effective. Concerns over the long term stability of foam filling, potential fire risks and the vulnerability of the material in the event of a fuel leak have made this option less favourable from an engineering point of view. A summary of the findings of the engineering feasibility assessment together with ranking for the various options is presented in *Table 12.1*.

12.7 A number of consultation exercises have been completed with a number of overseas experts with the objective of identifying the most appropriate site formation technologies. The findings from these consultation interventions have been summarised in *Section 7* of the report.

12.8 *Section 8* of the report has addressed the issues associated with the preliminary environmental impact associated with each of the proposed development options. The PER has identified baseline conditions, the likely environmental impacts associated with each of the proposed construction methodologies and makes proposals in respect of the most appropriate mitigation measures where these can be defined at the present stage in the project. Particular attention has been paid to the environmental implications of the drained reclamation options and the need for mitigation of adverse impacts associated with the expulsion of potentially contaminated pore water during the proposed fill consolidation. A summary of the findings of the preliminary environmental assessment is given in *Table 12.2*.

12.9 Statutory issues associated with achieving the necessary approvals for the proposed works have been reviewed and the issues have been addressed.

12.10 *Section 9* of the report addresses the risk issues associated with the various proposals, formulating a risk register together with proposed mitigation procedures. In general mitigation of the identified key risks has been concluded to be possible in all except one option where it has been deemed to be onerous to implement the semi buoyant construction option. Fire risk, uncertainties over the long term stability of foam based lightweight filling systems and risks associated with possible fuel spills and the major risk associated with handling large quantities of Styrene monomer required by the semi buoyant construction option. A summary of the findings of the risk assessments is presented in *Table 12.3*. We note that the adoption of the semi buoyant filling technique should not preclude the construction of piled building structures although the views of Fire Services Department will need to be assessed before the adoption of this revolutionary technique is recommended for use in Hong Kong where concerns over fire safety may well dominate.

12.11 Cost estimates for each proposed construction technology option have been developed and are presented in *Section 10* of the report. The conclusions drawn from the estimating exercise have been drawn based upon a risk based assessment procedure. The costs associated with each option have resulted in ranking of the options and it has been concluded that the proposed optimised combination of construction technologies will provide the most appropriate and cost effective solution to the reclamation issues. The results to the cost estimating exercise are presented in *Tables 12.4a,b and c*. assuming the baseline case, 50% and 25% productivity respectively.

12.12 *Section 11* of the report has resulted in the development of a series of possible construction programmes. The viability of the various construction techniques has been assessed by deducing the quantity of plant, working fronts etc applicable to achieving the overall programme objectives for the project. Feasibility of the adoption of each option has subsequently been determined by assessing whether the proposed production rates are feasible. In some cases the viability of the schemes will be dependent upon the availability of specialist plant from overseas. There are particular concerns in this respect where the amount of plant required for the completion of the project can be mobilised within the timeframe applicable to the project requirements. A summary of the findings in relation to the assessment of the programme implications for each option is presented in *Table 12.5*. The programmes have been developed by assuming 24 hour working throughout the year as a baseline case. Further programmes for the main development options have been devised assuming 50% productivity arising from the restriction of dredging, filling and piling activities between March and August each year during the CWD calving period. Similarly, development programmes have been devised assuming 25% productivity arising from the restriction of dredging, filling and piling activities between March and August each year during the CWD calving period and at night time.

Table 12.1 Summary of Engineering Issues and Engineering Ranking

	Process / Technology	Robustness of Technology	Tried and Tested at large scale	Potential Difficulties	Availability of Plant	Vulnerability	Need for maintenance	Engineering Ranking
1	Sand Capping Layer	10 years experience not used in HK	Yes	Control over sand loss and uniformity in strong tidal currents	Small number of large specialist barges	Potentially slow and vulnerable in strong tides	None required once installed	Fair
2	Prefabricated Vertical Band Drains (PVD)*	20 years + experience + widely used in HK	Yes	Efficiency may be compromised under large settlements due to smearing. May require repeat installation	Many rigs available – can be mounted on conventional barges	None once installed Performance restrictions may require repeat installation of say up to 25-30%	Not required once consolidation is complete	Good
3	Vertical Sand Drains *	50 years + experience rarely used in HK	Yes but few in HK	Difficult to control in very soft materials & may not be fully effective under large settlements	Many rigs available – can be mounted on conventional barges	Vulnerable to weather and tides during deployment	Not required once consolidation is complete	Good
4	Sand Compaction Piles (SCP)*	20 years + experience not used in HK	Well established in Japan but not in HK	Some issues over efficiency + Possible eruption of sand blanket	Many rigs required with semi specialist plant. Availability a serious issue	Surface eruption may restrict barge access	Not required once consolidation is complete	Good
5	Deep Cement Mixing (DCM)*	20 years + experience not used in HK	Well established in Japan but not in HK	May be restricted where water depths are limited. Issues over consistency and potential loss of cement	Many rigs required with highly specialist plant. Availability a serious issue	Excess cement requirement or loss might occur in poor quality mud areas	Not required once consolidation is complete	Fair
6	Piled Structures	Extensively used for over 50 years common in HK	Yes	Driven piles require very large – noisy hammers Founding restrictions may apply. Bored piles are slow and will need to found at deep levels	Very large number of percussive piling rigs required. Availability a serious constraint. Potential issues over ability to drive to required depth.	Corrosion sensitive	Continuous inspection, corrosion protection and cathodic protection	Fair
7	Vacuum Consolidation	10 years + experience Never used in HK	Few examples at large scale in China but not in HK	Achieving the required vacuum under water may be difficult to achieve in large areas	Plant would all need to be fabricated	Not vulnerable once consolidation is achieved	Not required once consolidation is complete	Fair
8	Semi Buoyant Construction*	20 years + experience Never used in HK	No	Fire and damage in the event of fuel spill are an issue. Requires fabrication factory and handling of styrene monomer.	Major production plant and storage tanks to be constructed on site	Fire and accidental spillage of solvents. Long term stability of foams for 120 years not proven	Not required once consolidation is complete	Poor
9	Floating Structure (VLFS)	Japanese experience at large scale in past	Only few examples none in HK	Requires large number of ship yards.	Shipyard availability will be a significant	Corrosion sensitive. Long term	Continuous inspection, corrosion	Poor

	Process / Technology	Robustness of Technology	Tried and Tested at large scale	Potential Difficulties	Availability of Plant	Vulnerability	Need for maintenance	Engineering Ranking
		10 years. Never used for commercial airport construction		Assembly difficulties may ne associated with such large structures on site. Large scale mechanical plant and reliability issues	issue	maintenance commitment	protection and cathodic protection Continuous maintenance of pumps and electricity costs	
10	Electro-osmosis*	Extremely limited experience Never used in HK	Very limited experience and none in HK or in flooded sites	Never tried or proven on a large scale. difficult to achieve electrical effectiveness under water	Plant would all need to be fabricated	May not be effective beneath water at high moisture contents	Not required once consolidation is complete	Poor
11	Deep Well Dewatering (DWD)*	Widely used but not for settlement control, often used in HK	Yes	Many wells required. Pumps need to be located in free standing bores in sea	Plant would all need to be fabricated	Not vulnerable once consolidation is achieved	Not required once consolidation is complete	Fair
11	Rock Armoured Seawalls **	Widely used for over 100 years common in HK	Yes	Source of rock filling to be identified. Large volumes of materials required. Time consuming fill placement	Conventional plant available in large numbers	Vulnerable to extreme storm events	Minimal. only following extreme storm events	Very Good

* to be adopted within containment seawall
**to be applied in combination with another

Table 12.2 Summary of CMPs Interfacing Environmental Issues and Environmental Ranking

Process	Sand Blanket	Surcharges	Pore Water Extraction	Contaminated Sediment Plumes	Sediment Removal & Treatment	Reclamation	Environmental Ranking	Main Consideration
Sand Capping Layer (Sand Blanket) **	Yes	NA	NA	Minor	No	NA	NA	NA
Prefabricated Vertical Drains (PVD)	Yes	Yes	No	Minor	No	Yes	Fair	▪ Sediment plumes related to the main reclamation
Sand Drains	Yes	Yes	No	Minor	No	Yes	Fair	▪ Sediment plumes related to the main reclamation
Sand Compaction Piles (SCP)	Yes	No	No	Minor	No	Yes	Good	▪ Eliminate surcharge (extra fills) requirement; ▪ Sediment plumes related to the main reclamation
Deep Cement Mixing (DCM)	Yes	No	No	Minor	No	Yes	Fair to Good	▪ Risk of cement leakage affecting the water quality; ▪ Eliminate surcharge (extra fills) requirement; ▪ Sediment plumes related to the main reclamation
Piled Structures	No	No	No	Minor	Yes, about 0.5 Mm^3 of spoils inside the pile columns which could be contaminated. .	No	Poor	▪ High noise impacts to CWD during piling (6,540 piles); ▪ Management of dredged spoil; ▪ No reclamation and minimal impacts to water quality and ecosystem
Vacuum Consolidation	Yes	No	Yes	Minor	No	Yes	Good	▪ Sediment plumes related to the main reclamation; ▪ Eliminate surcharge (fill) requirement
Semi Buoyant Construction	Yes	Yes (reduced)	No	Minor	No	Yes	Good	▪ Reduction of fill material requirement (~35%) and thus sediment plumes related to the main reclamation ▪ Reduce surcharge (fill) requirement
Floating Structure (VLFS)	No	No	No	Minor	Yes, a small amount of spoils inside the anchorage pile columns.	No.	Good	▪ Some noise impacts to CWD during piling; ▪ No reclamation and minimal impacts to water quality and ecosystem
Electro-osmosis	Yes	Yes	Yes	Minor	No	Yes	Poor	▪ Generation of reactive chemicals that can potentially highly toxic; ▪ Sediment plumes related to the main reclamation; ▪ Potentially added concern of DC field effects on the ecosystem
Deep Well Dewatering (DWD)	Yes	Yes	Yes	Minor	No	Yes	Fair	▪ Sediment plumes related to the main reclamation
Rock Armoured Seawalls **	Yes	NA	No	Minor	No	Yes	Good	▪ Minimise the impacts of sediment plumes during main reclamation;

**to be applied in combination with another option. All the environmental ranking are relative to the PVD.

Table 12.3 Summary of Risk Assessment Findings and Risk related Ranking

Process	Engineering Risks	Environmental Risk	Financial Risk	Programme Risk	Total Risks	Mitigation	Risk Ranking
Sand Capping Layer *	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Prefabricated Vertical Band Drains (PVD)	5H 3M	2H 1M	1H 3M	1H 3M	9H 10M	Yes	Good
Vertical Sand Drains	6H 3M	2H 1M	1H 3M	2H 3M	11H 10M	Yes	Fair
Sand Compaction Piles (SCP)	6H 3M	2H 1M	1H 2M	1H 3M	10H 9M	Yes	Fair - Good
Deep Cement Mixing (DCM)	6H 3M	2H 1M	1H 2M	1H 3M	10H 9M	Yes	Fair – Good
Piled Structures	6H 4M	2H 1M	1H 2M	1H 3M	10H 10M	No	Fair – Good
Vacuum Consolidation	4H 4M	2H 1M	1H 2M	1H 3M	8H 10M	Yes	Very Good
Semi Buoyant Construction	6H 3M	2H 1M	1H 2M	1H 3M	10H 9M	Yes	Fair – Good
Floating Structure (VLFS)	6H 5M	2H 1M	1H 2M	1H 3M	10H 11M	Yes	Fair – Good
Electro-osmosis	6H 6M	2H 1M	2H 1M	2H 1M	12H 9M	Yes	Poor
Deep Well Dewatering (DWD)	6H 2M	2H 1M	1H 2M	1H 3M	10H 8M	Yes	Fair – Good
Rock Armoured Seawalls **	6H 2M	2H 1M	1H 2M	1H 3M	10H 8M	Yes	Fair – Good

* used in combination with other systems
**to be applied in combination with another

Table 12.4a Summary of Cost Estimates and Cost related Ranking baseline case (24 hour unhindered working)

Process	Estimated Construction Cost	Estimated Whole Life Cost	Estimated Average Risk Cost	Estimated Maximum Probable Risk Cost	Ranking
Sand Capping Layer	Included	Included	Included	Included	Preferred composite scheme
Prefabricated Vertical Band Drains (PVD) + surcharge	28.64	60.89	64.52	68.72	4
Vertical Sand Drains + Surcharge	28.49	60.74	64.35	68.53	2
Prefabricated Vertical Band Drains (PVD) + vacuum consolidation	29.87	62.12	65.91	70.28	12
Vertical Sand Drains + Vacuum consolidation	29.57	61.82	65.57	69.91	10
Prefabricated Vertical Band Drains (PVD) + surcharge + deep well dewatering	29.04	61.29	64.97	69.22	8
Vertical Sand Drains + + surcharge + deep well dewatering	28.74	60.99	64.64	68.85	6
Prefabricated Vertical Band Drains (PVD) + partial surcharge + semi buoyant	28.56	60.81	64.43	68.62	3
Vertical Sand Drains + partial surcharge + semi buoyant	28.41	60.66	64.27	68.43	1
Prefabricated Vertical Band Drains (PVD) + vacuum + semi buoyant	29.79	62.04	65.82	70.18	11
Vertical Sand Drains + vacuum + semi buoyant	29.50	61.74	65.48	69.81	9
Prefabricated Vertical Band Drains (PVD) + partial surcharge + semi buoyant + deep wells	28.96	61.21	64.88	69.12	7
Vertical Sand Drains + partial	28.67	60.91	64.55	68.75	5

Process	Estimated Construction Cost	Estimated Whole Life Cost	Estimated Average Risk Cost	Estimated Maximum Probable Risk Cost	Ranking
surcharge + semi buoyant + deep wells					
Sand Compaction Piles (SCP)	29.90	62.15	65.94	70.32	13
Deep Cement Mixing (DCM)	50.32	82.57	88.96	96.33	15
Piled Structures	42.20	64.18	69.54	75.72	14
Floating Structure (VLFS)	91.37	120.29	131.88	145.27	16
Electro-osmosis	Not considered	Not considered	Not considered	Not considered	Not considered
Rock Armoured Seawalls **	Included	Included	Included	Included	Preferred composite scheme

**to be applied in combination with another

Table 12.4b Summary of Cost Estimates and Cost related Ranking - 50% productivity – no working during summer months

Process	Estimated Construction Cost	Estimated Whole Life Cost	Estimated Average Risk Cost	Estimated Maximum Probable Risk Cost	Ranking
Sand Capping Layer	Included	Included	Included	Included	Preferred composite scheme
Prefabricated Vertical Band Drains (PVD) + surcharge	34.16	64.64	68.98	73.98	1
Vertical Sand Drains + Surcharge	34.61	65.09	69.48	74.55	3
Prefabricated Vertical Band Drains (PVD) + vacuum consolidation	35.06	65.54	69.99	75.12	5
Vertical Sand Drains + Vacuum consolidation	35.50	65.98	70.49	75.69	6
Prefabricated Vertical Band Drains (PVD) + surcharge + deep well	34.51	64.99	69.37	74.43	2

Process	Estimated Construction Cost	Estimated Whole Life Cost	Estimated Average Risk Cost	Estimated Maximum Probable Risk Cost	Ranking
dewatering					
Vertical Sand Drains + + surcharge + deep well dewatering	34.96	65.44	69.87	74.99	4
Prefabricated Vertical Band Drains (PVD) + partial surcharge + semi buoyant	37.79	68.27	73.06	78.60	7
Vertical Sand Drains + partial surcharge + semi buoyant	38.23	68.71	73.56	79.16	8
Prefabricated Vertical Band Drains (PVD) + vacuum + semi buoyant	38.97	69.45	74.40	80.11	11
Vertical Sand Drains + vacuum + semi buoyant	39.42	69.90	74.90	80.67	12
Prefabricated Vertical Band Drains (PVD) + partial surcharge + semi buoyant + deep wells	38.43	68.91	73.79	79.42	9
Vertical Sand Drains + partial surcharge + semi buoyant + deep wells	38.87	69.35	74.29	79.98	10
Sand Compaction Piles (SCP)	46.75	77.23	83.16	90.00	14
Deep Cement Mixing (DCM)	85.67	116.15	127.02	139.57	15
Piled Structures	45.41	65.63	71.40	78.05	13
Floating Structure (VLFS)	90.92	118.08	129.61	142.93	16
Electro-osmosis	Not considered	Not considered	Not considered	Not considered	Not considered
Rock Armoured Seawalls **	Included	Included	Included	Included	Preferred composite scheme

**to be applied in combination with another

Table 12.4c Summary of Cost Estimates and Cost related Ranking – 25% productivity – no work in summer months or at night

Process	Estimated Construction Cost	Estimated Whole Life Cost	Estimated Average Risk Cost	Estimated Maximum Probable Risk Cost	Ranking
Sand Capping Layer	Included	Included	Included	Included	Preferred composite scheme
Prefabricated Vertical Band Drains (PVD) + surcharge	44.43	75.67	81.31	87.81	1
Vertical Sand Drains + Surcharge	46.16	77.39	83.25	90.01	3
Prefabricated Vertical Band Drains (PVD) + vacuum consolidation	45.33	76.56	82.31	88.95	5
Vertical Sand Drains + Vacuum consolidation	47.05	78.29	84.26	91.15	6
Prefabricated Vertical Band Drains (PVD) + surcharge + deep well dewatering	44.78	76.02	81.70	88.26	2
Vertical Sand Drains + + surcharge + deep well dewatering	46.51	77.74	83.64	90.57	4
Prefabricated Vertical Band Drains (PVD) + partial surcharge + semi buoyant	47.06	78.29	84.26	91.16	7
Vertical Sand Drains + partial surcharge + semi buoyant	48.78	80.02	86.21	93.37	8
Prefabricated Vertical Band Drains (PVD) + vacuum + semi buoyant	48.24	79.48	85.60	92.67	11
Vertical Sand Drains + vacuum + semi buoyant	49.97	81.21	87.55	94.87	12
Prefabricated Vertical Band Drains (PVD) + partial surcharge + semi buoyant + deep wells	47.70	78.94	84.99	91.98	9

Process	Estimated Construction Cost	Estimated Whole Life Cost	Estimated Average Risk Cost	Estimated Maximum Probable Risk Cost	Ranking
Vertical Sand Drains + partial surcharge + semi buoyant + deep wells	49.43	80.66	86.93	94.17	10
Sand Compaction Piles (SCP)	70.32	101.55	110.48	120.78	14
Deep Cement Mixing (DCM)	146.43	177.66	196.24	217.69	15
Piled Structures	49.01	69.98	76.20	83.38	13
Floating Structure (VLFS)	90.92	118.83	130.37	143.69	16
Electro-osmosis	Not considered	Not considered	Not considered	Not considered	Not considered
Rock Armoured Seawalls **	Included	Included	Included	Included	Preferred composite scheme

**to be applied in combination with another

Table 12.5 Summary of Programme Issues and Programme related Ranking

Process	Potential Risk to Programme	Plant Availability Constraints	Environmental Constraints	Programme Ranking
Sand Capping Layer	Medium	Medium - High	N/A	n/a
Prefabricated Vertical Band Drains (PVD)	Low	Small	Medium	4, 2, 3
Vertical Sand Drains	Low – medium	Medium	High	5, 4, 2
Sand Compaction Piles /drains(SCP)	Low	Medium	Medium	1, 3, 4
Deep Cement Mixing (DCM)	Medium	High	High	7, 4, 6
Piled Structures	Medium	Medium	Low	1, 6, 6
Vacuum Consolidation	Medium – high	Medium	Low	3, 4, 5
Semi Buoyant Construction	Medium	High	High	7, 7, 7
Floating Structure (VLFS)	Medium	High	Low	4, 1, 1
Electro-osmosis	High	High	Medium	7, 7, 7
Deep Well Dewatering (DWD)	Medium	Medium	Low	6, 4 ,5
Rock Armoured Seawalls **	Low	Low	Medium	Favoured

**to be applied in combination with another
Ranking a, b, c assuming 100%, 50%, and 25% productivity restrictions respectively

12.13 In *overall* terms it has been concluded that the adoption of a scheme combining a number of technologies including deep cement mixing to provide lateral stability to seawalls, sand compaction piles to enhance strength, stiffness and settlement control of the soft clays and selective filling and surcharging can be used to arrive at a robust solutions to the project requirements within the required timeframe. It has been concluded that the settlement issues can probably be addressed satisfactorily and that the environmental impacts can be mitigated, although there remain uncertainties in a number of areas. An overall summary of the option assessments from the following points of view has been presented in *Table 12.6*. This table assumes the baseline case of 24 hour working throughout the year. *Table 12.7* presents the overall summary of the option assessments assuming the 50% productivity scenario where no marine earthworks or piling work is undertaken during the summer months. *Table 12.8* presents the overall summary of the option assessments assuming the 25% productivity scenario where no marine earthworks or piling work is undertaken during the summer months or at night.

Table12.6: Summary of Ranking – baseline conditions

Process	Engineering	Environmental	Risk	Cost	Programme
Prefabricated Vertical Band Drains (PVD)	Good	Fair	Good	3, 4, 7, 8, 11, 12	4 =
Vertical Sand Drains	Good	Fair	Fair	1, 2, 5, 6, 9, 10	5
Sand Compaction Piles	Good	Good	Fair - Good	13	2
Deep Cement Mixing (DCM)	Fair	Fair - Good	Fair – Good	15	7 =
Piled Structures	Fair	Poor	Fair – Good	14	1
Vacuum Consolidation	Fair	Good	Very Good	9, 10, 11, 12	3
Semi Buoyant Construction	Poor	Good	Fair – Good	1, 3, 5, 7, 9, 11	7 =
Floating Structure (VLFS)	Poor	Good	Fair – Good	16	4 =
Electro-osmosis	Poor	Poor	Poor	Not considered	7 =

Deep Well Dewatering (DWD)	Fair	Fair	Fair – Good	5, 6, 7, 8	6
Rock Armoured Seawalls **	Very Good	Good	Fair – Good	Favoured	Favoured

** to be applied in combination with another
= equal ranking

Table12.7: Summary of Ranking –50% productivity – (No summer working)

Process	Engineering	Environmental	Risk	Cost	Programme
Prefabricated Vertical Band Drains (PVD)	Good	Fair	Good	1, 2, 5, 7, 9, 11,	4 =
Vertical Sand Drains	Good	Fair	Fair	3, 4, 6, 8, 10, 12	5
Sand Compaction Piles	Good	Good	Fair - Good	14	1 =
Deep Cement Mixing (DCM)	Fair	Fair - Good	Fair – Good	15	7 =
Piled Structures	Fair	Poor	Fair – Good	13	1 =
Vacuum Consolidation	Fair	Good	Very Good	5, 6, 11, 12	3
Semi Buoyant Construction	Poor	Good	Fair – Good	9, 10, 11, 12	7 =
Floating Structure (VLFS)	Poor	Good	Fair – Good	16	4 =
Electro-osmosis	Poor	Poor	Poor	Not considered	7 =
Deep Well Dewatering (DWD)	Fair	Fair	Fair – Good	2, 4, 9, 10	6
Rock Armoured Seawalls **	Very Good	Good	Fair – Good	Favoured	Favoured

** to be applied in combination with another
= equal ranking

Table12.8: Summary of Ranking – 25% productivity – (No summer or night working)

Process	Engineering	Environmental	Risk	Cost	Programme
Prefabricated Vertical Band Drains (PVD)	Good	Fair	Good	1, 2, 5, 7, 9, 11	4 =
Vertical Sand Drains	Good	Fair	Fair	3, 4, 6, 8, 10, 12	5

Sand Compaction Piles	Good	Good	Fair - Good	14	1 =
Deep Cement Mixing (DCM)	Fair	Fair - Good	Fair – Good	15	7 =
Piled Structures	Fair	Poor	Fair – Good	13	1 =
Vacuum Consolidation	Fair	Good	Very Good	5, 6, 11, 12	3
Semi Buoyant Construction	Poor	Good	Fair – Good	9, 10, 11, 12	7 =
Floating Structure (VLFS)	Poor	Good	Fair – Good	16	4 =
Electro-osmosis	Poor	Poor	Poor	Not considered	7 =
Deep Well Dewatering (DWD)	Fair	Fair	Fair – Good	2, 4, 9, 10	6
Rock Armoured Seawalls **	Very Good	Good	Fair – Good	Favoured	Favoured

** to be applied in combination with another
= equal ranking

12.14 Whilst this report has not been required to arrive at a single site formation option but to assess the feasibility of a wide variety of options, it is apparent that site formation over the contaminated mud pits, based on well tried and tested reclamation technologies can be made to work in a cost effective, robust and environmentally acceptable way.

12.15 Formation of the site using more specialist techniques will be possible with environmental benefits being achievable. The more specialist techniques have been concluded to be less attractive from the point of view of engineering simplicity, robustness and cost effectiveness.

12.16 Site formation making use of deep cement mixing or sand compaction piles used widespread across the site could not be achieved within the favoured timescale.

12.17 This report has been prepared making use of existing site investigation data together with results from a specifically designed site investigation undertaken while the study progressed. At the commencement of the assignment little directly relevant data was available in terms of the engineering properties of the contaminated materials lying within the mud pits. The SI undertaken as part of this assignment has generated useful data, enabling the assignment to be progressed in a subjective manner. The scope of the site investigation completed under this assignment has necessarily been restricted in order that the programme could be met. It is strongly recommended therefore that a more comprehensive SI must be carried out during detailed design works for the site formation, the objective being to fully explore the variability of the materials and to comprehensively assess the geotechnical properties of the soft clays within the contaminated mud pits. Particular attention is recommended in further study to investigate the variability of the clay materials and to search for particular anomalies which might exist as a consequence of the varied nature of the fill material.

12.18 The analysis of engineering options has arrived at the conclusion that with the exception of the semi buoyant and electro-osmosis approaches each of the proposed options could achieve the overall objective of achieving the reclamation or platform necessary to support a future 3rd runway at Chek Lap Kok airport. In general terms the most cost effective solutions have been derived through the adoption of seawalls incorporating rockfill bunds founded on a sand blanketing approach with vertical stability being provided by cement stabilisation of the marine muds beneath the core of the seawalls and lateral stability being enhanced through the adoption of sand compaction piling of the shoulders to the embankments. The reclamation contained behind the seawalls has been deduced to be achievable through the adoption of a variety of techniques. In general, the most cost effective solutions to the bulk reclamation will lie with the adoption of surcharging of the filling and acceleration of the consolidation of the underlying marine deposits being achieved through the use of either prefabricated vertical band drains or sand drains. Arguments have been presented in favour of both schemes with marginal differences being apparent between the two options. The adoption of deep cement stabilisation and the use of vacuum consolidation are generally less favourable in terms of cost effectiveness and reliability. Deep well dewatering may be beneficial from the point of view of minimisation of environmental impacts arising from the release of pore water as the adoption of this process will facilitate treatment of the pore water more readily than for other systems.