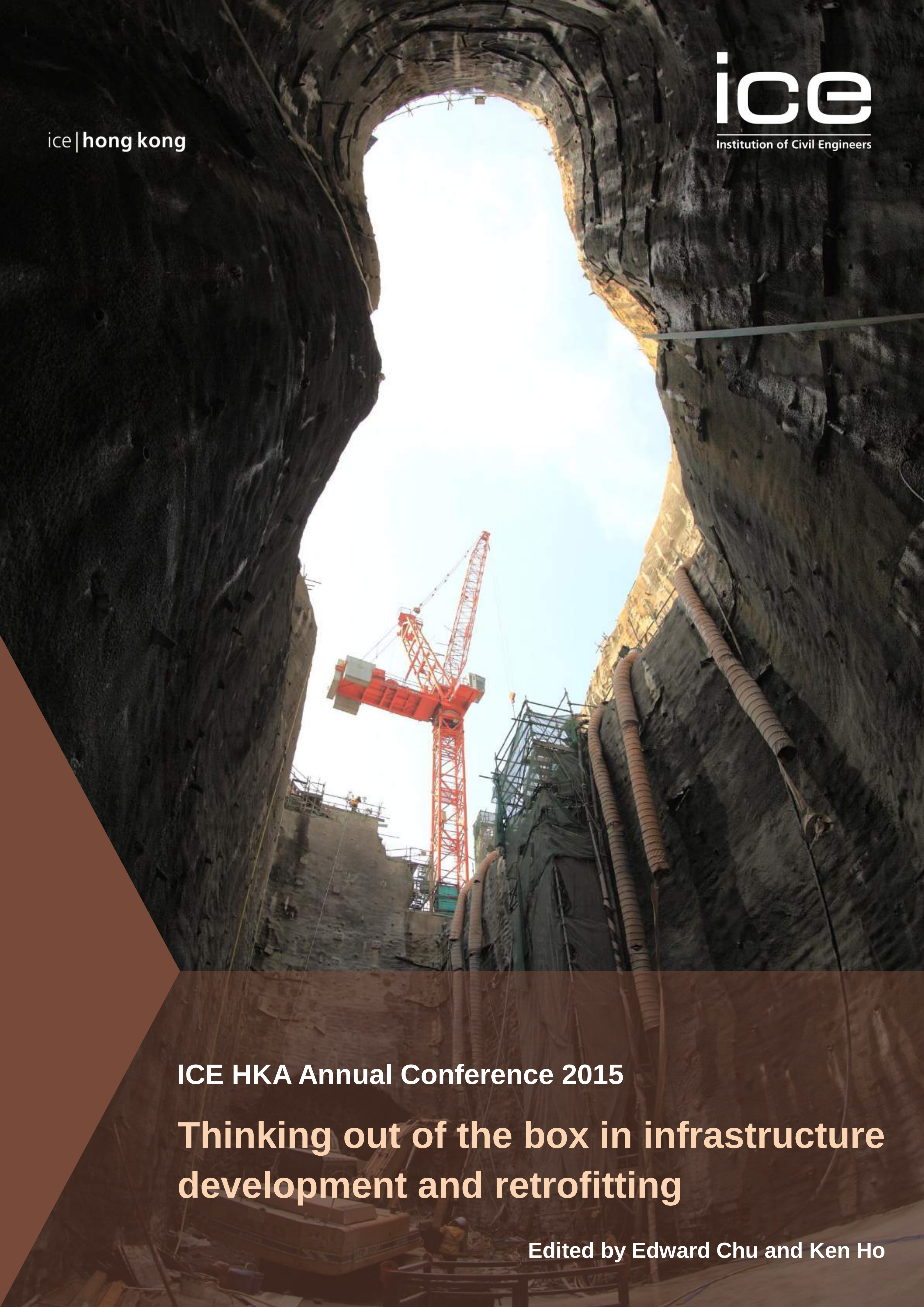

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ICE HKA Annual Conference 2015

Thinking out of the box in infrastructure development and retrofitting

Edited by Edward Chu and Ken Ho

Proceedings of the ICE HKA Annual Conference 2015

Thinking out of the box in infrastructure development and retrofitting

24 April 2015
Hong Kong

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Cover photograph (by Raymond HC Ho)
Deep excavation of intake structure for Tsuen Wan Drainage Tunnel

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Contents

Foreword

We are delighted to welcome all delegates to the ICE HKA Annual Conference 2015.

The theme of the conference is *Thinking out of the box in infrastructure development and retrofitting*. We have selected this theme to showcase the creative solutions involving innovations and new technology by practitioners and academia in tackling the engineering challenges that we are confronted with. The papers cover a wide range of topics including breakthroughs in material science, successful implementation of challenging infrastructure and building projects with major constraints, new visions in masterplanning, insights in enhancing city resilience against natural hazards, etc.

The 29 papers included in these proceedings represent a most valuable reference that provides vivid examples of the achievements made by civil engineers in using their ingenuity to solve practical problems. All papers have been peer-reviewed. The target of the Organising Committee was to strive for technical excellence. We believe this goal has been successfully achieved.

We are most honoured to have two eminent keynote speakers, namely Professor Victor Li from the University of Michigan and Dr Paul Toyne from the London Sustainable Development Commission, to share their wisdom with us. In addition, we are blessed with a large number of other distinguished speakers and authors, who are all leaders in their respective fields.

Special thanks are due to Professor Joseph Lee and Dr Andrew Chan who are our session chairmen, to Tim Warren who is the Chair of Asia Pacific Sub-Committee of ICE International Committee, and to the Organising Committee and Review Panel respectively for all their hard work behind the scene.

Ken Ho, JP
Chairman, Organising Committee

April 2015

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The Hong Kong Institute of Architects

Transport Department, HKSAR Government

Water Supplies Department, HKSAR Government

Reflections on the Conference

Tim Warren
Chair of Asia Pacific Sub-committee of ICE International Committee

1 OVERVIEW

As an overseas attendee to this prestigious conference I was surprised at the breadth and interconnectedness of the key points. These were extremely topical for me and were two fold – both technical (which was expected and provided information and insight for life-long learning at an international level) and, as importantly, on Civil Engineers learning to fulfil a social role in the development of modern cities. Specifically this second point came out as:

- Taking a more holistic review of the value and drivers of modern liveable cities,
- Raising the role of Civil Engineers professionally - in understanding that the needs of people are the reason for our work and therefore we have a role in shaping and guiding strategic policy and vision to satisfy people's social needs.

Traditionally engineers were thought of as those who provided technical solutions and innovations to improve construction efficiency and effectiveness. This conference showed us:

- Civil Engineers should again seeksolutions that reach past their projects, to understand the outcomes that they bring and how those outcomes can be focused on improving people's quality of life,
- Modern cities, to be successful, must be sustainable and provide an enhanced quality of life,
- Engineers who both engage in innovation and think about social dynamics will be able to serve their projects and employers much more effectively - neither of these are easy to achieve.

That, at a high level, Hong Kong clearly understands the changed drivers for a city to be 'world class' was clearly articulated. At a more detailed level the speakers' themes all touched on the competing issues and ideas that are the challenges of attaining this goal.

2 THOUGHT PIECE

I was really caught by two addresses - specifically:

Mr C.K. Hon, the Permanent Secretary for Development – in his Opening Address used the quote that "It is dreams that make change"[*1]. He noted that infrastructure should be developed for people's needs – and engineers need to be innovative to overcome significant current challenges (productivity levels, workforce availability, better H&S performance, constrained expenditure) to achieve these needs. He also highlighted a set of the Hong Kong Government's initiatives to help engineers in this challenge, including the use of BIM, ECI (NEC), improved collaboration and innovative tendering solutions.

Mr C.S. Wai, the immediate past Permanent Secretary – in his Special Lecture "The Ties that Bind"– suggested that raising both social interaction and social happiness is our goal as engineers and technology and the resultant move to a world dominated by mass immediate communication (through personal smart phones) was impersonalising us and weakening the ties between people. He concluded that Civil Engineers should work to bring people back together and cities such as Hong Kong need to be both high density and of high quality to be successful – and that this requires sustainability to be high on an engineer's agenda. He also noted the Hong Kong Association's connection with the ICE's Shaping the World initiative as a catalyst for improvement*[2].

The theme of 'thinking out of the box' was often mentioned – and for me the conference's best 'thinking out of the box' ideas are encapsulated by these two themes, alongside some really world class technical innovations.

3 SPECIFICS

All the presentations brought ideas and explored issues around innovation in developing and retrofitting – below are a few personal thoughts and comments on points that caught my attention:

- Prof Victor Li's Keynote Address looked at the application of innovative material solutions in the context sustainability and modern cities. It highlighted his success in connecting research to the market place – true innovation. This covered very clever material solutions for, amongst other things, bendable concrete (the earthquake resistant properties (plastic hinge) of this alone have applications for New Zealand and right around the Pacific).
- Dr Gary Chou's views from the supply side in using technical innovation to meet human needs (and those of race horses) was very incisive and well presented.
- Eng. Leung on working beyond the Euro Codes for seismic bridge loading on a world class project was excellent. His timely comments on the difficulty/constraints in getting innovative ideas accepted into the procurement and approval process sat very well with the audience as well.
- Dr Paul Toyne's Keynote Address explained the nine megatrends that are putting pressure on a city's quality of life. He then clearly articulated the need for cities to manage these and their own issues. He noted the best are moving from measuring resilience to thinking about 'betterment' and future proofing. He also noted the importance of sustainability, not only globally, but as a tool to directly improve city living and therefore the quality (and its world ranking) of that city – he was also able to illustrate this through examples from local contractor Gammon.
- Three presentations in the afternoon looked at a set of Hong Kong based practical examples – in covering both the benefits and challenges these were very useful. Specifically
  - The Hong Kong HA, as the world's largest property organisation, showcased ideas that other countries should adopt – such as planning for a 100 year operational life, understanding that, when introducing innovation, an engineer's skill set should include the ability to manage stakeholders and regulators and the use of a five step 'virtuous circle' for introducing innovation (including through the procurement process and in measuring effectiveness – both are often forgotten in other jurisdictions).
  - The smart grid presentation (with all the risks and drivers) was very well delivered, with a detailed analysis of the practical and commercial implications/risks of district heating systems– I found this discussion very pertinent to other cities – in Auckland distributed networks have been discussed and, like other cities, we know change is coming but how this will manifest itself is not clear. Therefore the insights given were very useful.
  - Phyllis Li's Special Address - although talking about Hong Kong specific transportation issues - touched on the important social issues of isolation in communities and the impacts of long commutes prevalent in many expanding cities. She provided an explanation of the concept of Smart Growth and, in acknowledging that land constraints and rising employment numbers will not go away, showed that innovative planning (both social and engineering) are needed to lift the quality of life.

4 CONCLUSION

These well attended and world class presentations covered diverse but linked themes and provided a set of real learnings for the attendees, whilst challenging some of our normal thinking both technically and socially. The organisation and facilities were quietly efficient and my congratulations go to Mr Ken Ho and his organising committee for a job well done.

Coming from Auckland in New Zealand – itself being a city that both has an aspiration to grow by being recognised as "the most liveable city' in the world in a country and that sits on the Asia Pacific's 'ring of fire' - it was refreshing and very interesting to compare and contrast the issues and innovation being applied in Hong Kong. We live in a global market and our peer groups and comparator cities stretch right across the region – and there was plenty for me to take back to discuss with officials and engineers in both NZ and Australia.

Looking forward from this successful conference, the ICE's Shaping the World initiative is a logical next place for attendees and interested parties to go to continue their life-long learning around these themes*2.

Notes:

*1 Martin Luther King Jr.
*2 ICE's Shaping the World funding has enabled the HKA to work with the Chinese University of Hong Kong's Institute of Future Cities to undertake a study looking at the long term infrastructure development needs to 2030. This Study will be published as a Report before the end of 2015.

Shaping The World

LEARNING AND EXHIBITION CENTRE

OVERVIEW

- The world's first state-of-the-art, interactive learning and exhibition space devoted to the broad disciplines of civil engineering
- Situated at the ICE headquarters in Westminster to open in October 2016 with an exhibition on transport and communications
- Patron: HRH the Princess Royal to host business receptions and events

TARGET AUDIENCE

- Government, industry, clients, supply chain, secondary/primary schools, civil engineering students, practising engineers, public and parents

OBJECTIVE

- Raise the profile of the built environment/impact of infrastructure for social good
- An platform to debate key issues and innovative solutions to global infrastructure challenges - urbanisation, climate change, energy shortages, demographic changes
- Support ICE thought leadership on industry themes – innovation, transformation
- Promote STEM activities to inspire the next generation to consider a career in the industry and prime the talent pipeline to address the lack of engineers

PROJECT OVERVIEW

The interactive space will bring engineering to life, supporting inspiration and skills development. The Centre will be designed to engage young people and will include areas such as exhibition spaces, 3D simulators and models. The design visuals above show a flexible space that will accommodate industry displays, case studies, simulated learning technology and corporate entertaining.

Technology	Hardware: Display equipment, interactive boards, computers, 3D printers
	Software: BIM and design software, virtual learning environment, 3D simulation, gaming, e-learning, digitisation of material
Training Aids	Interactive models, materials, stationery, 3D printer consumables, games, specialist furniture
Exhibitions	Models, archival material, display materials
Engagement	Staff involvement to educate and inform exhibition and learning attendees

SUPPORTING DATA

- There will be over 2.5m engineering job opportunities worldwide between now and 2022.
- An increasing, urbanising population, more frequent extreme weather and rapid digitalisation requires a broader mix of skills to meet the growing demands for infrastructure projects
- Civil engineering and the value of infrastructure have a low public profile.

Contact: Jeanette Grose, Head of Development Shaping the World jeanette.grose@ice.org.uk.

Keynote Address

Re-engineering concrete for resilient and sustainable infrastructure

Victor C. Li
University of Michigan, USA

Keywords:

ABSTRACT: This paper considers the requirements of construction materials to support civil infrastructure resiliency and sustainability. Relevant properties of a re-engineered ductile concrete, named Engineered Cementitious Composite (ECC), are reviewed under this framework. Based on a growing body of experimental data, it is suggested that the tensile and compressive ductility, the damage tolerance and tight crack width characteristics, and the self-healing functionality of ECC provides the foundation of a materials technological platform that contributes to structural durability and resiliency. The technology is undergoing a transition from laboratory studies to full-scale field applications. The state-of-the art of ECC technology is illustrated with highlights of an application in bridge deck retrofit aimed at enhancing infrastructure sustainability, and an application in a new building design aimed at enhancing building resiliency under earthquake loading.

1 INTRODUCTION

Civil infrastructures are products. Like all other products, the need for their design for sustainability is increasingly recognized. Unlike most other products however, civil infrastructures, such as building and transportation systems, and water and energy systems, are typically large in size and are supposed to last as much as a hundred years or more. They are often seen as public monuments. Their large size implies large material flow, and their long lasting nature implies a long life cycle, during which multiple repair events may be needed. Both features carry serious implications on sustainability in terms of material production, construction and reconstruction, reduced operation/occupation (down-time for maintenance) and associated economic and environmental costs. Suitable civil infrastructure design can have large and lasting impacts on the quality of life of citizens in developing and developed countries.

Civil infrastructures are often public assets. We depend on them to sustain our everyday activities. The importance of their resiliency to extreme loads whether natural or manmade, is well known. A lack of resiliency goes beyond the concern for public safety, but could lead to severe interruption of daily life and business that can have significant economic impacts, especially in dense urban communities.

It is perhaps obvious that civil infrastructures should be designed to be both sustainable and resilient, while being economical (part of being sustainable). In a recent article by Peng et al. (2012), it is argued that true sustainability should embody resiliency. While few will argue against this philosophy of integrated sustainability and resiliency, the question is how to achieve this in practice. This is a subject of active research.

For concrete infrastructure, there is intrinsic challenge due to the brittle characteristic of concrete material. Despite the common practice of steel reinforcement, severe damage or even collapse of infrastructure continues to occur in major seismic events, requiring expensive and lengthy periods of recovery, not to mention the loss of life. Under normal service loads, the problem of cracking continues to afflict both new and old structures, leading to poor durability and repeated repair requirements. These considerations suggest that we are far from the ideal of being able to build civil infrastructures that are both resilient and sustainable, despite a significant amount of research on structural safety design and on high performance concrete materials.

Much of the research that has gone into concrete materials over the last two decades have focused on either making concrete stronger (typically meaning higher compressive strength) or making concrete greener (typically using recycled ingredients and/or reducing the cement content). It is generally recognized that concrete with higher compressive strength actually becomes more brittle, and that greener concrete is not equivalent to more durable concrete. The challenge of creating damage tolerant concrete that supports infrastructure resiliency and sustainability remains.

This paper overviews the material characteristics required to support sustainability and resiliency of civil infrastructure systems. It is suggested that a newly developed ductile concrete named Engineered Cementitious Composites (ECC) provides a viable platform to meet this ambitious goal. Recent field applications of this material are used to illustrate how specific features of ECC are utilized to realize the infrastructure design goals of simultaneous resiliency and sustainability.

2 REQUIREMENTS FOR MATERIAL CHARACTERISTICS TO SUPPORT INFRASTRUCTURE SUSTAINABILITY AND RESILIENCY

The global construction and reconstruction of concrete infrastructures result in enormous flows of materials between natural and human systems. To support the sustainability of infrastructure, it is necessary for concrete material to be green with low embodied energy and carbon emissions in its production process. While this is a necessary condition, it is not sufficient. A green concrete that requires repeated repairs in the resulting infrastructure not only leads to increased volume of material used in the whole life cycle, but also negatively affects the use pattern of the infrastructure. For example, based on a detailed life cycle model, Keoleian et al. (2005) found that the use phase dominates over other phases of the life-cycle of a bridge deck; repair events and associated traffic impacts lead to primary energy consumption and equivalent carbon dioxide emission that overwhelms all other phases (material production for the initial construction, construction, transport and end of life demolition). This study highlights the important contributions of concrete infrastructure durability to sustainability indicators. For truly sustainable infrastructure design, an ideal concrete material should be both green in production and durable in use.

While a number of definitions on infrastructure resiliency have been offered, perhaps the concepts advanced by Bruneau et al. (2003) is most helpful for the present discussion. They stated that three characteristics must be met for an infrastructure to be resilient:

a) Reduced failure probability
b) Reduced consequences from failure
c) Reduced time (and cost) to recovery

Figure 1 (adapted from Bruneau et al., 2003) illustrates how the quality of an infrastructure degrades to 50% due to a major loading event at t_0, and gradually recovers to full functionality at time t_1. The green dashed line represents the quality profile of a more resilient infrastructure that degrades less by the same loading event, and recovers to 100% functionality at a time period below that of (t_1 - t_0).

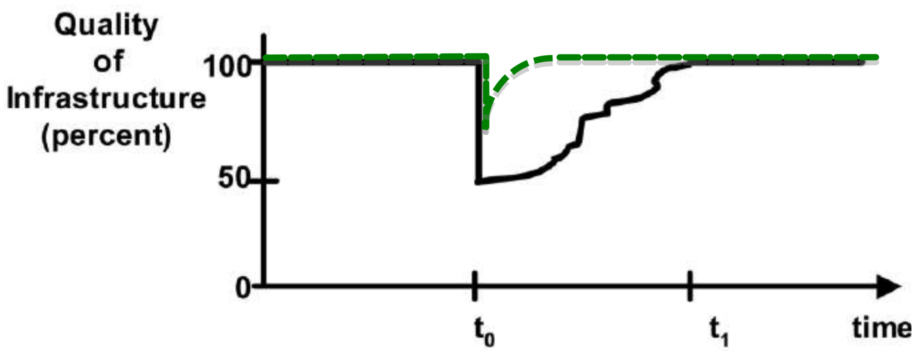


Figure 1 A resilient infrastructure degrades less and recovers faster after a major loading event (adapted from Bruneau et al., 2003)

From a concrete material standpoint, high material tensile ductility can be utilized to design structures with reduced structural failure probability (see e.g. Gencturk, 2013). This is particularly true as most concrete structural collapses result from brittle fracture rather then concrete compressive crushing. Further, material damage tolerance can directly contribute to reducing the consequences of failure (Gencturk, 2013, Fukuyama et al., 2000 and Billington and Yoon, 2004); controlled damage may lower the material and structural stiffness but retain load carrying capacity. Finally, if concrete crack width can be controlled so that they experience self-repair without external intervention, the recovery period will be shortened and repair cost will be reduced. A concrete that is ductile, damage tolerant and possesses the ability to undergo self-healing will contribute to enhancing infrastructure resiliency.

To summarize, the requirements for a structural material to support civil infrastructure sustainability and resiliency are:

a) The material should be green
b) The material should possess tensile and compressive ductility in addition to strength
c) The material should exhibit damage tolerance when overloaded
d) The material should be durable and should limit crack width in support of structural durability
e) The material should self-repair when damaged

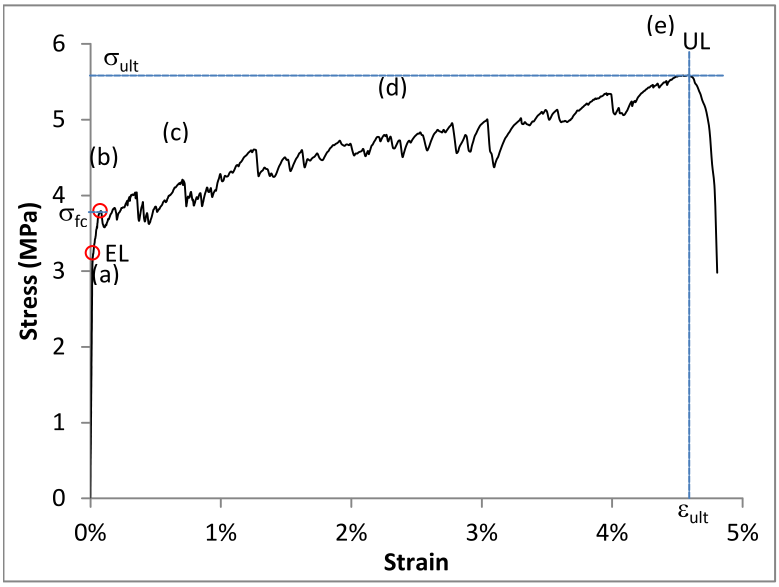
These requirements are difficult to meet, especially in a single material. The integration of these requirements, however, should be the goal of design for the next generation concrete. Implicit in the above, especially recognizing the large volume usage of concrete material in infrastructures is that these requirements need to be met with cost effectiveness in mind, in order for such a material to be meaningful in an industrial context. The pursuit of a single objective, for example, green or high strength, would not be adequate for future civil infrastructure systems demanding sustainability and resiliency.

3 A NEW CONCRETE FOR INFRASTRUCTURE SUSTAINABILITY AND RESILIENCY

A plausible technology platform to support infrastructure sustainability and resiliency is Engineered Cementitious Composite (ECC) (Li et al., 2001). The most important characteristic of ECC is its ability to strain-harden when deformed beyond the elastic limit under tensile or compressive loading. Under compression, the strain capacity is about twice that of normal concrete. Under tension, ECC exhibits over 3% strain, or more than 300 times the strain capacity of normal concrete.

A typical stress-strain curve of an ECC in tension is shown Figure 2. The elastic limit (EL), first cracking (σ_{fc}) strength, and ultimate limit UL at tensile strength σ_{ult} and strain capacity ε_{ult} are identified on this curve. Between the elastic and ultimate limits, multiple microcracking occurs, and gives rise to inelastic straining with increasing tensile stress. For this reason, the material is also known as Strain-Hardening Cementitious Composite (SHCC) (van Zijl et al., 2010). This strain-hardening branch on the stress-strain curve is typically absent in concrete materials, but is responsible for the unique tensile ductility and damage tolerance of ECC materials.

The development of crack pattern of an ECC (Lepech and Li, 2006) is unique among concrete materials. Instead of a single fracture leading to failure, distributed microcracking of tight cracks is associated with increasing load (Figure 3).



(a)

(b)

Figure 2 ECC shows (a) high tensile ductility with over 3% strain capacity, and (b) multiple saturated multiple cracking

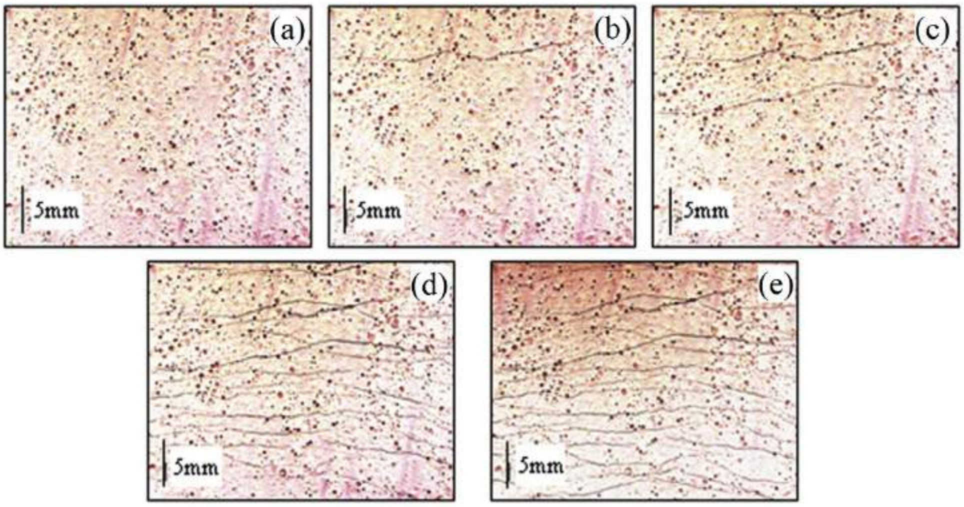


Figure 3 Development of crack pattern in ECC (Lepech and Li, 2006): Increasing number of cracks with limited crack width under increasing imposed tensile load. The five stages (a-e) of damage pattern correspond approximately to the loading stages indicated in Figure 2

The damage tolerance of ECC can be demonstrated by the response to uniaxial tensile loading of a double edge notched specimen. Figure 4 (Li, 2009) shows the distributed inelastic deformation away from the notched plane, a behavior distinctly absent for notch sensitive material such as normal concrete or ordinary fiber reinforced concrete.

Double edge specimens with different notch depths have been tested (Li, 2009). The tensile strengths of these specimens were found to remain essentially constant regardless of the notch depth.

The underlying principle of ECC design (Li, 2003, Yang and Li, 2010 and Li, 1997) is controlled load transfer between fiber and matrix, in such a manner that cracks emanating from initial defects in the composite matrix are bridged by fibers regardless of the length of the crack, and that tensile loading on fiber bridges are limited to below the bridging capacity. These requirements are embodied in a set of micromechanical tools that can be used to tailor fiber, matrix and fiber/matrix interface to attain the high tensile ductility (Li, 2012).

The same set of micromechanics tools can be utilized to control the width of the microcracks (Li, 2012). In ECC, microcracks are typically constrained to less than a hundred micron; in some cases as low as 20 microns depending on the fiber type, surface coating and interface tailoring.

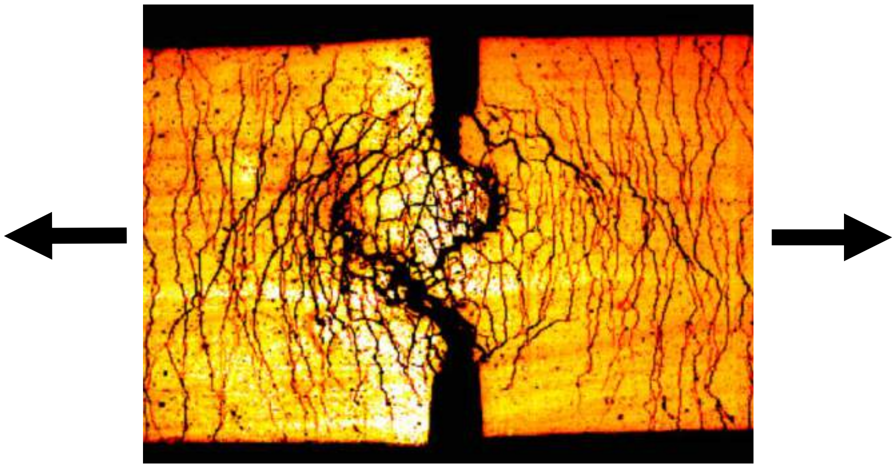
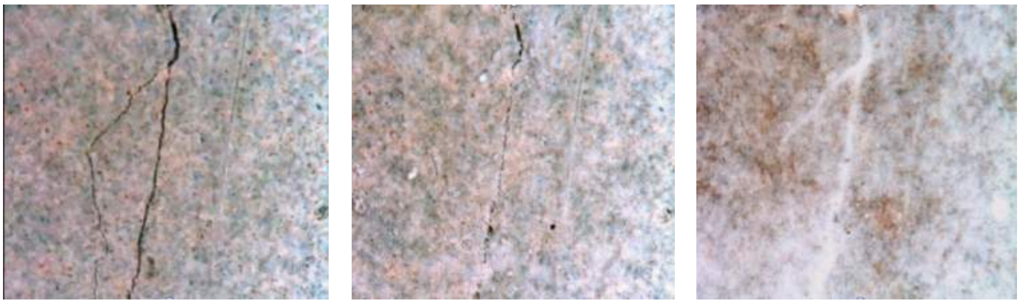


Figure 4 ECC shows high damage tolerance (Li, 2009) by diffusing inelastic deformation and delaying localization of fracture. The brittle fracture mode of failure is fully suppressed



(a) (b) (c)

Figure 5 ECC microcracks self-heal and recovers mechanical stiffness, strength and ductility: (a) load induced damage, (b) partial healing, (c) complete healing

The tight crack width of ECC can be exploited to aid in self-healing (Herbert and Li, 2013 and Yang et al., 2011) even after the composite has undergone imposed deformation of several percent. Self-healing occurs through a combination of continued hydration, pozollanic reaction, and calcite formation processes (Sakulich et al. 2010). Figure 5 shows the different stages of self-healing of preloaded specimens (to 3% in tension). Self-healing of the specimen took place by exposure to water and air.

A major drawback of ECC, similar to most high performance cementitious composites, is the fact that the material tends to have a higher cement content per unit weight of material, due to the elimination of coarse aggregates (Keoleian et al., 2005). This drawback impacts on economics, potential durability performance due to higher drying shrinkage, and on material greenness.

The concern on drying shrinkage of ECC has been addressed by several researchers (Yang, 2007a, Zhang et al., 2009 and Şahmaran et al., 2009). These researchers deploy shrinkage reducing, shrinkage compensating or internal curing agents to counteract the lack of internal shrinkage restraint by coarse aggregates. The resulting ECC has shrinkage similar to that of normal concrete.

The concerns on economics and material greenness have been addressed with material ingredient substitutions. For example, partial cement substitutions using fly ash, slags, or mine (iron ore) tailings have shown success (Huang et al., 2013a, Huang et al., 2013b, Huang et al., 2013c, Yang et al., 2007, Wang and Li, 2007, Lepech et al., 2008 and Zhou et al., 2010). Even total substitution of cement using fly ash base geopolymer has shown promise.

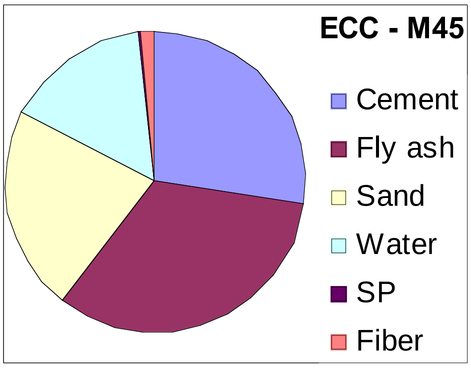


Figure 6 The weight fractions of ingredients in a “standard” ECC

Figure 6 shows the material composition of a “standard” ECC (M-45) that has been extensively studied. Yang et al. (2007) investigated ECCs with higher fly ash contents, and found that a F/C ratio up to 5.6 is feasible, although a reduction in compressive strength can be expected. However, they also demonstrated that an ECC with a F/C ratio of 2.8 could retain a tensile ductility of 3% and compressive strength of 35.2MPa at 28 days. Apart from enhancing economics and greenness, higher fly ash content and reduced cement content have been found to improve fresh properties, reduce drying shrinkage, heat of hydration as well as the width of the multiple microcracks.

Figure 7 (Huang et al., 2013a) shows the contributions to primary energy consumption and CO_2 emissions by the various material ingredients that make up ECC – M45 (Figure 6). The fly ash being a recycled material from coal-based power plants is assumed not to contribute to the carbon and energy footprints. From this figure, it can be seen that portland cement is the major contributor to CO_2 emission, whereas cement and PVA fiber contribute approximately equally to primary energy consumption. This implies that a more complete greening of ECC requires addressing also the fiber type used in ECC, despite its relatively small content in the composite.

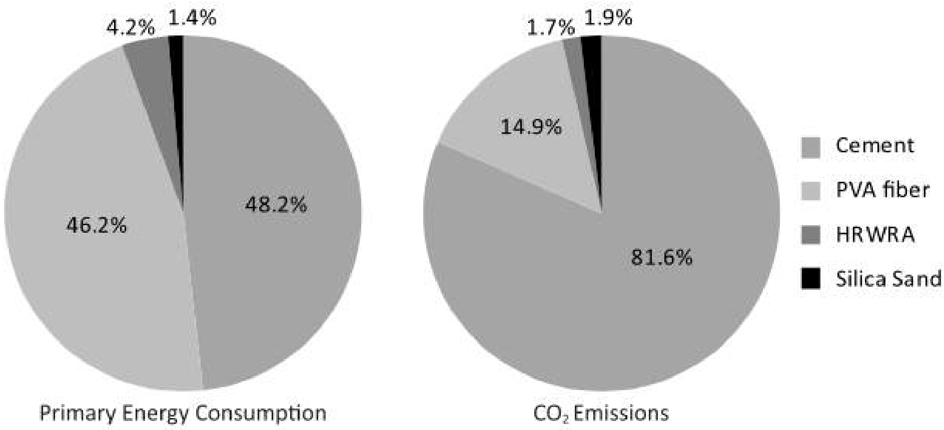


Figure 7 PVA fiber and portland cement dominates the carbon and energy footprints of ECC. These are targets for replacements to achieve greener ECCs

Primary energy consumption can be reduced by replacing PVA fiber with a density of 1.3gm/cc with PP fiber with a density of 0.9gm/cc. The reduction in density leads to a saving of about 30% purely because ECC tensile strength and ductility are governed by fiber volume fraction rather than weight fraction, assuming the same fiber content is employed. This is in addition to the lowering of energy in the production process of PP fiber when compared with PVA fiber. The development of ECC with PP fiber has been conducted by Yang (2007) and Felekoglu et al. (2014). Further reduction in carbon and energy footprints (and economic cost) can be achieved by the use of plant fibers that are renewable. Recent attempt (Soltan et al., 2015) to do exactly this indicates the feasibility of this approach, but a lowering of mechanical performance of the resulting ECC can be expected.

4 EXPERIMENTAL DEMONSTRATION OF STRUCTURAL RESILIENCY USING ECC

A variety of experimental testing of structural elements have demonstrated the ability of ECC to limit the amount of damage while maintaining structural load capacity, especially under simulated seismic loading (Fischer and Li, 2002, Fukuyama et al., 2000 and Billington et al., 2004).

Figure 8 (Fukuyama et al., 2000) shows the reinforcement detailing of a shear beam subjected to fully reversed cyclic loading. The level of damage experienced by the R/ECC beam is substantially smaller than that of the similarly reinforced R/C beam, with bond splitting and cover spalling completely suppressed (Figure 9). The hysteresis behavior of the R/ECC beam shows no pinching, but full loops, and stable load capacity up to a deflection angle of 5% radian (Figure 10).

The features of R/ECC with a large number of microcracks but no bond splitting and cover spalling, stable load and high energy absorption are common to many different experimental tests of R/ECC elements. They are the direct consequences of the high tensile ductility and damage tolerance of ECC material.

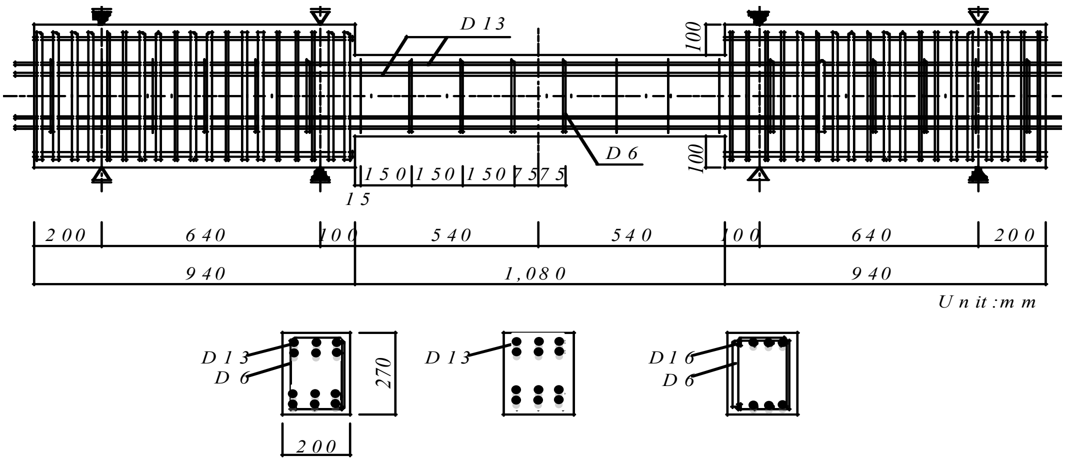
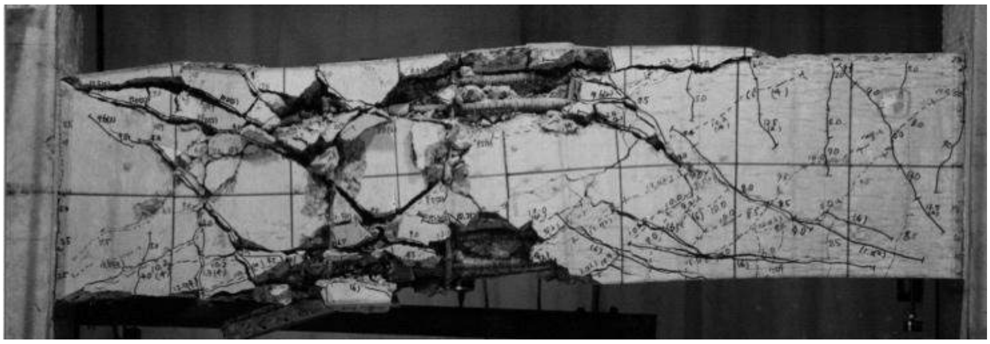
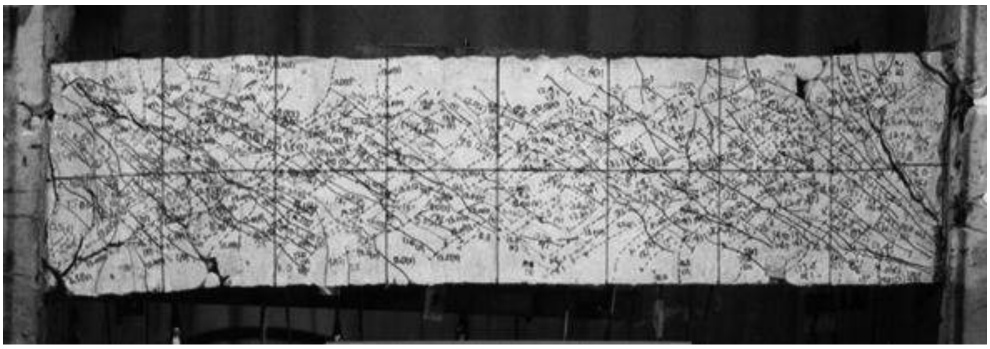


Figure 8 Reinforcement details of a beam subjected to fully reversed cyclic loads (Fukuyama et al., 2000)

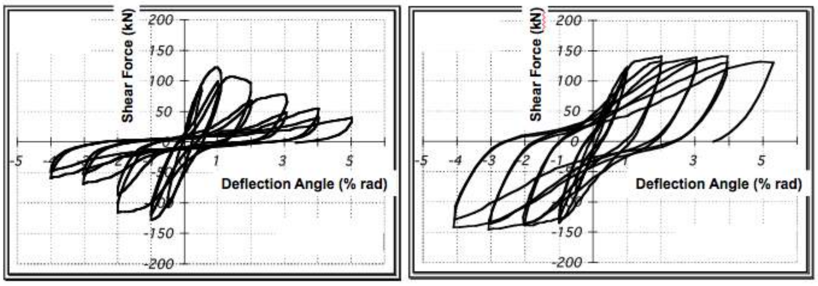


(a)



(b)

Figure 9 Damage level of (a) standard R/C member, and (b) R/ECC member, showing the suppression of bond splitting and cover spalling (Fukuyama et al., 2000)



(a) (b)

Figure 10 The hysteresis loops of (a) R/C member, and (b) R/ECC member. The latter shows no pinching and more stable load capacity even up to large deflection angle (Fukuyama et al., 2000)

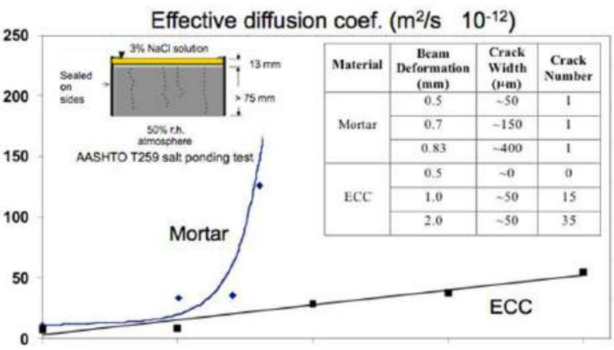
In a study on the connection of a hybrid steel beam – R/C column subjected to reversed cyclic loading, Parra-Montesinosand Wight (2000) found that the ECC shear panel experienced the least amount of damage when compared with eight other specimens with different materials (FRC) and reinforcement (stirrups, steel band plates, steel box cover plates) combinations, despite the deliberate absence of joint transverse reinforcement.

A recent study by Gencturk (2013) led to the conclusion of lower initial cost of ECC frames designed for seismic loads as a result of reduction in material and labor cost associated with the elimination of transverse reinforcement in the ECC frames when compared with R/C frames. The life cycle cost was also found to be reduced due to increased capacity and reduced demand for ECC frames.

5 EXPERIMENTAL DEMONSTRATION OF STRUCTURAL DURABILITY USING ECC

A variety of material durability studies have been carried out, including resistance to frost (Sahmaran et al., 2012), freezing and thawing (Sahmaran et al., 2009), deicing salt scaling (Sahmaran and Li, 2007), hot and humid environment (Li et al., 2004), alkali silicate reaction (ASR) (Sahmaran and Li, 2008), fatigue failure (Suthiwarapirak et al., 2002), and abrasion and wear (Lepech and Li, 2006). These studies indicate that ECC has improved durability over that of normal concrete.

Beyond material durability, infrastructure sustainability demands that the material supports durability of the structure under load. In concrete structures, the presence of cracks and subsequent rise in the effective diffusion coefficient of chloride ions is a major cause of loss of structural durability in many parts of the world, especially in coastal regions where the structures are exposed to a salt environment, or in cold regions, where salt is used in winter seasons for deicing purpose. Structural durability requires limiting the migration of aggressive agents through the concrete cover in order to delay the corrosion of the reinforcing bars.



Material	Beam Deformation (mm)	Crack Width (μm)	Crack Number
Mortar	0.5	~50	1
	0.7	~150	1
	0.83	~400	1
ECC	0.5	~0	0
	1.0	~50	15
	2.0	~50	35

Figure 11 Measured effective diffusion coefficient of ECC shows a slower rise with preload deformation level when compared with mortar (Sahmaran et al., 2007)

Sahmaran et al. (2012) studied the effective diffusion coefficient of ECC subjected to a 3% NaCl solution ponding. The specimens were obtained from a beam that has been deliberated preloaded to beyond the elastic stage. In order words, the specimens subjected to NaCl ponding had experienced microcracking, thus simulating a loaded structural environment. Figure 11 shows the effective diffusion coefficient as a function of preload deformation. In the case of ECC, this coefficient increases linearly with the amount of preload deformation, due to the increase in the number of microcracks that maintain a crack width of about 50μm. In contrast, the control mortar specimen shows a highly nonlinear increase of this coefficient, resulting from the increase in width of a single crack. The much slower linear increase of the effective diffusion coefficient of ECC suggest that ion penetration would be significantly slower, resulting in a time delay in corrosion of rebar and a longer service life of the structure.

A complementary investigation (Miyazato and Hiraishi, 2005) measured the corrosion rate in mm/year of the reinforcing bar inside the preloaded R/ECC specimen subjected to a 28 days chloride accelerated environment. This accelerated environment includes cycles of 2 day wetting (saltwater shower 90% RH), and 5 day drying (60% RH) while the macro crack in the R/C and the microcracks in the R/ECC were kept open (not unloaded). The result is shown in Figure 12. In the case of R/ECC, a small amount of corrosion was found along the length of the rebar, especially at locations where microcracks formed. In contrast, the control R/C specimen showed very high corrosion rate at the single crack site, while the other parts of the rebar remain pristine. This suggests that R/ECC structural elements will have longer service life corresponding to a longer corrosion initiation period, when compared with standard R/C elements in a similarly aggressive environment. Further, Sahmaran et al. (2008) investigated the residual flexural load capacity of R/C and R/ECC beams subjected to accelerated corrosion by an electrochemical method. The result is summarized in Figure 13. They concluded that the corrosion propagation period was significantly prolonged due to the damage tolerant and spall resistance of ECC.

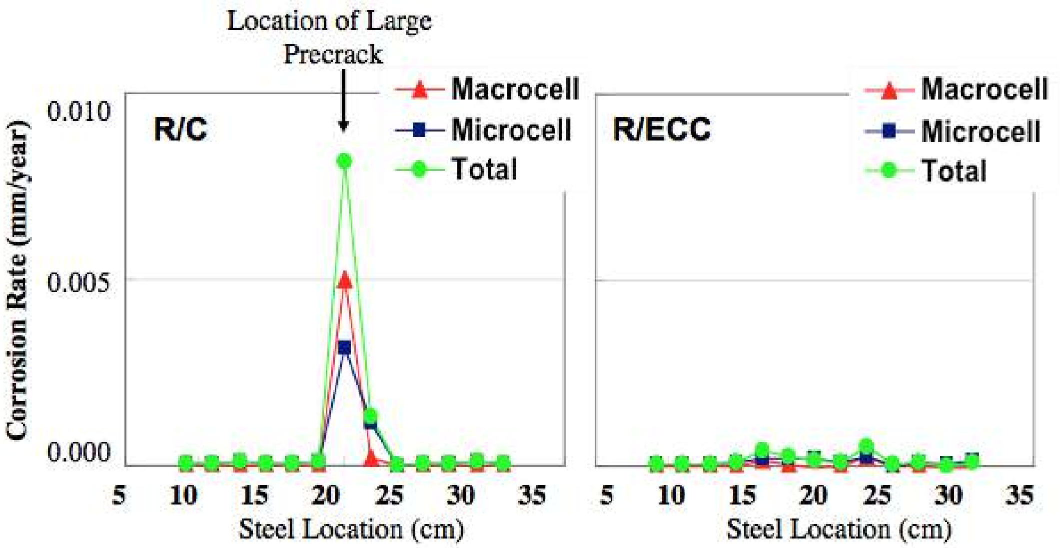


Figure 12 The measured corrosion rate in R/ECC beam subjected to an accelerated chloride environment is substantially lower than that of R/C beam. After Miyazato and Hiraishi (Miyazato and Hiraishi, 2005)

(a) ECC before accelerated corrosion (b) ECC after 150 hours accelerated corrosion

(c) Mortar before accelerated corrosion (d) Mortar after 50 hours accelerated corrosion

Figure 13 The damage tolerant behavior of ECC prolongs the corrosion propagation period and protects the beam element from failure, when subjected to an accelerated corrosion environment. After Sahmaran et al. (2008)

6 FIELD APPLICATIONS OF ECC

ECC has been applied in repair, retrofit, and new constructions. Repair applications include external walls of concrete buildings (Cheung et al., 2004), highway patch repair (Lepech and Li, 2006), railway infrastructure repair (Kunieda and Rokugo, 2006), irrigation canal repair (Uchida et al., 2008), and dam repairs (Uchida et al., 2008). Retrofit applications include bridge deck link-slabs (Lepech and Li, 2009), tunnel lining strengthening (Uchida et al., 2008) and viaduct deck-pier dampers for seismic retrofit. New constructions include exterior building insulation wall (Zhang, 2015), coupling beams in the core of tall buildings (Kanda et al., 2011) and composite bridge decks (Uchida et al., 2008). These applications cover the building and transportation, and water and energy domains. Application methods include cast-in-place, pre-cast, and spray (shotcrete). The Japan Society of Civil Engineers has published a recommendation document (Japan Society of Civil Engineers, 2008) for the design and application of this special class of ductile concrete materials.

One of the applications that best illustrate enhancement of infrastructure sustainability is the bridge deck retrofit in Southeast Michigan in the US (Figure 14), where ECC was applied as a deck link-slab replacing conventional expansion joints (Lepech and Li, 2009). This application was originally motivated by the frequent repair needs of expansion joints, the surrounding deck concrete and the steel or R/C girders supporting the deck below the joints. The combined traffic load and temperature load on the link slab demands a tensile strain capacity of 2%, which cannot be delivered by any concrete material reinforced or not. This is also an application that highlights a major value of ECC – high tensile ductility. High strength concrete material with normal ductility will not be able to meet the design requirement.

Figure 14 Application of ECC on deck retrofit of a highway bridge in Southeast Michigan

Figure 15 Bridge deck showing ECC link-slab (between dashed lines) after bridge re-opened to traffic

Figure 15 shows the bridge deck immediately after installation (2005) of the ECC link slab. Placement of the link slab was cast-in-place using a normal concrete ready-mix truck from a nearby batching plant. The total volume of ECC made for this application was 25.5m^3. While the ECC was designed to be self-consolidation, care was taken to ensure that the required crown of the roadway cross-section was maintained. Despite the severe winters in Michigan, the link-slab remains in working condition today with no maintenance since installation, offering confirmation of the durability of ECC material.

A detailed life-cycle analysis (Keoleian et al., 2005) was carried out for this bridge structure, comparing materials and energy inputs and waste and emissions for the case with conventional expansion joint and for the case with the ECC link-slab. The life-cycle model embodied a traffic model that accounts for changes in traffic due to reconstruction events during the 90 years service life assumed. Figure 16 shows the primary energy consumption and carbon emission results of this analysis. Savings of about 40% is obtained for the case when ECC link-slab is used to replace conventional expansion joint. The most important contributor to these savings derives from the differences in traffic patterns. In the case of the deck with ECC link-slab, the expected durability resulting from the reduced rate of re-bar corrosion and the spall resistance of ECC eliminated repair events and the associated traffic pattern interruptions. The second most important contributing factor to lowering the energy consumption and carbon emission derives from the lowering of the total amount of materials required over the service life as maintenance needs are reduced. This is despite the fact that the ECC used in this project and for the life-cycle analysis did not benefit from the greener versions that were later derived and described in an earlier section of this paper. The life-cycle analysis did not include costs associated with the repair of the supporting girder elements when conventional expansion joints fail. The economic and environmental savings would be even bigger had this been accounted for.

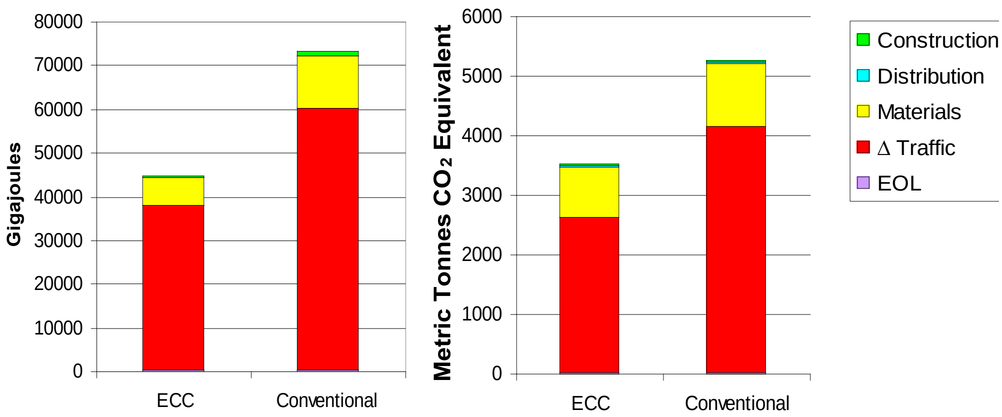
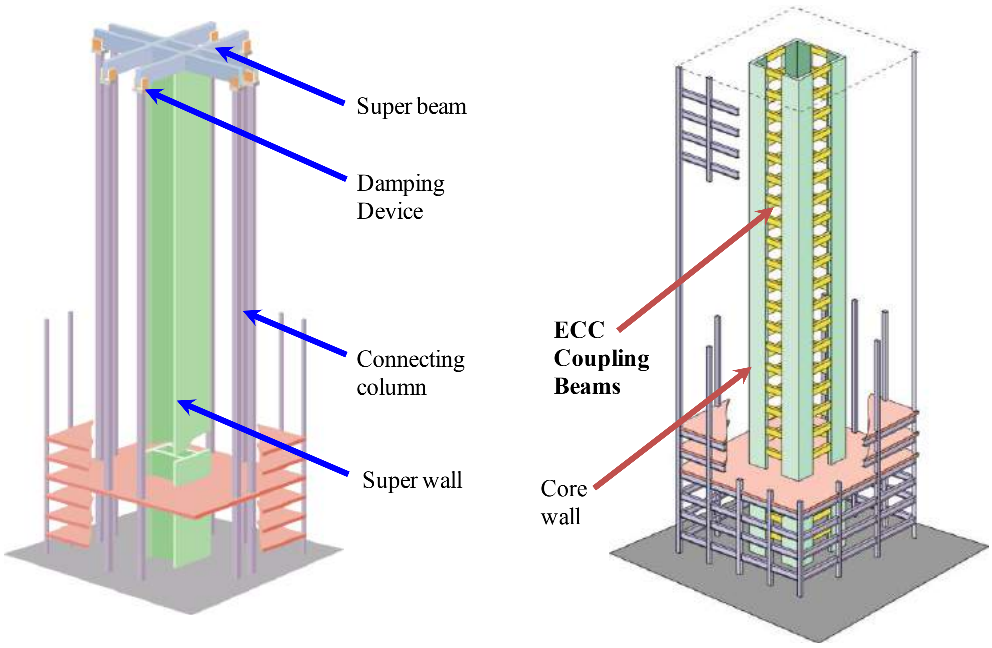


Figure 16 ECC link-slab extends service life and reduces primary energy consumption and carbon emission

One of the best illustration of the use of ECC in enhancing structural resiliency is the new building design created by Kajima Corporation in Japan (Kanda et al., 2011 and Yamamoto, 2008). In this new design, the structural load is mainly born by the core, made up of (two or four) ECC coupling beams on each floor between core-walls. The goal is to maximize floor space by eliminating the beams and columns commonly used in conventional rigid-frame structures. Although a similar goal was previously achieved using a superframe construction, the ECC coupling beam approach was considered easier to construct with dramatic labor saving, and also resulted in cost savings on the building unit level, even from an initial cost point of view.



(a) (b)

Figure 17 (a) Superframe design, and (b) ECC-coupling beam design of tall buildings realized by Kajima Corporation (Kanda et al., 2011)

Figure 17(a) shows the superframe approach which uses a super beam connected to the top of a pair of columns through dampers, to form a frame. Multiple pairs of superframes are deployed to achieve self-centering performance of the building during an earthquake. The super beams are about 3m in height and their weight requires construction of the beams on the building top. Figure 17(b) shows the new building design with ECC coupling beams. So far, three buildings in Japan has been constructed using this approach, including the Glorio-tower Roppongi (Figure 18(a)), the Nabule-Yokohma tower and residence (Figure 18(b)), and the 61 story Kitahama Tower in Osaka (Figure

18(c)), currently the tallest R/C structure in Japan. These buildings may be considered one of the first applications of ECC in enhancing infrastructure resiliency.

The ECC coupling beams were designed to meet three criteria: (a) shear or bond splitting failure must be fully suppressed, (b) load capacity remains stable even when the member drift angle reaches 4%, and (c) residual crack width after unloading must be limited to less than 0.3mm. Structural experiments similar to those described in Figures 8-10 confirmed the desired failure mode, hysteresis behavior and cracking behavior. The structural design strategy ensured member ductility and energy absorption for imposed seismic load profile.

The ECC coupling beams were precast off-site to ensure high quality control of these critical structural elements. They were cast together with part of the floor slab to safeguard a more gradual transition from the high performance ECC coupling beam to the standard R/C floor slab. The precast coupling beams with protruding re-bars were dropped into location at each floor before the core-wall was cast (Figure 19).

Kanda et al. (2011) reported no visible damage to the Glorio-tower and Nabule-Yokohma tower near Tokyo after the large Tohoku earthquake excitation in 2011.

(a) (b) (c)

Figure 18 (a) The Glorio-tower Roppongi, (b) the Nabule-Yokohma tower and residence, and (c) the Kitahama Tower in Osaka are recent tall buildings using ECC coupling beams for structural resiliency (Kanda et al., 2011 and Yamamoto, 2008)

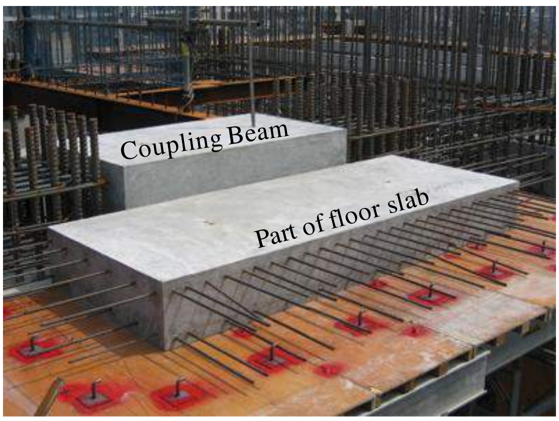


Figure 19 Precast ECC coupling beam dropped into location during building construction (Kanda et al., 2011)

7 CONCLUSION

In keeping with the goals of the global sustainable development efforts, next general civil infrastructure must be designed for sustainability. For truly sustainable infrastructures, they must also be resilient. These targets may initially appear contradictory since sustainability is often associated with less material consumption, while resiliency is often associated with more material for construction. However, it is argued that simultaneous sustainability and resiliency of civil infrastructure is not only necessary, but it is also feasible.

One approach to simultaneously attaining infrastructure sustainability and resiliency challenge is through advanced concrete technology. A re-engineered concrete that is green, ductile, damage tolerant, durable in use, and possesses the ability to self-heal when damaged, can serve this purpose. A growing body of experimental data collected under extreme durability testing conditions and mechanical test conditions suggests that Engineered Cementitious Composites (ECC) may provide a feasible platform to meet the infrastructure sustainability and resiliency challenge.

While continued refinement and improvements of ECC can be expected, this material is already emerging in a variety of full-scale structures, including in the building and transportation infrastructure, and the energy and water infrastructure domains. The field experiences gained in structural design, large scale material mixing, and construction methods, provide a valuable foundation for continued future development and broad deployment of this new technology.

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Keynote Address

Building resilience in the urban environment: the role of the engineer

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Keywords:

ABSTRACT: The paper describes the importance of building resilience into the urban built environment. It describes some of the trends and physical impacts which are influencing policy and the business response, with a particular emphasis on innovation and work undertaken in the United Kingdom. Five megatrends of urbanisation, demography, resource stress, climate change and technology are described. The importance of resilience and future-proofing the urban environment for these trends is discussed. Aspects of how built environment professionals are attempting to do this are explored. They include: the UK Government's resilience programme which includes assessments of key infrastructure assets and services; new standards in city metrics reporting (ISO 37120:2014) whereby a standard set of indicators can be used at a city level to indicate progress in sustainable development and quality of life for its citizens; the development of a standard for measuring and managing infrastructure carbon (UK PAS) by the UK's Green Construction Board; planning, design and new product development for flood resilience as part of the British Research Establishment's Resilient City programme; academic and industry partnerships for the deployment of wireless systems networks in tunnelling projects, and resource efficiency - the circular economy – a nascent concept gaining traction across Europe. It concludes with an assessment for what establishing resilient infrastructure might mean for engineers, highlighting the important role they play as integrators throughout the project lifecycle.

1 INTRODUCTION

The urban environment provides a home for more people than the rural environment with the rate of urbanisation likely to continue. The existing urban environment needs to adapt to cope with increased population, whilst new urban environments are being designed and planned as countries go through economic development. Some urban areas have grown without proper planning and have created conurbations that are considered vulnerable to disease and natural disasters (UNDESA, 2014).

In this paper, the trends that are expected to impact on urban living are summarised. Some of the characteristics of a resilient urban environment to manage these impacts are described. This is followed by a selection of responses from a policy and market perspective, looking at current activities by government, business and academia to make urban environments more sustainable and resilient. It concludes with a personal view of the role of the engineer in the context of the challenge of building resilience.

This papers draws on primarily examples from the United Kingdom looking specifically at the research from Innovate UK and the Building Research Establishment's Resilient Cities, and work I am personally involved in. The examples presented are illustrative of activities that seek to overcome challenges that exist today, specifically with a focus on innovation and future policy recommendations, and are not presented as definitive.

2 WHY DO WE NEED A RESILIENT URBAN BUILT ENVIRONMENT?

To best understand the need for more resilient infrastructure we should first briefly explore the issues facing citizens today and in the future. Famous recent examples of disruption to city living from infrastructure failure are – hurricanes Katrina and Sandy in the USA and Fukushima nuclear incident in Japan. Deconcini and Tompkins (2012) summarised some of the impacts caused by Hurricane Sandy such as estimated insured losses between $10 billion and $25 billion; 135 attributable deaths in the United States, with 60 in New York alone; and the U.S. Department of Energy reported that more than 8.5 million customers in 21 states lost power. Extreme weather events associated with a changing climate are likely to increase, and are commonly cited as a megatrend that will influence future urban living. There are others megatrends the impacts of which the urban environment will need to cope with.

3 THE MEGA-TRENDS IMPACTING FUTURE URBAN ENVIRONMENTS

There is a wide range of literature that looks at global megatrends and numerous commentators describe a "perfect storm" that means our city infrastructure needs to be effective, efficient, reliable, resilient and equitable. The Mowat Centre at the University of Toronto identified nine global megatrends in KPMG's Future State 2030 (KPMG 2013 and Figure 1). All of the megatrends will impact the ability of government and city administrations to provide suitable infrastructure. I have selected five of the nine to illustrate why infrastructure will be required to cope with an increasing population in a world with resource constraints and a changing climate. These megatrends include:

a) Demography – higher life expectancy and falling birth rates are increasing the proportion of elderly people across the world. Some regions are facing the challenge of integrating large youth populations into saturated labour markets, for example 1 million young people in India will enter the labour force every month for the next 20 years. By 2030, the number of people aged 65 and older will double to 1 billion globally (Future State 2030: the global megatrends shaping governments, KPMG, 2013). The implications of demographic changes include; an aging world, healthcare spending increases and public pension systems under pressure.
b) Urbanisation – currently 50% of the world's population reside in cities, this will rise to almost two-thirds by 2030 (United Nations population Division. 2012. "world urbanization prospects – the 2011 revision from KPMG.). 1 billion people currently live in city slums, this figure could double by 2030 (OECD in KPMG, 2013). The implications of urbanization include: large-scale infrastructure needs; resource flows from natural environment to the built environment; and urban poverty pressures including growing populations living in informal settlements.
c) Climate change – increasing greenhouse gas emissions in the atmosphere are causing climate change and creating changes to our natural environment which are placing stress on the resilience of natural and built systems. The consequences of climate change are: increased extreme weather events impacting on ecosystems; the pressure to adapt to "locked-in" effects of climate change; a greater role for urban environments.
d) Resource stress – the combined pressures of population growth, economic growth and climate change will place increased stress on essential natural resources including water, food, arable land and energy. By 2030, a 50% increase in food production will be required to feed the growing, affluent world population. Global food prices are predicted to double in the same

period. The consequences of resource stress include: demand rise for resources and energy; food and agricultural pressures; increased competition for metals and minerals and risk of resource nationalism.

e) Enabling technology – information and communications technology (ICT) has transformed society over the last thirty years; global internet users have increased form 360 million in 2000 to 2.4 billion in 2012. ICT has also enabled the research, development and growth of technology in many areas of the built environment including engineering and transportation. The consequences of enabling technology include: transformation of communication, transportation and construction with new ways of manufacturing; and challenges of coping with cyber-crime.

Figure 1 The nine megatrends that will impact of the resilience of today's and future infrastructure (redrawn from KPMG, 2013)

4 WHAT IS A RESILIENT BUILT ENVIRONMENT?

In general, the term "resilient" refers to the ability of something to continue to be of value into the distant future; that the item does not become obsolete and can cope with shocks and stresses. The concept of building resilience into a system is the process of anticipating the future and developing methods of minimizing the effects of shocks and stresses of future events. The Cabinet Office of the UK Government defined resilience as 'the ability of assets, networks and systems to anticipate, absorb, adapt and / or rapidly recover from a disruptive event'. They cited four types of resilience which are directly related to the five megatrends explored above:

a) Extreme weather – climate resilience
b) Natural and manmade disasters – disaster resilience
c) Terrorism and cyber-crime – security resilience
d) Burgeoning global population and increased urbanism - societal resilience.

In recent years the term resilience has been commonly used in government and also with built environment professionals with resilience being a design objective for buildings and infrastructure providing them the ability to absorb or avoid damage without suffering complete failure. There is some interchange in the use of resilience and future proofing. A recent definition for infrastructure future-proofing is "the process of making provision for future developments, needs or events that impact on particular infrastructure through its current planning, design, construction or asset management processes (McFarlane et al., 2014)".

Below I describe a set of conditions which, if present, would go some way to determining a "future-proofed" resilient built environment.

a) Not promote deterioration – do no harm. It is natural for all materials to deteriorate. Future-proof structures and products should not accelerate the deterioration of existing materials.
b) Stimulate flexibility and adaptability. Future-proof interventions should not just allow flexibility and adaptability, but also stimulate it. Adaptability to the environment, uses, occupant needs, and future technologies is critical to the long service life of a historic building.
c) Extend service life. Future-proof interventions in structures and products should help to make the building usable for the long term future – not shorten the service life.
d) Fortify against extreme weather and shortages of materials and energy. Future-proof interventions should prepare structures and products for the impacts of climate change by reducing energy consumption, reducing consumption of materials through durable material selections, and be able to be fortified against extreme natural events such as hurricanes and tornadoes.
e) Increase durability and redundancy. Future-proof interventions should use equally durable building materials. Materials that deteriorate more quickly than the original materials require further interventions and shorten the service life.
f) Reduce the likelihood of obsolescence. A future-proof structure or product should be able to continue to be used for centuries into the future. Take an active approach: regularly evaluate and review current status in terms of future service capacity. Scan the trends to provide a fresh perspective and determine how your historic building will respond to these trends.
g) Consider long term life-cycle benefits. Embodied energy in existing structures and products should be incorporated in environmental, economic, social, and cultural costs for any project.

The set of conditions cover a variety of the issues professionals in the built environment currently face – planning, design, technology, specific environmental conditions such as changing climate and materials sourcing – and that engineers need to be aware of.

5 HOW IS THE BUILT ENVIRONMENT SECTOR RESPONDING?

To answer this question I will explore, through illustrative examples, the latest developments in the following areas; Government policy; City planning; City metrics reporting; procurement strategies – the circular economy; product manufacturers; social value and industry standards, drawing predominately on evidence from the United Kingdom. This is not a comprehensive review, more an illustration of the latest thinking and responses in certain aspects of the challenges described above.

6 A POLICY RESPONSE TO RESILIENCE

The Cabinet Office of UK Government has prepared Sector Resilience Plans that set out the resilience of each national infrastructure sector to the relevant risks identified in the National Risk Assessment. The National Risk Assessment is the main document Government uses to assess the major threats (malicious terrorist attacks) and hazards (non-malicious risks such as human and animals diseases, industrial accidents and industrial action, natural hazards such as flooding and drought) the UK could face in the next five years. A public summary is available (Cabinet Office, 2012).

The Plans are placed before Ministers to alert them to any perceived vulnerabilities, with a programme of measures to improve resilience where necessary. Within the national infrastructure, there are certain critical elements, the loss or compromise of which would have a major impact on the availability or integrity of essential services leading to severe economic or social consequences or to loss of life in the UK. These critical elements make up the Critical National Infrastructure (CNI). The national infrastructure is categorised into nine sectors: Communications, Emergency Services, Energy, Finance, Food, Government, Health, Transport and Water. The UK's national infrastructure is defined by the Government as: "those facilities, systems, sites and networks necessary for the functioning of the country and the delivery of the essential services upon which daily life in the UK depends". (Cabinet Office, 2014)

7 STANDARDS

7.1 *City Reporting: A New Standard for City Metrics*

In this age of urbanization, city indicators can be used as critical tools for city managers, politicians, researchers, business leaders, planners, designers and other professionals to help ensure policies are put into practice that promote liveable, tolerant, inclusive, sustainable, resilient, economically attractive and prosperous cities globally. With this in mind an International Standard on City Indicators was developed by an international stakeholder community.

Established in 2014 the ISO 37120:2014 covers the following themes: economy; education; energy; environment; recreation; safety; shelter; solid waste; telecommunications and innovation; finance; fire and emergency response; governance; health; transportation; urban planning; wastewater and water and sanitation. It further defines 100 city performance indicators that could or should be measured, and how. Specifically, ISO 37120 defines 46 core and 54 supporting indicators that cities either "shall" (core) or "should" (supporting) track and report. ISO 37120 conformance will require third party verification of data, and the organization is in the process of defining an audit process with pilot cities. ISO 37120 also provides for a set of profile indicators, such as population and GDP, to help cities determine which cities are most relevant for comparisons.

The benefits of standardized indicators:

a) More effective governance and delivery of services
b) International benchmarks and targets
c) Local benchmarking and planning
d) Informed decision making for policy makers and city managers
e) Learning across cities
f) Leverage for funding and recognition in international entities
g) Leverage for funding by cities with senior levels of government
h) Framework for sustainability planning
i) Transparency and open data for investment attractiveness
j) Data is moving fast – big data and the information explosion – ISO can help to give cities a reliable foundation of globally standardized data that will assist cities in building core knowledge for city decision-making, and enable comparative insight and global benchmarking (http://www.iso.org/iso/37120_briefing_note.pdf accessed 1/03/20152)

The World Council on City Data is leading the efforts on global implementation of the standard by various cities. It coordinates efforts on open city data to ensure a consistent and comprehensive platform for standardized urban metrics. The WCCD is a global hub for creative learning partnerships across cities, international organizations, corporate partners, and academia to further innovation, envision alternative futures. (http://www.dataforcities.org/About_us.html accessed 28 March 2015)

7.2 *Industry Standards – A Standard for Infrastructure Carbon Measurement*

There is no recognised 'How to' guidance to measure and manage carbon from concept to on-site built-in economic infrastructure. There is sparse information on: how leading organisations measure and manage their carbon; what best practice looks like; how best to move from developing a number i.e. a carbon emission to creating value; how we compare and measure leading innovation; how do we get consistent measurement methodology through the value chain. This was apparent through the research commissioned by the Infrastructure Working Group of the Green Construction Board and HM Treasury. The resulting HM Treasury publication "Infrastructure Carbon Review" (HM Treasury, 2013), identified the need for a standard on how best to measure and manage carbon in infrastructure.

The principal purpose of the Infrastructure Carbon Publicly Available Standard (PAS) is to define good practice in the measurement and management of carbon in UK economic infrastructure. The Infrastructure Carbon PAS will be targeted at individuals or groups who are or want to be low carbon practitioners in all the sectors of UK economic infrastructure and it will satisfy Action 8 of the ICR.

By providing clear guidance on the measurement and management of carbon, the Infrastructure Carbon PAS will enable consistency across each sector and throughout the value chain. This is intended to join up the value chain and provide orthodoxy for consistent decision making. As well as consistency in reporting data on measured projects so that more informed investment choices and decisions can be made at earlier stages in future projects when more opportunities exist to make real carbon savings.

The PAS will be developed over 2015 and will be available for use in 2016. In developing the Infrastructure Carbon PAS, the following key principles will adhere:

a) It must drive towards reduced carbon and reduced cost
b) It must be relevant to all parts of the value chain as defined in the ICR
c) It must be as clear and simple as possible

The Infrastructure Carbon PAS is expected to meet the requirements of a "code of practice" PAS and it should be sufficiently specific to prevent spurious claims of carbon reductions, but it should not constrain future developments and innovations by being over-specific. It should be able to mitigate the unintended consequences of reducing emissions for one "owner" of carbon and displace their emissions to another asset or sector.

In general, the scope will mirror that of the ICR, therefore it will encompass:

a) UK economic infrastructure (Communications, Energy, Transport, Waste, Water)
b) Whole life carbon (including Capital, Operational and End-user Carbon)
c) Carbon that is under the control of UK economic infrastructure

As a minimum, the Infrastructure Carbon PAS will address the following aspects of the measurement and management of carbon in infrastructure:

a) How to define baselines
b) How to set appropriate targets
c) How to select appropriate tools/carbon models
d) How to establish effective KPIs and reporting
e) How to establish effective governance

In addition, it is anticipated that the Infrastructure Carbon PAS will address:

a) Measuring/managing carbon on both investment programmes and individual projects
b) Using carbon in optioneering and assessing innovations
c) Using carbon estimation techniques that are appropriate to the development of detail throughout the delivery process
d) Defining appropriate boundaries for carbon assessment
e) Defining the inputs and outputs required from carbon measurement tools
f) Encouraging outcome-based specifications

Finally, the PAS is not about: other aspects of the wider sustainability agenda; investing in renewables and decarbonising the grid; carbon associated with power generation; carbon that is not under the control of the infrastructure sector – end user carbon; climate change adaptation. It will also not address: buildings that are not part of economic infrastructure; national policy; carbon that is beyond the control (but within the influence) of economic infrastructure; and carbon at an individual corporate level.

8 PROCUREMENT STRATEGIES AND NEW BUSINESS MODELS

8.1 *The Circular Economy*

In Europe the megatrends of over consumption through population and resource scarcity has led to policy makers and business exploring the concept of the circular economy is gaining much interest in Europe. It promotes an alternative from the current linear business model where a resource is sourced, manufactured/processed, utilised then disposed of, to one that continues to use the material either by re-use, recycling or changing into another state, but not wasting it.

As construction uses around 30% of the world's resources there is a nascent working happening across Europe. In Copenhagen, Denmark projects are looking at construction waste and finding ways to use to use locally as a resource (Michael Johansen (www.cleancluster.dk, in pers comm, March 2015).

In Holland, Delta Development Group's Park 20|20 project, a full service office park in Werkstad A4 with offices, hotels and other facilities is currently deploying the principles of the circular economy. It is also attempting to make the links between property and health and well-being. For example the public realm design to encourage activity, and the choice of in-door finishes to promote health benefits. A driver has been the Municipality's 'quality of life' planning guidelines that incorporate the circular economy (http://www.deltadevelopment.eu/en/sustainability accessed March 2015).

In the UK, the environmental forum comprising of the 33 companies of the UK Contractors Group have drafted a position statement, with the intent to work on the topic (Andrew Kinsey, in pers comm. March 2015). Some construction projects have been designed to incorporate the circular economy and organisations like the Ellen MacArthur Foundation are bringing businesses together to learn from each other.

8.2 *Case Study of Gypsum*

A current European collaborative project between the recycling industry, the demolition sector and the gypsum industry (LIFE ENV/BE/001039) is assessing how to improve gypsum resource efficiency.

Gypsum products such as plasterboards and blocks can be counted among the very few construction materials where "closed-loop" recycling is possible. The recycling process separates gypsum from paper. Both materials return in their original products. The overall aim of the collaboration is to transform the gypsum demolition waste market to achieve higher recycling rates of gypsum waste, thereby helping to achieve a resource efficient economy. A video of closed loop illustrates this (https://www.youtube.com/watch?v=FyK4f8eTB1A&feature=youtube). Similarly, the construction product manufacturer, Saint Gobain has a similar project for glass, where glass is carefully removed during the demolition phase (Bekir Andrews, pers. comm. April 2015).

9 PLANNING AND DESIGN: FLOOD RESILIENCE

The Environment Agency suggests that over 5.2 million homes in England are at risk of flooding from rivers, sea or surface water. This equates to 1 in 6 homes. Annual costs of flood damage are currently at least £1.1 billion (due to all sources) and are expected to rise in coming years as the risk of flooding increases due to climate change. Currently 490,000 properties have a 1 in 75 or greater chance in any given year of flooding (from coastal waters or rivers) but by 2035 this will have increased by more than 350,000. In Scotland, there are currently estimated to be 2.5 million properties at risk. The University of Manchester and the University of Manchester Metropolitan with the Building Research Establishment with funding from the European Union have published a guide on flood resilience for local authorities and professionals (White et al., 2013).

9.1 *New Technologies and Products for Flood Defence*

Across Europe, flooding has been traditionally managed by large scale, engineering solutions whereby entire towns and communities are protected by hard flood defences. Although such defences will still have a role to play in creating resilient places, a rise in 'intra-urban' and surface water flooding which often occurs out-with the parameters of structural flood defences, requires a more effective and adaptive flood management approach. As such, attention has turned to integrating resilience into the built environment. Implementing flood resilience approaches could help the management of flood water and speed up the recovery of people and places. Flood resilience technologies are a cost effective method of providing flood protection in the UK and there are a lot of properties, both homes and businesses, which are at risk of flooding and would benefit from them. Many available products are low maintenance and are relatively cheap and simple installations. There are also many products which could equally be used to protect critical infrastructure elements from flooding.

For properties the main sources of water ingress from flooding have been researched and new products to help are coming to the market. For example, air bricks that are self-closing when or can take a de-mountable cover (Figure 1). Water enters through doors so new door guards are being developed (Figure 2). Costs are currently £1000 for door and guards, £200 for air bricks. Their benefit is that they stop or reduce internal flood damage to property and to the contents, depending on performance. But if the water overtops the treatment then damage can still occur to the same extent, so a good risk assessment is required.

Both temporary and permanent perimeter flood defence products have been developed (Figure 3). They offer varying degrees of performance, for example temporary blow up plastic defences may leak, whereas demountable schemes need a team that knows how to assemble and disassemble. Some barriers can be pre-installed with mechanisms that allow them allow them to close (Figures 4 & 5). Costs are variable dependant on size and type with demountable and temporary measures cheaper than pre-installed. The benefits are that vital infrastructure and communication network protected and able to continue to perform although maintenance access might be disrupted. If the flooding tops the barriers some damage will incur and another limitation maybe the potential to displace the flood elsewhere.

Figure 1 Coves for air bricks (left);
self-closing air bricks installation and diagram of component parts (middle and right)

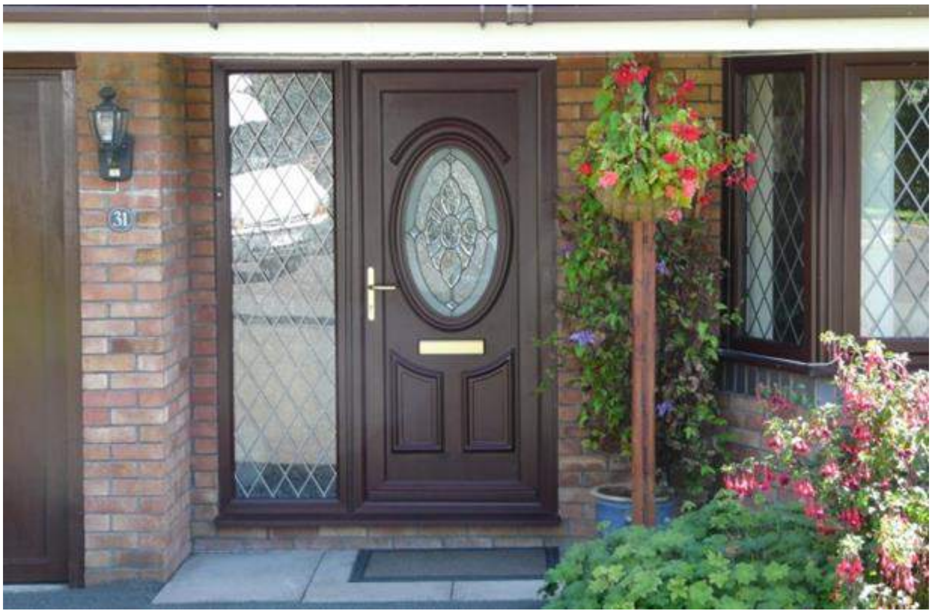

Figure 2 Demountable door guards (top); built-in water defence (courtesy of Whitehouse) and flood-proof doors in Germany but perforated bricks nearby reduce optimum performance (bottom)

Figure 3 Temporary and demountable perimeter flood barriers

Figure 4 Spring Dam prior to and after deployment: courtesy of TiltDam

Figure 5 Flood defence barrier

Design tools are being commercialised, one specific research project with EU and industry partner Toumazis allows consultants to design flood barriers specific to particular project requirements and derive the dimensions of the structural components. The tool allows the user to select materials for the pavement and flooded surfaces; investigate loading on flooded face, mounting mechanisms and costs.

With more than 10,000 properties each year built in flood risk areas in Europe new building products for flood defence properties will be required. Innovation hubs in universities and research are developing new products. By way of example, two new products from the Building Research Establishment's hub Aquobex are highlighted here (http://www.aquobex.com/).

Wall waterproofing is a common approach for flood risk, with such coatings as nanoShell Stone. Based on UV and high pH-resistant silicon nanoparticles, the colourless, water-repellent coating of nanoShell Stone can be applied to masonry, natural and composite stone. Lasting up to 25 years before re-application, the flood control coating maintains the original colour and texture of stone while preventing water and frost damage, mould, salts and more from attacking a property. Ensuring minimal water damage in the event of a flood, the coating also increases the insulation capabilities of a property and eliminates the need for cleaning as rainwater cleanses the surface.

Technitherm® is a cavity wall insulation that helps property flood resilience. Flood waters can penetrate unprotected cavity walls and cause major damage to the inner leaf, cavity wall ties, standard (fibre, polystyrene bead), cavity wall insulation materials, plaster, electrical appliances, furnishings and decorations. The cost of removal and/or drying out of saturated standard insulation materials can be expensive and disruptive for many months once flood waters recede. This flood resilient cavity wall insulation is a closed cell, seamless rigid polyurethane system. The solution will resist the passage of flood water ingress through the cavity wall, and the insulation will remain serviceable after the flood water is gone. The cavity insulation helps stabilise and strengthen property walls in preparation for flood risk, enabling them to withstand greater pressures from exerting flood water and debris. In a retrofit situation it provides a cost-effective alternative to metal cavity tie replacement and also delivers thermal insulation.

9.2 *New Property Flood Resilience Database for Insurers*

In recent years there has been a change in approach in flood management, from large-scale, engineered flood defences to one of flood resilience. Flood resilience can occur at a local scale (building to building) and requires householders to take responsibility for protecting their own properties. There has been great investment in flood resilience, particularly through property level protection (PLP). However, these improvements are not reflected in home insurance cover and premiums, as insurers currently have no data to refer to in order to assess reduced risk.

This is perceived as a barrier to the uptake of flood resilience measures. There is an increase in the number of properties at risk of flooding, due to increased urbanisation and climate change which results in more intense rainfall and more surface water flooding. There are, therefore, a growing number of householders who could benefit from flood resilience measures. To address this in the UK, the Building Research Establishment (BRE) has been working with the insurance industry, notably AXA Insurance and Lexis-Nexis, on the difficulty of insurers accessing information on improvements to buildings to manage flood risk.

The Property Flood Resilience Database (PFR-d) project is an Innovate UK funded feasibility study for the development of a database which allows resilience improvements made to a property to be lodged and 'scored' for use by the insurance industry. This project has developed a PFR-d which combines environmental datasets on flood risk with resilience measures installed at property level. The PFR-d addresses a gap in the current data relating flood risk and resilience measures applied to buildings. This will assist in providing more appropriate insurance pricing in high flood risk areas, or for properties which have suffered repeat flooding events. The existing datasets used by the insurance sector are flood risk data in the form of maps and exposure zones used to assess potential flood risk (depth and return period) in the future. The PFR-d allows insurers, local authorities and Government agencies to take into account the investment made by the insured parties and others to protect properties through implementing flood resilience.

An application for surveyors has also been developed, which can be used on-site, on a tablet or smartphone, to help standardise the data that is collected on property flood resilience surveys. This data is added to the PFR-d, once it is confirmed that work is complete. A PFR-d score is then calculated, and integrated into insurers underwriting systems to allow them to assess the residual risk.

The next steps are to commercialise the prototype database. This will include case study trials to further develop the surveyor’s application and to test the PFR-d. The PFR-d is owned through a collaboration agreement by BRE, LexisNexis and AXA.

10 INFORMATION TECHNOLOGY AND COMMUNICATION: CASE STUDY OF WIRELESS SYSTEM MONITORING

In the UK, the University of Cambridge the Centre for Smart Infrastructure and Construction's (CSIC) research focuses on the innovative use of emerging technologies in sensor and data management (e.g. fibre optics, MEMS, computer vision, power harvesting, Radio Frequency Identification (RFID) and Wireless Sensor Networks). These are coupled with emerging best practice in the form of the latest manufacturing and supply chain management approaches applied to construction and infrastructure (e.g. smart building components for life-cycle adaptive design, innovative manufacturing process, integrated supply chain management, and smart management processes from building to city scales). (http://www-smartinfrastructure.eng.cam.ac.uk/)

Many of their innovations are coming to the market. For example, with help from CSIC and Costain, Crossrail in London – the largest infrastructure project in Europe - has deployed an intelligent system using miniature wireless sensors, in a sprayed concrete lining (SCL) tunnel construction, at a partially sealed adit complex at its Eleanor Street site. The objective was to monitor the movements of the adit in real-time while new tunnel and shaft excavations were conducted. The system consists of self-powered wireless `UtterBerry' sensors, developed and patented by a PhD student at CSIC. UtterBerry sensors have already won industry awards and nominations.

The installation of the 29 sensors, each weighing 15g, required one person, who placed the sensors into position using just a 5m pole. The sensors were placed to measure temperature and humidity, as well as tunnel wall inclination and displacement in three axes. This rich dataset ensures all significant events are recorded and communicated in real-time, allowing engineers to quickly make better-informed and cost-effective decisions and reach accurate conclusions regarding structural movements.

The sensors have been in operation since April 2014, remotely monitoring the condition and structural health of the tunnel from their offices in a safe and effective manner.

The focus of new sensor systems for civil engineering application therefore should be to develop an integrated framework for planning, deployment and management of the system so that users can trust the data coming from it. One of the main missions of CSIC is to demonstrate the performance of various innovative sensor systems on real field sites.

In addition, CSIC technology CSattAR Photogrammetric Monitoring, a new digital image correlation (DIC) technology has been successfully deployed at a number of tunnels in the UK and abroad to monitor their structural behaviour - both long-term deformation and/or movement caused by nearby construction activity. The technology captures extremely precise movements at a wide range of points and operates at a fraction of the cost of conventional alternatives.

11 THE ROLE OF THE ENGINEER: THE NEW AGE OF ENGINEERING

Engineers develop and improve the services and facilities that keep society functioning. Citizens everywhere rely on engineering every day for a variety of things: from supplying energy and clean water to their homes, to processing and recycling their waste, to supplying transport and facilities for healthcare and education, and finding solutions to problems such as improving air quality.

The challenges facing society – in part created by the megatrends described at the beginning of the paper are not going to disappear. The solutions involve engineers; this makes the role of the engineer significant and challenging. The engineering professions will need to maintain their specialisms but also widen their knowledge base, as they will need to act as integrators interpreting the policy requirements, keep abreast of technology advances and innovations, and demonstrate the need to future-proof and de-risk both retro-fitting and new infrastructure developments. One of the recommendations in the State of the Nation (ICE, 2014) suggests dedicated multi-disciplinary engineering teams should be seconded directly into the latter stages of significant research projects with the task of implementing the benefits from academic research, so that they can be practically and efficiently applied to meet the UK's infrastructure needs. This I would support. But perhaps go further and challenge the training of today's new engineers and the professional development of the existing engineers facing these challenges. My point here is how best to up-skill and widen the engineers skill set to they can successfully take on an integrator role. There needs to be more knowledge sharing amongst between engineers and other professions to help facilitate this up-skilling. This has been recognised before and others have made similar requests; see for example Royal Academy of Engineering (2010) publication Engineering a low carbon built environment.

12 CONCLUSION

My conclusions based on this snapshot across the resilience landscape are about the inevitability of all these areas of work coming together, integrated to deliver quality of life outcomes for end users.

The application of emerging technologies such as new sensor technologies to advanced health monitoring of existing critical infrastructure assets will quantify and define the extent of ageing and the consequent remaining design life of infrastructure, thereby reducing the risk of failure. For new infrastructure, emerging technologies will also transform the industry through a whole-life approach to achieving sustainability in construction and infrastructure in an integrated way – design and commissioning, the construction process, exploitation and use, and eventual de-commissioning. The development of 'smart' infrastructure is essential to the viability of rehabilitation, repair and reuse.

This whole approach will feature resource efficiency, closed looped circular economy and influence procurement strategies. The outcomes of these activities will be measured and reported using standardised approaches, such as for carbon, and they will feed into a greater understanding of how they deliver improvements in city living. Engineers will need to adapt to the changing marketplace, as their role as an integrator, understanding how the infrastructure system work, will become increasingly more important.

13 ACKNOWLEDGEMENTS

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Special Lecture

The ties that bind – building a community with innovation in infrastructure development

C.S. Wai

Keywords:

ABSTRACT: Civil engineers build roads, bridges, railways and ports. The development of transport infrastructure in the past decades has brought people and places closer, and promoted global connections and interaction. Recent technological advancement has given us even more power to connect electronically with each other. The proliferation of mobile communications and the widespread use of social media, however, seem to have weakened inter-personal relationships. It seems that intimacy has been sacrificed in the pursuit of proximity.

Civil engineers are in a position to take the lead in re-establishing closer personal ties. In Hong Kong, this may be achieved in the planning, design and construction of community features in new development areas. Some examples are the use of public spaces and facilities, multi-disciplinary design solutions and adoption of sustainability development features. There is also scope for innovation in planning engineering infrastructure to achieve environmental protection, durability and people-oriented designs.

The planning of infrastructure involves striving for balance among the requirements of continuous development, environmental concerns and finite resources. Traditional planning tools may need to be updated. It is noted that research is being carried out to develop town or transport planning models that make use of “big data”. As “big data” can be harnessed from communication or social media usage, it is possible that more coherent future communities will be planned on the back of the same technologies that seem to have weakened social ties.

1 INTRODUCTION

Civil engineers are trained to use available resources to solve problems. When we see problems that have never been encountered before, or are more complicated than ever, we are the professionals who will use innovative ideas to tackle them.

Recently civil engineers in the Water Supplies Department of Hong Kong have developed an artificial intelligence system to optimize daily pumping programmes in the network using a genetic algorithm. The algorithm is a search heuristic that mimics the natural selection process, where candidate solutions to an optimisation problem evolve towards better solutions using techniques inspired by natural evolution such as inheritance, mutation, selection or crossover.

Did you find my introduction of this innovative technique a little hard to understand? Can you imagine how members of the general public think if they are given a description like that when we tell them about the contribution of civil engineers in Hong Kong society?

Of course the work of other professionals is often just as complicated:

In the legal field, a court ruling on a construction contract dispute about an "entire agreement" clause states that "the doctrine of freedom of contract allows parties to agree that a representation-free state of affairs exists even if prior representations have been made that might ground an estoppel".

Hard to understand isn't it?

In the medical profession, research into the common construction site injury of musculoskeletal disorder suggests that coherence of statistical and physiological information on injury cases could make intervention of risk factors more effective, even though it might be possible to do it by epidemiological approaches without knowing the underlying patho-physiology.

Again, not easy to follow…

Lawyers and doctors deal with issues as complex and challenging as the ones facing civil engineers. However, those professions and the practitioners often appear more approachable to the general public; whereas civil engineers seem to them to operate in a restricted zone where the public is not welcome. Is it true that civil engineers are not good communicators?

Maybe that perception is a result of our intensive technical training, or maybe our core duties do not require us to interact with people outside the profession. Doctors and lawyers have to explain their professional views to others in their day-to-day work. They have many opportunities to communicate with the public, even on TV, radio or news media. No wonder these other professionals seem to have higher profile or higher standing than us humble civil engineers in the community.

Therefore, I would like to talk about how we professionals can contribute to the society from the people-oriented aspect.

2 BROKEN TIES

Civil engineers may not be great communicators, we are however responsible for bringing people together in the recent past. It is almost a cliché to use the term "global village" to describe how closely linked we are. It is however easy to forget that not so long ago the world was a much bigger place. Transport infrastructure developed by civil engineers, such as roads, tunnels, bridges, railways, ports and airports, was primarily responsible for shrinking the big world into a smaller place, linking people, connecting lives.

These days there is more advanced communication technology that enables us to connect with all corners of the world instantaneously. However, in the pursuit of virtual proximity, we may have inadvertently sacrificed real intimacy. I have noticed recently that the advance in communication, that is supposed to keep us in touch with each other, has ended up keeping us further apart.

Why do I say that? Now that communication technology is so advanced, we seem to be constantly engrossed in the virtual internet world. People no longer engage themselves in reasoned debates. Many views or arguments are presented in online forums; protests actions are mobilized through social networks. When people play games, they do not meet on playing fields, they take on disguises in an online world with imaginary landscape and virtual currency.

Technology keeps people physically further apart than before too. Hong Kong transport officials reported last year that our metro system had a reduced service capacity as compared with that in the 80's or 90's. One of the reasons is that passengers use hand-held devices on board the trains and typically take up more personal space.

Looking at this state of things, it seems that although the world has become smaller, the ties among people and communities have been weakened by the advance in technology.

Luckily, civil engineers are trained to solve problems. I believe that civil engineers now have an additional challenge after having shaped the world – it is to come up with innovative ideas to build a community and bring people together again.

3 CONNECTING PERSONALLY

First of all, a strong community needs the infrastructure to bring people together.

It is common knowledge that Hong Kong is among the most densely populated cities in the world. In many other places, small personal space and high-rise living are catalysts for social discontent. However, in Hong Kong we manage to embrace high density living from which we derive social cohesion, harmony, order and safety. In this environment, we have developed a high quality urban living standard, together with an efficient transport system, accessible public facilities, and opportunities for close inter-personal relationships.

Most people still think fondly of the so-called "Lion Rock Spirit" as the exemplary model of Hong Kong community. This model sees people supporting each other whether it is in domestic chores or economic activities. The concerned communities lived in very close-knit living environment in public housing estates and shared long corridors or even communal kitchen or toilet facilities.

This ideal harks back to an earlier time, when personal income was low and technology primitive. In the twenty-first century, with the highly developed technology and infrastructure, can we revive a "Lion Rock Spirit" to promote a simpler and more organic lifestyle?

I think the answer is "Yes". The renowned architect Richard Rogers said that "Architecture is public space held by buildings". In our future cities, architects, planners, engineers, and other professionals need to consider the role of "public facilities", both in the physical and in the technological sense, to promote community life. In Hong Kong we are looking keenly into the Wise City planning concept – in our next generation of new development areas, emphasis will be put on walkability, human-scale urban design, city platform and the like to promote community interaction.

The civil engineer's part in this is not only in the planning and design of the infrastructure. We will do well to adopt a communicator's role so that we are better equipped to engage the public and provide for social interaction in the development area.

4 CONNECTING PROFESSIONALLY

The second issue to tackle is to recognise that the work of building a community requires expertise in many disciplines.

The professions of architect or town-planner as we know them today are in fact quite recent developments. Up to at least the seventeenth century, European buildings or urban development were studied and pursued by educated gentlemen well versed in science and mathematics. They involved themselves in improving the living environment mainly as a duty to the nation and the public.

Similarly, the profession of civil engineer was also once a more general field. It started around a thousand years ago against a military background. All engineering work, the building of roads, bridges or mechanical devices, was to facilitate military exercise. As the engineering expertise spilt over to serve civilian purposes, the discipline of "civil engineering" was born. So civil engineering started as an all-encompassing term for the non-military application of science and technology. The idea that engineering is "civil", reflects its primary interest in serving the public and community life.

The future city, or wise city, will require close collaboration of many disciplines. There is a need to connect those concerned professions. In essence, professionals of today may have to resume and re-gain the vision of past practitioners, to see their work as part of a wider whole, to understand and respect the ideas of other professionals and to target their effort at serving the community.

The civil engineer's part in this is not only in contributing engineering solution to the project team. We will do well to adopt a communicator's role so that we are better equipped to collaborate with other professionals to achieve a common vision and shared goal in the design and planning for the community.

5 CONNECTING GLOBALLY

The third thing that should be borne in mind when building for a community is to give it a farsighted and international context.

"No man is an island" and neither is a community. The building of communities and fostering of community spirit should not be limited to isolated or insular development nodes. Modern transport and communication infrastructures should be in place to facilitate necessary commuting or travel, also trade and commerce.

In this century we are faced with onerous global issues. Dwindling natural resources or the threats of climate change and global warming – these are issues that do not just affect regional welfare and stability, but the future of our living environment and the world itself. In the planning of new development areas, there should be a conscious adoption of responsible practice and sustainable materials.

The civil engineer's part in this is not only in paying attention to environmental protection. We will do well to adopt a communicator's role so that we are better equipped to understand global issues and keep up with international concerns.

6 SOME POINTERS FOR INNOVATION

I have earlier this month retired from an engineering career in the Hong Kong Civil Service. I still remember that, as young civil engineers, my colleagues and I dreamed of having the chance to be involved in mega-sized engineering projects. We wanted to build higher structures, longer bridges and deeper tunnels. No matter where our career paths have taken us, I think we all share that aspiration at some point in our life as civil engineers.

Having identified the role of civil engineers in building communities, I would like to propose a few pointers to guide us in our quest for innovative planning solutions. I will just summarise them in three aspects: High, Long and Deep.

a) High – We excel in Hong Kong in building tall iconic buildings. This is out of necessity because of our land shortage. There are therefore ample opportunities to explore "vertical mix-used building design concepts" which can bring about work-life balance and reduce the need for commuting. Many high-rise buildings already mix residential use with shopping centres, or offices with shops. We also have buildings, like the International Commerce Centre, that mix hotels, offices, shops and transport interchange in the same structure. A more radical example may be The Shard in London, an 87-storey building completed two years ago. It is described as a "vertical city" because in addition to the rather standard mixture of offices, shops, hotel or restaurant floors, it also has a number of residential floors, a business school and a metro station directly underneath. All interconnected by forty-four lifts. This may be a viable model for new communities in Hong Kong.

   Other than aiming high in our structures, we should also aim high in our environmental protection ideals. Many features can be built into our design to achieve low carbon emission, high energy efficiency, high degree of renewable power generation, or water recycling and harvesting. I hope that the ZCB in Hong Kong can be an icon and an inspiration for this high-sustainability ideal; and the BEAM-plus system a yardstick by which we measure our building design. This is without doubt a high expectation because Hong Kong development has been constrained physically by land shortage and ideologically by a profit-oriented design mindset.

b) Long – In Hong Kong we develop roads and railways at an astonishing speed. We also boast some of the longest bridge spans in the world. The Tsing Ma Bridge and Stonecutters Bridge have become international bywords for quality bridge design and construction.

   In further developing our transport system, we should not only aim at reaching long distance, but also long time into the future. Our long span bridges are not only designed and constructed well, they are under constant structural health monitoring; and are maintained to world-class standard. We should aim at adopting materials and designs that have durability and ease of maintenance in mind. The design should be future-proof, so that it includes provisions to cope with changes in climate or improved serviceability. Asset management systems may be applied to better monitor public facilities and the investment of resources in their upkeep and renewal.

c) Deep – Over the years we have made good use of tunnels in our transport network to overcome Hong Kong's constraints of hilly terrain and separation by the harbour. Our underground drainage systems, both in local or terrestrial scales, serve to protect us from wet seasonal monsoons or rainstorms. In enhancing land supply, we will be looking to make use of rock caverns and underground spaces. We have around five hundred kilometers of various types of tunnels in Hong Kong. The tunnel density, that is the ratio of total tunnel length to total area of the territory, should certainly be one of the highest when compared to other modern cities in the world.

   While we continue to look deep into the underground stratum for innovative engineering solutions, we should also make sure that we have the wellbeing of the community deep in our hearts. After all, the word "civil' in our profession is a constant reminder that we work for the people. As such, our ideas, our concerns and our targets should be for the good of the community and humanity.

7 TYING UP LOOSE ENDS

Having spoken about making connections, building communities and being innovative, are there some ideas that can demonstrate where the next phase of engineering innovation will be?

In my post in the Development Bureau, I was not only involved in aspects of the construction industry in Hong Kong, but also the direction in development of land and new towns. In that regard, I have thought much about the future of the city and how it can be better planned.

Planning in Hong Kong is confined and defined by many and various constraints. We are often faced with conflicting demands of development, environmental protection and finite resources. People say that "No one can serve two masters". Yet we are faced with at least these three harsh masters, plus any number of project specific issues and uncertainties. Civil engineers are often tasked with finding the balance among all these problems, or blamed when the balance tips. One area that seems sorely lacking in planning is a structured and scientific method to optimise land use presumptions and infrastructure needs. Such a system will be a valuable tool in the course of planning our new town projects.

Previously there was an attempt to use a Land Use-Transport Optimisation Model in Hong Kong to reconcile the inter-dependence of land and infrastructural development. This has not been brought up-to-date. With vastly increased computing power and data storage capacity, can we come up with a twenty-first century version of this innovative endeavour?

In transport planning, the classic four-stage model which is still in use is based on ideas and technology developed some 30-odd years ago. Is there an update of our transport planning modelling system to better reflect the trip demands, mode choices and behaviour of the modern-day traveller? In particular, advancement in communication technology and advent of home offices will have blurred the boundaries between commercial and residential areas leading to an entirely different commuting pattern.

I know that there are already studies being carried out to make use of big data or crowdsourcing for urban planning, transport modelling or land-use/transport interaction projection. The interesting aspect of using big data is that the data generating infrastructure is already established, such as the mobile phone network, online search engines, internet chatrooms, e-commerce and so on. The innovation comes from being able to harness the data and to come up with meaningful, site and user specific uses of the already available data.

I find it ironic that modern technology brought the world together to become a global village. Then the widespread use of the technology, like the mobile phone, the internet and so on, in turn causes us to abandon organic, physical and interpersonal interaction and threatens the spirit and integrity of community life. The next development, the raw materials that we may use to plan for a more coherent and people-oriented community, may however come from mobile phone or internet usages. The data generated from them may lead to more innovative planning and engineering solutions that will draw us back together, to complete a full circle, to bind us in our new future community.

8 CONCLUSION

In 2013, the Institution of Civil Engineers launched the "Shaping the World" initiative with a view to preparing the profession for the modern day challenges. Civil engineers have always taken the lead to shape the world. I mentioned in the beginning our effort in bringing people together through transport infrastructure development. I suggested that we should now take up the challenge of building future communities.

We should also continuously shape ourselves to enable us to address the demands of today, and to come up with the solutions of tomorrow. We have to be multi-skilled, and we have to develop communication skill and a global vision. It will be through shaping ourselves that we get to shape the world.

I look forward to many fruitful exchanges of innovative ideas in the Annual Conference this year. Thank you.

Special Lecture

New town development in Hong Kong: the road to smart growth

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Keywords: smart growth; create growth capacity; liveability; economic competitiveness.

ABSTRACT

> *"The story of Hong Kong has been one of enterprise and growth; of obstacles overcome and achievements forged from adversity ... But no chapter in Hong Kong's story has been more stirring, and no obstacle attacked with more determination than the continuing problem of providing homes for the millions ...."*
>
> "Hong Kong's New Town",
> the then Public Works Department, Hong Kong

Providing adequate homes has been an ongoing challenge for Hong Kong ever since the post-war period. While new towns have long been adopted by the Government to address the housing issue, the mode of new town development itself has been evolving over time. The prototype of Kwun Tong Satellite Town in the 1950s emphasised the provision of the much needed factory land and housing land at the urban periphery. In the early 1970s, building upon the emerging planning direction for decentralisation and driven by the Ten-Year Housing Programme, the Government embarked on new town development to provide a self-contained and balanced community. However, it is increasingly recognised that self-containment, especially in terms of local job provision, is difficult to achieve as people are mobile, the economy keeps restructuring, and business locational choices are broadening. Nowadays, with limited developable land but intense land demand to serve the population and different uses plus an aspiration for better quality of life, planners need to find a path to smart growth. New towns will continue to be the basic form of urban growth in Hong Kong. Yet, with hindsight of past new town planning experience, a new development model for the future is in the making.

1 THE STORY OF NEW TOWN DEVELOPMENT IN HONG KONG

"New town" in Hong Kong is a response to a unique set of planning and political circumstances such as demographic changes, economic restructuring, social aspiration, and government housing policies. While new town development is also prevalent in other parts of the world, Hong Kong's new town story has a unique flavor reflecting its unique context.

1.1 *Prototype Satellite Town in Kwun Tong*

Set in the context of a booming population in the post-war period, slums and squatters proliferated in the built up areas. The Government addressed the squatter problems by clearing and resettling them in the urban fringes. At the same time, the economy was shifting from entrepot trade to manufacturing. Kwun Tong, which was originally a reclamation scheme at the urban periphery, became a convenient location for siting Hong Kong's first factory satellite town that depended a lot on the linkage to Kowloon. Triggered by the disastrous Shek Kip Mei Fire in 1953, the Government launched a public housing programme. The emphasis then was on the expeditious provision of low-cost shelters. However, the prototype in Kwun Tong was often considered to be inadequately planned, especially in community facility provision. With hindsight, Kwun Tong is more of a "satellite town" than a "new town" as it lacks comprehensive planning as compared to subsequent nine new towns described below (Figure 1).

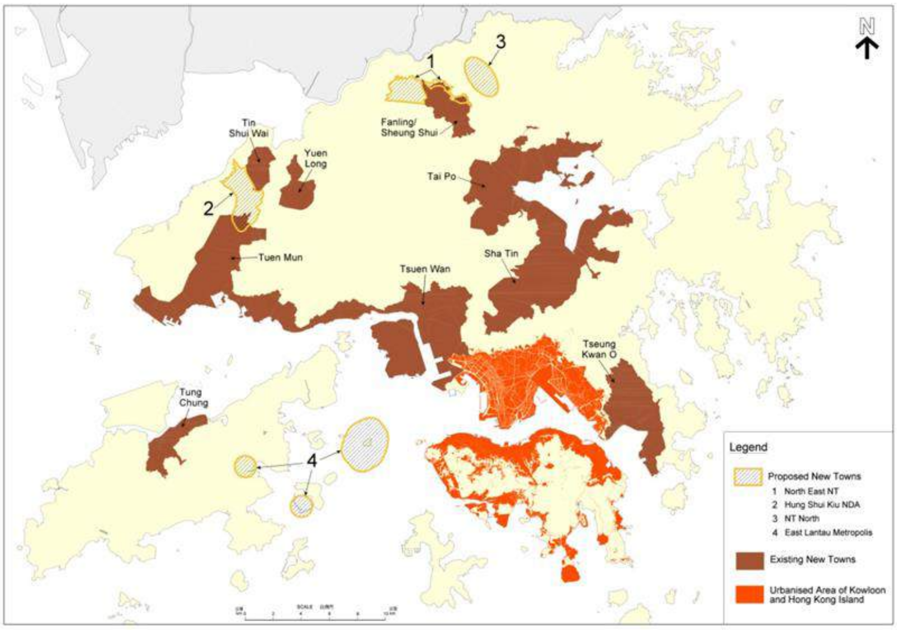

Figure 1 The existing and proposed new towns in Hong Kong

1.2 *First Generation New Towns*

Tsuen Wan, Sha Tin and Tuen Mun developments were initiated in the 1960s but with slow progress. At the strategic planning level, the Colony Outline Plan of 1972 set out a new town decentralisation strategy and environmental improvement initiatives for the existing urban areas. It advocated a well-thought approach to land development by delineating specific areas for residential, industrial and other uses at a district level. It also provided the standards and locational factors for various land uses, which represented the first set of planning standards and guidelines in Hong Kong. It called for a greater degree of vertical integration of urban functions centering around mass transit, laying the foundation of transit-oriented, compact and comprehensively planned new town development in Hong Kong.

The New Town Development Programme in 1973, driven by the Ten-Year Housing Programme, gave a major impetus to new town development in Hong Kong to accommodate about 1.8 million people outside the overcrowded urban area. The "first generation new towns" comprise Tsuen Wan, Sha Tin and Tuen Mun which have since then been developed at a much expanded scale. Below is an update of these new towns:

a) Tsuen Wan New Town: it covers Tsuen Wan, Kwai Chung and Tsing Yi, with a total development area of about 3,285ha for a planned population of 845,000. The current population is about 796,000. It has been developed partly on reclaimed land as an industrial extension of the urban area. Its industrial character still prevails today with the presence of container terminals and industrial areas such as Chai Wan Kok Industrial Area and Texaco Industrial Area. It is well-served by the MTR Tsuen Wan Line Extension and West Rail Line as well as the strategic road networks. Tsing Yi is accessible by the Tung Chung Line and Airport Express Line.

b) Sha Tin New Town: it includes Sha Tin and Ma On Shan (MOS), with a total development area of about 3,591ha for a planned population of 735,000. The current population is about 658,000. It has been built on land mainly reclaimed from the Shing Mun River valley. It includes Shek Mun, Siu Lek Yuen and Fo Tan Industrial Areas. It is served by the MTR East Rail Line and MOS Line as well as the strategic road networks.
c) Tuen Mun New Town: it covers a total development area of about 3,259ha for a planned population of 649,000, while the current population is about 496,000. It has been developed mainly on land reclaimed from Castle Peak Bay. The River Trade Terminal and Tuen Mun Industrial Area are located nearby. It is served by roads, the MTR West Rail Line and the Light Rail system.

Commonly underpinning the planning of the first generation new towns are the comprehensive large-scale development and concepts of "self-containment" and "balanced development". Hence, they are often provided with supporting infrastructure and community facilities serving residents' daily needs. Moreover, industrial areas for flatted factory buildings are typically found to provide local employment opportunities.

1.3 *Second Generation New Towns*

In the late 1970s, the Government was expanding the traditional market towns in the New Territories to new towns, now commonly classified as the "second generation new towns":

a) Tai Po New Town: it covers a development area of about 2,898ha for a planned population of 347,000. The current population is about 270,000. It is served by the MTR East Rail Line and a trunk road network.
b) Fanling/Sheung Shui New Town: it covers a development area of about 667ha for a population of 326,000 upon full development. The current population is about 254,000. It is served by the MTR East Rail Line and road links.
c) Yuen Long New Town: it covers a development area of about 561ha for a population of 196,000 upon full development. The current population is about 155,000. It is served by the MTR West Rail Line, the Light Rail system and a trunk road network.

Most of them comprise an industrial estate, e.g. Tai Po Industrial Estate and Yuen Long Industrial Estate in Tai Po New Town and Yuen Long New Town respectively to serve industrial diversification in the 1980s. In addition, a Science Park was built in the 1990s at Pak Shek Kok near Tai Po New Town.

1.4 *Third Generation New Towns*

The new towns built in the 1980s and 1990s are commonly called the "third generation new towns". They include:

a) Tin Shui Wai (TSW) New Town: it covers a development area of about 430ha for a planned population of 306,000. The current population is about 290,000. It has been built on land reclaimed primarily from fish ponds, and a wetland park is located to its north. It is served by the MTR West Rail Line, the Light Rail system and a trunk road network.
b) Tseung Kwan O (TKO) New Town: the total development area is about 1,738ha for a planned population of 450,000. The current population is about 386,000. It has been built primarily on the reclamation of Junk Bay, and includes TKO Industrial Estate. It is served by the MTR TKO Line and a trunk road network and major additional road infrastructure are being planned.
c) Tung Chung (TC) New Town: Phases 1, 2 and 3A were completed in 2003 as a component of the Airport Core Programme for the replacement airport at Chek Lap Kok. The current population is about 83,000. It is served by the MTR Tung Chung Line and North Lantau

Highway. The planning and engineering study on the extension of Tung Chung New Town has recommended further reclamation at Tung Chung East for a total population of 264,000.

These new towns are sited in the further parts of the New Territories, reflecting a deliberate decision to continue growth in the New Territories. Great emphasis has been put on transport infrastructure investment as a pump priming approach to new town development. The inclusion of multi-storey flatted factories in new towns for local employment provision is no longer considered a norm for the later generation new towns. TKO Industrial Estate, for example, is mainly for accommodating special industries such as data centres and media industries to serve territorial needs.

1.5 *Latest Situation*

The "Hong Kong 2030: Planning Vision and Strategy" (HK2030) places sustainable development as an overarching planning goal for Hong Kong. There has been a paradigm shift in planning from a "grandiose plan" to a "strategy for sustainable growth". The strategy highlights a preference for optimising available development opportunities, leveraging on the existing urban infrastructure, re-cycling old urban fabric, and being prudent on opening up greenfield land for development. New development areas (NDAs) in the North East New Territories (NENT) and Hung Shui Kiu (HSK) have been proposed to provide nodal clusters around rail stations. The feasibility of developing the NENT NDAs, including Kwu Tung North (KTN) and Fanling North (FLN) NDAs, has been established in planning and engineering studies. It will provide about 60,000 new flats for about 173,000 people, with first population intake envisaged in 2023. According to the Preliminary Outline Development Plan (PODP), the HSK NDA will provide about 60,000 flats for 218,000 people, with first population intake envisaged in 2024.

To plan for a longer perspective, the Government is also undertaking studies to review the feasibility of converting brownfield or deserted lands in the New Territories North (NTN) and Yuen Long South for development. It is also proposing to develop an East Lantau Metropolis (ELM) as a longer term strategic solution space primarily on artificial island(s) in the eastern waters off Lantau for housing and economic development in Hong Kong.

In embarking on another phase of major urban growth (Figure 1), it is time to rethink what is the best development model for the new/potential development areas.

2 A REFLECTION ON NEW TOWN EXPERIENCES

By the end of 1980s, with substantial completion of Fanling and Yuen Long New Towns, the target of housing 1.8 million people has been met. To meet population growth, the new town programme has been extended into the 1990s comprising a total of nine new towns with a total design capacity of 3.4 million people (Table 1). In parallel to the remarkable scale of housing development, significant investments in public works and strategic transport and infrastructure works have been made, making Hong Kong's new town programme one of the largest undertakings in the world.

Table 1 Total population of the new towns in Hong Kong

Year	1973	1983	1993	2013
Population	0.6 million	1.5 million	2.5 million	3.4 million

Source: Territory Development Department (1993) *New Town Development for Twenty Years*

2.1 *Major Achievements*

New towns have positive contribution to the development of Hong Kong. The major achievements are highlighted below:

a) Housing provision: shortage of land in the main urban areas and the dire need for better homes have rendered new town development at unprecedented speed and scale. The three generations of new towns now accommodate about 3.4 million population, nearly half of Hong Kong's population. They contain a diverse housing mix of both public and private housing.

b) More balanced population distribution: the new towns have successfully decentralised population from the urban core to the New Territories. About 41% of the population now live in the New Territories (excluding Tsuen Wan and Kwai Chung which are now classified as Metro Area) (Figure 2), thereby achieving a more balanced territorial development pattern.

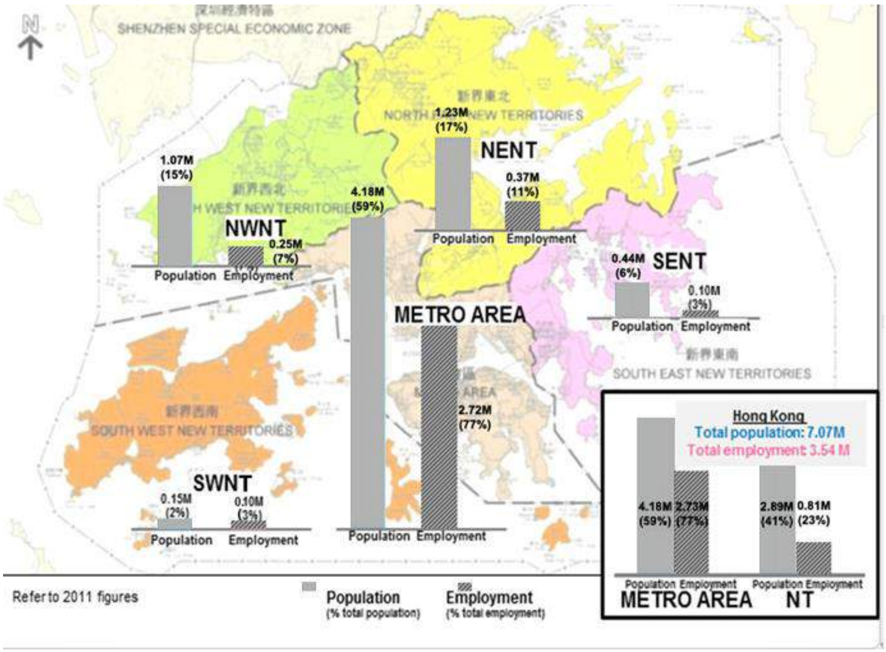

Figure 2 Population and employment distribution of the metro area and New Territories

c) Economies of scale: the clustering of developments in new towns enables savings over the use of land and the provision of public services and transport systems, hence preventing haphazard urban sprawl, protecting the natural resources and preserving the country parks.

d) Improved quality of living in old urban areas: new towns have provided lower density housing with more facilities and greenery outside the urban areas, thereby helping to relieve the congested urban environment.

e) Integrated land and infrastructure development: the new town development projects have involved a coordinated multi-disciplinary expertise set-up mobilising professionals including planners, engineers, architects and landscape architects in individual new town development offices under the then New Territories Development Department set up in 1973. Together, they have been overcoming complicated issues relating to land formation, infrastructure, land use, community facility provision, and implementation in an efficient and coordinated manner under a new town development programme.

2.2 *Major Challenges*

Nevertheless, new town development has also been subject to criticisms from time to time, as featured below:

a) Imbalanced community: TSW, in particular, has been criticised as a remote new town dominated by public rental housing (PRH) (a public to private housing split of 80:20). According to the study on TSW New Town undertaken by the University of Hong Kong (HKU) in 2009, despite the original planning intent of a balanced development in TSW, the change in housing policy (increasing demand for public housing and termination of home ownership scheme), the lack of private sector interest in the TSW development, and the relocation of manufacturing industries to the Mainland have resulted in the predominance of PRH and the removal of the industrial land reserved in TSW. The study concluded that one major challenge for future development of new towns is to ensure full implementation of the planning intention of developing a balanced community through an appropriate housing mix.

b) Large commuting flows: the concept of balanced development with respect to employment was largely based on the presence of manufacturing jobs. Owing to economic restructuring, self-containment in local job opportunities for the new town residents is not realised. Large commuting flows are resulted, as about 77% of the employment remains concentrated in the urban areas (Figure 2). The lopsided situation gives rise to problems such as traffic congestion, long work journeys, less family and leisure time and lower productivity. The significant expansion of high capacity, multi-modal public transport systems has further enhanced both worker mobility and locational choices of industrial/business entrepreneurs, rendering the ideal of balancing local labour force and local job opportunities difficult to attain.
c) "Hiccups" in facility provision: a mismatch in the notional and actual provision of community facilities is often observed, particularly in the early stage of new town development. This could be due to slow progress in the course of new town development or lack of effective inter-departmental coordination with dissolution of the new town development offices and new town development programme. Planning for the changing needs of a "developing community" also poses a challenge. New communities often face shortage of kindergartens at the beginning, followed by that of primary schools and then secondary schools, and ultimately the closure of some kindergartens, primary schools and so forth as the resident population is maturing.

3 THE ROAD TO SMART GROWTH

In the light of the major achievements and challenges in the trajectory of new town development, the Government has endeavoured to think out of the box to create land, infrastructure and a good environment for people to live, work and play. The quest for land for new development still prevails and the demand for various land uses has become more intense, but suitable readily available land resources are more difficult to obtain even in the New Territories. Amidst a land-scarce situation, we need to search for a smart growth strategy to cater for our population growth and the rising community aspirations for better quality of life.

In contemplating a smart growth strategy, reference has been made to Boyd Cohen's Smart Cities Wheel (Figure 3), which embodies key components for developing and implementing smart cities strategies. According to Cohen's work on "The 10 Smartest Cities in Asia-Pacific" with Urban Business Media (UBM)'s Future Cities, Hong Kong was ranked first. Hong Kong performed well for indicators relating to smart mobility (mixed-modal access, prioritised clean and non-motorised options, integrated ICT); smart people (21st century education, inclusive society, embrace creativity); smart economy (local and global interconnectedness, productivity, entrepreneurship and innovation); smart environment (green urban planning, green energy, green buildings); and smart governance (enabling supply and demand side policy, e-government, transparency and open data). Yet, our scores were not so well for indicators accounting for smart living (culturally vibrant and happy, safe and healthy).

In planning terms, our long-standing strategy of creating a compact, transit-oriented and sustainable city is on the right track to achieving smart growth. More emphasis would be needed to enable smart living, which denotes a holistic way of life in a quality environment. More emphasis would also be needed on "doing more with less". Adapting from Cohen's framework, a planning framework for a smart growth strategy in Hong Kong has been derived in pursuit of three major objectives, viz. "creating capacity for growth", "enhancing liveability", and "enhancing economic competitiveness" (Figure 4).

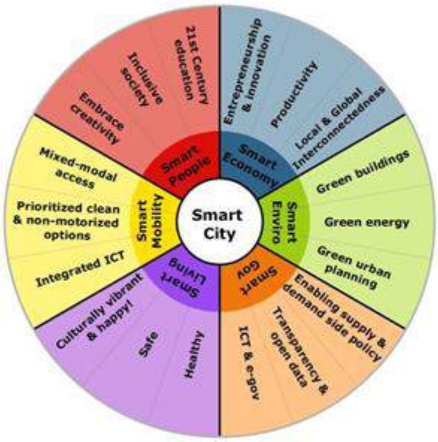

Figure 3 Boyd Cohen's Smart Cities Wheel

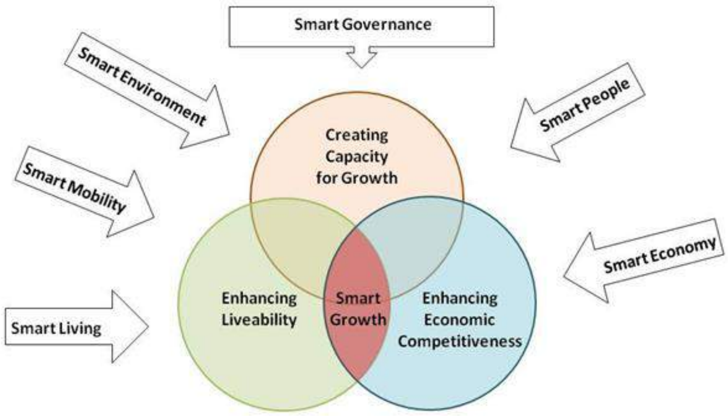

Figure 4 Planning framework for a smart growth strategy in Hong Kong

3.1 *Creating Capacity for Growth*

We need to find solution spaces to sustain population growth as well as social and economic development, while giving due respect to our stewardship to the environment. The key questions are: where, when, what and how to develop to meet the various land use needs. To achieve this, new urban growth could no longer be treated as a separate spatial entity anchored on the conventional notions of garden cities and self-containment. Rather, strategic considerations should play a crucial role.

a) Where and when? – the majority of our new towns have been built on reclaimed land while a few are expanded market towns. "Smart governance" is needed to strategically plan the right location for future solution spaces. In the medium and long term, new developments would be strategically located near the existing new towns to optimise the use of existing facilities, as in the case of the NENT NDAs in the proximity of Fanling/Sheung Shui New Towns. Similarly, HSK NDA is strategically sited in the North West New Territories (NWNT) as a regional centre, midway between Tuen Mun and TSW New Towns and served by railways and highways linking the Airport, Shenzhen and different parts of Hong Kong. Looking ahead in the longer term, ELM has been proposed on artificial island(s) as a solution space, capitalising on the strategic locational advantages of Lantau being in relative close proximity to Hong Kong Island and its hub function to be strengthened by new strategic infrastructures, notably Hong Kong-Zhuhai-Macao Bridge (HZMB) and the third airport runway.
b) What? – learning from our new town lessons, we realised that a sustainable new town must be comprehensively planned with timely provision of infrastructure and community facilities to suit changing needs of a "developing community". We learnt that an appropriate housing mix is significant for creating a balanced community. We learnt that the notion of achieving self-containment in local employment is over-idealistic. Instead, employment provision must be examined from a territorial perspective. "Smart mobility" networking homes and workplaces with efficient, convenient and pleasant transport and walking facilities, and providing for such mushrooming trends as home-based employment and co-working spaces should be considered. To bring jobs to homes and to create capacity for economic activities, it would be more pragmatic to promote mixed developments with diverse job opportunities to attract "smart people" (i.e. talents and workers).
c) How? – In a land-scarce situation, solution spaces have to be created innovatively and strategically, such as going atop or underground, in caverns, on rehabilitated quarries, on brownfield sites, on reclamation outside the Victoria Harbour, etc. In HSK NDA, for example, the Government is attempting to recycle or even "upcycle" brownfield sites by establishing multi-level storage compounds to make way for the NDA development. Constraints would

also have to be wisely overcome, for example, to release development potentials sterilised by consultation zone arising from chlorine storage in water treatment works.

3.2 *Enhancing Liveability*

Smart development should not only be people-oriented but also environmentally responsible, i.e. a "smart environment" for "smart living". To achieve this, the following initiatives are proposed for future new developments:

a) Urban-rural integration: the past generations of new towns tend to contain urban development amidst a rural/suburban setting, thereby creating an urban-rural dichotomy. Keeping the best of both city and rural living, a more appropriate direction for future development might be "urban-rural integration" which attempts to create synergy by blending both the urban and rural environment and activities in harmony. Such initiatives include, for example, conserving nature, preserving the rural heritage, preserving good quality agricultural land, encouraging urban farming, and exploring beneficial rural uses as an integral part of the new town development as far as practicable. For example, Long Valley Nature Park has been designated to preserve wet cultivation as a mean for habitat conservation and agricultural rehabilitation as part and parcel of the development package of the NENT NDAs. In the ongoing planning study for in NTN, a new form of urbanism embodying rural-urban integration would be explored.
b) Leveraging on green and blue spaces: new developments are not just dormitory towns but also provide for leisure, recreation and healthy living. Hong Kong has been successful in preserving much of its natural setting. As such, opportunities should be explored to enhance both "green spaces" (such as country parks, public parks, park connectors, etc) and "blue spaces" (such as coastlines, waterfront, lakes, rivers, drainage channels, urban water features, etc) and their connectivity for public enjoyment.
c) Diverse and inclusive community: different people may prefer different lifestyles. Building on the transit-oriented, compact and comprehensive planning and design framework for new town development, additional emphasis would be put on planning for a diverse and inclusive community with a multitude of living and activity spaces for groups of different ages and having varied needs and aspirations.
d) Smart, green and resilient initiatives: with a view to improving the quality of life, managing the resources prudently and being environmentally responsible, future developments and an integrated green infrastructure system should attempt to encompass smart measures in terms of energy use (e.g. renewable energy, district cooling systems, energy saving infrastructure); mobility (e.g. green transport, car pooling, car sharing); waste (e.g. waste-to-energy initiatives in water treatment and solid waste treatment process, waste minimisation); water (e.g. total water management); drainage (e.g. drainage rehabilitation and sustainable drainage); buildings (e.g. green buildings and green neighbourhoods). Such initiatives could include applying information & communications technology (ICT) platform for better planning and city management. To cope with natural disasters or hazards such as flooding and extreme weather arising from climate change, initiatives to enhance urban resilience to enable quick recovery and to minimise loss should be promoted. The concept is shown in Figure 5.

3.3 *Enhancing Economic Competitiveness*

New towns will be instrumental in bringing about economic benefits for Hong Kong and creating a "smart economy" for economic upgrading. Strategically locating and branding robust economic/employment clusters in the future development areas are highlighted below:

a) ELM is identified as our potential "CBD3", embracing the planning principles of Connectivity, Branding, Design, Diversity and Innovation Development. It is envisaged to be a future producer services hub with locational edge and strategic transport infrastructure connecting to the urban area, the airport and HZMB.

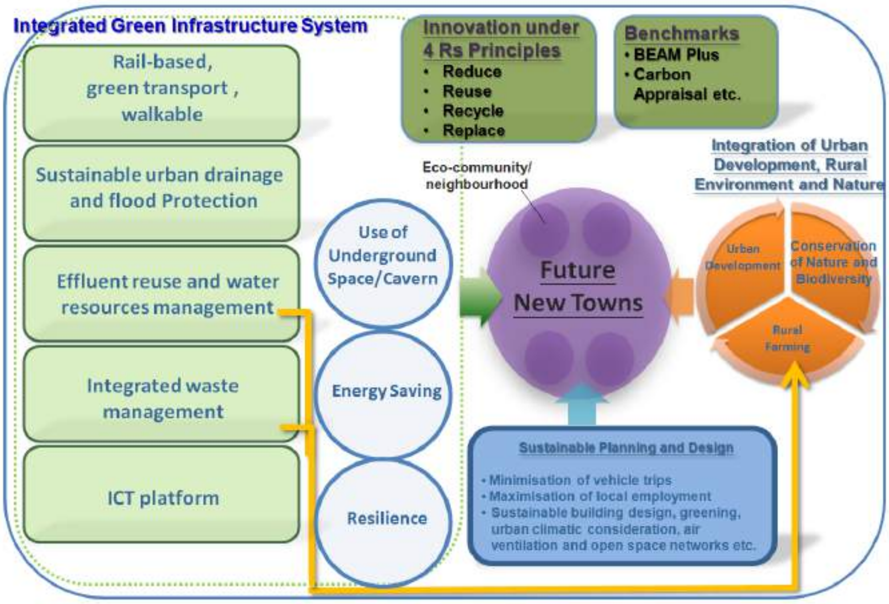

Figure 5 Smart, green and resilient city concept

b) In the NTN, much scope exists for the expansion of the innovation and technology sector along an existing knowledge corridor with the synergy of universities, science park and industrial estate along the NENT major transport corridor.
c) In KTN NDA, a cluster of “Commercial, Research & Development (R&D)” sites are designated to reap their locational advantages near the existing/planned boundary control points and the Lok Ma Chau Loop development. It has potential for developing into various types of offices and R&D uses.
d) The north-western part of HSK NDA is planned for a Logistics and Technology Quarter to house logistics facilities, testing and certification industries, and information technology and telecommunications industries. It is chosen to leverage on its transport hub function connecting with the NWNT and airport, which is conducive to generating economic/employment opportunities for the local and wider areas.

4 CONCLUSION

Planning for new towns has been guided by planning principles evolving over time to suit the specific development context in Hong Kong. The changing guiding principles are also reflective of lessons learnt from the hindsight of new town planning in the previous generations. In brief, the broad directions of new town development in Hong Kong has moved from “resettlement” to “self-containment and balanced development’, and is now moving towards “smart growth”. Addressing the increasing limitations, the smart growth strategy would aim to achieve the objectives of “creating capacity for growth”, “enhancing liveability”, and “enhancing economic competitiveness” for Hong Kong through the development of multi-functional development nodes with a variety of opportunities. Such opportunities would embrace not only housing and employment provision, but also economic, social, cultural, and conservation realms to promote quality way of living. The smart growth initiatives would benefit not only the new development areas but also promote the sustainable development of Hong Kong as a whole. With a projected population growth from the current level of 7.2 million to 8.5 million by 2041, the sheer scale of housing and other development needs must again be catered for in a coordinated manner involving the collaboration of all relevant professionals and parties. As in the past, we need the planning and development to be done with a strategy and the technology that are thought out of the box.

Figure 6 A bird's eye view of Sha Tin new town

(Aerial photo from Lands Department © The Government of the Hong Kong SAR Reference no. G4/2015)

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Innovation in underground explosive storage

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Keywords: underground; explosive storage; design; construction; safety.

ABSTRACT: In Hong Kong, land is a scarce resource and many people are living in very densely developed areas. Most areas suitable for development have already been used, for residential and commercial uses, or for infrastructure. Most of the remaining land is not suitable, typically steep, mountainous terrain, and designated as country parks. What small available spaces that remain are in great demand, particularly for residential development, and often have greater engineering challenges to development. This limitation of available land makes it particularly difficult to identify suitable areas for less desirable land uses, one of which is the storage of dangerous goods, including explosives. Aside from the difficulty of finding locations suitably remote from existing populations to meet the safety requirements, there is strong objection from residents and District Councils for such facilities. It is therefore necessary to consider less conventional approaches to the storage of explosives. One such possibility is the use of underground space, however large underground networks are incredibly expensive, and take a long time to construct. They also require a considerable amount of highly specialised underground installations, and a high level of maintenance is required. Therefore, creative solutions in providing safe underground storage of explosives but with minimal infrastructure and maintenance requirements have been developed. This paper will provide details of the planning process, design considerations, and briefly considers the potential application of underground explosive storage system.

1 INTRODUCTION

The object of this paper is to discuss the development of underground explosive storage facilities to suit the Hong Kong environment. With the progressive increase of underground projects in Hong Kong, many projects require construction by drill and blast method and the demand to provide explosives to all these major civil construction projects is proving difficult without the provision of additional site magazines. Hong Kong has an increasing population and growing economy but a very limited land area and the lack of sufficient surface areas means the choice of appropriate locations for surface explosive magazines is very restricted. For this reason, an underground explosive magazine is considered to be one of the best solutions. Underground storage helps in releasing surface land which can be used for other purposes. It also provides an efficient containment in the unlikely event of an accidental explosion, as the underground chamber will prevent the hazard extending beyond its boundaries.

Geological aspects have a great influence on construction of underground storage and is often more costly than above ground storage. Underground explosive storage sites should be located in sound rock, and extensive access tunnels may be required. However, careful site selection can allow solutions that do not require access tunnels, and simply consist of one or more storage chambers with direct access to a secure surface compound which can house the necessary administration and security facilities.

2 SCHEME OF UNDERGROUND EXPLOSIVE STORAGE

The primary restriction on the siting of an explosive store is that it must be located away from public inhabited areas. The safety distance criteria are based on the explosive storage recommendations by Health and Safety Executive (HSE) of the UK and Manufacture and Storage of Explosives Regulations 2005 (MSER). These include tables giving clearance distances for explosive magazines for various categories of public facility.

In addition, there are some other criteria applied, particularly with regards to the standard of access for delivery vehicles and emergency services, security measures, and provision of utilities for normal operation and also for use in emergencies.

Instead of constructing a standard above ground explosives store, it may be possible to construct an underground/cavern store. Such facility removes the hazard from aerial projectiles in the event of accidental explosion, but introduces other risk relating to stability of the tunnels and airblast from the portal. Underground/cavern storage complexes of this type have been constructed at Kau Shat Wan as the main Government Explosives depot, and at Victoria Road in Pokfulam for the MTR West Island Line project.

Such storage facilities require detailed safety review including Hazard to Life assessment. They require a substantial footprint of land beneath which the underground explosive store is constructed, and security facilities at the portal. They also require an extensive engineering assessment and design. The typical arrangement for underground storage for explosive is shown in Figure 1.

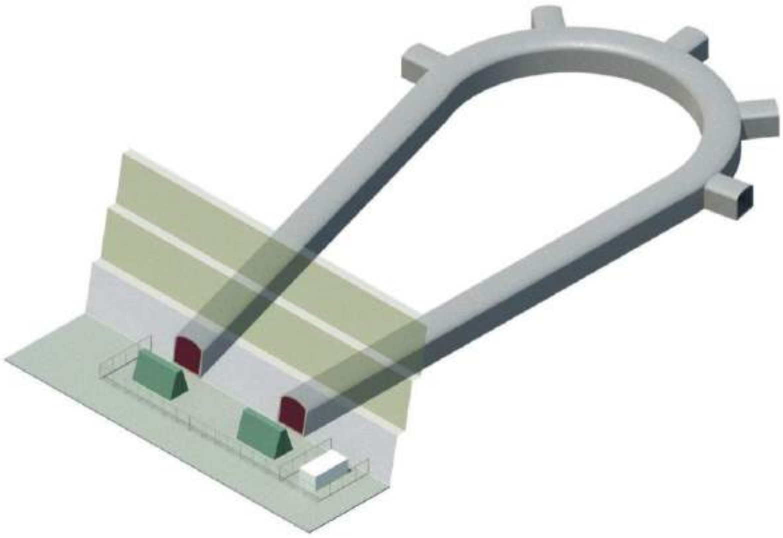

Figure 1 Typical arrangement of underground storage facilities for explosive

2.1 *Benefits of Underground Explosive Storage*

The advantages of underground explosive storage are:

a) Required smaller total land area than surface storage.
b) The use of underground storage complex allows explosive storage to be possible in a location where a surface store would not be permitted, either because of lack of space or because the location of the surface store in that location is not acceptable to the community.

c) The underground explosive store solution also allows one to build in closer proximity to existing project or on otherwise unbuildable sites, thus offering better solution in terms of shorter transportation time to the project.
d) Underground explosive store helps providing higher safety and lower risk to general public. The underground storage complex provides isolation from the community.
e) The underground explosive store also provides isolation from all types of climates. The temperature in underground storage sites is relatively consistent compared with the extremes of surface temperatures.
f) Underground structures are naturally protected from severe weather (typhoon, thunderstorms and other natural phenomena). Underground structures can also resist structural damage due to flooding, although special isolation provisions are necessary to prevent flooding of the structure itself.
g) Underground chambers may have a long life span compared to above ground structures, which are subject to the effects of weather.

However, a simplified underground explosive store concept has been developed, and this is shown in Figure 2.

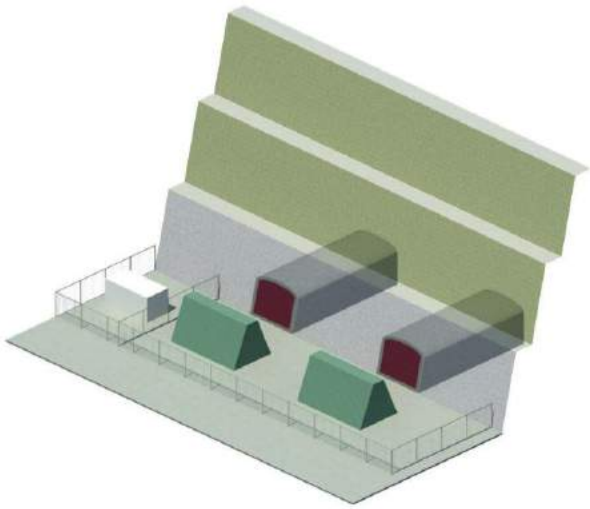

Figure 2 Typical details of simplified underground storage facilities for explosive

3 DESIGN OF SIMPLIFIED UNDERGROUND EXPLOSIVE STORAGE

3.1 *General Description of the Underground Explosive Storage*

Underground explosive storage chambers need be located in rock. A storage site may consist of one or more storage chambers depending on the required storage capacity and safety aspects in the environment of the storage site. A typical underground explosive storage could consist of a number of storage chambers, each with a storage capacity typically ranging from approximately 100kg to 500kg. The storage chambers would be designed to have sufficient rock cover to ensure stability of the chamber and surrounding rock mass even if subjected to the effects of an accidental explosion, and the separation between chambers should not allow communication of such accidental detonation with any adjacent chamber.

A typical underground storage chamber would be horseshoe shaped to maximise the stability of the chamber, typical dimensions of the chambers is shown in Figure 3.

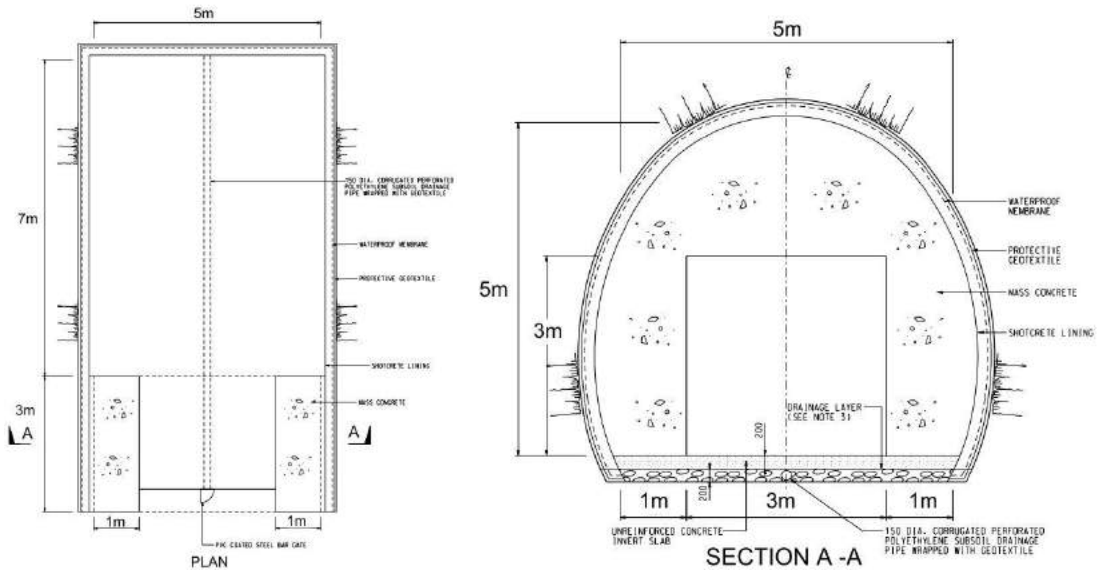

Figure 3 Underground storage chamber plan and section

3.2 *Design Standards*

The design of underground explosive storage facilities are normally based on the US Department of Defense (DoD), DoD 6055.9-M "DoD Ammunition and Explosives Safety Manual". Other guidelines are given in AASTP-1, Manual of NATO Safety Principles for the Storage of Military Ammunition and Explosives.

3.3 *Design Details*

In addition to the safety requirements stipulated in DoD 6055.9-M, the facility will also be designed to meet safety requirements stipulated by FSD and HKP. For such unique facilities, the exact requirements must be confirmed on a case by case basis. However, certain fundamental requirements can be anticipated, to assist with the initial development of the scheme. It is proposed to minimize the amount of infrastructure, such as electrical or mechanical equipment, installed inside the chambers. For fire protection, it is suggested to locate the chambers below the local ground level, and provide sufficient water supply to allow immediate flooding of the chambers in the event of an incident.

3.4 *Method of Construction of the Simplified Underground Explosive Storage*

Typical construction of the simplified underground explosive storage will adopt the following general procedure:

a) Construct access to portal area
b) Construct the portal site formation
c) Excavate the chambers and install permanent support, which may consist of a sprayed concrete lining for simplicity. The typical temporary and permanent support of the chambers is shown in Figure 4.
d) Complete the fitting out of the chambers and portal facilities, including blast wall and security features.

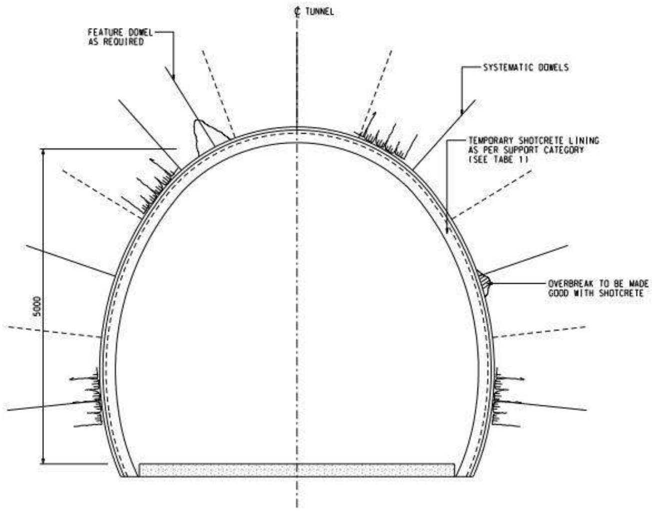

Figure 4 Typical temporary and permanent supports of underground storage chamber in rock

The main difference between the typical and the simplified underground explosive storage is the access tunnel. Normally, the typical underground explosive storage comprises a U shaped 5.5m span access tunnel with two portals. The construction duration of typical underground explosive storage is more than a year, as the tunnel has to be excavated before the chambers. For the simplified underground explosive storage, the construction of each of the chambers can be carried out concurrently. The construction duration of the simplified underground explosive storage can therefore be much shorter. The difference in construction duration between the typical and the simplified underground explosive storage facilities would depend on the length of the access tunnel.

3.5 *Hazard due to Accident Explosion during Operation of the Underground Explosive Storage Facility*

During operation of the underground explosive storage facility, there is a potential hazard to life associated with accidental detonation of the explosives within the underground magazine. Three hazards may be present, arising from ground vibrations, air-overpressure, or debris.

3.5.1 Ground shock

(1) Depth of cover above storage chambers

Underground explosive sites should be located in rock. the DoD manual includes guidelines on calculating the safe depth of rock cover to ensure the stability of the chamber in the event of accidental detonation, and this will vary depending on the explosive storage quantity.

(2) Safe separation distance to prevent rock spalling in adjacent storage niches

When a compressive wave in the rockmass is generated due to accidental explosion within one chamber, this wave would be reflected as a tensile wave when it meets the surface of adjacent chambers. This tensile wave can create spalling of the walls of the adjacent chamber, and the DoD manual includes a method to determine the safe separation distance to prevent such spalling.

(3) Safe vibration limit for nearby structures and slopes

Accidental detonation will induce ground vibration within the surrounding ground, the level of vibration varying depending on the explosive quantity and the decoupling factor (how much open space there is in the chamber). Guidelines are provided on the prediction of ground vibration in Hong Kong by Li and Ng (1992), and the DoD manual includes a method of assessing the decoupling factor. Based on these it is possible to predict the level of ground vibration, and carry out assessments to determine the effect on nearby structures, such as buildings, slopes etc. For slopes this typically follows the approach described in GEO Report 15, and for structures the guidelines described by Siskind et. al (1980) can be followed.

3.6 *Debris*

During operation of the underground explosive storage facility, there is a debris hazard due to possible fixtures and fitting being blown out of the portal in the event of accidental explosion. The risk of debris must be prevented by the construction of earth barriers immediately in front of the portal but with sufficient space allowed for explosive delivery to enter the underground chamber. The earth barriers should be centered on the extended axis of the niche that passes through the portal.

3.7 *Security*

The underground explosive storage chambers will be enclosed within a security fence, and 24 hour armed security guards will be employed. In order to provide protection to the contents of each chamber in the event of any unforeseen event, fireproof lockable gates should be provided at the entrance to each chamber. These will not be required to resist blast pressures resulting from accidental detonation of the explosives within the storage chamber.

4 CONCLUSION

For civil infrastructure projects requiring the use of explosives in an urban environment, underground explosive storage could be adopted which does not require surface space. Underground explosive storage may also be possible where surface storage is not acceptable to the nearby community. In terms of economical aspects, the construction cost of underground explosive complex is generally much higher than those at the surface. However, the simplified underground explosive storage concept presented in this paper comprising short chambers without any extensive tunnel infrastructure would drastically reduce the cost, and would also be much quicker to construct. The short construction duration of the simplified underground explosive storage facility can minimise the programme risk and enable the facility to be completed and commissioned by the time a project is ready to commence blasting.

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Innovative solutions in Hong Kong – Zhuhai – Macao Bridge (HZMB) Hong Kong Link Road Project

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Keywords:

ABSTRACT: The viaducts of the Hong Kong Link Road under Contract HY/2011/09 are precast prestressed concrete structures constructed using segmental balanced cantilever method with span length ranging from 35m to 180m. About 7 km of the viaducts are marine structures and some special techniques have been employed to tackle the challenges encountered in this environment. This paper covers the design and construction methods adopted in the marine viaducts to meet particular operation needs. It delineates the prestressed method employed in the formation of monolithic deck-column connections. Precast segmental piers are used in order to improve construction efficiency and works quality. The pier structural behaviour and measures to enhance the long-term performance are discussed. Pile caps in the marine viaducts are either built at +3.95mPD above the water in open sea at Western Waters or below existing seabed in Airport Channel where flow disturbance should be minimized. The pile caps above water are constructed using precast concrete shells designed as temporary work. Special design and construction considerations for the shells are discussed in this paper.

1 INTRODUCTION

The Hong Kong Link Road (HKLR) is a prominent structure that serves to connect the Main Bridge of HongKong-Zuhai-Macao Bridge (HZMB) at the HKSAR Boundary and the tunnel portal at Scenic Hill in Airport Island (See Figure 1). The works under Highways Department Contract No. HY/2011/09 comprises mainly design and construction of approximately 9km of viaducts supporting dual 3-lane carriageways passing through four zones.

In the west, it is open water with a vast horizon where the HKLR connects to the HZMB Main Bridge in the mainland. Two 1-way navigation channels, each with a minimum navigable width of 100m, are designed to have a clear spacing of 200m. See Figure 2a. Existing seabed in this area is generally underlain by 30m-40m of soft marine clay. Due to the presence of fault zones, the bedrock in some locations exceeds 100m deep.

As the viaducts approach San Shek Wan and Sha Lo Wan, the geology changes drastically where bedrock becomes very shallow. Since an archaeological site is located in the vicinity, no structures (either permanent or temporary) are allowed to be built in this sensitive zone and therefore, long-span decks (165m-180m) have been adopted in this area.

Long-span decks are continuously employed in the Airport Channel where a 2-way navigation channel of 46m is provided. A 180m long span is used to cater for the sharp skew angle of the navigation channel. All pile caps within the Airport Channel are embedded below existing seabed in order to minimise any hydrodynamic impact to the Channel, except those for the navigation span which are emerged with the pile cap top above the sea at +3.95mPD. See Figure 2b.

About 2 km of the viaducts are on land along the southern side of the Airport Island. One of the critical site constraints for constructing the viaducts is the requirement on Airport Height Restriction (AHR) pursuant to the Hong Kong Airport (Control of Obstructions) Ordinance. All works on the Airport Island have to be carefully planned and designed without imposing any risk to the normal operation of the airport.

There are many challenges in this project, namely, voluminous construction in marine environment that imposes difficulties in material logistics, working close to sensitive areas such as the archaeological site & Hong Kong International Airport (HKIA), foundation works in complex geology with significant ground variation and so on. This 12.9 billion Hong Kong dollar contract was awarded to Dragages-China Harbour-VSL Joint Venture (DCVJV) in June 2012. YWL Engineering Pte Ltd (YWL) was appointed as the Designer and construction engineering consultant for the marine viaducts.

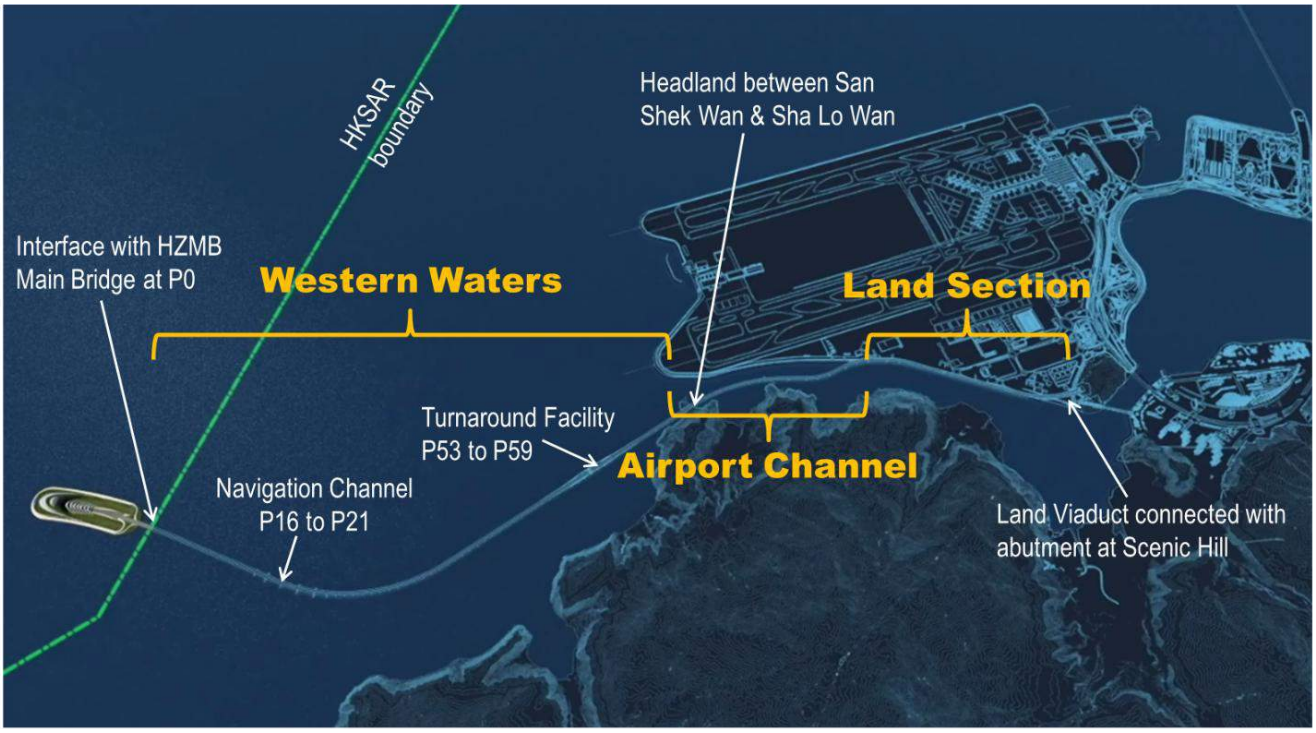


Figure 1 Location plan

2 CHARACTERISTICS OF THE VIADUCTS

Precast segmental technology has been proven to be an efficient method to large scale viaduct projects and has many successful examples in Hong Kong and worldwide. This technique was selected as the predominant form in the design of the viaducts in this project. Since the majority of the bridge spans are over 75m long, balanced cantilever method with epoxy glued joints was adopted due to its flexibility and cost effectiveness in construction.

The viaducts in this project consist of 115 spans with span length ranging from 35m to 180m. Conventional medium span decks were primarily used in the land viaducts. The typical span length of the marine viaducts in Western Waters is 75m with a constant box depth of 4m. In articulation design, a typical bridge unit of 8-span continuous deck (600m long) was adopted with optimal use of bearings and movement joints so as to reduce future maintenance effort. Other than the first internal piers, all other internal piers were constructed monolithically with the bridge deck (see Figure 4a and 4b). To improve the aesthetics, the length of the end span was made the same as the internal ones in order to provide a regular rhythm of the spans (see Figure 3a).

a. Viaducts in Western Waters

b. Viaducts in airport channel & headland

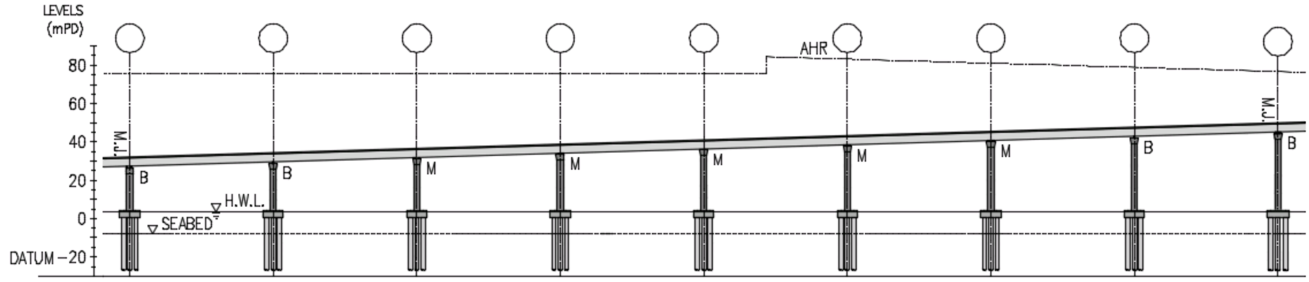
Figure 2 Artist impression

The functional requirement of the navigation channels in Western Waters was realized by a 5-span bridge unit with a configuration of 109m+3x150m+109m. Movement joints and bearings were provided only at the 2 ends of this 668m long bridge. The segmental box girder has a continuously varying depth from 7.936m at pier to 4m near the mid span. The internal twin-blade piers were monolithically connected to the deck. The form of these piers was selected with the objective to minimize the longitudinal stiffness of the bridge and reduce the lock-in effects due to creep, shrinkage and thermal deformations (see Figure 3b & 4c).

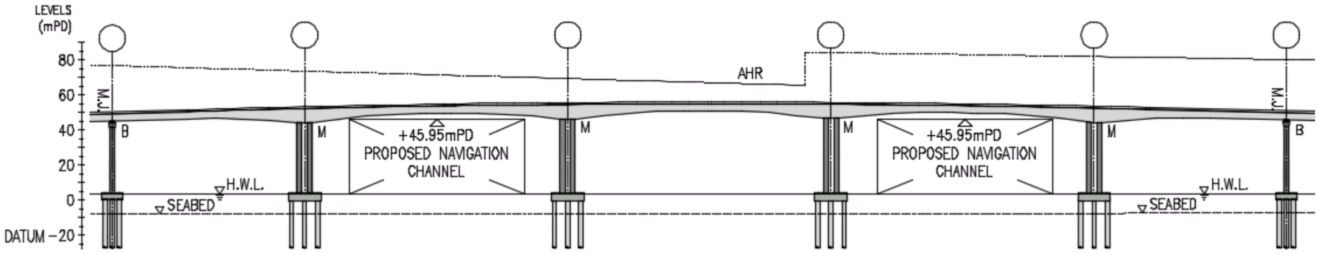
A grade-separated turnaround facility near San Shek Wan comprised of slip roads in the form of single-lane viaducts bifurcates from the HKLR mainline carriageways forming an elevated junction above. It consists of 6-span continuous mainline deck integrated with 2 ramps. A smaller box section of 9m wide was deployed for the ramps.

In the headland between San Shek Wan (SSW) and Sha Lo Wan (SLW), where an archaeological site had been located, a long-span deck of 180m overpassing the headland was proposed. The bridge configuration that provides a 180m long clear span in the centre is a 3-span continuous structure with two 115m long end spans. The segmental box girder has a continuously varying depth from 10m at pier to 4m near the mid span. The vertical alignment of the bridge was designed to satisfy the criterion that the span over the existing SLW pier will not obstruct its daily operation. The articulation of this bridge unit is similar to the one in the navigation channel with monolithic internal twin-blade piers and bearings at two ends (see Figure 3d).

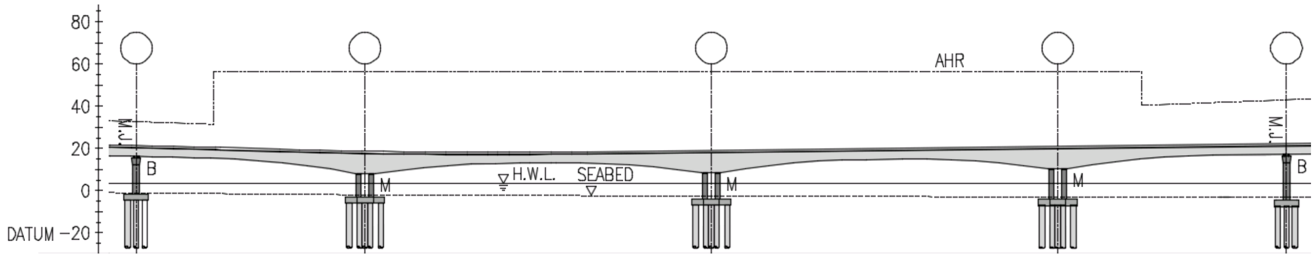
For viaducts in the Airport Channel, 4 bridge units with two generic configurations of 109m+2x165m+109m (see Figure 3c) and 115m+2x180m+115m were proposed. Their articulation is the same as other long span structure in this project.



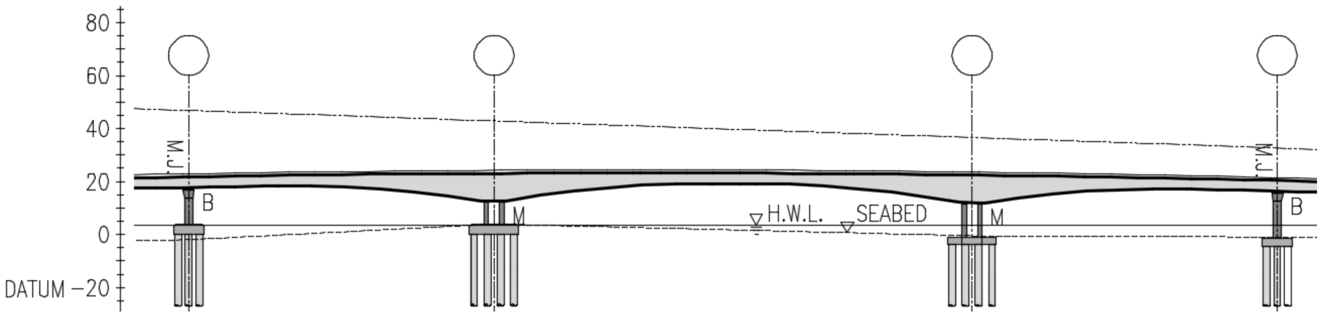
a. Typical bridge unit in Western Waters (8x75m)



b. Navigation channel unit in Western Waters (109.7+3x150+109.7m)

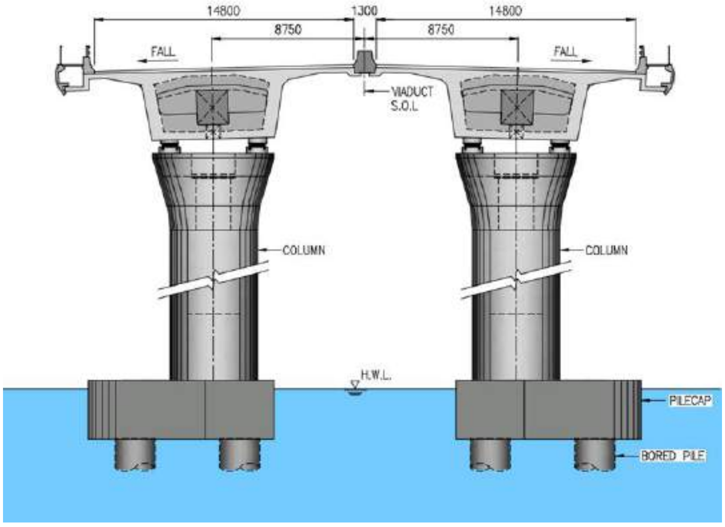


c. Airport channel unit (109+2x165+109m)

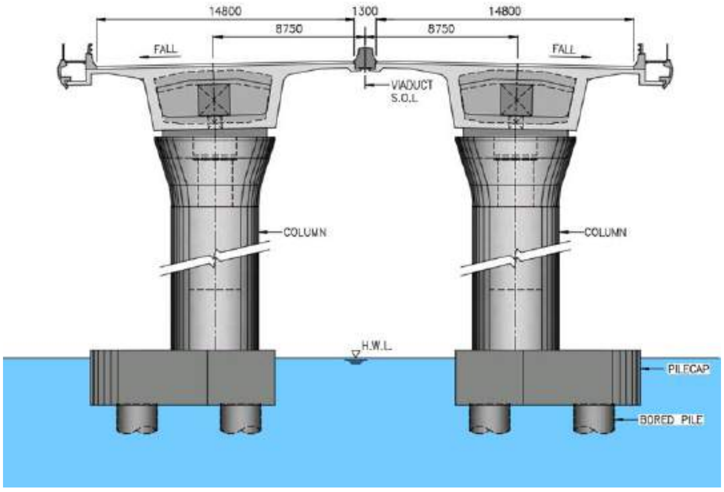


d. Headland unit (115+180+115m)

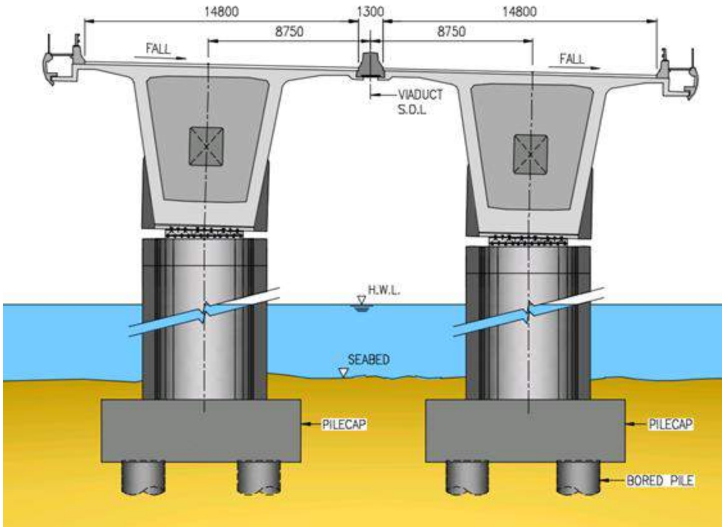
Figure 3 Bridge elevation



a. Typical pier bearings



b. Typical monolithic pier



c. Double blade pier for span larger than 100m

Figure 4 Bridge sectional view

In the superstructure design, a mixed prestressing scheme using both internal and external tendons was employed in order to minimize the self-weight of the deck and reduce the erection cycle time. Internal cantilever tendons were provided with an amount sufficient to support the self-weight of the cantilever decks while external continuity tendons & internal span tendons were to deal with the remaining loads. The viaducts were checked to ensure that failure of any 2 external tendons will not lead to collapse at ultimate limit state. All external tendons were designed to be replaceable. Spare ducts were provided for the tendon replacement operation. At deviator segments, the external tendons are deflected at an angle. In order to enhance the performance and constructability of these deflectors, a double curvature diabolo surface was adopted in the design instead of the conventional single curvature steel tubes that are commonly used in projects in Hong Kong. See Figure 5.

Figure 5 Prestressing ducts for mixed scheme

In this project, the precast concept was utilized not only in the superstructure design but also extended to the substructure and pile cap works in order to enhance the works quality and speed of construction in marine environment. Two specific and important areas of precast applications in this project are: (1) Prestressed Piers & Monolithic Pier-Deck Connection; and (2) Precast Concrete Shell for Pile Cap Construction.

In the Western Waters, tall piers except the twin-blade piers in long-span decks were designed as precast prestressed concrete structures. The typical pier section is hollow with typical external dimension of 5.0mx3.2m and internal dimension of 3.0mx1.5m. A typical precast column unit is 6m and an in-situ stitch of 400mm is used to connect the in-situ pier base and the first precast unit. U-shape internally prestressed tendons were used to connect the precast units. All prestressed structures in this project were designed as Class 1 structure under the service load combinations as stipulated in the Structures Design Manual for Highways and Railways (SDMHR) published by Highways Department, the Government of the Hong Kong Special Administrative Region. For the formation of monolithic connection between the pier/diaphragm segment and the pier, prestressing system was used instead of traditional reinforced concrete approach in the connection design. Details of the design considerations are discussed in a separate section below.

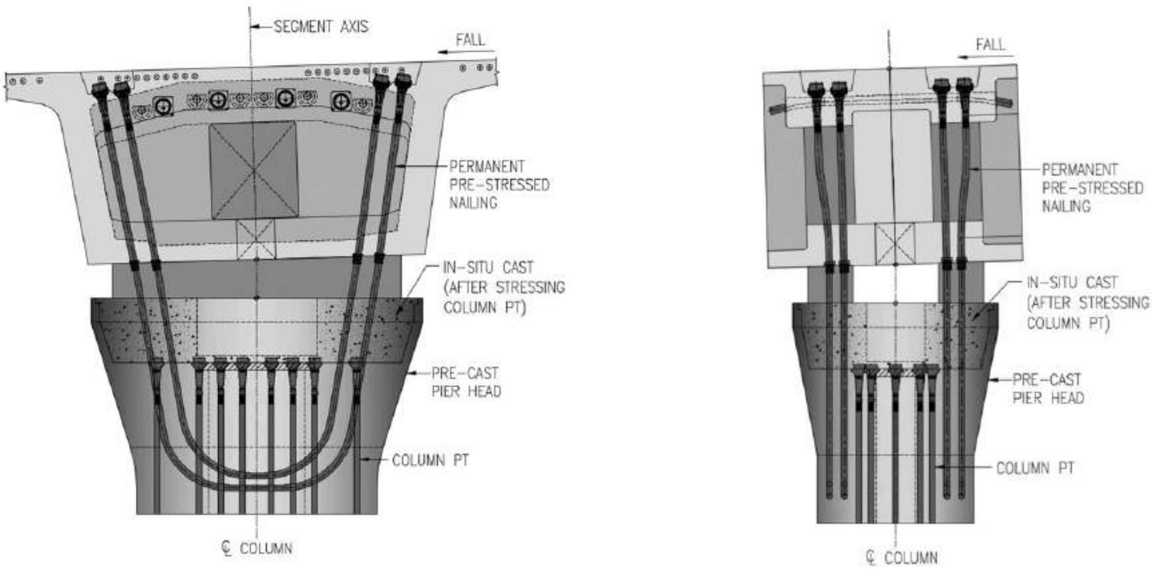
There are a total of 660 numbers of marine bored piles in this project. The diameters of the piles include 2.3m, 2.5m and 2.8m. Due to significant variation in geology, the pile length varies from 7m to 107m. For piles over 85m long, friction bored piles are employed and these piles are shaft-grouted. Pile caps are conventional reinforced concrete structures. There are 7 generic caps including the dolphins at navigation channels. In Western Waters, the emerged pile caps typically of 4m thick were proposed with the pile cap top at +3.95mPD. They were constructed inside the precast concrete shell special formwork that can provide a safe and dry environment to enable the work to be carried out in marine tidal zone. The precast shells design is unconventional due to the durability requirements. Special design concepts and considerations of the shells are discussed in a separate section below.

3 PRESTRESSED PIER AND MONOLITHIC PIER-DECK CONNECTION

In earlier projects in Hong Kong, deck-pier monolithic connections are typically formed by two methods. The first one is to cast the pier segment together with the pier as an integral unit by in-situ method. For geometry control purpose, wet joints are subsequently introduced between the pier segment and the first typical segments on both sides during deck construction. The second common approach is to precast the pier segment as a shell which will later be infilled with in-situ concrete and the connection joint is normally a reinforced concrete structure. If the pier shell segment is match-cast with the adjacent segments, the wet joints could be eliminated.

In this project, the monolithic connection between the pier segment and the pier was achieved using prestressing. All structural components were precast. This method is a new attempt in the local practice and is driven by the need to minimize the in-situ concrete works in the marine environment and optimization of the erection cycle time. After the pier segment was installed with geometry adjusted, its spatial position was fixed on temporary supports. The precast plinths would then be grouted prior to application of the vertical nailing prestressing in the formation of the monolithic joint.

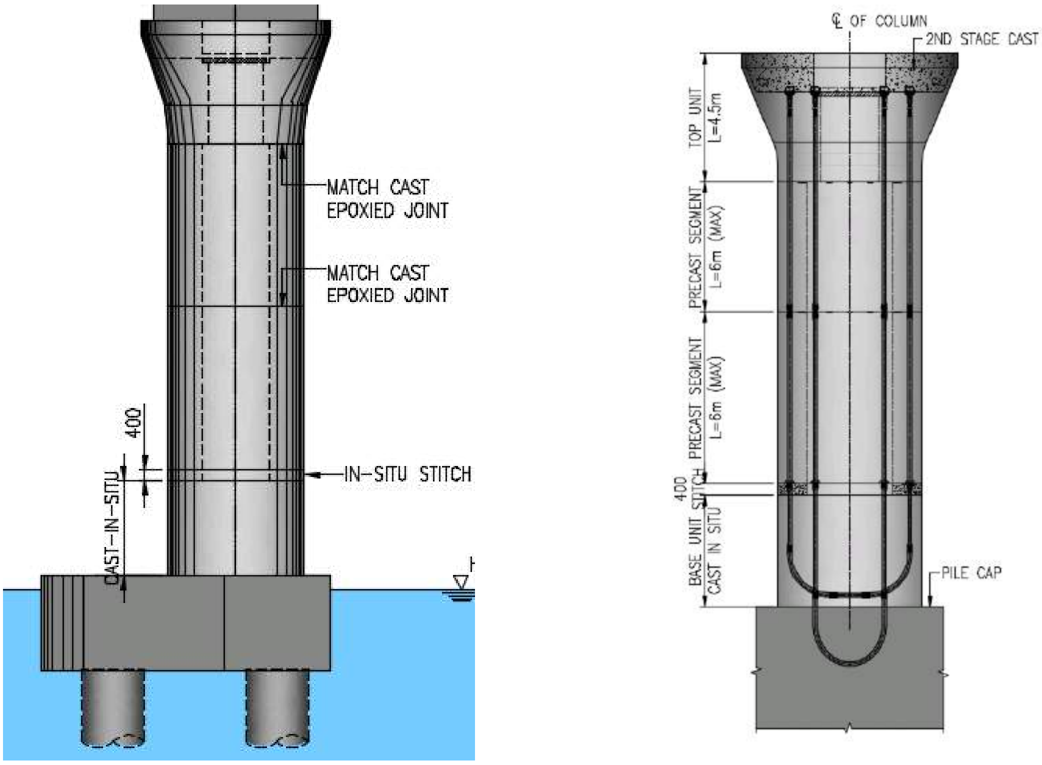
The U-shape nailing tendons used in this project were designed to be anchored on top of the pier segment so as to facilitate the stressing operation. They were embedded in carefully designed recesses of 420mm deep with at least 200mm of protection capping concrete on top in order to avoid accidentally damages by road surface milling operations. One of the design difficulties was to search for space to accommodate these tendons in the congested zone interfered by reinforcement and other prestressing tendons (see Figure 6a and 6b).



a. Sectional view b. Side view

Figure 6 Monolithic deck-pier connection with prestressing

In the Western Waters, the piers are generally much taller than those in the Airport Channel. Coupled with the geographic disadvantage of being far away from land in open sea, the precast option has become attractive. Precast prestressed concrete column is another novel element proposed in this project. A typical precast pier unit is 6m long. They are formed by match-casting technique (see Figure 7). A wet joint of 400mm is introduced between the cast in-situ pier base off the pile cap and the first precast column stem. Long U-shape vertical tendons are employed to connect the precast units together. Some tendons are embedded in the pile cap while some are in the cast in-situ base due to space limitation and ductility requirement at the column base. The tendons are anchored at the pier head instead of the pier segment so that these two construction activities are delinked.



a. Pier GA b. Tendon arrangement

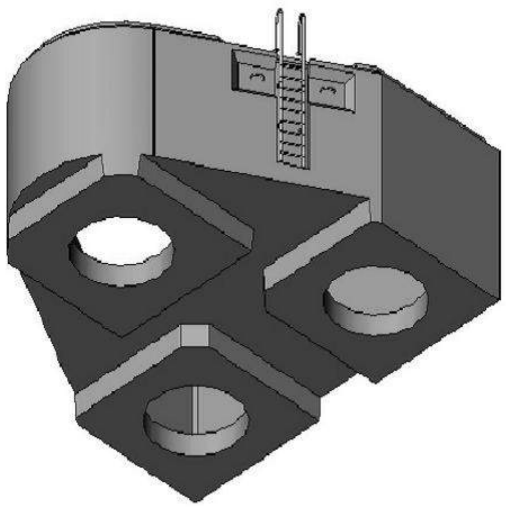
Figure 7 Precast and prestressed pier

During the design development, the ultimate limit state behaviour of the prestressed piers under seismic load scenarios was carefully investigated. Since the pier is mainly an axially loaded element, excessive deformations due to loss of prestressing as an advance warning may not be very noticeable as compared to a flexure member. However it was found that the governing load case for the design of these piers is mainly that from the construction stage loads. There are reasonable margin for the pier in service condition against a nominal reduction in strength due to tendon corrosion. The reliability of the system has been further ensured by the additional precautionary measures in the grouting operations and the provision of Electrically Isolated Tendons (EIT) system for corrosion monitoring. The EIT system involves complete electrical isolation at anchor heads to enable monitoring of the full tendon encapsulation by measuring the electrical resistance between the tendons and structures. It is an additional way to minimize stray currents as well as limit galvanic corrosion currents.

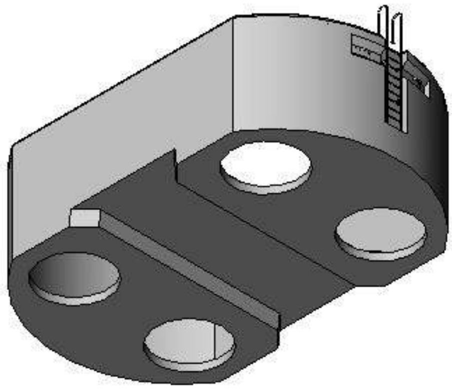
4 PRECAST CONCRETE SHELL FOR PILE CAP CONSTRUCTION

Except for the viaducts inside the Airport Channel, top level of the pile caps in the marine viaducts is at +3.95mPD so that they are observable by the vessels. The bored pile group in this project ranges from 3 to 6 with diameter of 2.3m, 2.5m or 2.8m. During the concept design stage, various temporary works options were considered. Conventional steel form mounted on a sacrificial concrete bottom slab was investigated. The main concern for this option is water tightness and the effort needed in demoulding as a significant amount of operation has to be executed in water. Therefore, it led to the development of the use of concrete shell as the temporary works. Initially, there were lots of debates on the possible integration of the shell to form part of the permanent structure. However, the amount of additional reinforcement needed, mainly stainless steel as required in the specifications for controlling the crack width to 0.1mm, for resisting the thermal stresses and the interface shear between the shell and second concrete pour invalidated this concept due to commercial reasons. The final decision was to use the precast concrete shells mainly as temporary works but integrated with the permanent cap as additional protection against corrosive marine environment.

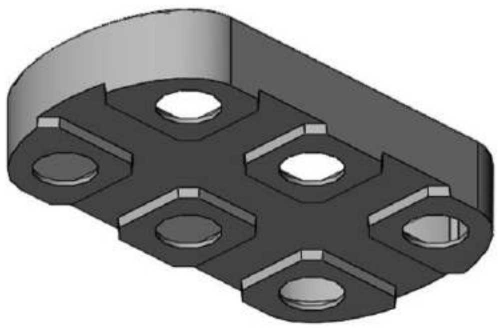
There are 7 types of generic concrete shells including that for the dolphin structure. In this paper, the discussion on the engineering behavior focuses only on the design of the CP1 shell which forms more than 50% of the caps in the marine viaducts. The CP1 shell is tailored for the 3-pile cap with a triangular shape of about 10.5m long and 3.7m deep. The wall thickness is 300mm and the bottom slab is 450mm. The 3D views of some shell structures are shown in Figure 8.



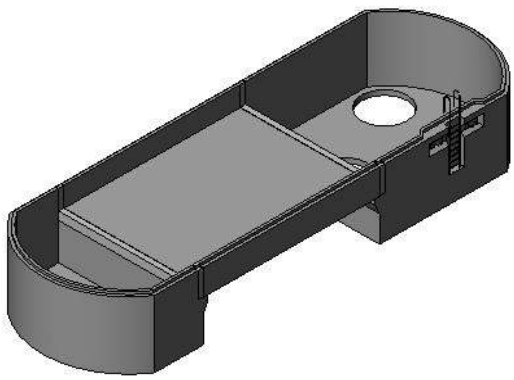
CP1



CP2



CP3



CP4

Figure 8 Precast shell - 3D views

Two aspects of the shell design were carefully considered, namely, strength requirement and durability performance. In strength design, the shell is to resist the concrete pressure from the permanent core, buoyancy force and wave load, construction load and thermal effects due to expansion of the core from heat of hydration.

Although the shell is not part of the permanent structure for resisting the service loads, it serves as a protection for the cap against the corrosive marine environment during its design life of 120 years in the durability assessment. Therefore, the crack width and lock-in stresses on the surface of the shell were carefully evaluated.

Due to the complex shell geometry, 3-dimensional finite element analyses were conducted to simulate the structural behavior of the system in different stages (See Figure 9). It was found that the thermal stresses induced due to concrete hydration in the permanent cap would be the governing load scenario. The tensile stresses could be in the order of 6-7MPa. Without controlling the early thermal effects (e.g. using cooling pipe), a significant amount of the tensile reinforcement would be needed in order to avoid cracks on the surface.

Driven by the need to release the thermal strain induced in the shell, a novel idea of thermal-structural insolation was conceived in the shell design. A thin layer of isolating material with specific mechanical and thermal property was introduced and mounted on all inner faces of the shell. This layer of material has Young’s modulus ranging from 500kPa to 4MPa and thermal conductivity around 0.04 kCal/h.m2.oC and was designed with 2 functions. Firstly, it serves as part of the formwork and transfers the wet concrete pressure to the shell. Due to its relatively small Young’s modulus compared to concrete, additional thermal strain developed at a later stage will be reduced. Secondly, it acts as a thermal insulation material to reduce the temperature gradient between the surface and the concrete core.

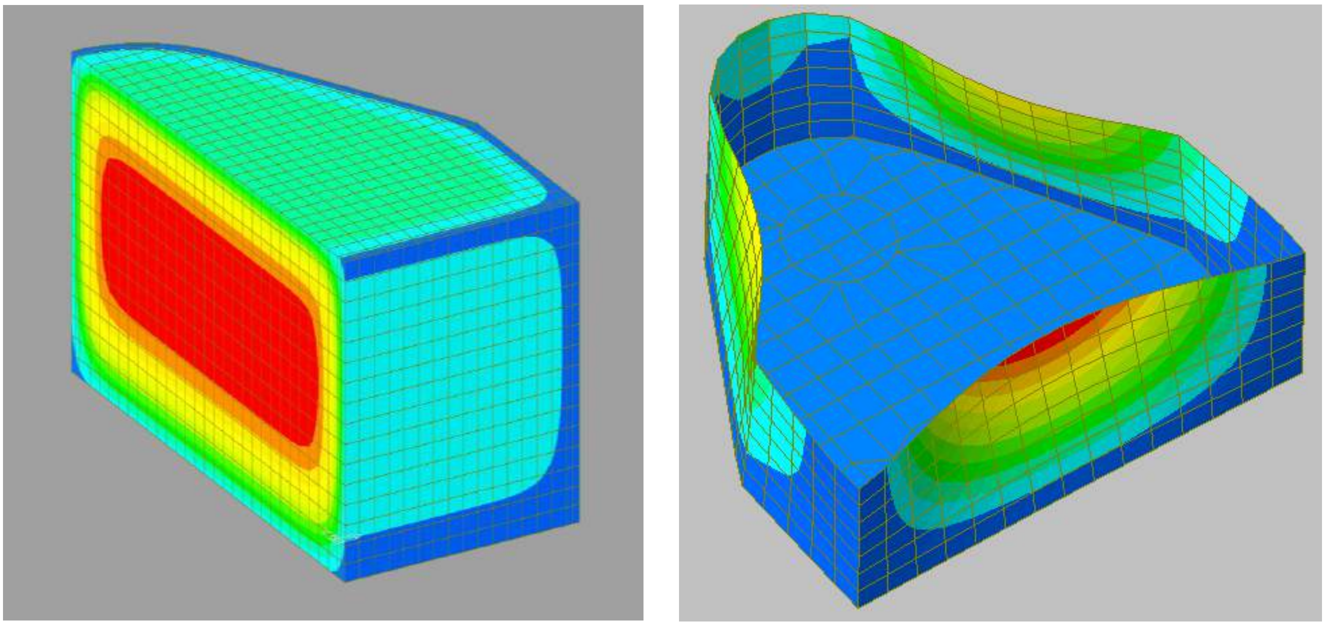


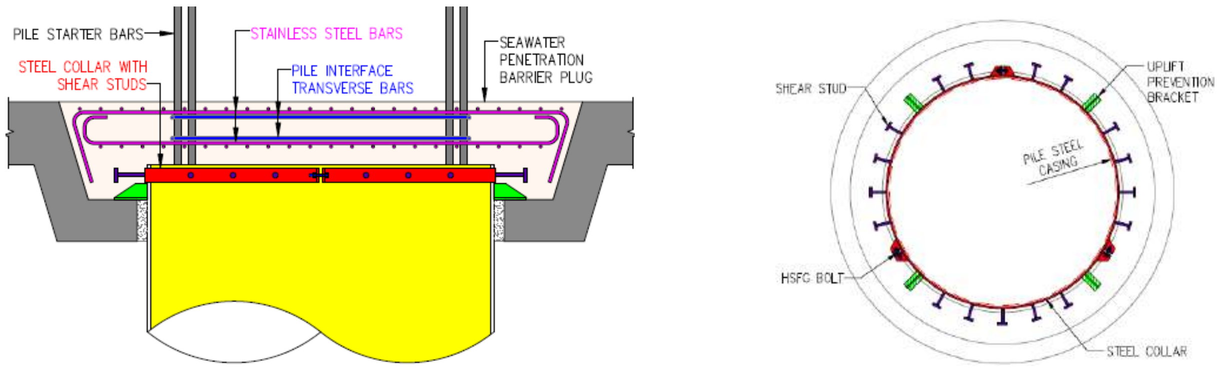
Figure 9 Finite element model of pile cap and concrete shell

Effect of shrinkage of the second concrete pour was also investigated and a special detail at the joint between the shell and in-situ concrete was adopted to avoid cracking at the joint and create a water leakage path. Stainless steel reinforcement was used to connect/integrate the concrete formed in two stages.

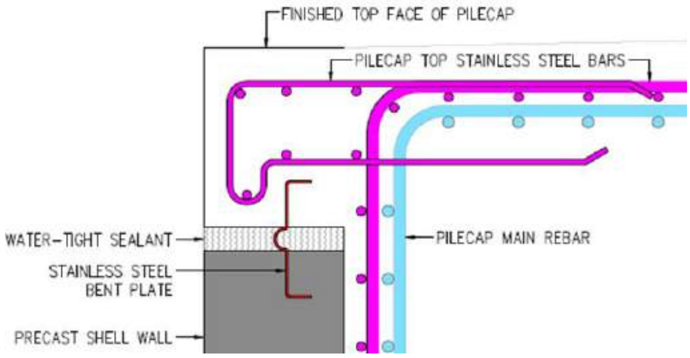
CP1 shell is the lightest one in this project. Its weight including the lifting beam and the installation accessories is 230ton. A 300ton crane barge was employed for the erection. The heaviest shell (Type F1) is about 560ton and it was installed using a 1000ton cane barge. All the shells were fully precast in the mainland China prior to being shipped to site for installation.

In the design, due considerations were given to construction tolerance and installation procedure. Since marine piles always demand higher tolerance limit, openings at the base slab have been oversized by 150mm for the external casing with a diameter of 2478mm OD. See Figure 10.

Installation of the shell involved a series of load transfer. Wire ropes were employed for lifting the shell and its installation accessories in the final position. The weight of the system was then transferred via the lifting beam, a key component of the installation accessories, to the king post embedded in the permanent piles. Partial water tightness was achieved by sealing the gap between the shell and pile with a rubber gasket before casting of the concrete plug. After the concrete plug was formed, the system would be completely water tight. All loads including the weight of pile cap wet concrete could then resisted by the reinforced concrete plug.



a. Formation of plug at pile location



b. Treatment on water-tightness at interface at pile cap top

Figure 10 Precast shell – interface details

Figure 11 Installation of precast concrete shell for pile cap construction

5 CONCLUSION

The precast segmental method has been commonly used in Hong Kong for superstructure works. Prestressed method in the formation of the monolithic deck-column connection and pier works is a new attempt in this project driven by the need to minimize the in-situ concrete works in the taxing marine environment and optimization of the erection cycle time. The special design and construction considerations for the concrete shells in the pile cap works are also novel ideas motivated by the construction requirements.

6 ACKNOWLEDGEMENTS

For a project of this scale and complexity, there are always many individuals who have contributed to the works; their names, however, would be too long to be all listed. Firstly the authors like to thank Highways Department to permit the publication of this paper. Special thanks are to those few who have significant help and influence on the authors' works, namely, Ir. K.Y. Yung (Highways Department), Mr. W.K. Poon (China Harbour Engineering Company Ltd), Mr. Yves RIALLAND (Dragages), Er. Shengfa CAO (YWL) and Er. Boonfei CHEW (YWL). Their efforts made this project happen.

Design consideration for artificial ground freezing for passenger adit construction for the Hong Kong MTR West Island Line

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Keywords: artificial ground freezing; temporary support; tunnel; creep.

ABSTRACT: Artificial Ground Freezing (AGF) techniques have been used in Hong Kong's for tunnel construction since 2001. This technique was first used on the Habour Area Treatment Scheme (HATS) Stage 1 project and subsequently adopted on other larger scale infrastructure projects such as HATS Stage 2A and MTRC West Island Line (WIL) Works Contract 703. This paper presents the design of a passenger adit excavation on MTRC WIL Works Contract 704 using AGF techniques. The passenger adit, is known as Sai Ying Pun (SYP) Adit B3-1 and B3-2, is 5.95m in diameter and 100m long, connecting the SYP Entrance B1/B2 and B3. AGF techniques were used because the adit was located in a densely populated urban area underlain by soft ground. In particular, a number of old buildings are located above the adit where the risk of damage to these structures caused by groundwater drawdown and ground settlement were of concern during adit construction. As such, AGF techniques were adopted and the design of a 1.3m frozen soil ring with an average temperature of -10^{o}C around the adit was undertaken. The frozen soil ring essentially acts as a water proofing layer which virtually eliminates settlement due to no groundwater inflows. The frozen soil also provides pre-support for the adit excavation to minimise settlement due to ground relaxation prior to installation of temporary support, thus minimising impact on the existing buildings above the adit. This paper presents the design considerations, material property derivation and the analysis undertaken for the design of the frozen soil ring adopted as part of the overall design of the SYP Adit B3-1 excavation.

1 INTRODUCTION

The West Island Line (WIL) is one of the priority railway extensions recommended in Railway Development Strategy 2000. The Works Contract 704 consists of caverns of Sai Ying Pun (SYP) and Hong Kong University (HKU), two connection tunnels from Kennedy Town Station (KET) to Sai Ying Pun Station (SYP) via Hong Kong University Station (HKU) and associated adits (Figure 1 refers). Gammon-Nishimatsu West Island Line Joint Venture (GNWILJV) was employed by MTRC as a main contractor for the project while Mott MacDonald Hong Kong Limited (MMHK) was employed by GNWILJV as contractor designers.

A SYP Passenger Adit B3 which consists of 80m long SYP Passenger Adit B3-1, 20m long Adit B3-2 and the junction area between Adit B3-1 and Adit B3-2. The SYP Passenger Adit B3-1 starts from the SYP Entrance B3 and passes underneath Queen's Road West, First Street, the SYP Passenger Adit B3-2 starts from the SYP Entrance B1/B2 and connects with the SYP Passenger Adit B3-2 under First Street. The adit's crown level (i.e. from -15mPD to -18mDP) is varying from 20m to 30m below the ground level (i.e. from +7mPD to +16mPD). There are several buildings, which are resting on friction piles or footings, are located about the SYP Adit B3 (Figure 2 refers). These buildings and their foundation are critical sensitive receivers for the SYP Passenger Adit B3 construction works. In order to minimize the adverse effects (e.g. settlement) on these buildings, a comprehensive temporary support system is required for the adit excavation. The adit is circular in shape with diameter of 5.95mm (including the permanent lining) and mainly embedded in completely decomposed granite, a cross section of SYP Adit B3-1 is presented in Figure 3.

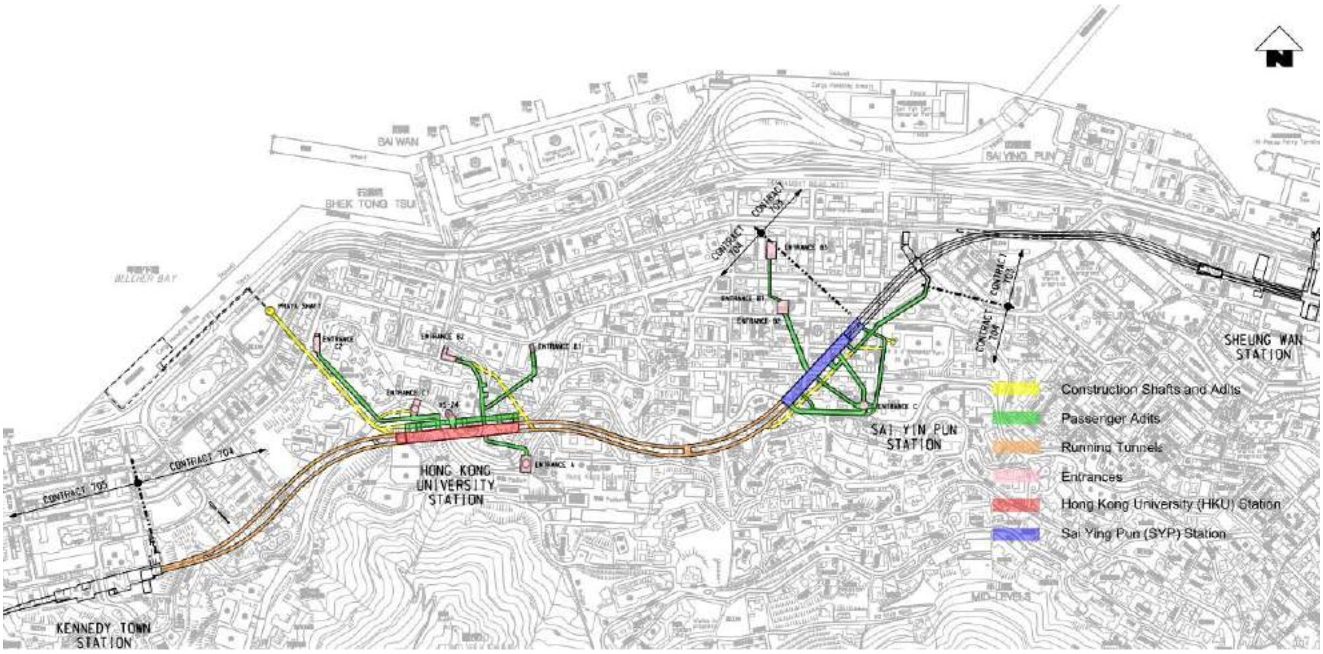

Figure 1 Project layout plan of West Island Line works Contract 704

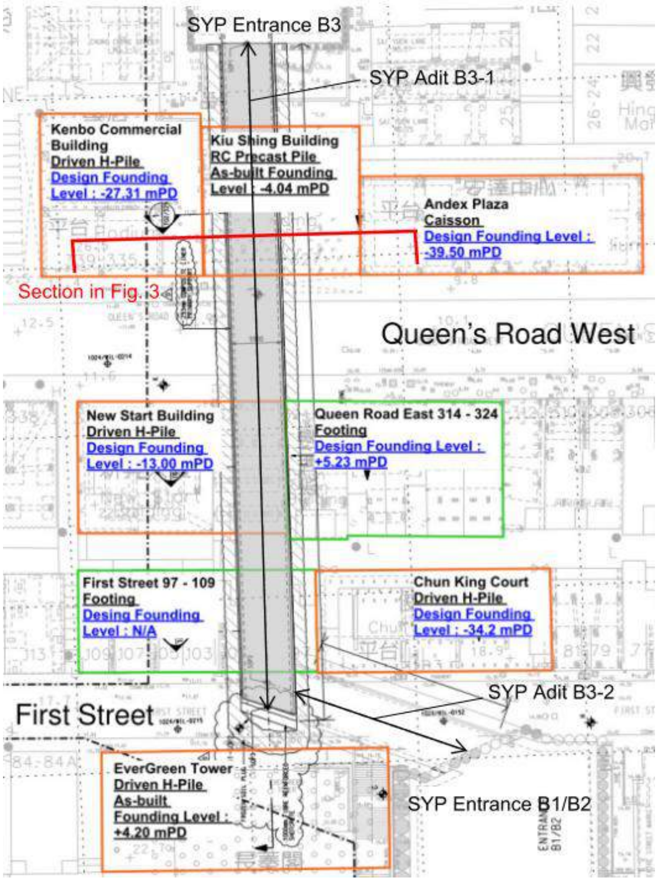

Figure 2 Layout plan of SYP passenger Adit B3

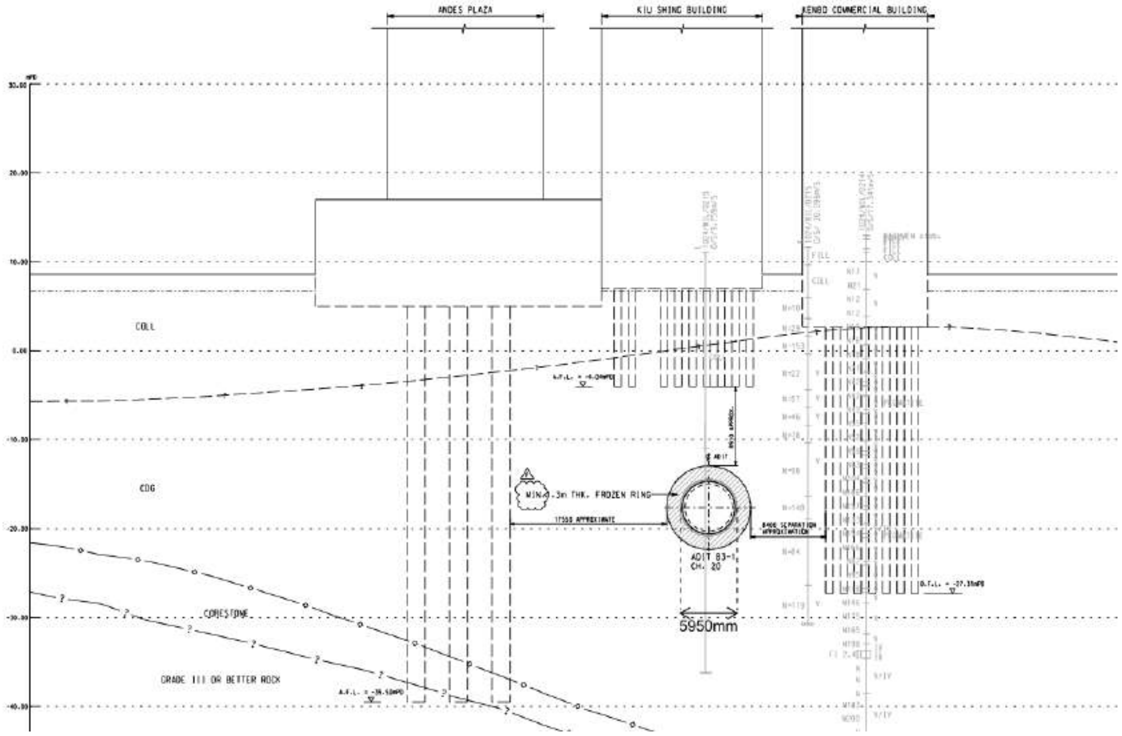

Figure 3 Cross section of SYP Adit B3-1

The surface and building settlement caused by adit excavation are due to groundwater drawdown and the deformation of the adit's temporary / permanent support lining. The settlement caused by lining deformation can be minimized by strengthening the temporary and permanent supporting system. The settlement caused by groundwater drawdown can be controlled by minimizing/ preventing the ground water inflow into the adit during excavation. In order to minimize/ prevent groundwater inflow into the adit, especially in soft ground condition, pre-grout and post-grouting are commonly adopted; however, grouting works cannot guarantee the performance of groundwater cut-off prior to the excavation works. Since a number of aged buildings are located above the SYP Passenger Adit B3, grouting method may be too risky to be adopted. Ground freezing method is an alternative method to prevent groundwater inflow into the adit; Ground freezing method has following advantage and disadvantage:

Advantage:

a) Water inflow into the excavation can be completed seal off by forming a frozen soil ring; minimize the ground settlement due to groundwater drawdown;
b) Frozen soil can be used as part of tunnel's temporary support system;
c) Minimize ground deformation due to ground deformation;
d) Performance (closure and thickness) of the frozen soil can be checked prior to the excavation works;
e) It is an environmental friendly method and the ground will not be occupied by other material (e.g. grout) after the works;

Disadvantage/ Limitation:

a) Ground freezing is a relatively expensive method for excavation;
b) Can only be used in region under groundwater table;
c) Ground heave during freezing and settlement during settlement may occur; especially when ground freezing in low permeability soil;
d) Special laboratory testing equipment are required for frozen soil sample testing;
e) Frozen soil is a creeping material; its properties will be deteriorated with time under stress;
f) Take time to freeze the soil and the freezing time is controlled by the soil thermal properties; salinity and flow of groundwater;

g) Working area is required for the refrigerator;

Ground freezing method for tunnel excavation has been adopted in previous Hong Kong following infrastructure projects.

a) Habour Area Treatment Scheme (HATS) Stage 1, Contract DC/93/10 (Pakianathan L. et al, 2002a & b)
b) Lok Ma Chau Sheung Shui to Chau Tau Tunnel, Contract LDB 201 (Storry R.B. et al, 2006)
c) HATS Stage 2A, Contract DC/2009/05 Pumping Station, Stonecutters Island Sewage Treatment Works and Connecting Tunnel (Leung, R.K.Y et al, 2012 and Tsang, L., et al, 2012)
d) MTRC, WIL, Sheung Wan to Sai Yin Pun Tunnels, Contract No. 703 (Polycarpe. S. et al, 2012)

In terms of excavation diameter and length of adit, the SYP Passenger Adit B3 excavation is the largest and longest adit excavated by ground freezing method in Hong Kong up to present history.

This paper concentrates on the design methodology for the SYP passenger Adit B3-1 which includes justification of the creep parameters of the frozen soil.

2 SITE GEOLOGY

Referring to the geological map in Figure 4, the majority of the SYP Passenger Adit B3-1 is located in medium grained Kowloon Granite while the northernmost part of the adit (i.e. portion adjacent to SYP Entrance B1) is found in beach deposit. Site specific ground investigation works were carried out and the adit was found to be in completely decomposed granite (CDG); Fill and Colluvium layer can be found above the CDG layer. A corestone layer is in between the CDG and the granite rockhead, which is expected to vary between -33mPD and -50mPD. The longitudinal geological profile of SYP Passenger Adit B3-1 can refer to Figure 5.

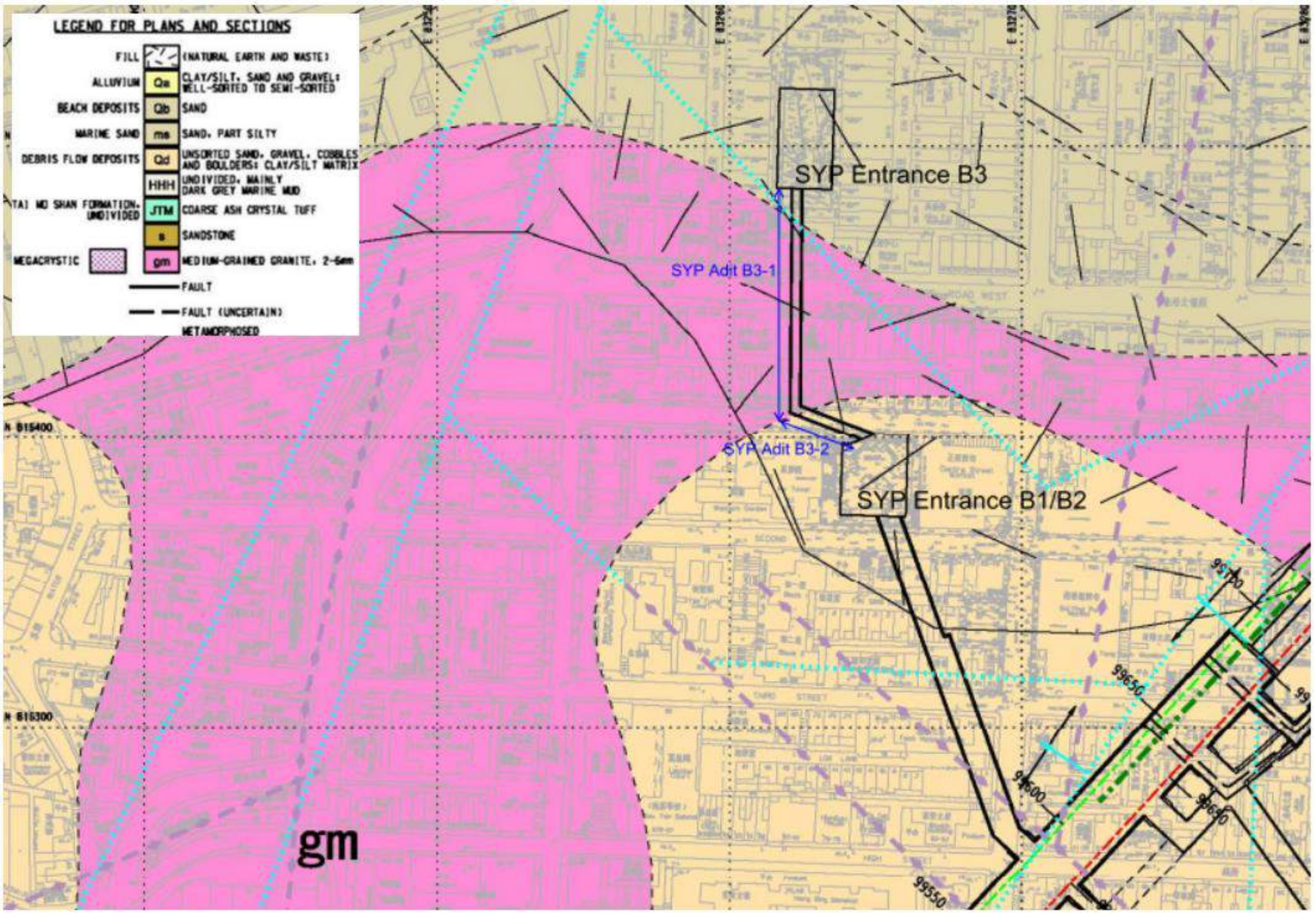

Figure 4 Geological map in SYP Adit B3 area

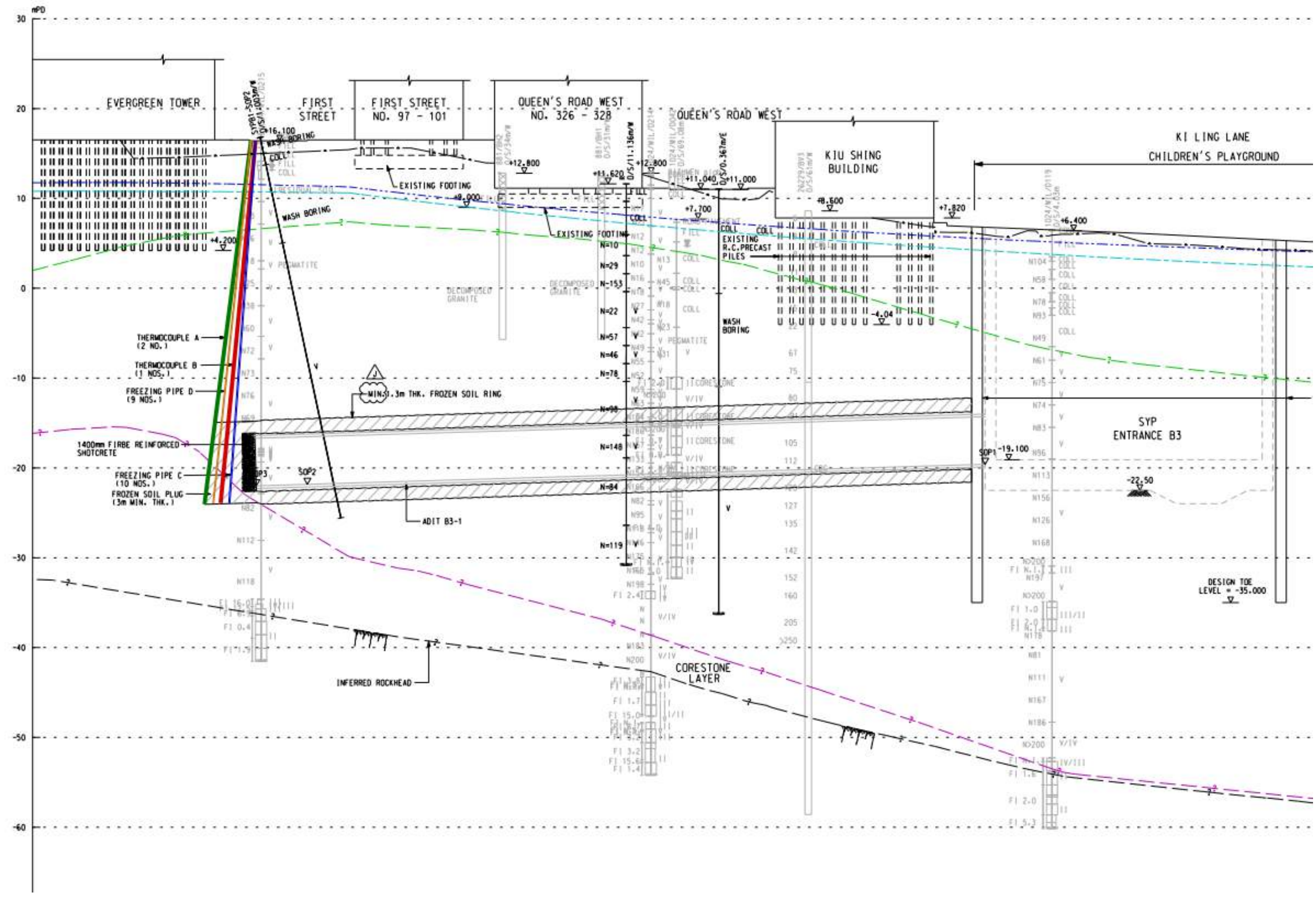

Figure 5 Geological map in SYP Adit B3 area

3 GROUNDWATER CONDITIONS

A long-term groundwater monitoring scheme was conducted throughout the WIL detailed design stage (2005) and the construction stage (2009). The monitoring date indicated that the groundwater level ranges from 3.7 m to 8.0 m below the existing ground level at First Street and from 2.5m to 6.5m below existing ground level at Queen's Road West. The groundwater was always above the proposed ground freezing zone.

4 MATERIAL PROPERTIES OF THE FROZEN SOIL

4.1 *Physical Properties of Unfrozen Soil*

The frozen soil is mainly composed of Completely Decomposed Granite (CDG). The particle size distribution curve in Figure 6 below indicated the CDG was sandy material with about 8% particles in clay size. The bulk density, dry density, moisture content and the porosity of the CDG at the Adit's level were about 19.5kN/m^3, 16.2kN/m^3, 20% and 0.37 respectively. The salinity of the CDG was also examined as it would affect the frozen process. The pH, NaCl content and salinity of the CDG are 7.9, 0.4g/l and 1.5g/l (i.e. 0.15%) respectively.

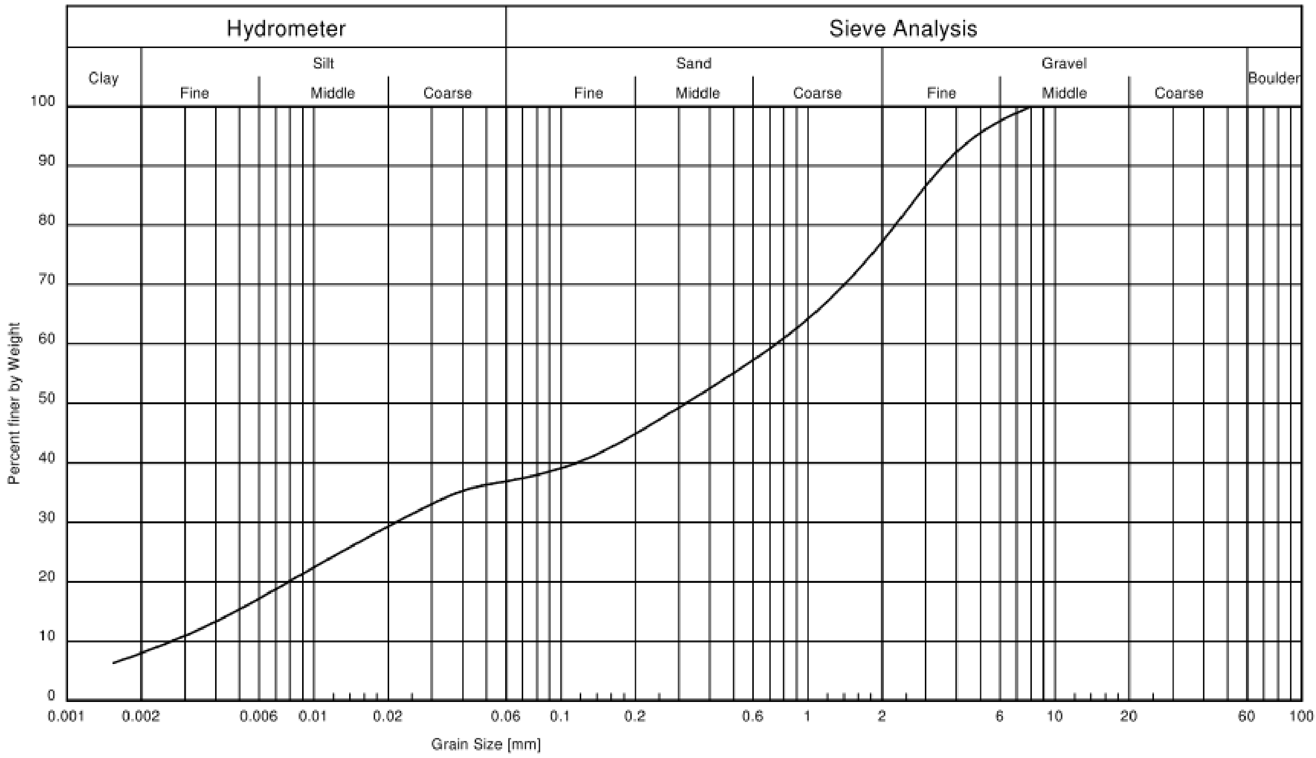

Figure 6 Particle size distribution curve of the CDG

Thermal properties of the frozen and unfrozen CDG at the Adit level were also elevated and summarized in Table 1.

Table 1 Thermal properties of the CDG and frozen CDG

	Thermal Conductivity (W/m^{o}C)	Heat Capacity (MJ/m^{3o}C)	Latent heat (MJ/m^3)
Unfrozen Soil	2.74	2.928	-
Frozen	4.56	2.161	141.3

4.2 *Strength of Frozen Soil*

As to determine the properties of CDG and frozen CDG, and their thermal properties; mazier samples at the CDG layer are collected from the boreholes adjacent to SYP Passenger Adit B3-1 for laboratory testing. The following tests are carried out to determine the frozen CDG properties.

a) Unconfined compression tests (with different frozen temperature);
b) Unconfined compression tests (with different strain rate); and
c) Creep test (i.e. unconfined compression tests at constant stress level)

The results of the unconfined compression tests with frozen temperature (i.e. -5^{o}C, -10^{o}C and -20^{o}C) are presented in Figure 7. The stress-strain graph reveals that the lower the freezing temperature, the high the peak strength and the stiffness of the frozen CDG samples. The failure mode of the frozen CDG sample changes from brittle to ductile with decreasing the frozen temperature.

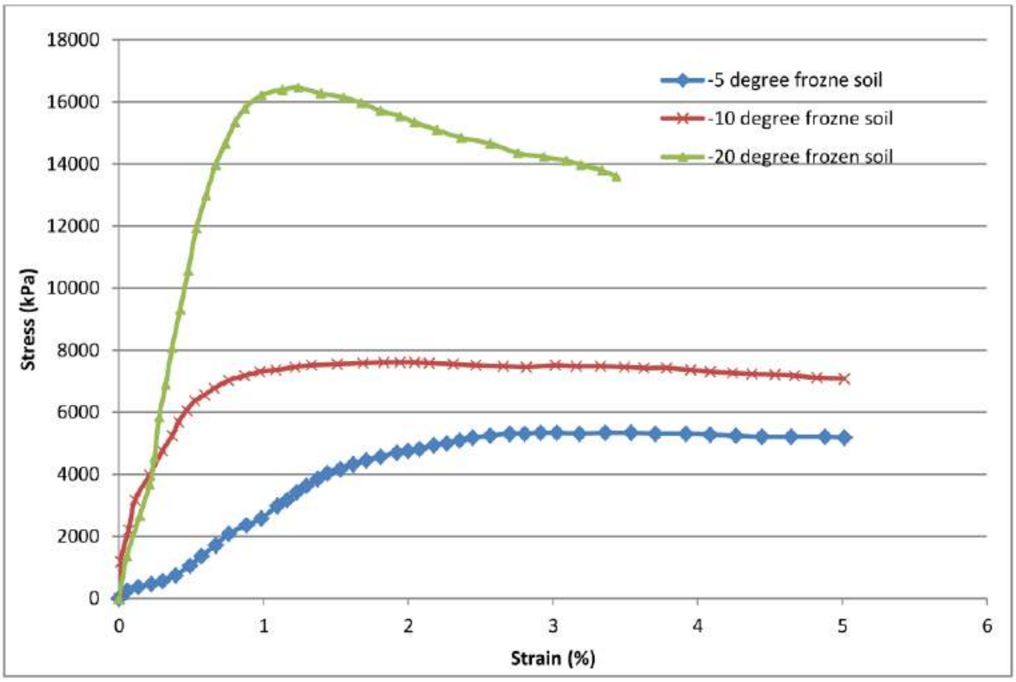

Figure 7 Unconfined compression test of frozen soil sample with different frozen temperature

It is considered that frozen CDG sample at -10°C is suitable to act as a temporary support for the captioned case in terms of its relatively high strength and stiffness, ductile failure mode and also cost and program for freezing the soil to the target temperature.

The failure strain of the CDG at -10°C is required as to determine creep strength of the frozen CDG (i.e. relationship between strength and time), which will be discussed in Section 4.3. The failure strains of the frozen samples with different compressive/strain rates (i.e. 1%, 0.1%, 0.01% and 0.05% strain rate) in unconfined compression conditions are determined. The results of the unconfined compression tests with different strain rates are presented in Figure 8. It can be observed that the faster the strain rate, the higher the peak strength and smaller the failure strain; the results of the tests are summarized in Table 2 below.

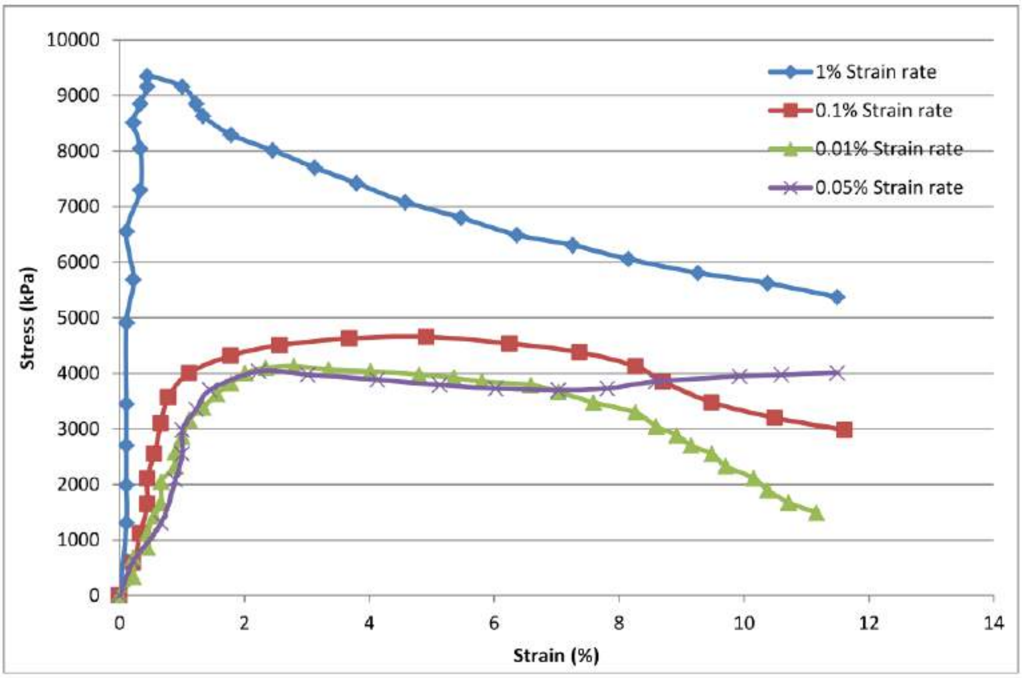

Figure 8 Unconfined compression test of frozen soil sample at -10°C with different compression/ strain rate

Table 2 Summary of the unconfined compression test for frozen CDG at -10°C with different compression/ strain rate

	Peak strength (kPa)	Failure strain (%)
-10° frozen CDG (1% strain rate)	~9500kPa	~0.5%
-10° frozen CDG (0.1% strain rate)	~4900kPa	~4%
-10° frozen CDG (0.1% strain rate)	~4000kPa	~4%
-10° frozen CDG (0.05% strain rate)	~4000kPa	~4%

The creeping behaviour of the frozen CDG sample at -10°C under sustained load are studied by creep test. In the tests, constant axial loads (1290kPa, 1720kPa, 2150kPa, 2580kPa, 3010kPa, 3440kPa, 3870kPa) are applied on the frozen sample in unconfined condition; the strain of the sample is measured with time. The results of the creep tests are presented in Figure 9. The results indicate that the sample deformations increase significantly with time if the axial load is greater than 3010kPa; and the creeping behaviour becomes less significant if the axial load is less than 2150kPa.

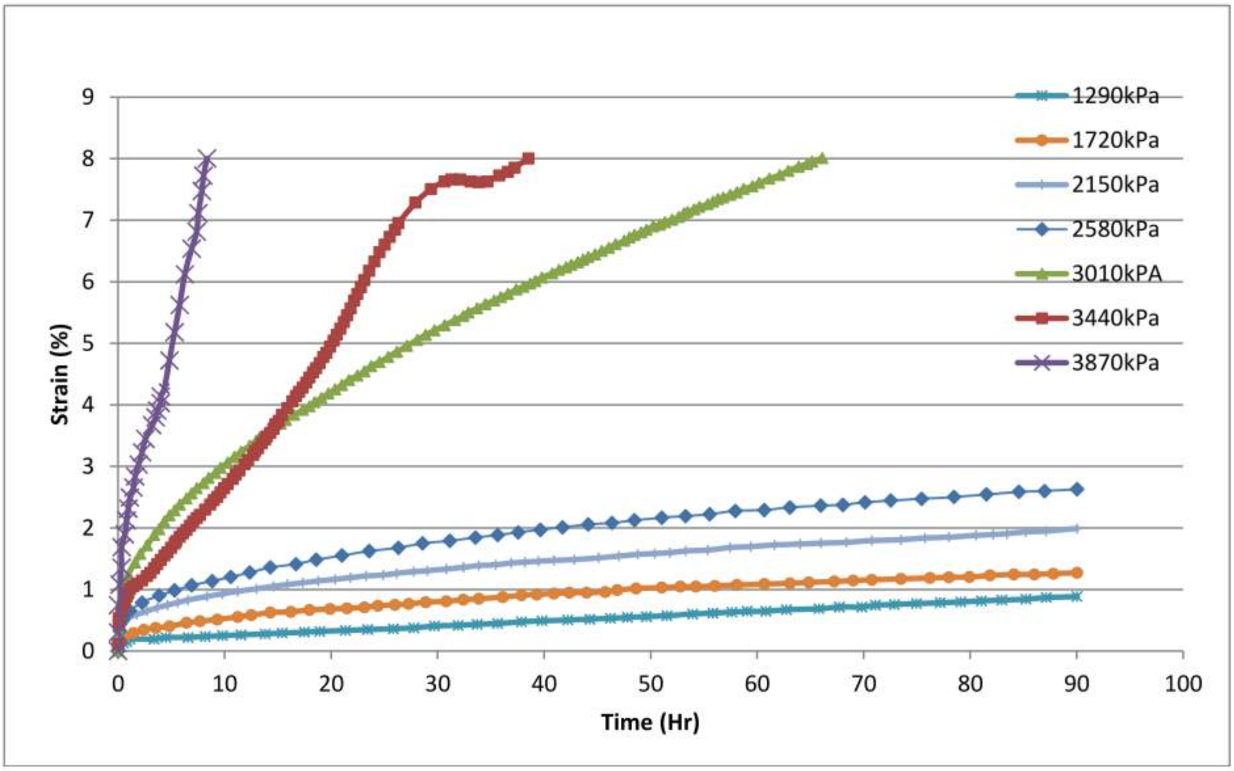

Figure 9 Creep test of frozen soil sample at -10°C with different loadings

4.3 *Creep Strength of Frozen Soil*

Referring to the creep test results, even the sustained load on the frozen soil sample is less than 2150 kPa, the sample deformations increase with time, i.e. creeping occur; hence, the time-dependent material behaviour need to be elevated for designing the geometry of the frozen soil temporary support and determine the construction sequence.

The idealised creep curve of the frozen soils is shown in Figure 10. The creep behaviour consists of three stages: primary (strain-hardening) with strain rate decreasing with time, secondary (linear) with strain rate reaching a minimum at a particular time (point of inflexion), and tertiary (strain-softening) with strain rate increasing with time. For the purpose of engineering design, it is considered that failure of a frozen soil occurs at a minimum strain rate, i.e. the inflexion point in secondary stage.

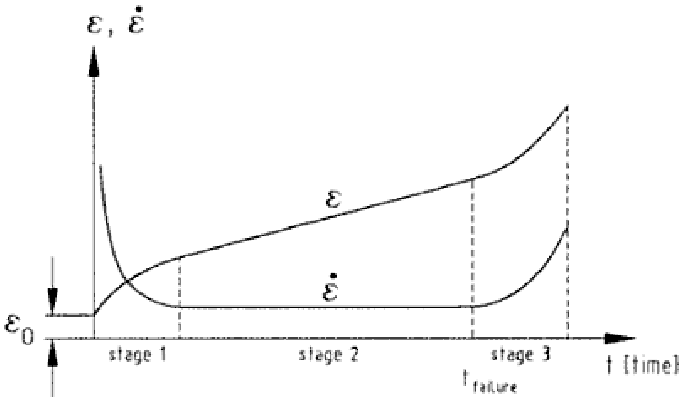

Figure 10 Idealised creep curve of the frozen soil

The distinct non-linear creep behaviour of frozen soil can be explicitly expressed in a formula. This formula is suitable to describe the creep behaviour of frozen soil obtained by the laboratory testing. The equation describes the creep behaviour of frozen soil by a potential law (Jagow-Klaff & Jessberger, 2001) is shown in follow:

$$\varepsilon = \sigma_1/E_0 + A \times \sigma_1^{B} \times t^{C} \quad \text{(Equation 1)}$$

where A, B and C are creep test parameters,
E_0 is initial Young's modulus,
σ_1 is constant axial stress and t is time.

The creep modulus A and exponents B and C for the frozen CDG at -10°C can be determined from the creep tests. Referring to the creep test results with constant load less than 2580kPa, A = 6.8E-4 $(m^2/MN)^B h^{-C}$; B = 2.090 and C=0.380 are determined. Creep formula with these creep modulus are used to determine the time-dependent time behaviour of the frozen CDG at -10°C with axial load smaller than 2580kPa. The curves calculate by creep formal and the creep curves obtain from creep tests under axial load 2580kPa are quite consistent with each other as shown in Figure 11.

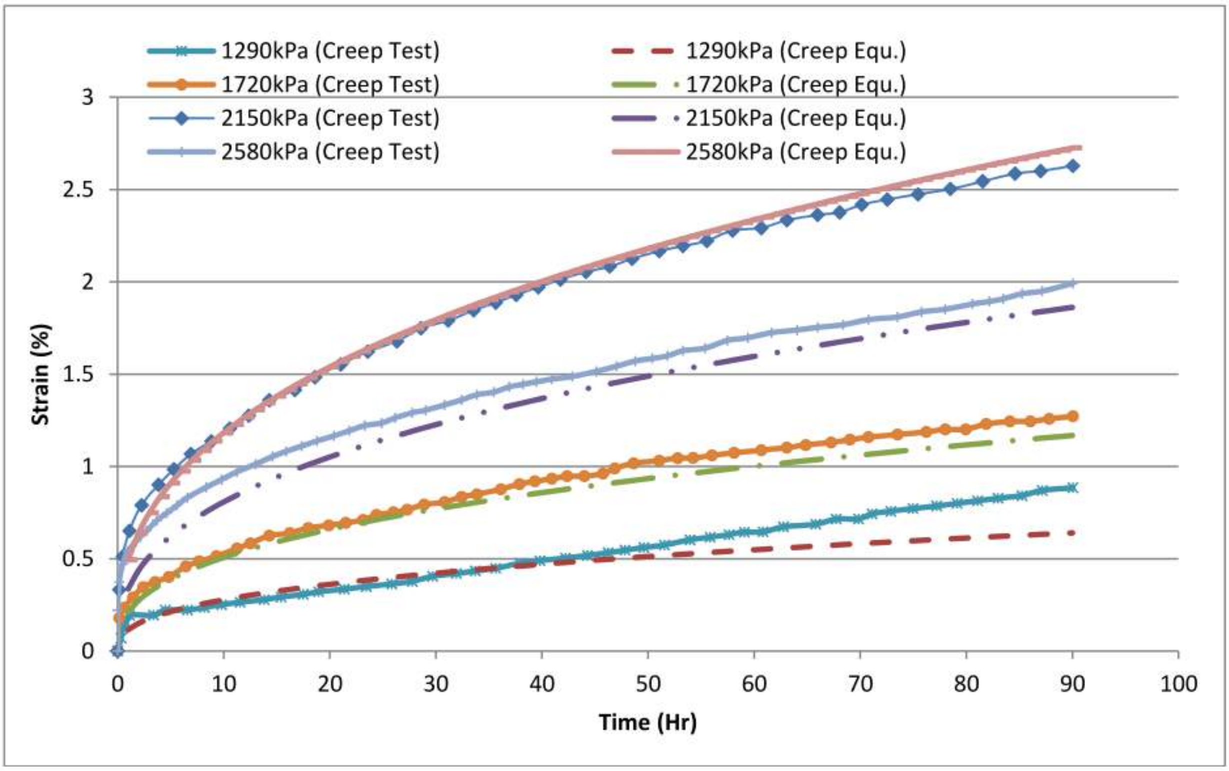

Figure 11 Creep curves of frozen soil from lab tests and creep formula

Rearranging Equation 1, the time dependent Young Modulus $E_g(t)$ can be defined by the following equation:

$$E_g(t) = [\varepsilon^{(1-B)} / (A \times t^C)]^{(1/B)} \qquad \text{(Equation 2)}$$

Time dependent stiffness of the frozen soil is defined as Secant Modulus at 0.35% strain.

Further rearranging Equation 2, time dependent allowable stress $\sigma_d(t)$ can be defined by following equation.

$$\sigma_d(t) = [\varepsilon_f / (A \times t^C)]^{(1/B))} \qquad \text{(Equation 3)}$$

where ε_f is the failure strain

Referring to the Table 2, it is noted that the ductile behaviour of the frozen soil samples was observed with peak strength occurring at ~ 4% strain in uniaxial compression test with compression/ strain rate less than 0.1% per min; from the creep equation (Equation 1) and creep test results as shown in Figure 11, the stain rate under the stress level less than 2580kPa is much smaller than 0.1% per min; thus the failure strain of the frozen CDG at -10^{o}C with stress level less than 2580kPa can be assumed at 4.0%.

The time dependent cohesion $c_g(t)$ is calculated based on Mohr-Coulomb failure criterion in consideration of the allowable compression stress, as follow:

$$c_g(t) = [\sigma_d(t) \times (1 - \sin \phi)] / (2 \times \cos \phi) \qquad \text{(Equation 4)}$$

where ϕ is the friction angle of the frozen CDG at -10^{o}C; ϕ is considered as time independent and defined through triaxial test.

The time-dependent frozen CDG parameter at -10^{o}C determined based on Equation 2 to Equation 4 are summarized in Table 3.

Table 3 Time-dependent frozen CDG parameter at -10^{o}C

Uniaxial compressive strength (q) MN/m^2	Friction Angle (ϕ)$^\circ$	Young Modulus ($E_g(t)$) MN/m^2					Allowable Compression Stress ($\sigma_d(t)$) MN/m^2					Cohesion ($c_g(t)$) MN/m^2				
		1d	1w	2w	6w	3m	1d	1w	2w	6w	3m	1d	1w	2w	6w	3m
8.6	13.3	351	246	217	178	155	1.97	1.38	1.22	1.00	0.87	0.78	0.55	0.48	0.40	0.34

5 FROZEN SOIL RING AS TEMPORARY SUPPORT FOR TUNNEL EXCAVATION

5.1 *Set-up of Temporary Support System for SYP Adit B3-1*

By elevating the above frozen soil parameters, actual site conditions, construction program and construction cost factors; a 1.3m thick frozen soil ring with average temperature -10^{o}C is proposed around the SYP Adit B3-1 as a pre-support system for the adit excavation prior to the steel rib and shotcrete composite lining installation; two-week frozen CDG parameters are adopted in the design; the overburden pressure above the adit crown would be supported by the steel rib & shotcrete composite lining only within two weeks after the excavation works. In order to get the required thickness of frozen soil ring in reasonable time, freezing pipes at maximum 1.0m spacing and brine solution at -25^{o}C was used as a freezing agent; the setting up of the freezing pipes can refer to Figure 14.

Numerical analysis was carried out to study the stress levels in the frozen soil ring during the adit excavation; the stress level inside the ring should be less than two weeks strength of the frozen CDG sample as specified in Table 3. The input soil parameters for unfrozen soil and steel rib & steel rib composite lining are listed in Table 4 below. The graphic numerical analysis input for a critical section is presented in Figure 12.

Table 4 Soil parameters for unfrozen soil

Geo- parameters	Unit	FILL	COLL.	CDG
γ	kN/m^3	19.0	20.5	19.0
Porosity	-	0.28	0.23	0.39
SPT 'N'	-	10	20	79
E'	MPa	1.0 × N	1.0 × N	1.4 × N
n'	-	0.3	0.3	0.3
ϕ'	deg	35	37	39
c	kPa	0.0	0.0	5.0

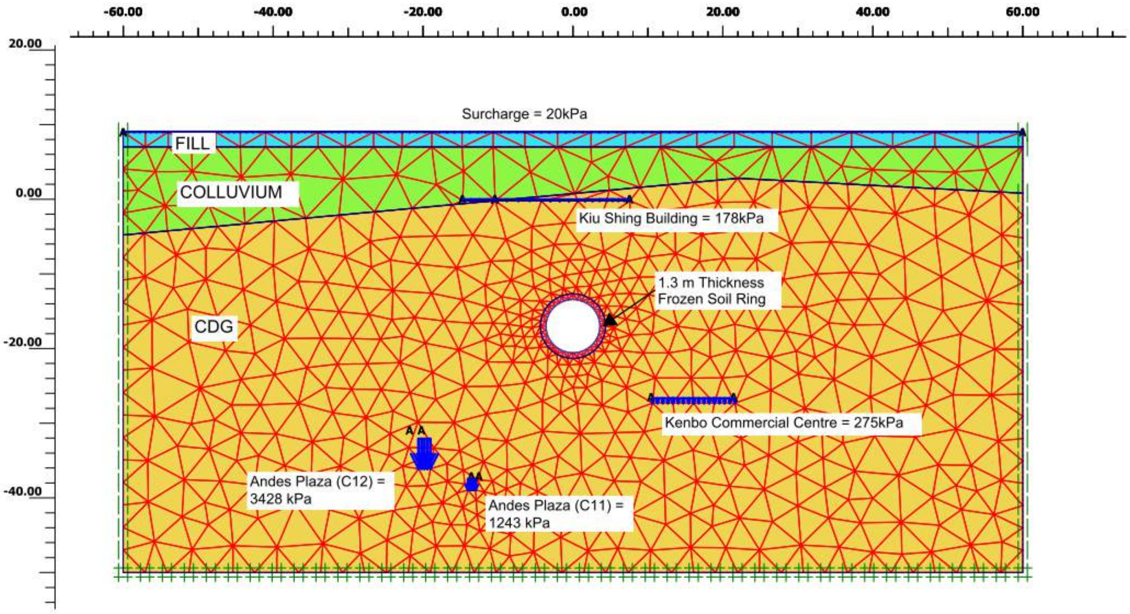

Figure 12 Numerical analysis input (Plaxis) for critical section

Stress levels inside the frozen soil ring from the numerical analysis are presented in Figure 13. It is noted that the stress concentrated at the spring line level inside the frozen soil ring and the magnitude (i.e. ~1100kPa) is less than the 2 weeks strength of the frozen CDG (i.e. 1220kPa).

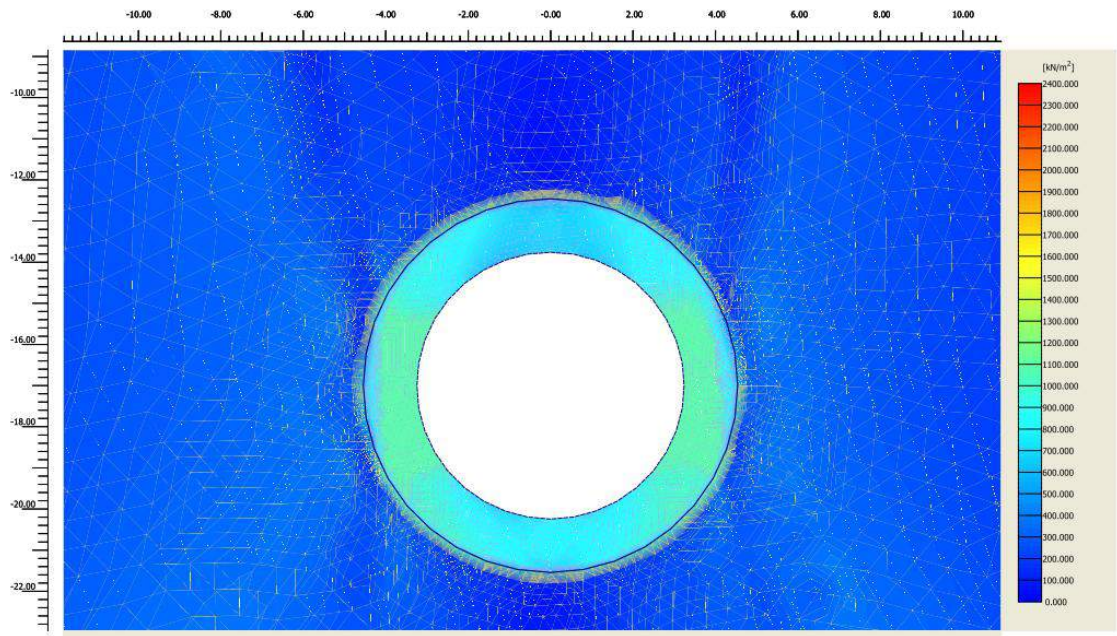

Figure 13 Stress levels in frozen soil ring from numerical analysis

It was considered that at 2-week time after the excavation works, the allowable strength of the frozen soil ring would decrease to a value that no longer can support the overburden pressure due to the creeping effect. Steel rib & shotcrete composite lining will be installed shortly after the excavation works. 1-week shotcrete strength was adopted in the steel rib & shotcrete composite lining design; with this design assumption, the composite lining need to be installed no later than 1 week after each portion of excavation; for example, full ring composite lining in the 3rd cycle zone in Figure 15 should be completed within 1 week after the commencement of excavation in 3rd cycle zone. 203 x 203 x 46kg/m UC section (Grade S275) at 1m centre to centre spacing together with 250mm fibre reinforced shotcrete is adopted for the composite lining.

The predict ground settlement due to the adit excavation works can refer to Figure 16. The settlement is solely due to the deformation of the temporary support system, i.e. the frozen soil ring, and the maximum predict ground settlement is about 21mm. It is relatively small for soft ground tunnel excavation by traditional methods.

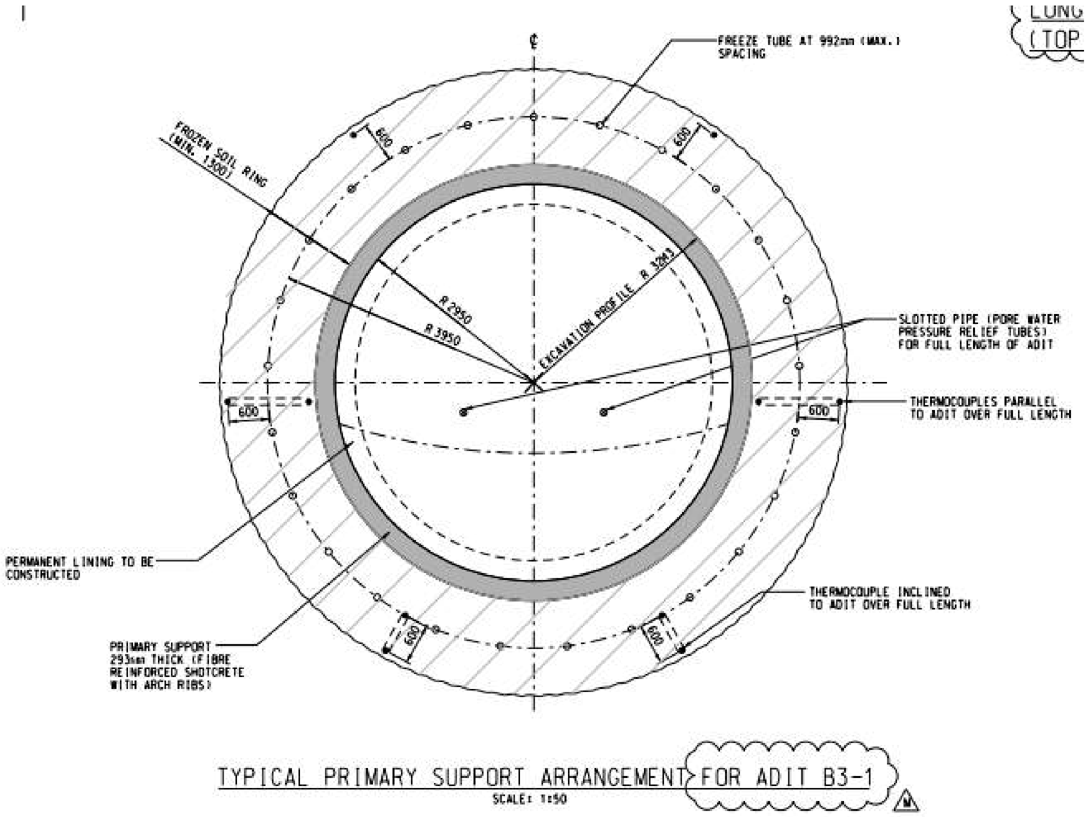

Figure 14 Typical temporary support arrangement from SYP Adit B3-1

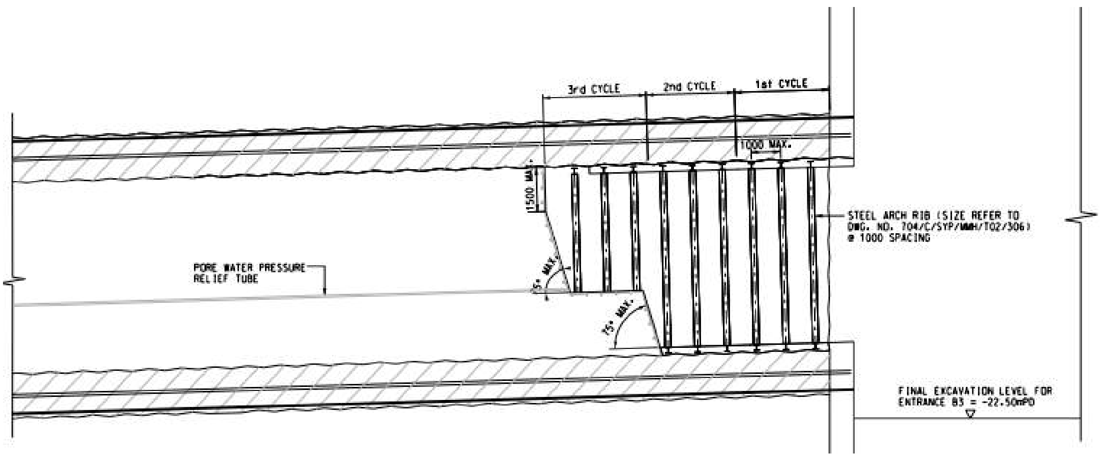

Figure 15 Construction sequence for SYP Adit B3-1

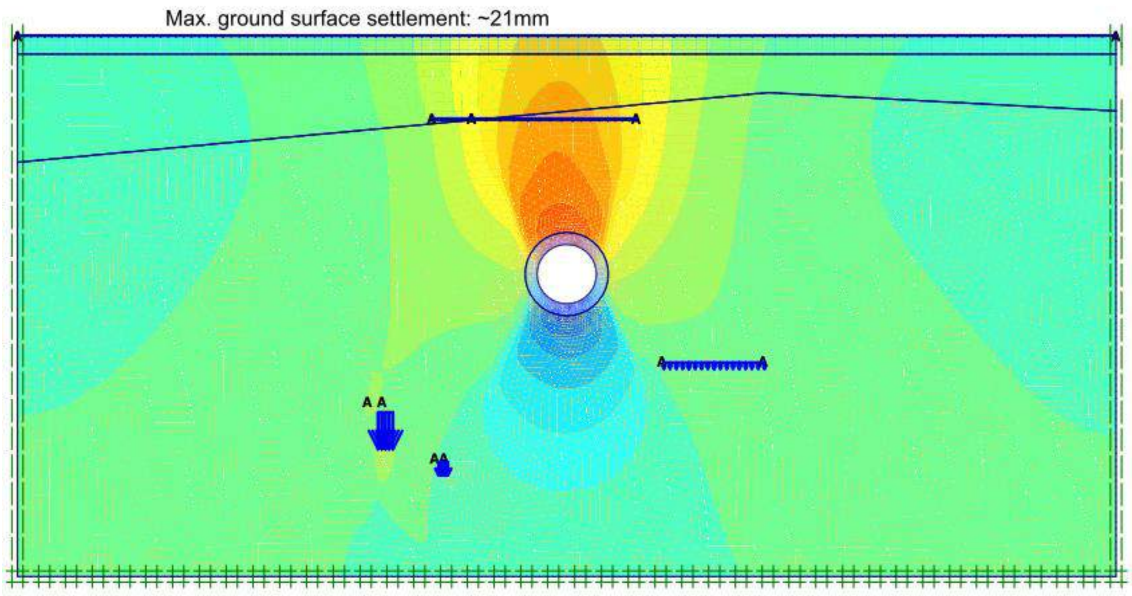

Figure 16 Settlement contour associate with the ground freezing excavation works

6 MONITORING FOR THE FROZEN SOIL RING

An intensive monitoring scheme is proposed to ensure the closure and the thickness of the frozen soil ring prior to the commencement and during the excavation. Before the commencement of the excavation, pore pressure in the pressure relief tubes as shown in Figure 14 were measured, the increase in pore water pressure indicated the closure of the frozen soil rings. Six numbers of thermocouples, in which 4 numbers run parallel to the adit alignment and 2 numbers cut across the frozen soil layer, as shown in Figure 14 are used to ensure the thickness of the frozen soil ring throughout the excavations works; real time monitoring for the thermocouples would be carried out. A back-up freezing plant is ready on site for emergency cases.

In addition, tunnel convergence array and strain gauge are proposed to place inside the tunnel and steel rib respectively to monitor its deformation with times and hence the creeping behaviour of the frozen soil; piezometers, ground and building settlement/lateral movement/tilting markers and Automatic Deformation Monitoring System (ADMS) for real time building movement monitoring are also proposed to monitor the ground response due to the excavation works.

7 CONCLUSION

Creeping effect of frozen soil under loading is the major design consideration in this paper; a series of laboratory tests for frozen soils in unconfined condition were carried out to study the creeping parameters of the frozen soil at -10^{o}C and time relationship for the material properties. The arrangement of the frozen soil ring, steel rib and shotcrete composite lining and the construction sequence were designed based on those laboratory testing results and numerical analysis.

The excavation works for SYP Adit B3-1 was finished in March 2014; all the monitoring records, including the convergence array inside the adit, ground and building settlement markers above the adit, indicate no adverse ground and buildings movement with the excavation works; in addition, the piezometer records showed that the groundwater table maintained in the steady level. The successful construction of SYP Adit B3-1 presented that the ground freezing method could be used to as a pre-support system for a relatively long (~80.0 m) and large span (excavation diameter ~6.3 m) adit in sensitivity area with high ground water table (~22.0 m above the adit crown), overburden pressure (~30.0 m) and aged buildings with shallow foundations or driven piles.

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Innovative design idea and low carbon construction – Happy Valley Underground Stormwater Storage Scheme

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Keywords: cost savings design; cut-off walls; CO_2 emission; movable overflow weir; water harvesting system.

ABSTRACT: In the midst of different expert views on whether radical actions are required on global warming, low carbon construction has momentously become a global trend. Sustainability on the design and construction of built environments has been encouraged by professional institutions and governments all over the world via policy setting and perhaps even financial support. This paper explains an innovative design idea realised through the Cost Savings Design mechanism for a Drainage Services Department's project in Happy Valley, Hong Kong. The innovative idea enabled mitigation of some 533 rock socketted H-piles. In addition to the saving in construction time and cost, this solution minimised adverse impacts on the neighbourhood, and avoided emission of 8,043 ton of CO_2-e which is similar to the effects of planting 205,938 trees in Hong Kong. This, coupled with other innovative features of the project, makes the Happy Valley Underground Stormwater Storage Scheme one of the substantial low-carbon and green construction projects in Hong Kong. This project achieved Platinum rating of Provisional Assessment under the BEAM Plus V1.1 for new buildings in January 2014.

1 INTRODUCTION

In Hong Kong, severe flooding incidents at Happy Valley and the adjacent areas occurred during major rainstorms in previous years. A long-term solution proposed by the Drainage Services Department (DSD) of the Hong Kong Special Administrative Region (HKSAR) entailed construction of an underground storage tank, storing temporarily part of the stormwater collected from the upstream catchment for attenuating the peak flow through the downstream stormwater drainage systems after heavy rainstorms. This scheme, known as Happy Valley Underground Stormwater Storage Scheme (HVUSSS), greatly reduces the risk of flooding to the low-lying areas of Happy Valley and Wan Chai District.

Chun Wo Construction & Engineering Company Limited (Chun Wo) has been commissioned by the DSD to undertake the construction works for the HVUSSS under an Engineering & Construction Contract (Contract No. DC/2012/03). Following the commencement of works, Chun Wo proposed an innovative Cost Savings Design (CSD) for the foundation of the underground storage tank in which 533 numbers of pre-bored H-piles up to 55m in length socketted into bedrock have been mitigated. This, coupled with other innovative features of the project, makes the HVUSSS one of the substantial low-carbon and green construction projects in Hong Kong, reducing the adverse effects owing to global warming.

2 SCOPE OF WORKS

The scope of HVUSSS comprises construction of an inlet structure, a twin-cell box culvert with overflow weirs and controlling penstocks, an underground storage tank and an integrated pump house under the Happy Valley Recreation Ground with minimum capacity of 60,000m^3 to receive overflow during severe rainfall events. The civil and structural works being constructed include:

a) Construction of an underground stormwater storage tank with a capacity of 60,000m^3 and a pump house at Happy Valley Recreation Ground;
b) Construction of about 400m long twin-cell (4,000mm×2,000mm) box culvert; and
c) Associated works including modification of an existing box culvert and construction of a stilling basin, a fan room, an access manhole, drainage and sewer diversion works.

The construction works are separated into phases, meeting the needs of the users of the existing Happy Valley Recreation Ground. In Phase 1, the construction works (in pink in Figure 1) include about half of the underground stormwater storage tank, a stilling basin, twin drainage pipes, a box culvert, a pump house and a fan room. Thereafter, sport pitches nos. 8, 2, 3 and 4 will be re-turfed sequentially, whereas in Phase 2 the remaining portion of the underground storage tank (in purple in Figure 1) will be constructed, followed by re-turfing at sport pitches nos. 6 and 12.

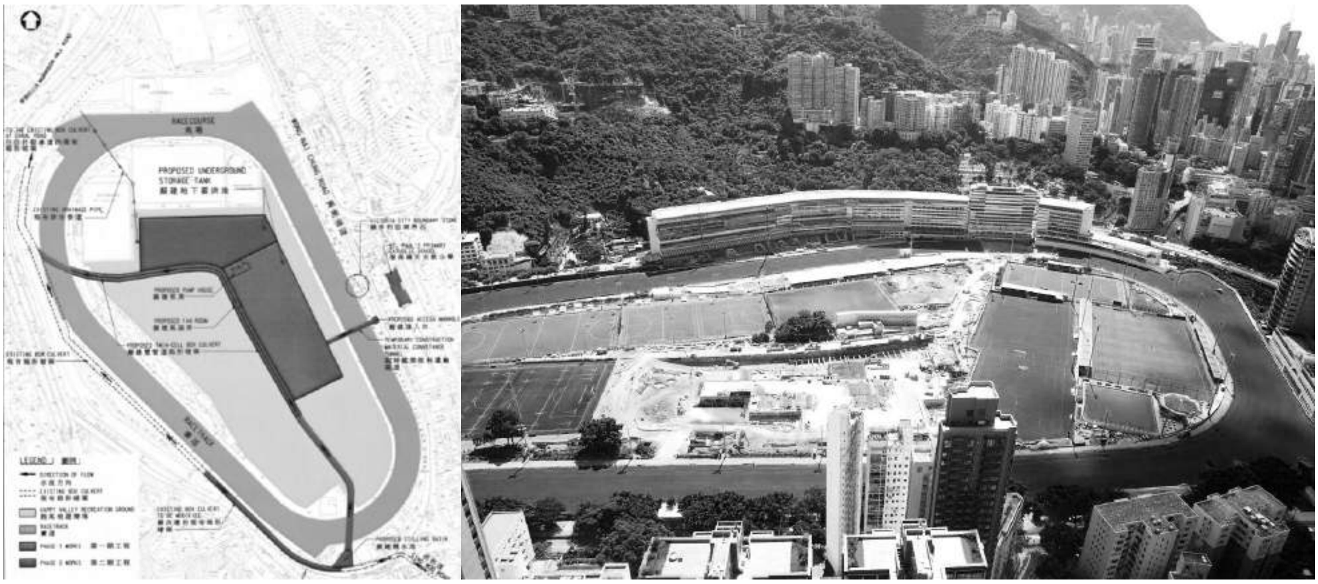

Figure 1 Working plan and arrangement of HVUSSS and bird eye view of the Happy Valley Recreation Ground during construction

3 CONFORMING DESIGN OF FOUNDATIONS

3.1 *General Arrangement*

The structure of the underground stormwater storage tank and the integrated pump house is L-shaped on plan, 58.5m to 69.5m wide, 161.1m and 231.5m along the outer edges of the legs. The invert of the tank varies from 1.10mPD at the outer corners around the pump house to –0.9mPD near the midpoint.

The contract drawings indicate that the underground storage tank and pumping station are to be supported by 533 nos. of pre-bored steel H piles (Grade S460 UC 305×305×180 and 305×305×223 kg/m, lengths from 40 to 60m) socketted into rock, sustaining compressive loads from 5045kN to 7354kN and tension loads from –431kN to –2457kN.

The dead weight of the underground storage tank is lower than the original soil mass replaced by the tank and the tank is thus subject to upthrust from the groundwater. Piles in the Conforming

Design are used to hold the storage tank down when it is empty in most of its operation life. The conforming piling layout is shown in Figure 2.

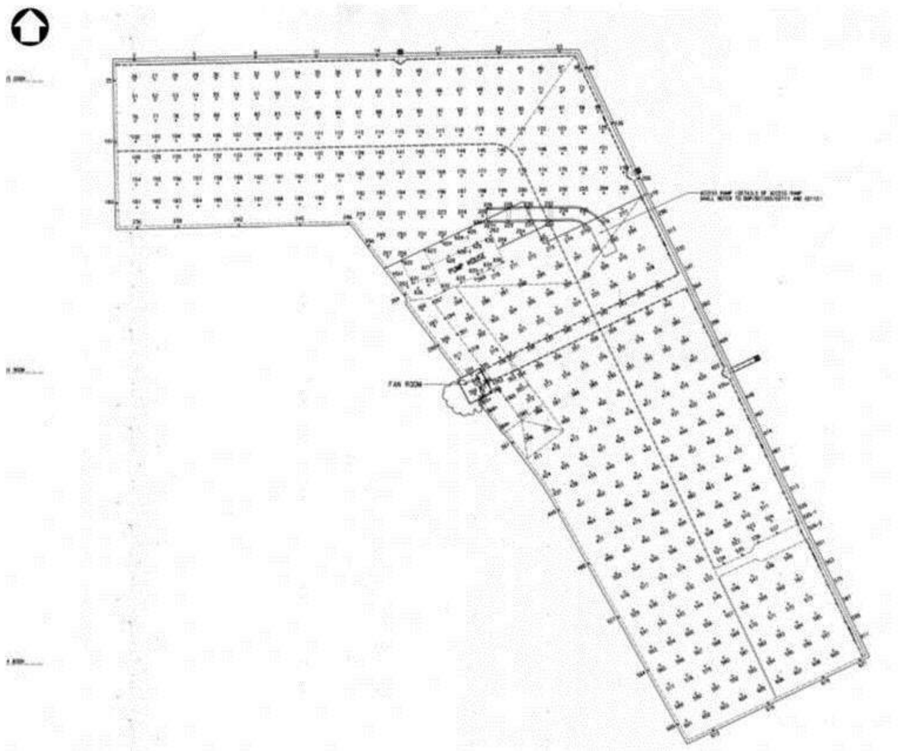

Figure 2 Conforming piling layout plan of the underground stormwater storage tank

3.2 *Construction Issues / Constraints*

Typical installation cycle for pre-bored H-piles comprises of drilling of hole, inserting H piles segments, splicing of H-piles by welding, testing of welds and grouting. The construction works are particularly prone to delays owing to the following issues and constraints.

3.2.1 *No obstruction to viewing of horse racing*
The horse field must remain operational during construction. The horse racing season runs in the dry season from September to July, when drilling works will be performed. All plants and un-spliced piles will have to be positioned and lowered before the racing in order not to obstruct the viewing of the racing.

3.2.2 *Potential impact on construction programme*
As mentioned above, construction works must be performed with minimal impact to operation of the horse field. Drilling operations and mobilization of drilling equipment will be limited to only the off hours each day. In addition, there is a risk of insufficient piling rigs in view of the constrained construction programme and the large amount of pile driving works involved.

3.2.3 *Risk of settlement owing to piling*
Pre-bored holes for H-piles are drilled using down-the-hole drills. During this drilling process, compressed air is injected to drive the drill bit and to expel the excavated materials out to the air at ground level. However, the compressed air may cause blow out and caving-in of weak layer forming voids around the drillhole. As a result, settlement of nearby ground may occur due to subsidence of ground towards the voids.

While settlement during construction is always undesirable, the adverse effect of settlement to this project would be particularly significant due to the potential danger presented to field users including jockeys and horses. Settlement may lead to potholes on the horse field, which is located adjacent the construction site. The uneven ground would be an unsafe environment for horse racing as horses may stumble during the race. Horse races would unavoidably be postponed or cancelled.

4 COST SAVINGS DESIGN OF FOUNDATIONS

In view of the constraints listed in Section 3.2 above and taken account of the need to counter the floating loading, several design ideas have been considered and the selected solution is explained Sections 4.1 to 4.5 below.

a) Bored piles: This design enhancement uses bored piles of 1.5m to 2m diameter to replace the H-piles. Preliminary assessment indicated that the number of could be reduced to about 150 numbers. Still, the relative long cycle of installation might cause overall delay on the foundation programme.
b) Mini-piles: This design enhancement entailed groups of 4 to 6 mini-pile to replace each H-pile. In comparison with H-piles, mini-piles installation employs lighter plants and no welding tests are required. However the large number of mini-piles to be required could offer small saving in time for the construction.
c) Ballasted Raft: This design enhancement employed thickened base slabs (and side walls) to counter act the upthrust from the groundwater table. Preliminary estimates indicated that the bases slab may need to be increased to 3.5m thick for inverts at –1.1mPD to provide the necessary dead weight to provide a FOS of exceeding 1.1 as required under Code of Practice for Foundation 2004. The deeper excavation and larger amount of excavated materials generated are undesirable in view of the limited access available for the Happy Valley site.

4.1 *Innovative Design*

After a series of deliberation with the relevant parties, a CSD of foundation was agreed in that the underground storage tank is supported on a raft foundation surrounded with cut-off walls. A system of sub-soil drains is employed to maintain the groundwater level at just below the foundation slab using small pump sumps. Since the water-table is to be maintained at a low level, there is chance to optimise the thickness of the base slab to further reduce the construction volume and time. The provision of cut-off walls and sub-soil drains will not only resolve the requirements to resist the upthrust in the Conforming design, but also create a favourable externally dry condition at the underground storage tank which can eliminate the groundwater infiltration into the tank in long term. Thus, the common maintenance problem due to water ingress through construction joints can be avoided.

4.2 *Water Cut-Off Walls*

The principal types of cut-off walls for this type of design are either sheet pile walls or diaphragm walls. Their respective merits/ demerits are discussed below.

4.2.1 *Sheet pile walls*

Sheet pile walls are interlocking sections of steel materials and are installed by driving or vibrating interlocking sheet piles into the ground. Since they can be installed without major excavation work, they are less expensive to build. Corrosion rate of sheet piles is minimal in common ground. The average corrosion rate of structural steel above seabed ranges between 0.04 to 0.08 mm/year. Durability of sheet pile walls installed in land with non-corrosive environment is expected to be more than 50 years. Leakage may occur through interlocks but can be prevented by incorporating sealing material that expands when hydrated.

4.2.2 *Diaphragm walls*

Diaphragm walls are usually constructed by slurry trench technique. The subsoils at the project site predominantly consist of alluvial clay and sand layers and are prone to induce large ground movements (due to caving in) which is highly undesirable in view of the stringent restrictions on ground movements under the contract. Diaphragm walls are also more expensive than sheet pile walls. For these reasons, sheet pile wall is chosen over diaphragm wall for the CSD.

4.3 *Provision of Subsoil Drains & Sump Pumps*

Groundwater flow within the confine of the cut-off wall is to be drained by a layer of 600m thick fine gravels with embedded 100 to 150mm diameter perforated HDPE pipes connecting to a pump sump (see Figure 3). A layer of non-woven polypropylene geotextile is placed between the drainage layer and native soil to provide separation and reinforcement. The drained groundwater is to be pumped out with a set of low cost small submersible duty and stand-by pumps. The collected groundwater is also used to supplement the water supply to the irrigation system for the football pitches within the Happy Valley Recreation Ground.

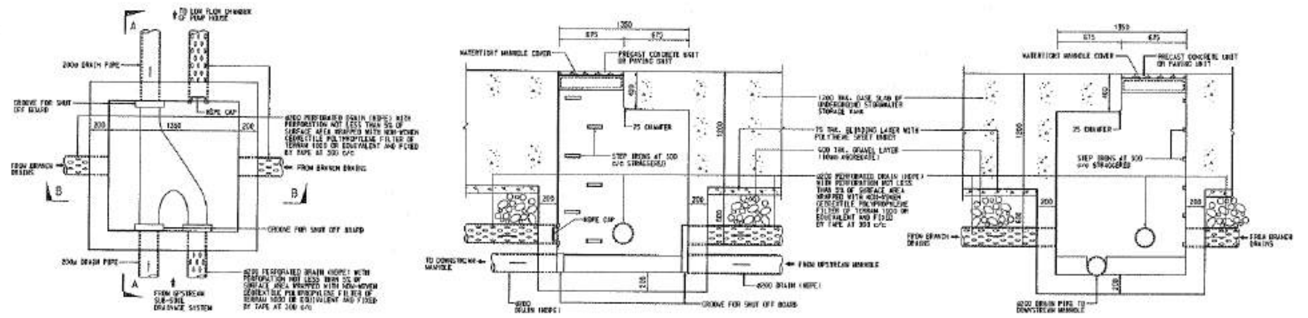

Figure 3 Diagrammatic detail of subsoil drainage system

As a fail-safe measure, a set of relief wells (total of 32) are proposed through the base slab to allow underground water to be drained into the stormwater storage tank in case of pump failures. Each relief well contains a 5m long perforated overflow pipe that embedded underground and opens to the bottom of the tank. If the pumps fail, groundwater can be drained via the flap valves at the top of the wells into the tank to prevent building up of excessive upthrust on the tank.

4.4 *Design Methodology*

4.4.1 *Cut-off and sub-soil drain*

The inflow of groundwater has been assessed using 2-D flow space governed by the Laplace's equation. The computer software SEEP/W is adopted for estimating the groundwater inflow quantities to be used in the design of the sub-soil drainage system. Some analysis results are shown in Figure 4.

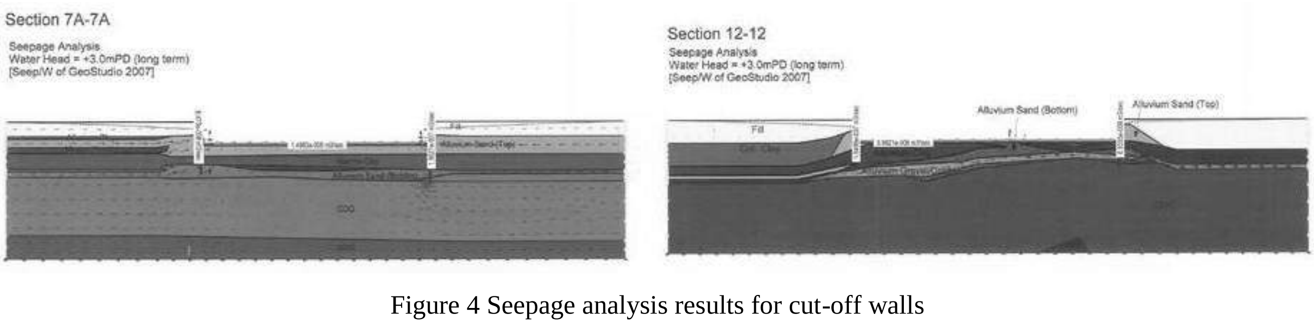

Figure 4 Seepage analysis results for cut-off walls

4.4.2 *Raft foundation stability*

Detailed calculations indicate that the change in contact pressure at the base slab is negative. The ultimate limit states of bearing failure of the raft foundation has been checked and confirmed using factored material parameters and loadings in accordance with Geoguide 1. The checking sliding and overturning failures are considered not necessary in view of the geometry and symmetric loading of the storage tank.

4.4.3 *Ground movement and long-term consolidation settlement*

The estimated foundation pressures are below that of the original in-situ pressures before the construction and will therefore not causing long term consolidation settlements. A detail check has been carried out to confirm the changes of ground stresses for the soils below the raft foundation in various stage of construction and operation of the tank.

The deformation of the soil layers and the tank at various stages of construction has been analysed using appropriate finite element models by PLAXIS to ensure the ground deformation of the vicinity of the excavation will not cause damages to adjacent installations and that no excessive stresses are induced in the storage tank structure. The long term consolidation settlement analyses have been performed taking account of the various timing and loading from the structures to demonstrate that no excessive total and differential settlement would be resulted (see Figure 5).

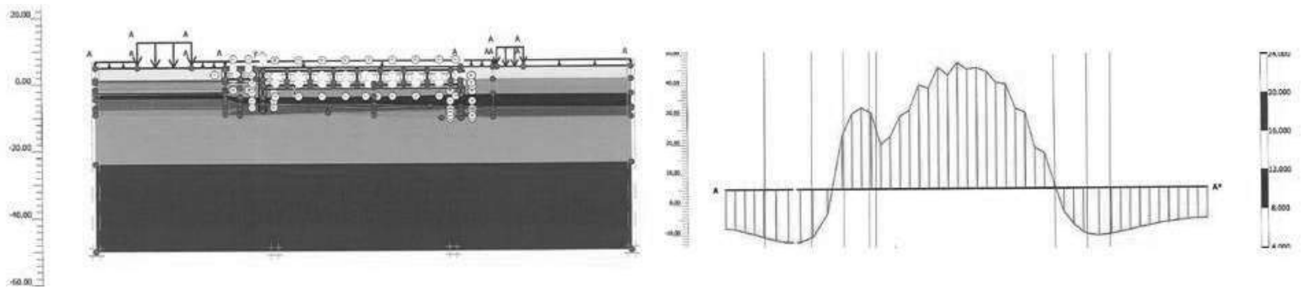

Figure 5 Deformation analysis results for underground stormwater storage tank

4.5 *Whole Life Cost*

Whilst the proposed CSD will decrease the need for future maintenance of the concrete structures of the storage tank and pumping station due to infiltration of groundwater, it will incur operation and maintenance cost to maintain the cut-off wall, sub-soil drainage and pumping system. The whole life cost of the CSD has been studied with respect to the items below, and it is estimated that there is a net cost saving to be gained.

4.5.1 *Underground storage tank*

Under the Conforming scheme, the underground concrete structures will be in contact with groundwater fluctuating with tidal cycles. It is expected that concrete repairing works due to water ingress through construction joints are frequently required. This cost can be avoided in the proposed CSD in which the outer faces of the external walls of the storage tank are kept in a dry condition.

4.5.2 *Submersible pumps*

Under the CSD, additional maintenance of the duty and standby pumps in the groundwater withdrawal chamber is required. The pump sump should be cleaned by either digging the sediment out of the silt traps or by using a vacuum device at least once a month. Sump grates should be clear of rubbish and silt. Carrying out these maintenance works requires approximately 30 man-hours per month. In addition, the pumps will need to be replaced every 10 years or a total of 4 times over the design life. The pump will operate 24 hours a day all year, and it is estimated that the annual power consumption of pump will be 125,000kWh.

4.5.3 *Drainage system*

Under the proposed scheme, additional works are required to maintain efficient operation of the subsoil system. Regular inspections and maintenance of subsoil drains are necessary to prevent blockage by fine sediments and/or debris that infiltrated into the system. Maintenance works required for the sub-soil drainage system include regularly monitoring the discharge rate at the outlets, as well as monthly cleaning of the junction manholes and annual inspection of pipes and manholes. The maintenance includes:

a) Discharge rates monitoring from sub-soil drains by automatic senor at v-notch weirs with monthly manual measurement of v-notch for calibration;
b) Monthly cleaning of junction pipes and manholes;
c) Annual inspection of pipes, by CCTV if needed. Clearing of debris in the HDPE pipes can be carried out by rodding and flushing with air/water jets.

4.5.4 *Monitoring system*

Unusual rises in piezometric heads could be an indication of pump failure or reduced performance of the subsoil drainage system. Therefore, a monitoring system includes piezometers inside and outside the cut-off wall at various locations is necessary to demonstrate that designed groundwater levels within the cut-off walls are maintained. Monitoring works will include regular checking and recording of the underground water level during high and low tides. A monitoring program will be needed to identify monitoring and emergency protocol procedures. Implementation of this monitoring program will include development and adoption of the protocol procedures, as well as trainings for maintenance workers. It is estimated that monitoring and the associated works will require 20 man-hours per month.

4.5.5 *Irrigation system*

According to the requirements in the contract, the rain gun flow rate for sport pitches 2, 3, 4, 6, 8 and 12 is 14 litres/second. At this rate, the estimated volume of irrigation water saved by the CSD is 46,000m^3/year. Further explanation of the irrigation system is given in Section 5.2.

5 GREEN & LOW CARBON CONSTRUCTION

5.1 *Movable Overflow Weir*

The DSD has been keen to identify a sustainable alternative to simply just ‘throwing more drains at the flooding problem’. HVUSSS is Hong Kong’s first application of movable overflow weir system with Supervisory Control And Data Acquisition (SCADA) to collect the excessive stormwater more effectively and hence resulting in a smaller size storage tank. The shallow tank design also allows less energy consumption and less time and cost for construction. The HVUSSS provides a sustainable solution which addresses both the needs for cost effective flood alleviation and environmental protection.

The ingenuity of the scheme comes from the SCADA real-time monitoring of water and tide levels, which allows the control of weir crest level in a real-time manner, to ensure that the filling of the storage tank would start at the most optimal time to prevent pre-mature or late overspill of stormwater into the storage tank. Hence, the design capacity of the storage tank can be reduced by as much as 30%. It also achieves sustainable development by minimizing the amount of excavation for construction and thus the total construction time. Moreover, the shallow tank design and the adoption of movable overflow side weir system can allow over two-thirds of stored stormwater to be discharged by gravity, which significantly reduces energy consumption required in the operation stage and achieves sustainability (see Figure 6).

This project is also Hong Kong's first example of applying state-of-the-art hydraulic modelling techniques including 1-D and 2-D hydraulic models for drainage network and overland flow, as well as a 3-D computational fluid dynamics model for analyzing and understanding hydraulic performance of particular parts of the hydraulic system. Hence, the hydraulic performance of the flood alleviation design has been greatly enhanced.

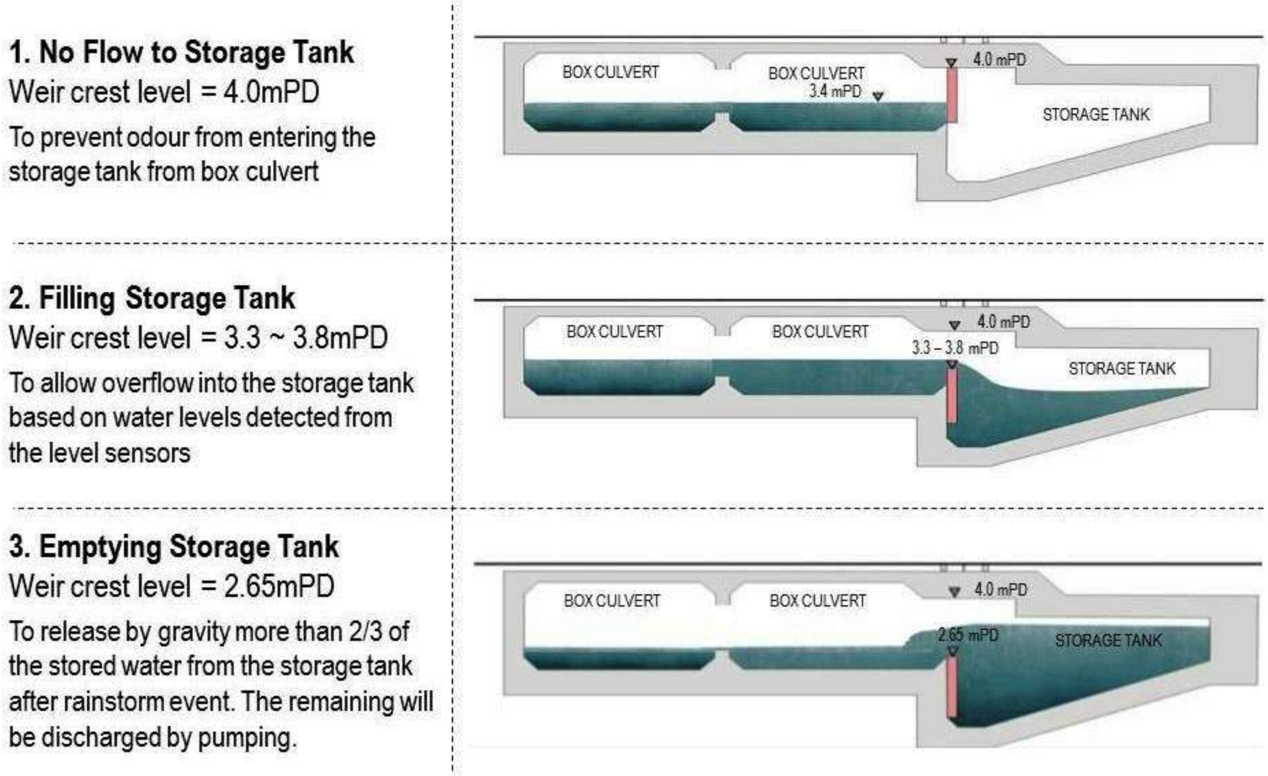

Figure 6 Different operation modes of movable overflow weir between storage tank and box culvert

5.2 *Water Harvesting System*

An element of the works under the HVUSSS is the reconstruction of an irrigation system comprising an above ground water tank ($144m^3$) next to the LCSD management office connected to WSD supply and a system of distribution pipes and watering points. A viable option for the use of collected underground water is the provision as a source of water supply for the irrigation system. As a result of the CSD, the clean underground water collected below the raft can be used to supplement the irrigation water. The preliminary calculation indicates a flow rate of about $2.9x10^{-5}m^3/s/m$ run of section or about $1000m^3$ per day. The amount is subject to confirmation by the results of further ground investigation including pumping tests but the order of magnitude of inflow indicates a good opportunity for providing a good supply for the irrigation water.

The proposed system includes an underground storage and sedimentation chamber at the downstream end of the groundwater collection pipes and feeds to a pump chamber. Submergible pumps remove the collected underground water and feed it to the above ground water tank. The new system for conveyance of underground water comprises submergible pumps and connects to the above ground water tank that supply to the watering points. Pipes provide connection to the $144m^3$ water tank of the irrigation system for delivery of underground water for irrigation use.

5.3 *Omission of Pre-Bored H-Piles*

As a result of the CSD, 533 nos. of pre-bored steel H piles (Grade S460 UC 305×305×180 and 305×305×223kg/m, lengths from 40 to 60m) have been omitted. Taking an average length of 55m for each pile, it can be demonstrated that approximately 4,040 ton of structural steel has been saved.

5.3.1 *Life cycle and embodied carbon of structural steel (H-Piles)*

The life cycle of structural steel can be separated into 7 stages from the extraction of raw materials through the manufacture of the product (sometimes referred to as 'cradle-to-gate') to final material disposal and recycle (see Figure 7).

The carbon dioxide (CO_2) or carbon dioxide equivalents (CO_2-e) emitted into the atmosphere in order to produce structural steel can be measured as Embodied Carbon (EC). Embodied carbon contributes a significant portion of the life cycle carbon footprint of the built environment. The construction sector is the second largest contributor of the carbon footprint in Hong Kong and 85% of the carbon footprint associated with the construction sector is embodied in imported goods and services.

According to the values shown in the ECO-CM website of the Hong Kong University of Science and Technology (http://ihome.ust.hk/~cejcheng/ec/carbonInventoryLocalized.html), the embodied carbon for steel sections is 1.988kg CO_2-e/kg (cradle-to-gate) and 2.037kg CO_2-e/kg (cradle-to-site). Given the steel tonnage saved, emission of 8,043 ton of CO_2-e has been prevented, giving a similar effect of planting 205,938 nos. of trees in Hong Kong.

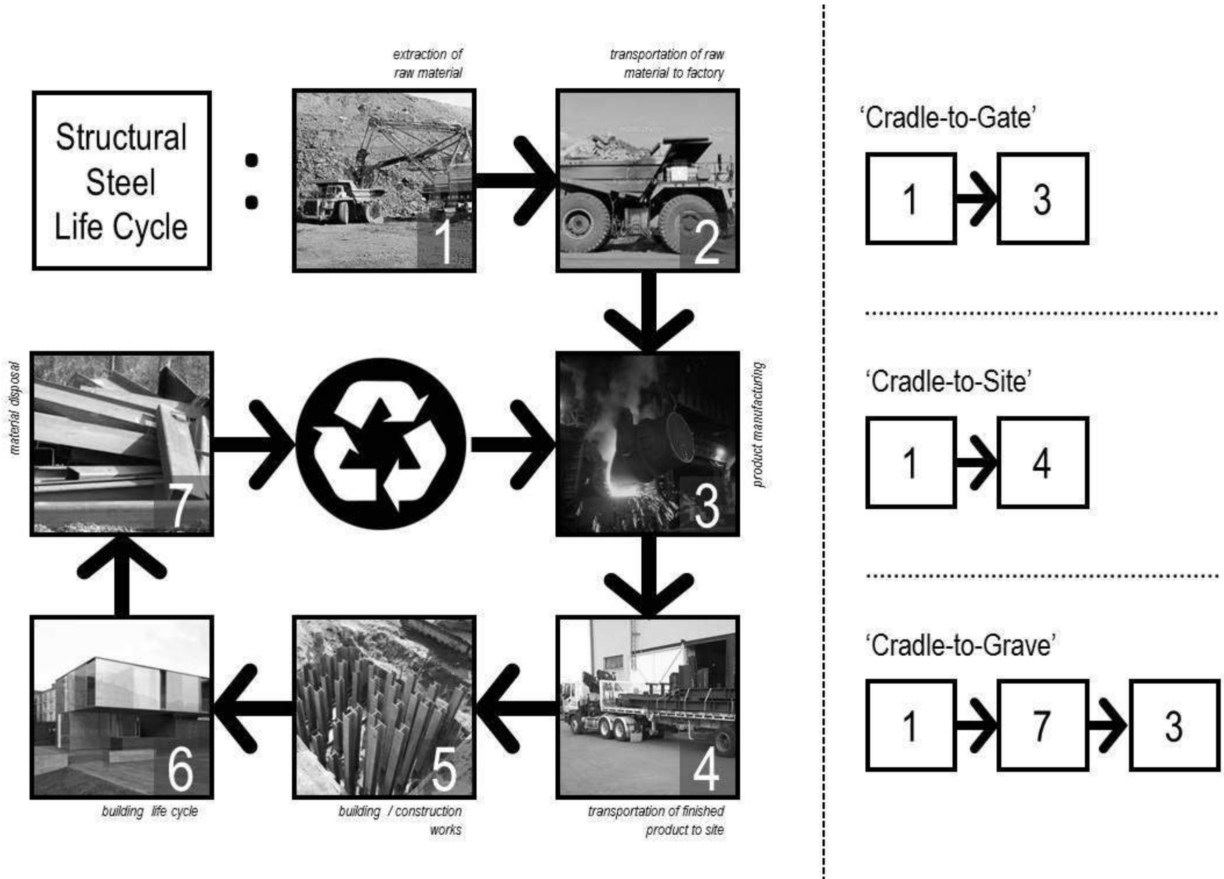

Figure 7 Life cycle of structural steel

5.3.2 *Other benefits*

Other benefits that have been gained via the CSD are as follows.

a) Substantial reduction of traffic impact to existing road network – Due to the reduction in deployment and mobilization of construction plants and equipment under the innovative design enhancement, vehicular traffic destined to/from the project site is substantially reduced. The lowering of the traffic load/demand on existing streets/highways allows for better safety and operations.
b) Reduction of nuisance – Due to the shortened construction programme and the elimination of pile foundations, noise, and polluted water generated from construction activities such as

mobilization and pile driving is reduced. Project risks associated with disturbance to local residents and operations of the horse field are minimized.

c) Improvement in air quality – The reduction in emission from associated construction vehicles or idling equipment would reduce exposure of individuals in residences and businesses in the vicinity of the staging area to pollutants in the exhaust. Construction equipment may include, but is not limited to, dump trucks, loaders, excavators, diesel driven generators, and compressed air units for construction power.

d) Reduced impact on racing track and adjacent facilities – The existing race track and the some of the HKJC's facilities are located in close proximity of the site. The pre-boring for piles in the original scheme is prone to ground movements due to caving in when the pile hole is being formed through the upper alluvial deposit. The innovative design enhancement removed this risk due to the elimination of piles.

The selected progress photos in Figure 8 show the construction of part of the underground stormwater storage tank.

Figure 8 Construction of the underground stormwater storage tank

6 ACKNOWLEDGEMENTS

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Effective use of high performance steel in construction: innovative design and construction using Q690 steel materials

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Keywords: high strength steel materials; structural adequacy; flexural buckling; section resistances; member resistances.

ABSTRACT: Engineers embrace innovation. Engineers are always able to face challenges and provide effective solutions through innovative use of materials in design and construction. In Hong Kong, as there are many large scale infrastructure projects in full swing at present, there is an urgent need for the local construction industry to improve its productivity through innovative use of materials, design and construction. In simple terms, to build differently!
With the support of the China Iron and Steel Association and the National Engineering Research Centre for Steel Construction, the Hong Kong Constructional Metal Structures Association launched a comprehensive industry-led research and development programme entitled “Effective Use of High Performance Steel in Construction” in early 2012. The key objective of the programme is to promote innovative design and construction of high performance steel in construction of heavily loaded structures such as long span bridges, high-rise buildings and structures with large enclosure among design and construction engineers. This paper presents key findings of an experimental investigation on the structural behaviour of Q690 steel columns of welded H-sections under compression recently completed at the Structural Engineering Research Laboratory of the Hong Kong Polytechnic University.
A total of 9 stocky columns and 6 slender columns of Q690 welded H-sections under compression have been successfully carried out, and both local plate buckling behaviour in stocky columns and overall flexural buckling behaviour in slender columns were observed in tests. Test results of these columns have been used to verify the structural adequacy of the Column Buckling Design Method given in the Structural Eurocodes in order to establish effective use of Q690 steel members. The research work aims to establish the use of Q690 steel members in building structures, and provides essential technical data to designers to facilitate their application in construction.

1 CHALLENGES TO LOCAL CONSTRUCTION INDUSTRY IN HONG KONG

1.1 *Shortage Supply of Aggregates in Hong Kong*

Aggregates are essential construction materials in Hong Kong. In 2000, there were four local quarries, namely, Anderson Road Quarry, Shek O Quarry, Lamma Quarry and Lam Lei Quarry, producing at their peak capacities and meeting up to about 50% of the local aggregate demand, in the range of 6 to 9 million tonnes per annum. The other 50% demand was met with imports from the Mainland China.

Owing to various infrastructure projects undertaken by the Government in the recent years, the annual demand of aggregates in the local construction industry has increased steadily to 20 million tons in 2013. On the other hand, with the closure of Lamma Quarry in 2002 and Shek O Quarry in 2012, only Anderson Road Quarry and Lam Tei Quarry were in operation in 2013 with a total annual rock production of 3 to 3.5 million tons. Hence, the aggregate supply in Hong Kong relies heavily on imports from the Pearl River Delta (PRD) Region in China.

It should be noted that according to the Civil Engineering and Development Department of the Government, Anderson Quarry will cease its production in 2016 while Lam Tei will continue its operation with an annual production of merely 1.5 million tons per year on average up to 2022. As the annual demand of aggregates in the next eight to ten years is estimated to range from 16 to 22 million tons, there is an urgent need to explore alternative constructional materials in Hong Kong. Otherwise, the local construction industry will have to rely almost totally on Mainland supplies, and there is uncertainty in supply as well as a likely price hike on the aggregates.

1.2 *Abundant Supply of Constructional Steel from the Pearl River Delta Region*

In order to minimize any impact of shortage of aggregates to construction projects in the coming eight to ten years, it is time for the local construction industry to explore the use of alternate constructional materials. Structural steel is an attractive alternative owing to abundant and steady supply of constructional steelwork in the PRD Region. According to the World Steel Association (www.worldsteel.org), China produced about 779.2 million metric tons (mmt) of steel materials in 2013, which was about 49.2% of world production.

The Annual Report of the China Steel Construction Society released in April 2014 stated that about 50 million metric tons of the steel materials were constructional steel materials, such as structural steel plates, sections and tubes. At present, there are currently about 300 quality steel fabricators in the PRD Region each with an annual production exceeding 20,000 tons, and about 10 quality steel fabricators each with an annual production exceeding 50,000 tons. For easy reference, the steel tonnage of a typical 50 storey commercial building in Hong Kong ranges from 20,000 to 28,000 tons, depending on the geometrical dimensions and structural form of the building. It should be noted that some of these leading steelwork fabricators have been supplying to Macau since late 2000s on various infrastructure projects, public buildings, hotels and resorts. Hence, Chinese structural steelwork has been widely accepted in building and civil engineering projects in Macau because of their structural adequacy, reliable supply and timely delivery. Many of these projects are designed, constructed and quality controlled by Hong Kong engineers.

The following additional advantages associated with steel construction will be able to improve the overall productivity and efficiency of construction projects in Hong Kong:

a) prefabrication with a good control on quality and workmanship,
b) reduced demand on the number of semi-skilled and unskilled workers on site,
c) fast erection and assembling of structural members and building materials on site,
d) increased usable floor areas for better use of the space in buildings,
e) reduced self-weights of structures, and hence, reduced costs for transportation, foundations and superstructures,
f) simple addition and alteration in steel frames through bolting and welding
g) easy recycling of constructional steel materials at the end of their service life,
h) reduced life cycle cost, and
i) lower carbon emission in manufacturing and transportation, when compared with concrete materials.

1.3 *Collaborative Research with China Steel Construction Industry*

With the support of the China Iron and Steel Association and the National Engineering Research Centre for Steel Construction, the Hong Kong Constructional Metal Structures Association launched a comprehensive industry-led research and development programme entitled “Effective Use of High Performance Steel in Construction” in early 2012. The key objective of the programme is to promote innovative design and construction of high performance steel in construction of heavily loaded structures such as long span bridges, high-rise buildings and structures with large enclosure among design and construction engineers. This paper presents key findings of an experimental investigation on the structural behaviour of Q690 steel columns of welded H-sections under compression recently completed at the Structural Engineering Research Laboratory of the Hong Kong Polytechnic University.

It should be noted that EN 1993-1-1 (2005) provides a comprehensive set of design rules for structural steel design in buildings, and it covers steel materials with yield strengths ranging from 235 to 460 N/mm^2. Additional design rules for steel materials with yield strengths up to 700 N/mm^2 are given in EN 1993-1-12 (2007). However, only few additional clauses are provided, and they follow closely to those given in EN 1993-1-1, despite being rather conservative at the same time. Similar design rules may be found in the Hong Kong Steel Code (2011).

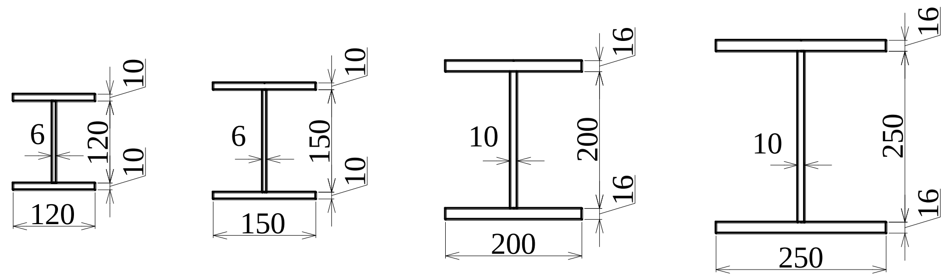
Owing to recent advances in steel production technology in China in the recent years, a number of reputed Chinese steel mills have been producing and exporting high performance Q690-QT and Q960-QT steel plates for lifting equipments overseas. Hence, it is reckoned that these high performance steel materials are able to offer effective structural solutions to heavily loaded structures. Complementary welding procedures (Peter & Bernt, 2005; Dainelli & Maltrud, 2012; Lee et al 2012) with suitable choices of electrodes are widely available in the literature. In order to ensure quality of all steel materials and structural steelwork, it is necessary for the local construction industry in Hong Kong to establish accredited quality assurance systems on production and supply of steel materials as well as fabrication and inspection on structural steelwork.

2 EXPERIMENTAL INVESTIGATION

A total of 9 stocky columns and 6 slender columns under axial compression have been carried out in order to provide test data to verify structural adequacy of various design rules given in the Structural Eurocodes, in particular, EN 1993-1-1 and 1993-1-12. A comprehensive test programme on Q690 steel members has been devised as follows:

a) Tensile tests
   A total of 3 tensile tests on coupons cut from Q690 steel plates of 3 different thicknesses were conducted to obtain basic mechanical properties of Q690 steel materials for subsequent studies.
b) Compression tests
   A total of 9 compression tests on stocky columns of welded Q690 steel H-sections with three different cross-sections, namely, Sections C1, C2 and C3, were conducted to obtain their cross-section resistances.
   Moreover, a total of 6 compression tests on slender columns of welded Q690 steel H-sections with four different cross-sections, namely, Sections C1, C2, C3 and C4 with two different lengths were conducted to obtain their member resistances.

The cross-section dimensions of the welded H-sections are illustrated in Figure 1. Section classification of the four sections according to EN 1993-1-1 is also conducted.



(a) Section C1 (Class 1) (b) Section C2 (Class 3) (c) Section C3 (Class 2) (d) Section C4 (Class 3)

Figure 1 Cross-section dimensions of Q690 welded H-sections

3 TENSILE TESTS ON Q690 STEEL MATERIALS

A total of 3 tensile tests on coupons cut from Q690 steel plates with different thicknesses were conducted, and the measured material properties of Q690 steel plates are summarized in Table 1.

Table 1 Measured material properties of Q690 steel plates

Nominal thickness (mm)	Measured thickness (mm)	E (kN/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)	f_u / f_y	Elongation at fracture
6	5.68	210.4	781.2	821.8	1.05	12.9 %
10	9.83	207.7	754.4	807.0	1.07	16.2 %
16	15.86	208.4	799.8	858.7	1.07	19.7 %

Notes: E is the elastic modulus;
f_y is the yiled strength;
f_u is the tensile strength.

Hence, the measured yield strengths are found to range from 754.4 to 799.8N/mm^2, which correspond to 1.09 and 1.16 of the nominal value of 690N/mm^2. As the tensile to yield strength ratios for all the steel plates are larger than 1.05 and their elongation at fracture exceed 10%, these steel plates are shown to be readily adopted for structural applications according to Clause 3.2.2 (1) of EN 1993-1-12 (2007).

4 COMPRESSION TESTS ON WELDED H-SECTIONS OF Q690 STEEL MATERIALS

4.1 *Compression Tests of Stocky Columns*

In order to determine the cross-section resistances of stocky columns of welded H-sections, a total of 3 series of stocky columns were tested to failure under compression, and there were 3 welded H-sections with nominally identical dimensions in each series. Figure 1 illustrates the cross-section dimensions of these H-sections while the general test set-up is shown in Figure 2.

Figure 2 General test setup of compression tests on stocky columns

In each test, both the applied loads and the axial deformations of the welded H-sections were measured continuously. The test would be terminated when there was a significant reduction in the applied load. Hence, the maximum applied load would be taken as the cross-section resistance of the column. Typical failure modes of the stocky columns are shown in Figure 3 while typical load-deformation curves of the compression tests are presented in Figure 4. The measured cross-section resistances of all the stocky columns are summarized in Table 2.

(a) Test C1S-A (b) Test C1S-B (c) Test C1S-C

Figure 3 Typical failure modes of stocky columns after tests

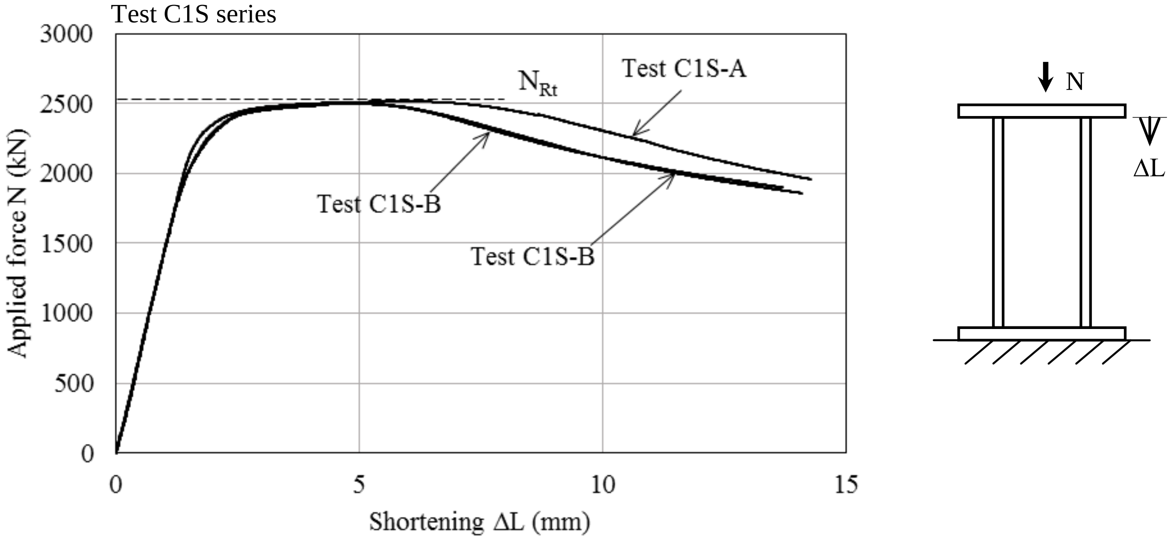


Figure 4 Typical load-deformation curves of compression tests on stocky columns

In all stocky columns, excessive local plate buckling in both the flange and the web plates were apparent at failure.

Table 2 Cross-section resistances of stocky columns under compression

Test		N_{Rd} (kN)	N_{Rt} (kN)	$\frac{N_{Rt}}{N_{Rd}}$
C1S	- A	2316	2515	1.09
	- B	2311	2496	1.08
	- C	2316	2504	1.08
C2S	- A	2892	2998	1.04
	- B	2904	3029	1.04
	- C	2921	2994	1.03
C3S	- A	6581	7055	1.07
	- B	6624	7084	1.07
	- C	6625	7066	1.07

Notes: N_{Rt} is the measured section resistance against compression;
N_{Rd} is the design resistance based on measured yield strengths;
All the specimens failed in yielding and plastic plate buckling.

4.2 *Compression Tests of Slender Columns*

In order to determine member resistances of slender columns of welded H-sections, a total of 6 slender columns were tested to buckle in an overall flexural buckling mode under compression. The test programme is presented in Table 3 while the cross-sectional dimensions of the four welded H-sections, namely, Sections C1, C2, C3 and C4 are illustrated in Figure 1. The general test set-up is shown in Figure 5, and all the slender columns will bend about their minor axes when undergoing overall flexural buckling.

In each test, the applied load as well as both the axial deformations and the lateral displacements at mid-height of the welded H-sections were measured continuously. The test would be terminated when there was a significant reduction in the applied load. Hence, the maximum applied load would be taken as the member resistance of the column. Typical failure modes of the slender columns are shown in Figure 6. It should be noted that local plate buckling in the flange outstands is apparent in some cases, in particular, in sections with large flange outstand to thickness ratios, i.e. Sections C2 and C4. The measured member resistances of all the slender columns are summarized in Table 3.

Table 3 Member resistances of slender columns under compression

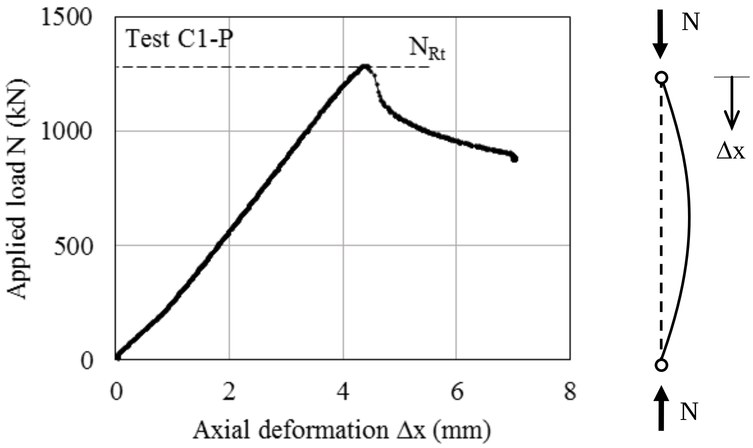
Test		L_s (mm)	L_{eff} (mm)	λ_z	$\bar{\lambda}_z$	N_{Rt} (kN)
C1	- P	1610	1990	64	1.30	1284
C2	- P	1610	1990	52	1.03	2714
	- Q	2410	2790	72	1.44	1510
C3	- P	1610	1990	39	0.78	5924
	- Q	2410	2790	55	1.09	4644
C4	- Q	2410	2790	44	0.87	7284

Notes: L_s is the member length;
L_{eff} is the effective length strength of the test specimen;
λ_z is the slenderness;
$\bar{\lambda}_z$ is the non-dimensional slenderness;
N_{Rt} is the measured resistance of the column under compression;
All the specimens failed in overall flexural buckling.

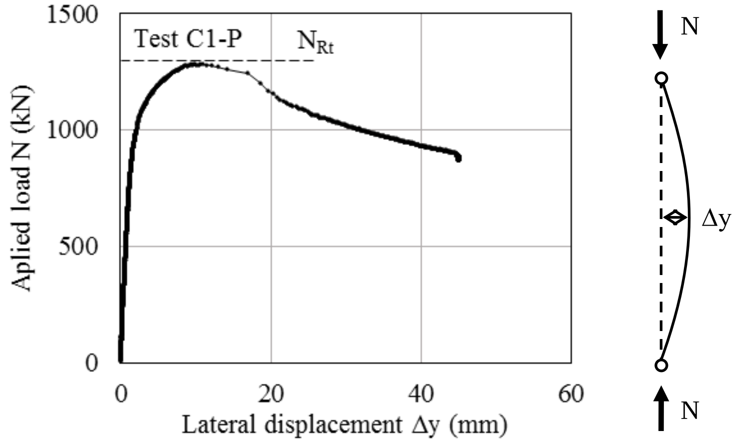
Figure 5 General test setup of compression tests on slender columns

Figure 6 Typical failure mode of a slender column

Typical load-deformation curves of the compression tests are shown in Figure 7. A sudden drop in the applied load is apparent in the graphs, and it is taken to be the member resistance of the column against overall flexural buckling for bending about its minor axis.



a) Load-axial deformation



b) Load-lateral displacement curves

Figure 7 Load-deformation curves of slender columns under compression

5 CALIBRATION AGAINST EN 1993

Table 4 summarizes the measured resistances of both the stocky and the slender columns together with the design values determined in accordance with EN 1993. The design rules given in Clause 5.5 of EN 1993-1-1 for member resistance of slender columns are re-presented in Appendix A for easy reference.

All the measured member resistances of the columns are plotted onto the same graph of the column buckling curves in Figure 8 for direct comparison. It is shown that comparison between the column buckling curves and the measured resistances over a wide range of the non-dimensional slenderness ratios is highly acceptable.

Hence, it is found that

a) All the test data follow closely to the column buckling curves, and hence, the column buckling curves are readily applicable to determine the member resistances of welded Q690 columns;
b) All the test data lie well above Curve c, and hence, it is possible to improve the structural efficiency of the Column Buckling Design Method by an appropriate choice of the buckling parameter, α ; and
c) All the test data lie closely above Curve a_o, and hence, this curve may be adopted to give a conservative and yet efficient design.

These findings agree broadly with the work of other researchers (Shi et al., 2012), and more data are needed to fully justify the proposed change in the Method.

Table 4 Member resistances of slender columns under compression

	Test		L_s (mm)	L_{eff} (mm)	λ_z	$\bar{\lambda}_z$	N_{Rd} (kN)	N_{Rt} (kN)	$\frac{N_{Rt}}{N_{Rd}}$
Stocky columns	C1S	- A	400	280	9.2	0.18	2316	2515	1.09
		- B					2311	2496	1.08
		- C					2316	2504	1.08
	C2S	- A	400	280	7.4	0.14	2892	2998	1.04
		- B					2904	3029	1.04
		- C					2921	2994	1.03
	C3S	- A	550	385	7.6	0.15	6581	7055	1.07
		- B					6624	7084	1.07
		- C					6625	7066	1.07
Slender columns	C1	- P	1610	1990	64	1.27	1222	1284	1.05
	C2	- P			52	1.02	2060	2714	1.32
	C3	- P			39	0.78	5729	5924	1.03
	C2	- Q	2410	2790	72	1.42	1271	1510	1.19
	C3	- Q			55	1.09	4342	4644	1.07
	C4	- Q			44	0.87	6730	7284	1.08

Notes: L_s is the specimen length;
L_{eff} is the effective length;
λ_z is the slenderness;
$\bar{\lambda}_z$ is the non-dimensional slenderness;
N_{Rt} is the measured resistance of the column under compression.
N_{Rd} is the design resistance based on the measured yield strengths, and Curve a_0 is adopted for design against compression.

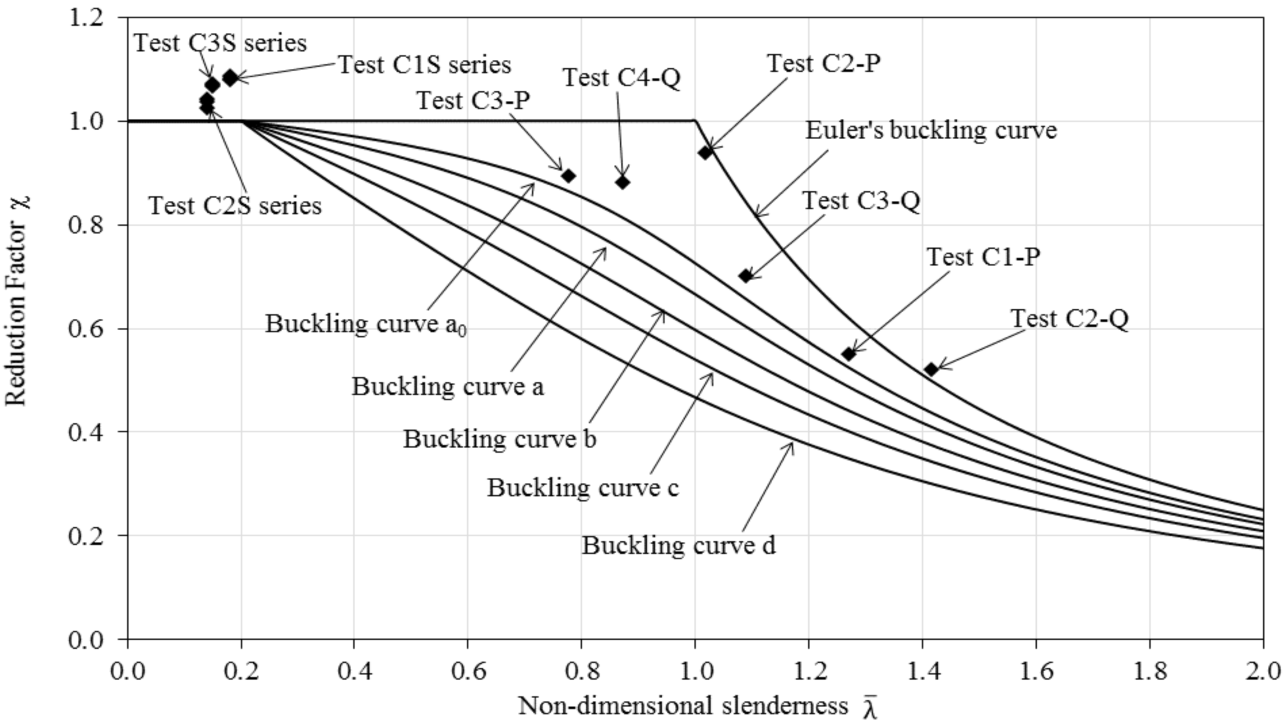


Figure 8 Comparison between measured and design resistances of columns of Q690 welded H-sections

6 CONCLUSION

In Hong Kong, as there are many large scale infrastructure projects in full swing at present, there is a urgent need for the local construction industry to improve its productivity through innovative use of materials, design and construction. Structural steel is an attractive alternative because of abundant and steady supply of constructional steelwork in the Pearl River Delta Region. Owing to recent advances in steel production technology in China in the recent years, it is reckoned that high performance Q690-QT and even Q960-QT steel materials are readily available for construction of heavily loaded structures. Complementary welding procedures with suitable choices of electrodes are widely available in the literature.

A total of 9 stocky columns and 6 slender columns under compression have been successfully conducted, and both local plate buckling in stocky columns and overall flexural buckling in slender columns are observed in tests. The test results of these columns have been adopted to verify the structural adequacy of the Column Buckling Design Method given in the Structural Eurocodes. Hence, these high performance steel materials can be designed effectively. The research work aims to establish the use of Q690 steel members in building structures, and provides essential technical data to designers to facilitate application in construction.

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APPENDIX A COLUMN BUCKLING DESIGN METHOD TO EN 1993:1-1

1. Determine the buckling length of the steel column for both axes.

2. Calculate N_{cr} and Af_y.

3. Calculate the non-dimensional slenderness, $\bar{\lambda}$ of the steel column.

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i} \times \frac{1}{\lambda_1} \qquad \text{for Class 1, 2 and 3 cross-sections}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} = \frac{L_{cr}}{i} \times \frac{1}{\lambda_1}\sqrt{\frac{A_{eff}}{A}} \qquad \text{for Class 4 cross-sections}$$

where A is the cross-section area,
A_{eff} is the effective cross-section area of Class 4 sections,
f_y is the yield strength,
$N_{cr} = \frac{\pi^2 EI}{L_{cr}^{\ 2}}$ which is the critical flexual buckling load/elastic critical force

and
L_{cr} is the buckling length in the buckling plane considered,
$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9\varepsilon$, where $\varepsilon = \sqrt{\frac{235}{f_y}}$

4. Choose a suitable flexural buckling curve for rolled and equivalent welded sections in Table A1, and hence, the imperfection factor, α, is obtained from Table A2.

5. Determine the parameter Φ.
$$\Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

6. Calculate the buckling reduction factor, χ
$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \le 1.0$$

7. Calculate the design buckling resistance, $N_{b,Rd}$.
$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$$
where γ_{M1} is the partial factor for resistance of the steel column to instability.

Table A1 Selection of flexural buckling curves for rolled and equivalent welded cross-sections

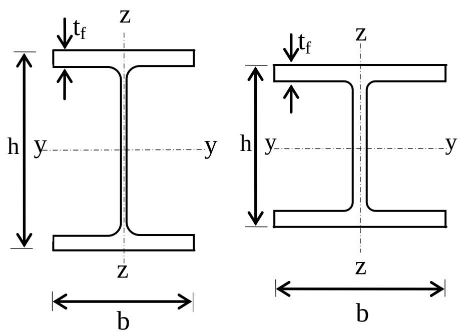
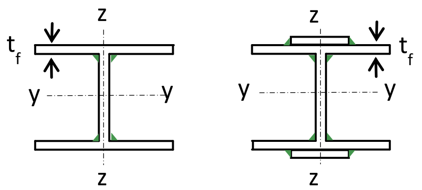
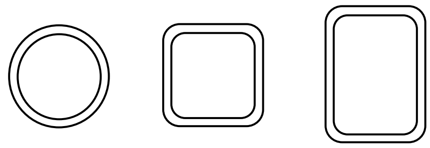
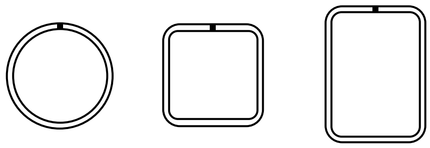
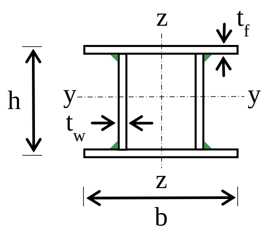
	Cross section	Limits		Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S460
Rolled sections		$h/b > 1.2$	$t_f \le 40$ mm	y-y z-z	a b	a_0 a_0
			40 mm $\le t_f \le$ 100 mm	y-y z-z	b c	a a
		$h/b \le 1.2$	$t_f \le 100$ mm	y-y z-z	b c	a a
			$t_f > 100$ mm	y-y z-z	d d	c c
Welded sections			$t_f \le 40$ mm	y-y z-z	b c	b c
			$t_f > 40$ mm	y-y z-z	c d	c d
Hollow sections			hot finished	any	a	a_0
			cold formed	any	c	c
Welded box sections			generally (except as below)	any	b	b
			thick welds: $a > 0.5t_f$ $b/t_f < 30$ $b/t_w < 30$	any	c	c

Table A2 Recommended values for imperfection factor, α, for various flexural buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor	0.13	0.21	0.34	0.49	0.76

Innovation and sustainability in public housing development

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Keywords: innovation; sustainability; public housing.

ABSTRACT: Hong Kong Housing Authority (HA) is responsible for the construction and maintenance of the affordable public housing to the low-income families in Hong Kong. To achieve the quality, cost effectiveness and sustainability of public housing, HA has strong motivation in adopting and developing innovative solutions in design and construction. In the past decades, we have been achieving these goals through the implementation of innovations in different aspects including planning, design, new materials, construction technology, information technology, etc. Some of these innovations have been developed based on our experience while others have been adopted from external sources. Currently, we are proud to be the pioneer in the use of building environmental modelling and precast concrete for the sustainability of public housing development. The paper summarizes the use of the innovations in public housing design and construction in the recent years.

1 INTRODUCTION

To cope with the public demand on the affordable public housing for low income families, HA has been carrying out the vital task of producing large quantity of affordable quality public housing. The workload on HA has been increasing continuously in past few years and the production quantum will continue to increase in the coming years. Furthermore, HA has also been facing a host of complexities in terms of technical, legal, social, environmental and economic aspects for almost all new housing sites. To cope with these challenges, HA has strong motivation in adopting innovative solutions to solve problems in design and construction, maintain the quality and quantity of public housing under very tight resources. Propelled by this momentum, we have been implementing new initiatives through applied research and technologies that have embraced planning and design, advanced information technologies, different approaches in procurement together with new materials and innovative construction techniques. The application of some of these technologies such as mechanized construction with precast technology has been favoured by the extensively repetitive nature of public housing production. In addition, new innovations have also been introduced to protect the environment and to achieve sustainability which are part of our social responsibility. Given the multi-disciplinary matrix setup of our organization, HA possesses an advantageous edge to innovate with an approach. Whenever there is a problem, it should be an opportunity to innovate. Most of the innovations are project driven, whilst others may follow the normal R&D life cycle of exploring, piloting, reviewing, before mass application. The major innovations for sustainable housing development are described in the coming sections of this paper under four categories, many of which are award winning innovations in recent years –

a) Innovative approach on planning and design,
b) Application of sustainable materials and construction,
c) Use of information technology in sustainable design and construction, and
d) Sustainable procurement approach.

2 INNOVATIVE APPROACH ON PLANNING AND DESIGN

Optimisation of the use of natural ventilation, daylight, sun-shading, reduction of solar heat gain and urban heat island effect by modern technologies would lead to a healthy living environment and reduce energy consumption. The following examples illustrate the major achievements of the innovative design at early stage of planning and design.

2.1 *Passive Design Assisted by Micro-climate Studies*

Since 2004, we have been applying the micro-climate studies in all public housing projects during the early planning and design stages, to create quality living environment and improve the environmental performance of the projects. With the use of computerized simulation models, the local climate can be optimized to enhance wind environment of the site, natural ventilation, daylight and solar heat gain for the domestic flats. The findings of the micro-climate studies properly help designers to optimize the planning and design of buildings and outdoor spaces to create "urban oasis" in estate layout, as well as detailed design using wing walls to bring wind into corridors on domestic buildings. We conduct environmental studies and Air Ventilation Assessment at early planning stage to verify no air, noise and water pollution and other hazard to the development. The studies cover the wind environment, outdoor thermal comfort and sun shadowing analysis at site levels, the Vertical Daylight Factor (VDF) for all flats, as well as indoor environmental quality of flats for each building block. This helps to provide a healthy and quality living environment for residents through optimal use of the natural environment such as wind environment, daylight and solar radiation as well as energy consumption. We conducted on-site measurement at post occupation stage to validate the design. Encouraging feedback from tenants on environmental performance was collected in resident survey conducted in recently completed projects. Figure 1 illustrates study results by micro-climate.

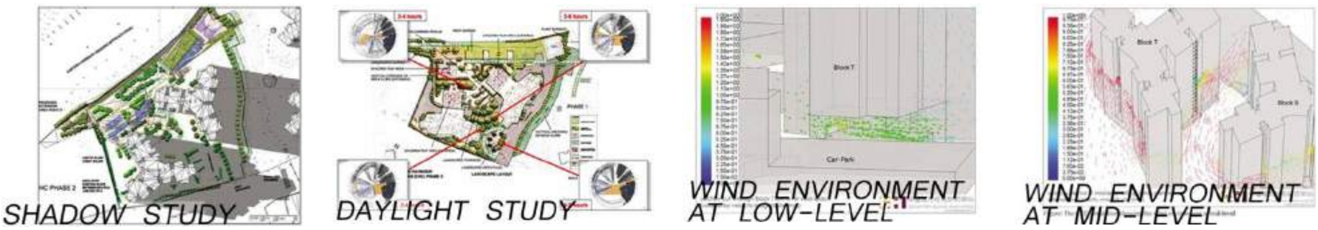
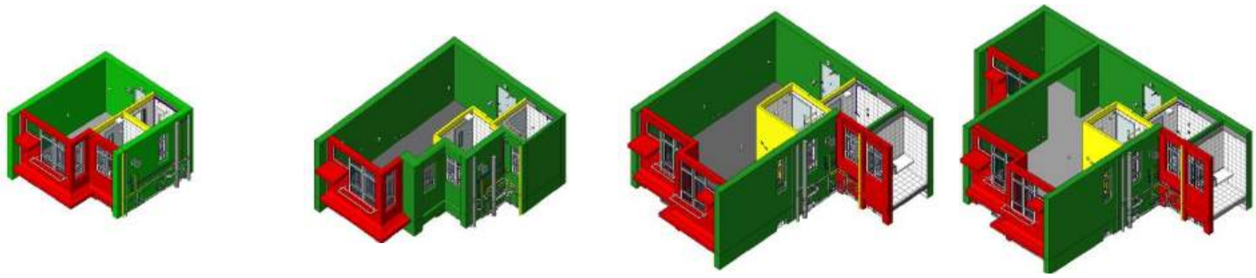


Figure 1 A demonstration of some study results by micro-climate

2.2 *Modular Flat Design*

After adoption of site-specific design approach, HA further rationalised its tool kits for mass customization. In 2008, in line with the principle of "functional and cost-effective" design, HA developed a new library of Modular Flat Design for use in public rental housing. This aims to achieve the best value and practice in sustainable housing design and construction, and to strive for greater efficiency and productivity through wider use of mechanized building process promulgated under Quality Housing Initiatives, whilst at the same time maintaining a certain level of design control over standard of provision and maintaining consistency across different projects. The new library covers a whole spectrum of small modular flats (1-Person / 2-Person and 2-Person / 3-Person flats) and family modular flats (1-Bedroom and 2-Bedroom flats) (Figure 2). Individual designers may articulate these standard modular flats in designing site specific domestic buildings to suit individual site configuration.



1-Person/2-Person Flat | 2-Perso/3-Person Flat | 1-Bedroom Flat | 2-Bedroom Flat

Figure 2 Modular flat design

2.3 *Noise - mitigation Building Modelling*

Due to the scarcity of land, many public housing developments in Hong Kong are located close to major traffic routes. To provide a better and noise acceptable living environment, we adopted a number of noise mitigation measures, ranging from re-arrangement of block layouts to provision of “noise tolerant” buildings. Since conventional mitigation measures might not be sufficient to reduce noise level, the innovative arc screen and acoustic window design were developed to alleviate the problem. The acoustic balcony design with arc screen had been initially adopted in Sai Chuen Road Public Rental Housing (PRH) development and it could deduce 2.5dB(A) and 6.4dB(A) on the lower and higher levels respectively. However, the balcony structures could not be sufficient to alleviate the noise impact to acceptable level in some sites close to heavy traffic roads such as San Po Kong PRH development adjoining Prince Edward Road East, HA, in collaboration with the Environmental Protection Department and the Hong Kong Polytechnic University, further developed the acoustic window of a modified double-glazed window with offset openings to allow natural ventilation as illustrated in Figure 3. The validation by laboratory test and site mock-up revealed that the acoustic window could achieve noise attenuation up to about 8 dB(A). With the innovative design of acoustic window and arc screen balcony, a comfortable and noise nuisance free environment were provided to the residents.

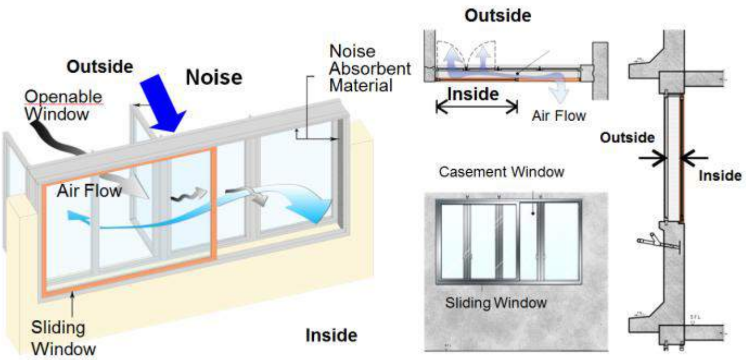


Figure 3 Acoustic balcony and window

2.4 *Carbon Emission Estimation*

The construction and maintenance of building is one of the major sources of carbon emission which would lead the global warming. To provide a green and sustainable building design, we found the need to devise a straight and practical indicator to quantify the comprehensive impacts of residential buildings on the environment. Since 2011, HA has developed and implemented a Carbon Emission Estimation (CEE) methodology, which is conducted during design stage of a public housing development, to holistically gauge the whole life carbon emission of new public housing developments for a building life of 100 years. The model covers six aspects as illustrated in Figure 4 The six aspects of CEE, which have implications on carbon emission, reduction and absorption. The six aspects cover the total carbon emission of a building and estate which will be compared with the benchmark block and estate to gauge the overall life cycle performance in terms of carbon emission.

Aspect	Embracing
I : Materials Consumed During Construction	■Timber formwork for substructure & superstructure ■Steel formwork for superstructure
II : Materials for Building Structure	■Concrete for substructure & superstructure ■Steel for substructure & superstructure
III : Communal BS Installations	Lighting, Lift, Water Supply, Security, CABD, A/C & Ventilation, Fire Services, Electrical Distribution System
IV : Renewable Energy	Solar and/or wind powered system
V : Trees Planting	Trees taller than 5m
VI : Demolition	■Dismantling of building ■Transportation of building debris from site to landfill

Figure 4 The six aspects of CEE

2.5 *Improving Energy Efficiency*

In December 2011, HA developed and rolled out the Energy Management System (EnMS) by modelling on the ISO50001 best practice framework to verify the energy performance of communal building services installations of HA's new domestic blocks systematically and was awarded the first ISO 50001 certificate on residential building design in Hong Kong in June 2012.

To secure low carbon emission and good energy performance in public housing, HA has implemented a number of energy-efficient designs. Over the past decade, significant reduction in annual electricity consumption in the order of 42% in new housing projects has been achieved by a number of energy saving initiatives, including the adoption of renewable resources (using photovoltaic (PV) system), energy-efficient lighting system and innovative lift system.

2.5.1 *Energy-efficient lighting system*

Lighting is one of major energy consumptions in public housing. We developed the two-level lighting design in the common areas of the domestic block enables high efficiency lighting and saving in electricity. A minimum lighting level of 30 lux for typical corridors and staircases and 50 lux for typical lobbies was maintained for safety and security considerations around the clock. Installation of manual switch integrated with the door phone handset in each domestic flat and the provision at strategic positions at the lift lobby and corridors enable the required illumination level up to 85 lux. Apart from the two-level lighting system, using electronic ballasts, T-5 fluorescent tubes, motion and photo sensors would also optimise the energy-efficiency. Figure 5 illustrates the arrangement of the Two-level Lighting System in a typical residential floor.

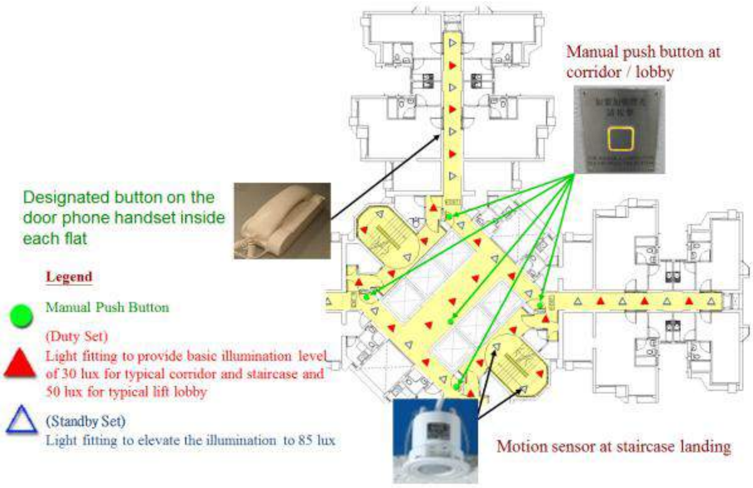


Figure 5 Two-level lighting system

2.5.2 *Innovative lift system*

Starting from 2013, we specified gearless drive for lift installations in our housing developments. Depending on the energy performance of gearless lift drive, it can achieve a running electrical power at more than 10% below the maximum electrical power limit stipulated in the Building Energy Code for lifts adopted before. Meanwhile, current state-of-art technology enables capturing the regenerative power from lift system during heavy load down and light load up conditions for feeding into the grid for immediate consumption by other installations (

Figure 6. Other energy-efficient lift system such as variable voltage variable frequency (VVVF) lift driving system, optimization of lift provision, reduction of decoration weights of lift car, utilization of regenerative power were adopted in HA.

Figure 6 Gearless permanent magnet synchronous lift motor

2.6 *Water saving and enhancing healthy living*

We harvest rain water from the tower roofs and sterilise for reuse in irrigation of the planter at grade and podium. Where automatic irrigation is not feasible for technical or security reasons, we provide lockable water points to allow for manual watering. Recently, we also explore innovative Root Zone Irrigation System (Figure 7) and Zero Irrigation Planting System, which will direct supply water from the roof to the plant, can achieve further saving in irrigation water.

Figure 7 Root zone irrigation system

Normally, water supply would be suspended for about 4 hours for the cleansing process and often caused disturbance to residents. In 2008, we pioneered the twin-tank system, with the water tank divided into two compartments, to provide an innovative ‘alternative operating’ approach ensuring continual uninterrupted water supply to tenants when one of compartments is being cleaned. Less water wastage is assured under well-planned cleansing cycle for all the major water tanks (Figure 8), as well as providing convenience and better hygiene condition.

Figure 8 Twin-tank system

3 APPLICATION OF SUSTAINABLE MATERIALS AND CONSTRUCTION TECHNOLOGY

The following new materials and construction technology have been explored and used in the construction projects of HA.

3.1 *A New Concrete Production Process "iCrete"*

Concrete, which is the major construction materials of HA buildings, contributes a very significant part of the construction cost. However, the design of the concrete has generally been uneconomical due to the great variability of concrete strength which forces manufacturers to over-design so as to minimise the risk. iCrete, a concrete production process, provides a optimization process to design concrete with less variability and sufficient confidence over the design requirements through their expertise in concrete technology. The optimization process mathematically relates the properties of strength, slump and other aspects of concrete, such as cost, cohesiveness and durability which are based on the concentrations and qualities of the various raw material inputs. So close monitoring of the properties of the concrete constituents including moisture contents with extensive use of computational methods and new theories of concrete technologies have been employed. An important approach that has been utilized is the well-designed grading of the aggregates to achieve optimum particle packing resulting in less voids which will in turn lead to stronger concrete with less cement to fill the voids as illustrated in Figure 9. iCrete has been used in the HA project of Kai Tak Phase 1B. In the pilot project, around 30% of cement has been saved as compared with ordinary concrete which has resulted in less cost and great contribution to reduce carbon footprint. In addition, as denser concrete is produced, durability is also enhanced.

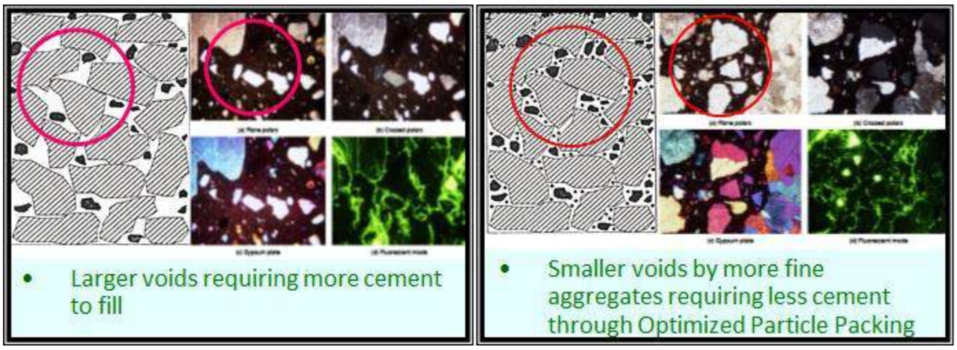


Figure 9 Packing of aggregate particles in ordinary concrete and iCrete

3.2 *New Application of Ground Granulated Blastfurnace Slag (GGBS)*

Ground granulated blastfurnace slag (GGBS) is a by-product of the steel manufacturing industry which had been disposed in the past (Figure 10). However, like pulverized fuel ash (PFA), the product can act as fillers and partial replacement of cement in concrete. The benefits are many including lower heat of hydration during concrete setting, production of concrete of higher long-term strength and stronger durability against chloride ingress. More importantly, disposal of the GGBS can be avoided and carbon emissions in the manufacture of cement can be reduced which constitute elements of environment protection. Realizing the benefits, HA has commissioned a consultancy study on the applicability of GGBS in their precast concrete elements in 2008. In parallel, the Public Works Central Laboratory has carried out testing on the GGBS concrete under the support of the Standing Committee on Concrete Technology. The studies confirmed that concrete with GGBS can attain the required strength with lesser cement and higher durability. HA's study has further indicated that a saving of carbon emission of 20% can be achieved through reduced use of cement and the adhesion performance by tiles on the concrete surface is satisfactory. Meanwhile HA has specified all precast concrete elements to use GGBS except semi-precast slabs in which the compatibility of the insitu ordinary concrete topping and GGBS precast concrete plank is being studied. In view of the coming shortage of PFA, HA is planning to extend the use of GGBS concrete to in-situ mass concrete elements. The trial of GGBS in pile cap construction in a HA project has been completed. The logistic and supply chain would be reviewed comprehensively before full implementation.

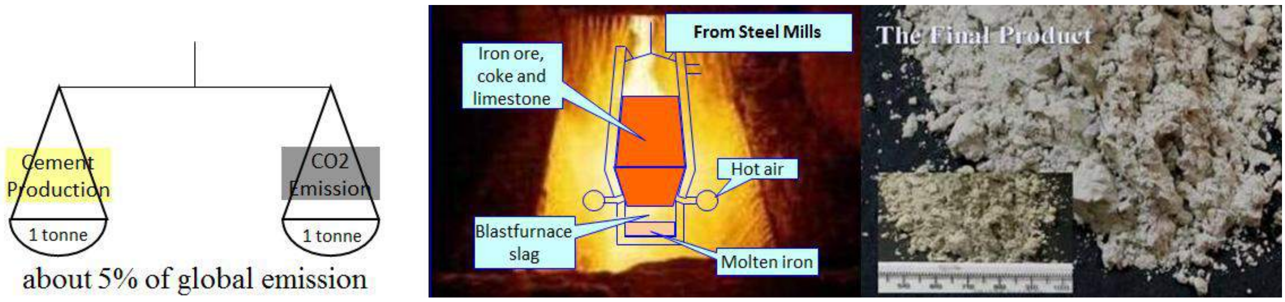


Figure 10 Ground granulated blastfurnace slag (GGBS)

3.3 *New Treatment of Marine Mud in Insitu Backfilling*

Marine mud is often found in construction sites of HA during excavation which is not suitable for backfilling. A common way of handling marine mud in local construction industry is disposing to either landfills or marine dumping sites. However, recent public concerns are growing on the rapid depletion of landfill space in Hong Kong, and also the high levels of heavy metal contents found in some common seafood species due to serious sea water pollution. These environmental concerns have catalysed the development of a green treatment process to convert the 12,000m^3 of marine mud sediment found in the Kai Tak Public Rental Housing Development Site 1A into an inexpensive and sustainable earth filling material. This green initiative has extended successfully to other development sites. The green treatment of marine mud is the conversion of soft marine mud into a material suitable for earth filling by a cement-stabilization process. It aims at obtaining a green recycled earth filling material which is environmental friendly and can be compacted by common machinery to achieve the engineering properties similar to the parent ground such that it can provide similar lateral resistance to the foundation. It is designed as a mixture comprising mainly marine mud, granular material and Portland cement. The green treatment process of marine mud for backfilling is undoubtedly an innovative and self-developed technique being successfully implemented in Kai Tak Site 1A for the first time in Hong Kong. It has also been proved to be effective, inexpensive and environmental friendly. Figure 11 illustrates the mixing, backfilling, compaction and testing of the cement stabilized marine mud.

Figure 11 Mixing, backfilling, compacting and field testing of cement stabilized marine mud

3.4 *Innovative Prefabrication Technology*

The comparatively large scale and extensively repetitive nature of the public housing construction in HA permit the use of highly mechanized construction method involving standardized elements. With such characteristics, HA has developed various construction technologies which not only expedite construction, but also improve qualities and protect environment. Ever since the commencement of precast concrete construction in 1980s starting with precast staircase, HA has extended the precast concrete construction to various elements including facade, tie-beams, refuse chutes, semi-precast slabs and recently to volumetric precast units including bathrooms and kitchens which signifies a great advancement from planar to volumetric precast. These precast elements are manufactured off-territory which helps alleviate the current labour shortage problem in the construction industry. We have successfully installed about 13,500 volumetric precast elements in Kai Tai Site 1A & 1B and extended using this technology in So Uk and Hung Shui Kiu projects. The valuable precasting knowledge and experience gained in public housing development will continue to be a prime mover for sustainable construction in the industry. Currently, the percentage of precast construction in a public housing block has reached to about 35%. Figure 12 shows the semi-precast slab and volumetric precast bathroom construction.

Figure 12 Semi-precast slabs and volumetric precast bathroom in public housing

4 USE OF INFORMATION TECHNOLOGY IN SUSTAINABLE DESIGN AND CONSTRUCTION

In recent years, we have been using extensively information technology in planning and design of the public housing which has proven to be very effective in facilitating design and monitoring, as well as construction, thereby optimizing resources, reducing wastes, avoiding clashes and enhancing safety.

4.1 *Application of Appropriate Design Tools - Building Information Modelling (BIM)*

BIM effectively produces integrated building design in multi-dimensions with graphics and properties, making it easier to identify and decide on design-related issues, avoiding clashes and abortive work. We lead the industry in BIM application and have succeeded in optimizing estate design encountering complicated topography and site constraints, and avoided abortive construction works at subsequent stage. We have constructed a common Building Information Modelling platform for sharing design practices among project teams in estate planning and building design. Furthermore, we establish a Building Information Modelling Design Library containing not only the modular flat design but also the miscellaneous building components such as doorsets and windows etc. for common application during design development. These basic components can be used to create more complex standard modular flat models and assemble to form building wing(s), floor(s) and block(s). Together with integrated application of BIM and GIS (Geographic Information System), we can apply environmental studies including lighting, ventilation, energy, carbon emission, green design etc., prepare quick design visualization for visual impact assessment, thereby enabling our public housing more user-friendly and avoid undesirable clashes of building services installation and underground utilities (Figure 13). These Construction Information Technologies effectively enable us to execute our work from macro to micro levels in an integrated approach, as we see a paradigm shift when we have cascaded the use of BIM through the supply chain. Figure 13 illustrates the use of BIM in the determination of extents of cut-and-fill work with precise quantities in one of HA's project in Tung Tau.

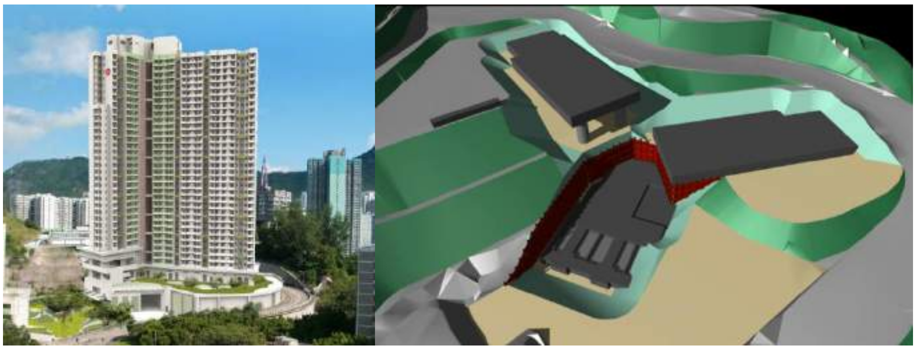


Figure 13 Illustrations of the use of the BIM technique in determination of cut-and-fill quantities

5 SUSTAINABLE PROCUREMENT APPROACH

We have adopted an innovative procurement approach in two projects with more synergistic integration of designers' and builders' expertise upfront in particular aspect of sustainable development. To further enhance the quality of construction material delivered to sites, we have promoted the product certification to ensure the quality, safety and suitability of products through upstream control.

5.1 *Integrated Procurement Approach (IPA) with 3-envelope Tendering System*

Procurement is a key factor in the public housing development process. We therefore place strong emphasis on quality products and services in the selection of our business partners. Integrated Procurement Approach (IPA) aims at earlier and better integration of design and construction expertise from designers, builders and manufacturers so as to achieve a holistic solution for design and construction, offering cost-effective, quality and environment-friendliness. The approach is particularly beneficial in large scale construction projects. HA has first implemented IPA on the public housing development at Kai Tak Site 1B. In the approach, a 3 envelope tendering system has been adopted with Envelope 1 containing “Corporate Information and Technical Submission”, Envelope 2 containing “Proposal for Innovation” and Envelope 3 containing the “Price”. In tender assessment, the main benefits identified are that the approach could effectively mobilize the tenderers to think innovatively and a large number of innovative proposals could be obtained. These proposals include scheme and detailed design, use of new materials, new construction methods, environmental consideration, etc. Effective use of overall project delivery time is also achieved. The second project with IPA implemented is the Public Rental Housing Development in Anderson Road Phase A & B. As a concluding remark, IPA is proved to be a successful step to integrate Contractor’s expertise and innovation to promote the sustainability in public housing development. HA has been seen as a proactive corporation pioneering an integrated production process with added value of innovation in design and construction offered by Contractors. The major innovative proposals received from the contractors are listed below:

a) iCrete
b) Root Zone Irrigation System
c) Materials containing titanium dioxide
d) Precasting technology
e) Renewable energy
f) Permeable External Pavers
g) 100 Years Design Life for Structure

3-envelope tendering system itself is in fact a major innovation in the procurement field, and we have successfully turned the tendering process into a learning process for professional development of HA’s staff as well as tenderers, in quest of sustainability and innovations. Figure 14 illustrates the three envelope system.

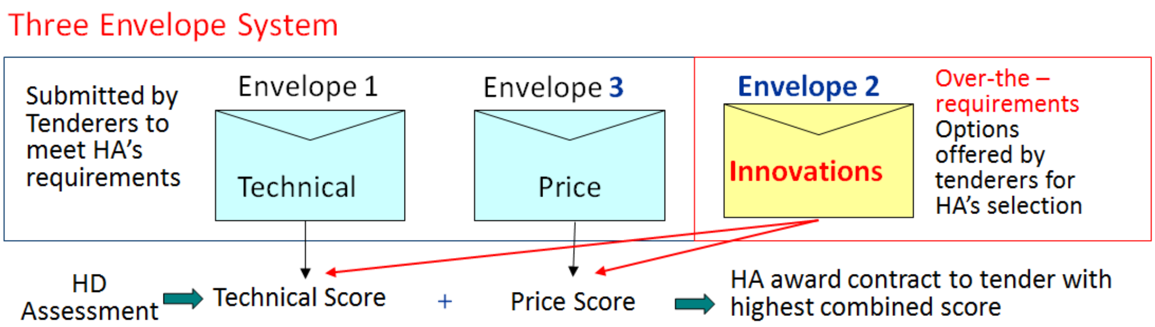


Figure 14 Three envelope system

5.2 *Product Certification*

Each year, HA procures a large volume of building and engineering products through its construction contracts for public housing development. As a major buyer, HA is very conscious of the quality of building and engineering products that supplied for use in public housing projects. A good quality product not only can serve the purpose of its use but also helps reduce the number of replacements due to early deterioration which in turn reduce the amount of natural resource consumption and waste generation. To uphold the quality of building and engineering products for use in public housing development, HA has spearheaded the implementation of product certification in its construction projects in stages since 2010 and up to now ten major building and engineering products such as fire-rated timber doors and mesh reinforcement are required to have obtained product certification before they are approved for use, among other things, in public housing projects. Product Certification is the process whereby a product is certified to have achieved prescribed standards and quality requirements, through regular factory surveillance and periodic random sampling and testing of products by third party of accredited certification body. It is an upstream control of the supply chain. Instead of merely relying on the test certificates provided by manufacturers and surveillance checks, product certification at production line offers a better guarantee for quality, safety and reliability of products. It puts more emphasis on technical capability of factory, which is different from ISO 9001 certification which emphasizes more on factory's quality management system.

At micro-level, product certification adds confidence and assurance on product quality and enhances product reliability. At macro-level, product certification enables sustainable development through less natural resource consumption and waste generation. Figure 15 illustrates the product certification process by HA.

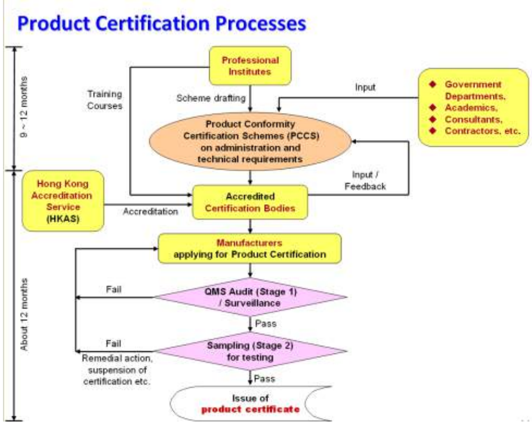


Figure 15 An illustration of product certification process in HA

6 CONCLUSION

To enhance the sustainability of public housing development in face of challenges encountered on various fronts, HA has been actively inventing and using innovative technologies in different stage of design and construction, all to be developed in line with our Core Values of "4Cs" - Caring, Customer-focused, Committed and Creative. The use of the innovative approach has generally proven to be successful. Today we are the one of the more advanced fore-runners in the use of a number of these technologies in the local industry and advancing the frontiers of human knowledge for construction industry in the global arena. HA would continue with this path of exploration for further success, in partnership with business stakeholders, in particular when in face of more challenges as we see the huge public housing demand in coming years.

Sustainable Midfield Concourse Development at Hong Kong International Airport

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Keywords: new visions and insights in sustainability and resilience.

ABSTRACT: In today's world, consideration of sustainable development is an integral part of the engineering profession. This paper sets out the guidelines and parameters for the inclusion of Green Design principles in the new Midfield Concourse Development at Hong Kong International Airport. It illustrates how sustainable initiatives were taken forward through the different stages of the project from a planning, engineering and building services perspective. Methodologies and practical solutions are offered which can help to fulfil corporate social responsibility demands by achieving holistic and sustainable development. By working closely with stakeholders, employees, suppliers and the wider community to create long-term value for Hong Kong International Airport, the Midfield Concourse Development will make a significant contribution to the sustainable development of Hong Kong.

1 INTRODUCTION

The five-level, 105,000 m^2 Midfield Concourse (MFC) at Hong Kong's International Airport will provide 20 new parking stands, connected to Terminal 1 via an extension of the existing automated people mover system. The design has incorporated a wide range of sustainable initiatives that were established early in the design through the development of a project specific "Green Airport Design Strategy" by the Airport Authority and the design consultant Mott MacDonald – ARUP Joint Venture (MAJV), which is referenced throughout this paper. The initiatives can be categorized into different aspects, such as Site Aspects, Material Aspects, Energy Use, Water Use and Indoor Environmental Quality. This paper will focus on Material Aspects and Energy Use which have been incorporated throughout the design and construction process. It is the desire of the Airport Authority to deliver a sustainable building that is specific to the airport environment within the context of Hong Kong.

2 MATERIAL ASPECTS

The materials used in the construction, fitting-out, operation and maintenance of buildings use large amounts of energy and natural resources including fuel and energy to extract the required raw materials, process, transport and install them.

There are opportunities to reduce environmental impacts through good design, good materials selection, and high quality and efficient installation methods. Major considerations are:

a) Quantity of material used (both virgin and recycled).
b) Energy, resources and fuel use in manufacturing and transportation.
c) Waste generated and the potential for minimizing waste, re-using materials and recycling.
d) Energy and resource use during building operation.
e) Atmospheric and other pollution arising from construction, manufacturing, transportation and building operation.
f) Ability of the building to adapt to future change of use or operational procedures.

Figure 1 Overview of MFC

2.1 *Timber Use*

The extraction of timber from forests in an unsustainable manner destroys valuable eco systems and reduces the size of CO_2 sinks around the world. Historically the construction sector in Hong Kong used large quantities of timber wastefully. The MFC however reduces the amount of timber form work through the extensive use of steel shuttering and precast concrete floors. Where necessary for temporary site works the use of virgin timber is avoided and there is a requirement that timber products used within the MFC must be sourced from Forest Stewardship Council (FSC) certified sources.

2.2 *Modular and Standardized Design*

Large quantities of materials can be unnecessarily wasted during the construction process particularly where building forms are unnecessarily complex. The MFC building function determines that the form of the concourse is elongated; the MFC capitalizes on this by incorporating details that promote the use of modular systems where possible.

The use of a standardized grid system allows standard size factory built and assembled components to be used leading to efficient use of building materials. It also has quality and environmental cost benefits as it simplifies design and site operations. The MFC makes extensive use of precast structural floor units (Total: ~1,800 units and ~6,000m^3 concrete used), prefabricated steelwork and modular roof units.

Figure 2 FSC timber used in MFC

Figure 3 Steel column shutter used in MFC

Figure 4 Beam shutter used in MFC

Figure 5 Precast structural floors

Figure 6 Prefabricated steelwork

2.3 *Recycled Construction Materials*

The extraction and use of raw materials and subsequent manufacture / processing of building materials uses a large quantity of energy and resources. The use of recycled building materials can lead to good savings in energy and resource usage. The Midfield Concourse will make use of:

a) Recycled structural steel, minimum of 60% (Up to Jan. 2015, over 4,000 tonnes of recycle materials contributing over 60% of total structural steel had been used);
b) Recycled cold formed steel, minimum of 10%;
c) Recycled content in reinforcement bars, minimum of 10% (Up to Jan. 2015, over 8,500 tonnes of recycle materials contributing over 60% of reinforcement bars had been used);
d) 10% of all building materials to be recycled; and
e) Pulverized fuel ash for cement mix.

2.4 *Regional Manufacture*

A requirement for local sourcing of materials reduces the environmental impacts primarily from transportation of required materials and structural elements. Cement, pulverized fuel ash and natural stone aggregates used to make concrete are sourced from within 800km of Hong Kong and the project has a target of sourcing over 20% of building materials locally or from within 800km from HK.

Figure 7 Concrete batching plant in MFC construction site

2.5 *Waste Management*

Construction and Demolition waste (C&D) is generated wherever any construction/demolition activity takes place. C&D waste consists mostly of inert and non-biodegradable material such as concrete, plaster, metal, wood, plastics, etc. C&D waste will be generated throughout the entire development life-cycle of the Midfield Concourse. There are principally two types of construction waste: inert or non-inert construction waste.

Non-inert construction waste typically makes up around 20% of the total and usually comprises timber, vegetation, packaging waste and other organic materials. Some of these can be recycled while others are disposed of at landfills. In contrast, inert construction waste - otherwise known as public fill - mainly includes construction debris, rubble, earth, bitumen and concrete, which can be used for land formation. Materials like concrete and asphalt can also be recovered for re-use in construction.

The major approach to managing construction waste in Hong Kong is the use of public filling areas for inert construction waste and landfills for non-inert construction waste. However, landfill sites are currently close to reaching their capacity in Hong Kong and as such all measures to reduce non-inert waste must be considered.

The following hierarchy for construction waste is controlled during this project through the use of a construction waste management plans:

a) Sort mixed construction waste and not dispose of it in a single place;
b) Reuse and recycle as far as possible; and
c) Design better and construct more efficiently to minimize waste.

3 ENERGY USE

3.1 *Green Airports and Energy*

In Hong Kong, buildings account for over 50% of total energy consumption. Using energy more efficiently reduces the amount of fuel required to produce a unit of energy output and reduces the corresponding emissions of pollutants and greenhouse gases. New buildings have an opportunity to make use of best available technologies for building energy efficiency and can significantly reduce energy demand whilst delivering the required internal environment.

To profile energy consumption and potential savings for the MFC a detailed energy model was constructed taking into account building envelope parameters, mechanical and electrical services, climatic data and occupancy schedules. Considering the breakdown of energy consumption enabled the design team to focus efforts on effective energy saving strategies and assess the potential benefits.

3.2 *Passive Design Methodology*

Passive design techniques were adopted and have embedded energy use reductions into the MFC from the outset. The MFC capitalizes on the use of natural daylight to offset demand for artificial light use.

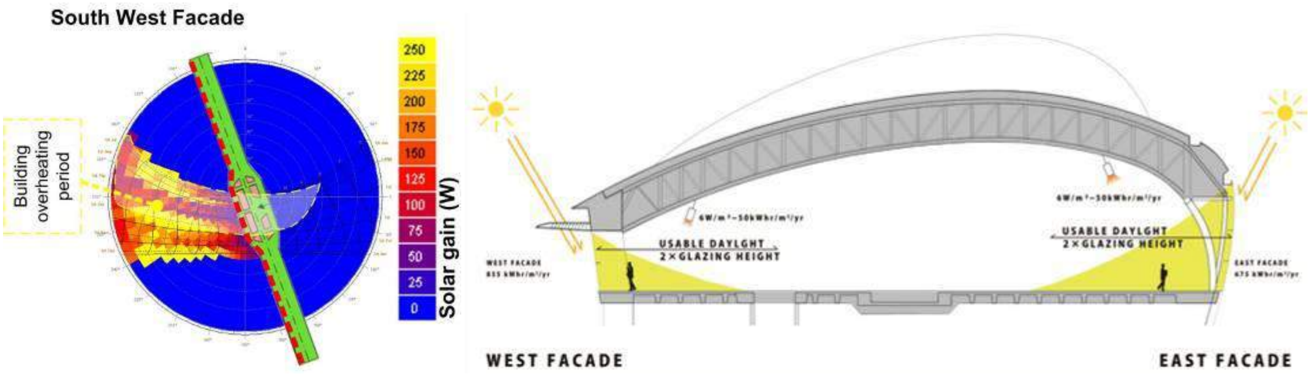

Figure 8 Energy balances of east and west facades

3.2.1 *Reducing artificial lighting demands*

Energy modelling of the MFC shows lighting energy accounts for almost 30% of energy consumption. The design of a good natural daylight system will optimise the amount of daylight entering the space, decreasing energy bills and increasing occupant connectivity with the outdoors.

The concourse has two long facades with tall windows on either side; allowing daylight to be introduced through vertical glazing. The MFC is oriented almost north-south. Due to lower annual solar heat gains on east facades in Hong Kong it is possible to increase the glazed area on the east to receive more daylight, at a relatively low solar heat gain penalty.

Performance is further enhanced by incorporating over-head north lights (true north-facing). These strategies work in parallel with intelligent daylight and occupancy controls to optimize performance. North lights block the majority of direct sunlight (from the south) but allow diffuse light to enter (from the north).

This system has the following benefits:

a) Reduces the dependency on electrical lighting by emitting controlled daylight to the space;
b) Occupants feel a greater connectivity with the outside;
c) North lights' roof areas (south-facing) are used to support PV cells which are optimally inclined to capture sunlight;

d) Relatively low glare issues associated with the predominantly diffuse light from the north;
e) Access gantries have been provided to give safe access for maintenance of the north lights; and
f) North lights are integrated with smoke ventilators.

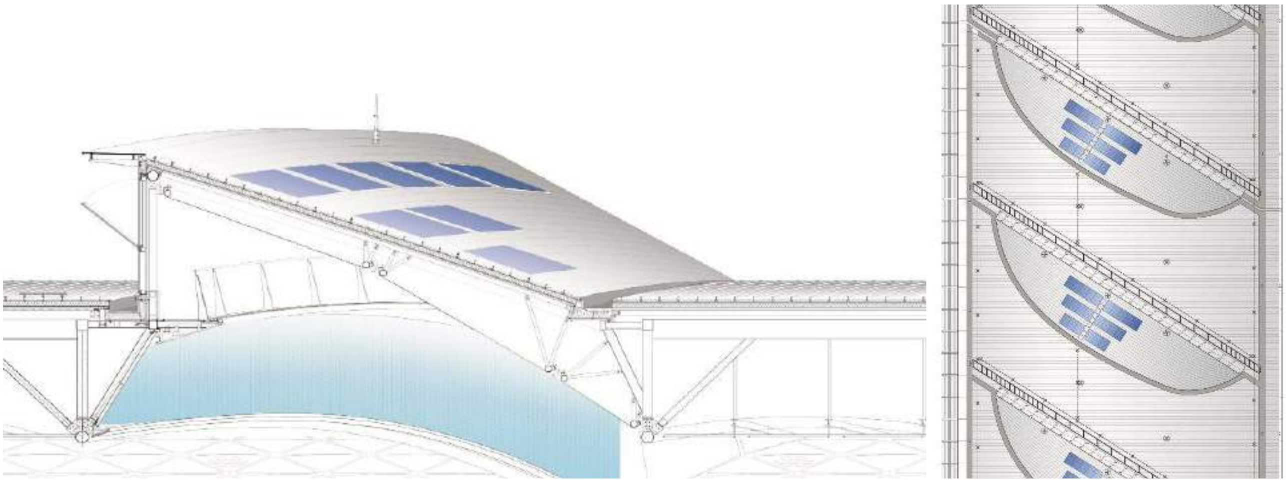

Figure 9 North lights located on the roof along the east west axis

3.2.2 *Reducing solar heat gain*

Solar penetration and associated heat gains are minimized to reduce cooling system energy consumption. This was achieved using the following hierarchy:

3.2.2.1 *Considerations*

a) Solar heat gain to a space can significantly increase the cooling demand for space;
b) The south facing facade is minimized. This adds benefits as in Hong Kong's climate, this facade receives the highest quantity of solar heat gain throughout the year;
c) Special considerations have to be given to the large E-W facades that receive low angle sun;
d) Direct light can also lead to glare issues, which causes problems with the use of flight information displays and computer screens due to high levels of light contrast.

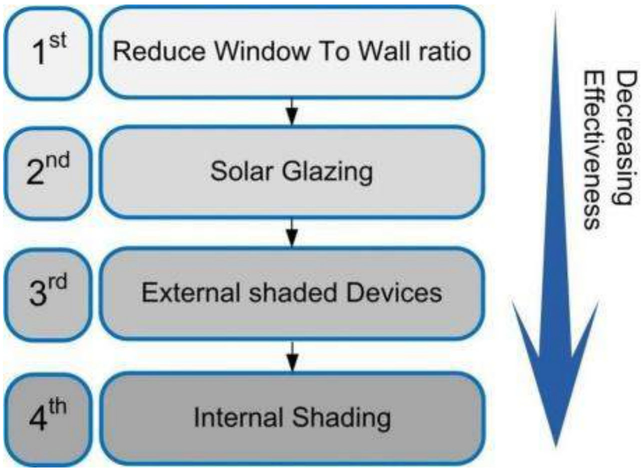

Figure 10 Shading hierarchy

3.2.2.2 *Step 1window to wall ratio*

The size of the windows is directly proportional to the solar heat gain emitted through the window; reducing the window to wall ratio will reduce the cooling demands of the building. This is a dominant factor in facade heat gain.

American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE) Standard 90.1 recommends a maximum window to wall ratio of 40% as a starting point. This target is used in the lower floors. In the Departure Concourse a more conventional window to wall ratio of around 60-80% is used.

The MFC adopts an asymmetric cross-section at the Departure level, with a lower West facade to mitigate the much higher heat gains on this surface, with the following benefits:

a) Reducing facade heat gain directly reduces cooling loads and associated running costs; and
b) Internal comfort will be increased as temperature asymmetry between the glazing and occupants is reduced.

3.2.2.3 *Step 2 high performance glazing*

High performance glazing is a highly effective method to reduce solar gain, and has the following benefits:

a) Permanent solution which requires no additional maintenance compared to tradition facades;
b) Reduced cooling loads, due to the reduction of solar loads entering the building, facade heat gain of the MFC will be reduced;
c) Internal comfort will be increased as temperature asymmetry is reduced; and
d) The choice of a glass with low solar heat transmission but high visual light transmission minimizes heat gain but maximizes natural daylight benefits.

3.2.2.4 *Step 3 external shading systems*

The use of external shading devices or overhangs blocks direct solar heat gain from specific orientations and solar angles with the following benefits:

a) Permanent solution, that can be integrated architecturally into the facade;
b) Low maintenance due to no moving parts;
c) Reduced cooling loads due to the reduction of solar loads entering the building. Facade heat gain of the MFC will be reduced; and
d) System can be integrated with window maintenance strategy.

The MFC incorporates:

a) Overhangs on the southern elevation which are determined by Hong Kong’s solar conditions, this will minimize direct sunlight on the floor plate;
b) External louvers on the West facade, where the MFC rises to allow an additional level of accommodation in the node, fritting is also provided to the glazing to reduce solar heat gain; and
c) External shading louvers are included on west fixed link bridges which have south facing glazing. Where north glazing is provided on the eastern fixed link bridges, louvers are not required and as such are excluded to minimize unnecessary construction.

3.2.2.5 *Step 4 opaque façade elements*

The MFC has a relatively large roof area compared to the conditioned volume of the building. When the sun hits the roof a significant amount of energy can be transferred to the building which increases cooling loads. To mitigate solar heat gain, thermal insulation is used in the roof reducing the U-value to below $0.3 W/m^2K$.

3.3 *Active Design Methodology*

As well as the passive design strategies, low energy active design techniques are helpful for delivering energy in the most efficient way.

The energy modelling process highlighted that the lighting and MVAC systems consume a large portion of the total energy demand. Therefore, a number of innovative and effective energy saving methods have been adopted in the MVAC and lighting systems.

3.3.1 *Best practice cooling solutions*

Several best practice MVAC solutions have been incorporated into the MFC, with the following elements exceeding the requirements of Hong Kong’s Building Energy Codes.

3.3.1.1 *Variable air volume system*

Variable air volume (VAV) systems for fresh air delivery are proposed for the MFC. Air Handling Units (AHUs) will be provided at each floor, consisting of mixing box, coiling coil, fan and filter. The major advantages of VAV system are:

a) VAV systems allow quick response to the dynamic cooling loads in predetermined zones. Each VAV box will be controlled by its own CO_2 sensor and modulates appropriately to maintain the environmental requirements within the zone it serves.
b) VAV systems require minimal maintenance within the actual occupied space. The majority of items requiring maintenance are in the plant rooms.
c) VAV systems are self-balancing as variable control dampers are mounted as necessary in the system ductwork and the VAV boxes modulate to ensure that the required airflow rate is delivered to all areas served by that system.
d) The high level of fresh air supply will provide a pleasant and comfortable atmosphere for occupants.

3.3.1.2 *Binnacle air delivery for departure hall*

Binnacle air delivery systems are used in the Departure Concourse which has large floor to ceiling heights and tie in with the AHU systems, and with variable speed fan systems. Energy efficient characteristics of binnacle air systems are:

a) Delivers air to the space up to approximately 4m above the ground;
b) Cooled air falls with gravity around the occupants in the occupied zone;
c) Only the lower zone is cooled.

3.3.1.3 *Chiller water loop – variable primary flow*

Since the MFC is long and narrow building (700m), chilled water will need to be pumped for large distances requiring a large amount of energy to move the chilled water along the length of the building.

Variable primary flow systems are adopted and these save energy by reducing the flow of chilled water when demands are low, so that pumping energy is not wasted.

3.3.1.4 *Water cooling system*

Water cooled systems offer significant energy savings compared to air cooled systems. Five variable speed cooling towers are provided to serve the typical MFC building loads. These are served by a variable speed condensing water pump, which ensures pumping energy is not wasted.

Water recycling systems are a sustainable initiative whereby. The grey water from washing basins, cleaner’s sinks, showers and kitchen sinks, will be collected and treated by the grey water treatment plant and then used to supply cooling tower water makeup at the MFC Chiller Building at the MFC.

3.3.2 *Active design of renewable technologies*

The MFC will use over 1200m^2 of photovoltaic panels to convert the sun’s energy into electricity and offset the use of grid electricity. The panels are mounted on sloped south facing areas of the north lights. This gives the panels a better performance compared to flat mounting. These PV panels will provide over 0.5% of the building’s energy.

Figure 11 PV panels at MFC

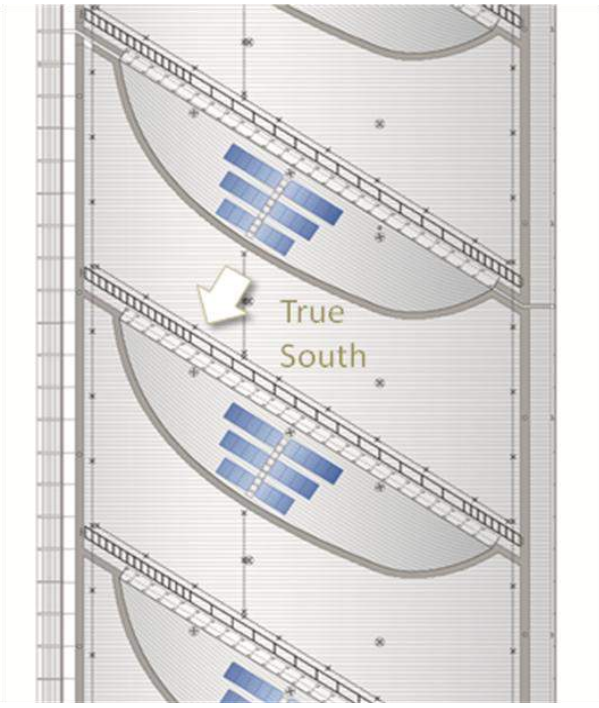

Figure 12 Orientation of PV panels

3.4 *Commissioning, Operation and Maintenance*

It is essential that the building systems are set up to function efficiently to allow optimal performance. This will be achieved through an extensive commissioning process. Operations and maintenance staff will be trained to use the systems effectively so as to realize all benefits.

The benefits of extensive commissioning and staff training include reduced energy use, lower operating costs, fewer contractor call-backs, better building documentation, improved occupant well-being and productivity, and verification that the systems are able to perform in accordance with project requirements and expectations.

The MFC will use a detailed commissioning plan to:

a) Ensure full and complete commissioning of all systems, equipment and components that impact on energy use and indoor environmental quality;
b) Ensure proper training and handover to the operation and maintenance teams; and
c) Report on the outcomes of the commissioning.

In conjunction with the commissioning operation, maintenance and energy conservation manuals will be prepared for the MFC.

3.5 *Transportation Design*

People move relatively large distances around airports. Whilst maintaining timely efficient and comfortable transportation several initiatives have been included to minimise energy consumption from MFC transportation:

a) The APM train system will incorporate regenerative power from the braking function;
b) Vertical transportation will use regenerative power, and presence detectors in lift cars for lighting, ventilation and inverter drive units; and
c) Horizontal transportation (moving walkways and escalators) will slow down when not required.

4 CONCLUSION

The Midfield Concourse Development has incorporated a wide range of sustainable initiatives into the design, construction and operation of the facilities. The adoption of the sustainable initiatives listed in this paper is at the core of the Airport Authority's commitment to develop Hong Kong International Airport in the most environmentally friendly and sustainable ways, whilst always delivering the highest quality and world's best airport facilities. With the inclusion of the Green Design principles in the Development, it is anticipated that there will be major improvement on the green performance compared with the existing airport developments.

5 REFERENCES

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Sustainable transport management solutions for mature cities

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Keywords: sustainability; maturity models; real-time data; information mapping.

ABSTRACT: Incorporation of sustainable transport solutions into mature cities is often considered to be costly and extremely difficult due to existing infrastructure, utilities and the built environment. This paper aims to show how challenges can be transformed into opportunities and how cities are currently seeking to make the "journey" more enjoyable, healthy, reliable, safer, cost effective and connected. Through a series of examples this paper aims to demonstrate that even in the most mature of cities, opportunities for incorporation of sustainable transport exists, and even in cities which have excellent public transport, cycle lanes and the like still more can be done. The examples will draw on cities which have not just considered issues of significant infrastructural improvements but also societal opportunities for revitalisation and changes to the living and working environment. Examples will also be cited of applications of smart technologies for sustainable transport options using real time monitoring and data applications. The thread running through the paper will be the introduction of the use of maturity matrices to demonstrating the benefits of decisions made in respect of sustainable transport planning in terms of performance measures such as air quality, noise, journey times etc., as well as the communication to a wide range of stakeholders. Using this approach the return on investment can be articulated more cogently than merely by stating the cost of designing, construction and operating the transport modes.

1 INTRODUCTION

It is an acknowledged fact that the world is rapidly urbanising and by 2030 over 60% of the world's population will live in cities (Economist Intelligence Unit, 2007, p.1). An increase in population densities and migrations into cities can place significant demands on urban infrastructure and, as a result, many cities face multiple and significant infrastructure challenges. Transport issues have been identified, by some, as being the most significant concern across cities with more issues to unfold as cities expand. Inadequate transport networks have the ability to cause significant impacts on economic development and a city's global competitiveness. From a societal perspective, transport infrastructure is a significant factor when rating the liveability of a city and quality of life in general, which is increasingly important for mobile workforces and economic migrants. The increased awareness of public health and environmental impacts associated with certain forms of transport when coupled with the economic and social impacts arising from a poorly planned system (or a system under considerable strain), further affect the "city indices". A concerted effort is needed by policymakers, city planners and engineers to address the issues through the development of innovative sustainable transport solutions, which strike the balance between economic competitiveness, environment and quality of life for urban residents. Stakeholders including residents and commuters within the sphere of influence of the transport networks also have a role to play in supporting initiatives to improve mobility options.

This paper focusses on some of the transport issues and solutions facing mature cities (as opposed to emerging cities), which are generally defined as cities that have older population profiles, over 100 years old and have built out their basic infrastructure to serve their populations one or two generations ago (Economist Intelligence Unit, 2007). Mature city governments such as those of Paris, New York, Tokyo, Berlin and London have explicitly or implicitly defined long ago that regardless of traffic conditions no more road infrastructure would be built in their core areas. Instead, resources would be concentrated on public transport (Penalosa, 2007). Mature cities provide an interesting perspective on mobility and various factors in terms of modal choices influence the final solution. It would be impractical to suggest opportunities for enhanced mobility choices arise from infrastructure only; policy, technological and innovative solutions could also be considered.

This paper aims to demonstrate that even in the most mature of cities, opportunities for incorporation of sustainable transport exists, using examples where societal opportunities for revitalisation and changes to the living and working environment, and the application of smart technologies for sustainable transport options using real time monitoring and data have been adopted. The use of maturity matrices is introduced to demonstrate the benefits of decisions made in respect of sustainable transport planning in terms of performance measures as well as the communication to a wide range of stakeholders.

2 TRANSPORT CHALLENGES IN MATURE CITIES

Despite a redirected focus away from road infrastructure towards public transport by some leading governments, there are a number of examples of mature cities where significant transport issues exist and require innovative measures to ensure improvements.

Los Angeles (United States), for example, is known to be a city with serious congestion issues, as seen through the dominance of car travel (85% of trips) when compared to biking and walking (13% of trips) and transit (2% of trips). Despite the implementation of policy on innovative fare pricing to incentivise public transit travel in 1985, ridership declined over the subsequent period and a further dependence on car travel prevailed (Wolch, Pastor, Dreier, 2004). The city has recently addressed these issues through various planning measures outlined in a 30 year long-term plan to ensure service requirements meet future needs. The transport plan outlines a mix of solutions including a new metro, expanded bus networks, carpool lanes and various initiatives to promote rideshare.

Sydney (Australia) was, until recently, considered to have a poor performing transport network, and journeys involved long waits for connecting services and multiple tickets due to the lack of integration of services. Acknowledging the urgent need for rail development and service integration linking Suburban and Sydney, the government approved the implementation of the city’s first rapid transit system alongside an integrated ticketing program and commuter parking options to improve Sydney’s integrated transport network (Transport for New South Wales, 2012).

For the past half century the development of the City of Cambridge (United Kingdom (UK)) and surrounding areas has been severely constrained due its rapid growth which has resulted in a severely congested road network (Cambridge Futures, n.d.). Employment, especially manufacturing, was encouraged to locate or relocate elsewhere in the region. The growth of Cambridge city began when it was connected to London by the M11 in the early 1980s, and to the north by the M1/M6 highways. Major expansion of the electronic ‘High Tech’ industries began to flourish due to the close proximity to the universities and good communications. Tourism and other service industries also experienced major expansion. As a result, the growth rate of population and industry in the City was one of the highest in the UK for over a decade, and now stands at over 200,000 (from 108,000 in 2001). As a result of this rapid growth, traffic also increased by 47% on the main radials into the city and issues relating to congestion and transport options for residents became significant (Oldridge, 1995). As a mature city, Cambridge has been challenged to develop innovative measures to maintain and improve economic development in this area and to ensure social equity.

Hong Kong is a mature city which, despite having an efficient integrated public transport system, experiences major issues relating to vehicle access around some of the old and upcoming commercial centres. Access lanes or restricted times, for example for deliveries, coupled with pedestrian movements continue to present challenges in some locations. Car journey speeds on some major traffic corridors during weekday morning peak hours are recorded to be around or even lower than 10km/h, which is marginally faster than the average walking speed of an adult at 4 to 5 km/h and a lot less “green”. To address these issues, the Hong Kong Government has been striving to foster a safe bicycle-friendly and pedestrian-friendly environment in new towns and new development areas for short-distance travel or leisure purposes, demonstrating that even mature cities with efficient rapid transit networks are privy to improvement opportunities.

3 DERIVING SUSTAINABLE TRANSPORT SOLUTIONS

Sustainable transport solutions need to cater for the mobility needs of the current and future generations to support economic activities while minimising impacts on the environment (Haque, Chin & Debnath, 2013). Options include but are not limited to integrated land-use and transport planning, designing compact city plans, implementing transit-oriented development, increasing green mass transit solutions, increasing active transport and incorporating environmentally friendly technologies into transport planning. All of which are enjoyed to some extent in Hong Kong. A number of innovative smart technologies are readily available to be incorporated into the transportation systems to make them more effective even in the mature city environments.

Common transport supply measures include rapid transit systems, bus rapid transit systems, integrated rail and bus services and light rail systems. These obviously require analysis with respect to costs, affordability, safety and cultural acceptance, in addition to which physical impediments such as existing land uses, built form of existing narrow streets, safety of transport corridors, congested utilities both above and below ground are also essential elements in a decision making process. As do opportunities for revitalisation and changes to the living and working environment which can contribute to building a well-connected global and vibrant city. The following section discusses options to develop sustainable transport solutions, through built infrastructure solutions, government policies and innovative technologies.

3.1 *Determining the Optimal Infrastructure Solution*

Some mature cities have overcome transport issues through innovation and the use of exemplar planning approaches (Curitiba in Brazil for example). To demonstrate how a multi-criteria decision making tool can assist in the planning process, reference is made to the use of Mott MacDonald’s Sustainability Decision Model (SDM) where the SDM allows multifarious options to be considered, analysed, modified and retested in terms of sustainability performance and a rational basis for analysis is provided as an output. The tool allows the integration of sustainability into the planning or design optioneering phases of a project such that the outputs can be carried over into the subsequent design, development and implementation phases. Such tools have been used on many projects which consider integration of rapid transit or light rail systems into existing towns/cities, integrated railway and highway schemes, water based transport and air based transport schemes.

3.1.1 *Case study 1: Masdar City*

Masdar City was envisioned as a sustainable, carbon-neutral and zero-waste city that would be totally reliant on on-site renewable energy and become the first eco-city of the world. In terms of transport, key challenges facing the infrastructure project team included the interaction with service corridors, aesthetics, and long term changes in demand and modal shifts.

Mott MacDonald’s Sustainability Decision Model (SDM) was conceived as a tool for supporting cost-effective decision-making through the integration of sustainability principles into the infrastructure component design. The tool assessed various technically feasible design options, recorded and tracked engineering design decisions and provided a transparent and consistent framework to integrate sustainability into infrastructure design. Through stakeholder engagement, the tool ultimately set relevant priorities with client buy-in and assessed options by way of a multi-criteria decision analysis approach. Figure 1 below outlines this process and demonstrates the structured approach to decision making using sustainability objectives.

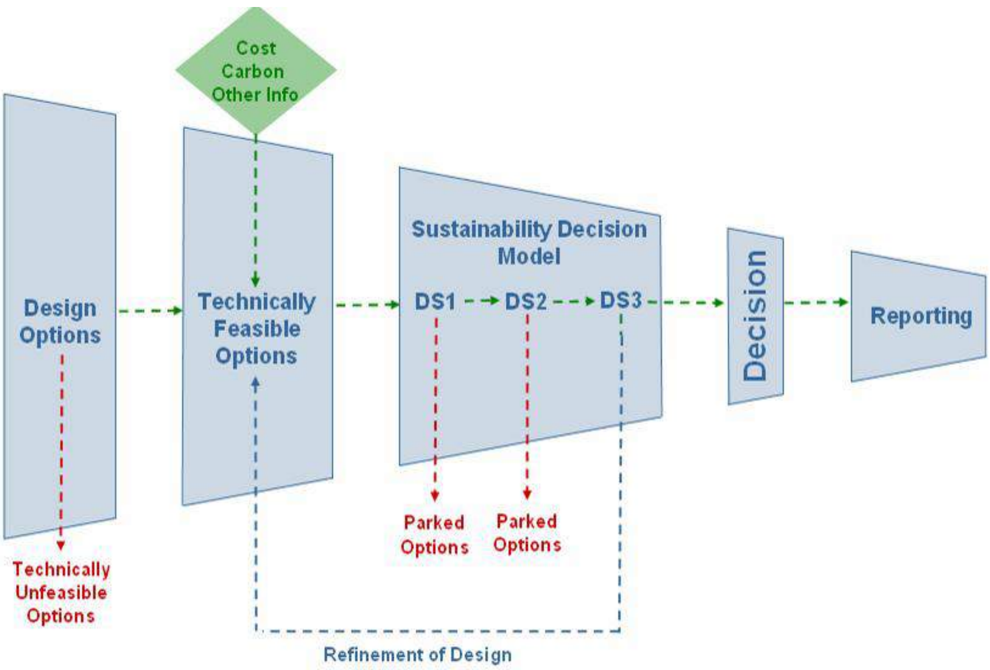

Figure 1 Sustainability Decision Model approach through the design optioneering phase.

As a result of this process, Masdar City's transportation network has been developed to connect with the external network.

3.1.2 *Case study 2: Leeds New Generation Transport*

A key component of the Leeds transport strategy was the integration of a light rail project into the city's existing network to help meet the growing demand for sustainable travel whilst underpinning social inclusion and economic regeneration initiatives. The proposed Leeds New Generation Transport (NGT) system comprised a core three-line radial network of approximately 20km connecting to a city centre loop; with further lines to be considered as part of ongoing strategic transport plan. The baseline scheme comprised the first modern trolley bus system in the UK and tram options were also considered as part of the appraisal process. The NGT network was planned to include a mix of on and off street running with significant corridor capacity enhancements and traffic management (Mott MacDonald, 2014).

Sustainability input was provided into the options evaluation exercise for the scheme through the use of Mott MacDonald's in-house Sustainability Decision Model (SDM), to assess each of the Leeds NGT options. The assessment was based on a set of objectives derived from the UK Department for Transport's web-based multimodal guidance on appraising transport projects and proposals, but was expanded to include additional social and economic dimensions. Objectives and criteria were prioritised and weighted and each option was then scored against each objective to determine their sustainability performance.

As a result of this process the final outcomes relating to route, mode and operating strategy options were developed as part of a staged appraisal using the SDM process to provide sufficient design definition and a robust case to government decision makers. Whole project lifecycle outcomes, including initial strategic fit, through to the planning, design and implementation stages were analysed using the Sustainable Decision Model.

3.1.3 *Case study 3: East Coast Main Line (ECML)*

The ECML is one of the busiest and most successful railway lines in the UK providing services from London to Edinburgh and beyond. Network Rail (United Kingdom) was faced with the challenge of improving the efficiency of passenger and freight services to meet the future demands for a low carbon transportation system. The scheme involved the construction of a three kilometre (km) railway flyover over the ECML and a separate highway bridge. Once operational, the scheme will provide a direct link between two key freight railway lines, removing slow moving freight trains off the ECML and provide greater capacity and efficiency on the ECML for high speed passenger trains.

Throughout the complex, evolving design process, detailed and thorough consideration was given to key aspects of the scheme including the structural options and embankment fill options. Mott MacDonald tailored its Sustainability Decision Model (SDM) for the scheme and worked together with Network Rail to develop a total of thirteen customised project sustainability objectives for the scheme, covering aspects such as materials management, climate change adaptation and health and safety that were aligned to cover all the topics in Network Rail's internal sustainability policy. These objectives formed the basis of the SDM for the detailed design optioneering process by which the scheme proposals were tested against to determine whether the contribution towards sustainability could be improved. Ultimately, the SDM provided a sustainability score for each of the proposed options, and the estimated construction cost and embodied carbon were also considered when selecting the final option.

The SDM proved to be a key aspect of the decision making process and a valuable mechanism for integrating sustainability into the planning and design of the project and therefore being able to consider the long-term improvements against social, economic and environmental objectives. In adopting the SDM for the project, the tool served to change the way design teams think about sustainability and also built capacity in the client organisation in terms of awareness raising and systems and processes (Leather, Parker, Knight, 2011).

3.1.4 *Case study 4: Abu Dhabi Surface Transport Plan 2030*

The Abu Dhabi Government commissioned a study to address transport issues arising from population growth, urban expansion, industrial development and the rise of Abu Dhabi as a tourist destination and a global hub. Abu Dhabi has a diverse and multicultural society and solutions to its transport challenges had to take account of the social customs, environmental factors relating to hot climates and economic aspects relating to fuel prices. To address these issues, sustainability aspects were considered central to the approach in developing Abu Dhabi's future surface transport plan and a sustainability framework was developed to assess transport options and scenarios. From the outset of the project, the three pillars (economic, social and environmental) of sustainable development were embedded in technical deliverables and consultation exercises.

For the Abu Dhabi Surface Transport Plan, climate, culture, existing transport systems, social acceptance of change and responsiveness to change options were considered priorities. A proactive approach was taken using a bespoke Sustainable Decision Model which included aspects relating to energy conservation, material use and sourcing, noise and air quality, social equity, economic efficiency and ecological enhancement and protection (Mott MacDonald, 2009). As a result of this process, final transport options sustainable solutions largely depended on environmental and social factors and included bus rapid transport, light rail and metro systems along with improved road networks and connections. These options were tested through a rigorous process based on sustainability performance of options using the Sustainability Decision Model.

3.2 *Policy Solutions*

With the lack of physical space, resources, and time to expand transport infrastructure to cope with the ever-increasing demand for greater transport capacity, mature cities have needed to resort to other means to improve existing transportation networks and systems. Recognising this issue has brought about new strategies and schemes that help minimise congestion and increase transport efficiency, and through the intervention of local, regional, and governmental policy has been able to drive successful implementation in some cities.

Sustainable and innovative transport strategies that have been developed can be generally categorised as those that: (i) make transportation systems more efficient; (ii) provides more travel options; (iii) provides commuters with accurate and more connected information; (iv) makes pricing and payment more convenient; and (v) reduces journeys and traffic (Transportation of America, 2010). Enhanced transport strategies have been adopted by megacities worldwide, and a number of these strategies have been discussed in the following case studies.

3.2.1 *Case study 5: Minnesota Urban Partnership Agreement*

As part of the Minnesota Urban Partnership Agreement (UPA), strategies have been implemented to set priority lanes for buses and prices have been set to safeguard free-flowing traffic by requiring cars with one occupant to pay a toll to access the high occupancy vehicle lanes during peak hours. Some of the toll revenue has been used to fund fare discounts for commuters who use the new bus rapid transit network. Another element of the UPA is enhancing telecommuting, where the existing Results-Only Work Environment programme has employers agree to provide employees the flexibility to telecommute or shift their hours to avoid congested commutes. Administrators are targeting large employers in the region to participate in the programme with aim to reduce 500 daily peak-period trips (Minnesota Department of Transportation, 2013).

3.2.2 *Case study 6: Intelligent Strategy for Transport, Finland*

Finland already first published a national Intelligent Transport Strategy (ITS) back in 2009, and in 2013 has released the second generation ITS that focuses on four main areas of: (i) improving the level of transport-related information services; (ii) improving the productivity and efficiency of the transport system; (iii) implementation of the new transport policy through pilot projects; and (iv) promotion of ICT innovation and opportunities. The ITS will be implemented via a number of key projects to be primarily carried out as joint venture projects between the public and private sectors, costing a total of approximately EUR 300 million between 2013-2017. The ITS will gradually put into practice the EU White Paper’s principles of where the “user pays” and “polluter pays” (Ministry of Transport and Communications, 2013).

3.2.3 *Case study 7: City-wide bicycle sharing schemes*

As an alternative to region-wide strategic movement in transport, many major cities have been promoting bicycle sharing schemes on a more local scale. For example, Central London has introduced a bicycle-sharing scheme since 2008, which has been successfully implemented due to pro-cycling leadership on a political level. The scheme allows people to hire and ride a bicycle at an affordable rate that consists of a basic bicycle rent rate plus the first 30 minutes free and charges applying thereafter. Returning the bicycle is also made easy as it can be returned to any one of the 700 automated docking stations located across the city. (Transport for London, 2014).The scheme provides an environmentally friendly alternative mode of transport for daily commuters and also enhances the city experience for tourists and visitors.

Many other cities worldwide have implemented similar schemes to promote bicycle riding to improve mobility for the public at a relatively low cost with the minimal construction required, and also improve on the quality of life by reducing air and noise pollution and congestion.

3.3 *Innovative Technologies*

A growing trend in improving transportation has been to revitalise existing transport systems, rather than investing in whole new infrastructure projects. This has led to the development of various innovative technologies that can be integrated to existing transport systems to improve the commuter’s experience and system efficiency.

3.3.1 *Real Time Passenger Information and Intelligent Transport Systems*

Real Time Passenger Information (RTPI) has increased its application for modes of urban transportation including bus services, train systems, and tram networks. Basic functions of RTPI serve to let passengers know when the next ride is scheduled to arrive and informs how long the wait will be. However, with technological advancement Intelligent Transport Systems have been developed with the integration of Global Positioning Systems (GPS) to track vehicle location, Urban Traffic Control System (UTC) to coordinate traffic flow, and Automatic Number Plate Recognition (ANPR) to provide mass surveillance. These various systems are linked together through a common database to provide traffic survey data and management information and enable appropriate responses to be made to improve road and rail traffic management (Onslow, 2014).

3.3.2 *Case study 8: Storstockholms Lokaltrafik integrated travel information system*

In Stockholm, the drive for more efficient public transport information is partly due to the rapid increase in population, and also other external factors such as imposing new restrictions on road traffic, stricter environmental regulations and the rising cost of driving. Hence, it is essential that public transport steps up to meet the public's growing demands and expectations. The Storstockholms Lokaltrafik (SL), the organisation running all of the land based public transport systems in Stockholm County, carries over 700,000 passengers on more than 2.44 million journeys every working day and incorporated an integrated travel information system in 2007 named "JustNu", which translates to "Right Now". The system combines real time transport information through satellite communication and disseminates through the Internet, ensuring that commuters are able to obtain up to date information on specific times rather than scheduled times, journey diversions and route disruptions, thus optimising public transport operation and enhancing resource use. (AB Storstockholms Lokaltrafik Annual Report, 2007).

3.3.3 *Case study 9: Mobility on demand system, Helsinki*

A "mobility on demand" system is currently under development in Helsinki to transform its conventional transport system into a smart public mobility network by 2025. The mobile application aims to merge journey planner and payment platforms together with all information related to public transport, driveless cars, buses, ferries, taxies, etc. Ultimately the whole system will act as a public utility when developed and will encourage less dependency on car ownership (Greenfield, 2014).

3.3.4 *Case study 10: Getaround Inc. smartphone application*

Getaround Inc. is a California-based company that enables sharing of the 280 million personal vehicles in the U.S. that sit unused 93 percent of the time, by making use of a smartphone application that matches users with available cars within walking distance. This concept increases options for non-car owners without adding more vehicles to the road and this shift to innovative car-sharing is set to expand to more cities and suburban areas across the US (Transportation of America, 2010).

3.3.5 *Case study 11: Pittsburgh ACCESS service*

In Pittsburgh, the integration of technology has made public transportation for the elderly and people with disabilities more economical, convenient, and shortens waiting time through its ACCESS service. The service is a door-to-door, advance reservation, shared ride transportation service in Pittsburgh and the surrounding area, allowing the elderly and those with disabilities to be able to live and travel independently. The use of a low-cost, real-time information system to support drivers' schedules and improve arrival times, whilst providing detailed information to customers has enabled ACCESS to become an efficient service with an average of 5,800 weekly journeys (Transportation of America, 2010).

4 ANALYSING THE EFFECTIVENESS OF THESE SOLUTIONS (MATURITY MODEL)

In addition to sustainable transport solutions relating to infrastructure, innovative policy and smart technologies, some organisations gauge the success of their measures using a maturity model. A strategic maturity model is a tool that can be used to measure the sustainability performance over the longer term. This approach places continuous improvement at the centre of organisational or project performance. Maturity models are a part of BS8900: 2006 Guidance for Managing Sustainable

Development and it is proposed that bespoke maturity models could be used to help illustrate how outcomes of the sustainable transport solutions perform over time.

As an example, MTR Corporation (MTR) use maturity models in its annual reporting to measure and present corporate sustainability performance against global best practice. MTR has identified criteria against which the organisation can measure its performance and develop a road map for improvement. Figure 2 represents MTR's maturity matrix:

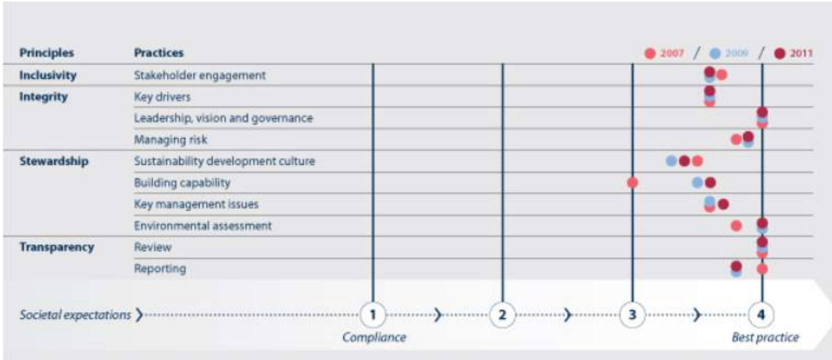

Figure 2 Maturity matrix

In this case, performance is measured through a series of principles that benchmark strategy against accepted universal principles that drive sustainable development (MTR Corporation Limited, 2012). Progressive steps from Compliance to Best Practice are defined by sets of initiatives within the Materiality Map of the BS8900: Guidance for Managing Sustainable Development. The resulting performance can be illustrated in annual plotting of performance at year-end to illustrate the effectiveness of management action and resource. Similarly, this approach can be applied to individual transport solutions, with criteria outlining success provided through levels of improvement or achievement to show the improving performance of the solution.

5 CONCLUSION

In conclusion, this paper identifies some of the challenges occurring in mature cities which have put pressure on the need to improve existing transportation systems that have been outgrown over time. It is evident that no solution is able to fit all cities, but the fundamental tenet is that solutions must be geographically and culturally relevant, robust and include affordable options for users. A range of sustainable transport solutions have been explored, including planning approaches that highlight the importance of integrating sustainability in the design process; solutions advocated through policy changes; and the use of innovative technologies to provide efficient transportation and real-time information to improve commuters' experience. The examples show that it is not necessary to focus on improving major transport infrastructure alone, and simple solutions can also have long term benefits, such as the integration of technology provides accessibility, convenience, and relevance to modern day society. The development in mobility options today promises to be far reaching in terms of social, economic and personal terms – and if the visionaries are to be believed it will result in a rethink of the way people view their second most valued possession (after their home).

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Temburong Bridge – A new 30km road link for Brunei

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Keywords: infrastructure development; sea crossing; viaducts; cable stayed bridge; Eurocode.

ABSTRACT: The new 30km Cadangan Projek Jambatan Temburong (Temburong Bridge Project) in Brunei Darussalam will connect the relatively isolated district of Temburong with the more developed Brunei-Muara district. Improved connectivity will enhance the movement of labour, goods and services to and from Temburong, and will facilitate the development of eco-tourism in the area. The paper describes the objectives of the project, the project planning, the procurement strategy and the design for the marine structures including precast segmental marine viaducts and 2 concrete deck cable stayed bridges. Design has been carried out to the Eurocode design standards. Innovative elements of the project planning and design are highlighted.

1 INTRODUCTION

Temburong District is isolated from the rest of Brunei by the Brunei Bay to the north, and Malaysian state of Sarawak to the south, east and west. Its only land-based access is the road that passes through Limbang, Sarawak. Its isolated location and lack of connectivity with the main commercial districts of Brunei and the associated port and airport infrastructure essential for global trade and commerce has constrained the economic growth in the district.

The Cadangan Projek Jambatan Temburong (Temburong Bridge Project) will connect Temburong with the more developed Brunei-Muara district. Improved connectivity will enhance the movement of labour, goods and services to and from Temburong, and will facilitate the development of eco-tourism in the area.

The new 30km link will include marine structures across the Brunei Bay comprising 14.6km long marine viaducts and two cable stayed bridges, as well as 12km of elevated structures across the Temburong peat swamp forest and a small area of mangroves, and approximately 3.6km road in Brunei-Muara district where 3 lengths of tunnels are required as well as at-grade roads and viaduct ramps to link with the existing road network. The construction will commence in early 2015 with completion targeted in 2018. Key challenges include very soft ground conditions, shallow waters, difficult access, lack of local raw materials and the required fast-track programme.

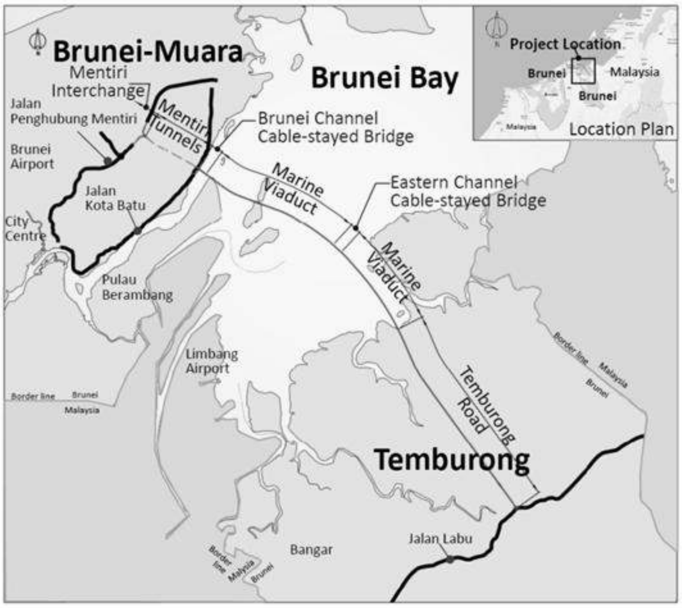

Figure 1 Layout plan of Temburong Bridge

2 OBJECTIVES OF THE PROJECT

The new link will bring significant socio-economic benefits to Brunei. These include:

a) Providing a road transport link for the movement of labour, goods and services to and from Temburong, hence encouraging more business activities and development there.
b) Allowing direct movement of international visitors into Brunei and onwards to Temburong, which has been earmarked as Brunei's centre for eco-tourism. Potential tourism clusters can be developed that include not only Brunei but also surrounding Malaysia, by developing packages that provide a range of locations and sights within the one visit. This has downstream effects to Brunei tourism sector as a whole including the national airline, hotels and restaurants which potentially could capture a wider tourism market that visits Temburong and onwards to other areas.
c) Increasing mobility of the workforce so that more citizens can settle in quiet Temburong and yet work and enjoy leisure time in the capital or other parts of Brunei. Over time, it is expected that the link will allow a resident population within Temburong to establish and grow with it commercial and industrial sectors and the necessary social infrastructure such as medical and educational facilities. This will potentially result in greater social and economic development for Brunei as a whole.
d) Providing easy land transport from Temburong to the port in Brunei-Muara, or to the shops and markets in other districts of Brunei. At present, farmers in Temburong practice subsistence agriculture rather than commercial agriculture. Temburong can be transformed into the country's source of commercial agriculture to help diversify the country's oil-based economy and help in reducing food commodity imports.
e) Providing a strategic link to allow Brunei emergency response teams to be dispatched to Temburong quickly, or vice versa.

3 PROJECT PLANNING

The Government of Brunei Darussalam has recognised the geographical separation and lack of connections to and within Temburong which have constrained development and growth within the district and development of its full socio-economic potential. In 2010, Arup was commissioned to carry out the feasibility study to provide a road and bridge link between the districts of Brunei-Muara and Temburong.

A route selection exercise was carried out to determine the most favourable route across Brunei Bay as well as the optimum connection points in both Brunei-Muara and Temburong districts. The route selection study took into account a comprehensive range of criteria, including transport planning, engineering, land matters, environmental issues, impact on local community, programme and cost. During this exercise, land use and environmental information were collected in Geographical Information System (GIS, see Figure 2 for screenshot) to better understand the relationship between alignment options and various constraints. Stakeholders were also consulted to understand their opinions and other constraints.

After the route selection exercise, more in-depth engineering assessment, cost estimation and cost-benefit analyses were carried out. The conclusion of the feasibility study was that the Project is technically feasible, economically viable and that the environmental impacts can be managed or mitigated and hence acceptable.

Following the feasibility study, the Government decided to proceed with the construction of the link. In early 2013, the Government engaged Arup to carry out detailed design and support services for the procurement of the Project, including the prequalification and tendering processes, management of the construction contracts and other associated engineering support.

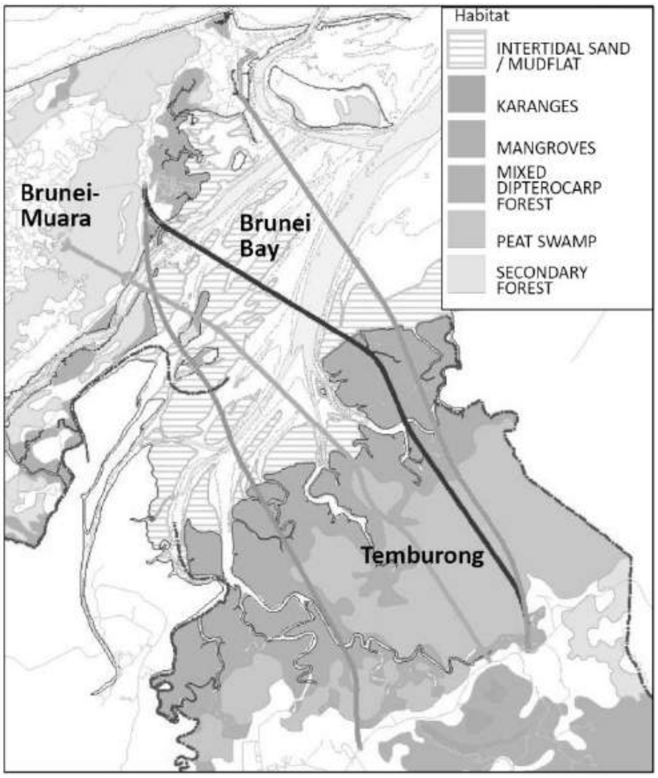

Figure 2 Alignment options and environmental constraints from GIS model

4 PROCUREMENT STRATEGY

In order to achieve the target opening date of 2018 for the project, a fast-track approach is essential. Delivering the project on a fast-track basis requires innovative contract packaging and phasing of works. Activities which might traditionally be carried out sequentially are carried out in parallel and the schedule is managed to focus on critical path activities and give priority to construction activities that take the longest time.

4.1 *Procurement Arrangement*

The Project is being procured as a traditional engineer-design, contractor-construct arrangement under various contract packages. These construction contracts are procured by selective tendering method. Following an Industry Awareness Day to promote the project to international contractors who may wish to bid, prequalification has been adopted, and the subsequent tenders invited from prequalified contracting teams only. One of the criteria for prequalification is the inclusion of a local contractor in the team. The prequalification and tendering process are carried out separately for each construction contract.

4.2 *Contract Packaging*

The civil works are divided into five construction packages to enhance competition and permit an early start to the construction works with the longest duration. This also gives more opportunity for local contractors to be involved in contracts suitable for their skills and experience. Five was the optimum so as not to introduce excessive numbers of interfaces.

Table 1 below summarises the key scope of each contract:

Table 1 Summary of contract packages

Contract	Key Contract Scope	Length
CC1 – Mentiri Interchange and Mentiri Tunnels	Grade-separated connection to Jalan Utama Mentiri. Tunnels across Mentiri Ridge; At-grade roads and viaducts between tunnels.	Interchange about 3.6km
CC2 – Marine Viaducts	Marine viaducts in Brunei Bay Turnaround facility at Pulau Baru-Baru; Provision for future connection to Pulau Berambang.	about 13.4km
CC3 – Navigation Bridges	Navigation bridges across the Brunei Channel and Eastern Channel; Viaducts between tunnel portal and Brunei Channel navigation bridge; Connecting ramps to Jalan Kota Batu; Administration building at Jalan Kota Batu.	about 1.1km (and slip roads)
CC4 – Temburong Road	Land viaducts and river crossing bridge; Turnaround facility; Reconfiguration of Jalan Labu at connection point.	about 11.8km
CC5 – Traffic Control and Surveillance System	Route-wide traffic control and surveillance systems.	Route-wide

4.3 *Tender and Construction Programme*

In order to achieve the target date, the programmed construction period is 45 months. This is governed by Contracts CC2 Marine Viaducts. Therefore this package was tendered first. It was estimated that Contract CC3 and CC4 would each take approximately 36 months to construct. These packages are therefore tendered in a second wave. By staggering the various contracts, the project procurement team and bidding contractors can focus on each tender one at a time.

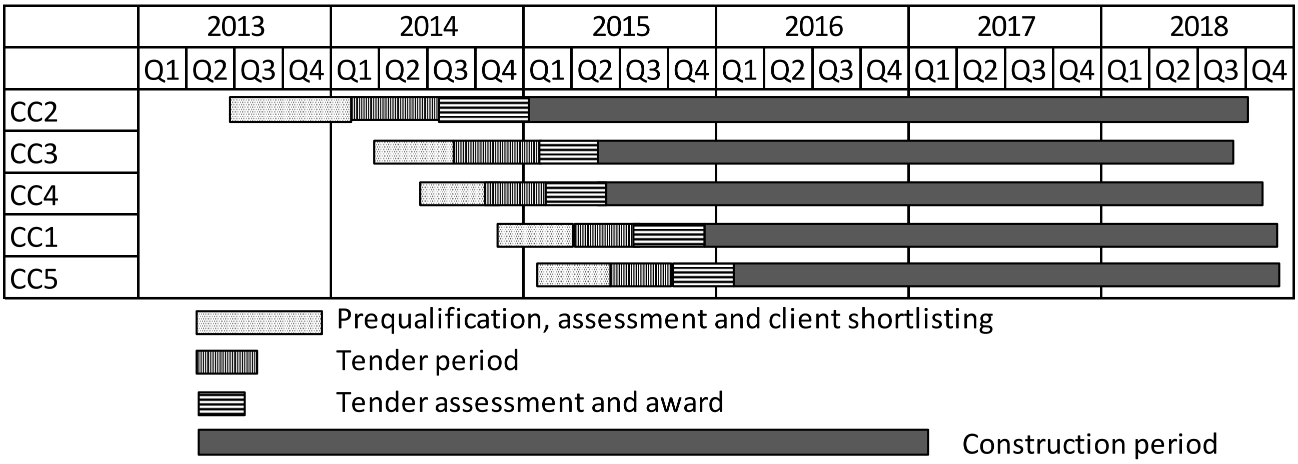

Figure 3 Tender and construction programme

5 DESIGN CRITERIA

5.1 *Functional Cross Section*

The Project will be a dual two-lane highway with the provision of paved shoulders, except that there is a short section of dual 3-lane carriageways between the future connection to Pulau Berambang and slip road connections to the existing road Jalan Kota Batu along the coast of Brunei-Muara. The wider section is required otherwise there will be merging traffic from slip road and diverging traffic from mainline leading to safety issues and congestion.

5.2 *Design Standards*

Historically Brunei has adopted the British Standards supplemented by local standards. As British Standards have been superseded by the Eurocode in the United Kingdom (UK), the Brunei authorities decided to adopt the Eurocodes with the UK National Annex for the project. These standards are supplemented with project and location specific requirements, in particular to take account of differences in climate and seismicity between Brunei and the UK. A project-wide Master Design Criteria was developed to clarify how the Eurocode rules shall be applied in the design, and supplemented with additional project specific criteria where necessary to cover issues such as seismic action, wind climate and ship impact.

The main loading considered are briefly described below:

a) General actions – Dead loads and superimposed dead loads are implemented in accordance with the relevant parts of Eurocodes and the UK National Annex.
b) Traffic action – The UK National Annex traffic actions of Load Model 1 and Load Model 3 – SV196 are adopted as other bridges in Brunei have been designed for UK Highways Agency BD37 loading previously. This ensures that the bridges in the transport network are designed for similar loading.
c) Wind action – A site-specific wind climate assessment was carried out based on historic data. Considering the thunderstorm phenomenon which could lead to sudden burst of high wind speed, the 10-minute wind speed with mean return period of 50 years is conservatively taken as 26m/s at 10m above mean sea level.
d) Thermal action – A statistical analysis of historical data was carried out and the maximum and minimum air shade temperatures are 40℃ and 15℃ respectively.
e) Action during execution – This is specified for each type of structure. In particular, parts of the marine viaduct substructures and the cable stayed bridges are governed by the execution stage loading.

f) Accidental action –The cable stayed bridges are designed for sudden rupture of stay cables and 50MW gasoline spill fire on bridge carriageway. Ship impact is discussed further in section 5.3.

g) Seismic action – A site specific probabilistic seismic hazard assessment was carried out for Brunei to establish the bedrock seismic response spectra (see section 5.4).

5.3 *Ship Impact*

Apart from the two navigation channels in Brunei Bay, the rest of the Bay is very shallow. Current vessel movements are very scarce in the Bay, with the largest vessel recorded being 36m long. Despite that there is no known plan for future port development, a larger 80m long rivertrade vessel was selected as the design vessel to safeguard future development opportunities.

The bridges are designed for ship impact in accordance with BS EN1991-7. As there is generally no specific rule on the probability based analysis in this standard, the methodology in AASHTO (American Association of State Highway and Transportation Officials) Guide Specification and Commentary for Vessel Collision Design of Highway Bridges was adopted to carry out the probability based risk assessment. A large number of vessels was assumed conservatively such that the design ship impact forces are essentially established in a deterministic way. For piers further away from the navigation channels, vessel collision associated with drifting vessels at the speed of water current has been considered for robustness.

5.4 *Seismic Analysis and Design*

Based on the probabilistic seismic hazard assessment, it can be established that Brunei is a low seismicity region based on the definition in BS EN1998-1. Despite this fact, considering the importance and scale of this project, seismic design is essential.

Seismic analysis and verification have been carried out in accordance with BS EN1998-1 and BS EN1998-2. The ground comprises thick layer of soft material and is hence classified as Type S, which means that site specific response analyses are mandatory to establish the ground displacement and design seismic response spectra (see Figure 8).

The bridges are designed for two performance levels - Ultimate Limit State with 975-year return period earthquake and Structural Integrity Limit State with 2475-year return period earthquake. In order to achieve this, the analysis and verification rules in Eurocode 8 are followed for the 975-year earthquake. Additional checks are then carried out for the 2475-year earthquake to ensure that there is no brittle failure of non-ductile structural elements and no unseating of superstructure.

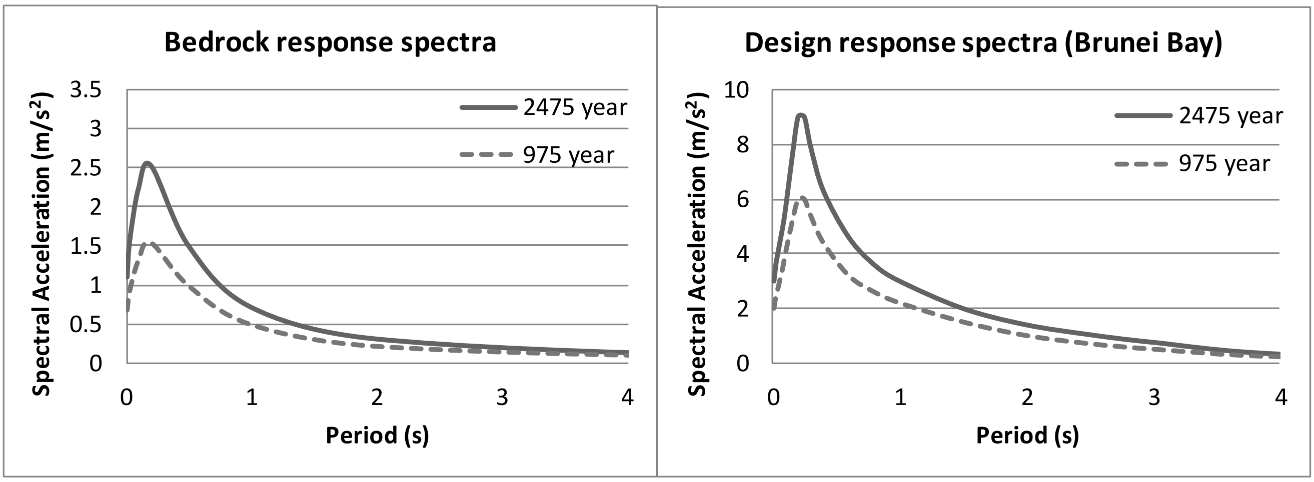

Figure 4 Bedrock and design response spectra

6 CC2 MARINE VIADUCTS

Contract CC2 comprises viaducts across Brunei Bay, which is shallow except at the two navigation channels. There are mud flats in the Bay, as well as mangroves near the landing point on Temburong side and other islands. The ground is very soft, with typically 20 to 30m of soft marine clay. The shallow waters, very soft ground conditions and the fast track programme pose significant challenges to both design and construction.

6.1 *Superstructure*

The viaducts will be in the form of twin post-tensioned concrete single-cell, box girders with exclusively 50m spans. Typical modules are formed by 6 spans of continuous deck. To fit between the contract boundaries, some modules are formed by 5 spans. These standard modules have typical box cell with 3.8m wide soffit. Figure 5 shows a typical cross-section of these modules.

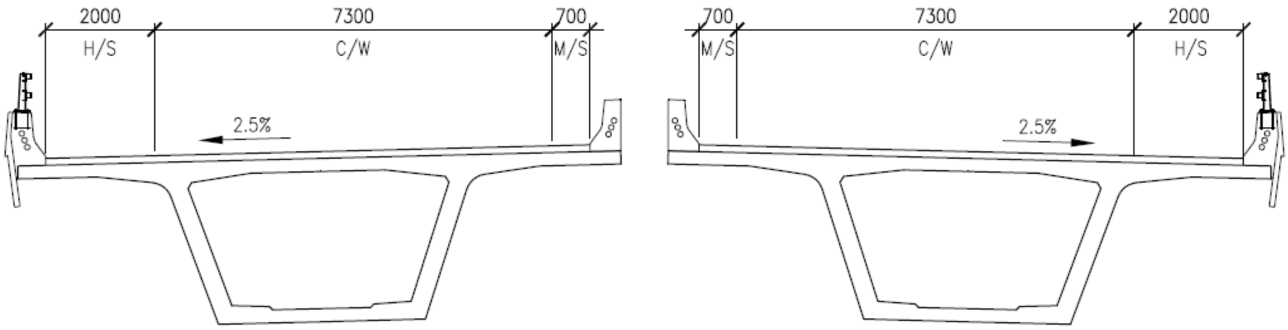

Figure 5 Typical cross-section of standard twin box girders

A turnaround facility at Pulau Baru-Baru, the dual 3-lane section and the transitions to these require a much wider deck area compared to the typical. This is achieved by adding a narrower box cell with 2.8m wide soffit at each side, which subsequently peel away as ramps.

The box girders will be post-tensioned with a combination of internal and external tendons. To achieve the 45-month programme, the viaducts are designed for construction by precast segmental span-by-span erection method. Construction by precast full-span launching method is also permitted. Continuity between separate spans is achieved by a 200mm wide cast in-situ stitch. This minimises the amount of in-situ concreting and more importantly, the narrow gap will facilitate the delivery of material such as precast segments along the erected deck.

6.2 *Substructure*

Due to the shallow water, dredging or temporary access bridge will be required to gain access to carry out piling works. A number of foundation options were studied. Bored piles were initially considered but to achieve the required programme and to minimise cost, it was decided that driven piles would be adopted instead. Three types of piles are used:

a) Majority of the piles will be 1m diameter concrete spun piles, which are precast circular hollow sections with high density concrete achieved by spinning during the manufacturing process
b) Steel tubular piles of 1m diameter are adopted for locations where hard materials are shallow
c) Steel tubular piles of 1.6m diameters are adopted at piers subject to higher ship impact forces near the navigation channels

To construct the piles rapidly, it is envisaged that contractors would drive the full length of 30 to 70m long piles in one piece on site. This will require the use of large piling rigs on barges, but with minimum draught, to limit the amount of dredging required.

The soffit of pile caps will be above the Mean High High Water level for ease of construction. The pier columns are solid for short piers and hollow for piers taller than 5.5m. To simplify construction, there is no pier crosshead and the deck will sit on the pier columns.

6.3 *Articulation*

One of the key drawbacks of the use of concrete spun piles is that they have limited capacity to resist bending and can behave in a brittle manner. Compound with the fact that the ground investigation information was not available until a late stage during detailed design, the design team decided to use high damping rubber bearings for all the bearings of marine viaducts. Not only this reduces the seismic force on the foundations, it also reduces the sensitivity of the design to the ground conditions.

7 CC3 NAVIGATION BRIDGES

There are two navigation channels along the marine section of the crossing – the 130m wide Brunei Channel and the 235m wide Eastern Channel. To cross these channels cable stayed bridges will be constructed - the Brunei Channel Bridge (BCB) will be a gateway between the Brunei-Muara and the Temburong districts and hence designed as an iconic bridge. The Eastern Channel Bridge (ECB) will follow a similar form.

7.1 *Brunei Channel Bridge*

The Brunei Channel Bridge (BCB) is a single tower cable stayed bridge with a 145m navigation span and a symmetrical 145m side span.

The all-concrete ladder beam deck is 37.2m wide and formed from longitudinal edge girders with transverse cross beams at 4.15m spacing. The stay cables are anchored into the edge girders. The edge girders and the transverse cross beams are post-tensioned.

The general arrangement is illustrated in Figure 6.

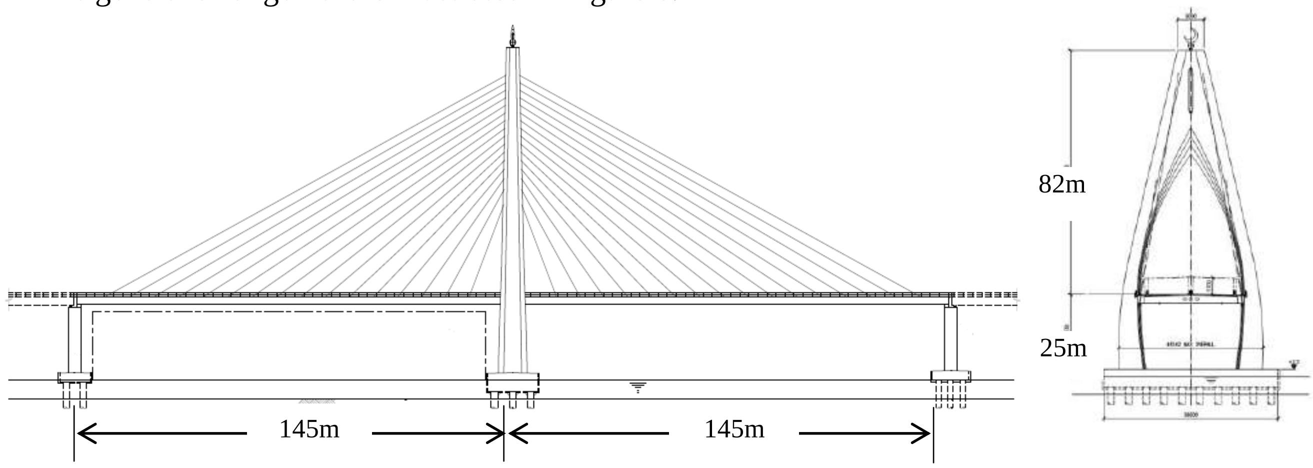

Figure 6 Brunei Channel Bridge general arrangement

The main tower is a sculpted A-shape, 107m tall, supported on a group of 2.2m diameter bored piles. Stay cable saddles support the cables through the towers. The deck is monolithic with the tower and the deck is supported on high damping rubber bearings at the end piers which also support the marine viaducts.

7.2 *Eastern Channel Bridge*

Despite a longer overall span requirement and differences in deck width, the Eastern Channel Bridge (ECB) is conceptually the Brunei Channel Bridge with two towers.

The main span is 260m with 130m side spans. The all-concrete ladder beam deck is 30.2m wide with a transverse cross beam spacing of 4.9m. The towers are of a similar form to the BCB tower, but slightly taller at 110.5m and slightly narrower to suit the deck width.

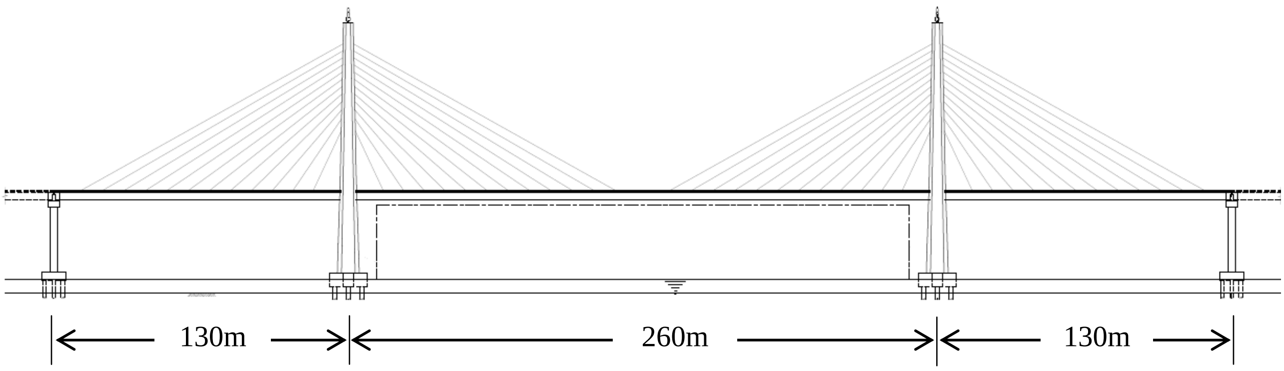

Figure 7 Eastern Channel Bridge general arrangement

7.3 *Parametric Analysis and Design*

The two cable stayed bridges have a similar architectural language which reinforces the idea of two bridges being part of the same infrastructure link. This has been translated into common shapes and details, even though the overall arrangement, span lengths and deck widths are different. The same structural system has been chosen for both bridges and the similarity of the detailing and the geometry will have a positive impact for the construction phase as, in the case the bridges will be built in series, the same equipment can be used on one bridge might be re-adapted or even re-used for the erection of the other one.

Common structural features of the two bridges have led to an efficient design process. In fact, given the geometrical and conceptual similarities between the two bridges, parametric modelling, analysis and design verification systems for them were effective and led to an optimized design of both structures. A common database to both bridges was created through which, by only varying the few distinguishing geometrical variables of the two bridges, it was possible to generate the structural modelling of them from a unique source. The common model generation was also tied with similar verification tools adopted for both structures and this process helped to improve the efficiency of the design process and reduce the total time spent on the design of both bridges.

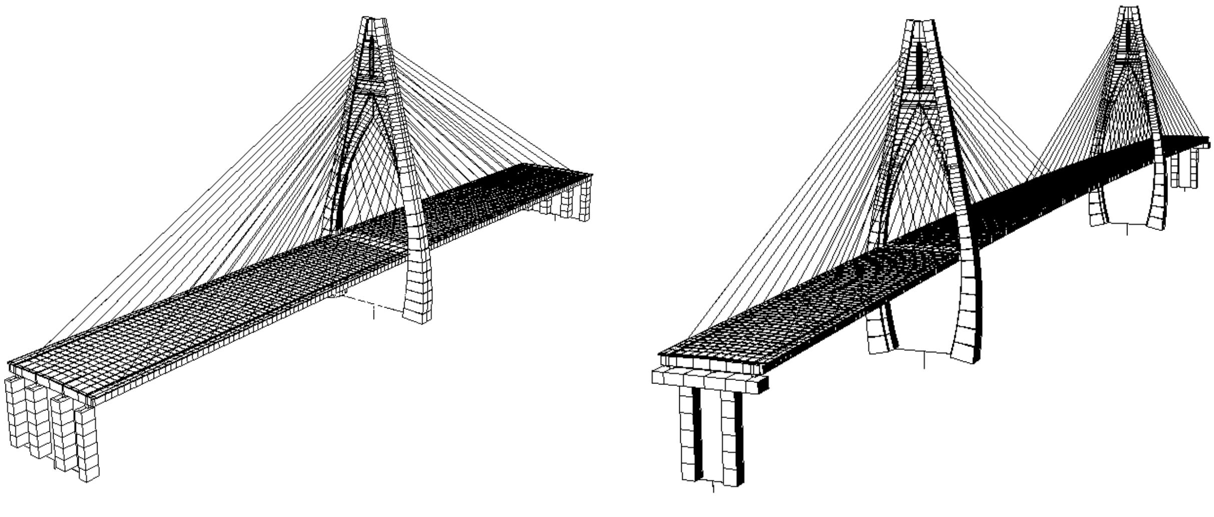

Figure 8 Structural analysis models for BCB (left) and ECB (right)

7.4 *Construction*

The tower construction sequence is conventional, but with some additional complexities due to the shape. A two-stage cast in-situ deck cycle is developed in order to limit the out of balance effects on the tower. The stay cables, which utilise saddles in the tower, are partially stressed against the formwork prior to pour 1 (Figure 9) of the east and west segments being cast, either simultaneously or sequentially. This is followed by pour 2 of each segment. Finally the second stage of stressing of the stay cables from both ends is performed.

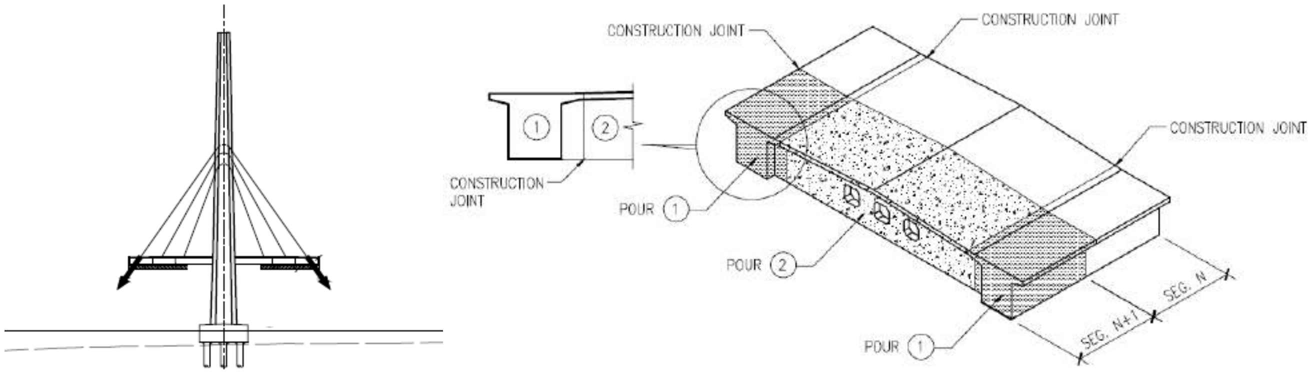

Figure 9 Deck casting sequence

8 CONCLUSION

The new 30km Cadangan Projek Jambatan Temburong will facilitate the development of Temburong and Brunei Darussalam as a whole. During the project planning stage, a comprehensive feasibility study was carried out, considering several route options to select an optimal solution. The project was evaluated to be technically feasible, economically viable and the environmental impacts can be managed or mitigated.

To achieve the opening target date, an innovative procurement strategy has been adopted with a fast-track programme. The marine viaduct construction is critical and has been designed in such a way that maximises the use of precast elements to minimise the construction time. The solution is precast concrete box girder erected span-by-span with the use of driven concrete spun piles and steel tubular piles. To make the bridge an iconic structure in Brunei, the two navigation channels will be crossed by two cable-stayed bridges with unique concrete pylons that incorporate Islamic architectural features. Efficiencies are possible in both design and construction due to the similar forms.

The Project is one of the first major projects in East Asia that is designed to Eurocodes.

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Calculating the effects of debris impact from first principles

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Keywords: debris; impact; rockfall; dynamics.

ABSTRACT: In extreme weather conditions such as windstorms and hailstorms impact by debris materials including hailstones, windborne gravels and broken tiles has generated a great deal of damage to building facades and roof coverings. Boulders rolling down a hill slope also constitute a significant element of hazards to the built-infrastructure following heavy rain. The same applies to water borne objects in flash floods. At the moment there is no clear guidance on how to estimate such impact actions as the emphasis in design codes of practices has been on wind pressure in a storm scenario. Difficulties with codifications for impact actions by solid objects stem from a lack of fundamental understanding of the underlying principles. Civil engineers have been educated and trained to analyse the effects of a given load pattern (such as wind pressure). However, the loading function cannot be pre-defined in an impact scenario as much depends on interactions between the debris object and the target being struck. This paper presents a new approach to the analysis of impact actions. It is shown that many common impact scenarios can be analysed by a page of simple hand calculations whilst more complicated problems can be solved by employing newly developed experimental methods.

1 INTRODUCTION

Most structural analyses in a contemporary design office are done by the computer. Meanwhile, civil engineers are expected to exercise their basic skills in the evaluation of computer output to ensure that static equilibrium is satisfied and that bending moment values in beams and columns are within constraints of (well known) solutions that are based on idealised conditions. There is clearly a desire by engineers who are responsible for the safety of the structure that they design to be able to interpret, and confidently quantify, various actions from the environment in a form that is familiar to them. For example, wind actions are still commonly considered as static forces in the design of most structures even though computational aerodynamics is well established. Factors such as drag coefficients have been used to incorporate the effects of complex dynamic phenomena in order that analysis of wind actions can still be based on statics. The amount of pressure generated by the flow of a medium can be estimated when its density and velocity of flow are known. Thus, dynamic actions associated with water flow, and mud flow, can also be represented in the form of dynamic pressure (like wind pressure) in order that static analysis can be applied. A dynamic amplification factor (commonly taken as 2) has been recommended to represent the effects of an abrupt application of loads.

In a similar manner, equivalent static forces have been prescribed by highway codes of practices to simplify collision actions in static terms even though the phenomenon involved is actually much more complex (AS5110.2, 2004; BSI, 2008; AASHTO, 2012). The desire for transparency in design and analysis is driven by the need of the engineer to retain good control of the design process. This best practices norm is underpinned by the expectation that the engineer in charge must always be able to verify results no matter it is done by hand, or by the computer.

A commonly used alternative approach to analyse the effects of an impact is to equate kinetic energy delivered by the impactor to the energy of absorption of the barrier (or the target) which receives the impact (e.g. Yang et al., 2012). Equations (1a) and (1b) are, well established, expressions used in highway codes for practices for finding deflection of the target (Δ) and the corresponding equivalent static force (F) based on energy principles as described.

$$\Delta = \frac{mv_0}{\sqrt{km}} \tag{1a}$$

$$F = v_0\sqrt{km} \tag{1b}$$

where m is the mass of the impactor;

v_0 is the incident velocity of the impactor;

k is the stiffness of the targeted element (eg. parapet) which is subject to the impact.

It is uncertain if equations (1a) and (1b) can be used for checking the impact resistance of components other than metal parapets in highway accident scenarios. Equation (1a) appears to be able to also provide an estimate of the amount of deflection of a column, for example, which is subject to the impact of a fallen rock; and equation (1b) is the amount of equivalent static force to emulate the effects of the impact on the column including bending moments and shear forces. What are required for input into the two equations are simply the mass and incident velocity of the fallen rock and the horizontal stiffness of the column which is struck.

However, the basis of these simplified methods, and their limitations, are not all that well understood by engineers and many of them have not been properly verified by research that is reported in peer review literature. In a similar fashion empirical models have also been developed to estimate the value of the peak blast pressure generated by an explosion. Because of uncertainties with the generality of recommended expressions, the analysis of extreme actions such as effects of impact and blast are normally undertaken by rigorous numerical simulations involving non-linear finite element analyses or by testing of the prototype in the field, or in the laboratory (e.g. Gabauer et al., 2010; Zinedddin & Krauthammer, 2007).

Sophisticated finite element software (e.g. program LS DYNA) can be used to simulate impact actions but there are uncertainties over parameter values for input into the analysis for characterising dynamic properties of the impactor and target. Verifications of analytical results are usually beyond the means of designers and system developers. Impact experiments by the use of a dropweight, or gas gun, for accelerating specific impactor object on the surface of a target are common but this type of testing is mostly undertaken to check compliance and to observe permanent damage to the specimen, and is seldom performed repetitively to model variability. Information obtained from physical experimentation has been very restrictive given that it is specific to the impact scenario and target sample employed in the test setup. Consequently, little is known of the damaging potentials of projected scenarios of debris impact. Limitations with current assessment methodologies as described are major obstacles in the development of effective, and reliable, approach to disaster mitigation during the service life of the building. The scope of this paper covers impact by storm debris and hailstones on glazing panels and metal claddings, and also fallen rocks from a hillslope onto the vulnerable parts of the building support.

2 THINKING OUT OF THE BOX

Most of the tasks used to be done manually are now performed electronically. Whilst this makes sense in view of improved efficiencies fewer engineers are now able to evaluate results reported by advanced computations including those to assess the effects of blast and impact. The common strategy adopted by vendors of computational tools is offering the users “total solution” to the problem in one analysis (e.g. Timmel et al., 2007; Fan et al., 2011). In a total solution every details of stresses, strains and deformation are reported and this requires full details of both the target and the impactor. The simulation outcome of the total solution is often a 4D display of how the damage would evolve in the course of the event along with the 3D image of the damaged component. This type of simulations is certainly appealing to the eye but the validity of a model is not reflected in its appearance. The merit of the model is in how it behaves, and not in the presentation of the solution. The onerous is on the user to type in all the correct instructions into the program. The literature offers little help in this regard given that articles which present results of analyses do not usually provide a listing of input parameters along with explanations of the choice of the values. The myth held by many designers, and stakeholders, is that the accuracies of the solutions is controlled by the capability of the software.

In reality, the accuracies of the solution to an analysis is limited by (i) assumptions made in the analysis (ii) knowledge on the potential environmental actions and the target the information of which are required for input into the software. Adopting a powerful software can be counter productive if knowledge on the input parameter is lacking. On the other hand the more is understood of the underlying phenomena the simpler the analysis. As for many well known methods of analysis in the field of solid mechanics predictive relationships which were originally derived from first principles can be simplified to address what matters most whilst accepting some errors. Acceptable solutions for impact actions can be derived from first principles too, or modified from existing expressions to take into account the effects of the key influential factors.

An impact action can be resolved into the global deflection demand (i.e. impulsive effects) of the impact and the localised contact force. It has been shown that impulsive actions of the impact can be estimated by equation (1) provided that it has been modified to take into account energy losses on impact. The deflection of an element (e.g. a column) resulted from the impact can be calculated by the use of simple algebraic expressions. An equivalent static force could then be applied to generate deflection to match with that estimated from energy principles. Stresses, strains and deformation so obtained from the static analysis could then be taken as solution to the impulsive effects of the impact.

The amount of contact force generated by the impact is much higher than the equivalent static force (or quasi-static force) because of interferences from the inertia forces (Figure 1). The contact force value is another critical piece of information for it controls the piercing of metal cladding or the probability of damage to glazing facades. However, common calculation methods based on energy principles can only be used to quantify the impulsive effects of the impact but not the contact force. The harder the impactor material the shorter the duration of contact and hence the higher the amplitude of the contact force in delivering a given amount of momentum from the impact. Representative information in relation to such hardness properties can only be obtained experimentally and such experiments would need to be repeated to allow for variability between impactor samples. Example contact force and reaction force time-histories which were measured from a specially made device in a simulated hailstone impact is shown in Figure 2 (Sun, 2015). Note the difference in scales used for quantifying contact force on the left and reaction (quasi-static) force on the right. The very different duration of the two types of forces should also be noted.

The rest of the paper is divided into two parts: (i) introducing the modified energy approach for calculation of the impulsive effects of the impact and (ii) introducing ways of solving for the magnitude of the transient force developed at the point of contact between the impactor and the surface of the target.

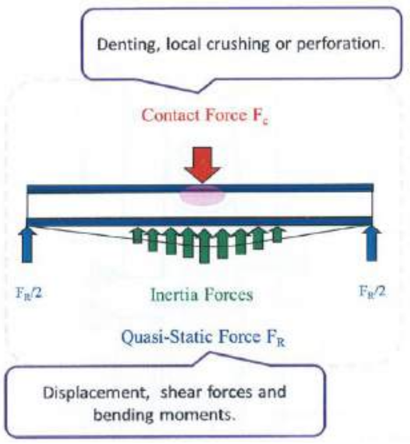

Figure 1 Impact forces

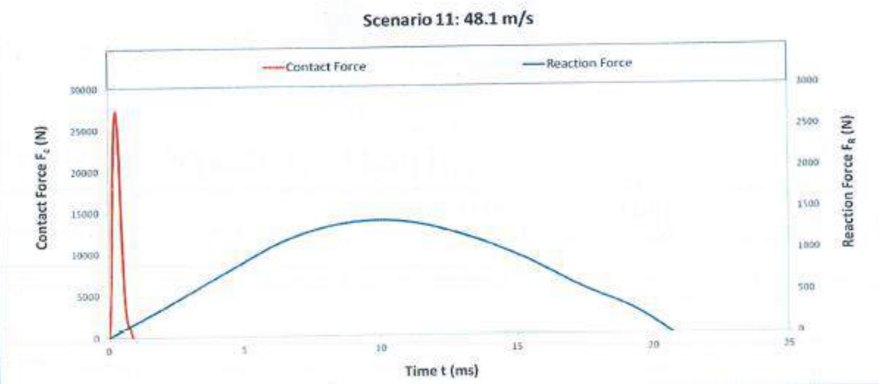

Figure 2 Example impact force time-histories

3 IMPULSIVE EFFECTS OF IMPACT

For estimating the impulsive effects of an impact the following relationship was derived by considering the principles of equal momentum before and after contact is made (as represented by the terms to the left and right of the equal sign respectively) as shown by equation (2).

$$m v_0 = (\alpha m) v_2^{'} - m v_1^{'} \tag{2}$$

where m is the mass of the impactor;

v_0 is the incident velocity of the impactor;

αm is the generalised mass of the targeted element;

$v_2^{'}$ is the velocity of the target following impact;

$v_1^{'}$ is the velocity of the impactor on rebounce in opposite direction.

Equation (1) is strictly speaking only valid for impact between two free bodies in space, and can be adapted for cases where the targeted body is not a free body but supported by a spring which has stiffness value to emulate the behaviour of a simply-supported beam as depicted in Figure 3.

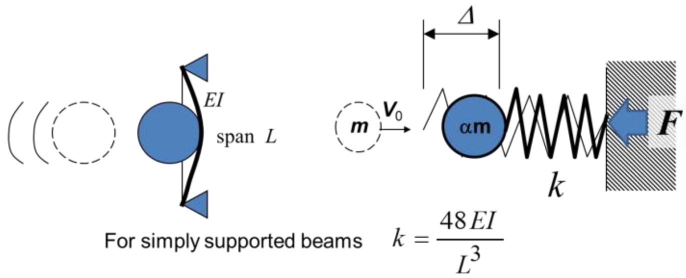

Figure 3 Model for contact behaviour

Given that the Coefficient of Restitution (COR) of the impact is defined by equation (3):

$$COR = \frac{v_1^{'} + v_2^{'}}{v_0} \tag{3}$$

the velocity ratio, and the kinetic energy ratio of the impact (which is indicative of energy losses) is accordingly obtainable from equations (4) and (5) respectively for cases where the impactor does not become embedded into the surface of the target following the impact.

$$\frac{v_2^{'}}{v_0} = \frac{1 + COR}{1 + \alpha} \tag{4}$$

$$\frac{KE_2}{KE_0} = \frac{\frac{1}{2}\alpha m.\left(v_2^{'}\right)^2}{\frac{1}{2}m.\left(v_0\right)^2} = \alpha\left(\frac{1+COR}{1+\alpha}\right)^2 \tag{5}$$

Equation (5) can be modified as follows to incorporate parameter λ which is either equal to 1 (to represent the effects of an embedded impactor) or 0 for cases where the impactor is detached from the target following the impact:

$$\frac{KE_2}{KE_0} = \left(\lambda+\alpha\right)\left(\frac{1+COR}{1+\alpha}\right)^2 \tag{6}$$

The deflection of the target and the corresponding quasi (equivalent) static force, or reaction force, is obtainable using equations (7a) and (7b)

$$\Delta = \frac{mv_0}{\sqrt{km}}\beta \tag{7a}$$

$$F = v_0\sqrt{km}\beta \tag{7b}$$

where the *mass reduction factor* $\beta = \sqrt{\frac{KE_2}{KE_0}}$ (8)

The value of β is therefore given by equation (9) which takes into account the effects of both the mass of the target and the coefficient of restitution (COR).

$$\beta = \sqrt{\left(\lambda+\alpha\right)\left(\frac{1+COR}{1+\alpha}\right)^2} \tag{9}$$

Equation (9) is reduced to equations (10a) and (10b) for the idealised conditions of perfect elastic impact with perfect re-bounce (COR=1, λ=0) and inelastic impact of an embedded impactor (COR=0, λ=1) as presented in Ali et al. (2014).

$$\beta = \sqrt{\frac{4\alpha}{1+\alpha}.\frac{1}{1+\alpha}} \tag{10a}$$

$$\beta = \sqrt{\frac{1}{1+\alpha}} \tag{10b}$$

Consider an impactor which has mass equal to one-fifth of the generalized mass of the target (e.g. a RC column): $\alpha = 5$ the deflection demand, and equivalent force, generated by the impact is only about 0.4 times of that estimated by equation (1a) and (1b) if the conditions of inelastic impact is assumed. Thus, an estimated deflection of 100 mm is reduced to 40 mm. The deflection is increased slightly to 45 mm if there is a small amount of re-bounce (COR=0.2) of the impactor according to equation (9). If the idealized conditions of perfect elastic re-bounce is assumed the deflection is increased to the upper limit of 75 mm. An equivalent static force is then applied to generate a deflection which matches with that estimated from energy principles. These estimates can be obtained conveniently by expressions presented above. Importantly, the accuracies of these estimates have been verified by comparison with results from both finite element analyses and physical experimentation on miniature scale models (Ali et al., 2014). Without these expressions it would have taken a great deal of effort to set up elaborate finite element models for obtaining total solutions from the so called state-of-the-art software.

The estimated contact force values can be applied to tiles specimens (Figure 4) and glazing panel specimens (Figure 5) on a normal test rig to assess the risks of potential damage to these components in projected impact scenarios. It would have been a great deal more costly to test these specimens dynamically when the amount of force to be applied is uncertain.

Figure 4 Testing tiles to failure on a normal test rig (Sun *et al.*, 2015)

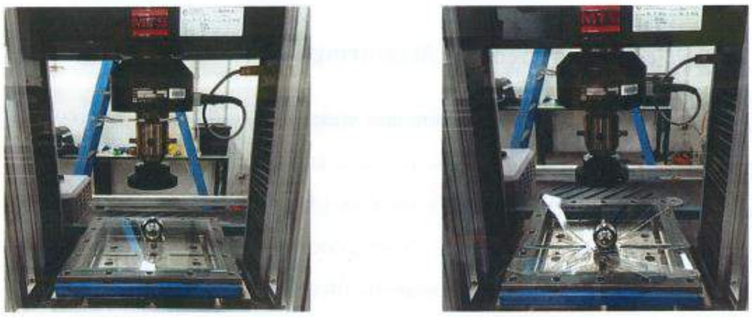

Figure 5 Testing glazing panels to failure on a normal test rig (Sun *et al.*, 2015)

4 LOCALISED EFFECTS OF IMPACT

The much higher amplitude, shorter duration, contact force can be determined by considering the impactor as a lumped mass which is connected by a spring to half-space (in a single lumped mass system), or to another lumped mass representing the target (in a two lumped masses system as depicted in Figure 6). The hardness of the impactor and that of the surface of the target is reflected in the stiffness of the connecting spring in the lumped mass model. In situations where the duration of contact is so short (typically in a few ms) that movement of the target is expected to be negligible during the course of contact the half-space modelling approach can be considered as valid. When this assumption is made there is only one lumped mass and one connecting spring to consider in the model. The value of the maximum contact force (F_{cmax}) can be calculated by equating kinetic energy delivered by the impactor with energy absorbed by the compression of the connecting spring up to the limit of F_{cmax} (Yang et al., 2014).

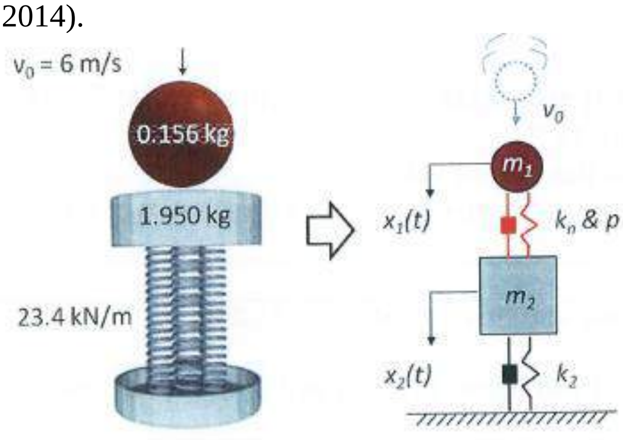

Figure 6 Example two lumped masses model for simulating impact of a fallen object

The maximum contact force associated with a given amount of absorbed energy depends on the hysteretic relationship adopted in the modelling. The simplest hysteretic contact model is that of linear elastic behaviour (Figure 7a) which has neglected the well known stiffening behaviour of two objects coming into contact. The alternaive non-linear elastic model (Figure 7b) is consistent with observations of quasi-static testing on the impactor and the surface of the target, and is consistent with Hertz Law in which the value of p is taken by default as 1.5. By contact mechanics based on Hertz Law the value of contact force (F_c) generated by a rigid sphere indenting into the surface of a half-space made of materials of Young's modulus (E) is defined by equations (11a) – (11c).

$$F_c = k_n \delta^p \tag{11a}$$

where $$k_n = \frac{4}{3} E \sqrt{R} \tag{11b}$$

$$p = 1.5 \tag{11c}$$

The same expression may be used if the half space is assumed to be rigid instead and the impactor is made of materials of Young's modulus (E). An example of this is a piece of concrete debris of 0.5m in size, weighting 160 kg, falling on a metal target from a drop height of 2m (i.e. velocity on impact of 6.3 m/s). The value of E for the piece of debris is assumed to be 20GN/m^2 which is about 1/10th of that of the metal target which can be treated as half space in an analysis. Thus, only the compressibility of the debris material is taken into account. The value of k_n is estimated from equation (11b) to be $\frac{4}{3} \times 20\sqrt{0.25} \approx 13\,\mathrm{GN/m^p}$ based on treating the debris piece as a spherical object.

In situations where the impactor (fallen debris), and target, have comparable values of Young's modulus E_1 and E_2 the value of E to be used for the purpose of eqn (11b) can be calculated using eqn (11d).

$$\frac{1}{E} = \frac{1}{E_1} + \frac{1}{E_2} \tag{11d}$$

The value of the maximum contact force (F_{cmax}) can be found using equation (12) which can be derived using equal energy principles (Sun, 2015).

$$F_{c\max} = k_n \left(\frac{p+1}{2k_n} m v_0^2 \right)^{\frac{P}{P+1}} \tag{12}$$

where m is the mass of the impactor and v_0 the incident velocity of impact

The maximum contact force generated by the impact of the piece of concrete debris is accordingly:

$$13 \times 10^9 \left(\frac{1.5+1}{2 \times 13 \times 10^9} \times 160 \times 6.3^2 \right)^{\frac{1.5}{1.5+1}} \times 10^{-3}\,\mathrm{kN} \approx 2400\mathrm{kN}$$

In situations where the Young's Modulus (E) of the impactor object is unknown, which is normally the case when the impactor is not made of an homogeneous material (e.g. a golf ball or cricket ball), an equivalent value of E can be found by quasi-static load testing of the object followed by calibrating the value of E in a finite element model simulating the test setup to match with the test results. Substituting the value of E into equation (11b) will give the value of k_n which can in turn be used to estimate the force-displacement relationship using equations (11a) – (11c).

Alternatively, the force-displacement curve can be obtained by taking readings directly from the test rig in a static test but the curve has to be rotated about the origin to allow for an increase in stiffnesses in dynamic conditions. A correction factor of 2.3 is recommended in Sun et al. (2013) to allow for the difference in boundary conditions of the impactor in a static test (where the impactor bears against load platens of the test rig on two sides as shown in Figure 8) and the impactor in a real impact scenario (where the impactor bears against the surface of the target on one side only).

Even more accurate solutions can be found by adopting the non-linear visco-elastic contact model of Figure 7c but dynamic testing and calibration would be required for the determination of the value of the modelling parameters.

The estimated contact force value can be applied quasi-statically to a metal panel specimen in a normal test rig for estimating potential damage by indentation in projected impact scenarios (Figure 9).

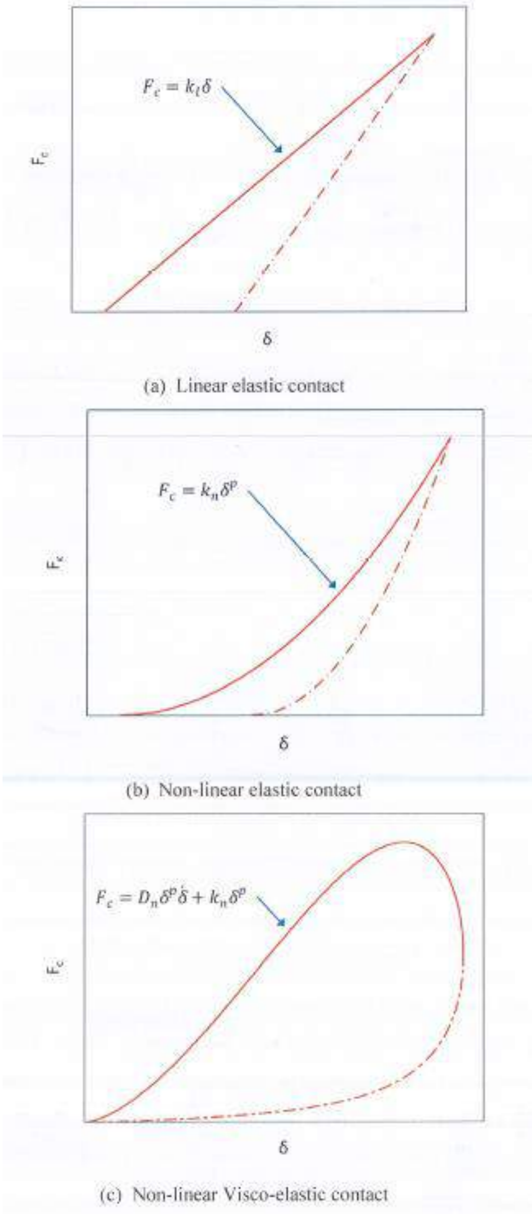

Figure 7 Models for contact behaviour (Sun, 2015)

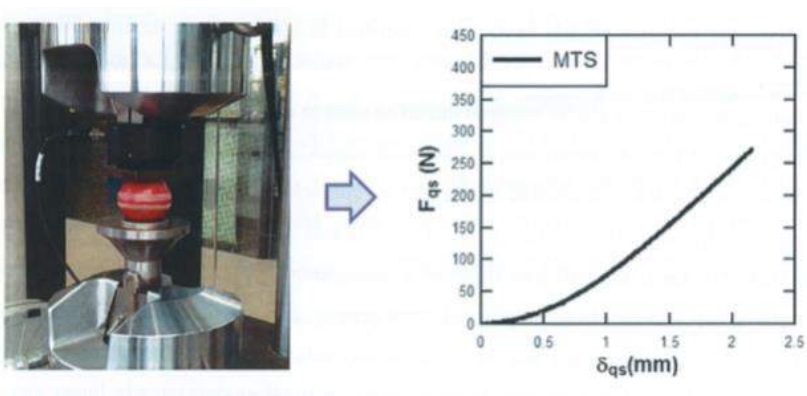

Figure 8 Testing impactor stiffness (Sun *et al.*, 2014)

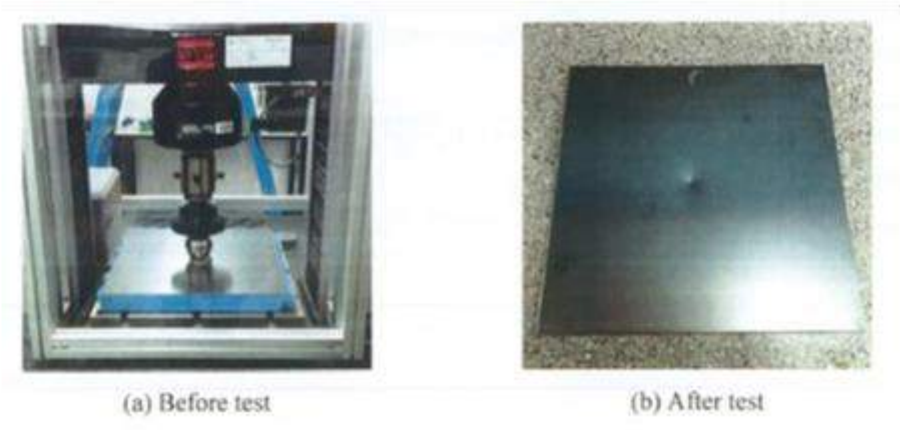

Figure 9 Testing aluminum panel for indentation (Sun et al., 2015)

5 CONCLUSION

Simplified methods for estimating the amount of force generated by the impact of a solid object have been presented in this paper based on resolving the impact action into (i) the impulsive component which results in the deflection of the target along with the associated bending moments and shear forces and (ii) the contact force component which results in localized damage such as indentation into the surface of the target and perforation. Algebraic expressions have been presented in the early part of the paper for estimating the deflection demand of the target, and hence the quasi-static force, for given impactor mass, incident velocity of impact, mass ratio and coefficient of restitution. The presented expression takes into account the significant mitigating effects of the target mass and provides much more accurate predictions than existing relationships that are currently used in highway codes of practices. The analytical procedure is generic in nature and can be used for assessing the risk of failure of a piece of tile, a glazing panel or a concrete member subject to impact by a solid object. A separate set of expressions has been presented in the later part of the paper for modelling contact force which is typically of much higher amplitude, and shorter duration, than the quasi-static force generated by the impact. The important influence of the hardness of the impactor object (and that of the surface of the target) has been taken into account by the presented estimation methodologies.

6 ACKNOWLEDGEMENTS

Collaboration with Professor Emad Gad and Dr Ruan Dong from Swinburne University of Technology and with colleague Dr Lihai Zhang from The University of Melbourne in the co-supervision of research higher degree candidates undertaking research in this theme is gratefully acknowledged. Much of the materials reported in this paper are based on work of PhD graduates Yi Yang, Muneeb Ali and Jing Sun.

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Innovative engineering solution for underground sewage infrastructure construction

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Keywords: innovation; drop shafts; downpipe; diaphragm wall; corestone.

ABSTRACT: An innovative method has been employed by Gammon Construction Limited for the construction of drop shafts for an underground sewage treatment project in Hong Kong. Harbour Area Treatment Scheme Stage 2A requires the construction of drop shafts at North Point, Wanchai East and Central to transfer effluent from preliminary treatment works to deep sewage tunnels. This paper explains the technical background to the development of an innovative method, involving the use of large diameter bored piling and raise boring excavation techniques, to form drop shaft downpipes. The adopted engineering solution not only accelerated shaft construction and reduced overall construction cost, but also made the shafts easier to build safely and minimized the impact on the environment and natural resource demand.

1 INTRODUCTION

The Harbour Area Treatment Scheme (hereafter as "HATS") is being constructed to improve the water quality in Victoria Harbour through the collection and treatment of all waste water from industries, homes and businesses prior to final disposal into the harbour. Under the scheme, the effluent is collected by an extensive network of deep tunnels from the urban areas of Hong Kong Island and Kowloon, and conveyed to the sewage treatment works at Stonecutters Island for centralized treatment. The treated effluent will be discharged through the submarine outfall tunnels into the western harbour of Hong Kong.

The overall scheme is being implemented in two stages by the Drainage Services Department. In Stage 1, 24km of deep tunnels were constructed from a catchment at the northeastern part of Hong Kong Island and Kowloon to Stonecutters Island Sewage Treatment Works. In Stage 2, presently under construction, is divided into two phases. The first phase, namely Stage 2A, comprises a sewage conveyance system, comprising 21km of interconnected deep tunnels and drop shafts at the northern and southwestern part of Hong Kong Island. The second phase, namely Stage 2B, aims to add an underground biological treatment facility adjacent to the existing Stonecutters Island Sewage Treatment Works to provide secondary sewage treatment.

Construction of HATS Stage 2A commenced in 2009. As part of the system, existing sewage treatment works in North Point, Wanchai East and Central require the construction of drop shafts to transfer the screened effluent from the treatment works to a system of deep tunnels. In July 2009, Gammon Construction Limited (hereafter as "Gammon") was awarded Contract DC/2007/23, HATS Stage 2A, Construction of Sewage Conveyance System from North Point to Stonecutters Island by the Drainage Services Department. AECOM Asia Company Limited was appointed as the Engineer to undertake the schematic design and the construction supervision of this Contract. This paper presents an innovative method used to construct the drop shafts developed by Gammon. The merits of using this method with emphasis on its economic, sustainability and technical advantages are discussed.

2 SHAFT DESCRIPTION AND REFERENCE BASE SCHEME

Gammon's scope was to design-and-construct a 12km long section of deep tunnel between North Point and Stonecutters Island together with associated drop shafts. The alignment of tunnel in relation to its drop shafts is shown in Figure 1 below.

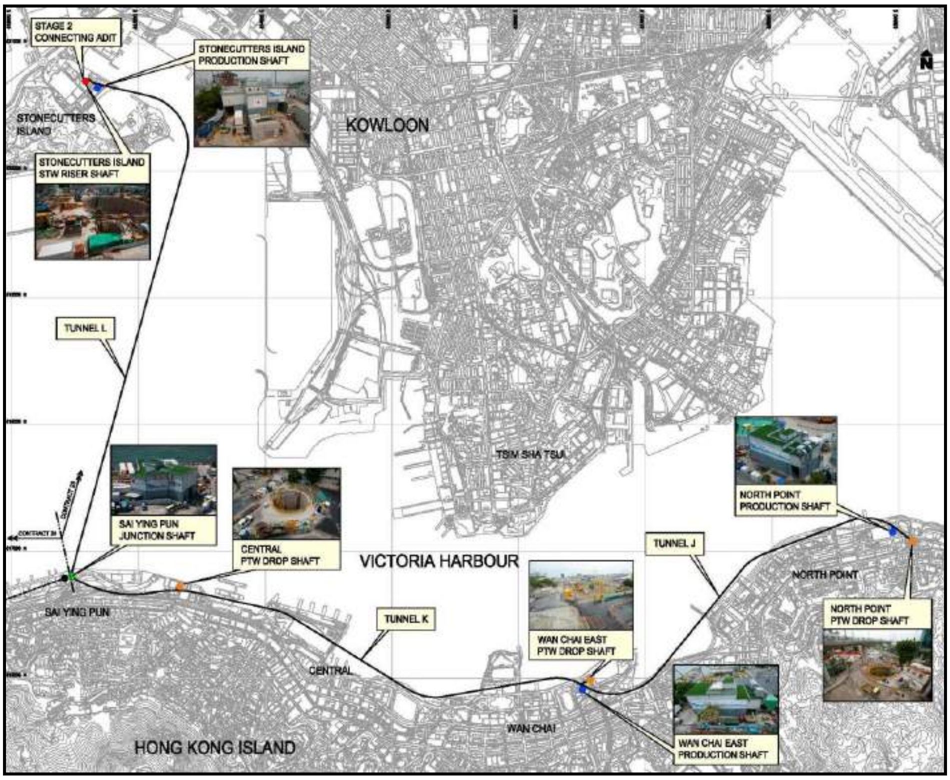

Figure 1 Layout plan of tunnel and drop shafts

The vertical drop shafts are circular shaped in the form of an intake chamber with a diameter of 12m to 13.5m and depth ranging between 13m and 26.5m below ground surface. The chamber is designed for installation of a scroll-type vortex inlet which minimizes air entrapment and dissipates excess flow energy within the effluent before being discharged into the tunnels. The vortex intake structure transforms the upstream straight flow into a swirling flow in order to prevent choking phenomena and cavitation damage. A pair of 2.7m diameter downpipes were built to connect the chamber through soil and rock down into the deep sewage tunnels, which were constructed below the rockhead level and some 160m below sea level.

The Engineer had developed a base scheme using cast in-situ circular diaphragm walls formed to competent bedrock as excavation support. The proposed arrangement would allow operations to carry out the excavation without the need for installing any internal props or ties, simply relying on the self-propping action of the continuous reinforced concrete rings as structural support. Excavations had been planned in two stages. The initial stage involved a large-scale excavation to rockhead level, which covered the entire footprint of intake chamber and the upper portion of downpipes, as the anticipated rockhead level was below the formation of chamber. The second stage involved excavating the lower portion of the downpipes in rock using the raise boring technique, using a reamer, rotating it and pulling it upwards from the tunnel level through the rock mass. Upon completion of the excavation, the upper portion of the downpipes in soil would be constructed in an open circular diaphragm wall cofferdam. The voids between the diaphragm walls and downpipes would be backfilled up to the soffit of the chamber during subsequently construction of the downpipes.

3 MAJOR TECHNICAL DESIGN PROBLEMS AND CONSTRUCTION CHALLENGES

The drop shafts were constructed along the existing shoreline of Hong Kong Island on old marine reclamation. The geological setting at the shaft locations covers a wide variety of superficial deposits comprising a lower alluvium that overlies saprolite formed by in-place weathering of the underlying granitic bedrock, and an upper marine mud and sand. The marine deposits are overlain by a 20m thick blanket of variable fill. Competent bedrock described as strong to moderately strong, slightly to moderately decomposed granite, was encountered below the formation of the intake chamber. The rockhead profile at the shafts is irregular due to preferential weathering along local joint patterns. A review of the site investigation data shows that the rockhead level at the drop shafts varies from about 38m to 50m below the existing ground level. Core samples recovered from the boreholes indicate the saprolite contains an appreciable amount of corestone above the rockhead level.

The rockhead level is uneven over the footprint of the shafts. In the event of diaphragm walls extending to rockhead, additional excavation of rock to equalize the panels' toe to the deepest level is required to ensure continuity of hoop stress transferring through the panels. Diaphragm wall excavation is challenging in strong rock due to tool wear that results in slow production rates. Corestone layers above rockhead at the shaft sites would have proven tremendously difficult to excavate panels through. Also, corestone can adversely affect the verticality of diaphragm wall panels resulting in adverse bending and structural distress. If the diaphragm wall panels are misaligned to the extent that the beneficial hoop action is no longer sufficient to fully support the load, the wall would need to be supported laterally by supplementary bracing such as ring beams.

As a further challenge, it was recognized that in the first few meters below rockhead level, there was a number of weathered seams and sheeting joints within the rock mass. These geological features can significantly increase the hydraulic conductivity of the rock mass. A more permeable zone at rockhead may serve effectively as a confined aquifer beneath the less porous saprolite. Risks of excessive groundwater inflow, associated ground settlement and base heave when the excavation approached the confined aquifer were identified. Mitigations would have involved pressure relief provisions and extensive groundwater cut-off grouting, from the toe of the diaphragm wall through the jointed rock mass.

4 CONTRACTOR'S SOLUTION

To tackle with these geotechnical challenges, Gammon and its in-house engineering design consultant, Lambeth Associates conducted a series of qualitative workshops to determine the excavation support system for constructing the drop shafts. The team carefully assessed different construction methodologies such as freezing the overburden, employing the large diameter bored pile technique, and using the traditional slurry wall system in conjunction with pre-boring. A comprehensive study was undertaken to evaluate the methods against several criteria including the deadline for completing the construction works, constructability, safety risks, environmental benefits and the potential impacts on construction cost. The large diameter bored pile technique was ultimately identified as the most appropriate construction method because it removed major safety risks, offered time and cost benefits and minimized impacts on the environment.

The innovative method involved the excavation of soil using traditional long arm excavators and support from diaphragm wall hoop stress to the soffit level of the intake chamber. This prevented bringing the entire footprint of the temporary access shaft down to the rockhead level, and considerably reduced the required depth of excavation for the shaft. The portion of downpipe through the soil was constructed by adopting conventional bored piling methods to sink a large diameter steel casing. The verticality tolerance for installing the steel casing was controlled to 1:200. The steel casing acted as an access tube to support the ground, and the soil inside the steel casing was excavated using a hydraulic grab. A 2m thick concrete plug was placed at the bottom of the steel casing while leaving the rest of the casing empty for raise boring. The steel casing was designed to socket into the competent bedrock with permeation grouting around the periphery of the casing at the rockhead interface. This ensured an effective seal for raise boring in the dry. The raise bore operation was finally carried out from the soffit level of the intake chamber instead of from the rockhead level. The temporary surcharge effect of intake chamber due to construction dewatering has been incorporated into the steel casing design. The load carrying capacity of soil foundation has been verified against the maximum ground bearing pressure at the base of drop shaft.

This engineering solution prevented sinking the diaphragm walls through the problematic corestone bearing stratum. Toe levels of diaphragm wall panels were raised and a groundwater cut-off, formed by permeation grouting below the diaphragm wall toe to rockhead level, was provided. At North Point drop shaft, which has a particularly shallow intake chamber, pipe piles with provision of grout curtain and ring beams were used to replace the diaphragm walls. By using this method, the induced settlement due to shaft excavation was greatly reduced since the excavation of the entire footprint of the shaft was limited to the upper intake chamber structure. Sections showing the construction method for the Central Drop Shaft can be seen in Figure 2.

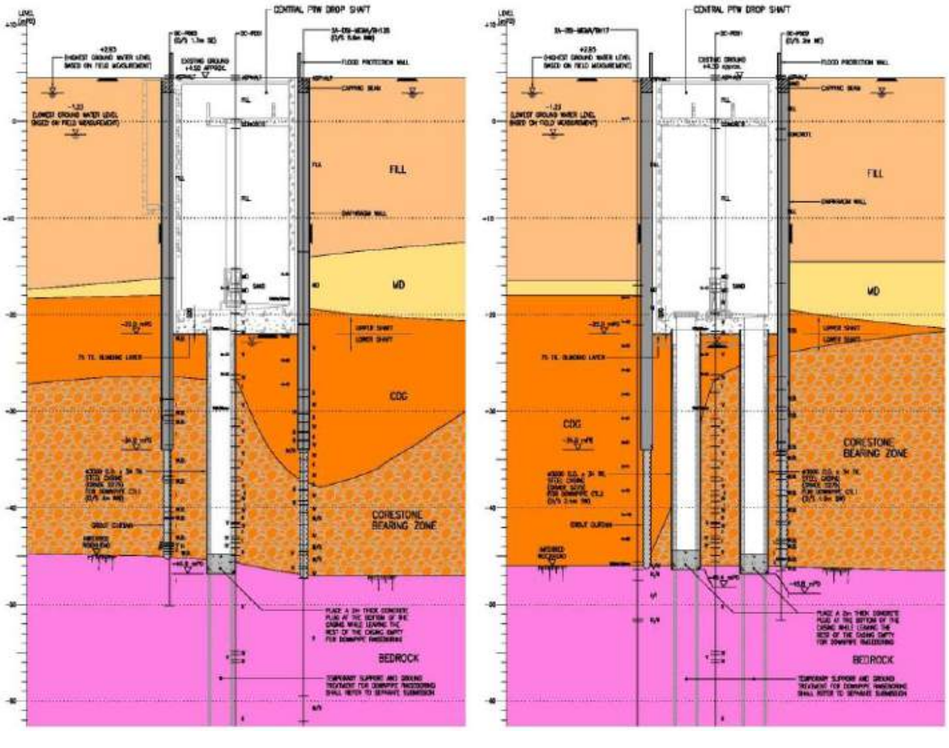

Figure 2 Cross sections of central drop shaft

The construction of drop shaft downpipes has been successfully executed by utilizing the bored pile casing method together with the raise boring technique. The adopted construction sequence is shown in Figure 3 below.

Stage 1

Divert the existing utilities at the shaft area and construct the diaphragm wall (in Central and Wanchai East) and pipe pile wall (in North Point)

Stage 2

Install the steel casings by Reverse Circulation Bored Pile Drilling Machine

Stage 3

Carry out the grouting underneath the diaphragm wall panels down to the rockhead and around the periphery of the steel casings at the rockhead interface

Stage 4

Construct the capping beam and shaft collar and complete the excavation within the steel casings to the rockhead level using hydraulic grab and the chisel

Stage 5

Complete the excavation of the intake chamber and remove the exposed temporary steel casings subsequently by flame cutting during the excavation of the drop shaft

Stage 6

Place a 2m thick concrete plug at the bottom of the temporary steel casings and carry out downpipe raise boring at the soffit of intake chamber

Figure 3 Sequence of construction for drop shaft

5 BENEFITS OF CONTRACTOR APPROACH

This innovative approach leveraged Gammon's extensive experience in construction of large diameter bored piles to excavate the drop shaft downpipes through the problematic corestone bearing stratum. The method adopted offers many advantages. It not only lowered the estimated cost and conserved resources, but also provided a safer working environment and improved work efficiency throughout the work process. The key advantages of this new construction method are summarized as follows:

a) Eliminated the buildability challenges of constructing diaphragm walls through corestone bearing saprolite, and removed the need to excavate huge volume of soils and rock, and to bring down the entire footprint of the temporary access shaft to the rockhead;

b) Greatly reduced the risk of excessive groundwater inflow and the potential of base heave because the toe of diaphragm wall panels would not be exposed during the excavation of drop shafts;
c) Substantially reduced the removal of excavated material for off-site disposal and import of the backfilling material, lowered the carbon emissions and fuel consumption during the construction process, and eventually reduced the cost of construction;
d) Reduced the total volume of excavated material for the drop shafts in Central, Wanchai East and North Point by approximately 8,500 cubic metres;
e) Mitigated the risk of any significant ground settlement in the worksite surrounding areas because the excavation of drop shafts was highly confined and localized; and
f) Reduced the duration of excavation activities inside the temporary access shaft, lowered the level of construction noise and dust for nearby urban areas, and minimized the need of frontline staff to work in the confined environment.

6 CONCLUSION

This case study has far-reaching practical significance in underground engineering. It has demonstrated that a successful project requires a combination of solid scientific knowledge and the art of making sound engineering judgment at the various stages of construction. To overcome the adverse geotechnical challenges in the sewage infrastructure construction project, Gammon succeeded in employing the large diameter bored pile technique to avoid buildability challenges of bringing down the entire footprint of the temporary access shaft to the rockhead. This innovative method proved to be beneficial on many fronts; it enhanced works safety, lowered the construction cost, and protected the environment. By adopting this method, Gammon has positioned itself as one of the most innovative and competitive enterprises in Hong Kong's fast-growing urban construction industry.

7 ACKNOWLEDGEMENTS

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Application of BIM in civil and infrastructure project

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Keywords: Building Information Modeling (BIM); parametric element.

ABSTRACT: In recent years, Building Information Model (BIM) starts to be applied in the civil and infrastructure projects, and continues to grow in greater importance. How BIM can actually help the civil engineers in their daily design works and be physically implemented in different stages throughout the entire project cycle become concerned and imperative subjects. This paper briefly discusses the BIM methodology, provides various examples on BIM applications, and illustrates the special features of BIM. With its diversified applications at different stages of project, BIM has brought to the industry numerous benefits.

1 INTRODUCTION

Building Information Model (BIM) technology has been widely adopted in the building industry for many years. However, BIM concept has just begun for civil and infrastructure works. Large scale civil and infrastructure projects include site formation, slope works, trench, roads, tunnels, bridges, piers, etc. Unlike buildings, they include horizontal aspects to construction, and their model elements are non-standardized. As a result, it increases time and effort to create BIM for civil and infrastructure projects. Consequently, we could ask ourselves, why do we need to adopt BIM for civil and infrastructure projects? And, how can BIM be implemented? The purpose of this paper is to provide an overview of BIM, and introduce its benefits towards and applications for civil and infrastructure projects.

2 OVERVIEW OF BIM

2.1 *What is BIM*

BIM is a 3D model (geometry) consists of all project components, and the elements of the component linked with a database containing the attributes. This database also contains traditional documents such as drawings, specification and construction details. It allows BIM to provide a common platform for information sharing. A 'smart' feature of BIM is the parametric design of elements, which define model elements as parameters with relationships to other model elements. The relationships can be embedded in the model. Whenever a parametric element is modified, the BIM software determines which other elements need to be updated based on the embedded relationships.

2.2 *Why BIM*

The life cycle of civil and infrastructure projects consists of planning, preliminary design, detailed design, construction and operation. Typically the information that is carried over between each stage is the drawings. However, these usually exclude the very information necessary for effective design evaluation and construction, such as contract documents, timelines, specifications, installation guides. BIM is a data repository for design, construction and maintenance information for the whole project life cycle. All the stakeholders (owner, engineers, contractor and management officer) can share information through BIM to ensure consistency. As the information is shared in a single platform for the whole project life cycle, it can reduce the need for re-gathering or re-formatting of information.

BIM is an integrated, dynamic model linked with documents (drawing list, drawings, 3D renderings, etc). Through this dynamic model, the design and the drawings are coordinated automatically. That means when there are changes to the design, documents update automatically to reflect those changes. BIM can generate documents on demand to present views of current BIM.

3 BIM APPLICATIONS

BIM has a lot of different uses at different stages of civil and infrastructure projects. The following examples present the applications of BIM and the benefits.

3.1 *BIM for Geological and Terrain Models*

When carrying out design of a large scale civil and infrastructure projects such as tunnels, bridges, reclamation works and site formation works, engineers need to quickly visualise the ground condition to understand how the geology will interact and influence the designs.

Terrain models can be created from different type of data sources, such as LiDAR, contour, spot height. Engineers can quickly compile a 3D ground model by inputting geological data from the ground investigation. Rockhead level, geological strata, rock grading, results of all kinds of GI testing and groundwater tables can be input into BIM and provide a 3D overview on geological condition. The 3D overview facilitates a comprehensive interpretation of ground condition for geotechnical assessment of the proposed development.

2D rockhead contours can be generated from the 3D geological model. A geologist shall check the contour in order to ensure the accuracy. Subsequently, the geological model can be updated by the interpreted data. In addition, cross sections in any orientation and angle easily be generated from BIM for review from a geological perspective.

These cross sections can be used as a basis for further development of detailed designs and the associated drawings. Engineers can also use the contoured surfaces to speed up the establishment of geotechnical ground models in other specialist software such as Slope/W or Plaxis.

In applying this technology, engineers can fully understand the ground condition in a fast and easy way. Identification of gaps in geological information, or complex ground conditions can be identified, additional ground investigation can be planned in a cost effective manner. Better understanding of ground condition results in safe and economic geotechnical designs, and minimizes the construction risk due to impact of unforeseen conditions.

3.2 *BIM for Road Alignment Modeling*

Hilly terrain is a dominant feature in Hong Kong and constructing new roads in such environment usually involves a substantial amount of earthworks. BIM allows engineers to visualise the alignment in order to minimise the amount of cut and fill.

The design typically begins with BIM of existing surfaces from topographical data and as-built records. The surfaces formed typically include the existing ground levels, existing road networks, and adjacent buildings. From these surfaces, the road/footpath alignments can be created based on the design requirements specified by government authorities.

Not only can the earthworks quantities be calculated from the models, the cross sections in any orientation can also be developed. This enables designers to compare the constructability of various viable schemes, while allowing a smooth progression of the detailed design in subsequent stages. The models also help engineers to identify quickly any elements that fail to comply with statutory requirements, such as those found in the government's Code of Practise for Barrier Free Access (BFA), Code of Practice for Fire Safety in Buildings Part D section 6 – Provision of Emergency Vehicular Access.

3.3 *BIM for Schematic Design of Road Widening*

For schematic design, the engineer prepares BIMs of different design options. The BIMs are created based on hand sketches of the road widening design as shown in Figure 1. Figures 2, 3, 4 show different design options for the road widening. For each option, the earthworks quantities can be calculated. The engineer is able to walk through all the virtual models and visualise impact to adjacent areas, to decide the best option to fit his requirement.
The use of BIM helps to make a better decision. In addition, the BIM model with touch-up effects can be used for public consultation or project video.

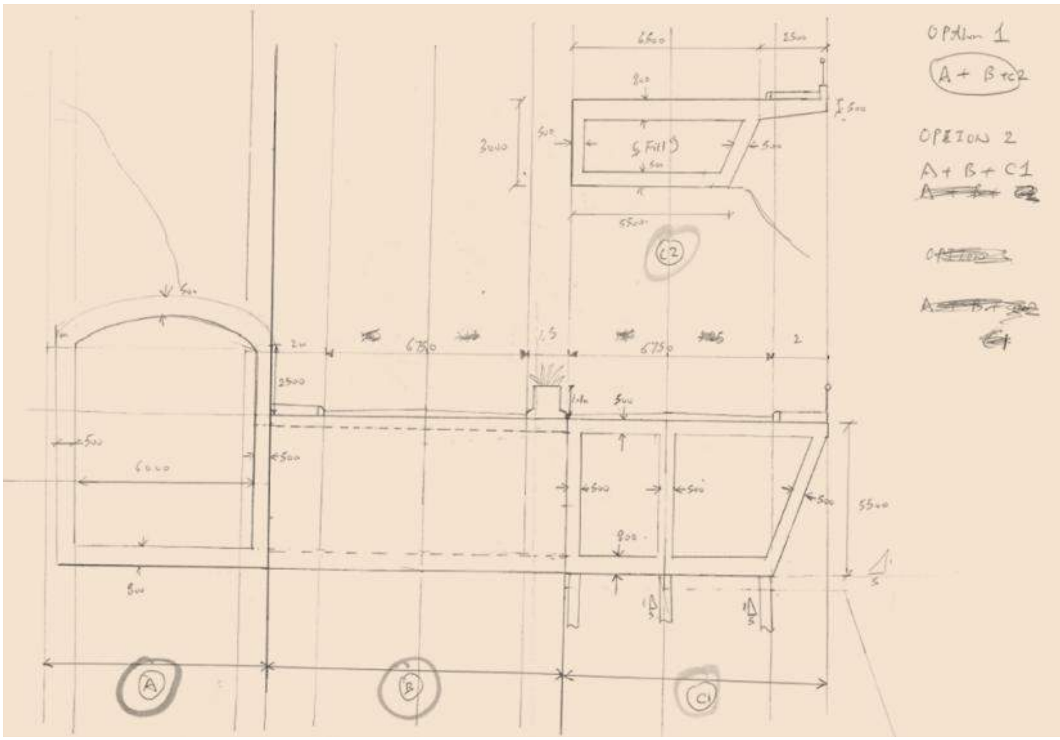

Figure 1 Hand sketch of road widening design

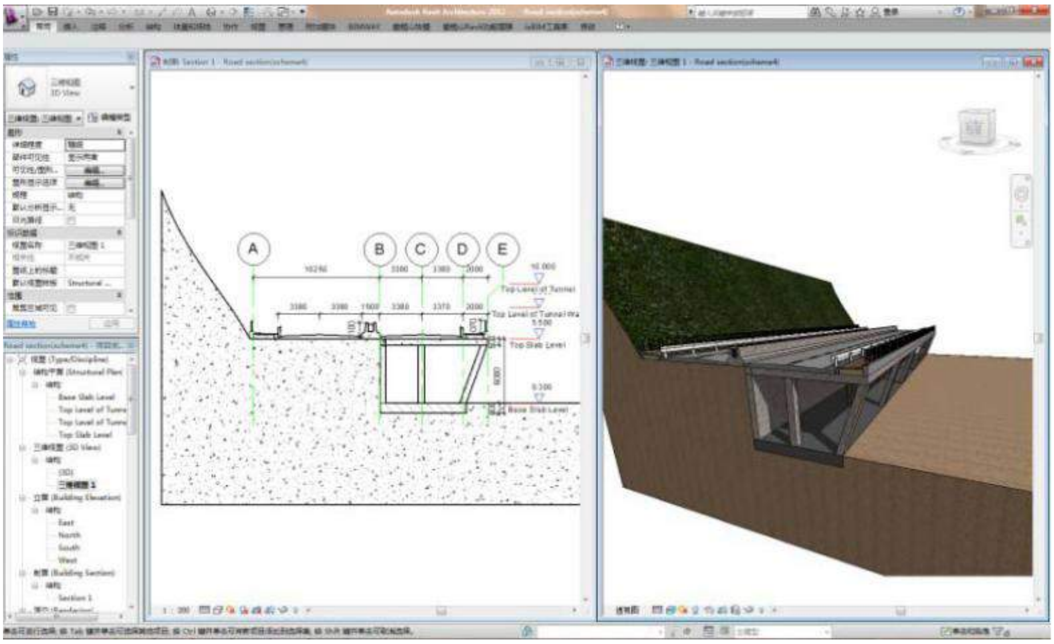

Figure 2 Option 1 of road widening design in BIM

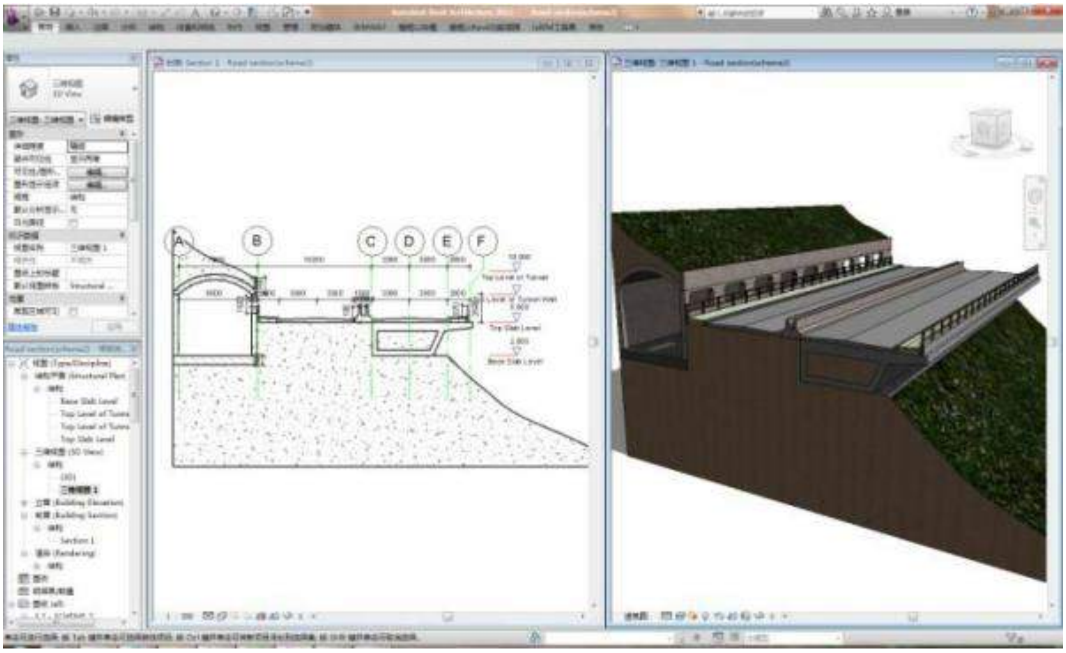

Figure 3 Option 2 of road widening design in BIM

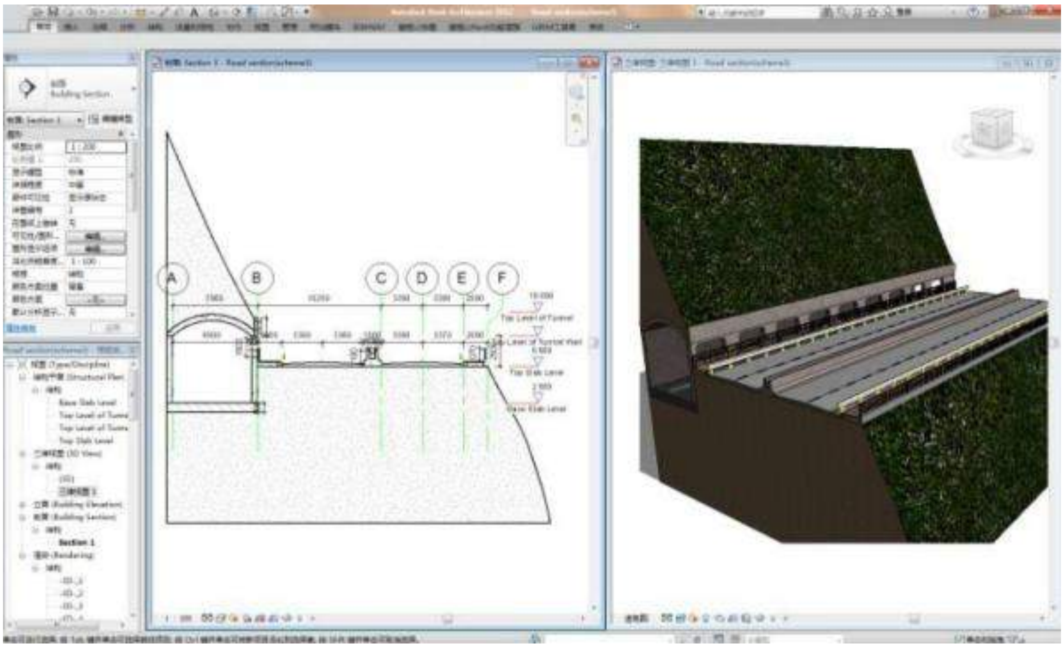

Figure 4 Option 3 of road widening design in BIM

3.4 *BIM for Sub-sea Tunnel*

Immersed tube tunnels are constructed by sinking precast concrete boxes into a dredged channel and connecting them under water. The BIM is able to show the excavation profile, backfill profile and geological information. It can therefore calculate the dredging volume along the immersed tube tunnel. The quantities for different types of excavation (Marine Deposits, saprolite or rock) and backfill can be calculated easily using BIM. The cross sections in any orientation can be also developed with backfill, inferred geological profile, etc.

3.5 *BIM for Construction Sequence*

In BIM technology, each element in the 3D model can be linked to the construction programme. This 3D model with construction sequencing activities can display the progression of construction over time, and is called a 4D model. The 4D model can be applied in design and construction phases. At the design stage, the 4D model allows the engineer to carry out constructability assessment and develop a phasing sequence. At the construction stage, 4D simulation of the construction process allows the contractor to easily visualise and identify incompatible construction sequences (clash analysis), which optimize the construction planning. In addition, the owner and contractor can have a better comparison between the planned and the actual construction sequences.

In applying this technology, many design issues can be identified in the 3D model, which may be difficult on 2D plans. The identified design issue can be resolved before construction stage. It results in fewer Request for Information (RFIs) and field coordination problems during the construction stage.

3.5.1 *4D BIM for Large Scale Civil and Infrastructure Project*

This technology can also be applied to large scale multi-disciplines civil and infrastructure projects consisting of roads tunnels, bridges, reclamation works and site formation works. However, it takes time and effort to build the model. If the construction duration of any model element changes, that element needs to be re-defined. For example, the whole excavation period for a tunnel is established to be 20 months. If construction progress by the 10^{th} month changes from the assumed 100m to 80m, the elements of the model from the 10^{th} to 21^{st} months will need to be re-defined. For a 4D site formation model, the excavation period may be 20 months and the programme interval is 1 month. That means total 20nos. of model elements need to be built. If the design of the site formation changes, all 20nos. of model elements need to be re-built.

It is difficult to view the construction progress of underground structures (subway, tunnel, etc.) and its model elements (tunnel lining, tunnel paving, etc.). It is recommended that the overall 4D model for large scale projects should be as simple as possible, using colour to represent the construction status, such as green colour to represent the tunnel excavation and blue colour for completion of tunnel permanent works. The 4D model should be formed after completion of the design works (to minimise the rework), to improve the return on investments for BIM. Other specific examples of detailing of 3D BIM models for infrastructure include:

a) Replace the ground surface of the 3D model by 3D contour lines, making it possible to view underground structures, which would otherwise be obstructed by a 3D surface;
b) Details such as rebar of tunnel linings or permanent structures should be excluded.

It is difficult to view the construction progress of reclamation in 4D BIM for large scale projects. The detailed construction sequence of reclamation including foundations and/or ground improvement needs a special 4D implementation technique. It is also difficult to visualise parts of the model below sea level as they will be obscured by the sea element of the model.

Forming the 4D model before tendering will help both the client and the contractor understand the project better, and will result in more reasonable construction cost.

3.5.2 *4D BIM for traffic works*

For traffic improvement works or diversion works, 4D model can be developed to enhance the planning of construction sequence and extent. The advantages of using this 4D model are:-

a) To effectively assess the space required for construction for and therefore minimize the needs to reduce or shifting traffic lanes, hence reduce the impact on local traffic;
b) To better control the safety, quality, sequence and progress of works;
c) To effectively handle any diversion or removal of existing utilities and box culvert construction;
d) To be used for public consultation and minimise complaint from the public.

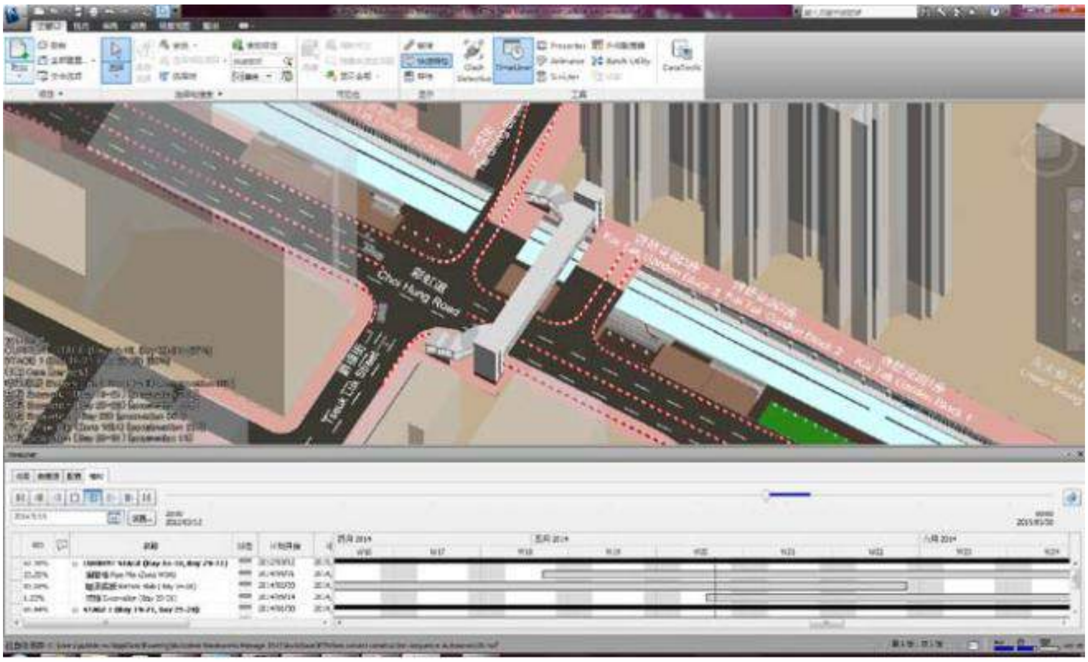

Figure 5 4D BIM for traffic diversion

4 CONCLUSION AND NEXT STEPS

The use of BIM is just beginning in civil and infrastructure construction. BIM can be applied to many different kinds of civil and infrastructure project. However, the development of BIM model takes time and effort to create, which would not be reusable in other projects. As there are a lot of benefits of using BIM, it is worth to develop and implement BIM in civil and infrastructure construction, but with careful consideration of the level of details to be incorporated. In particular, BIM can be separately applied to local element or construction site in a large scale civil and infrastructure construction project. The key benefits of using BIM include better collaboration, better project prediction, better overview and understanding, risk reduction, fewer RFIs, and decreased need for redesign.

4D BIM is one of the BIM applications which can be widely used in different kinds of civil and infrastructure project. The 4D simulation is of most benefit to stakeholders and the public, and worth to be further promoted.

The application of GIS is widely adopted in civil and infrastructure projects. BIM can be integrated with GIS to increase the BIM value for civil and infrastructure projects.

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Innovation and simplification on seismic design and analysis of artificial islands

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Keywords: Seismic Design, Artificial Islands, Man-Made Islands, Reclamation, Ground Response Analysis, Soil Dynamics, Liquefaction, PLAXIS, FLAC, SHAKE91.

ABSTRACT: This paper presents the technical key points for the seismic design and analysis of the artificial islands in the Hong Kong - Zhuhai - Macao Bridge (HZMB) Link Project. The design process included evaluation of seismicity, establishment of design methodology, and numerical analysis with the multiple computer methods including finite element method and finite difference method. At the start of the project, an in-depth literature study was carried out to establish a robust set of design parameters. Ground response analyses, liquefaction evaluation, as well as deformation analyses were performed and the results were cross-verified by multiple methods. The design work had been carried out in an efficient way to satisfy design requirement while catering for the very tight design time schedule.

1 INTRODUCTIO

The Hong Kong - Zhuhai - Macao Bridge (HZMB) Link Project, situated at the waters of Lingdingyang of Pearl River Estuary, is a large sea crossing linking Hong Kong, Zhuhai and Macao. It consists of a main bridge in Mainland China waters together with the boundary crossing facilities and link roads within the three cities. The main bridge includes a 29.6km dual 3-lane carriageway in the form of bridge-cum-tunnel structure comprising a tunnel of about 6.7km; two artificial islands for the tunnel landings west of the HKSAR boundary and associated works (see Figure 1). This paper is on the seismic design and analysis of the artificial islands which are the transition places connecting the bridge and tunnel sections. The two islands of oval shape are named as the West and East Island respectively with a dimension (length x width) of 625m x 183m for the west and 625m x 215m for the east island.

2 ESTABLISHMENT OF DESIGN AND ANALYSIS APPROACHES

The technical content of the design and analysis includes several interconnected parts, listed as below:-

a) Establishment of seismic design criteria,
b) Critical review and selection of seismic input parameters,
c) Liquefaction analysis and seismic response analysis, and
d) Dynamic deformation analysis of the artificial islands.

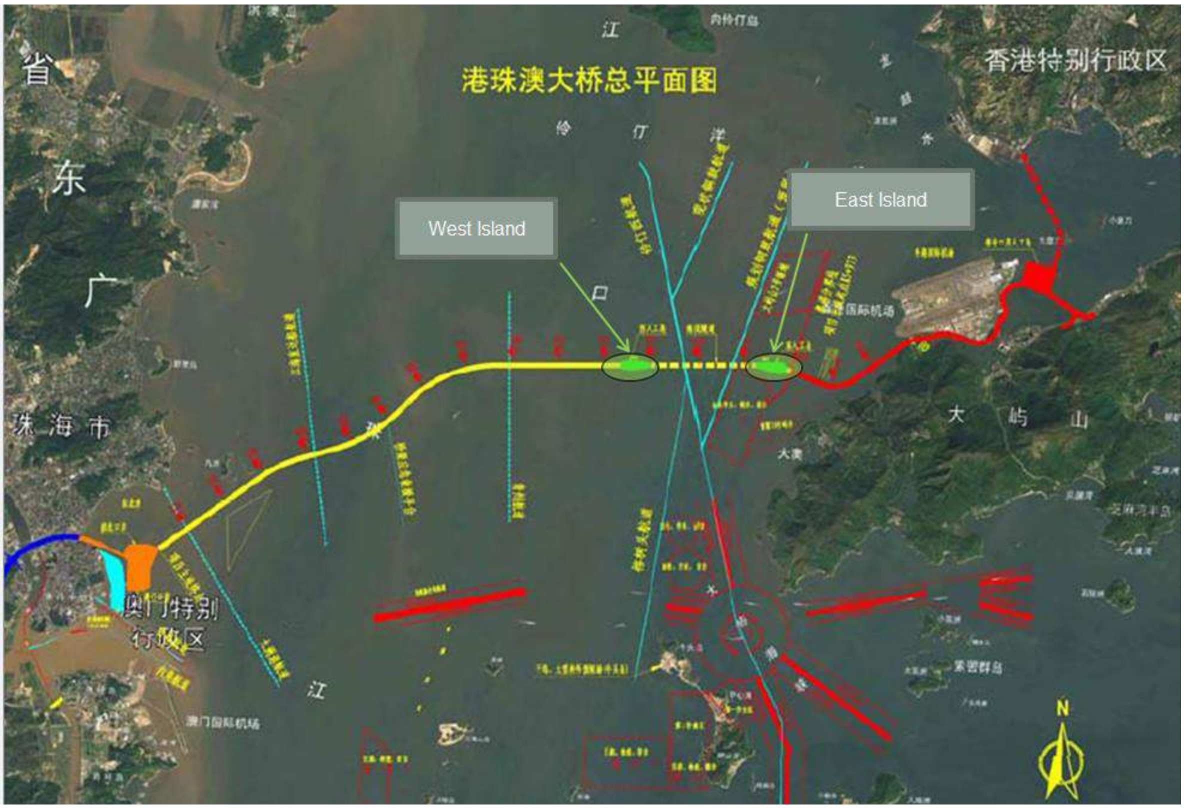


Figure 1 HZMB layout plan and locations of West and East Islands

This work was challenging in both technical and nontechnical perspectives. First, there had been significant controversy with regarding to the seismicity of the region, as the project site is located in a region with significant controversy regarding the level of seismicity. The controversy has been regarding to whether the region falls into the category of "low" or "intermediate" seismicity area. The site-specific seismic hazard analysis for this project tends to support the category of "intermediate" level of seismic hazard. Secondly, although scientific methods for seismic design have been quite developed, the usage of advanced analytical methods in local design practice was not a common practice.

In view of these challenges, the design team had conducted extensive research, literature review and review of past design practice relevant to the project site. During the execution of project design, the authors conducted series of literature review on local seismicity and the state-of-the-art design practice for man-made islands. Based on results of the literature review and considering the seismicity level and the project importance, we proposed to carry out three types of analyses, including ground response analysis, liquefaction analysis and deformation analysis.

3 CRITICAL REVIEW ON THE DYNAMIC SOIL PROPERTIES

As a key part in seismic design, the dynamic responses of the artificial islands under design level of seismic loadings need to be clearly understood. For this purpose, we have carried out various ground response analyses, for which the main input parameters were the input ground motions and dynamic soil parameters. For input ground motions, we studied the seismic hazard analysis report prepared by China Earthquake Administration, and extracted the input ground motions relevant to the site of the two artificial islands. With the input motions given, the ground response is critically dependent on soil properties and the ground profile. As it was understood in soil mechanics, soil properties such as stiffness and damping ratios under seismic loading would vary with development of shear strains. At small strains (less than 0.001%) the soil stiffness is high and damping is low; at large strains, the stiffness tends to be only a fraction of the small strain stiffness, and the damping ratio is much higher as shown in Figure 2.

For conventional engineering applications under static loadings, the design soil properties are commonly determined at large strains. For seismic engineering applications, the strain level is usually much smaller than that of conventional engineering design where only large-strain static soil parameters are required. As a result, the dynamic soil properties are significantly different from the soil properties in static design.

To obtain the dynamic soil properties, for this project a series of laboratory and in situ geophysical tests were performed and summarized in a preliminary report. Key data from these tests include initial/maximum soil modulus and soil dynamic curves.

In order to establish a sound basis for engineering design, we have to carry out a critical review of basic design parameters for the dynamic analyses. It depicted two specific issues. First, the number of soil tests appeared not sufficient considering the scale and the importance of this project. Second, some dynamic soil curves appeared abnormal trend. In view of the inadequacy of test data, we decided to make use of a set of internationally accepted soil dynamic curves EPRI (1993) to compare with those provided in the preliminary report and conducted a further critical review. Figures 2 and 3 illustrate a comparison of lab test curves and EPRI (1993) curves.

In this exercise, we identified a few materials, for which the damping ratios from the lab tests are significantly higher than the normal range found in the literature (EPRI, 1993). We then analysed the differences and concluded that some lab test results may not be credible due to the scarcity of the data for the potentially problematic soils. For a certain soil layer under seismic loading, as damping ratios increases, more energy would be dissipated during wave propagation and the amplitude of ground response would reduce. As a conclusion of the investigation and some sensitivity analyses, we found that certain test data if directly used would lead to non-conservative results in ground response. The under-prediction of the ground response would then potentially lead to an unsafe design.

Based on both validated laboratory tests and the internationally accepted curves by EPRI (1993), we obtained a set of design parameters which ensured the robustness of the design and analyses.

4 GROUND RESPONSE ANALYSIS

In a preliminary design stage, we have chosen three locations for each island to perform ground response analysis. These three locations were chosen to be at the top, middle and bottom of the artificial/man-made island embankment to cover different ground profile as shown in Figure 4. SHAKE91, a computer program developed by Idriss and Sun (1992) was chosen as the tool for analysis.

In this process, we noticed that, due to the particular nature of the ground profile, both islands would contain a weak layer of soils despite the soil improvement techniques. This weak layer would be created when the sand compaction piles are used to treat the in situ soft clay.

However, due to limitation of soil improvement equipment and technique, the range of application only included the upper part of the soft clay layer. The lower part is untreated as it was, considered as both difficult and expensive to do so. As a result, the initial ground profile would become a sandwich type, where a weak layer is overlaid by treated soils above and in situ hard soil below. During ground response under seismic excitation, the weak layer, due to its strong damping effects actually would help to reduce the magnitude of acceleration. This was clearly demonstrated by the analyses, and later on led to a technical paper (Li, Yang, et al., 2011b) in which the authors further explored the weak layer effects and brought to the industry and the academia a potential area of further study for innovative engineering applications.

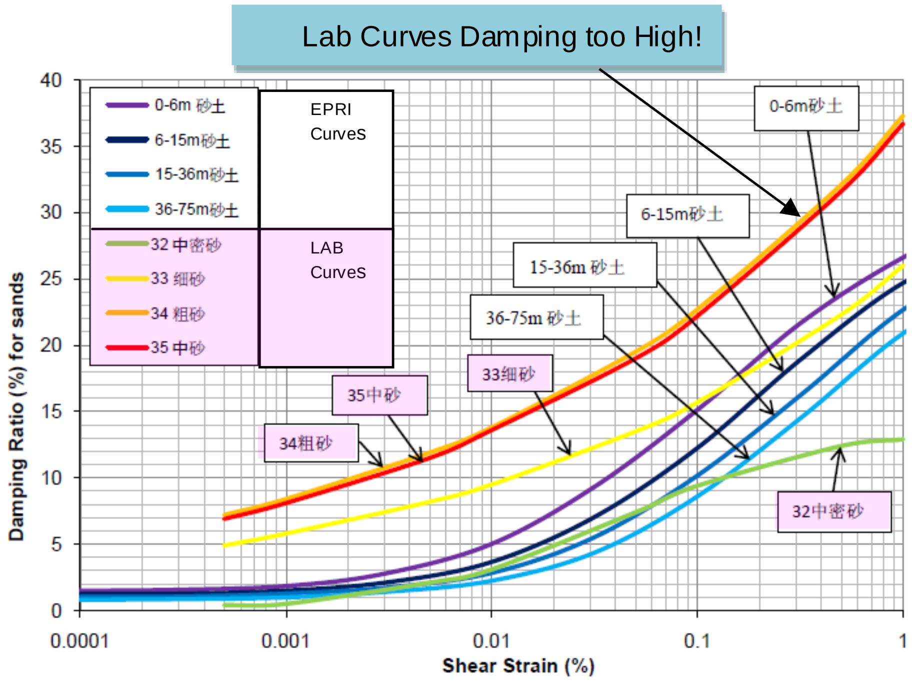


Figure 2 Dynamic soil curves - shear modulus degradation curve

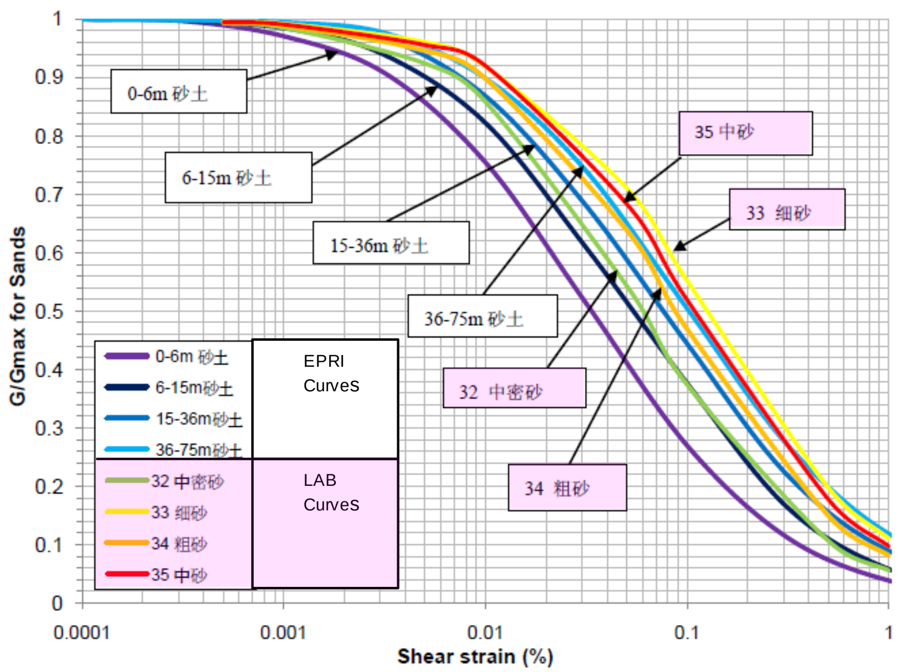


Figure 3 Dynamic soil curves - damping curves

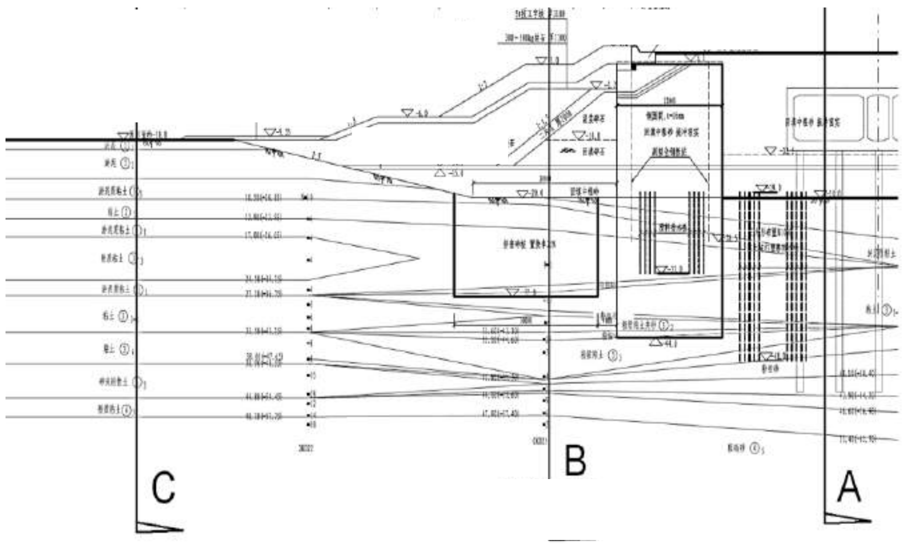


Figure 4 Illustrative ground profile and analytical sections

5 LIQUEFACTION ANALYSIS

Traditional liquefaction analysis was normally based on a series of publications by Seed et al (1971). For this project, we used one most updated version by Youd et al. (2001), which represents the state-of-the-art evaluation methods following the "simplified approach". The liquefaction Factor of Safety (F.S.) is defined as the ratio of CRR to CSR, where CRR is the cyclic resistance ratio and CSR is the cyclic stress ratio. From the design ground acceleration provided by China Earthquake Administration, we established a series of work sheets to evaluate the liquefaction potential on a borehole-by-borehole basis. Eventually we evaluated and demonstrated the sufficiency of the construction plans in which the factor of safety of all boreholes would be satisfactory.

6 DYNAMIC ANALYSIS USING COMPUTER METHODS

The last and most challenging part of design was the deformation analysis under dynamic loadings. 2D Dynamic analysis should account for both the 2D geometry effects and 1D ground response analyse. It is much more computationally intensive than a static 2D analysis since a dynamic analysis basically involves solving a nonlinear static problem at each time step. It is also much more complex than 1D ground response analysis, as it solves the problems in 2D and in time domain, while the 1D ground response analysis is solved in frequency domain. Two commercial computer software were used for this project, i.e., PLAXIS 2D ver. 8.0 (a finite element program) and FLAC 2D ver. 3.4 (a finite difference program). It was realized that both computer programs, however, are not directly suitable for the intended dynamic analysis, mainly for the reason of inconveniency in directly including the dynamic soil parameters which changes with shear strain level.

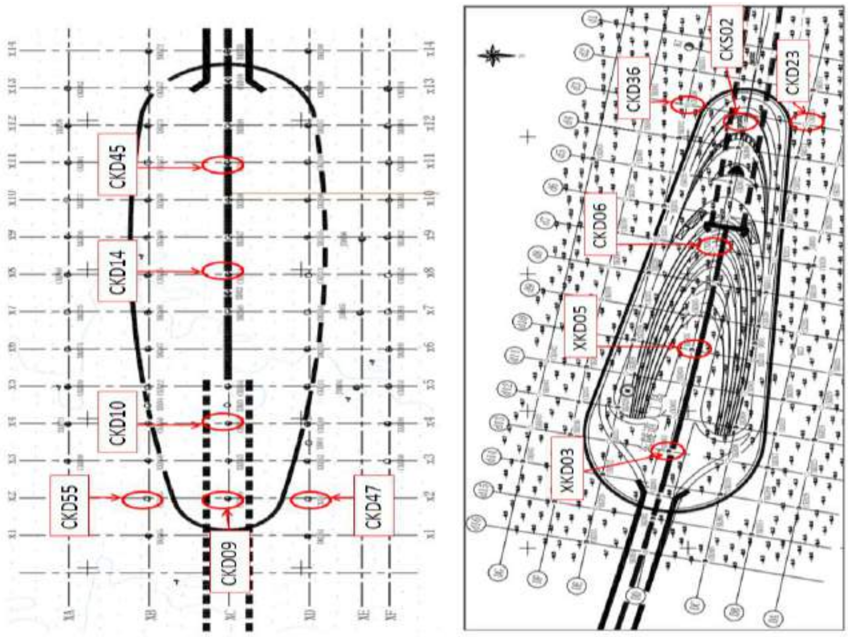


Figure 5 Illustrative borehole plan for liquefaction evaluation

Table 1 A sample calculation of liquefaction evaluation

Soil No.	Depth (m)	SPT-N Value	Elevation mPWD	$N_{1,60\text{-}CS}$	CRR	CSR	F.S.	Remarks
j_2	0.0	2	-8.70	-	-	-	-	Clay
j_3	9.0	2	-17.70	-	-	-	-	Clay
l_3	18.0	8	-26.70	-	-	-	-	Clay
m_3	25.3	24	-34.00	15.19	0.233	0.134	1.74	✓
l_2	27.0	28	-35.70	16.97	0.260	0.131	1.99	✓
m_5	36.0	34	-44.70	15.02	0.231	0.109	2.12	✓
m_3	42.0	50	-50.70	19.61	0.304	0.104	2.93	✓
m_5	46.5	50	-55.20	18.08	0.278	0.100	2.77	✓
m_3	51.0	50	-59.70	16.78	0.257	0.097	2.65	✓
m_5	52.5	50	-61.20	16.39	0.251	0.096	2.61	✓
m_3	57.4	50	-66.10	15.22	0.234	0.093	2.51	✓
m_5	59.1	39	-67.80	11.59	0.184	0.092	1.99	✓
p_3	69.8	40	-78.50	10.32	0.167	0.086	1.94	✓
p_3	71.4	50	-80.10	12.65	0.198	0.085	2.31	✓

The common used constitutive models in PLAXIS 8.0 consider only constant soil modulus during analysis. Our exploration on FLAC suggested that it was possible to use a special constitutive model (UBC Sand) for the sand. However, there would be the problem of establishing the material parameters for the advanced constitutive model. Laborious calibration work with experimental data to establish high quality input parameters are required, in addition to relevant past experience regarding the usage of this constitutive model.

In view of the complexity of the problem and the tight program for project delivery, we came out with an efficient way of linking the 1D ground response analysis with the 2D analysis.

First, based on the ground response analysis work, we had established a set of converged soil parameters considering the dynamic characteristics of soils for typical locations at the island profile. The converged soil parameters are associated with the expected strain level and therefore suitable for dynamic deformation analysis.

This set of soil parameters, if input into a dynamic PLAXIS or FLAC to model a 1D problem with the same setup as SHAKE91 would lead to approximately the same prediction with that from the ground response analysis.

To account for 2D effects in a straightforward way, we established a model which has many subdivisions in the horizontal direction in order to capture the 2D effects. Each subdivision therefore can be viewed as a 1-D soil/rock column which extends from bedrock to the ground surface of the island. This scenario will be approximately correspondent to the 1D problem.

In other words, for the same soil layer under static loadings, the dynamic properties can be different depending on the location relative to the slope of the artificial island. These different converged properties were established from SHAKE91 analysis and then applied to the subdivisions in 2D analysis (see Figure 6 for the geometry of modelling section, considering different soil blocks). In this way, 2D PLAXIS and FLAC were used to perform the dynamic analyses with good agreement with each other and with SHAKE91. See Figure 7 for comparison of acceleration time history results. A diagram illustrate the process of dynamic deformation analysis is shown in Figure 8.

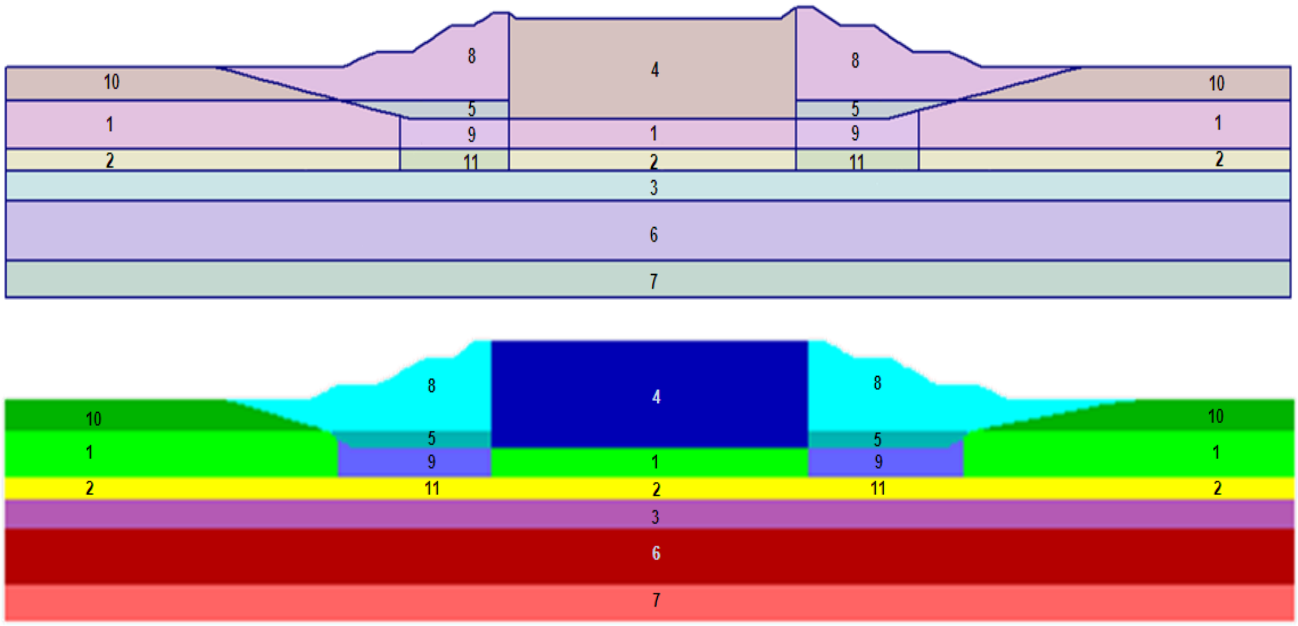
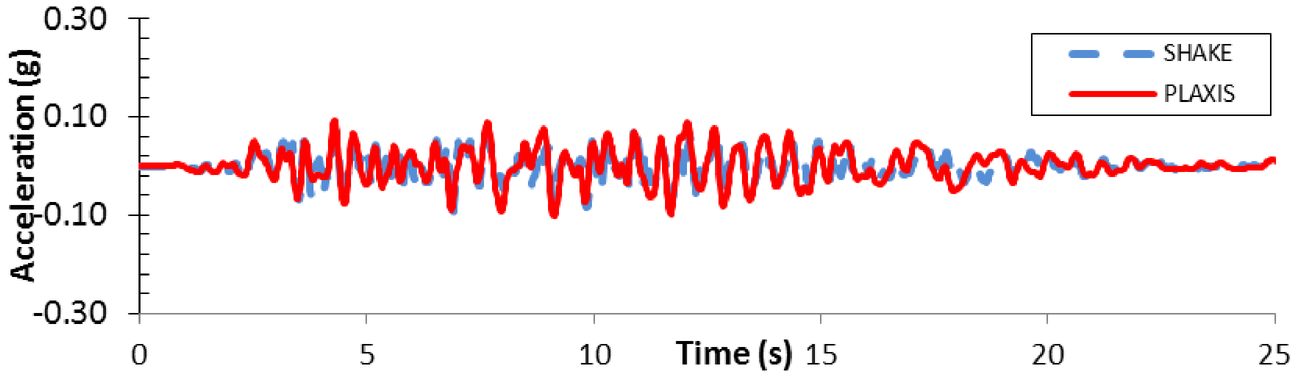


Figure 6 Set-up of analysis sections for PLAXIS and FLAC



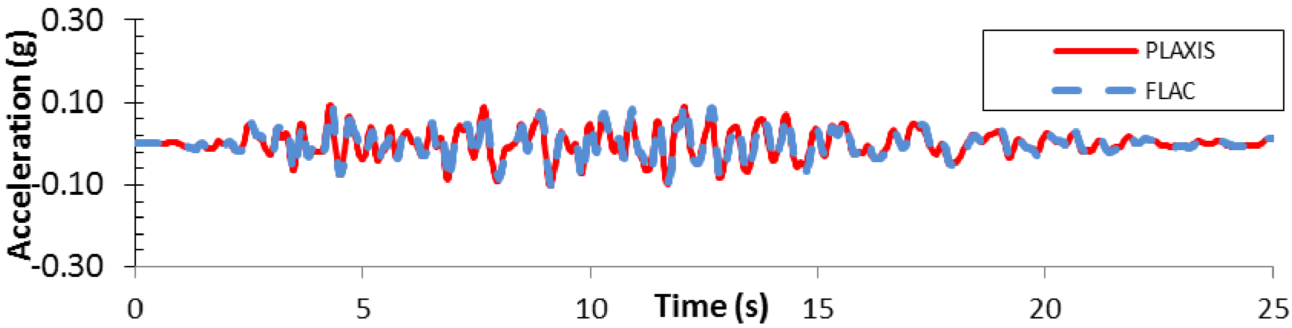


Figure 7 Comparison of analysis results among SHAKE91, PLAXIS and FLAC

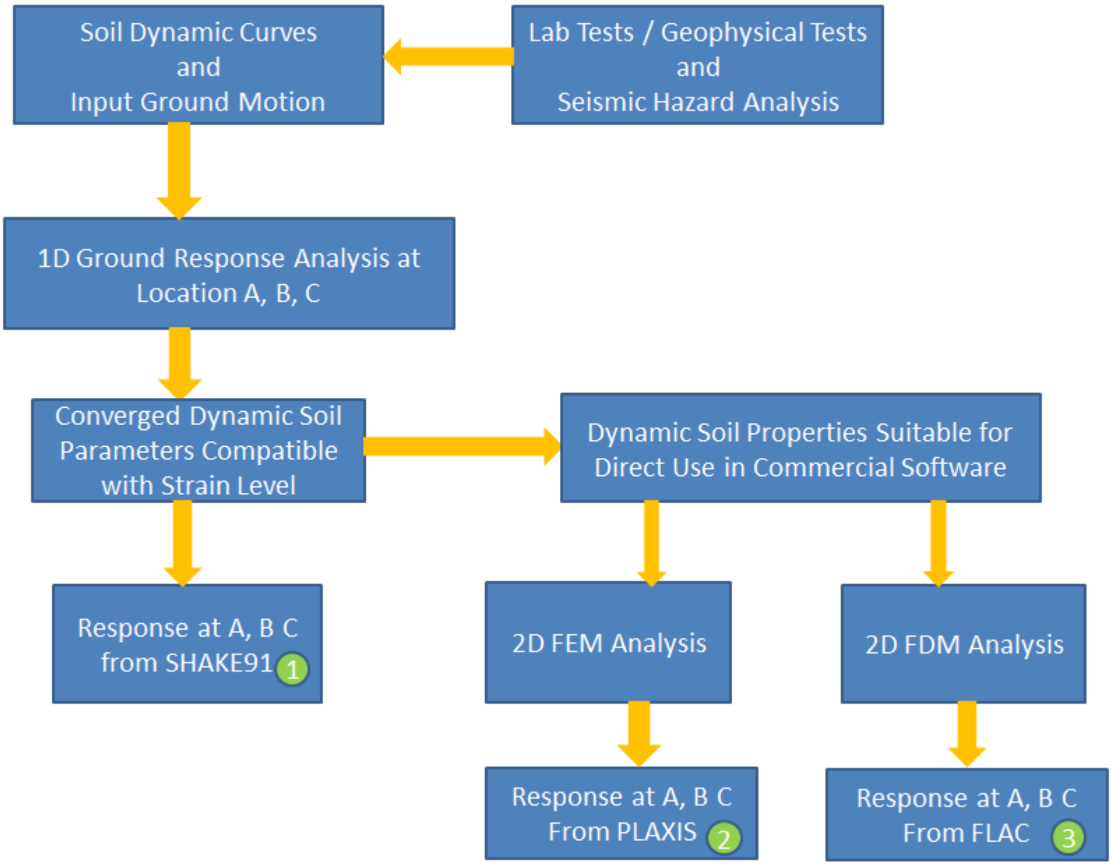


Figure 8 Flow-chat of process of dynamic deformation analysis using three computer software

7 CONCLUDING REMARKS

The seismic design of the two artificial islands (man-made islands) became a successful application of state-of-the-art design practice to a challenging engineering project. During the execution of the project, we have identified issues with data sufficiency and validity for seismic design. A practical way of performing dynamic deformation analysis was adopted to produce consistent results between 1D ground response analysis in frequency domain and finite element and finite difference time history analyses.

The lessons learned include the following:

a) The fundamental inputs to the problems should always be independently reviewed to make sure that the project team are fully aware of the limitations. The best chance to identify and correct any problem is at the start of a project. The success of this project was critically dependent on the initial critical review of the dynamic soil parameters provided by a third party.

b) We had successfully used multiple methods, including numerical analysis methods, analytical methods and design spread sheets, to examine one project. This approach not only firmly establishes confidence in design, but also promotes opportunities in spotting errors and ensures high-quality end results.

c) As consulting engineers are oftentimes found struggling between complexity of problem and scarcity of time, personnel and resources, we have solved the complex dynamic deformation analysis by combining limited-function 2D numerical analysis methods with 1D ground response analysis. This kind of innovation had helped the project to be completed within the time/budget, and avoided delay to project construction.

d) By combining scientific approach with engineering applications, after the completion of the project, the authors continued their research into relevant aspects such as the objectivity of ground response analysis, and liquefaction analysis considering existence of a weak layer (Li, Yang et al., 2011a, 2011b and 2011c). This is an example of engineers' use of out-of-the box thinking and also going an extra mile beyond the normal engineering practice, and consolidating engineering experience into knowledge which can benefit other practitioners and researchers.

e) A closer interaction between the industry and the academia is therefore called upon to foster knowledge growth as well as innovation to the major infrastructure development. This implies the requirement on the engineers' continual involvement in research works and smart way of applying scientific theory; also it calls on the academia to reach out to the industry and to address the limitations of the engineering practice and make contributions to continual development.

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Road to achieving zero carbon emissions

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Keywords: zero carbon building; zero energy building; carbon emission; bio-fuel; renewable energy; PV panel.

ABSTRACT: Completed in June 2012, ZCB is the first zero carbon building in Hong Kong with the aim to showcase the latest eco-building design and technologies and to promote the low carbon living concept. It is designed to achieve life cycle zero carbon emissions with on-site renewable generation, not only achieving energy self sufficiency during the operation stage, but also offsetting the embodied energy of the major construction materials and the construction process. This paper presents an overview of the energy performance of the ZCB during its first two and half year operations.

1 INTRODUCTION

Developed by the Construction Industry Council (CIC) in collaboration with the Hong Kong Government and completed in June 2012, the ZCB is the first zero carbon building in Hong Kong. It aspires to showcase the latest eco-building design and technologies applicable to Hong Kong which is of a high population density in the subtropical climate, and to serve as an education centre to inspire positive behavioural changes towards low carbon living.

Located in Kowloon Bay, Hong Kong, it covers a total land area of 14,700 m^2 surrounded by high rise office buildings (Figure 1). It comprises a 3-storey building with a footprint of 1,400 m^2 and a total gross floor area of 3305 m^2 and, a landscape area including an urban native woodland. The major components of the ZCB are listed as follows:

a) An indoor exhibition and education area to show case the latest zero/low carbon design and technologies;
b) An eco-home to promote low carbon living;
c) An eco-office for staff;
d) An eco-café to promote low carbon living;
e) A multi-purpose hall for conferences, seminars, exhibitions and corporate functions in a zero carbon environment;
f) A public green leisure space, which can also be used for outdoor exhibitions and events;
g) An urban native woodland to promote biodiversity; and
h) A ring path to demonstrate the concept and principles of "One-planet Living".

Figure 1 Perspective view of ZCB

Following a brief overview of the design principles and design target, the actual energy performance including performance gaps, improvement measures taken and experienced gained so far is presented in this paper. The ZCB has been in operation for 2.5 years since its practical completion in June 2012. As much of the data record during the first 0.5 year is either not reliable or incomplete due to the fact that most of the building systems, including the Building Management System (BMS), were not fully functional. The analysis presented in this paper was mainly based on the information collected during the most recent 21 months from May 2013 to January 2015.

2 DESIGN PRINCIPLES

The ZCB has its mission in carbon neutrality. That is, the building has net zero consumption of energy generated from fossil fuels. The energy needed for its operation is provided by renewable sources on site. The building is connected to the city grid for exporting surplus renewable energy and when needed, for importing electricity from the city grid. Production of on-site renewable energy offsets the power consumed from the grid on an annual basis.

The ZCB also goes beyond the common definition of a Zero Carbon Building by exporting surplus renewable energy to the local grid to offset the embodied carbon of its construction process and major construction materials.

The main consideration in planning for the above various components was to create a real use building blended with the environmental design and latest technologies.

Being in the subtropical area, Hong Kong's summer is long (May to October), hot and often humid. Afternoon temperatures frequently exceed 31^{o}C. The remaining months of the year are generally considered pleasant. As such building energy consumption peaks in summer due to air-conditioning. Keeping the building cool whilst minimising energy consumption in summer is the focus of the design consideration.

In responding to the aspirations set for this project, the following 4 "E" principles were adopted in the design:

a) "Experimental" – The ZCB shall be an experimental ground for the latest eco-building design technologies.

b) "Evolving" – Technologies evolve fast. The ZCB is designed to have as much flexibility as possible for upgrade with the future technologies.

c) "Evaluation" – As not all technologies adopted have been proven in Hong Kong, evaluation of those design and technologies in terms of capital costs, operation costs, energy saving and carbon reduction would provide valuable information for promoting their wider application to the construction industry in Hong Kong. Accordingly, over 2800 sensors are installed throughout the building to record and monitor the key environmental performance parameters, such as CO_2, temperature, humidity, renewable energy generation, energy consumption, energy import from the city grid, surplus energy export and light level. Those information is constantly fed into the BMS for recording, analysis, reporting and optimisation of building system performance.

d) "Education" – Even with the best design and all the latest technologies, it is still the people who use the buildings matter most in terms of energy saving and reducing carbon emissions. Much of the emphasis of the ZCB during the operation stage is education, particularly for the younger generation. Three sessions of guided tours are provided each day to the public with interactive displays to explain the design strategies, rationale and benefit of the key design features and technologies adopted.

3 DESIGN PERFORMANCE

On top of meeting the statutory requirement stipulated in the Building Energy Codes, the ZCB aims to achieve a further 45% energy saving through the various passive design measures and active systems. Table 1 summarises the designed energy consumption by the key components of the ZCB whilst Table 2 shows the expected energy savings compared to a similar building of the current standard (i.e. meeting the minimum requirements stipulated in the Building Energy Code).

By design, 60% of the energy need, or 87 MWh/year electricity, is met by the electricity generated from the PV panels. A Combined Cooling, Heating and Power Generation (CCHP) system provides another 110% of the energy needs, or 143 MWh/year, using waste cooking oil. As such, about 100MWh/year surplus electricity will be exported to the city grid to offset the embodied carbon. Translating to the CO_2 equivalent, this means a carbon reduction of 50 tonnes/year over the 50 year building life as illustrated in Figure 2.

Table 1 Operation stage – Distribution of energy consumption by design

Key Components	Energy	
	(MWh)	(%)
Mechanical Ventilation	10	10
Lighting	14	13
Cooling	41	39
Office Equipment (computers, printers etc)	34	32
Others	6	5
Total of Building	105	100
Other Activities in Landscape Areas	15	
Total Site Emissions	120	
Total Renewable Contribution	-230	

Table 2 Operation stage – CO_2 reduction and energy savings in building by design

Typical Design	Building Energy Consumption 200MWh/yr	
Energy Efficient Measures Taken in Design Energy Consumption	Building Energy Saving (MWh/yr)	Energy Saving (% of total building)
Envelope design	9	4
Ventilation design	14	6
Lighting design	47	20
Cooling design	38	15
Total Reduction	108	45

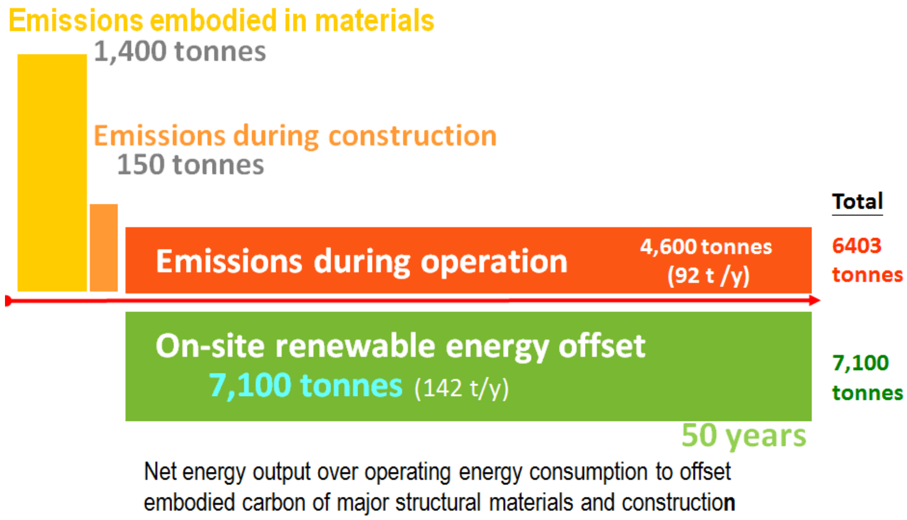

Figure 2 Carbon strategy

4 ACTUAL PERFORMANCE

4.1 *Extended Testing and Commission Period*

After its practical completion in June 2012, the ZCB went through an extended testing and commissioning period. A total of three rounds of energy audits were carried out. This was in addition to the daily monitoring by the operation staff of the ZCB through the BMS. Both the energy audits and the daily monitoring prompted a continuous fine tuning, testing and commissioning of various building systems, primarily the CCHP system, the BMS and the air conditioning system. During the period, the CCHP system suffered two major breakdowns. Although the nature of the breakdown was not serious, it took a few month each time for the ordered replacement parts to arrive and for the contractor to make sure the fault was fixed properly. Adding the fact that the CCHP was designed to operate in summer months only and was switched off during the rest of the year, the actual run time of the CCHP from July 2012 to the end of 2014 was only 7 months approximately.

There are a total of over 2800 sensors connected to the BMS for data recording, reporting and management. Many data gaps and abnormal records were identified and rectified during the study period.

As a result, the operation period up to the end of 2014 was really an extended testing and commissioning period and from the energy performance perspective the ZCB was in the process of continuous improvements.

4.2 *Free Cool Period*

The design assumed that the ZCB would be operating under the free cool mode for approximately one third of the year. In 2013, the free cool period was about 5 months, or 40% of the time. In 2014, the free cool period was 6 months, or 50% of the time. Whilst the actual duration of the free cool period depends on the weather of the year, it can be said that the cross-ventilation design strategy has been effective under the free cool mode.

However, coming with the open windows is the nuisance of mosquitoes. In spite of the regular spray of pesticides, presence of various mosquito traps and no apparent presence of stagnant water nearby, the nuisance of mosquitoes has been persistent in this low rise building. Other measures will have to be considered to improve the office comfort level.

During the free cool period, the indoor air quality can only be as good as the ambient air quality in the outdoor. The indoor air quality is classified as "Good" under the Indoor Air Quality Certification Scheme for Offices and Public Places. Surrounded by the busy traffic roads, the indoor air quality could not meet the "Excellent" class in terms of the NO_2 level and the respirable suspended particulates level.

4.3 *Energy Consumption*

The total energy consumption in the two year period from 2013 to 2014 was 663 MWh and the total renewable energy generation during the same period was 402 MWh, leading to a carbon footprint of 183 tonnes because the renewable energy generation was less than the energy consumption.

The yearly average energy consumption was 332MWh/year, 155% over the design estimate of 130MWh/year. The key factors of the energy performance gaps have been identified in the following:

a) The ZCB was designed to open for 5 days a week. It was actually open for 7 days a week, leading to 40% increase in energy consumption over the design estimate.
b) The design assumed negligible energy consumption during the non-operating hours from 19:00 to 08:00, but the actual energy consumption during the non-operating hours was about 22% of the total daily energy consumption in the summer and 30% in the winter. The major energy consumption during the non-operating hours was by the irrigation, landscape lighting, audio and video equipment on standby mode which could not be conveniently switched off, BMS, CCTV, emergency lighting and some evening events held at the Multi-Purpose Hall. The office staff remaining to work at the office well after the office hours also contributed to the increase in the energy consumption during the non-operating hours.
c) The energy consumption related to non-essential services such as the landscape lighting, basement floor energy consumption and pumping and drainage, was found to be significantly higher than expected. Figure 3 illustrates the distribution of the energy consumption by various systems in 2013. The distribution in 2014 is similar.

Based on the energy audits, a number of improvement measures had been undertaken to reduce the energy consumption. Those included operation improvement measures and facility management improvement measures as highlighted in the following:

a) All staff working in ZCB office were required to exercise strict energy control practice as recommended in the ZCB House Keeping Rules; the House Keeping Rules were incorporated as part of the induction package for all new staff;
b) Optimisation of ventilation operation for various plant rooms and utility rooms;
c) Optimisation of irrigation regime;
d) Optimisation of outdoor lighting schedule and emergency lighting schedule;
e) Provision of the switch off function for all audio and video equipment wherever possible.

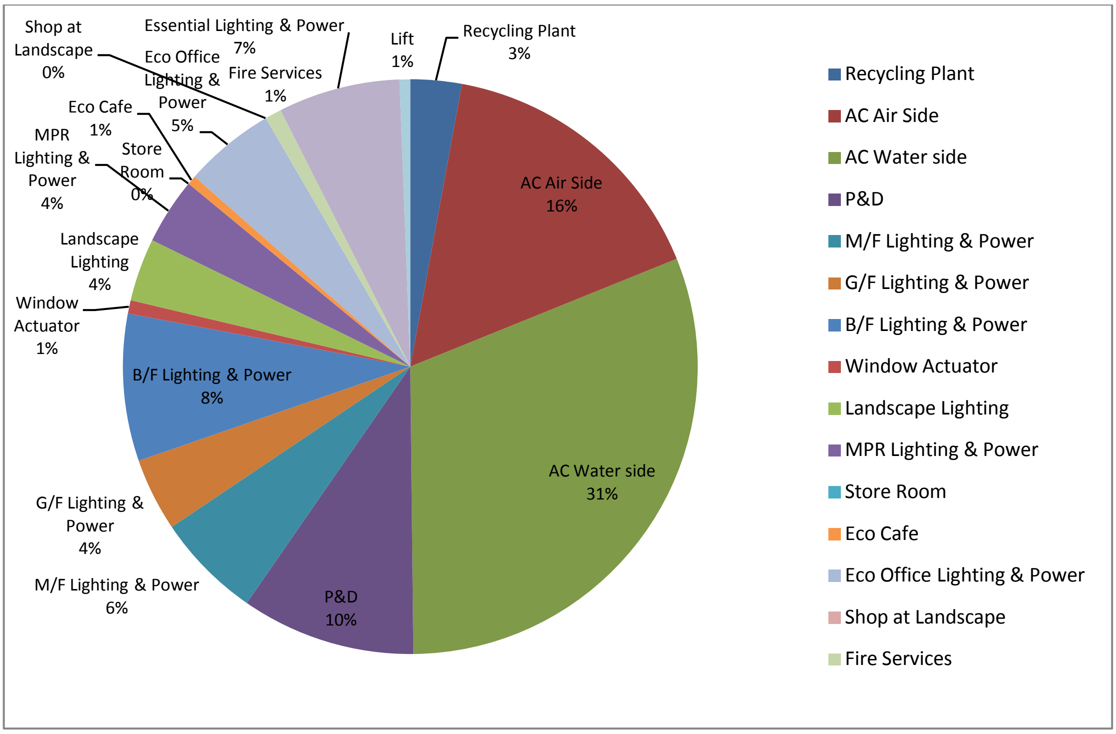

Figure 3 Distribution of energy consumption in May-Dec 2013

Implementation of those measures was completed over the period from July to December 2014. The average monthly energy consumption was reduced from 36MWh/month during the period from July to December in 2013 to 27MWh/month during the same period in 2014, as illustrated in Figure 4. The one month data in 2015 is not adequate to make a meaningful comparison. However, it does indicate a continuation of the improvement trend resulting from the measures undertaken from the latter half of 2014.

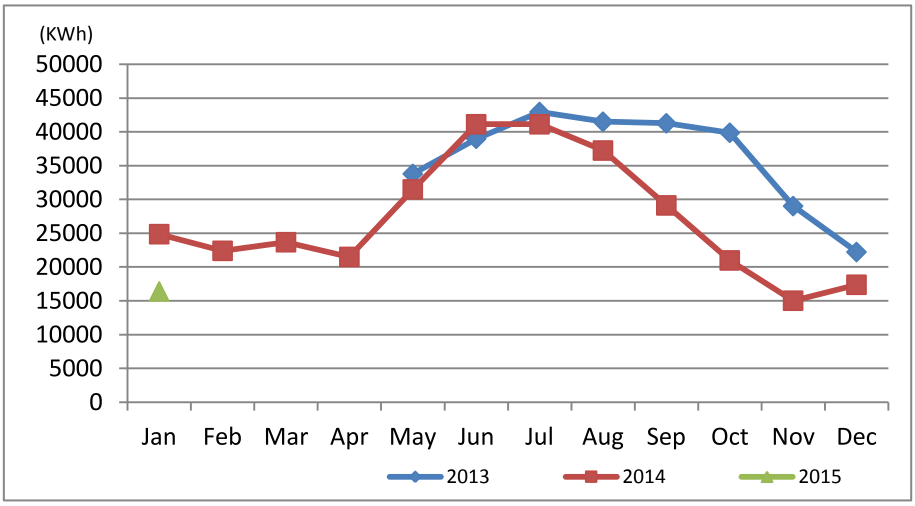

Figure 4 Comparison of monthly energy consumption in 2013, 2014 and 2015

Although some 26% reduction in energy consumption was achieved from those improvements, it is not adequate to offset the increased energy consumption due to the increased usage of the ZCB and the essential energy consumption during the non-operating hours. Following further measures have been undertaken since 1 January 2015 in order to achieve the zero carbon emission target:

a) Exclusion of the energy consumption in the landscape area (such as landscape lighting, eco-cafe and landscape irrigation) from the energy budget, which would also lift the energy constraints on hosting more outdoor activities;

b) Extending the CCHP operation from summer monthly only to all year round.
Figure 5 is a snapshot of the dashboard display as at 11 February 2015.

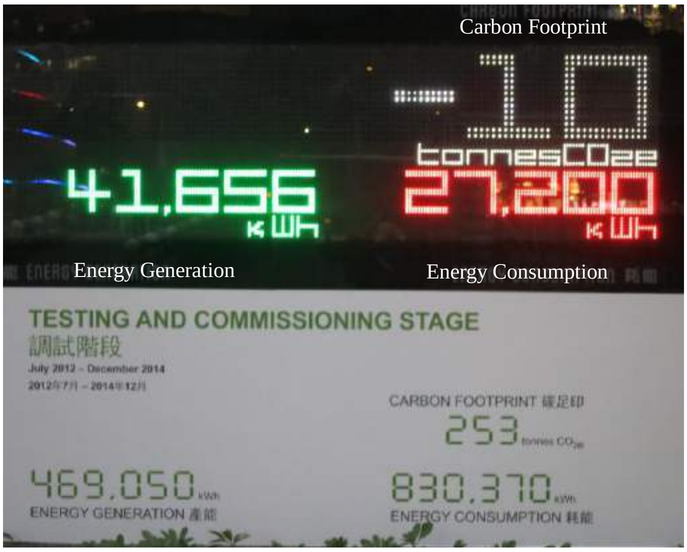

Figure 5 Dashboard display, 11 Feb 2015

As at 11 February 2015, the total renewable energy generation and energy consumption since 1 January 2015 were 41,656KWh and 27,200KWh respectively. The export of the surplus renewable energy of 41,656 - 27,200 = 14,456 KWh means a carbon reduction of 10 tonnes over the same period. Although the energy needs are expected to increase in the summer months, as to be illustrated in the later part of this paper, the renewable energy generated from the PV panels is also expected to be 60% to 100% higher in the summer months than in the winter months. The output from the bio-fuel generator of the CCHP system will remain largely constant throughout the year, provided no significant breakdowns. As such, the net carbon reduction of between 60 and 90 tonnes per year is projected for 2015, which exceeds the net carbon reduction requirement of 50 tonnes/year (Figure 2) in order to attain the life cycle net zero carbon emissions target, or 55 tonnes/year after taking into account the additional embodied carbon accumulated over the extended testing and commissioning period.

4.3.1 *Renewable energy generation*

The renewable energy generation was 14% short of the design target primarily due to a number of long duration breakdowns of the CCHP system. The PV panels performed consistently and generated 106 MWh electricity in 2013 and 108 MWh in 2014, which represents 22% and 24% respectively above the design output. The bio-fuel generator of the CCHP system suffered breakdown during May and June in 2013 and again from August and November 2014. As a result, the energy generation by the bio-fuel generator was well short of the design target.

However, the generator, running at 9.5 hours per day, had the output of 24MWh per month when it was in full operation. Six month operation in the summer months each year would generate a yearly total of 144 MWh/year, well in line with the design target of 143 MWh/year. Figures 6 and 7 illustrate the renewable energy generation by the PV panels and the CCHP system from 2013 to 2015.

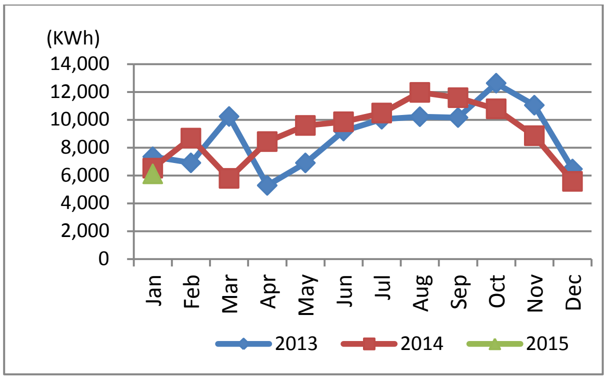

Figure 6 Energy generation by PV panels

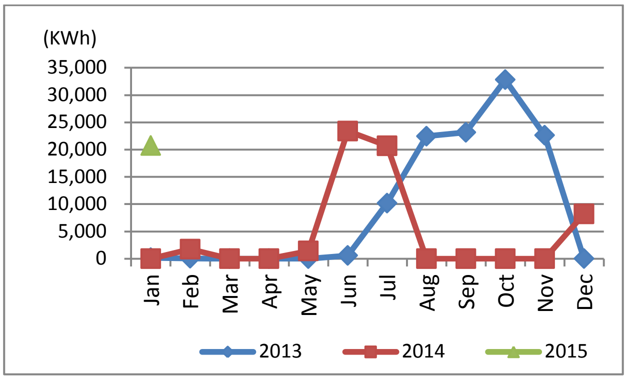

Figure 7 Energy generation by CCHP

5 SUMMARY

The assessment of the ZCB was based on its performance over the first 2.5 year operations from July 2012 to December 2014. The energy consumption of the ZCB was found to be significantly higher than the design estimate. The major courses were found to be the much higher usage of the ZCB facilities than the design assumption, essential energy consumption during the non-operation hours which was not expected in the design, and higher than expected energy consumption by the non-building related activities such as landscape lighting and pumping for irrigation, grey water recycling and wastewater recycling.

Some 26% reduction in the energy consumption was achieved through the implementation of a range of operational and management measures, which highlighted the importance of the human behaviour in achieving the zero carbon emission target. However, the total energy consumption still more than doubled the design assumption.

The renewable energy generation by the PV panels outperformed the design assumption. The CCHP suffered badly from some long duration down time primarily due to the waiting time needed for parts replacement. However, the CCHP was capable of generating the designed energy output when operating normally. The adsorption chiller of the CCHP system was performing satisfactorily, but the desiccant dehumidifier, which was designed to be powered by the waste heat of the bio-fuel generator, was left idle as all the waste heat from the bio-fuel generator was diverted to the adsorption chiller in order to maximise the cooling efficiency by the adsorption chiller.

To achieve the zero carbon emission target, the energy consumption has to be further reduced, or the renewable energy generation needs to be increased, or both. To this end, as from 1 January 2015, the energy consumption in the landscape area has been excluded from the ZCB energy budget, which also has the benefit of lifting the energy constraints on hosting more outdoor events, and the operation of the bio-fuel generator of the CCHP system has been extended from the summer months only to all year round. The operation since 1 January 2015 so far indicates that the ZCB is well on its way to achieve the life cycle zero carbon emission target.

The operation track record of the CCHP system has so far not been satisfactory. In spite of the enhancement of the maintenance regime including keeping stock of the essential spare parts, the risks of the CCHP system breaking down again in the future cannot be ruled out. The provision of some spare capacity or a backup system, will be necessary.

Comparative analysis of technologies and methods for automatic construction of building information models for existing buildings

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Keywords: Building Information Modelling (BIM); existing buildings; as-built BIM; data capturing; data processing.

ABSTRACT: Building Information Modelling (BIM) provides an intelligent and parametric digital platform to support activities throughout the life-cycle of a building and has been used for new building construction projects nowadays. However, most existing buildings today do not have complete as-built information documents after the construction phase, nor existed meaningful BIM models. Despite the growing use of BIM models and the improvement in as-built records, missing or incomplete building information is still one of the main reasons for the low-level efficiency of building project management. Furthermore, as-built BIM modelling for existing buildings is considered to be a time-consuming process in real projects. Researchers have paid attention to systems and technologies for automated creation of as-built BIM models, but no system has achieved full automation yet. With the ultimate goal of developing a fully automated BIM model creation system, this paper summarises the state-of-the-art techniques and methods for creating as-built BIM models as the starting point, which include data capturing technologies, data processing technologies, object recognition approaches and creating as-built BIM models. Merits and limitations of each technology and method are evaluated based on intensive literature review. This paper also discusses key challenges and gaps remained unaddressed, which are identified through comparative analysis of technologies and methods currently available to support fully automated creation of as-built BIM models.

1 INTRODUCTION

BIM intends to develop the process and use of a computer generated model to simulate the planning, design, construction and operations and maintenance of a building (Azhar, 2011). The resulting building information model not only focuses on creating a simple 3D model but a data-rich, object-oriented, intelligent and parametric digital representation of the building to support diverse activities throughout the life-cycle of the building. This smart model providing an intelligent platform could extract and analyse the whole construction project achieving various users' needs, such as energy simulation, structure analysis, and construction plan, etc.

Buildings can be classified into three types according to their ages: new buildings, existing buildings and heritage buildings. The majority of existing buildings and heritage buildings constructed before the BIM was introduced do not have initial or updated BIM models. With the development of the BIM platform today, the industry has started to use BIM-based tools for new construction projects, and furthermore the emphasis related to the BIM research also include operational phase and maintenance phase or even retrofitting/refurbishment phase of existing buildings. As shown in Figure 1, process (1) stands for creating a BIM model for new building from the design phase through the retrofitting phase, while process (2) presents creating an as-built BIM model for existing buildings.

Missing or incomplete building information is one of the main reasons leading to manage existing building projects with low-level efficiency. During the operations and maintenance phase or the retrofit phase, uncertain recognition of existing building documents and inaccurate assumptions about the condition of the building might result in unintended errors or even accidents. Since the BIM is accepted to act as a digital platform with comprehensive building information, it also should be applied to operate and manage existing buildings. In reality, however, there are a lot of challenges in creating BIM models for existing buildings due to, for example, 1) Missing or lacking complete or effective building documents; 2) the whole process of creating an as-built BIM model needs efforts, costs and time. Skilled workers are also necessary to successfully complete the process (Figure 1).

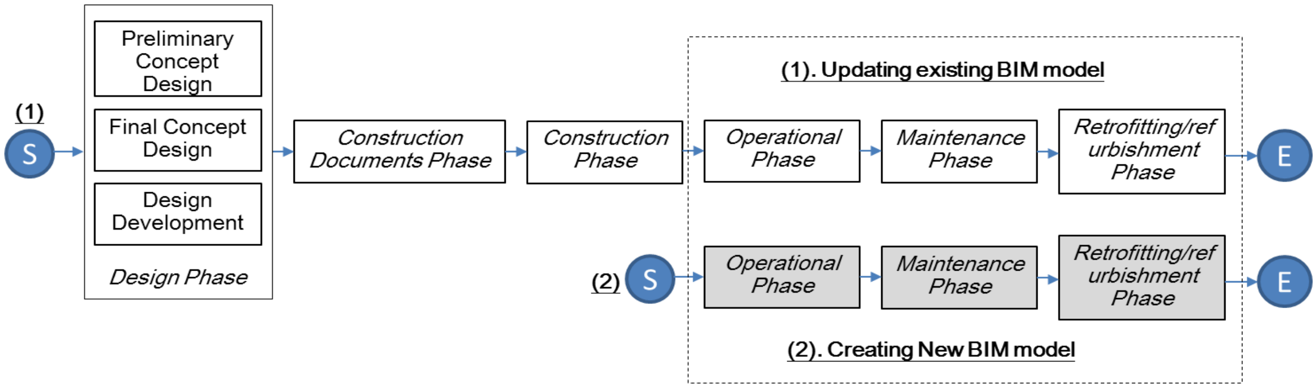

Figure 1 Comparison existing buildings and new buildings of BIM model creating process in LC

Considering that it is a time-consuming and costly manual process to create an as-built BIM model, research has concentrated on how to make the process fully automatic. On one aspect, a BIM model is not a simple 3D geometry model, but each component is registered and connected with their building information. On another aspect, building elements hidden behind another elements are hard to identify from the actual building (e.g. pipeline systems are always settled under the decoration layer) (Volk et al., 2014). Hence, it is indeed a difficult research problem to automatically create existing building models, which has been investigated by many research groups over the last 30 years, while the industry has not gotten enough breakthroughs so far (Nagel et al., 2009). These difficulties have been mentioned by many researchers before and some of the key difficulties are summarized as following:

a) Redundant efforts and resources are needed. As we create an as-built BIM model, both geometry information and building information are required. Sometimes it is not easy to collect all necessary data. Extra time and effort are the must (Arayici, 2008);
b) Complex processes of input data and reconstructed models (Nagel et al., 2009);
c) Extra data errors and inaccuracies; There exist uncertainties in interpretation and an additional verified process is needed (ibid).

In recent decades, there are a lot of contributions in this area. This paper mainly focuses on comparing and analysing technologies to create as-built BIM models. In the following sections of this paper, data capturing and building surveying technologies applying in real projects are explained. Data processing technologies and object recognition approaches are also compared and summarized in the second part, followed by describing as-built BIM modelling. Based on the knowledge and lessons learnt in this study, research conclusions and future work are described in the last chapter.

2 COMPARISON AND ANALYSIS OF CURRENT TECHNOLOGIES & METHODS

Creating an as-built BIM model can be differentiated between for new and for existing buildings, as the degree of difficulty, the quality of building information and the availability and functionality requirements of building information are varied from each other (Volk et al., 2014). For new buildings, according to various requirements of designers and owners, the BIM model is created in an interactive way using BIM-based design authoring applications (e.g. Revit, ArchiCAD) in the design phase. For existing buildings, however, there are two different scenarios. If there exists an initial BIM model of the target building, it is only required to update the model with missing and newly-created building information in the construction or operation phases, and correct inaccurate information. If there is no existing model, a new BIM model needs to be created by directly gathering all the information from the actual building and available resources such as drawings and specifications documents.

Automated as-built BIM creation means achieving a streamline, which starts with an input (e.g. point clouds/images/video/others) and ends up with the as-built BIM model, while the whole intermediate processes apply semi-automated or automated techniques saving effort & time and improving efficiency (Figure 2).

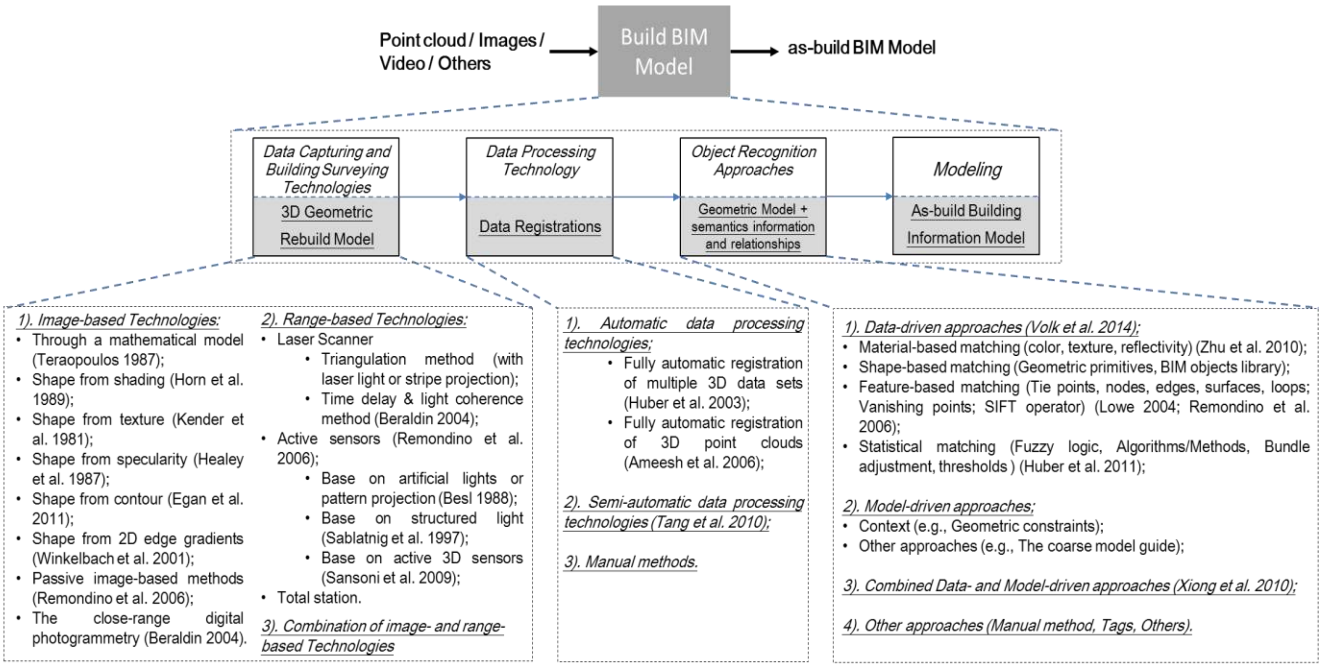

Figure 2 Technologies and processes of reconstructing as-built BIM model

As shown in Figure 2, creating an as-built BIM using different technologies can be divided into four main steps: 1) data capturing step in which various building surveying technologies can be chosen and 3D geometric models are created; 2) data processing step in which all measurements from the 1st step (e.g. point clouds) are combined and registered into one representation in a global coordinate system; 3) object recognition step in which the 3D geometric models in the global coordinate system are complemented by semantic information and objects also are given relationships with each other; 4) modelling the as-built BIM model step in which the primary partial information model will become a semantically rich BIM. In this model, every object has its meaning and information. Further, the whole process and primary goal should be gained at reasonable time and cost.

2.1 *Data Capturing and Building Surveying Technologies*

Creating a 3D geometry model of an existing building is a process of applying data capturing and building surveying technologies to measure and to capture existing objects and/or around environments, and then choosing reasonable software or techniques to get a complete 3D model. Data capturing technologies can be divided into contact based and non-contact based technologies. Considering that 3D geometry models are usually produced using non-contact based technologies in recent years, this paper concentrates on discussing this type. They can be further classified into three sub-categories: image-based, range-based, and comprehensive technologies, as shown in Table 1. Image-based technologies depend on geometry or surface characteristics of building elements. For example as shown in Figure 2, accurate 3D geometry models are created from shading, texture, and contour of buildings from their images, while range-based technologies obtain 3D models directly from the actual building according to geometric information of buildings with high accuracy (such as laser scanners or total stations). However, generally the cost of the range-based technologies is higher than the image-based technologies.

Hence, it is usually not an easy task to choose a suitable technology. There are a lot of factors that should be taken into consideration. When researchers evaluate and confirm a reasonable data capturing and building surveying technology, important specifications and factors affecting the decision should be determined. GSA (2009) has mentioned about some of these factors including project requirements (e.g. measurement uncertainty, resolution, and level of detail), project schedule, time, and costs. Based on discussions and prior research, we have identified eight factors in the decision process as following:

a) Degree of automation (1. automatism of the whole correspondence process; 2. meeting project requirements) (Kandil et al., 2014);
b) Applicability to free-form objects and fitting for BIM transformation (Mian et al., 2005);
c) Accuracy (reducing errors and level of details) (ibid);
d) Efficiency with respect to time (ibid);
e) Robustness to the range image resolution and surface sampling (ibid);
f) Robustness to dealing with adjacent views (Volk et al., 2014);
g) Data Volumes (ibid);
h) Cost (ibid).

Table 1 summarizes the results of evaluating some technologies currently available by these eight factors. The use of laser scanners shows promising properties of creating a complete 3D model, but needs relative large data volumes and high cost.

The table also shows that any single technology is not possible to meet all the requirements. So combination of different technologies is the comprehensive way to overcome and supplement limitations of each individual technology (Volk et al., 2014). Photogrammetry and laser scanning have often been used together to capture complex or large objects of a building. In this way, a complete and detailed 3D model data can be obtained efficiently. For example, Liu et al. (2012) used a laser scanner and a camera to build a 3D model in only achieving incomplete as-built data condition.

In real projects, it is not always practical to scan all the information of a building in the light of laser scanning. Besides, the image-based technology is difficult to deal with irregular surfaces and cannot provide the model automatically. So the most logical and effective method is: the basic outlines (e.g. the building surfaces) are captured by image-based technologies (e.g. photogrammetry) and the details are modeled through range-based one (El-Hakim et al., 2002).

2.2 *Data Processing Technology*

Data processing technology aims at transferring the image-based and range-based geometry data in its local coordinate system to the global coordinate system, as the initial data are expressed from different kinds of data capturing methods in their local coordinate frames. All of the data should go through this transformation process to be presented in the common global coordination system. This process is also known as registration as shown in Figure 2 (Tang et al., 2010). Some automatic data processing technologies have been available. For example, Ameesh et al. (2006) developed a technique, which could register three dimensional (3D) point clouds automatically. This comprehensive technique is established basing on the two related Extended Gaussian Images (EGIs) in the Fourier domain and taking advantages of harmonic transforms. It could fully automatically transfer point clouds with little overlap. Huber et al. (2003) also introduced an automatic method to register multiple 3D model data. This registration algorithm tries to transfer the input data into surface meshes through a surface matching engine. However, they are still a manual or semi-automatic process.

Table 1 Summary and comparison of data capturing technologies
(Furukawa et al. 2009; Nagel et al. 2009; Tang et al. 2010; Volk et al. 2014)

Data Capturing Technologies	Types of Technologies		Characteristics of Technologies 1	2	3	4	5	6	7	8
Non-contact Technologies	Range-based Technologies	Total Stations	M	M	M	M	M	M	H	M
		Laser Scanning (e.g. terrestrial/airborne laser scanners, LADAR, LiDAR)	Y	Y	Y	M	Y	M	H	H
	Image-based Technologies	Photogrammetry	M	N	M	Y	M	M	M	L
		Videogrammetry	M	N	Y	M	M	M	M	M
	Other Technologies	Pre-existing information (e.g. photo, geometry)	N	M	Y	Y	N	N	M	M
		Tagging (e.g. RFID, Barcodes)	M	N	M	Y	Y	Y	L	M
Contact Technologies	Manual Technologies	CMMs Callipers Others	N	N	M	N	N	N	L	L

*Y stands for fully satisfying all the requirements; M stands for marginally satisfying the requirements (1-6) and the medium level for data volume and cost (7 and 8); N stands for dissatisfying the requirements;
*H and L stand for high and low, respectively;
*LADAR: Laser Detection and Ranging; LiDAR: Light Detection and Ranging; RFID: Radio-frequency Identification; CMMs: Coordinate Measuring Machines.

2.3 *Object Recognition Approaches*

The third step is to recognize building components and connect them with their building information. For example, object recognition approaches need to recognize classes of objects, such as recognizing all the window objects with various height, width, etc. As shown in Figure 2, object recognition approaches can be divided into four types: data-driven, model-driven, combined data- and model-driven approaches, and others. Data-driven approaches recognize objects and sort building information based on their captured data. As shown in Table 2, these methods can be mainly classified into feature-based matching, shape-based matching, material-based matching and statistical matching, while model-driven approaches require the predefined structures to extract building information (Volk et al., 2014). These approaches always rely on the architectural information or context. Table 2 also summarizes disadvantages and advantages of the approaches that are the most commonly used ones nowadays. However, comprehensive methods (i.e., combined data- and model-driven approaches) have also been applied in order to integrate different methods and overcome disadvantages of each approach. For instance, Xiong and Huber (2010) used building context to create semantic 3D models of indoor environments. Despite the efforts, manual identification is still one of the most common options in the practice.

The statistical matching method is considered to have higher geometric accuracy. In particular, the fuzzy theory has been highly chosen by many researchers as a natural representation framework for real-world concepts (Lee et al. 2009). The fuzzy theory is also recognized as a promising method applied in object recognition approaches. Kim et al. (2009) tried to achieve real-time object recognition by using neuro-fuzzy to control workload-aware task pipelining.

Table 2 Summary and comparison of object recognition approaches
(Zhu et al. 2010; Lowe 2004; Volk et al. 2014; Huber et al. 2011; Tang et al. 2010; Lee et al. 2009)

Object Recognition Approaches		Types of Approaches	Advantages	Disadvantages
Data-driven Approaches	Feature-based matching	Tie points, nodes, edges, surfaces, loops	Flexible (allowing for large image variations); Automatically extracted and matched features;	Unsuitable for similar elements detection; Redundant information (too much for 3D model); High requirements during this process;
		Vanishing points		
		SIFT operator		
	Shape-based matching	Geometric primitives	Divided into explicit shape, implicit shape and relationship representations (Explicit one is suitable to describe free-form objects); Implicit one can describe geometric needs accurately for building BIM model;	Explicit method sometimes may lead to mix up the volumetric objects and surface ones; Implicit one is not well suited for algorithms, which use for automatically recognise objects;
		Segmentation, cell decomposition		
		BIM objects library		
		Images/videos		
	Material-based matching	Colour/Texture	Being suitable for collecting information of structural elements;	Unable to identify correctly structural element (e.g. these elements connected each other or made of the same material);
		Reflectivity		
	Statistical matching	Fuzzy logic	Higher geometric accuracy; Being independent from the shape of components;	Complex images fitting should be achieved manually; Semi-automatic;
		Algorithms/Methods (FEM, LSM)		
		Bundle adjustment		
		Thresholds		
Model-driven Approaches	Context	Constraints	Full automation; Being reliable and accurate for create building models.	High requirements (e.g. feature detection and closely spaced images).
	Other Technologies	The coarse model guide		

*SIFT: Scale-invariant feature transform; FEM: Finite element method; LSM: Least square method.

2.4 *Creating As-built BIM Models*

Creating an as-built BIM model aims at: firstly, the representation of the BIM model can match and express geometric components accurately, which contain individual surfaces and volumetric shapes. Secondly, the geometric components of the BIM model also need to be labeled with an object category. Furthermore, relationships among different components should be described clearly. For instance, walls must be connected to slabs at the bottom (Volk et al., 2014). If the as-built building data comes from data capturing and building surveying, it requires the building information in details and with high accuracy as well. The "level of detail" (LoD) is an effective method to describe the information richness and the level of accuracy based on the primary goal of the construction project (Leite et al., 2011). When we create building information models, the LoD defines the quality and integrality. We also use it to verify the accuracy of an as-built BIM model, when different methods are chosen to construct the BIM model.

As mentioned earlier, it takes a significant amount of time and effort to build an as-built BIM model in real projects. It often needs several months even for highly skilled modelers to build a BIM model for an existing building just with common size, which will depends on the requirements and details about the project model (Tang et al., 2010). Figure 3 lists examples of the software applications commonly used in the practice, which output BIM models or CAD models. Although the

Potomodeler claims to generate 3D models based on images taken from a typical camera, it does not achieve fully automated process of creating as-built BIM models.

Some researchers intended to reduce time depending on pre-existing information, as the preexisting information of a building can predigest the whole BIM construction process and it provides a reference for further progress. However, it is still needed to gain extensive attention and deeper research on this topic. An automatic or semi-automatic system is necessary and imperative to improve efficiency of building as-built BIM models for existing buildings.

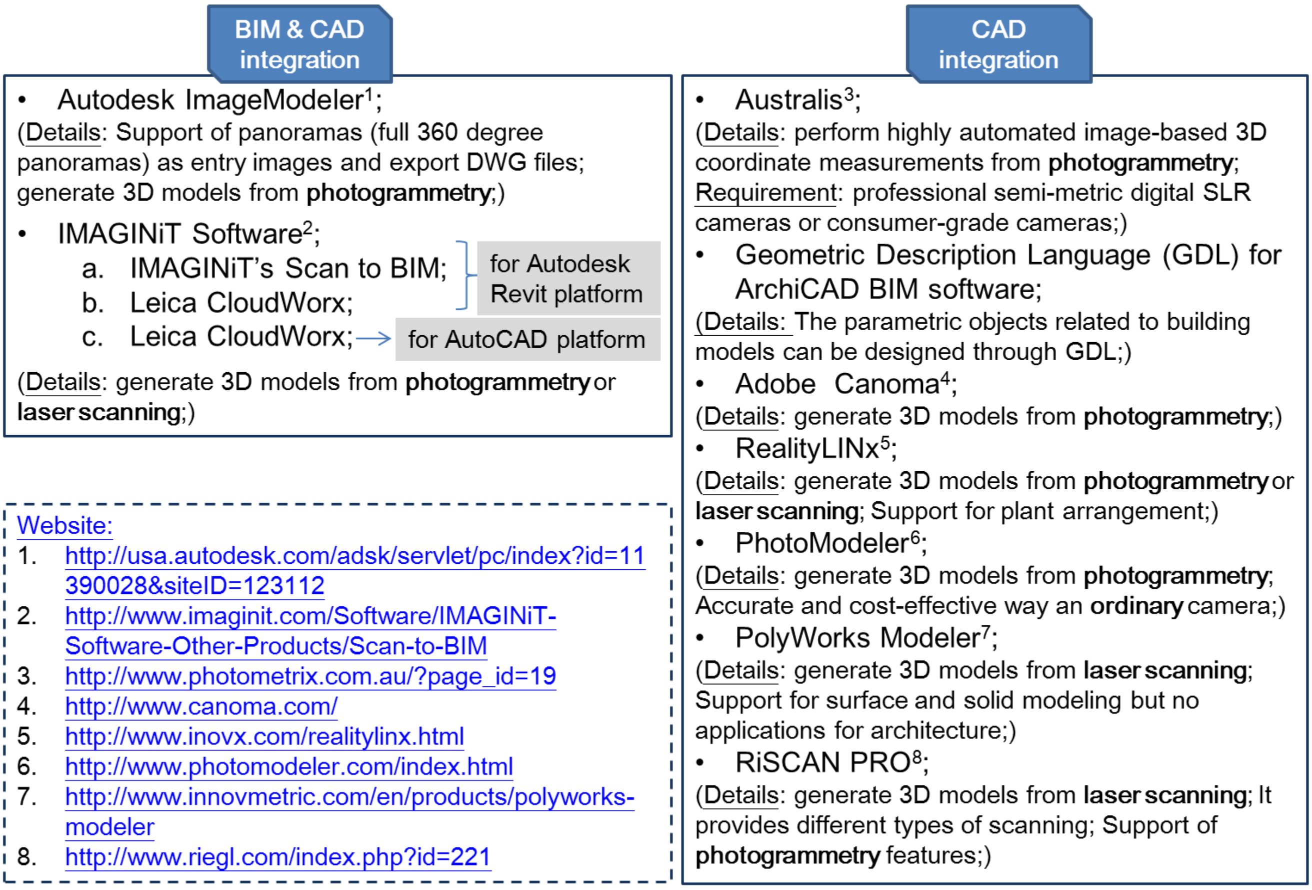

Figure 3 Summary of capturing and modeling software with respect to BIM or CAD integration

3 OVERVIEW OF SEMI-AUTOMATIC OR AUTOMATIC METHODS OF CREATING BIM MODELS FOR EXISTING BUILDINGS

The creation of an as-built BIM model focuses on surveying the geometry and surface of an existing building, improving this collected information into a primary semantically rich model and finally achieving a building information representation referring to LoD. In Chapter 2, the process of creating automated or semi-automated as-built BIM has been introduced and four steps of this streamline with existing techniques & methods within the relevant fields are also compared and analysed (Figure 2).

This chapter has twofold aspects: 1) to list the state-of-the-art systems for creating as-built BIM models; 2) to evaluate their merits and systems for automating the process and to put forward the possibility and main gaps to achieve fully automating the process.

3.1 *State-of-the-art of Creating As-built BIM Models*

Nagel et al. (2009) developed a two-step BIM model construction process. The process starts from 3D geometry models, which can be generated from different methods, such as laser scanning or photogrammetry. Then, they choose CityGML as an intermediate layer transferring geometry models to CityGML building models. The last step is to create IFC (Industry Foundation Classes) building models from CityGML. Since there are considerable gaps between the pure 3D geometry model and the as-built BIM model, it is difficult to achieve a fully automated creation process directly. In order to simplify the overall process and to increase the flexibility and operability, Nagel and his research team proposed the system. However, limitations include: 1) formalizations for CityGML or IFC are needed; and 2) the optimal and suitable data interpretation is also an issue.

Brilakis et al. (2010) proposed a framework combining video capturing and laser scanning technologies for automated as-built BIM creation. A 3D geometry model is first created through recording the geometry information and generating semantically needful components (e.g. columns and beams) as well. Then, fulfilling relationship definitions and fitting object recognition algorithms achieve as-built BIM construction. The main contribution of this research is reducing redundant tasks automatically and allowing for concurrent modelling processes.

Murphy et al. (2013) devote on Historic Building Information Modeling (HBIM). The HBIM is a smart library of parametric objects for historic buildings, which matches point cloud and image data. These parametric objects were created depending on Geometric Description Language (GDL), which is a kind of programming language embedded in the ArchiCAD, a commercial BIM authoring application. The final HBIM would automatically create cut sections and 3D models. However, it is still under development for real projects.

Kandil et al. (2014) provided a case study in which they captured a progressive history of a renovation project using a laser scanner and a camera to develop a complete as-built BIM model. However, the difficulty may concentrate on how to integrate different information sources.

There are also some researchers who have tried to start from a category of buildings or environmental components. For example, Livny et al. (2010) provided the tree reconstruction method and it can automatically rebuild multiple tree models directly from laser scanning data.

3.2 *Analysis*

Current as-built BIM creation processes are time-consuming and costly, and even sometimes the as-built BIM model may be considered to be meaningless and counteract the benefits for civil infrastructure projects, when comparing with effort and time. Although many systems have been proposed and developed, it is still a long way to achieve automated as-built BIM creation for existing buildings. The main research areas needed to be concentrated on and addressed include:

a) How to choose suitable capturing technologies with relatively low cost;
b) How to model complex structures;
c) How to easily distinguish the target building from surrounding extra environments;
d) How to represent models automatically using volumetric components rather than surface representations; and
e) How to transfer the input data effectively during data processing phase.

The whole papers described show promising opportunities of automated as-built BIM creation in future, but required more automated and intelligent way to achieve this goal.

4 CONCLUSION AND FUTURE WORK

It is beneficial to create an as-built BIM model for existing buildings as the as-built BIM models can help improve data management, support decision makings, increase management accuracy in the post-construction phases, and facilitate operations and maintenance of the building. However, current methods and technologies of creating as-built BIM models mainly depend on human effort. Although data may be collected automatically from diverse sources and methods such as laser scanners or camera, integration of the raw data, recognizing building objects and building logical relationships between the objects are all performed manually. In this paper, we began with the introduction of the streamline of creating an as-built BIM model. Four steps have been compared and analyzed in details, which contain data capturing, data processing, object organization and BIM model creation. Then, we focused on surveying and comparatively analyzed potential techniques and currently promising systems to automatically create as-built BIM models. Both promising opportunities and challenges are also discussed in this paper.

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The use of Building Information Modelling

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Keywords: Building Information Modelling (BIM); infrastructure; design and construction.

ABSTRACT: Digital data is now integrated with the concept, design, construction and maintenance stages of a civil engineering projects; its application is often termed Building Information modelling (BIM). With recent advances and increased understanding of its uses it has become a routine part of engineering contracts. An example of its application has been for utility installations, which are typically located below ground, out of view and within a congested space in Hong Kong. Suitable application of BIM for utility detection is therefore vital to limit impacts and interfaces with other construction works. Despite the advances challenges exist in the effective use of BIM, mainly compatibility and transfer of data between different software types and application and a consistent communication between the many specialist parties using BIM. This paper provides an overview of the history, definition and understanding of BIM and its application to civil engineering. A summary of published data with particular focus on its use for utility design and construction for infrastructure projects is provided.

1 INTRODUCTION

BIM was originally referenced in the early 1990s and became a more common used terminology for software users during the last decade. Its use for visual communication is vital and is advancing with technological advancement and the range of the available software. However, like many other specialist fields of civil engineering, effective application and interface of BIM within a management framework is important for its successful use in an engineering project. This paper provides a brief definition and history of BIM, an overview of its application for civil engineering projects. An outline of the work flow and interfacing typically adopted in Hong Kong and published examples of its application for infrastructure projects are provided.

2 BUILDING INFORMATION MODELLING

2.1 *Definition*

BIM was originally defined as the "creation of and use of coordinated, computable information about a building project in design and construction" (Van Nederveen et al, 1992; Autodesk, 2007). With increased BIM application within the construction industry the definition(s) have since expanded generally being referred to as a "digital representation of the physical and functional characteristics of a facility" (Down-to-Earth technologies, 2014). BIM is a shared knowledge resource for information about a facility forming a reliable basis for decisions during a project life-cycle from earliest conception to completion, either demolition, as built or maintenance stages (Wikipedia, 2014). It can be used to represent the characteristics of facilities through the planning, design, construction, operation and maintenance stages with applications ranging from different physical infrastructures, such as water, wastewater, electricity, gas, refuse and communication utilities to roads, bridges and ports, from houses, apartments, schools and shops to offices, factories, warehouses and prisons. Information can be exchanged in files, termed Building Information Models (BIMs), to support decision-making processes (HKBIM, 2014 and Smith, 2007).

2.2 *History*

BIM was initially referenced in 1992. It became more popular during the past 10 years later as software producers adopted this term as a normal and popular terminology reference, initiated by Autodesk as a standard term for modelling multiple views on buildings (Down-to-Earth technologies, 2014). The term was then used for digital representation of the building processes by other software producers, such as Graphisoft (Virtual Buildings), Bentley Systems (Integrated Project Models) and Autodesk (Vectorworks)

2.3 *Utility Application*

Utilities are often located in congested out of sight spaces, in urban environments with limited access from the surface. In particular older utilities were typically placed with inaccurate or non-existent alignment survey data. Accurate location needs to be provided for inclusion in the BIMs prior to construction. Following the utility location, using either indirect techniques, such as Ground Penetrating Radar or utility cameras or, if accessible, by a direct survey, position can be captured using mobile phones or tablets (see Figure 1). Using standardised data storage, for example using OXF and landXML files, can allow rapid data transfer to the BIMs using compatibility tools such as Infrakit. Following upload, potential clashes with the construction works can be identified and appropriate decisions made for the type and method of works to be carried out. Examples of BIM presentations for a variety of congested utilities and pipe-works, mainly for Mechanical and Electrical Plant (MEP), is presented in Figure 2.

Figure 1 Mobile phone survey data

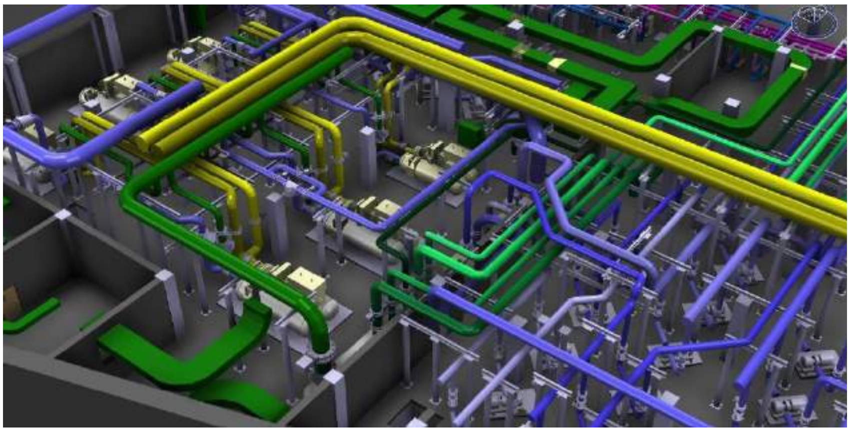

Figure 2 BIM showing congested pipework and utilities

2.4 *Software*

Some of the design software used for the BIM process during different stages of the works include databases, such as Pitney Bowes MapInfo; Geographical Information systems (GIS), such as Esri ArcGIS; Computer Aided Design (CAD) with 3D capabilities, such as Bentley MS; Innovyze Infoworks CS; Autodesk AutoCAD; Autodesk Civil 3D; Autodesk Revit; Autodesk Robot; Autodesk Navisworks; Autodesk 3DS Max; and miscellaneous software applications such as Unity 3D; Lunas

Finite Element Analysis; MasterSeries Structural Design Software; Nemetchek Vectorworks and Acecad StuCad (Ward et al, 2014). At different stages of the project choices need to be made to interoperate and exchange the design and / or construction data. Given the variety of software, the different uses, the interfaces and the demand for a rapid processing a major challenge is establishing a Computer Aided Design (CAD) team with the necessary experience to ensure the BIM workflow is carried out effectively.

An important part of the BIM process is capturing site data and rapidly transferring this to the BIMs. GPS equipment used to capturing site data may include Trimble 61000-00 GeoXH and Pro X provide. Avoiding disruption to existing utilities in the USA for example is now a federal and state practice requirement needing the submission of Subsurface Utility Engineering (SUE) methods to accurately identify and map underground utilities. This can be carried out using surface geophysics, surveying, data management and non-destructive excavation techniques, with an estimated cost saving quoted to be $1 on SUE at least $4.62 saved for the overall project (Purdue University 1999). In order to achieve high levels of accuracy, GPS data communication techniques such as Real Time Kinematics (RTK) is used; in the absence of ambient radio signals, RTK improves the GPS accuracy to a few centimeters (Ehsani et al. 2004), see Figure 3.

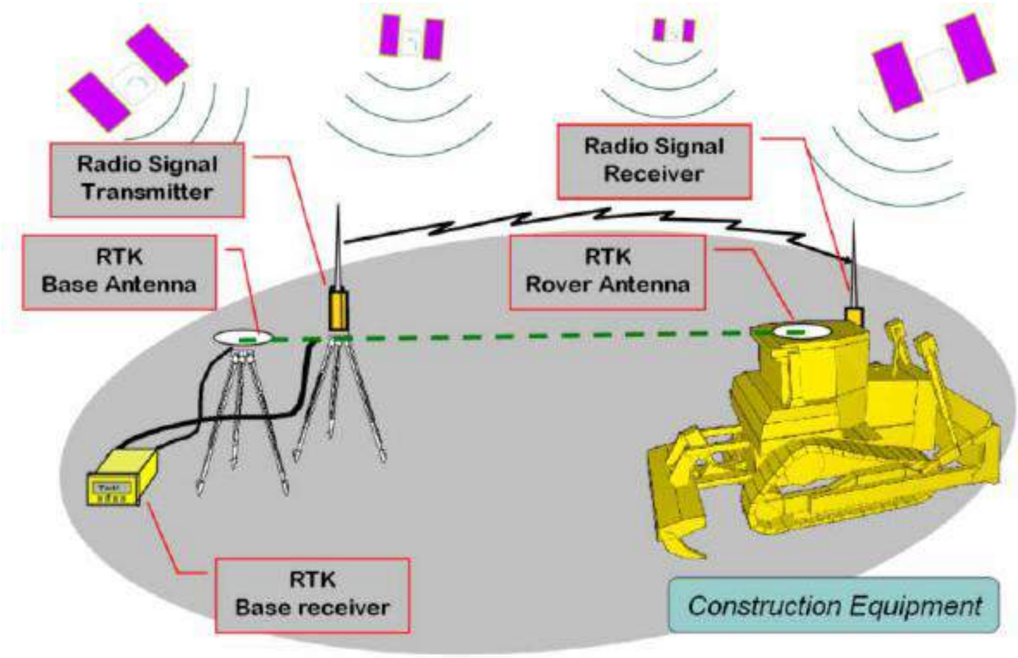

Figure 3 RTK communication to improve GPS data accuracy (Behzadam et al, 2009)

An RTK base antenna and radio signal transmitter is mounted on a remote spot with known global position. An RTK rover antenna together with a radio signal receiver, placed on the excavation equipment, can then continuously communicate the data to the base station. The exact position of the RTK rover antenna can then be ascertained for input to a visualization system. This can include software such as AutoCAD 360, Turbo View Pro (IMSI design), BIMx (Graphisoft), Genius Scan (Grizzly Labs), Photo Measures (Big Blue Pixel) and MagicPlan (Sensopia), Ward et al, 2014.

2.5 *Work Flow and Standards*

The BIM standards, mainly applicable to Hong Kong, are generally categorized into the Project Execution Plan (PEP), modelling method, level of detail and component style, with aims summarized in Table 1 (HKBIM, 2014).

The initial BIM set up, as summarized in Table 1, is typically initiated by the client prior to receiving appropriate data from the design consultant and the contractor at later stages. See Figure 4 for the process (HKBIM, 2014).

Table 1 BIM categorization and aims

	Standards Category	Aim
i	PEP	Project management and execution definition for the strategy, collaboration, production and data segregation;
ii	Modelling Methodology	Model development, including the efficient use and consistency of the BIM data applicable to each discipline;
iii	Level of Detail	Specify the intended graphical scale, level of detail (architectural and structural) models for each project stage, including the design (concept, preliminary and detailed), approval, construction and as-built;
iv	Presentation style and data organisation	Standardisation of appearance, style, size, properties, categories, units and measurement, data structure and nomenclature.

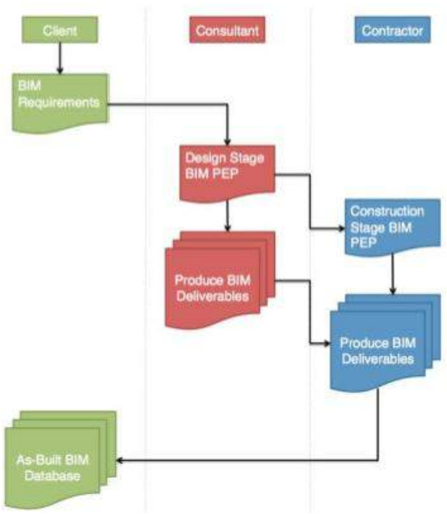

Figure 4 Overall work flow

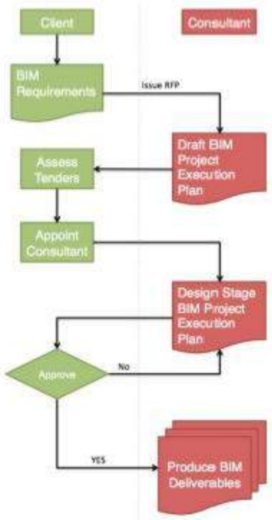

Figure 5 Client / consultant work flow

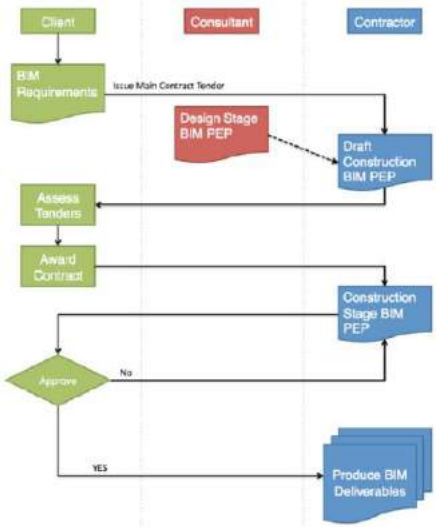

Figure 6 Client / contractor work flow

During the design and construction stages suitable approaches, capabilities, capacities for the type of work and competence for the BIM updates needs to be assessed by the client or its representative during tender assessment of both the design consultant and contractor (see Figures 5 and 6). At the construction stage the designer BIM PEP should be passed to the tendering contractors for review and consideration for amendment and update as required to ensure successful production of the project deliverables (see Figure 6). Agreed software and file exchange models, such as DWG, DGN, DWF, PDF and IFC files need agreement at each stage (HKBIM, 2014).

An example of the application of BIM standard is set out within the Hong Kong Mass Transit Railway Corporation Limited (MTRCL) Specifications. A typical standardised drawing folloing the MTRCL CADD Manual is required with definitions provided for the Modeling and Level of Detail covering the full extent of the civil, and architectural works. The building services and ystemwide Electrical and Mechanical (E&M) input is also needed with the aim of providing effective transfer between the client and different parties involved in the project. The requirements are typically provided for the naming conventions, families, file exchange protocol and symbols and annotations, typically initiated by a nominated BIM originator, who then typically sets up the following requirements:

a) BIM Authoring Application Version;
b) Model Geometry (co-ordinates, setting out, levels & units);
c) Model Division (model demarcation, central/local file setting & workset setting);
d) Model Project Browser Set-up;
e) Model File and Revit Family Naming Convention;
f) Model Maintenance and Quality Control Procedures;

g) Revit Family Library Descriptions;
h) Clash Analysis System;
i) BIM Deliverables Descriptions (native files, review files, clash analysis reports structure & file/folder structure).

3 INTERNATIONAL CASE HISTORY EXAMPLES

3.1 *Port Mann Highway, Canada*

BIM was used to provide visual presentations and as a management device through distinct phases of the design and construction process for the Port Mann Highway Interchange project (NCEI, 2013), located near the Port Mann Bridge, Highway 1, Vancouver Canada (See Figure 7).

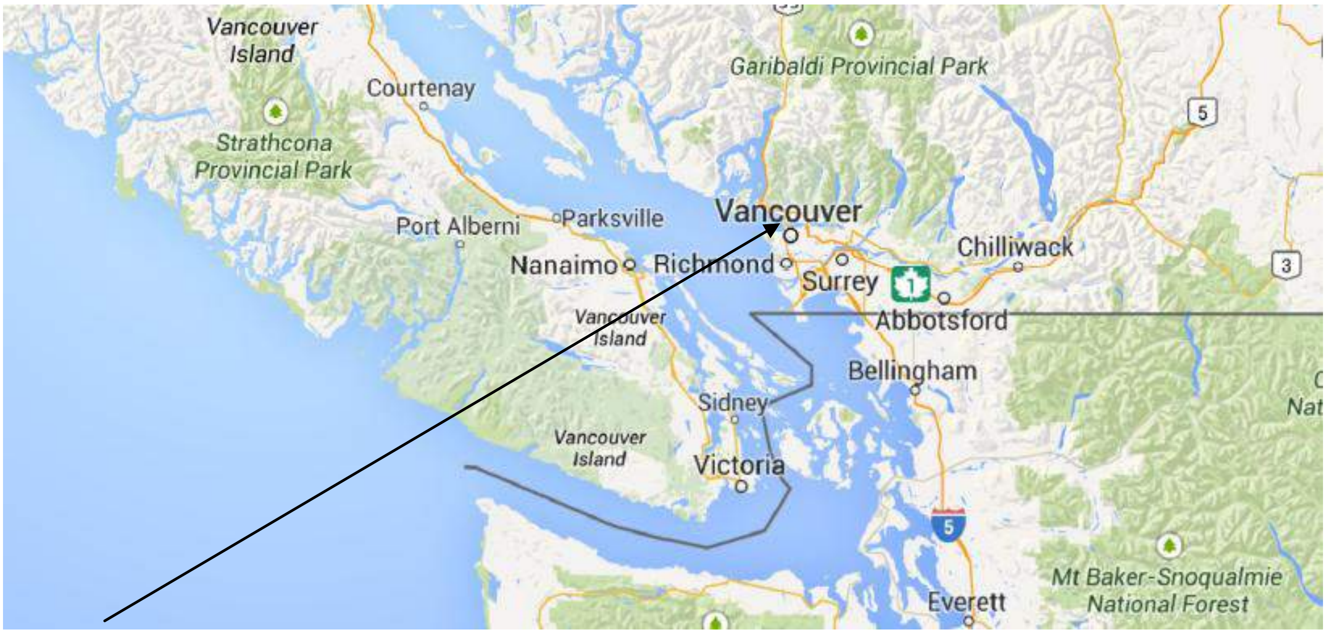

Figure 7 Port Highway Interchange, Vancouver, Canada (Google Earth, 2011)

The initial stage involved the use of Autocad software to create a virtual 3D corridor, providing the highway grading, layout and geometry for the 37 kilometre alignment; See Figures 8 and 9.

Figure 8 BIM model, Cape Horn Interchange transport corridor, Vancouver, Canada

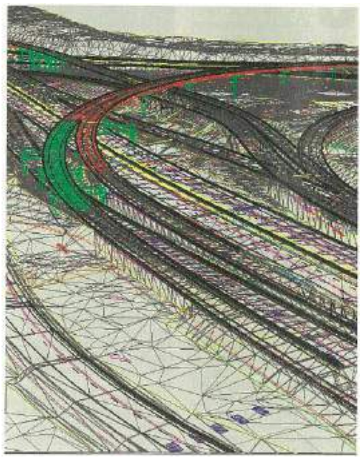

Figure 9 Cape Horn Interchange model

The initial 3D visualization provided an overview of the utility interfaces, which allowed minor adjustments to the alignment to limit interference as far as practical. The main interfaces included drainage culvert, power line and sewer utilities, which were located and incorporated into the BIM model as more detail became available (NCEI, 2013) and utility locations relocated as needed. Utility realignment was considered for the sewers, culvert junction boxes, street lamps and major drainage elements. Once the alignments were set the construction sequences could then be determined. The main challenge for the use of BIM on this project was interfacing and model exchange between different parties involved on the project. Software compatibility, providing the appropriate level of detail, was required for the type of work and methods adopted.

The alignment was placed over ground terrain models determined by topographic detail provided from on-site laser scanners to provide detail in the congested locations, particularly within, or in close proximity, to the alignment corridor.

3.2 *Corrib Pipeline, Galway, Ireland*

The Corrib pipeline transfers gas from an offshore reservoir to the landfall at the landfall installation plant at the estuary of the Corrib River, County Galway, Ireland (See Figures 10 and 11).

Figure10 Corrib location, Ireland (Google Earth, 2011)

Figure 11 Pipeline landfall

From the landfall compound the gas is transferred to the Bellanby Bridge gas terminal for onward distribution, supplying up to 60% of Ireland’s gas requirements (Ward et al, 2014); see Figures 12 and 13. The pipeline was 0.5m inside diameter with a 27mm thick pipeline.

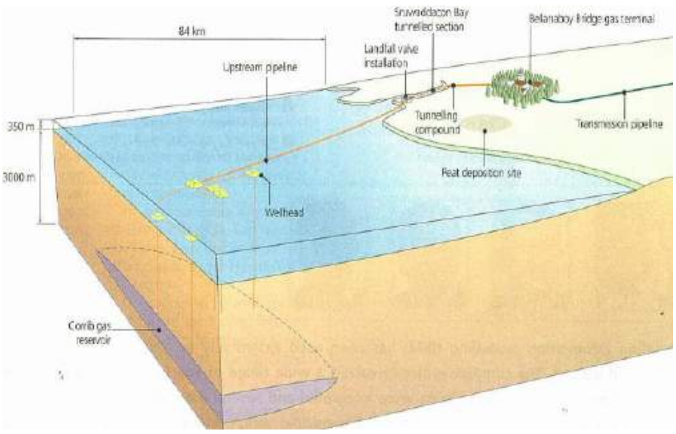

Figure 12 Sketch presentation, Corrib pipeline location

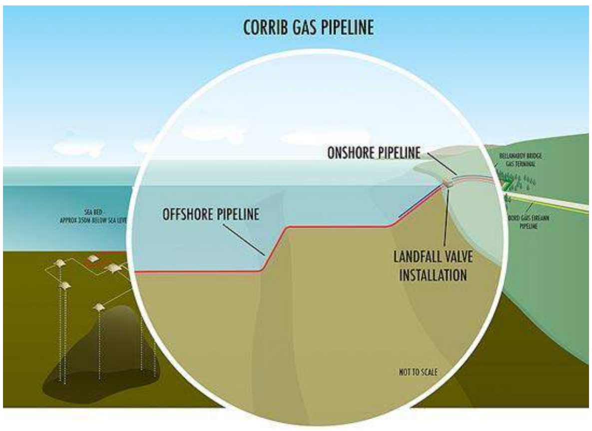

Figure 13 Offshore pipeline location

The initial route selection involved environmental impact study and preliminary design carried out by placing available digital data into a database (ArcGIS) used to store, organization and manage the information (Ward et al, 2014). The various pipeline alignments were then considered using visual presentations to show interaction with the landfall facilities (see Figures 14 and 15) and highlight the main environmental data, stored in ArcGIS, to show the environmental impacts (See Figure 16).

Figure 14 3D BIM model of the pipeline compound

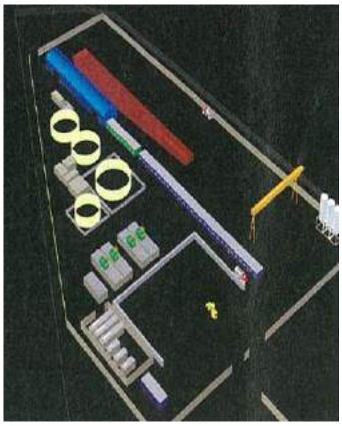

Figure 15 Preliminary model

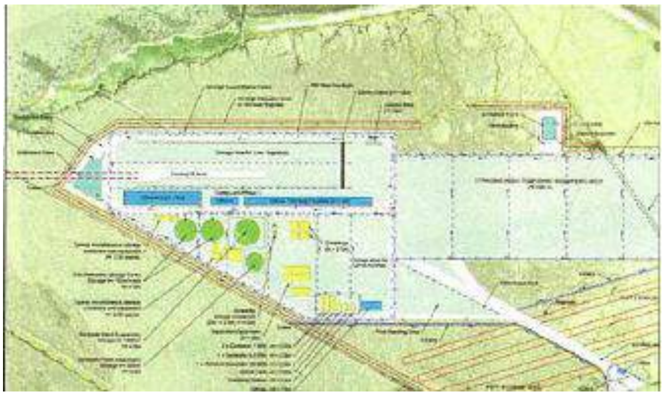

Figure 16 Tunnel compound, Environmental Impact

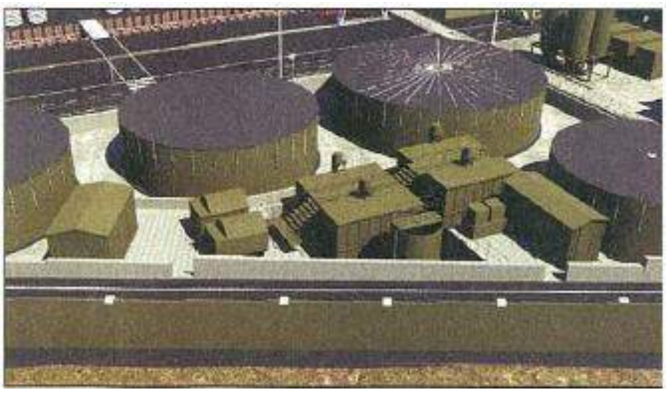

Figure 17 Surface Water Treatment plant model

During the subsequent detailed design increased interaction and access to the BIM models was required between different parties involved on the project. These included geotechnical, civil, structural, electrical, planning, visual assessment, surface-water management, water treatment and environmental specialists. One of the considerations was the plant, equipment and operations design within the compound. This was presented as a 3D visualization to all parties providing rapid planning and technical collaboration between all relevant parties (see Figure 17). As the pipework data was provided during the later program stages, a rapid clash detection assessment was needed to provide design feedback, see Figure 18. The process was continued through to construction for the drainage layout design in congested locations allowing rapid layout optimization, see Figure 19 (Ward et al, 2014).

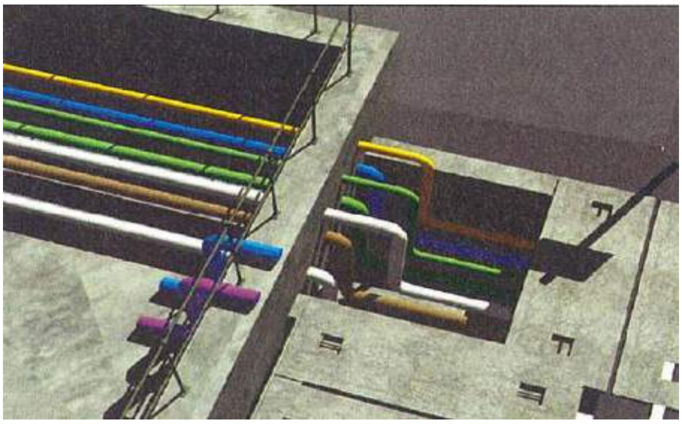

Figure 18 Pipework and structure interface used for clash detection

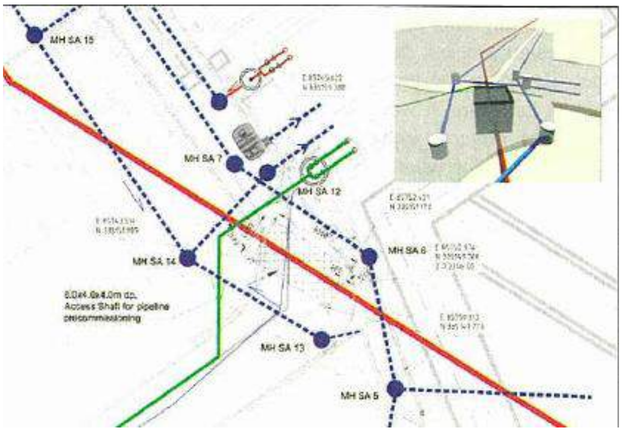

Figure 19 Drainage diversion modelling

3.3 *Tottenham Court Road Station, London, UK*

Tottenham Court Station is located in central London at the intersection of the Central and Northern Lines of the London underground transport system. The station upgrading works commenced in 2007 and is due for completion 2016, at a cost of approximately 1 billion pound sterling (TfL, 2013). Refer to Figure 20 for a visual presentation of the site (Li et al, 2014). BIM was used in the design and construction process for this project to:

a) Design the constraints imposed by the limited space available for the site works;
b) Provide visualization to assist interfacing between the different design disciplines (see Figure 21);
c) Integrate the design model with the site topography to ensure compatibility with actual site conditions;
d) Visualise the model at different construction stages.

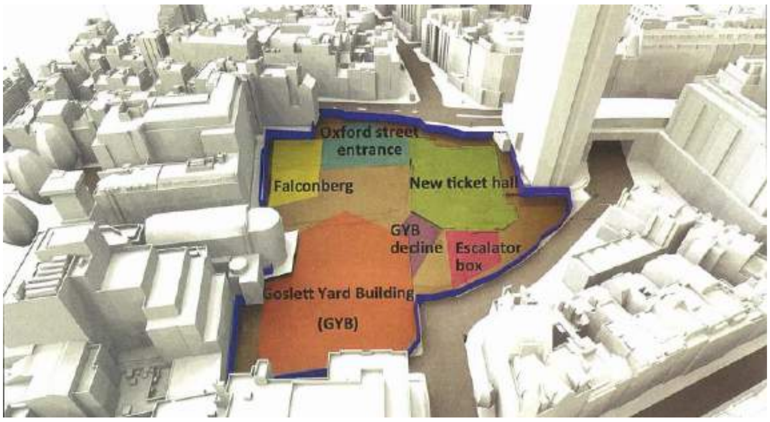

Figure 20 General sketch presentation, Tottenham Court Road.

Figure 21 3D construction model.

Upon completion of stage of the works program a visual image was produced to record the site progress at that stage to provide a 4D view of the works (see Figures 22 and 23, Li et al, 2014).

Figure 22 Site photograph of the works (11/12).

Figure 23 Visual image of the works (11/12).

The up to date records provided at each stage of the works, Figures 22 and 23, were used to update data for the future BIMs, adopting a 3 to 4 month look ahead with associated construction program updates accordingly.

In addition to the use for planning and programming BIM was also provided overlays for other specialist input requirements and updates. Part of this process was enhanced by regular survey data updates from Ordnance Surveys and laser imaging detection and ranging (Lidar) data. The 3D information included the topographic map, utility services, the site boundary, temporary works, station architectural details and the ground model as presented in Figure 24.

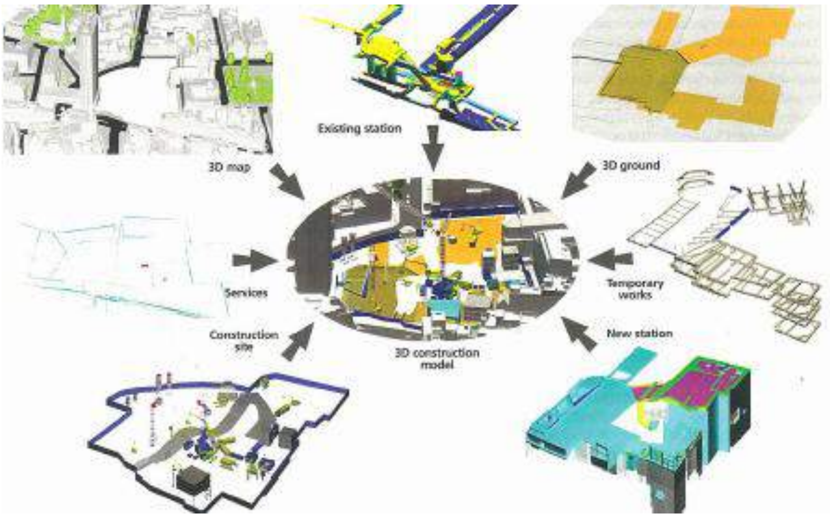

Figure 24 3D presentation of Tottenham Court Station with the building, existing station, ground model, temporary works and site facility overlays.

4 CONCLUSION

The use of BIM as a project management tool in planning and visualizing construction projects is becoming a standard requirement and has successfully used through design and construction stages of a range of major infrastructure projects. Attention is needed in consistent use of software and transfer data to ensure its efficient use through each project stage. Its use can lead to significant cost and time savings and when used as process for visualising utilities surveyed using non-destructive excavation techniques was estimated to save US $4.62 for each US $1 invested for the overall project (Purdue University, 1999). The use of 4D simulation, using input to the planning through interaction with Primavera 6 and 5D simulation, for use in cost predictions, is currently being advanced.

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Design of new railway projects – challenges and innovative solutions

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Keywords: expanding railway network in Hong Kong; developed urban area; operating area; lift-only entrance.

ABSTRACT: The railway network of Hong Kong is being expanded with the implementation of five railway extension projects. Upon completion of these five railway extension projects, the railway network will be expanded by 56km. The alignment of these new railway projects pass through developed urban area and/or existing operating area. Implementation of these projects are therefore very challenging. This paper outline some of the challenges encountered during planning and design stage of the West Island Line and Shatin to Central Link project and solutions adopted to overcome these challenges, e.g. creating space to construct the stations and tunnels in a developed area and use of lift-only entrance.

1 INTRODUCTION

The railway network of Hong Kong is being expanded with five railway extension projects. These include West Island Line, South Island Line (East), Kwun Tong Line Extension, Express Rail Link and Shatin to Central Link. The 3-km West Island Line is an extension of the Island Line from Sheung Wan to Kennedy Town. The South Island Line (East) will extend MTR services from Admiralty to the Southern District of Hong Kong Island, with a route length of 7km. The 2.6-km Kwun Tong Line Extension will extend the Kwun Tong Line to Whampoa. The 26-km Express Rail Link will provide high speed cross-boundary rail services connecting Hong Kong and the high speed rail network in the Mainland of China. The Shatin to Central Link includes an 11-km section between Tai Wai and Hung Hom and an 6-km section between Hung Hom and Admiralty. Upon completion of these five railway extension projects, the railway network will be expanded by 56km.

Implementation of these railway extension projects are very challenging in terms of technical difficulties, scale of construction, site constraints and/or interfaces with existing facilities. One of the common challenges is that the stations/tunnels are constructed in developed urban areas, where available space for constructing the new railway works is very limited and the construction is severely constrained by utilities, buildings and other infrastructures. For works on Hong Kong Island, the works are also constrained by the hilly terrain.

Another challenge is interfaces with the operating railway. The West Island Line and Kwun Tong Line Extension are natural extension of the Island Line and Kwun Tung Line respectively. The terminus station of the South Island Line (East) is located at Admiralty. The Shatin to Central Link connects the existing Ma On Shan Line and West Rail Line, and extends East Rail Line across Victoria Harbour to Hong Kong Island. The works have to be constructed without affecting railway operation.

This paper highlights some of the challenges encountered during design of the West Island Line and Shatin to Central Link and solutions adopted to overcome these challenges.

2 WEST ISLAND LINE

The Island Line opened in 1985 and it provides rail services to major catchment areas on the north shore of Hong Kong Island, from Chai Wan to Sheung Wan. With the implementation of the West Island Line project, rail services is expanded to serve major catchment areas west of Sheung Wan. As the alignment passes through developed urban area in the western district, a major challenge is to find suitable space to accommodate the stations, entrances and ventilation shafts without resumption of private buildings. The hilly terrain makes planning for the project even more challenging.

During planning and design stage, a review of the catchment areas was conducted and it was recommended to construct three new stations to serve population and employment in the western district. [Note : These three stations are now named Sai Ying Pun Station, HKU Station and Kennedy Town Station.] A review of available land was also conducted to identify suitable sites for construction of these stations. However, no vacant sites could be identified. Two possible options were then proposed. The first option was to construct the stations in a rock cavern while the second option was to make use of government sites. Following detailed studies, it was recommended to construct Sai Ying Pun station and HKU station in rock caverns, and to construct Kennedy Town Station at a government site.

The selected alignment is to extend the Island Line tracks at Sheung Wan in a south-westerly direction to arrive at Sai Ying Pun Station, and then to run in a westerly direction to connect with HKU Station and Kennedy Town Station. A turnback tunnel is provided west of Kennedy Town Station to enable westbound trains from Chai Wan to turn back and then travel in an eastbound direction towards Chai Wan.

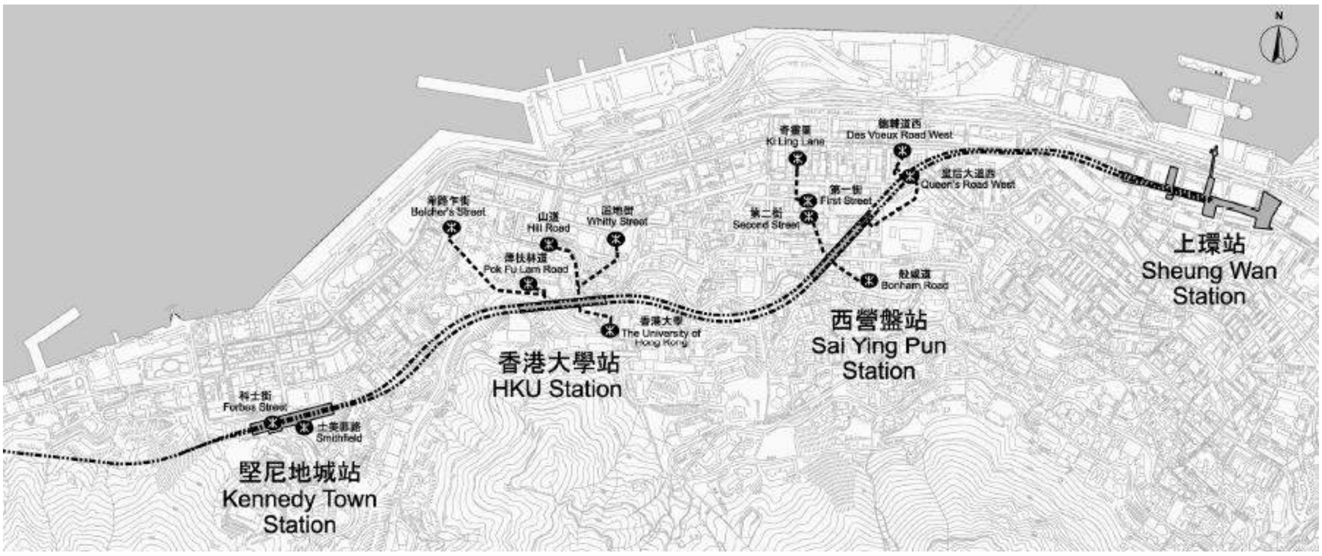

Figure 1 West Island Line

Sai Ying Pun Station is located in a south-westerly orientation, with the station bound by Eastern Street on the east, Western Street on the west, Bonham Road on the north and Second Street on the south. At this location, the terrain of the ground directly above the station box is quite steep and the station is at a distance horizontally from the tramway at Des Voeux Road West. HKU Station is located under Pok Fu Lam Road, in the vicinity of Hill Road. Similarly to Sai Ying Pun Station, HKU Station is at a distance vertically below street level and at a distance horizontally from the catchment area at low levels. Kennedy Town Station is located at a site originally occupied by a public playground and swimming pool.

At these locations, both Sai Ying Pun Station and HKU Station are deep stations and the ground is general in good rock. Drill-and-blast method is adopted to form a two-level station in a rock cavern. A remote access shaft is constructed for mucking out of material generated from drill-and-blast construction. The excavated material is then delivered to a barging point in Kennedy Town. The selected method of construction for the station box of Sai Ying Pun Station and HKU station therefore does not affect ground level facilities and hence minimise impacts to the community.

Having resolved construction of station box for Sai Ying Pun Station and HKU Station, another challenge to overcome is to provide adequate number of entrances to serve the community. While the station box can be located underground, entrances must be located at ground level for passengers to enter and leave the station during normal operation and for evacuation of passengers in the event of emergency. Location of entrances is of great interest to local stakeholders. Extensive consultation has been conducted during planning and design stage.

As a result of the extensive consultation and after detailed studies, a number of entrances are provided at Des Voeux Road West, Queen’s Road West, First Street, Second Street, Ki Ling Lane Children’s Playground and Bonham Road for Sai Ying Pun Station, and at Whitty Street, Hill Road, Belcher’s Street and Pok Fu Lam Road for HKU station. Most of these entrances are equipped with escalators and staircases which are conventional vertical circulation elements widely used for MTR stations. For the entrances at Bonham Road and Pok Fu Lam Road, the vertical distance between these entrances and station concourse is quite large and the use of escalators and staircases is considered not practical and not effective.

An alternative proposal is therefore developed for the deep entrances at Bonham Road and Pok Fu Lam Road. Instead of providing conventional escalators and staircases, the preferred option is to use high capacity lifts to transport passengers between ground level and station concourse level. This is a new idea for MTR stations which was first proposed in 2004 during the feasibility study stage and further developed during the preliminary design stage in 2006. These studies conclude that the use of lifts for deep stations in lieu of escalators is more time effective when the rise is approximately 30m and the time benefits become greater with increasing rise. Further, the area of shaft required to accommodate the lifts is smaller than the area required to accommodate escalators.

Unlike lifts for high-rise buildings which stop at many levels, lifts for entrances at Bonham Road and Pok Fu Lam Road only stop at designated levels. Separate lobbies for boarding and for alighting are provided in order to avoid conflict of passenger movement. Walk-through lift design is adopted to allow passengers to leave the lifts first via doors on the side of the alighting area, and then to let passengers to board the lifts via doors on the side of the boarding area.

In the event of emergency where a fire occurs at the adit linking entrance with lifts only and the station concourse, passenger on the concourse side of fire can evacuate towards the concourse and then leave the station via entrances with escalators and staircases. However, passengers on the lift lobby side of the fire must use the lifts to leave the station. This is a new evacuation strategy for MTR stations in Hong Kong.

The innovative approach towards lift-assisted evacuation for deep stations is developed in collaboration with statutory authorities and emergency services personnel over a period of approximately six years. The lift-assisted evacuation proposal has been submitted to the Safety and Security Coordinating Committee (which is responsible for railway safety and security in Hong Kong) and the Fire Safety Committee convened by Buildings Department and it has been approved by the Electrical and Mechanical Services Department. In the second edition (September 2013) of the “Guidelines on Formulation of New Railway Infrastructures “document published by Fires Services Department, lift-assisted evacuation (associated with lift-only entrance) has been included as an example of special means of escape.

Passengers at the lift lobby side will use lifts for evacuation in the event of a fire incident in an adit between station concourse and the lift lobby. Passengers will be directed to a refuge lift lobby located one level above the concourse lift lobby and then leave the station using the high capacity lifts operated under evacuation mode. The refuge lift lobby is pressurised and is equipped with the necessary fire safety provisions to provide a safe environment for evacuated passengers to wait for the lifts.

The entrance at Bonham Road is located at the site originally occupied by David Trench Rehabilitation Centre. Prior to demolition of this building, the existing facilities have to be reprovisioned at a site in the vicinity of the building. After detailed land search, the Ex-Crime Wing of Hong Island Regional headquarters at High street is considered to be suitable for reprovisioning of David Trench Rehabilitation Centre. The original building is refurbished and a new annex building is constructed in order to provide adequate area to accommodate the necessary facilities. New lifts are provided to enhance accessibility from High Street to the reprovisioned David Trench Rehabilitation Centre.

The entrances for HKU Station are located at Pok Fu Lam Road in the vicinity of the campus of Hong Kong University. In addition to serving local residents, these entrances are designed with convenient connection with the campus.

Unlike Sai Ying Pun Station and HKU Station which are deep stations, Kennedy Town Station is a relatively shallow station. It is located at the public playground and swimming pool at Forbes Street. Prior to occupying the site for construction of Kennedy Town Station, a new swimming pool is constructed at a vacant site at Shing Sai Road. The remaining area of the site is used for construction of an access shaft for mucking out of excavated material from Sai Ying Pun Station and HKU Station. After completion of station construction, this area is used for expansion of the swimming pool complex.

3 SHATIN TO CENTRAL LINK

The Shatin to Central Link project includes an 11-km section between Tai Wai and Hung Hom and a 6km section between Hung Hom and Admiralty. The Tai Wai to Hung Hom section is an extension of the existing Ma On Shan Line from Tai Wai to connect with West Rail Line at Hung Hom, via the old Kai Tak Airport and To Kwa Wan area. Upon completion of this section, an East West Corridor between Wu Kai Sha and Tuen Mun will be formed, allowing passengers to travel between Wu Kai Sha and Tuen Mun directly. The Hung Hom to Admiralty section is an extension of the existing East Rail Line from Hung Hom across Victoria Harbour to north Wan Chai and Admiralty. Upon completion of this section, an North South Corridor between Lo Wu/Lok Ma Chau and Admiralty will be formed, allowing passengers to travel directly from the border of the Mainland of China to the central business district on Hong Kong Island. Passengers travelling on East West Corridor and North South Corridor can interchange at Hung Hom, where new platforms will be constructed.

The alignment of the East West Corridor and North South Corridor passes through the existing operating area at Hung Hom. One of the major challenges is to construct the tunnels and station without affecting operation of East Rail Line and intercity services. For the East West Corridor, the alignment has to connect with the existing at-grade West Rail track at Hung Hom. Therefore, its track level at Hung Hom is approximately at ground level. For the North South Corridor, track level at Hung Hom has to be below ground level in order to allow the alignment to cross Victoria Harbour. With these constraints, it is a major challenge to develop an alignment and method of construction that does not affect operation, and provide new underground platforms for North South Corridor and at-grade platforms for East West Corridor at Hung Hom.

Currently, the East Rail Line, West Rail Line and intercity services terminates at Hung Hom. There are four at-grade platforms for East Rail Line and West Rail Line and two at-grade platforms for intercity services, just to the east of the toll plaza of the Cross Harbour Tunnel. As operation of East Rail Line, West Rail Line and intercity services has to be maintained during construction of the Shatin to Central Link project, new platforms have to be provided to the east of the existing platforms, with platforms for North South Corridor underneath platforms for East West Corridor. Interchange between these platforms will be via escalators and staircases. With this configuration for new platforms, the alignment for North South Corridor and East West Corridor at Hung Hom has to run on the east of the existing East Rail tracks.

The proposed alignment for North South Corridor north of Hung Hom station is to bifurficate from the existing at-grade East Rail tracks west of Oi Man Estate, with the horizontal alignment running parallel and to the east of East Rail Line and the vertical alignment descending towards the new underground platforms at Hung Hom. For the East West Corridor, the vertical alignment will climb up vertically after leaving Ho Man Tin station and merge with the horizontal alignment on plan of the North South Corridor south of Chatham Road, with the tracks for East West Corridor above the tracks for North South Corridor. There are four major challengers along this alignment, including the existing slope west of Oi Man Estate, Chatham Road overbridges above East Rail Line, major operating facilities along the alignment, and foundations/columns of the podium structure at Hung Hom.

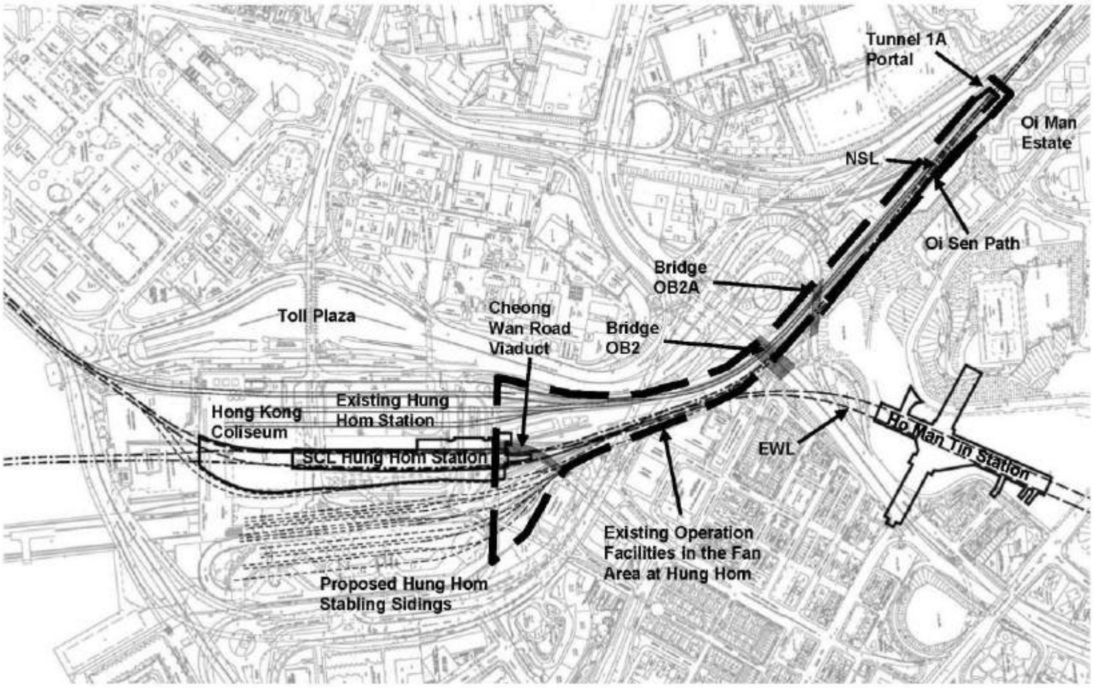

Figure 2 SCL Alignment at Hung Hom area

After bifurficating from the existing at grade East Rail tracks, the horizontal alignment of North South Corridor has to run on the east of East Rail Line at a safe distance in order to facilitate construction of new tunnels. However, the space between the existing East Rail Line and the slopes is not adequate for construction of the new tunnels. In order to create space for the new tunnels, the existing slopes have to be cut back temporarily during construction. The extent of the temporary slope modification has to be optimised such that adequate space is provided for construction of the tunnels and provision of the necessary railway protection measures, and at the same time minimise any impact to the existing slopes.

The space created by cutting back existing slopes is very tight and the construction works is close to the operating tracks. An ADMS monitoring system is provided to ensure any movement which may be caused during construction is within the acceptable limits. A continuous barrier is provided to separate construction area from the operating area. A trip wire system is provided on the barrier to ensure a clear demarcation of construction area and operating area. In the event of excessive movement that activates the trip wire system, an alarm will be sent to the signalling system to stop train operation. If these happens, a detailed investigation will be conducted to identify causes of the movement and measures to be implemented to allow train operation to resume as soon as practical.

The temporary slope modification works affect Oi Sen Path (located at the first berm of the slopes) and CLP cables at Oi Sen Path. It is a requirement that a pedestrian route has to be maintained during construction. Therefore, temporary diversion of Oi Sen Path has to be implemented. The CLP cables have to be locally slewed and protected. After completion of tunnel construction, the slope will be reinstated. Where space is tight and the slope cannot be reinstated to the original profile, the tunnel walls are designed to support loadings from the slope and to keep the slopes stable.

Further south, the alignment clashes with the abutments of the overbridges at Chatham Road. These overbridges allow road traffic to run above East Rail Line. They also provide support to the overhead line and other utilities. The solution to resolve these conflicts is to divert the traffic, construct temporary support to the bridge deck, build the new tunnels and then build new support for the bridge deck.

New temporary steel bridges are constructed above East Rail Line and adjacent to the existing overbridges. New bridge beams are provided to span across East Rail Line and they have to be installed at night time after train services stop. Traffic on Chatham Road is diverted to the new temporary bridges in a number of stages. Temporary support are constructed adjacent to East Rail tracks to support the deck of the existing overbridges.

Construction of the new tunnels for North South Corridor starts after traffic on Chatham Road overbridges is diverted to the new temporary bridges and temporary support to the overbridges is in place. The existing bridge abutment is demolished in order to creat adequate space for construction of the new tunnels. A temporary cofferdam is constructed to allow excavation to proceed and construction of the new railway tunnels which are supported on new foundations. A new bridge abutment resting on the new tunnel box is then constructed to support the decks of the existing overbridges. Following this, traffic will be diverted back to Chatham Road overbridges in stages and the temporary bridges will be demolished.

South of Chatham Road overbridges, the alignment of both North South Corridor and East West Corridor pass through the operating area north of Hung Hom station where a number of essential E&M and maintenance facilities are located. These facilities are essential for operation and maintenance of the railway and they have to be reprovisioned prior to commencement of construction of the tunnels of the Shatin to Central Link. As there is no space available on site, these facilities have to be relocated off-site, in close proximity to the existing East Rail Line.

Following detailed search of suitable sites, it is identified that there is no single site of sufficient size to accommodate all the affected facilities. However, a number of smaller sites are identified to reprovision the various facilities. The locomotive maintenance facility is relocated to Lo Wu and the ground level space of the Hong Kong Polytechnic University Phase 8 development at Chatham Road. Some maintenance facilities are relocated to the disused freight yard at Mongkok (adjacent to Mongkok East Station). The E&M building is relocated to the space just west of the existing Hung Hom Station.

Within the operating area north of Hung Hom station, the alignment of North South Corridor and East West Corridor clashes with a portion of Cheong Wan Road Viaduct. This portion of the viaduct has to be reprovisioned such that road traffic and pedestrians can be diverted to the new section in order to allow the portion of the viaduct in conflict with the new railway alignment to be demolished.

The new platforms at Hung Hom are located on the east side of the existing platforms, underneath the existing podium structure. Some of the existing columns/foundations are in direct conflict with the new platforms while some are located very close to the new platforms. Underpinning of these colums/foundations are therefore required. The underpinning work has to be carried out without affecting station operation.

For the columns/foundations in direct conflict with the new platforms, the ground levels columns supporting the podium structure are supported by piles founded on rock. The foundations below ground level are located very close to the edge of the underground platforms of the North South Corridor and have to be removed. The columns at ground level are located further away from the edge of platforms of the East West Corridor at ground level and they can be retained. The proposed scheme is to construct diaphragm walls as permanent walls for the new underground platforms and as part of the cofferdam during construction, and to construct base slabs and ground level slabs to form the new platform structure. Loadings of the existing ground level columns are first transferred to the new ground level slabs via some temporary structure. The existing piles then become redundant and they are removed during construction of the underground platform structures. After completion of the new platform structure, the existing columns are then connected monolithically with the new ground level slab and the loadings are transferred to the new ground level slab.

Some of the existing columns are located close to the diaphragm walls of the new platforms. These columns are supported on friction piles. Prior to construction of the diaphragm walls, new piles founded on rock are constructed to support the columns such that any movement occurred during construction of the diaphragm walls and bulk excavation will not affect the existing columns and the podium structure.

In order to ensure normal operation of the station is not affected, load transfer of the affected columns is carried out during non-traffic hour. A comprehensive monitoring plan (including ADMS systems) is also implemented. Also, a detailed condition survey of the existing structure is conducted prior to commencement of construction.

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Turning the myth of high-rise zero carbon buildings into reality: a socio-technical framework of design strategies

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Keywords: zero carbon building; Hong Kong; high-rise; design strategies.

ABSTRACT: The zero carbon building (ZCB) approach is innovative in reducing buildings' energy consumption and carbon emissions, with hundreds of 'zero carbon' or 'zero energy' projects reported worldwide. However, very few of them are of 10 storeys or above. There is perception that high-rise ZCB is a myth. The aim of this paper is to develop a novel socio-technical framework of design strategies for realising high-rise ZCBs. Through an onerous literature review, only two papers were found to have examined zero energy design strategies for high-rises, rendering a significant gap in knowledge. Also, the lack of a common definition of high-rise ZCBs and the energy/carbon scope hampers the uptake of ZCB design strategies. Another deficiency in literature is the ignorance of the interdependence among possible design strategies. The paper thus develops a socio-technical framework of design strategies, which integrates both technical and social aspects. The technical aspects are: 1) improving energy efficiency; 2) adopting on and off-site renewable energy technologies; and 3) installing carbon offsetting systems. The social perspective should be addressed from the chain of the innovation, diffusion, adoption and implementation of these technologies as well as through engaging the key stakeholders.

1 INTRODUCTION

The zero carbon building (ZCB) approach is regarded innovative in transforming the carbon-intensive construction industry. In Hong Kong, the first ZCB was completed in 2012 but remains as a prototype 'to showcase state-of-the-art eco-building design and technologies to the construction industry internationally and locally and to raise community awareness of sustainable living in Hong Kong' (Construction Industry Council – CIC 2012: 2), far from mainstream practice. Also, very few ZCBs exist in the high-rise high-density subtropical urban settings. This knowledge gap hampers the uptake of the ZCB approach in urban environments with the subtropical climatic conditions.

This paper aims to develop a socio-technical framework of design strategies for realizing high-rise ZCBs. The specific objectives are to review the state-of-the-art of low/zero carbon design strategies in high-rise buildings and identify possible gaps in knowledge for future studies. This literature review will pave the way for future studies on ZCBs in high-rise settings.

2 THE CONCEPT OF HIGH-RISE BUILDINGS

Table 1 presents four definitions of high-rise/tall buildings, which are measured by heights of meters or the number of storeys. This study adopts the local definition provided by the HK Fire Services Department, which refers high-rise building to a building of 30m above (see last row).

Table 1 Definition of high-rise/tall buildings

Reference	Descriptions
Code for Design of Civil Buildings (GB50352-2005) P. R. China	A high-rise building is a building of ten or more stories for residential building; or over 24 meters in height
National Fire Protection Association 101®, Life Safety Code, 2012 edition	A high-rise building is a building more than 23 meters in height, measured from the lowest level of fire department vehicle access to the floor of the highest occupiable story.
Council on Tall Buildings and Urban Habitat	A tall building is a building of perhaps 14 or more stories (i.e. or over 50 meters in height); A super-tall building is a building over 300 meters in height, and A mega-tall is a building over 600 meters height.
Code of practice for minimum fire service installations and equipment and inspection, testing and maintenance of installations and equipment (Fire Services Department HK, 2012)	Any building of which the floor of the uppermost storey exceeds 30 m above the point of staircase discharge at ground floor level.

3 A CONCEPTUAL FRAMEWORK OF HIGH-RISE ZCB DESIGN

While it is widely recognized that design strategies for achieving zero carbon should be constructed in a tier-based approach (e.g. from passive design to energy efficiency building services, to renewable energy) (see examples in Table 2), “very few guidelines exist, on the architectural design of multistory buildings, to improve their energy efficiency and solar potential” (Hachem, Athienitis and Fazio, 2014). Tentatively, this paper proposes a three-tier technical design framework for delivering high-rise ZCBs covering: 1) improving energy efficiency; 2) adopting appropriate on and off-site renewable energy technologies; and 3) installing carbon offsetting systems. This technical framework guides the process of literature review.

4 LITERATURE REVIEW STRATEGY

This study adopts the searching inputs from Pan and Ning (2014a) who conducted a comprehensive literature review of sustainable/green building. The key words are “carbon neutral*” OR “sustainab*” OR “carbon emission” OR “energy saving” OR “green” OR “zero carbon” OR “low carbon” OR “passive” OR “zero energy” OR “autonomous” and “building” OR “hous*” OR “home” OR “project”. In addition, another two key words, namely “high rise” or “tall”, were added. The search scope is confined to the title, key words and abstract. In addition, we restricted the literature to nine well-established journals. These are Energy and Buildings, Building and Environment, Applied Energy, Renewable Energy, Renewable and Sustainable Energy Reviews, Indoor and Built Environment, Resources Conservation and Recycling, Automation in Construction and Construction Management and Economics.

Gathered were 28 empirical articles after screening the abstracts and paper contents. A further search was then undertaken to seek the paper citing these 28 articles and the paper delivered by researchers from the region or countries of a prevalence of high-rise buildings, like Hong Kong. In the end, another 24 articles were added, yielding a total sample of 52 articles.

Table 2 Strategies for achieving zero carbon

Target/concept	Strategies	Reference
Net zero energy building	Passive design Service system Renewable energy power generation	Deng et al., 2014
Zero energy building	Energy-efficient measures Renewable energy and other technologies	Li et al., 2013
Carbon neutral	Reduce loads/demand Meet loads efficiently and effectively Use on-site generation/renewables to meet energy needs Use purchased Offsets	Boake, 2014
Very high energy performance building	Passive design strategies Energy efficiency technologies Energy from renewable sources	Rodriguez-Ubinas et al., 2014
Zero carbon homes	Good fabric energy efficiency Inclusion of on-site low carbon heat and power technologies Use of allowable Solutions	ZCH, 2013
Zero carbon homes	Insulation Airtightness Thermal bridging Heating systems efficiency Renewable energy Community level solutions	EST Code Level 5, 6

5 LITERATURE REVIEW RESULTS

The results show that only two articles addressed the achievement of realizing zero energy in high-rise buildings (Fong and Lee, 2014; Hachem et al., 2014). Fong and Lee (2014) examined the solid oxide fuel cell (SOFC) tri-generation system under two conditions, partial-SOFC strategy (with connection with the grid) and full-SOFC strategy (without connecting with the grid) in a 28-storey office building in Hong Kong. They found that the full- and the partial-SOFC tri-generation systems have a 51.4% and 23.9% carbon emission reduction respectively. Hachem et al. (2014) investigated an integration of current PV systems in façades and roof in buildings of 3 to 12 floors, with eight apartments per floor. They found that the total energy generation could make up 90% and 50% energy consumption of a 4-story and a 12-storey building respectively.

Among the remaining, the majority focused on the passive design measures to reduce energy consumption or carbon emissions (see Appendix 1), followed by studies examining the effects of energy efficient services systems (see Appendix 2). Also, there are a small number of studies examining the adoption of renewable energy in high-rise buildings (see Appendix 3).

Appendix 1 shows a wide range of passive features examined by prior studies. Appendix 2 presents the examples of investigating the energy efficient building service systems. It is a common case that the innovative building service system combining multiple measures together, such as HVAC system combining chilled-ceiling with desiccant cooling (Niu et al. 2002), combination of cooled ceiling, microencapsulated phase change material slurry storage and evaporative (Wang et al. 2008).

Appendix 3 shows that PV systems are the most popular technology to generate renewable energy. However, it is recognized that the use of PV faces great challenges in the high-density urban environment. None of the studies claimed to achieve net zero energy in high-rise buildings of ten storeys or above. Yet, Hachem et al. (2014) pointed out that improving energy production efficiency of PV systems especially those integrated into the facade has a great potential to reach the zero target.

Notwithstanding these empirical evidences, the experiment or real-case examination of the building of 40 storeys is scarce.

6 DISCUSSION

6.1 *Missing Connection between ZCB Definition and Design Strategies*

The lack of zero carbon related research in the high-rise building setting may indicate a significant gap in knowledge. The review results show that while the design strategies examined by prior studies are proved to have a potential, albeit to a varying extent, to reduce energy consumption in high-rise buildings, none of them explicitly claimed an ambitious zero carbon target or even nearly zero target.

It is also argued that the absence of a common ZCB definition of high-rise buildings may challenge the design decision-making. The first concern facing the design strategy identification is the scope of energy consumption. For example, excluding the lighting appliances and plugs from the ZCB definition (see the UK example) would cause designers to attribute a low priority to the use of energy-efficient lightings.

Second, the debate about the inclusion of embodied energy from the life-cycle perspective might also confuse the decision-making at the design phase, especially the material selection and renewable energy supply. For example, upgrading a low energy building to a net zero target may result in an increase of the embodied energy (Berggren et al., 2005). The use of photovoltaics and fuel cells technologies require more energy to produce than they themselves will ever generate (Kibert, 2010). If taking the embodied energy into account, the design solutions would be accordingly adjusted from the sake of energy conservation.

6.2 *Scarcity of Studies Examining the Interdependence between the Design Strategies*

The review results show that the bulk of the literature focused on individual low carbon strategies or a combination of a few, without addressing the systems features in achieving low/zero carbon. Probing into the systems features are of great importance to the zero carbon delivery as some effects will only emerge in the system evolution process. The emerging effect indicates that two design measures working together will not be a simple aggregation, but a complex nonlinear interaction. For example, dwellings in tropical climates must be kept open for thermal comfort, whereas acoustic comfort is achieved by closing the windows (Garde et al., 2004). Therefore, besides a comprehensive understanding of the systems elements, the examination of the interdependence between the system elements is also of equal importance to address the socio-technical systems in the zero carbon delivery (Pan and Ning, 2014a).

While the tier-based design strategies are prevalent in guiding ZCB design (see Table 2), the understanding and empirical examination of the interdependence between these tiers are quite limited. Nevertheless, three approaches used to interpret such interdependence are identifiable. The first is a hierarchical approach, which indicates that the design strategies should start with energy-efficient fabrics to more efficient equipment, then to the micro-generation (see Xing, Hewitt, & Griffiths, 2011). This approach, albeit intuitively understandable, might run the risk of oversimplifying the interdependence among the tiers.

Second, researchers use the concept of ‘matching’ to describe a two-way interaction between energy needs, energy generation on site and the grid (Voss et al., 2010), specifically consisting of load matching and grid interaction (Salom et al., 2011). Load matching refers to “how the local energy generation compares with the building load”; grid interaction refers to “the energy exchange between the building and the grid” (Salom et al., 2011). By distinguishing these two matches, Voss et al. (2010) contended that overall energy performance could be improved by adjusting the demand to the generation; or by adjusting the generation to the needs.

Third, interdependence could also be described from the spatial perspective (Pan and Ning 2014a). For example, Pan and Ning (2014a) proposed that the interdependence of the design strategies also exists in the spatial dimension, ranging from components within building through building as a whole to building beyond.

Notwithstanding the recognition of the interdependence, it is found that none of the studies empirically examined the interdependence within the high-rise ZCB systems.

6.3 *The Lack of a Socio-technical Framework*

Prior studies found that current technologies might not be able to achieve the ambitious zero emission target in high-rise buildings (Pan and Ning 2014b). It is, therefore, urgent to explore more advanced technologies. Hachem et al. (2014) argued that advanced design of façades can technically achieve zero energy in buildings of eight storeys. However, it is also recognized that the technical aspects only advocate a 'hero story' of the zero-carbon target (Janda & Topouzi, 2013, p. 229), which is, however, considered insufficient to achieve the ambitious zero emission target in reality. Pan and Ning (2014b) argued that "the delivery of high-rise ZCBs should not be regarded solely as a technological challenge, but a socio-technical uphill battle."

Thus, to realize a zero carbon target in the high rise buildings, a socio-technical framework of design strategies would be helpful. In addition to the technical aspects of: 1) improving energy efficiency; 2) adopting appropriate on and off-site renewable energy technologies; and 3) other carbon offsetting systems, the social-technical systems framework should further address another two social aspects (Pan and Ning, 2014a, b). First, instead of focusing on the implementation of the technologies, the social perspective should be addressed from the chain of the innovation, diffusion, adoption and implementation of technologies (Pan and Ning, 2015).

Second, the role of key stakeholders should be clarified. Pan and Ning (2014a) addressed that specifying a possible contribution from the industry and the power sector in the technical solutions is urgent to the ZCB delivery roadmap. Besides the technical problems, it will be a much more complex socio-technical problem to engage key stakeholders. This is because the collaboration among multiple stakeholders involves close social interactions; but each party has their own value propositions.

7 CONCLUSION

This paper has developed a socio-technical framework of design strategies for delivering high-rise ZCBs. A literature review was carried out; the results show that only two papers examined zero energy design strategies in high-rise buildings, rendering a significant gap in knowledge. Also, it is found that a common definition of high-rise ZCBs, albeit influences the design strategies significantly, is absent.

Another important implication is that the design strategies for achieving zero carbon should be addressed in a systems manner, which not only consider the system elements (e.g. passive design solutions, high energy-efficient service systems and renewable energies), but also address the interdependence among these system elements.

Lastly, zero carbon building is increasingly considered as a socio-technical system, which is, however, understudied in high-rise buildings. The vast majority of previous studies focused on the technology perspective, presumably arguing that the implementation of these technologies will achieve the outcome as intended. This approach is, however, criticized due to its ignorance of the social aspects. This paper has thus proposed an integrated socio-technical framework of design strategies, which not only admits the importance of technical aspects (e.g. energy efficiency design, on and off-site renewable energy technologies), but also raises the awareness of the social aspects. With the socio-technical design strategies integrated the myth of delivering high-rise ZCBs will be possibly turned into reality.

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Appendix 1 Passive design measures in high-rise buildings

Location	Design measures	Effects	References
Hong Kong	Daylight performance for kitchens	The small kitchen size has a significant effect on daylight level and distribution within kitchens	Lau et al. 2006
Hong Kong	Envelope color and thermal mass	The use of lighter surface color and thermal mass can reduce maximum indoor temperatures	Cheng et al., 2005
Hong Kong	Shading effects of neighboring buildings	The degree of shading at the local peak design condition (i.e. 15:00 July) ranges from 25 to 31%	Lam, 2000
Hong Kong	Envelope construction	Reduce the external heat gain and cut the cooling energy consumption by as much as 35% compared with a poor envelope design.	Chan and Chow, 1998
Korea	Building shape	Plate-type buildings consume less energy than tower-type buildings.	Choi e et al., 2012
Hong Kong	Advanced glazing	Low-e glazing leads to a reduction in cooling electricity use by up to 4.2%. Low-e reversible glazing: 1.9%; Double-clear glazing: 3.7% Clear plus low-e glazing: 6.6%.	Bojić and Yik , 2007
Hong Kong	Overhangs and side fins	Overhangs reduce the electricity consumption by up to 5.3%. Side fins with overhangs: 1.4%	Bojić, 2006
Hong Kong	Switchable glazing	Reduces annual cooling electricity consumption: 6.6%	Yik and Bojić , 2006
Hong Kong	External walls and partitions	Insulating the envelope and the partitions reduce space cooling load: 38%, but could either increase 19% or reduce 16% of the peak cooling demand.	Bojić and Yik, 2005
Hong Kong	Envelope and partition	Applying thermal insulation to external walls without increasing the thickness of the concrete layer increases yearly cooling load when the concrete layer is thickened but without addition of insulation. Improving the thermal insulation of the partitions separating air-conditioned and non-air-conditioned spaces within the apartments is an effective way of reducing cooling load.	Bojić et al., 2002a
Hong Kong	Thermal insulation layer in the fabric components	Up to 9.1% and 10.5% of yearly cooling load reduction and maximum cooling demands could be obtained respectively when a 50 mm thick thermal insulation layer is placed at the indoor side of the walls that enclose the cooled spaces.	Bojić et al., 2002b
Hong Kong	Windows (i.e., clear, tinted, reflective and tinted + reflective)	Highest decrease in Q of roughly 10% and that in D of 11% are obtained for the flat facing west when the clear glass is replaced with the reflective, tinted glass.	Bojić et al., 2002c
Hong Kong	Thermal insulation position in building envelope	Adding a 5 cm think thermal insulation layer inside the residential flat leads to the highest decrease of 6.8% of the yearly cooling load.	Bojić et al., 2001
Hong Kong	Improved building envelope design	For a predominantly night-occupied apartment, improving the thermal performance of external wall is more effective than those for windows. A saving of 31.4% in annual required cooling energy and 36.8% in the peak cooling load can be achieved with the improved building envelope design.	Cheung et al., 2005
Southern China	Facade design	Coloring or shading of the external façade are more cost effective means to reduce air-conditioning loading than the application of insulation.	Niu, 2004
Germany	Double facades	For specific locations (wind or noise) double facades save more energy as compared with conventional solutions with full air conditioning.	Pasquay, 2004
Chongqing, China	Building orientation and spacing	Optimized design reduces the age of air to less than 6 minutes in 90% of the rooms, as compared to an age of greater than 30 minutes in 50% of the rooms in a conventional design.	Zhou et al., 2014

Location	Design measures	Effects	References
Jakarta, Indonesia	A life cycle energy assessment of an apartment on the 20th floo	Enclosures contribute 20-30% of the total heat gain. The potential of double walls could be improved by increasing U-value and reducing the thickness which will reduce its specific heat.	Utama and Gheewala, 2009
Iran	Vertical double glazing windows	The alteration could reduce energy consumption to approximately 30–35% in the highest level in comparison with an ordinary building.	Lotfabadi ,2014
Hong Kong	Raising the indoor temperature, thermal insulation, double glazing and tinted glass	Raising the indoor temperature has the best mitigation potential.	Wong et al., 2010
Hong Kong	Day lighting and OTTV designs	With day lighting designs, the reductions for the peak cooling load and annual electricity consumption for the base-case model were 10 and 13%, respectively.	Li et al., 2002
Hong Kong	Light-pipe system	Using on–off and high frequency dimming controls, the lighting energy expenditures were respectively, 811 and 635 kW h. These represent 54% and 42% of the lighting energy use without any lighting control.	Li et al., 2010
Singapore	Green roof	Lower surface temperature significantly	Nyuk et al. 2007
Hong Kong	Building envelope construction	Yearly cooling load increases when the thermal insulation was applied to external walls without increasing the thickness of the concrete layer, and when the concrete layer was thickened but without addition of insulation	Bojić et al., 2002e
Hot summer and cold winter zone China	Shape coefficient, Heat-transfer coefficient of wall, roof and window, Solar absorptance of wall and roof, WWR, Shading coefficient	In cooling season, shading coefficient and WWR are the most vital factors; in heating season, wall heat-transfer coefficient and shape coefficient have crucial effects when WWRs are 25% and 50%, respectively	Yu et al.., 2013
Hong Kong	Green roof	Top barriers are lack of promotion and incentives from governments and the increase maintenance cost	Zhang et al., 2012
Korea	Light-pipe systems	Lighting energy usage saving: 30%	Shin et al., 2012
Seoul, Korea	Exterior shading devices	The cooling energy saving potential: 20%, The horizontal overhang and the vertical panel decrease the cooling energy 19.7% and 17.3%, respectively	Cho et al., 2014
Singapore	Double-skin facade	South-facing facade has the best outcome in the month of January. North-facing and West facing facades fail to provide an acceptable indoor thermal comfort for the purposes of office function	Wong et al., 2008
Dubai	Glass type, visible light transmittance, reflection and relative heat gain	Most the glass/glazing was misused in 70% of buildings in intermediate and low performance groups.	Aboulnaga, 2006
Hong Kong	Insulation, thermal mass, colour of external walls, glazing systems, window size and shading devices	Saving of 31.4% in annual required cooling energy and 36.8% in peak cooling load	Cheung et la., 2005
Hong Kong	Light-pipe system	A high potential to reduce the electric lighting energy consumption.	Li et al., 2010
Sydney	Horizontal light pipes and vertical light pipe, wind siphonage, long volume turbines	They all have the potential to reduce energy consumptions.	West, 2001

Appendix 2 Energy efficiency measures in HVAC systems

Location	Measures	Effects	References
Hong Kong	HVAC system combining chilled-ceiling with desiccant cooling	Chilled-ceiling combined desiccant cooling could save up to 44% energy consumption	Niu et al., 2002
Hong Kong	A combination of cooled ceiling, microencapsulated phase change material slurry storage and evaporative cooling technologies in air-conditioning	Energy saving potential up to 10%	Wang et al., 2008
Hong Kong	Combining cooled ceiling and a microencapsulated phase change material (MPCM) slurry storage tank	Shift the part of cooling load from the daytime to nighttime	Wang and Niu, 2009
Shanghai, China	Changing the secondary chilled water pumps and hot water pumps from constant speed into variable speed	Save 3.9% primary energy; decreasing the lighting power density from 12 to 9.31 W/m^2	Pan et al., 2007
Hong Kong	Secondary chilled water system	Annual energy savings: about 1,034,594 kWh.	Ma et al., 2009
Hong Kong	Liquid desiccant cooling system (LDCS)	The electricity driven LDCS not suitable for local commercial buildings.	Qi et al., 2012
Bangladesh	Lift	15% energy saving	Ahmed et al., 2014
Hong Kong	Multi-zone demand-controlled ventilation (DCV) strategy with two schemes using limited sensors for multi-zone offices.	First scheme: 50% saving when using the DCV strategy having full sensor instrumentation was 52%. The second scheme saved 45% energy	Shan et al., 2012
Supper high-rise building	Fault-tolerant and energy efficient control strategy for primary–secondary chilled water systems	The energy saving can be up to over 70% and 50% at system starting and normal operation periods respectively.	Cao et al., 2011

Appendix 3 Literature related to renewable energy

Location	Measures	Effects	References
Hong Kong	Centralized solar water-heating system	Annual efficiency of the vertical solar collectors could reach 38.4% on average, giving a solar fraction of 53.4% and a payback period of 9.2 years	Chow et al., 2006
India	Harnessing energy from grey water	Expected outcome is 6.85 kWh	Sarkar et al., 2014
Hong Kong	Full- and the partial-solid oxide fuel cell-trigeneration	have 51.4% and 23.9% carbon emission cut respectively	Fong and Lee, 2014
Hong Kong	Building integrated photovoltaic	The output energy ranged from 10 to 360 kWh with an average value of 116.3 kWh.	Li et al., 2013
Malaysia	Power augmented vertical axis wind turbine	the power output increment of the rotor with the power-augmentation-guide-vane (PAGV) was 5.8 times at the wind speed of 3 m/s.	Chong et al., 2013
Malaysia	3-in-1 wind-solar hybrid renewable energy and rain water harvester	the estimated annual energy generated and savings is 160 MW h.	Chong et al., 2012
Malaysia	Wind-solar hybrid renewable energy system with rainwater collection feature	Estimated annual energy savings is 195.2MWh/year	Chong et al., 2011
Montreal, Canada	PV systems in roofs and façades	Electricity production of up to 90% and 50% of energy use for 4-story, 12-storey respectively.	Hachem et al., 2014
Hong Kong	Semi-transparent amorphous silicon PV module together with the dimming controls	The annual building electricity saving of 1203 MWh and peak cooling load reduction of 450 kW	Li et al., 2009

Emergency preparedness of disasters for safe and sustainable development

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Keywords: emergency preparedness; landslides.

ABSTRACT: The Hong Kong Slope Safety System, which is administered by the Geotechnical Engineering Office (GEO), has been evolving with time in response to historical rainfall and landslides events and to incorporate enhancements arising from improved knowledge and practice in the past 30 to 40 years. Apart from being regarded as a role model in landslide risk management, the system has also contributed to support urban and infrastructure development as well as the economic growth of Hong Kong. In recognition of the likelihood of more frequent occurrence of extreme rainfall conditions due to the effects of climate change, the GEO has made an expanded effort to assess and improve the resilience of the Hong Kong Slope Safety System in combating extreme rainfall events. The key issues including identifying the nature and scale of the extreme landslide events, assessing the severity of the landslide consequences, evaluating the capacity of emergency management and improving crisis preparedness as well as the need for adopting an updated approach to enhance community resilience, are highlighted in this paper.

1 INTRODUCTION

Natural disasters are hitting every corner of the earth, bringing grave consequences with an increasing scale and frequency. According to the United Nations Office for Disaster Risk Reduction, 42 million life years were lost between 1980 and 2012 in internationally reported disaster each year and the economic losses from disasters are now reaching an average of US$250 to 300 billion (UNISDR, 2015). The number and severity of impact from these disasters are on the rise and the trend is likely to continue given the influence of global climate change.

It is recognised internationally that good governance and disaster risk reduction are mutually supportive objectives at various scales from international, to regional, national and local. Efforts to reduce disaster risks should be integrated with Government policies, programmes and plans for sustainable development (UNISDR, 2007).

Hong Kong has a population of 7 million and a small land area of 1,100 km^2, only 15% of which is developed land. The terrain is hilly (with 75% of the land steeper than 15^o and 30% steeper than 30^o). The vulnerable setting, comprising dense urban development, a legacy of inadequate geotechnical control before the 1970s, coupled with high seasonal rainfall has given rise to acute landslide problems as one of the most deadly disasters affecting Hong Kong.

The Geotechnical Engineering Office (GEO), which was established by the Hong Kong Government in 1977, has implemented a comprehensive Slope Safety System. This embraces a range of initiatives covering policy, strategic, legislative, administrative, technical and procedural frameworks that serve to manage landslide risk in a holistic manner. The Slope Safety System has been evolving with time in response to historical rainfall and landslides events and to incorporate enhancements arising from improved knowledge and practice and is renowned for its performance in reducing and managing landslide risk in Hong Kong, as well as supporting the safe and sustainable development of Hong Kong. This is evident from the significant reduction in landslide fatalities compared with the population growth in Hong Kong (see Figure 1), and its rapid urban and economic development in the past few decades. The key components of the Slope Safety System are shown in Table 1.

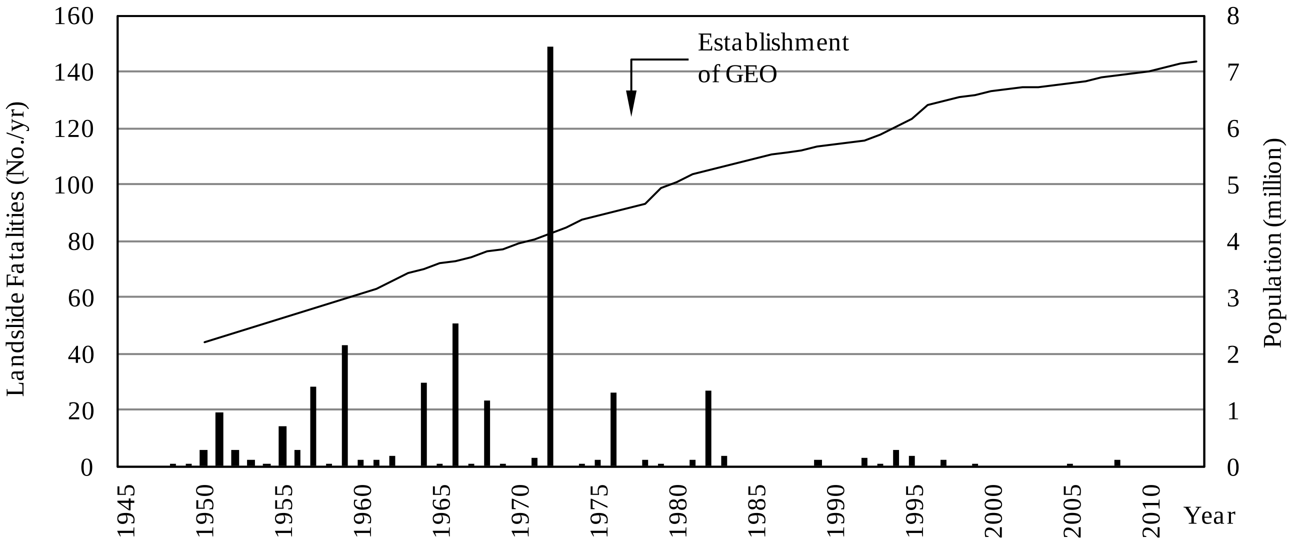


Figure 1 Landslide fatalities and population in Hong Kong

Table 1 Key components of the Hong Kong Slope Safety System

	Contribution by each component		
	reduce landslip risk		address public attitude & perception
	hazard	consequence	
Policing			
cataloguing, safety screening and statutory repair orders for slopes	✓		
checking new works	✓	✓	
slope maintenance audit	✓		
inspecting squatter areas and recommending safety clearance		✓	
input to land use planning	✓	✓	
Safety requirements, technical standards and research			
[e.g. natural terrain hazard study and mitigation, debris mobility, etc.]	✓	✓	✓
Works projects			
upgrading existing Government man-made slopes	✓		
mitigating natural terrain landslide hazards	✓	✓	
Education and information			
slope maintenance campaign	✓		✓
personal precautions campaign		✓	✓
slope safety awareness programme	✓	✓	✓
information services	✓	✓	✓
landslide warning and emergency services	✓	✓	✓

Note: Regular maintenance of registered Government man-made slopes and natural terrain defence/stabilisation measures is carried out by the responsible Government departments.

The multi-pronged approach includes a range of initiatives in the System to manage landslide risk in a holistic manner, via: (i) improving slope safety standards, technology and administrative and regulatory frameworks, (ii) ensuring safety standards of new slopes, (iii) rectifying sub-standard Government slopes and maintaining them, (iv) ensuring that private owners take responsibility for slope safety, and (v) promoting public awareness in and response to slope safety.

So far, the system has generally proved to be effective in coping with the prevailing landslide problems. However, the more frequent occurrences of extreme rainfall causing serious landslides and major casualties in different parts of the world in recent years have highlighted the impact of extreme rainfall on the community and concerns about the possible effect of climate change on landslide hazards.

2 EXTREME WEATHER-RELATED LANDSLIDE EVENTS

2.1 *Extreme Weather-related Landslide Disasters around the Earth*

According to the Intergovernmental Panel on Climate Change (IPCC), extreme weather refers to “the occurrence of a value of a weather or climate variable above (or below) a threshold value near the upper (or lower) ends of the range of observed values of the variable” (IPCC, 2012). Around the globe, catastrophic landslides induced by extreme rainfall causing serious casualties have been reported increasingly in recent years. For example, the landslide at Xiaolin village during the passage of Typhoon Morakot in Taiwan in 2009 killed more than 400 people; the debris flow in Gansu in 2010 led to more than 1,400 fatalities; the rain-induced landslides in Brazil in 2011 caused more than 900 fatalities; the torrential rain in South Korea in 2011 brought about more than 30 landslide-related fatalities; Tropical Storm Washi brought flash floods and severe landslides, resulting in 1,010 fatalities in the Philippines in 2011; Severe Typhoon Wipha struck eastern Japan in 2013 and caused more than 40 landslide-related fatalities, etc.

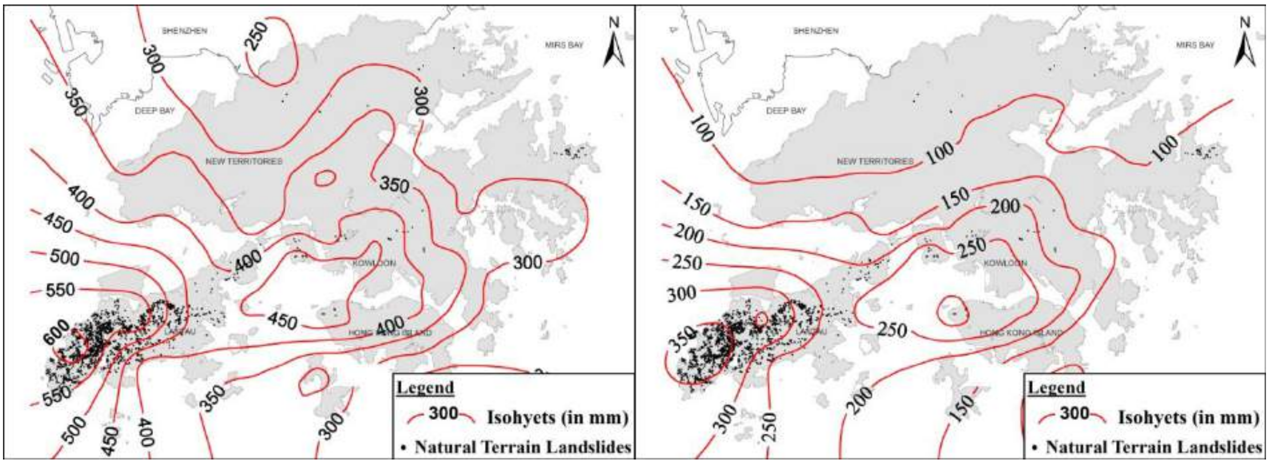
2.2 *Extreme Weather-related Landslide Disasters in Hong Kong*

Hong Kong was not immune from extreme rainfall events. The following is a brief account of the severe rainstorms and landslide damages experienced in Hong Kong since the 1960s:

a) Rainstorm of June 1966 - The rainstorm primarily struck Hong Kong Island and triggered hundreds of landslides and flooding which resulted in 64 fatalities, >2,000 people homeless, >8,600 people temporarily evacuated and >400 houses damaged (Chen, 1969).
b) Rainstorm of June 1972 - The severe rainstorm caused many significant landslides in various places of Hong Kong, including the landslide disasters at Sau Mau Ping and Po Shan Road which killed more than 130 people.
c) Rainstorm of May and August 1982 - Over 600 mm of rainfall was recorded within the four days from 28 to 31 May 1982 and more than 520 mm of rainfall from 15 to 19 August 1982. The two intense rainstorms triggered over 1,400 natural terrain landslides and caused serious damage particularly to squatter areas (with over 20 fatalities) (Hudson, 1993; Lee, 1983; Wong et al, 2006).
d) Rainstorm of November 1993 - Over 700 mm of rainfall was recorded in 24 hours over Lantau Island triggering >800 natural terrain landslides and >300 man-made slope failures, which resulted in closure of roads and evacuation of houses. (Wong et al, 1997 and Wong & Ho, 1995)
e) Rainstorm of July 1994 - During the heavy rainfall episode from 22 to 24 July, over 600 mm rainfall was recorded at Hong Kong Observatory, together with record-breaking maximum 1-hour, and 24-hour rainfall of 185.5 mm and 954 mm respectively recorded at Tai Mo Shan. This resulted in about reported 200 landslides, 5 fatalities, 4 injuries, closure of roads and evacuations of houses (Chan, 1996).

f) Rainstorm of August 1999 - Over 500 mm of rainfall was recorded in 24 hours over the southern part of the New Territories triggering >200 man-made slope failures, which resulted in 1 fatality and permanent evacuation of 3 public housing blocks (Ho et al, 2002).

g) Rainstorm of June 2008 - This very severe rainstorm resulted in widespread natural terrain landslides, with many large scale and mobile failures, and two fatalities. The maximum rolling 4-hour and 24-hour rainfall recorded for the rainstorm were 384 mm and 623 mm respectively (see Figure 2) with the statistical return period in the order of 1,000 and 200 years respectively. This was probably the most severe rainstorm since rainfall record began in Hong Kong in 1885. The torrential rain in June 2008 fell largely on Lantau Island, the largest outlying island and one of the sparsely populated areas in Hong Kong. The consequences could have been much more serious had the 2008 rainstorm hit the more densely developed urban areas of Hong Kong.



(a) Maximum rolling 24-hour rainfall distribution (b) Maximum rolling 4-hour rainfall distribution

Figure 2 Maximum rolling rainfall distributions for the 7 June 2008 rainstorm

2.3 *Effect of Climate Change*

In 2013, the IPCC in its Fifth Assessment Report of Working Group I (WGI AR5) reaffirms that under the high concentration scenario (i.e. Representative Concentration Pathway RCP8.5), most land regions on the Earth (including southern China) would experience an increase in extreme rainfall in term of annual maximum five-day precipitation for the rest of this century (IPCC, 2013).

In the last 60 years, Hong Kong's annual precipitation exhibited an increasing trend of 36 mm/decade, though statistically insignificant. On the analysis of the past trend in extreme rainfall in Hong Kong, Wong & Mok (2009) found that the return periods of short-duration extreme rainfall events had decreased significantly from the year 1885 to 2009. For example, the return period of hourly rainfall of 100 mm or more shortened from 37 years in 1900 to 18 years in 2000, indicating that such heavy rainfall events have become more frequent.

The maximum hourly rainfall record of the Hong Kong Observatory (HKO) Headquarter has been broken several times since 1885 and the time interval between new records is getting shorter. In particular, the record was broken three times since 1966. The latest record of 145.5 mm was set on 7 June 2008, breaking the previous one by a wide margin of 30 mm (Figure 3). In particular, the HKO has conducted climate projection studies using the technique of statistical downscaling. Under the scenario RCP8.5, the annual rainfall in late 21st century is expected to increase by about 180 mm compared to the 1986-2005 average (with a likely range of -250 mm to +800 mm) (See Figure 4). To conclude, climate change is liable to render extreme rainfall events more frequent, as well as more intense.

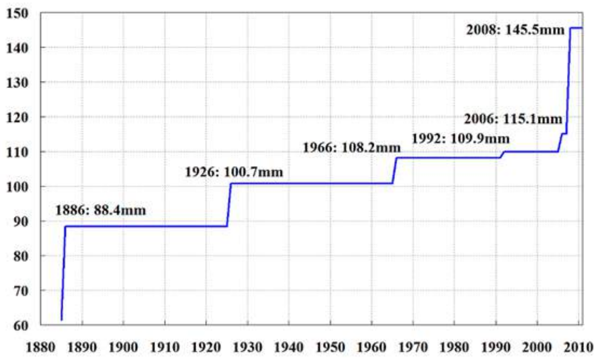
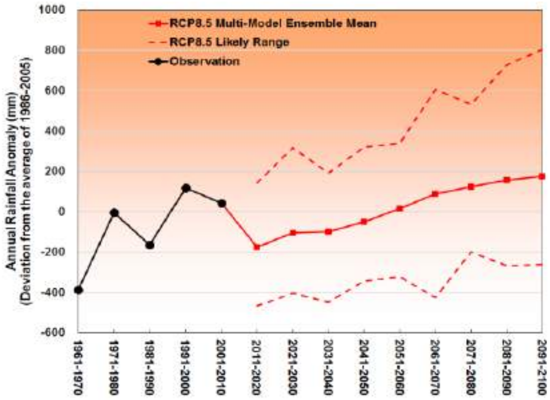


Figure 3 Maximum hourly rainfall recorded at Hong Kong Observatory
(Source: Hong Kong Observatory)



Note: Likely range refers to the region embraced by the 5th and 95th percentiles of the multi-model ensemble.

Figure 4 Past and projected annual rainfall anomaly of Hong Kong under the RCP8.5 scenario

(Source: Hong Kong Observatory)

3 UNDERSTANDING THE RISK

3.1 *Nature and Magnitude of Extreme Rainfall*

The more frequent occurrences of extreme rainfall causing major casualties around the world in recent years have highlighted the real threat of the potential impact of extreme rainfall on community and concerns about the possible effect of climate change on slope safety. The theoretical upper limit of the amount of rain that can fall over a given area is defined by Probable Maximum Precipitation (PMP) (WMO, 2009). PMP is commonly used in reservoir and dam engineering with no allowance made for long-term climatic trends.

The PMP for Hong Kong was first derived in 1968 for waterworks developments by the HKO (Bell & Chin, 1968). In 1998, the GEO and HKO collaborated on a study to update the 24-hour PMP (HKO, 1999). This PMP was used by the GEO to develop possible extreme landslide event scenario for reviewing the adequacy of the Government's landslide emergency preparedness (Sun et al, 2001).

In 2014, the GEO completed a study to further update the 24-hour PMP estimates. In this third-generation update, four major rainstorms in Taiwan (viz. 1996 Typhoons Herb, 2004 Aere, 2005 Haitang and 2009 Morakot) were transposed to Hong Kong by using the storm separation technique (namely the Step Duration Orographic Intensification Factors (SDOIF) Method). The Depth-Area-Duration curve of the updated PMP estimate is shown in Figure 5. The updated PMP for 10 km^2 and 100 km^2 were estimated to be 1,510 mm and 1,350 mm respectively (AECOM & Lin, 2014).

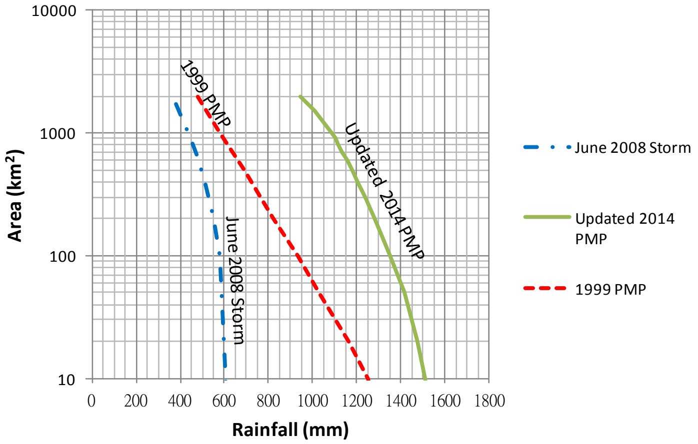


Figure 5 Depth-Area-Duration curves for updated 24-hour PMP for Hong Kong

3.2 *Extreme Landslide Scenarios*

By nature, PMP is similar to the 'Maximum Credible Earthquake' concept in seismic hazard assessment and evaluation of extreme seismic event. In practice, realistic and credible extreme rainfall scenarios can be established by reference to the PMP estimates for a 'stress test' on the prevailing landslide emergency system (Wong, 2013).

For this purpose, the GEO opted to consider two extreme rainfall scenarios separately, namely (a) a near-miss event transposed to Hong Kong Island (the densely populated urbanized area in Hong Kong), and (b) a rarer and more severe but credible rainfall scenario striking Hong Kong Island taking into account the effect of climate change. The former one is relevant to a rainstorm of a return period in the order of 1,000 years. For the latter scenario, rainfall isohyets of Typhoon Morakot that hit Taiwan in 2009 are transposed to Hong Kong taking into account the topography of Hong Kong and the possible increase in tropical cyclone rainfall due to the effect of climate change projected to the end of 21st century. This also corresponds to 70% of the 2014 PMP with a notional return period in the order of 10,000 years under the current climate condition. It should be noted that this notional return period would be reducing with time under the effect of climate change. The assessment carried out so far is preliminary in nature and subjected to refinements with gain in new knowledge. The results of the preliminary assessment are indicated in Table 2.

Table 2 Expected consequences in landslide scenarios

Landslide scenarios	Total no. of landslides	No. of landslides affecting buildings/roads
Near-miss Event (Transposing June 2008 rainstorm to strike Hong Kong Island)	2,000	200 – 300
More Extreme Event (Transposing 2009 Typhoon Morakot rainstorm to strike Hong Kong Island with climate change effect projection to end of 21st century)	50,000	4,000 – 9,000

The assessment indicated that, for scenario of a near-miss event, about 2,000 landslides could occur, some 200-300 of which would impact on buildings or roads. For the more extreme event, the assessment indicated that about 50,000 landslides could occur, some 4,000 to 9,000 of which could impact on buildings or roads. The landslides would result in the detachment of about 10% of the natural terrain area. It should be noted that there are many uncertainties in the assessment and, hence, the numbers of landslides predicted should be treated as rough figures indicating the order of magnitude of the possible event, rather than precise scientific results.

4 HAZARD MANAGEMENT FOR SAFE AND SUSTAINABLE DEVELOPMENT

4.1 *Development, Asset and Hazard Management*

Echoing with UNISDR's advice that efforts to reduce disaster risks should be integrated with Government development policies, programmes and plans for sustainable development, safe and sustainable development of a modern city, likes Hong Kong relies on a three-fold management strategy:

a) Development Management – Planning and implementation of sustainable development initiatives as a driver to meet the short and long term needs of the community;
b) Asset Management – Maintaining and operating the infrastructure as an asset to serve the vibrant community;
c) Hazard Management – Improving the engineering of the infrastructure and the preparedness of the system and community, to effectively manage the risk and enhance the resilience of the community in overcoming extreme events.

Development and asset management are usually well addressed so as to deliver projects in a timely and cost-effective manner and maintain the infrastructure in safe and functional conditions. Hazard management is usually less emphasised in the community. In the face of the increasing scale, frequency and severity of natural disasters, greater emphasis needs to be placed on hazard management. To support the long term sustainable development of Hong Kong under the threat of climate change, there is the need for enhancing the resilient of the Hong Kong Slope Safety System, which includes heightening the public's awareness for vigilance and emergency preparedness within the community.

4.2 *Hazard Management*

Hazard management is the discipline that deals with or mitigates the risk of weather-related extreme events or natural disasters. It serves to protect the community from the adverse consequences of disasters, via developing and implementing processes to manage a range of related phases and activities. In the context of landslide hazards, it includes:

a) Prevention: This involves preventive activities to provide protection from disasters, e.g. setting up and enforcing the use of suitable slope investigation, design, construction, supervision and maintenance standards.
b) Mitigation: This involves implementation of engineering measures to lessen the impact of landslides, e.g. retrofitting substandard slopes to reduce the chance of landslide, or providing mitigation measures (e.g. landslide barriers or check dams) to reduce adverse landslide consequences.
c) Preparedness: This focuses on formulating procedures, managing human resources, setting up emergency systems, preparing equipment, etc. for prompt mobilisation when a disaster occurs. It also involves preparing the vulnerable community to respond promptly and effectively to the landslide disaster, e.g. taking suitable personal precautionary actions during Landslip Warning.
d) Response: The response phase of an emergency may involve search and rescue, as well as evacuation and fulfilling the basic humanitarian needs of the affected population. Emergency inspections for identification of any imminent danger and advice on necessary response actions, safe settlement of those people who have been evacuated and provision of relief measures, are examples of response activities.
e) Recovery: The recovery phase starts after the immediate threat to life has been dealt with. The goal is to bring the affected area back to normal, e.g. carrying out landslide repair or mitigation works.

While slope safety systems commonly include provision for emergency management, the existing systems are rarely designed for, nor tested against, extreme events. The Hong Kong Slope Safety System is of no exception. The System has proved to be effective in managing landslide risk and in dealing with emergency management under more commonly occurring severe rainfall conditions (i.e. the rainfall levels below extreme value). However, the System has not been specifically designed to cope with extreme events.

Traditional disaster risk management focuses on preventing particular events occurring or by mitigating the consequences. Instead, resilience approaches risk from a different perspective centred on developing strategies to deal with disruptive events if and when they occur. Emphasis is placed upon anticipation, preparedness and recovery rather than prevention, and on the inherent ability of the system to respond and adapt to disturbances rather than hazard-specific risk mitigation (da Silva, 2012).

To combat the risk of extreme weather events, resilience of the community should be enhanced against the weather-related hazards. Resilience can be enhanced through an engineering approach to upgrade Hong Kong's infrastructure, making it more robust in protecting the community from serious hazards. This encompasses the adoption of improved design standards and practice, as well as implementation of retrofitting and improvement works to upgrade facilities. It also calls for support through research, development and technological innovation.

As part of the System, old man-made slopes have been upgraded and natural terrain landslide hazards have been mitigated systematically through the completed Landslip Preventive Measures Programme and the dovetailed Landslip Prevention and Mitigation Programme. Over the last three decades, more than 7,000 slopes were dealt with under the Programmes at a total expenditure of about $12 billion, bringing about over 75% reduction in the overall landslide risk.

Following the June 2008 rainstorm, enhanced technical guidance was promulgated on the delineation of adverse site settings for the potential development of highly mobile channelised debris flows associated with major drainage lines fed by sizeable catchments. In addition, guidance was issued on the appropriate parameters for landslide debris runout modelling based on back analyses of actual mobile debris flows in 2008. Furthermore, enhanced technical guidance on natural terrain hazard studies, together with empirical design of standard barriers for open hillside landslides, was promulgated in 2013 following consolidation of experience in the past few years and a probabilistic assessment of the scale and mobility of open hillside landslides triggered by the June 2008 rainstorm.

While improving the design and upgrading infrastructure facilities are important to hazard prevention and mitigation, this calls for significant financial investment. Hence, it is not sufficient, nor cost-effective, to count solely on engineering solutions in hazard management. The engineering approach needs to be supplemented with non-works measures covering emergency preparedness, response and recovery. In contrast to engineering works, these non-works measures entail with ‘software’ aspect to improve the robustness of our system in coping with emergency and enhance the resilience of the community in surmounting crises. In practice, it includes setting up warning systems for the relevant hazards, educating the public, providing emergency services in times of crisis to rescue and evacuate the population at risk, carrying out urgent repairs, implementing recovery initiatives, etc.

4.3 *Landslide Emergency Management*

The GEO provides a 24-hour year-round landslide emergency service to give professional advice to Government departments on actions to be taken in case of landslide danger. The GEO endeavours to attend to each significant or serious landslide incident in order to undertake an inspection and give advice on the immediate precautionary measures (e.g. evacuation or road closure) and urgent works needed to remove the immediate landslide danger and safeguard public safety. The aim is to minimise casualty and damage to property, and facilitate recovery under severe rainfall conditions. The findings from the above scenario-based assessments were used as a form of stress test to review the adequacy of the GEO’s prevailing landslide emergency preparedness.

When it is predicted that numerous landslides will occur based on recorded and forecasted rainfall, the HKO in consultation with the GEO will issue the territory-wide Landslip Warning to alert the general public of the potential landslide danger. As shown in Table 3 below, the landslide emergency service has played a significant role in minimising the exposure of the general public to landslide danger, thereby reducing landslide risk.

Table 3 Summary of GEO’s landslide emergency service from 1994 to 2013

No. of Landslip Warning issued	No. of reported landslide incidents	No. of building units evacuated *			No. of road sections closed
		Block	House	Flat/Unit	
62	5,274	21	119	1,003	2,434

Note: * A ‘block’ is a multi-storey building, which may comprise up to several dozens of flats/units. A ‘house’ is typically within 3 storeys, which comprises several flats/units.

An assessment has been made of the likely capacity of the landslide emergency management system, based on consideration of the available human resources for emergency inspection of landslide incidents that require GEO’s input in emergency response (Wong, 2013). The findings indicate that the current system is able to handle about 200 to 300 landslides on natural terrains or man-made slopes affecting buildings or roads following the current practice of dealing with reported landslides.

It is expected that a rainstorm with intensity equivalent to the near-miss event transposed to Hong Kong Island would stretch the existing system to the limit. In comparison, about one-third of the system capacity was mobilised in June 2008 when the rainfall event hit the relatively sparsely populated and less accessible western part of Lantau Island.

If a more extreme rainstorm corresponding to 70% of the 2014 PMP were to hit Hong Kong Island, the very large number of predicted natural terrain landslides would completely overwhelm the capacity of the current landslide emergency management system. Both the response and recovery phases would face acute challenges and the landslide consequences are likely to be very serious. The existing mode of emergency service will become impractical under this scenario. Apart from the very large number of landslides that will be reported, many of them will likely become inaccessible as a large number of roads and footpaths are expected to be blocked by landslide debris, or flooding. A new strategy for managing the landslide emergency is therefore needed.

4.4 *Enhancing Emergency Preparedness*

4.4.1 *Strengthening and streamlining landslide emergency service*

Based on the findings of the stress tests using the scenario-based assessments, the Hong Kong's landslide emergency system can barely cope with the first extreme scenario (a near-miss event transposed to Hong Kong Island). The following have been undertaken recently to strengthen and streamline the emergency service:

a) Draw up a list of more experienced geologists/geotechnical engineers as a contingency provision for prompt deployment to the field to assist in assessing the residual risk of major natural terrain landslides.
b) Put in place arrangements for GEO works contractors to be mobilised, if and when needed, to undertake emergency repair or risk mitigation works on large-scale natural terrain landslides. Flexible barrier components have been stockpiled for possible use in emergency works.
c) Use of modern information technology for sharing of key information and enhanced communication in order to facilitate prompt emergency response. A new, web-based landslide information management system was developed. This system is equipped with a mobile map and geo-location service that enables users to identify and record landslide locations conveniently. The timely reporting of key landslide information to aid identification and classification of landslides is particularly important under an extreme landslide scenario, as it will support the Emergency Manager in decision-making, action planning and setting priorities for the deployment of emergency teams.
d) Put in place an Emergency Command System to handle situations that individual infrastructure departments might be unable to cope with the widespread damage in order to enhance the coordination of repair and recovery works.

There is scope for further streamlining and continuous improvement. Potential areas include the following:

a) Sourcing additional and contingency resources to narrow the gap between the demand for emergency services and the existing emergency management capacity (e.g. develop plans for mobilisation of resources from the private sector and quasi-government organisations to assist in emergency management under extreme landslide events).
b) Streamlining the procedures with focus on setting up priority for emergency response actions.
c) Revamping the emergency management strategy to make it more pragmatic and effective to implement under the constraints of extreme landslide events. For example, geotechnical engineers may be deployed in district police/fire service command centres for more effective provision of geotechnical advice on emergency rescue actions.
d) The GEO has recently developed a separate set of landslip alert criteria particularly for natural terrain to supplement the current Landslip Warning system (Chan et al., 2012). These have been in use since 2012 for internal reference by the GEO Emergency Manager for a timely

alert on possible widespread occurrence of natural terrain landslides. As natural terrain landslides are usually activated at higher rainfall level than that for Landslip Warning, the natural terrain landslip alert criteria may be further developed and used for forewarning in the extreme event.

e) Revamping the emergency information system to facilitate sharing of information and communication among the different Government bureaux/departments/agencies responsible for different types of hazards and emergency responses via a GIS common operating platform.
f) Providing enhanced advices and training to the Police and Fire Services Department on appropriate actions to be taken by their front-line staff on emergency duties for typical landslide incidents.
g) Developing an emergency service continuity plan to react efficiently to unexpected situations, e.g. prolonged interruption of utility services (including power grids and communication networks).

Other bottlenecks will almost certainly exist in emergency management for extreme events, such as transport arrangements, communication facilities, and provision for prompt and safe settlement of the affected community, which are constrained by the capacity of other Government agencies or non-government organisations involved in emergency management. All these issues need to be addressed in a holistic manner as part of the emergency preparedness for extreme landslide scenarios.

4.4.2 *Enhancing coordination of Government emergency services*

The Hong Kong Government has put in place a Contingency Plan for Natural Disasters to ensure that all departments concerned will respond quickly and effectively in a coordinated manner to deal with emergency situations. The Government conducted an inter-departmental exercise in April 2012 to test its response and capabilities in the event of a serious nuclear incident at the Daya Bay Nuclear Power Station. The exercise included a drill on emergency evacuation of about 120 people caused by a landslide scenario that might occur during prolonged period of heavy rain incidental to the nuclear accident. The landslide drill provided an opportunity to test the Government's response at various critical stages of emergency evacuation, including evacuation of occupants, leading them out of the danger zone, taking stock of the evacuees and transporting them to temporary shelters.

Experience suggests that failures of leadership, communications and information-sharing during disasters are not uncommon. These problems are to be reduced through running of large-scale drills and simulation exercises. Realistic scenarios and involvement of the key agencies responsible for the response are of essence for the drills and simulation exercises.

4.4.3 *Enhancing public education and communication*

The resilience of the Slope Safety System in combating extreme events is critically affected by the resilience of the vulnerable population and the relevant stakeholders in facing the events. Maintaining vigilance of the extreme landslide scenarios and awareness of emergency management is essential to crisis preparedness. Hence, public education and communication are as important as other technical approaches (Tam & Lui, 2013).

The potential areas of improvement may include (a) communication with the public during a crisis, such as pre-written press releases and clear protocols in terms of the officers that will engage with the media and (b) public awareness of possible extreme landslide scenarios and equipping the general public with the knowledge and skills required for emergency management, e.g. launch appropriate publicity campaigns and public education programmes. Members of the public will need to be empowered and encouraged to take appropriate emergency response actions in order to protect themselves and their families, and to minimise their exposure to landslide hazards. This may be done by providing them with the necessary knowledge and information on the latest landslide and rainfall situations in a timely manner so as to assist them to make their own decisions on emergency response, before the arrival of the GEO staff or other emergency staff. An analogy will be the general public taking appropriate emergency response actions in the event of a fire before the arrival of the firemen.

Focused and enhanced publicity and public education efforts are warranted to improve the community resilience against extreme landslide events.

4.5 *Multiple Hazards*

By virtue of the vulnerable and densely urbanised settings, it is perceivable that an extreme weather event hitting Hong Kong could result in multiple hazards, such as landslides, flooding, storm surges, tree falls, damages to squatter huts and buildings by wind, breach of reservoirs and catchwaters, damage to port facilities by waves, etc., which call for different forms of emergency operations as well as coordination and collaboration. For example, there are six coastal low-lying spots (viz. two in Tuen Mun, and one in Sai Kung, Sham Tseng, Lei Yue Mun and Tai O each) in Hong Kong that are vulnerable to storm surge flooding during typhoons. The Government has established early warning systems to deal with the storm surge flooding in these spots. However, if an extreme event corresponding to a typhoon similar to Super Typhoon Usagi, but taking the unfavourable route as discussed above, occurs, the storm surge would lead to series flooding in many parts of the urban areas in Hong Kong in addition to the six low-lying spots.

The storm surge flooding itself is a serious threat to the public. Tremendous emergency resources may be deployed as the affected area may be extensive. It should be noteworthy that the combined effect of the storm surge with other rainfall-related hazards such as landslides would be even more serious. The flooding would not only affect the population in low-lying areas, but also would block the escape routes for people in danger due to other concurrent hazards, hinder the mobilisation of the emergency services and leave many of the temporary shelters and emergency supplies inaccessible.

To effectively handle the impacts of multiple hazards that could happen concurrently, the adoption of a holistic approach is of the essence in assessing the severity of the plausible combination of extreme events and development of suitable crisis preparedness plans.

5 CONCLUSION

The Hong Kong Slope Safety System has been performing well in reducing and managing landslide risk in Hong Kong and in supporting the development of the city. Under the threat of climate change, there is the need for enhancing the resilient of our Slope Safety System for landslide hazard management of Hong Kong and to support its long-term sustainable development. In facing the extreme landslide hazards that may be brought about by climate change, it is neither practical nor cost-effective to rely solely on engineering solutions to manage the risk of extreme rainfall. Non-works ‘soft’ measures involving enhanced emergency preparedness, response and recovery should be in place in addition to the engineering approach.

The GEO landslide emergency system currently may not be properly geared up to deal with extreme rainfall events in terms of preparedness, response and recovery. Resilience to landslides under extreme rainfall conditions needs to be enhanced. Under the second extreme scenario (i.e. a more extreme rainfall scenario striking Hong Kong Island taking into account the effect of climate change), the current landslide emergency system will break down. This calls for a new strategy for managing the emergency, in partnership with Government bureaux and departments.

For enhanced slope safety preparedness and resilience to extreme rainfall events to take root, it has also to be embraced by the community. It is of the essence to engage the public for community support, education and preparedness and to promote self-protection and neighbourhood support. The population should be better informed about how the dangers of extreme weather can be avoided and the community can recover more quickly afterwards.

6 ACKNOWLEDGEMENTS

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The intrigue of creative design

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Keywords: cable-stayed bridges; suspension bridges; river crossings; railway bridges; design; construction; construction engineering; aerodynamics; wind tunnel testing; earthquake resistant design; seismic isolation.

ABSTRACT: The human thought processes involved in creative design are an intrigue. Decades on since his research into artificial intelligence in conceptual bridge design, the author describes case examples of landmark bridge projects, past and present, with special reference to design as an intellectual and practical endeavour. The paper illustrates the catalysts for innovation in bridge design; and adds to the debate on whether innovation is best engendered through evolution or by revolution. The paper also provides a glimpse of the future in bridge design, and the potential success which can be enjoyed by the engineering design profession.

1 INTRODUCTION

The human thought processes involved in creative design are an intrigue. Decades on since his research into artificial intelligence in conceptual bridge design, the author will describe case examples of landmark bridge projects, past and present, with special reference to design as an intellectual and practical endeavour. The paper illustrates the catalysts for innovation in bridge design; and adds to the debate on whether innovation is best engendered through evolution or by revolution. Innovation is often perceived as either a process of evolution or a revolution that shapes current thinking and steers future trends. Evolution arises from our desire to utilise past experience to perfect our art. However, if a teething problem is encountered, which defies a solution by existing techniques, and then new technologies will have to be invented. The landmark bridge projects presented in this paper have many vivid examples of innovation through evolution, and by revolution. The paper will also provide a glimpse of the future in bridge design, and the potential success which can be enjoyed by the engineering design profession.

2 THE SOUTHERN BRIDGE

With the rejuvenation of a former landfill site in Tseung Kwan O, new infrastructure links are being proposed to improve access to the area. One of these new connections is an aesthetically-striking long-span footbridge across the Eastern Channel, a waterway which is more than 100m wide and flows into Junk Bay in Hong Kong.

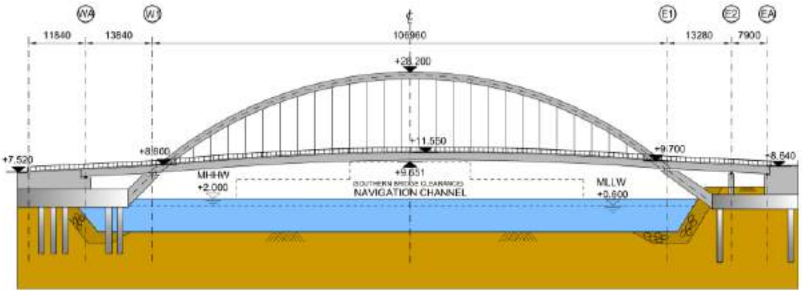


Figure 1 General arrangement of Southern Bridge

The history of the Southern Bridge dates back to 2002, when the Civil Engineering & Development Department of the Government of Hong Kong Special Administrative Region commissioned a feasibility study into further development of Tseung Kwan O. The study, which was completed in 2005, recommended provision of a strategic infrastructure to support the recreational development, including improvement of connections from the former landfill site to adjacent areas such as Tseung Kwan O town centre and other nearby developments.

To provide a catalyst for the land-based leisure facilities as well as water sport facilities, the government planned strategic infrastructure in the area, including improvements intended to enable the site to blend in with its surroundings.

Detailed studies were conducted on the visual interaction and aesthetic progression between the Southern Bridge, the Northern Bridge and the future Cross Bay Link. The latter is a dual two-lane highway over Junk Bay; it is approximately 1.8km, with a dramatic arch bridge flanked by marine viaducts, and connecting Tseung Kwan O – Lam Tin Tunnel to Wan Po Road at the southeastern part of Tseung Kwan O.

This family of bridges will be visible from many vantage points in Hong Kong, and they can create a cluster of structures adorning the landscape. For this reason there was a strong motivation to enhance the aesthetics of these landmark bridges. Different forms of arch and varying ratios of arch rise to span were examined, and perspectives of the three bridges and the landscape, from different vantage points were studied. The proposed footbridge had to reach high aesthetic standards, to be in harmony with neighbouring bridges and to demonstrate a theme of progression in this context.

The concept chosen focuses on a theme of waves, inspired by the fact that the Southern Bridge is at the mouth of the Eastern Channel and a curved bridge deck girder would represent waves from Junk Bay rolling into the channel. A wave-form in the bridge alignment creates a pleasing transition from the straight Eastern Channel to the curve of Junk Bay and offers some continuity to the surrounding landscape. Close to the bridge site, the eastern and western banks are used for recreational development and activities and a large waterfront park and leisure area for Tseung Kwan O New Town will also be developed. This design will enable the Southern Bridge to complement the leisure environment and romantic setting.

A range of feasible bridge schemes was rigorously appraised, and compared against a robust set of criteria, including (1) Cost Effectiveness, (2) Viability to Construct, (3) Viability to Operate and Maintain, (4) Aesthetics and Characteristics and (5) Sustainability (carbon footprint and impact to marine environment). The bridge schemes included splayed arches and arch forms with straight girders and girders curved-in-plan. A wave-form bridge scheme was judged to be superior to the other options in terms of its aesthetics, increased torsional stiffness, dynamic and aerodynamic performance, shorter construction period, minimal construction impact, ease of maintenance, and reduced construction and operation costs. So this was carried forward into detailed design in accordance with Eurocode and the Structures Design Manual for Highways and Railways 2013 Edition.

The structure selected is a long-span steel arch footbridge, with a single arch rib which crosses diagonally over a curved deck girder, and supports the deck girder via steel plate hangers. The evolution of the design has gone through improvements to the overall plan curvature of the bridge to enrich the distinctive character of the wave form; enhancements to the shapes of the arch ribs, hangers, and edge detail of the deck girder and to the integration detail of the bridge with its surroundings.

The wave-form bridge alignment also enables a smooth transition from the straight Eastern Channel to the curved shape of Junk Bay and creates a sense that the bridge is a harmonious continuation of the surrounding environment. In addition to addressing the structural adequacy, the shape and sizes of the main structural members were optimised to accomplish a slimmer and more elegant scale for a pedestrian bridge. The shape of the arch rib was enhanced from a 1.5m-diameter circular hollow section to a 1.4m-high diamond-shaped hollow section. The width of the steel hangers was reduced from 1000mm to 350mm through the use of high-strength steel, and the fascia of the edge girders was streamlined to enhance its visual appearance.

Figure 2 Hook-shaped steel plated hangers

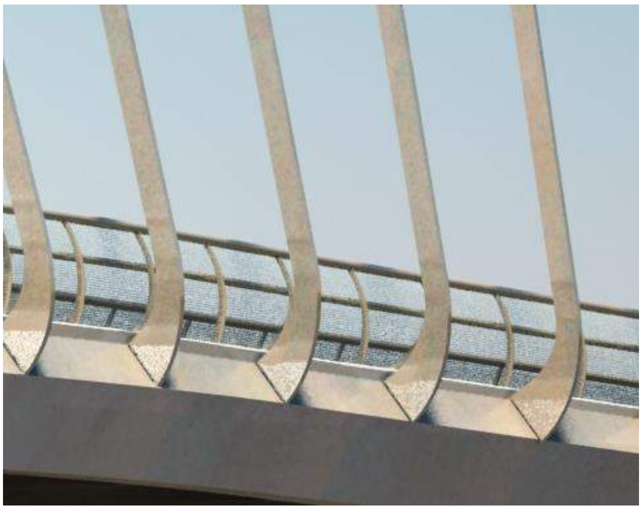


Figure 3 Streamlined fascia of deck edge girder

A single arch rib crosses diagonally over the deck girder, which on plan is designed to have a wavy shape. The curved deck girder is supported on its inner edge by steel plate hangers, which in turn transfer the loads to the arch rib. This structural system is well-balanced, with torsional and other effects being tackled efficiently.

Pedestrians on the bridge will enjoy a spacious environment and changing views. The hook-shaped hangers are intended to provide plenty of clearance at deck level, and the reverse-curvature of the girder creates a series of different views for pedestrians. The bridge geometry and spatial correlation of the structural members were configured to maximise views towards Junk Bay.

3 THE SECOND PENANG BRIDGE

The Second Penang Bridge was officially opened on 1st March 2014, celebrating the successful completion of the largest civil engineering project in the past 20 years in Southeast Asia and one of the world’s largest sea-crossing projects in recent history. The Second Penang Bridge is a 24km sea crossing linking the Penang Island and the Malaysia Peninsula. Southeast Asia is a region surrounded by aggressive volcanic and oceanic environments and is prone to natural disasters. This sea crossing is the longest in Southeast Asia, and it consists of precast segmental concrete marine viaducts in 55m span modules and an in situ concrete cable-stayed bridge 475m-long, founded on extensive marine piling.

The cable-stayed bridge over the main navigation channel is a prestressed concrete deck girder supported by two planes of cables. The span arrangement is 117.5m+240m+117.5m. The cable spacing is 6m at the deck end and a typical vertical spacing of 2.5m at the pylon end. The deck girder is cast monolithically into the pylon crossbeam, to gain advantages in the static and dynamic response of the frame system.

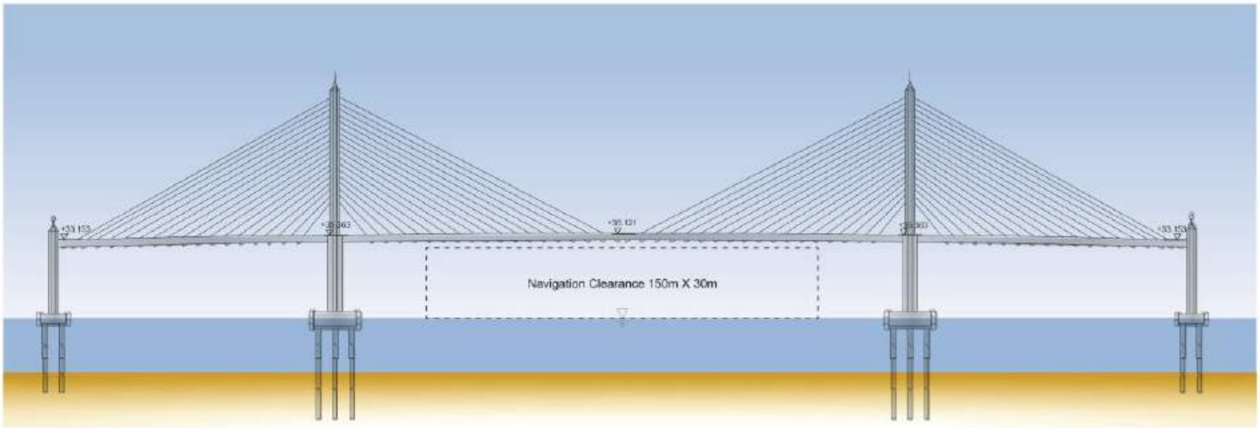


Figure 4 The Second Penang Bridge – cable-stayed bridge over the main navigation channel

3.1 *Integrated Cable-Stayed Bridge Design and Erection*

In this project, an integrated design and intelligent constructing engineering method was successfully implemented, and it succeeded in:

a) Establishing a methodology for the construction of the bridge which did not cause overstress to any part of the permanent bridge structure during construction
b) Safeguarding structural adequacy, integrity and stability throughout the erection stages including bridge completion
c) Achieving the reference states of the permanent works, including the permanent load geometry and force distribution defined by Contract Documents

Integrated design and intelligent bridge geometry control methods, already built into the optimized design, and implemented in construction were used in a prediction, survey, re-analysis and possible adjustment cycle in the bridge erection. It ensured that the final, target geometry of the completed structure was achieved without unacceptable locked-in stresses and that the structure had adequate strength and performance at all construction stages.

The combined permanent work design and intelligent construction engineering made possible a highly optimum design and fast-track construction, without reliance on temporary strengthening was achieved. The work advanced the fundamental understanding of cable-stayed bridges, and resolved a chronic problem in industry.

3.2 *Design and Construction of Marine Piling*

The entire marine piling operation posed significant risks to the project, but with careful planning, design execution, high precision and good quality were achieved within programme. The geological and hydrological conditions at the bridge site, together with marine piling in aggressive environments, thoroughly taxed pile design and installation technology.

In the shallow water zones, the standard penetration test values of the soil were high, 1.0m-diameter concrete spun piles could be adopted for the majority of these regions. On both sides of the main navigation channel, the water is deeper, the pile free length was long and the loading was high. This would result in a substantial increase in the number of piles. If 1.0m-diameter concrete spun piles were used under these conditions involving 32 piles (with a common pile cap for the two piers at each support location). The minimum pile spacing of 3 x diameter governed the size of pile cap. Therefore, consideration of structural design optimization led to the use of 1.6m-diameter tubular steel piles for foundation works at these locations, with discrete pile caps. The tubular steel piles saw the most extensive use of cathodic protection in Southeast Asia

3.3 *Precast Concrete Hollow Spun Piles in Bridge Construction*

The Second Penang Bridge project saw the most extensive use of precast concrete hollow spun piles in bridgework construction. Intense efforts were expended on design optimization, and development of specifications to clearly define the design intent, quality control in production, shipment, and every stage of installation. The bridgework involved was a major sea crossing in an aggressive marine environment, and a large-scale piling operation successfully installed a record number of spun piles totaling 5,168. The body of knowledge acquired in this project on precast hollow concrete spun pile technology and field operation has been substantial and comprehensive. Strict quality control was applied to the pile manufacture and pile driving. After the trials at the initial stage of the project, the joint detail between spun pile sections was improved, and further drivability analysis for hammer and pile shoe selection. In field action, the damage rate in driving the precast concrete spun piles was kept very low. For durability saline coating was applied to the pile shafts.

3.4 *Large Diameter Bored Piles*

The cable-stayed bridge over the main navigation channel and the marine viaducts near the Peninsula are founded on bored piles socketted into bed rock at depths over 100m below water. The bored piles for the main cable- stayed bridge pylons are amongst the longest attempted; their design lengths being 103m to 127m. Given the water depth at the main channel, the design construction and quality control of the foundation piles posed significant challenges. The installation of bored piles consisted of drilling, reinforcement cage installation and concreting. Bentonite was tested during the drilling. After drilling, the verticality, diameter and settlement of the drilled bore hole was inspected by the Supervision Consultant. Upon completion of the bored pile installation, all piles were tested with cross-hole sonic logging to verify the integrity of concrete. Four cross-hole sonic logging tubes were inserted in the pile together with the reinforcement cage. Six tests (2 cross + 4 adjacent) with four classifications were conducted on each bored pile from the cross- hole sonic logging, Statnamic Test was conducted on the bored piles to verify the bearing capacity.

(a) (b) (c)

Figures 5 (a) Tubular steel pile driving; (b) Precast concrete hollow spun pile driving and (c) In situ concrete bored pile installation

3.5 *Bridge Aerodynamics and Wind Tunnel Testing*

A carefully planned aerodynamic investigation of the main cable-stayed bridge consisted of static and dynamic section model testing to measure the static loads and the dynamic response of the bridge deck; aeroelastic model testing of a freestanding bridge pylon, together with aeroelastic bridge model testing of two critical erection stages.

The section model testing was carried out in the SWJTU-1 wind tunnel at Southwest Jiaotong University in China, at 1:50 scale. The aeroelastic bridge pylon model testing and aeroelatic bridge model testing were conducted respectively at 1: 40 and 1:60 scale, at the 22.5m-wide, 4.5m-high and 36m-long SWJUTU-3 wind tunnel, which is the world's state-of-the-art and largest boundary layer wind tunnel in operation.

(a)

(b)

Figures 6 (a) Aeroelastic bridge model testing and (b) Aeroelastic bridge pylon model testing

3.6 *Navigation Risk Assessment*

The marine viaducts traverse the Penang Strait in primarily shallow water, with some areas of inter-tidal flat. The main navigation channel is located to the east of the crossing adjacent to Penang Island and this is spanned by the cable-stayed bridge. For safety, reliability and economy a navigation risk study was conducted to examine the risk exposure to the principal bridge elements (main span and approach piers). Further assessment, to a high level of sophistication, was made to more rigorously appraise the risks involved and to pursue further design optimization. The risk to piers located further away from the main navigation channel was scrutinized, taking into full account the influences of water depths, tidal levels and current movements, and marine traffic, to determine appropriate ship impact probabilities and forces. The additional work verified that resistance inherent in the marine viaduct piers was more than adequate. A number of ship impact protection measures were studies - energy dissipation facilities such as steel fender systems; artificial islands; fender piles, sand and injection shell cofferdams, and floating buoys. After a careful study of the project constraint, technical feasibility and sustainability, a steel fender system was adopted for ship impact protection of the main bridge.

3.7 *Tsunami and Resulting Liquefaction Phenomena*

The tsunami waves study was based on a simulation of the tsunami wave generation and propagation hydrodynamic model. The simulation was then calibrated with records from the poignant tsunami event of 26 December 2004, which inundated Pulau Langkawi and the northwestern coasts of Peninsular Malaysia. Additional tsunami scenarios were simulated using historical records of seismic activities in the Andama Sea where there is a high probability of tsunami generation into the Melaka Straits and the Second Penang Bridge site. The simulation results were expressed in time-series wave propagation, maximum wave height distribution and arrival time of maximum tsunami waves. The results showed that a maximum wave of 2.17m was produced by the Year 2004 tsunami, affecting the Batu Maung end of the Bridge with an arrival time of 5 hours after the tsunami generation.

The bridge design took full account of this predicted wave height. Potential soil liquefaction resulting from a tsunami was investigated. Several soil strata consisting of saturated silty and sandy constituents are potentially prone to liquefaction. An analysis was conducted on selected borehole samples for different types of soil with the lowest standard penetration test values, representing the worst case scenarios for potential liquefaction. The liquefaction potential along, and in the proximity of the bridge alignment were found to be generally less that 50% for earthquakes with magnitudes less than seven with a maximum acceleration of 0.1g at a 500-year return period.

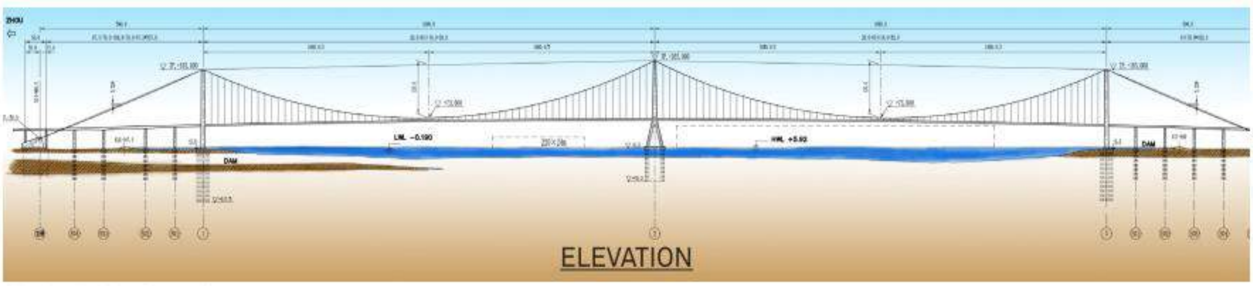
3.8 *Seismic Isolation*

The design championed the theory and practice of seismic isolation to achieve resilience, rather than rely on mere provision of strength. Seismic isolation in bridge structures can be achieved through the use of many types of seismic device. Through their installation - the seismic isolation devices alter the bridge's fundamental resonant frequency and shift it away from earthquake ground motion frequencies. In this project seismic isolation was achieved by the use of high damping natural rubber bearings in the marine viaducts, altogether some 1,400 in number. The earthquake intensities, the vertical loadings, the range of displacements involved, and the cost-competitiveness of rubber bearings in Malaysia, pointed to the use of high damping natural rubber bearings for seismic isolation for the viaducts. After further investigation, this type of bearing was adopted for seismic isolation between the marine viaduct superstructure and the viaduct piers.

4 TAIZHOU BRIDGE

The Taizhou Bridge, which was opened to traffic in November 2012, epitomizes the pursuit of excellence through vision and creativity. The bridge is a pivotal element of infrastructure in the east of China and will play a vital role as a link in the freeway network in Jiangsu Province and the Yangtze Delta region. The bridge is part of a new USD 1.5 billion, and 62km-long freeway that connects the cities of Taizhou, Yangzhou, Zhenjiang and Changzhou in Jiangsu Province. The freeway network will serve as a catalyst for further economic growth in the eastern part of China.

The main crossing over the Yangtze River is a spectacular suspension bridge of three pylons, and two continuous 1,080m main spans. It carries dual-six traffic lanes, with a deck width of 33m. The central pylon is of an inverted-Y shape on elevation, 200m tall, constructed in steel, and founded on a 58m by 44m caisson. The pylons at each end of the main bridge spans are concrete frame structures, 180m tall, and each founded on 46 number 2.8m diameter friction piles. At the bridge site, the river is some 2.1km wide and the riverbed has channels at each side, with the deeper channel on the south side at a water depth of some 30m, and reducing to some 17m towards the central region of the river. The channel therefore has a w-shaped cross section at the bridge alignment. A three-pylon, two-main span suspension bridge scheme was chosen for environmental reasons, as a means of minimizing impact on the river hydraulics and ecology through reducing the number of bridge piers in the water while providing for two main navigation channels to facilitate ship movements and encourage the development of port facilities in the region. The scheme is also cost-competitive given the site constraints and project conditions. All elements of the planning, design and construction have been a determined effort to minimizing the ecological impact on the environs.



Taizhou Bridge General Arrangement

Figure 7 Taizhou Bridge general arrangement

The three-pylon, two-main span integral suspension bridge system enabled not only a breakthrough in spanning over large distances, but also the beginning of a new generation of multiple-span continuous ultra-long-span bridges for conquering difficult terrains and obstacles.

4.1 *Daunting Challenges*

The project posed numerous daunting challenges, including:

a) This is the world's first attempt to design and construct such a long-span, three-pylon suspension bridge. Very substantial research and developments efforts were absolutely necessary to ensure safety and performance.
b) The structural behavior of a three-pylon, continuous suspension bridge system is different from that of a conventional two-pylon suspension bridge system. The design must ensure no cable slip occurs over the cable saddles under all loading conditions, in order to prevent collapse. Conflicting demands on the central pylon stiffness therefore arise – a flexible central pylon helps prevent cable slip but is ineffective in the control of girder deflection; a stiff central pylon renders it difficult to help prevent cable slip, although it improves on deflection control of the girder.
c) The caisson foundation of the central pylon is located in the middle of the 2.1km-wide river and founded at some 70m deep in the riverbed. The conditions therefore led to the deepest underwater bridge caisson construction. This warranted a sophisticated control system to mitigate oscillations induced by wind and current; together with a high-precision positioning system, for caisson construction.
d) The central pylon is of steel construction and, the control of stresses demanded a very substantial quantity of thick steel plates; some 60 percent of which were of thickness 50mm-60mm. The distribution of weld seams, the torsional effects induced by welding and the geometric complexity rendered the control of welding distortion and geometry control very difficult.
e) Cable anchorages - at the bridge site, bedrock is at some 190m deep with no impermeable bearing layer of soil above. The north and south cable anchorages were constructed as combined systems of gravity anchorages with caisson foundations measuring 68m x 52m in plan; and 57m and 41m deep respectively in the north and south caissons.
f) The superstructure construction scheme for a three-pylon suspension bridge system is much more complicated than that for a two-pylon suspension bridge system, particularly in the main cable erection, main girder erection and bridge geometry control.

To tackle the unprecedented challenges, substantial research, development and investigation were conducted to derive techniques for design, fabrication, welding, construction and geometry control.

4.2 *The First Attempt to Create a Long-span, Three-pylon and Two-main-span (2 x 1,080 metres) Continuous Suspension Bridge*

Under the most adverse loading conditions with full live load on one main span, and none on the other, the deflections of the deck must be controlled to within limits. The friction between the main cables and the saddles must be sufficient to prevent cable slip. Load testing was carried out on the resistanace against cable slip on the saddles. While meeting these two criteria, the stability and structural adequacy of the central pylon must also be preserved. Conflicting demands on the central pylon stiffness therefore arise – a flexible central pylon helps prevent cable slip but is ineffective in the control of girder deflections; a stiff central pylon renders it difficult to help prevent cable slip, although it improves on deflection control of the girder. Three pylon types were investigated – an inverted Y-frame, an A-frame and an I-shape. The inverted-Y pylon scheme was found to offer adequate resistance against cable slip and sufficient control of stresses and deflections. Further investigations and optimizations were carried out, to determine an acceptable longitudinal stiffness in the pylon, through exploring different controlling parameters, including the pylon height, cross sectional dimensions, the longitudinal separation between the pylon feet, and the height at which the pylon legs join together. After exhaustive investigations, the final solution evolved as a longitudinal inverted Y-frame, and transverse portal-frame type pylon of steel construction.

4.3 *The Steel Central Pylon with a Height of 200 Metres*

The central pylon was fabricated in chamfered steel segments with a rectangular cross section; the maximum segment dimensions being 5m by 12.7m and the maximum segment height being 15m. Geometry control and stress criteria dictated the dimensional tolerances to - longitudinal and transverse directions (+2mm), diagonal (3mm), flatness (0.25mm); together with other stringent tolerances. The demands for high-precision imposed significant challenges to the control of welding deformations, marking positioning, mechanical processing and measurement. Since some of these segments were 15m in length, shop assembly of segments could involve 30m of module. To ensure safety and to mitigate temperature effects, pre-assembly of two adjacent pylon segments horizontally, assisted by computer simulation, was adopted. For geometry control, a technique based on advanced survey, digital alignment, efficient state control, and a normal processing machine, was developed after extensive research, for use in the fabrication process.

4.4 *The Deep Underwater Caisson Foundation for the Central Pylon*

The caisson has cross sectional dimensions of 58m by 44m, an overall height of 76m and divided into two halves each 38m high. The lower part is a prefabricated rectangular steel shell and the upper part is a concrete caisson. The first part of the steel shell was fabricated onshore, and then floated to the bridge site where construction continued to its 38m height. The steel shell was precisely set out at its permanent location by position control system, where it was sunk to the riverbed by flooding it with water. An overall accuracy within 500mm in plan, 1/150 gradient and 1 degree torsional angle was imposed. Once in position, the chambers of the steel shell were filled with concrete. Under water concrete casting was used to complete the central pylon foundation construction.

4.5 *The Erection of the Long Suspension Cable with two 3,110-metre-long Main Cables*

Each main suspension cable consists of 184 prefabricated parallel wire strands, with each strand consisting of 91 high-strength galvanized 52mm-diameter steel wires with a minimum tensile strength of 1,670MPa. The unstressed length of each strand was about 3.1km and its weight was 47-tonne. Inside the main cable, the void ratio was 18%, while outside the cable clip it was 20%. The coefficient of friction between the main cable strands and the cable saddle was 0.2, with a factor of safety of at least 2 against cable slip. Suspender cables were made of prefabricated parallel high-strength, 5mm-diameter steel wires, with a minimum tensile strength of 1,670MPa. Suspenders cables have a typical longitudinal spacing of 16m, and the pylon centreline is 20m from the nearest suspender cable.

4.6 *The Concurrent Erection of Two Long Suspended Deck Girders, in an Integral Suspension System*

Research was undertaken into the type of catwalk that would be most suitable for the construction of a three-pylon continuous suspension system, and it concluded in a continuous catwalk with four spans being used. Investigations were also conducted to explore options in either erecting the deck girder segments from the centre of a span or from the central pylon symmetrically outwards. The evaluation criteria included deflection of the central pylon, safety factors against cable slip, the need to limit variations of the forces in suspenders to a minimum, and also the objective to limit the shear forces and axial forces in the main girder to a minimum. Against this set of criteria, detailed work led to the decision to erect girder segments from the middle of the span outwards.

5 PADMA BRIDGE

The 6.15km-long Padma road and rail bridge will become a landmark structure in Bangladesh and one of the largest river crossings in the world. The design encountered significant engineering challenges, particularly from the hostile site conditions and the merciless forces of nature. During the monsoon season the Padma River becomes fast-flowing, and is susceptible to deep scour, demanding deep piled foundations. The bridge site is also in an area of considerable seismic activity, leading to significant seismic loads being exerted on the structure. In the design, extensive engineering studies were conducted, advanced computational analyses were employed and innovative engineering solutions were developed, to ensure that the bridge will be able to survive the challenges of nature in its long design working life. The project has accumulated a significant body of knowledge in seismic resilient and scour tolerant design, and it has advanced our understanding of bridge behaviour in conditions of severe earthquake and deep riverbed scour.

5.1 *Severe Constraints on the Initial Scheme*

In previous studies for the bridge a number of options for the bridge form had been examined, with a single-level extradosed bridge with spans of 180m being proposed. Detailed investigation of this bridge form was conducted through extensive finite element modeling.

Figure 8 Padma Bridge – a two-level rail-road bridge

The extradosed bridge investigated consisted of a concrete box girder superstructure, supported by stay cables in order to reduce the girder depth. The freight railway however posed very tight tolerances on displacement and rotation, and in attempting to meet these tolerances the girder would need to be stiffened, reducing the benefit of the stay cables and increasing the weight of the deck. This conflicted with another constraint - with poor ground conditions and onerous loading combinations, it was imperative to minimize the loads transferred to the foundations, to curb the cost escalation in foundation construction, which already constituted a high proportion of the overall cost of the bridge. Because of these severe constraints the extradosed bridge was found not to be the preferred option. Alternative concrete superstructure forms were investigated. Three examples are shown in Table 1, an extradosed concrete truss bridge, a concrete girder bridge and a steel truss bridge. In each case a two-level structure was proposed because it has significant advantages over the single level structure:

a) Separate highway and railway envelopes enable improved operation, inspection and emergency evacuation for the bridge.

b) The maximum permissible gradient on the railway is 0.5%, requiring long lengths of approach viaduct for the railway to descend to ground level. By reducing the structural depth beneath the railway (in a two-level structure the railway runs inside the structural cross section), the length of the railway approach viaducts can be minimized.
c) Construction cost – a two-level structure is more efficient, with a much reduced overall width of the structure.

Table 1 Comparison of alternative bridge forms

		Advantages	Disadvantages
	Extradosed concrete truss bridge	A truss structure enables significant weight savings over concrete box girder schemes Use of cable supports will increase potential span lengths	Truss connection nodes would be difficult to construct, leading to longer construction period and additional costs Heavier than steel deck girder schemes
	Twin-box concrete girder with the railway carried by a perforated beam-and- slab system spanning transversely between the boxes	A straightforward erection method, similar to that used in other major bridges in Bangladesh	A heavy girder leads to increased demand on the foundations and limits the span lengths. Increased costs for foundations and deck girder owing to additional weight An enclosed railway is a potential safety hazard
	Steel truss bridge with composite concrete top slab	A steel truss is the lightest girder option, leading to a reduced number of piles and lowest overall cost The truss is relatively rigid and does not deflect excessively under railway loading	The steelwork will require repainting at regular intervals

Analytical models were developed for each of the bridge forms to determine member sizes and in particular the weight of the superstructure. The steel truss bridge was found to be the most efficient with the lightest deck. Further studies were conducted on this scheme to determine the optimum span length. Overall deck weight and foundation loads were compared for three span length modules: 120m, 150m and 180m. From this data a construction cost was estimated for each span length module, and the optimum span length was found to be 150m.

The conclusion of the studies carried out on the bridge deck, was that the steel truss bridge with a concrete top slab acting compositely with the truss, would be the most economic and suitable bridge form for the river crossing. The fundamental principles are that this structural form has a relatively high stiffness to mass ratio and it therefore has advantages in (i) the control of deflections and instantaneous longitudinal gradient under the freight railway loading, and in (ii) its seismic performance, by virtue of the reduced sprung mass to be carried by the pier-and-piled foundation system in the event of an earthquake. This two-level, combined rail-road bridge scheme, comprising of a steel truss superstructure acting compositely with a concrete roadway slab, was adopted for the detailed design. The detailed design was completed in 2010, and the construction contract was awarded in June 2014, marking a significant milestone in the history of modern bridge engineering.

6 A GLIMPSE OF THE FUTURE - CROSS BAY LINK BRIDGE

Figure 9 Cross Bay Link Bridge

The Cross Bay Link is a 1.8-kilometre long sea link carrying a dual two-lane highway, a footpath and a cycle track across the Junk Bay, connecting the Tseung Kwan O – Lam Tin Tunnel to Wan Po Road near Area 86 of Tseung Kwan O.

The bridge is located at one of the most scenic settings in the world, in full view of the famous landmark Victoria Harbour. The most striking icon of the Cross Bay Link will be the Eternity Arch, a unique 400-metre long steel arch bridge over the main navigation channel.

The aesthetics design evolves from a theme of "timeless elegance, diamonds are forever". Creative design seamlessly fuses with engineering fundamentals to fulfil our social responsibility, creating a unique bridge that serves the community. The design of the Cross Bay Link is in progress, and the intrigue of its creative design will be published in the near future.

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a) Civil Engineering & Development Department of the Government of Hong Kong Special Administrative Region (the Southern Bridge and the Cross Bay Link).
b) Jambatan Kedua Sdn. Bhd. Malaysia (the Second Penang Bridge).
c) China Harbour Engineering Co. Ltd., China (the Second Penang Bridge).
d) Jiangsu Provincial Yangtze River Highway Bridge Construction Commanding Department, China (the Taizhou Bridge).
e) Bangladesh Bridge Authority, Bangladesh (the Padma Bridge).

A Sustainability Construction Scorecard for Hong Kong

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Keywords: sustainability; scorecard; construction; projects; resources; process; machinery; people; value.

ABSTRACT: Typically, the performance of construction projects is rated by how well cost, scope, time and outputs are managed. Often though, the delivery of construction projects is complex involving machinery, people and processes. It is also a requirement to make sure that resources such as energy, materials and water are used efficiently and sustainably. In addition, the value that the project contributes not just to the immediately affected community but to society and the gaining of valuable knowledge should be taken into account as well. The objective of the Sustainability Construction Scorecard is to score the performance of a project from a holistic point of view using the 5 parameters of process, machinery, people, resources and value. Each parameter has a set of related aspects, which are individually assessed in terms of three performance levels. The scorecard serves to set a baseline for measuring project performance, guidelines on performance improvement, and KPIs to measure performance achievements so that projects of a similar nature can be benchmarked. The scorecard further accommodates the range of users who would use the results for different operational and strategic purposes. This paper sets out the scorecard, explains the different parameters and provides the preliminary findings of its application to a series of case studies in Hong Kong. The results are examined together with user feedback on the wider applicability of the tool.

1 INTRODUCTION

In the construction sector, cost, scope, time and deliverables are crucial factors for the successful completion of a project. Measuring these factors tells one how well the project has been delivered and these are often the parameters used to gauge the performance of contractors or consultants but not the project itself. Consideration of the energy, materials and resources used in the project is a useful way to show how efficiently the project has been managed, with environmental benefits as well. However the sustainability performance of the project in terms of its wider impacts on the community and society at large are still not addressed, nor are other aspects such as how much value the project has contributed to economic progress or the capturing of key knowledge for future projects. The use of a scorecard, in this case, a Sustainability Construction Scorecard, that rates the performance of a project from the viewpoint of different stakeholders is an alternative way of assessing construction projects.

2 BACKGROUND REVIEW

The idea of a scorecard as a management tool is not new. Kaplan and Norton (1996) were the pioneers of the Balanced Scorecard, which measured organizations according to the parameters of financial, customers, internal business processes and learning & growth. Companies were able to use this tool as a means of setting strategic objectives and cascading the latter down to operations using the same parameters to come up with actionable plans based on agreed targets aligned with the company's strategic goals. Managers were assessed on their ability to achieve personal targets prescribed on their individual balanced scorecards according to relevant key performance indicators (KPIs).

The adoption of the scorecard concept at an industry level, on the other hand, has not been widespread. Manufacturing industries have used it to measure productivity and the hospitality industries have applied it to measure service quality, but in general, the scorecard has remained largely a management tool rather than a means of serving an industry's development.

Hence, it is no surprise to find that the scorecard approach has not been widely applied in the construction sector. Assessment methods generally include, but are not limited to profitability, schedule, quality and safety and relate more to the performance of a contractor rather than the project itself. However, there have been a handful of cases where this approach has been used for a variety of purposes. Based on a statistical analysis, the US Construction Industry Institute (2003) developed a scorecard to assess a project owner's level of involvement in safety issues and tested this scorecard on a range of projects. It was concluded that better project safety performance could be accomplished when owners set safety objectives, select safe contractors, and participate in safety management during construction. Constructing Excellence (2006) in the UK established a scorecard that applies ten environmental KPIs for construction projects in achieving excellence. Tennant and Langford (2011) extended this idea by combining the Constructing Excellence KPIs with the four inter-related business perspectives advocated by Kaplan and Norton within a performance measurement framework. Their study found that the application of a construction project scorecard aligned with the established practice of performance goal-setting facilitated the introduction of project team-based reward initiatives and supported project monitoring and control practices. Additional benefits were realized in the form of organisational learning and benchmarking.

In Hong Kong, the construction industry is a major sector dominating much of the city's economic growth. Whilst the adoption of construction methods and techniques in Hong Kong have resulted in many iconic and world-class buildings and infrastructure, there have been concerns raised within the industry on the quality of the work and workforce productivity set against a backdrop of pressing contract deadlines. Construction quality and productivity problems are commonly attributed to the performance of contractors and subcontractors entrusted to complete the actual works and means of assessing the performance of these parties are invaluable for maintaining project quality and timely outputs. Ng (2007) used a balanced scorecard model to appraise contractor performance to achieve reliable and practical means for performance evaluation. Tam and Khoa (2007) adopted a systematic approach in assessing environmental performance in the construction industry setting performance against environmental operational indicators and correlating input (environmental site planning, energy consumption and maintenance of equipment) with output (waste generation and accident rate) measures using survey and interview methods in Hong Kong and Australia. Liu and Wan (2004) linked traditional quality concepts to the financial performance of construction projects by linking Quality Management Systems to a balanced scorecard using the latter as a performance measurement tool to complement the achievement of quality objectives.

In recent years, sustainability has become a further driver for project performance both during the construction phase and post completion in terms of the social, environmental and economic impacts as arising from the project. The Advanced Performance Institute provided the idea that a 'scorecard is increasingly being seen as a way to promote, measure and profit from sustainable development' (Lin and Khoa, 2007). Butler et al. (2011) suggested that scorecards should include four perspectives: sustainability, stakeholders, processes and learning. The sustainability perspective emphasizes the triple bottom line of economic prosperity, environmental quality, and social justice. The stakeholder perspective incorporates measures of business ethics, labour practices, and impact on society; the processes perspective focuses on specific organizational internal and external processes, products, tools, and systems; and the learning perspective stresses organizational synergy, training, and research and development. In addition, any framework for measuring success in sustainability must have performance metrics that can be clearly understood and communicated, must add value and be integrated into the main value-adding systems of the organization as well as being extensively supported by existing management tools and resources.

3 A SUSTAINABILITY CONSTRUCTION SCORECARD FOR HONG KONG

The assumptions used in the development of Sustainability Construction Scorecard (SCS) include:

a) Traditional project processes can be enhanced through efficiency and best practices;
b) Advances in technology and automation would change the use of on-site machinery and plant;
c) New materials and lean construction practices will change the way resources are used on-site;
d) Application of human resources practices like communication, training and empowerment can improve people performance; and
e) The intrinsic value of the project can be shown as increased economic, environmental and societal value.

The SCS proposed in this paper comprises five parameters as follows:

a) Processes;
b) Machinery;
c) People;
d) Resources; and
e) Value.

The objective of the SCS is to score the performance of a project from a holistic point of view using these five parameters. Each parameter has a set of related aspects, which are individually assessed according to three performance levels. The scorecard serves to establish a baseline for measuring project performance, guidelines on performance improvement and KPIs with which to measure performance achievements so that projects of a similar nature can be benchmarked. The scorecard further accommodates the range of users who would use the results for different operational and strategic purposes. Examples of the scoring criteria are given below.

3.1 *Processes*

Processes on site broadly encompass traditional project management processes like managing the schedule and maintaining project quality. The scorecard should not attempt to replace conventional project management tools like project plans and procedures but should look at improvements in outputs if certain performance criteria are met. In addition to schedule and quality, decision making is a crucial process and the scorecard should indicate whether the right level of time is being taken to reach a certain decision. The aim of this is to reduce layers of bureaucracy and increase project efficiency. Safety and environmental management are added as the other critical processes. In all cases, the KPIs are only useful if the baseline is known.

3.2 *Machinery*

Site machinery tends to be specific according to the nature of the project. For instance, a slope stability project would be quite different in terms of machinery used compared with site formation for a major public works. Hence the listing of machinery must be adapted for each type of construction project. A sample is given below. As before, the KPIs are only useful if the baseline is known.

3.3 *People*

There are several stakeholders involved in a construction project. Each party has a different contribution towards the success of the project. For example, a client needs to give clear direction for all the parties to deliver the project, the site management and contractors need to work in harmony according to the project program and affected stakeholders must be engaged together with the general public to give the project the level of approval and acceptance to meet social and - sometimes - political goals. A sample is given below. As before, the KPIs are only useful if the baseline is known.

Table 1 Performance level criteria for processes

	Level 1	Level 2	Level 3	KPIs
Managing schedule	Overrun	On-time	Ahead of schedule	% days saved/lost
Managing quality	Poor quality	As stated in specification	Added value	% reduction in non-compliances
Decision making	High. bureaucracy (centralized)	Low bureaucacy (delegation of authority)	Flat (effective management system)	% time saved
Managing safety	Business as usual	Safety management system in place	Robust safety culture	% accidents & incidents reduced
Managing environment	Business as usual	Environmental management system in place	High environmental awareness	% environmental improvements

Table 2 Performance level criteria for machinery

	Level 1	Level 2	Level 3	KPIs
Lifting-crane	Business as usual	Well maintained equipment with minor enhancements	Latest lifting-crane machinery with enhanced lifting capability	% time saved % labour saved,
Grout mixer	Manual methods	Experienced manual operators	Automation with data logger	% time saved % labour saved
Air-compressors	Business as usual	Well maintained equipment	New model of high performance air-compressors	% time saved % labour saved
Pile drivers	Business as usual	Well maintained equipment	Latest machinery e.g. concentric drilling with casing	% time saved % labour saved
Scaffolding	Business as usual	Minor improvements	Better scaffolding connections	% time saved % labour saved

Table 3 Performance level criteria for people

	Level 1	Level 2	Level 3	KPIs
Client	Business as usual	Minor improvements in communication and reporting with project team	High levels of communication and direction with project team	% client performance score (rated by other stakeholders
Site management	Business as usual	Minor improvements in training	High levels of training & communication	% consultant performance score (rated by client)
Contractors	Business as usual	Minor improvements in training	High levels of training & communication	% contractor performance scores (rated by site management)
Affected stakeholders (local communities)	Business as usual	Minor improvements in engagement (carry out surveys)	Highly engaged (carry out surveys and town halls, proactively seek views and discuss)	% reduced no. objections
General public	Business as usual	Minor improvements in education and consultation (active efforts to reduce disruption)	High public awareness and acceptance Very responsive to complaints	% reduced no. complaints % increase in compliments

3.4 *Resources*

Sustainability, as stated earlier, is a key outcome for the project. To achieve this, resources must be managed carefully to ensure efficiency and avoid wastage. In most cases, this makes sense to reduce project costs. However, where pricing of externalities does not cover loss of resources, wasteful behaviour still persists e.g., wasting water or leaving generators on when not necessary because water and fuel bills respectively do not reflect the true cost of the resource. The adoption of clean technologies like solar or biofuel also takes a lesser priority if payback periods are not immediate enough. A sample table is given below. As before, the KPIs are only useful once the baseline is established.

Table 4 Performance level criteria for resources

	Level 1	Level 2	Level 3	KPIs
Building materials	Business as usual (on-site assembly)	Use of prefabricated or precast structures	Lean construction	% reduction in use of materials
Recycled construction waste (wood, concrete, plastics, glass, spoil)	Business as usual (no waste recovered – sent off to landfill)	Waste recovered and taken off-site for recycling	Waste recovered and reused on site	% recycled construction waste (%reduced off-site trips)
Energy - oil	Business as usual – standard grade diesel	Use of bio-fuel blended fuel	Use of renewable energy e.g. solar cells	% fossil fuels reduced
Energy - electricity	Business as usual – use of mains electricity	Energy efficiency and good site practice	Use of renewable energy e.g. solar cells	% fossil fuels reduced
Water	Business as usual – no specific water initiatives	Collect and recycle water (e.g. grey water)	Water conservation measures (low use appliances)	% water volume reduction

3.5 *Value*

Ultimately, the project's success must be measured according to the amount of value delivered to society and the environment. Another intrinsic value of the project is when the construction knowledge captured and the number of innovative ideas increases on an industry level. A sample table is given below. As before, the KPIs are only useful if the baseline is known.

Table 5 Performance level criteria for resources

	Level 1	Level 2	Level 3	KPIs
Design excellence	Business as usual allow for rework costs	Follow design so that project sum within budget	Incorporate innovative ideas to achieve project savings	% savings through design
Local community	Business as usual – no added value to assets	Increase in value of assets through better functionality	Sustainable asset value – assets are lasting and durable (e.g. climate resilience)	% increase in value of assets, quality of life
Knowledge management	Business as usual – no lessons learnt	Some new ideas adopted and recorded	Active KM program showing lessons learnt plus innovative ideas	% reduction in error costs
Organizational development	Business as usual – organization does not develop	Improvements in capability	Organization improves as a learning organization	% increase in managerial/ leadership capability
Societal	Business as usual	Minor contribution to HK society	Major contribution to HK society	% GDP increase, quality of life

4 CASE STUDY

The close proximity of steep slopes to buildings and infrastructure in Hong Kong, coupled with seasonal torrential rainfall, inevitably brings with it the risk of casualties due to landslides. The Landslip Preventive Measures Programme focuses on substandard man-made slopes that pose the greatest landslide risk in Hong Kong. Consultants are engaged in landslip prevention and mitigation engineering studies to upgrade Government man-made slopes together with mitigation measures for natural hillside catchments.

The SCS was tested on five slope projects in Hong Kong to assess the applicability of the scorecard. The rankings of 1, 2 and 3 were assigned scores of 5, 10 and 15 marks respectively. Although empirical in nature, responses from the project teams yielded a series of interesting results as below:

Table 6 Scorecard results

	Site 1	Site 2	Site 3	Site 4	Average
Process	67%	53%	73%	67%	65%
Machinery	47%	33%	27%	33%	35%
People	87%	33%	87%	60%	67%
Resources	60%	53%	40%	47%	50%
Value	40%	40%	53%	67%	50%

From the findings, the project teams interviewed expressed satisfactory levels of confidence regarding site processes and people management aspects. This is in part due to project management practices which are well established and monitored in Hong Kong government projects. In addition, government has its own rating systems on which to base performance levels in the industry. Site 2 however scored a low result on ‘People’, due to the complexity of this particular project, which involved several stakeholders in the community and different agencies. Stronger engagement practices would help this project improve its score.

The lowest scores were observed on ‘Machinery’. This could be due to the limitations to test new technologies or contract constraints in doing so on projects. In most cases though, it should be noted that clients do actually want innovative ideas to save costs and increase efficiencies. To facilitate this, a good practice is to run innovation workshops at the outset of projects. This should be done on a regular basis to maximize the benefits to the project and to score higher on this parameter. Notably for instance for soil nail drilling, novel ideas like concentric drilling or reverse circulation drilling techniques could be employed instead of conventional drilling techniques. In addition, it should be pointed out that most of the site responses to improvement towards level 2 in ‘Machinery’ only extend to better administration practices and contractual amendments. The greater improvements will come in the form of process automation, which only comes with higher levels of cost investment.

In all cases, management of resources and the actual value delivered by the project came out average i.e. these met basic requirements rather than aspired to achieve more. ‘Value’ can be increased by better knowledge capture at the end of the project; however project closure is not always done as systematically as it should be.

Other comments from participants of this exercise included:

a) Absence of a baseline to carry out an accurate assessment;
b) Lack of incentives for improvement;
c) The large difference between levels in order to make the higher score (in most cases, government policy encouraged the use of biodiesel and recycling of construction waste so the transition from 1 to 2 was easy, but not to level 3);
d) Lack of consistent guidelines in measuring (many of the indicators proved difficult and difficult to measure).

For ease of use, it was suggested that the indicators should be best prescribed in more practical measurements e.g. number of skips to landfill or number of non-compliances to the contract. This enables an easier grasp of measurable data and would ease user acceptance.

5 CONCLUSION

The authors have developed a Sustainability Construction Scorecard that attempts to score the performance of a project from a holistic point of view using five parameters: process; machinery; people; resources; and value. Each parameter has a set of related aspects, which are individually assessed according to three performance levels using established baselines.

The main differences between the SCS and other scorecards are:

a) The SCS takes a holistic view of the project performance both during execution and the longer lasting effects as well as from different perspectives i.e. client, contractor, site management and community;
b) SCS is simple and user-friendly, and it is easy to apply;
c) Using the SCS, it is intended to make the project easy to monitor; and
d) The wider society at large is considered.

The scorecard provides KPIs to measure performance so that projects of a similar nature can be benchmarked. The usefulness of the scorecard will be determined in the end by users. The final table below explains what users will get out of the scorecard:

Table 7 What do users get out of the Scorecard

	How well is the project performing?	Where are the gaps in performance?	What short-term value is the project delivering?	What is the strategic value of the project(s)?
Client	√	√	√	
Consultant	√	√	√	
Contractor	√	√		
Community	√		√	
Sponsor			√	√
Society at large			√	

A new user is introduced in the form of the sponsor, commonly the government policy agency overseeing the entire development of the project. In this case, whilst most parties are focused on the short-term benefits, the sponsor has a much broader brief and will be focused on the strategic value of the work done and how it fits with the overall sustainability goals of the city or nation. The sponsor is often not the same as the client.

In the end, if projects of a similar nature are benchmarked against each other using the scorecard, knowing that a minimum standard is required in all five areas, the sponsor together with the other project stakeholders will have aligned expectations and get what has been mutually agreed. Key to note is the need to establish agreed baselines as to what is business as usual compared to what is an aspiration towards achieving improvement and, ultimately project excellence.

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Adelaide's Riverbank Footbridge

Harry Turner
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Nick Southward
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Keywords: architecture; urban design; design competition; glass; box girder; post-tensioning.

ABSTRACT: This elegant structure, opened in early 2014, is a key element of the revitalisation of Adelaide's Riverbank Precinct. The area is one of Adelaide's iconic public spaces, offering panoramic views across the River Torrens, Parklands and Adelaide Oval. Crucially, the footbridge provides a river crossing and rail station access for crowds attending major sporting events at the recently upgraded Adelaide Oval. The bridge selection was based on a major design competition for this $40M project. The winning entry satisfied the Client's brief for a "design that will meet the highest levels of functionality and aesthetic resolution for the constituents and the city at large." The Urban Design was considered by the panel judges to be bold, elegant, striking and aesthetically pleasing. This paper discusses the urban design philosophy and presents some of the engineering challenges that were overcome in order to achieve this ambitious vision. An artist's impression is shown as Figure 1.

Figure 1 Artist's impression of the footbridge and Riverbank Precinct

1 INTRODUCTION

1.1 *Background*

In February 2012, South Australia's Department of Planning, Transport and Infrastructure (DPTI) issued a call for expressions of interest, "seeking to engage a highly motivated and committed design led multi-disciplinary team to provide an outstanding design and functional solution for a new pedestrian bridge and allied works connecting the north and south banks of the Torrens Lake in the Riverbank Precinct". This was essentially a design competition, requiring a concept design to be submitted with the expression of interest.

The Expression of Interest document noted that "The proposed bridge is an integral element in the recently released Master Plan for Adelaide's Riverbank Precinct. The bridge will assist in the revitalisation of this important part of the city and will play a major role in the movement strategy for the new 50,000-seat stadium at Adelaide Oval".

The Project was to be completed by December 2013, in time for a major Sporting event, being the Ashes Cricket Test Match. Given the ambitious nature of the project, this timeframe was clearly a challenge.

1.2 *Context*

Adelaide's Riverbank Precinct lies immediately north of the CBD. The location and character of this area are critical elements in the layout and make-up of the City of Adelaide. In particular the Riverbank Precinct provides an important link between the City and its picturesque river environment.

The Precinct contains some of the highest profile institutions and businesses in Adelaide. They include Parliament House, Adelaide Festival Centre, Adelaide Railway Station - all of heritage significance, along with the Adelaide Casino, Intercontinental Hotel, Riverside Office complex, the Adelaide Convention Centre and one of the City's pre-eminent outdoor gathering spaces, Elder Park.

1.3 *The Team*

Based on an enhanced schematic design, the LEAP Consortium, comprising Aurecon (lead consultant), landscape architects Taylor, Cullity, Lethlean and building architects Tonkin, Zulaikha, Greer was awarded the contract to design the bridge and associated works. Under the terms of DPTI's procurement method for this project, the consortium was eventually novated to the Contractor for the project for the final stage of the design and construction.

In order to preserve the integrity of the urban design vision, yet ensure constructability, DPTI devised a procurement method whereby the concept design was developed with controlled input from shortlisted independent Constructor teams. DPTI required construction staff to review and advise on cost, constructability issues and opportunities for the preferred initial schematic design. The designers worked collaboratively with DPTI and the shortlisted Constructor teams through this process to ensure that the schematic design developed could be constructed within proposed timeframes and budget.

The contract for the construction was eventually awarded to McConnell Dowell Constructors (MacDow). As part of its tender, MacDow proposed an alternative to the steel design developed by Aurecon for the bridge superstructure. MacDow, employing Tony Gee & Partners of Hong Kong as designers, developed a prestressed concrete option for the deck. This was accepted by DPTI and the detailed design of this element of the project was undertaken by Tony Gee.

2 THE VISION

For such an important environment, many guiding principles were considered by the team in developing the concept. From this work, two key design principles were distilled – minimum intrusion into the valued landscape experience of the Torrens, and maximum lightness and sculptural resolution.

The slim bridge depth, triangular planar glass surfaces and V-shape support structure was carefully designed to create the visual impression of a super light structure spanning effortlessly across the River Torrens with the V-pier just touching the water surface. The sweep of the river valley as defined by its northern and southern banks and adjacent road bridges, is dominated by significant trees and the urban form of the city. In the centre of this 'postcard' of Adelaide there is little place for another major vertical expression, rather the appropriate gesture is one of horizontality – linking, travelling, experiencing, not dividing and exclaiming.

Within this constraint however, the bridge should not be recessive – it is a significant contribution to the public life of the city, and therefore it needs to be a work of art, a response to the aspirations of the community, a thing of beauty in its own right.

In plan the sweep of the bridge is a pure arc, its carefully-defined level providing the most effective linkage from bank to bank. Supporting this arc is a structure of extreme slenderness – under 2m in depth for the majority of its length. A southern pier is located in the water, being a landmark sculpture of intersecting prismatic shapes, lightly touching the water, opened up to the vistas of dappled light. The north pier is similar, but engages with the sloping landform rather than the water, soaring lightly over the river-front pathway.

The surfaces of the bridge are clad with laminated glass panels, forming a flowing series of planar surfaces to celebrate its delicate leap across the river. These sculpted planes interpret the structural forces of the bridge, whilst providing facetted surfaces to reflect light and the patterns of the water. Their warm white colour responds to the dominant forma of the surrounding landmarks – the sails of the Festival Centre and the vaults of the new Stadium. The simplicity of the bridge is enhanced by the resolved design of the handrail, open and glassy, with its integrated lighting. The pavement is in sawn granite, a durable low maintenance material.

To the east, the arc of the bridge continues, defining the edge of the Oval Plaza, and extends out over the water as a poetic final destination, a viewing place facing Elder Park. From this final celebration of the bridge a flow of water aerates the Torrens in a dramatic gesture of water, environment and the rich experience of the city.

3 DESIGN EVOLUTION

3.1 *Geometry*

One of the most unusual aspects of the architecturally derived design was the asymmetry of the structure. The architect's vision required an unsymmetrical triangular cross-section, a horizontally curved alignment and tilted/unsymmetrical V piers. The need to achieve a very slender deck and accommodate the various imbalanced dead load generated forces caused by this unusual geometry posed many challenges for the design team. The elevation of the bridge is shown as Figure 2 below.

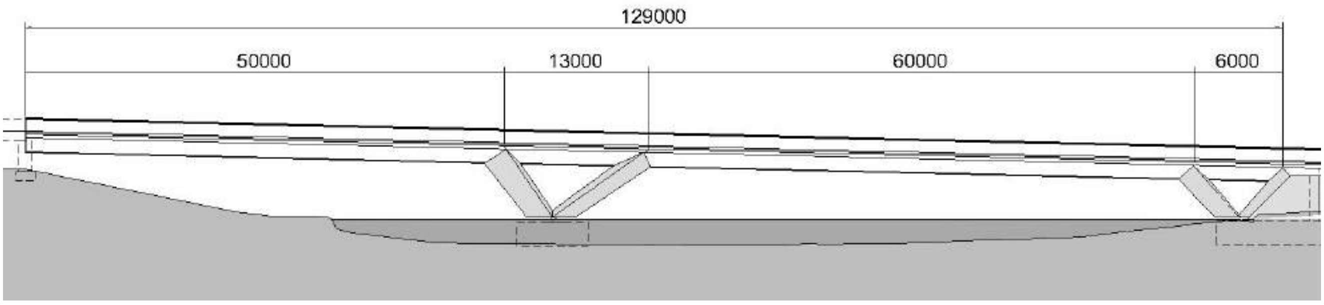

Figure 2 Bridge elevation

3.2 *Width*

The footbridge needs to cater for events at Adelaide Oval and forms the main river crossing for spectators accessing the nearby Adelaide Rail Station and the city in general. The notional width envisaged by DPTI at the start of the project was 10-12m, however a rigorous modelling exercise was undertaken to arrive at an optimal width. DPTI engaged WS Atkins (UK) to perform this modelling, considering various width scenarios of 6, 8, 10 and 12m. A rigorous modelling process was required in order to inform the trade-off between form, function and cost. Obviously, the greatest width would allow for the quickest egress, however a step change in cost can occur with only a marginal increase in capacity (or vice versa).

The "central" design case for the bridge was for a crowd of 45,000 spectators. A capacity Design Case of 54,000 spectators was also considered. The alternative (existing) crossing is via the nearby King William Street road bridge. Microsimulation software Legion Spaceworks Enhancement was employed for the modelling.

The conclusion from the modelling was that on balance, there was a credible case for an 8m wide footbridge that delivers a system performance that far exceeds the existing condition, that limits lane closure of the adjacent road bridge to about 5 times per year, and which facilitates a good level of service most of the year round and reasonable levels of service under even the most extreme demand conditions. Based on this conclusion and after considering cost and aesthetic issues, a width of 8m was adopted.

3.3 *Cross-section*

A triangular shape is of course structurally inefficient. Both the original steel and final concrete designs comprised truncated triangular box girders, with acutely angled webs. In order to achieve the required capacity, the steel design required very heavy steel plates in the narrow bottom flange, whilst the final post-tensioned concrete design has a very heavy tendon arrangement.

3.4 *Steel Deck*

The steel solution developed as the winning concept design is illustrated in Figure 3. The construction philosophy was to pre-fabricate the deck into the largest and heaviest units that could be transported and lifted into place by crane. A longitudinal split was envisaged in order to limit size and weight of modules, requiring a full length welded connection on site.

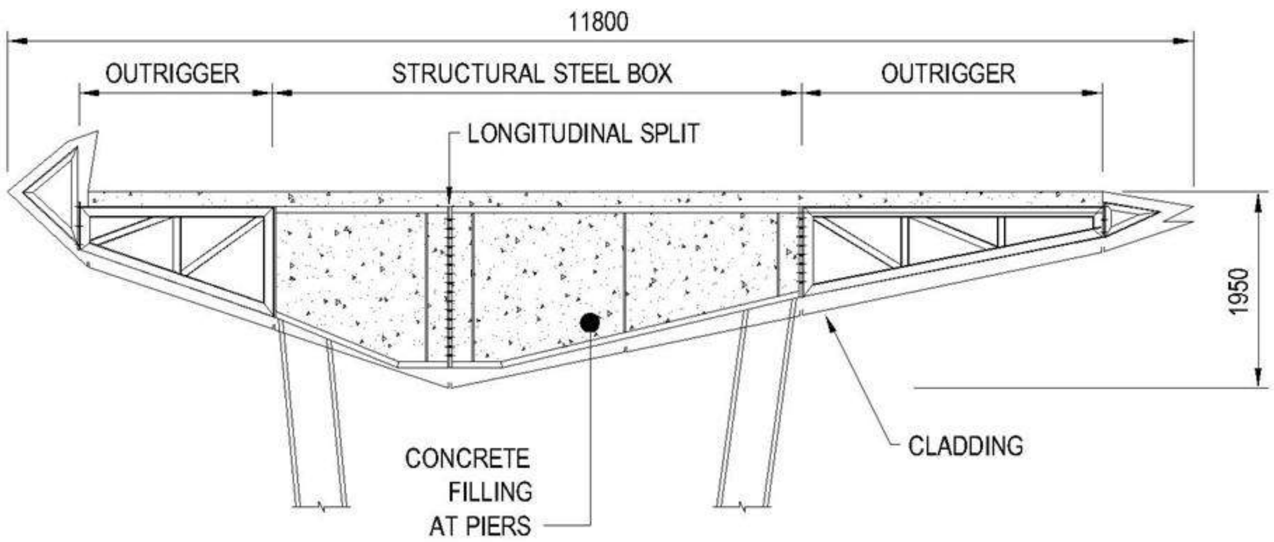

Figure 3 Cross-section of proposed steel concept

3.5 *Articulation*

The unusual curvature, unsymmetrical cross-section and asymmetric V-column bridge supports limited the ability to adopt typical bridge construction techniques/philosophies with movement joints at each main support. The resultant of this unique bridge structure was permanent lateral forces in both radially and tangentially to the bridge at the V-column bridge supports.

The bridge's performance was found to be highly sensitive to the overall stiffness of the bridge foundations, and arrangement of movement joints. A detailed sensitivity analysis was undertaken to trial various bridge restraints, and the theoretical upper and lower stiffness of the foundation system to determine the maximum loadings on the bridge.

The sensitivity analysis also considered potential changes to soil conditions over the 150 year design life.

The final articulation arrangement comprised a bearing/expansion joint at the southern end, allowing movement in the radial and tangential direction of the bridge axes and fixed connections for the asymmetric V-column supports and the northern end of the bridge.

3.6 *Dynamic Behaviour*

The extreme slenderness of the superstructure meant that vibration issues were key considerations from the outset. With a span/depth ratio of 32, it was no surprise that natural frequencies were in the order of 1.5 Hz, resulting in potentially unacceptable accelerations. From the very early stages of the design, it was considered that some form of artificial damping would be required in order to keep vibrations within appropriate limits and satisfy AS5100.2 clause 12.4. A concept design incorporating tuned mass dampers (TMDs) was developed. The TMDs were to be encapsulated within the box.

Horizontal excitation was never considered problematic. The relatively wide deck, combined with the arching effect resulting from the curvature, meant that lateral vibrations were modest.

Reference is made to the final section of this paper for discussion on the analysis method used to justify the final (alternative deck) design dynamic behaviour.

3.7 *River Loading*

The Torrens River in this location is not truly navigable, being essentially a lake artificially created by the presence of a major weir. The only vessels in use here that could potentially impact a pier were assessed as having a negligible effect on the structure. Likewise, water velocities under all conditions are not significant, so water related forces did not govern any aspect of the structural design.

3.8 *Glass Cladding*

Unusually for a bridge structure, a cladding system formed part of the design. The urban designers' vision required the surfaces of the bridge to appear as a flowing series of planar surfaces clad in a smooth reflective material. These sculpted planes interpret the structural forces of the bridge, whilst providing facetted surfaces to reflect light and the patterns of the water. The finally adopted solution comprised some 2,000 glass panels, each panel approximately 1,800mm by 900mm in size, and forming a "snake-skin" type pattern. The polar white colour was selected to respond to the dominant forms of the surrounding landmarks - the sails of the Adelaide Festival Centre and the vaults of the new Adelaide Oval Stadium.

For the original steel deck concept, the cladding was also intended to provide greater durability for the structure. In the same manner that steel buildings are protected with sealed facades, it was intended that the cladding form a semi-impervious barrier to the atmosphere. Each glass panel was uniquely designed to allow individual panels to be removed as necessary for periodic inspection of the structure using a deck mounted vehicle with an under-bridge inspection boom.

While the glass clad system still provides a semi-impervious barrier and thus greater durability performance, this was less critical for the concrete bridge deck compared with the steel version. The function of the cladding was therefore reduced to that of a mainly aesthetic nature, consisting of white opaque, laminated glass panels fixed to the concrete underside of the bridge. The engineering design of the glass clad system is unusual, incorporating aluminium sections that were specifically extruded for this project and allowing for tolerance during construction is all directions. The typical fixing detail is shown in Figure 4.

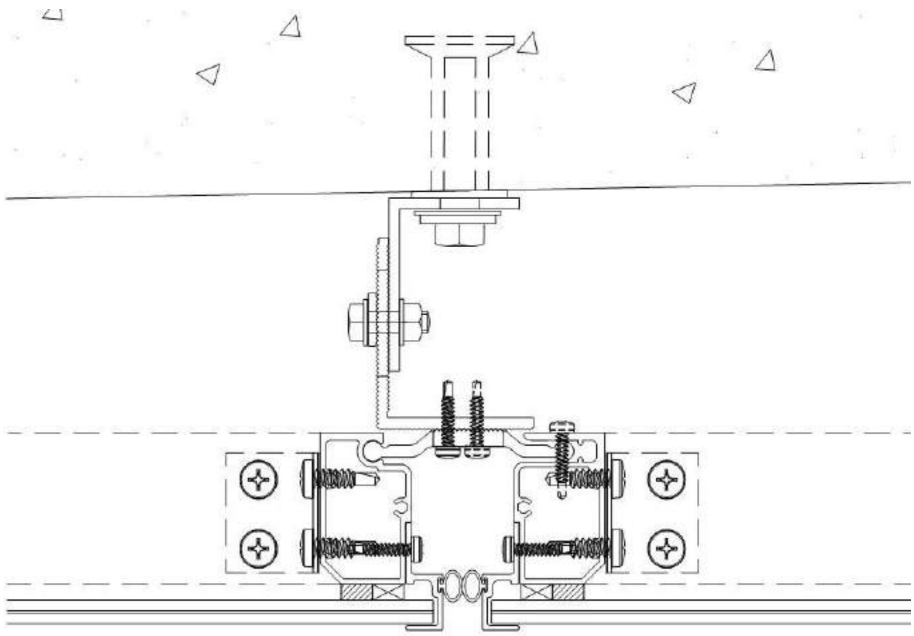

Figure 4 Glass cladding fixing detail

The method of attachment of the cladding to the deck and piers had to be designed so that the relative movements of the bridge deck and piers could be accommodated without distortion to the glass panels and consequent cracking. To achieve this, the relative movement of each element of the structures were calculated and provided to the cladding designer. The relative movements between piers and deck were found to be too large so that the cladding on the piers is isolated from that on the deck.

3.9 *Piers*

The superstructure is supported by two structural steel V piers - one in the river and another adjacent to the northern bank and concrete abutments. The columns are fabricated steel sections inclined at varying angles to create the asymmetric shape of the clad V piers. The structural steel columns are braced in the radial direction to resist torsional loading and the out-of-balance loading between columns caused by the asymmetric nature of the bridge structure and the curvature of the bridge in the horizontal plane.

4 FOUNDATIONS

4.1 *Geotechnical Summary*

The soil profile consists of typical alluvial conditions, with interbedded sandy clays and gravels over dense sand.

4.2 *Piled Foundations*

As noted above, the bridge is curved and incorporates multiple asymmetries. As a result, significant horizontal forces and overturning/torsional moments are transferred to the footings, in addition to conventional vertical forces. A high proportion of the out of balance loading is permanent. From consideration of the soil profile, it was clear that piled foundations were appropriate, founded in the stiff sandy gravels.

A number of design options were investigated for the bridge foundations including, caissons, driven & cast-in place, bulbous based ‘Franki’ double bulb piles, steel driven tube piles and continuous flight augered (CFA) piles.

CFA piles were adopted for all bridge foundations, based on the CFA's cost effectiveness and ability to deal with the presence of water and large permanent lateral forces. In order to construct the foundation system within the river, steel driven sheet piles were first installed from a barge around the pile cap. Waler beams were installed and a temporary causeway and piling platform constructed to allow access for a piling rig, and later for support of the temporary formwork for construction on the main bridge deck.

Following a durability review (refer to discussion below) a 50MPa concrete mix was chosen, with 25% fly ash and 90mm cover. Piles on the north bank, north landing and under river (south) required sulphate resistance treatments.

4.3 *Durability*

Unusually, the client's brief called for a 150 year design life for the bridge – 50 years more than the Australian Bridge Code norm. The Designers were required to produce a durability plan demonstrating that this life could be achieved with the design with minimal maintenance. The durability plan included an analysis of the existing and expected future environmental conditions for the bridge. Climate change over the next 150 years was taken into account.

A number of potential hazards were identified in this riverine environment including sulphate attack, acid sulphate soil degradation, alkali-aggregate reaction, leaching and delayed ettringite formation.

Special, sulphate resistant concrete mixes and high concrete covers (90mm) were specified in order to deal with the identified hazards for the piles, pilecaps and column cladding. Allowance for future cathodic protection was made within the design of reinforcement within piles and to pile caps and plinths.

5 DETAILED CONSTRUCTION DESIGN

As part of its winning construction tender, the bridge Contractor McConnell Dowel submitted a cost effective alternative concrete design for the superstructure which was accepted by DPTI. The advantages and disadvantages of the concrete alternative compared to the original steel version are summarised in Table 1:

Table 1 Advantages and disadvantages of the concrete alternative compared to the original steel version

	Superstructure type	
Criteria	Steel	Concrete
Depth / Aesthetics	●	•
Cost	•	●
Constructability	•	●
Construction impact on river	●	•
Construction impact on Bank Precinct	•	●
Durability	•	●
Vibration	•	●

● Positive / low impact
• Relatively less positive / higher impact

The prestressed concrete box girder alternative developed by Tony Gee was detailed to match the external shape of the cladding envelope, but required a small increase in the structural depth at the apex of the soffit. Otherwise the appearance and layout of the structure was unchanged from the concept design developed by Aurecon.

This section of the paper focusses on those aspects of the design that significantly departed from the concept described above or are considered worthy of further description.

A cross-section of the Alternative Deck is shown below as Figure 5.

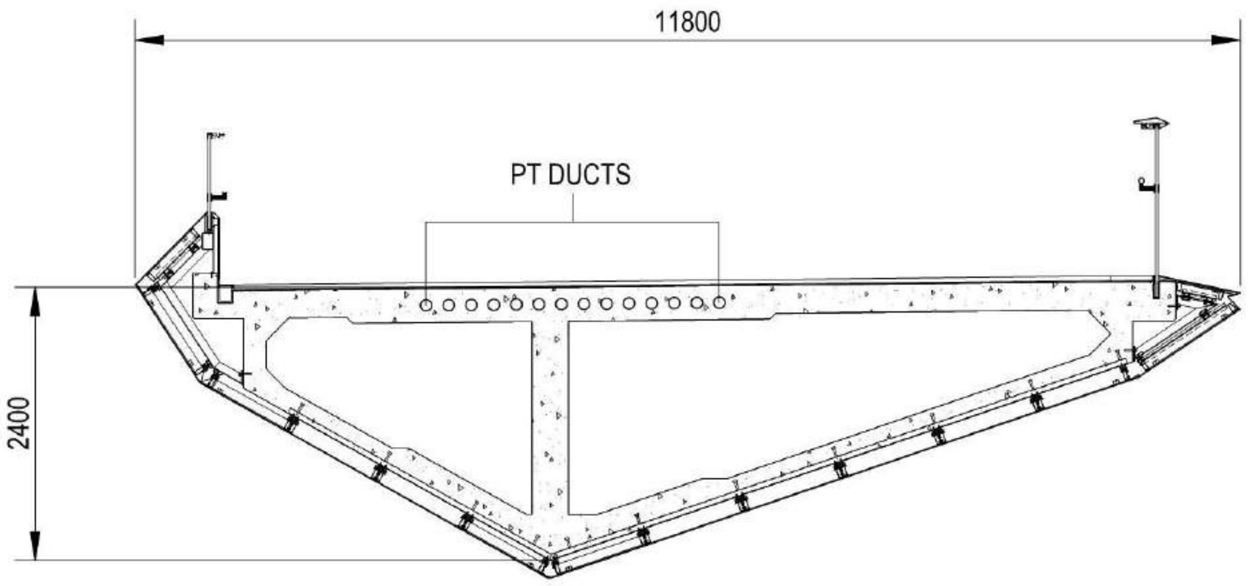

Figure 5 Final concrete deck cross-section at pier

5.1 *Structural Form*

The prestressed concrete box girder superstructure was designed to be cast in-situ and is structurally continuous between abutments. The box girder was designed to carry its shear forces on the central web, with the torsion created by the span curvature being resisted by the exterior box section, thus creating the most efficient section possible within the constraints of the cladding envelope.

5.2 *Bridge Articulation*

The Northern abutment is designed to be a fixed point and to "encastre" the bridge span, which is monolithically connected to abutment walls. The span applies a permanent overturning moment to the abutment. A counterweight soil box behind the abutment is used to minimise this effect. The General Arrangement of the structure is shown in Figure 6.

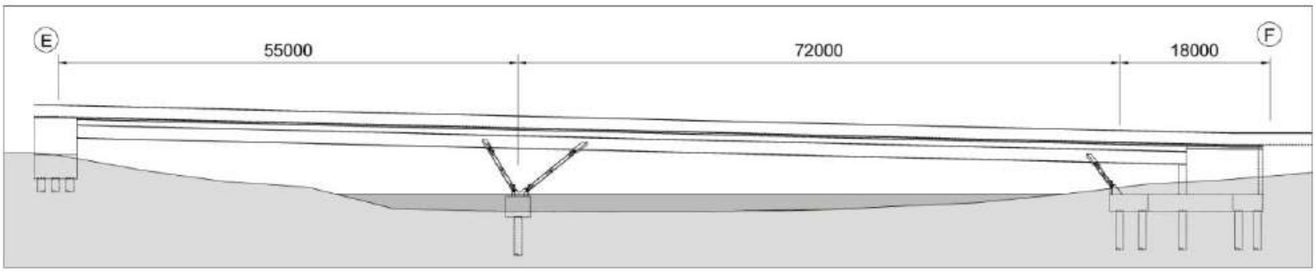

Figure 6 Bridge general arrangement and articulation

5.3 *Column Details*

At intermediate points the bridge deck is supported by a pair of diagonal steel column sections that frame down to the pilecaps. The steel columns consist of twin 400x303 WC welded steel column sections, which are braced together with diagonal bracings. A view of the hidden structure of the river pier is shown as Figure 7.

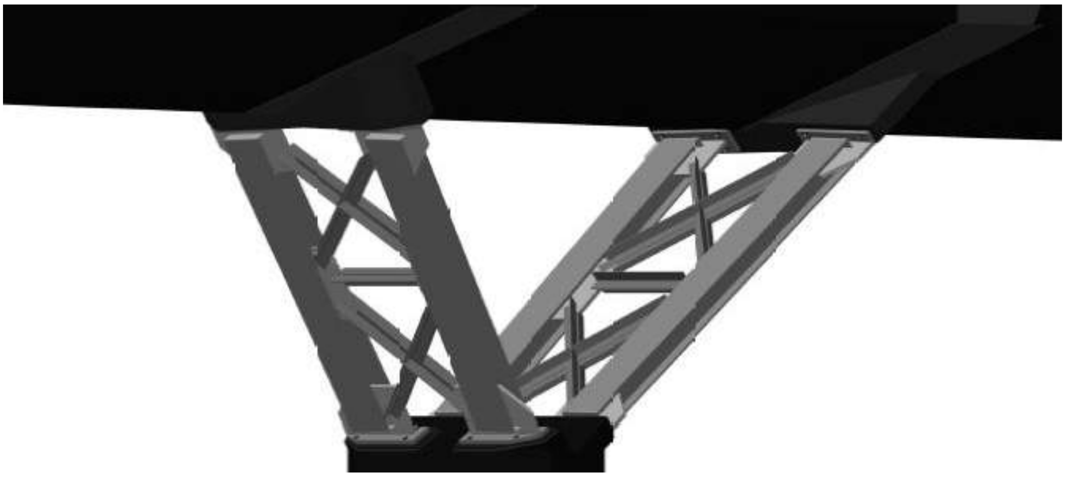

Figure 7 View of steel columns of central pier

In the concept design the top and bottom of the columns had full moment connectivity by bolting to the deck and pilecaps. This arrangement created large longitudinal bending moments in the columns and foundations and these were exacerbated by the elastic shortening, creep and shrinkage movements of the alternative concrete deck. To solve the problem the connection detail was changed to that of a pinned connection at the top and bottom of the columns, significantly reducing the moments.

To avoid the use of expensive pinned connections a vertical plate was used to connect the ends of the steel members with the concrete supports. This plate acts as a dummy pin or “fuse” by means of its reduced stiffness and allows for movement to occur so the columns behave as if they are pinned top and bottom. Overall stability of the bridge is ensured by means of the encastre connection at the northern abutment. The plate “pin” detail is shown as Figure 8.

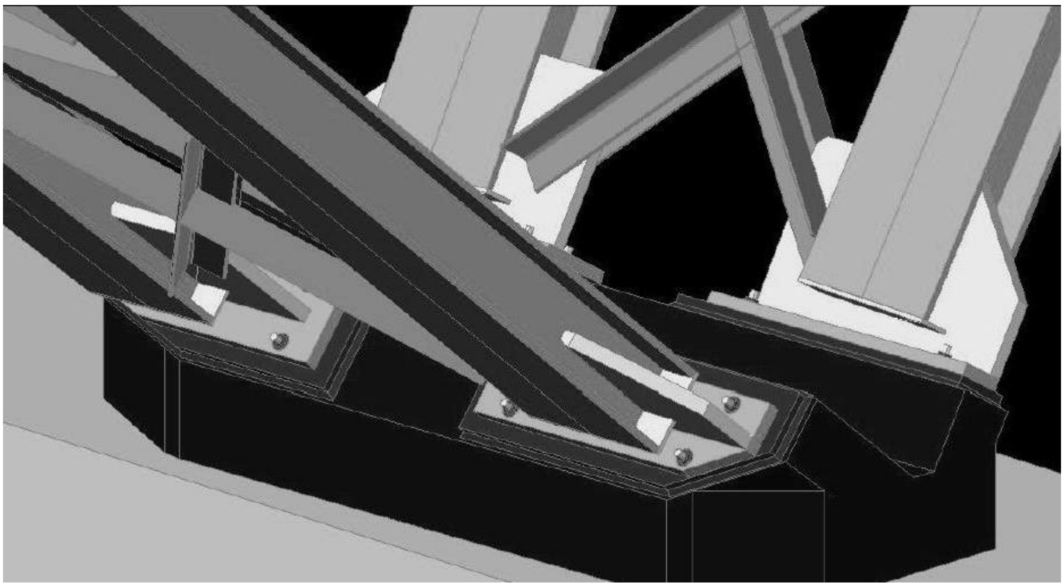

Figure 8 View of pin plate detail at base of inclined columns

A concrete downstand and pilecap support plinth was used to bolt the steel columns to the structure. The plinths were carefully dimensioned to suit the internal envelope of the cladding.

5.4 *Foundation Layouts*

Although the top and bottom of the columns are pinned steel members, the diagonal orientation or framing of those members mean that the columns behave like a truss and force the pilecap to move longitudinally with the deck as a rigid body. Therefore the arrangement of the piles was made to allow the piles to flex longitudinally with the pilecap movement by using a line of piles arranged in a row perpendicular to the bridge deck centreline, but rotated by approximately 3 degrees clockwise. The effect of this orientation is to optimise the alignment of the piles as best possible by minimising the transverse moments that are created on the pilecap by the effect of longitudinal creep and shrinkage of the deck. The single row of piles allow the pilecap to move tangentially under creep, shrinkage and thermal effects with this movement being accommodated by flexure of the top part of the piles.

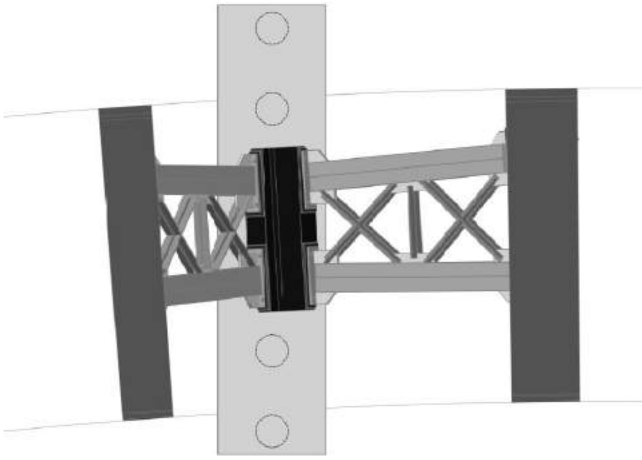

Figure 9 Central pile cap layout

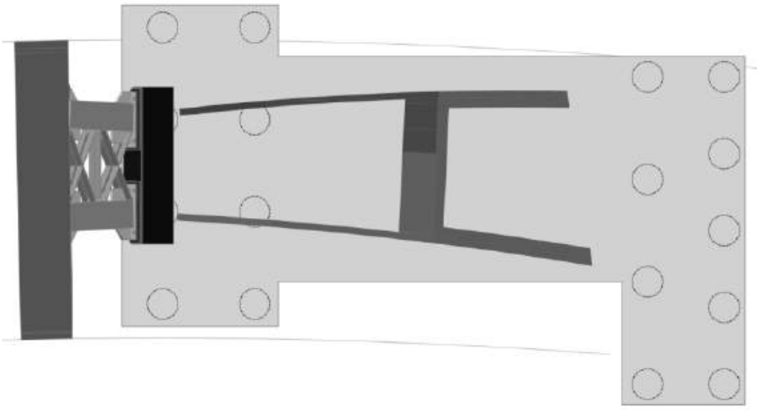

Figure 10 North abutment pile cap layout

The span curvature and the fixed moment at the North Abutment required an asymmetric pile layout, which was orientated so as to create an even set of pile loads and to minimise the pile tensions that occur in the ultimate load combinations on the back row of piles. Piles are in compression in the service conditions. These foundations are shown as Figures 9 and 10.

5.5 *Prestressing Layouts*

The prestressing for the bridge spans use conventional internal post-tensioning layouts, with the tendons were located in the central web of the section, but were distributed into the top and bottom flanges of the cross section at the supports and midspan regions respectively. Supplementary short tendons were used in the at the supports and midspans to provide the required design strength.

The prestressing layout is illustrated in Figure 11 below.

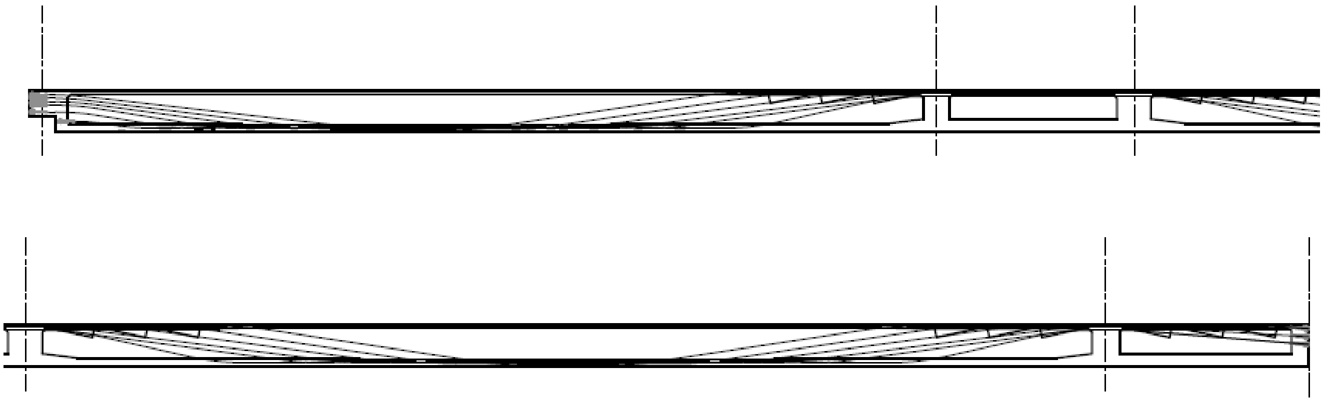

Figure 11 Layout of principal prestressing tendons

5.6 *Pile Design*

Further to the discussion above, of particular concern in the pile design was that of cyclic loading effects due to thermal expansion and contraction of the deck and the impacts this would have on the bending moment created in the piles. The effect of cyclic loading is to soften the engineering stiffnesses of the soil, so this effect was considered by adopting the following:

a) Upper and lower bound stiffnesses of the soil
b) The weak soil over the upper 4 to 5m of the pile yields
c) The pile bending design is governed by upper bound stiffnesses.

5.7 *Pedestrian Excitation*

The vertical and horizontal fundamental frequencies of the concrete alternative for the footbridge were both found to be in the order of 1.2Hz. This meant that as the natural frequency of the bridge was below 5Hz, further dynamic analysis was required to calculate the accelerations of the bridge under various live loads to ensure that these accelerations were within the acceptable limits.

The dynamic structural analysis under wind induced excitation of the bridge, confirmed that the cross-section and aerodynamic profile of the bridge mitigated likely vortex shedding. Further dynamic analysis of the bridge focused on calculating acceleration values from pedestrian induced vibrations. The criteria on whether the use of TMDs were required are based on the human comfort factor in accordance to SÉTRA Technical Guide. The guide defines the acceptable comfort levels in terms of different acceleration ranges in the vertical and horizontal direction. There are 4 frequency ranges, which correspond to a decreasing risk of resonance, Range 1 being at maximum risk of resonance down to Range 4 at negligible risk of resonance. Our dynamic analysis checked against these 4 frequency ranges under 3 classes of human traffic loading -standard, heavy and dense crowds.

The conclusion from the dynamic analysis was that for the 3 classes of human traffic loading, the acceleration ranges calculated from the corresponding frequency ranges were within the design limits.

6 CONCLUSION

This paper has discussed how an ambitious urban design vision was developed for this unique site and how various challenges were overcome in order to deliver the promise encapsulated within the winning design competition entry. In particular, the unusual geometric requirements of the urban designers constituted a major challenge to be overcome. Innovations in the area of structural modelling, articulation, vibration assessment, foundation design helped to overcome these challenges. Strikingly, the adoption of a glass cladding system – a first for a footbridge in Australia, helped achieve the ambitious architectural vision.

A "soft" opening of the bridge was successfully made in December 2013, in time for the second Ashes cricket test match at the newly upgraded Adelaide Oval. Since this time, the bridge has catered for a full season of AFL Football games and many other capacity events at the Oval. The reaction from the public has been overwhelmingly positive. The final structure is a spectacular yet harmonious addition to the landscape of Adelaide's Riverbank.

7 ACKNOWLEDGEMENTS

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a) The South Australian Department of Planning, Transport and Infrastructure (DPTI)
b) Urban Designers Taylor Cullity Lethlean and Tonkin Zulaikha Greer
c) Constructors McConnell Dowell

Development of eco-cities through increasing eco-efficiency in urban construction and innovation – energy in cities

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Keywords: energy supply; demand side management; distributed generation; district cooling system; smart grids and microgrids; data in smart cities.

ABSTRACT: Eco-city development is a new concept of city development adopting the scientific principles of sustainable development. The essence of eco-city development is to achieve a balance on sustainability between the impact of socio-economic development and the ecological and environmental quality of the city. In the implementation of an eco-city, key focus shall be on the planning and development of the eco-city construction, the requirements of eco-efficiency in urban construction according to the process of sustainable development and the concept of innovation. In this paper, the eco-efficiency in urban construction such as energy in cities will be examined.

1 INTRODUCTION

Eco-city development is a new concept of city development adopting the scientific principles of sustainable development. The essence of eco-city development is to achieve a balance on sustainability between the impact of socio-economic development and the ecological and environmental quality of the city. In the implementation of an eco-city, key focus shall be on the planning and development of the eco-city construction, the requirements of eco-efficiency in urban construction according to the process of sustainable development and the concept of innovation. The eco-efficiency in urban construction such as energy in cities will be examined.

2 ENERGY IN CITIES

The way we generate, distribute and consume energy is changing. The scale and rate of this change is a major challenge for existing and new cities alike, but also brings enormous economic, social and environmental opportunities.

Cities consume over three-quarters of the world's energy and account for 80% of its greenhouse gas emissions. Under conditions of resource scarcity and climate change, future cities will need to lower their energy demand and consumption whilst supporting larger, more affluent populations and meeting people's expectations for local environmental quality.

There shall be a plan to help cities reduce energy demand, enhance efficiency of supply and increase use of low carbon and renewable energy. Arup was the consultant to Hong Kong Green Building Council to investigate the HK3030 Campaign in which a comprehensive plan and strategies had been established to reduce the demand side electricity consumption of Hong Kong by 30% in 2030 compared to the 2005 (Hong Kong Green Building Council, 2014). The systems and technologies featured in this paper were selected to show that the innovative energy solutions that could enable cities to achieve successful energy transitions.

a) Energy Supply – Low Carbon & Renewables
b) Energy Storage
c) Smart Grids and Microgrids
d) Data in Smart Cities
e) Building Systems

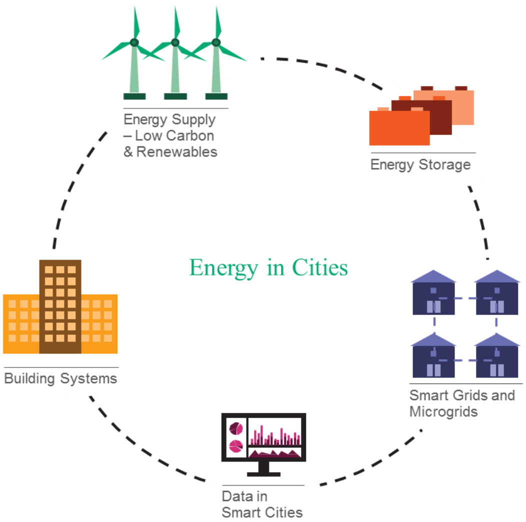


Figure 1

Global energy demand is expected to double or even triple by 2050, by which time 70% of the world's population will be living in cities. Clearly, cities have a decisive role to play in tackling the energy challenges ahead and driving the transition towards a low carbon energy future.

Energy is essential to all city functions and services; it is at the centre of both the challenge and the solution to achieve healthier, more resilient and prosperous cities. But while urbanisation drives economic growth, innovation and social progress, it is also contributes to vulnerability in cities. Climate change is inextricably linked to urbanisation; a global problem with very local impact for cities' energy services and beyond. As the world urbanises, cities will need to provide more with less. Innovative approaches to energy delivery and management are fundamental to tacking that challenge.

3 ENERGY CHALLENGES IN CITIES

In addition to increasing population, cities are threatened by rising energy demand, ageing infrastructure, volatile energy markets and climate change impacts. The need to update and coordinate energy services has seldom been greater.

The energy trilemma encapsulates three distinct objectives for future systems while also recognising the tension between these objectives:

a) Maintaining a reliable and secure energy supply,
b) Ensuring long term affordability of the energy system and
c) Drastically reducing GHG emissions associated with energy supply.

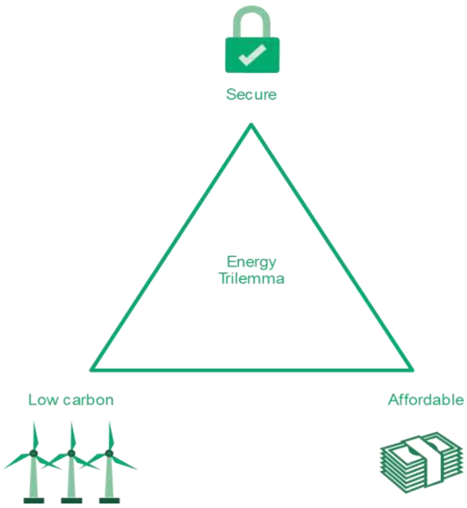


Figure 2

Cities can help solve this trilemma by adapting their energy delivery services to become more flexible, responsive and decentralized. These adaptations will enable a greater share of energy to be supplied by renewable and low carbon sources; however they may also increase the complexity of the urban energy systems.

Beyond the technical challenges, cities will also need to ensure regulatory conditions and governance are appropriate to encourage new business models under alternative market conditions.

Cities are a powerful force for delivering action. They are increasingly showing they have the economic and political power to lead where nations and governments have failed.

Governance - Energy assets such as district heating/cooling networks, wind farms and rooftop solar PV panels may be owned and operated by a range of stakeholders from state or city authorities, to private sectors, community groups and individuals. Regulatory mechanisms will be needed to protect such group, whilst ensuring supply and demand is balanced in a predictable and reliable fashion. As the energy transition gathers pace, cities must support the roll-out of evolving technical solutions with governance strategies that favor integration of assets and services and engagement from stakeholders at multiple levels. Resilience, efficiency and long-term cost savings are amongst the opportunities that may be achieved.

Knowledge Networks – Cities are already proving their capacity for sustained, transformative, and even radical action through networks like the C40 City Climate Leadership Group, 100 Resilient Cities and the Asian Cities Climate Change Resilience Network (ACCCRN). Networks serve to connect the multitude of stakeholders that operate at different levels of city governance, providing a platform for action and engagement and a medium for sharing knowledge and learning.

4 ENERGY TRANSITION PLANNING IN CITIES

Cities have a key role to play in the shift towards a sustainable energy future.

The energy transition will not happen overnight. For the foreseeable future, cities will continue to rely on legacy systems and fossil fuels to help meet basic energy services including heating/cooling and transport. The transition process needs to be well thought-out to guide the change, helping us go from where we are now to where we want to get to.

Modelling and Scenario Planning – Energy modelling and scenario planning will help cities set out targets and create realistic plans and programmes to transition to a low carbon future.

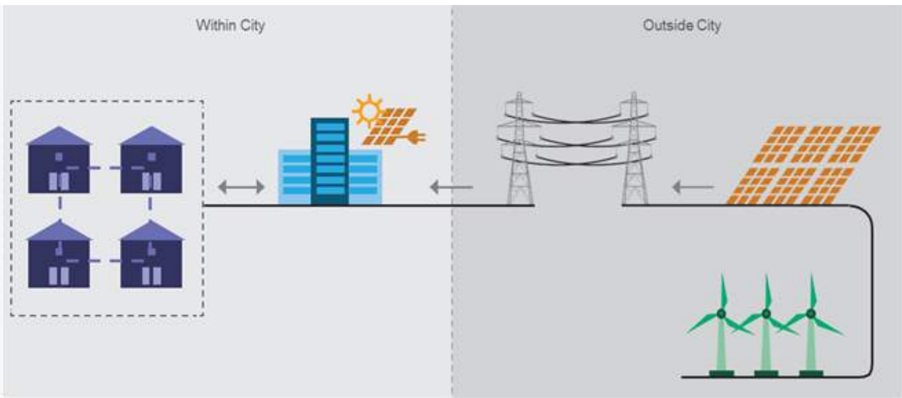
	1	2	3	4	5
Solution	**Energy Supply – Low Carbon & Renewables**	**Energy Storage**	**Smart Grids and Micrograds**	**Data in Smart Cities**	**Building Systems**
Capability and Benefit to Cities	Renewables will be deployed at both the national grid scale (in the form of large-scale biomass, wind, solar, hydro and tidal plants based outside the city) to replace traditional centralised power stations; and at the distribution or building scales (in the form of micro renewables and low carbon heat sources such as heat pumps, biomass and waste CHP and solar thermal), within the city. Both will help cities meet growing energy demand while reducing GHG emissions.	Energy storage enables optimisation of supply and demand, helps mitigate grid infrastructure constraints and assists the seamless integration of renewable energy. Storage helps increase efficiency and reliability at various points on the system using solutions including batteries, compressed air, flywheels, hydrogen and supercapacitators. District heating with heat storage can provide additional balancing (when supplied from heat pumps or CHP).	Smart grids is an umbrella definition which includes a number of technologies and operational strategies implemented into power networks which enable the delivery of flexible, affordable, reliable and efficient power. The bidirectional flow of power and data is the key operational feature in smart grids. Microgrids (a subset of smart grids) assist localised electricity distribution with smart coupling of decentralised electricity generation and demand response for reliable and high quality supply.	Smart cities use technology to transform infrastructure and make better use of energy and resources. Information and communications technologies (ICT) can be deployed to create new intelligent ways of making urban centres more efficient. Real time data informs smart systems which facilitate the shift towards behaviours that contribute to the development of the low carbon economy.	Energy reduction in buildings can make a significant impact on the overall energy use of a city. Increasing operational efficiency in existing and new building stock may be achieved through a combination of design, regulation, materials and technical solutions. For example, integrated building management systems allow energy use and demand to be monitored and optimised which can result in lower energy usage, costs and carbon emissions.

Figure 3

5 ENERGY SUPPLY – LOW CARBON & RENEWABLES

Supplying low and zero carbon energy will require a combination of centralized and decentralized sources to meet growing demand and global greenhouse gas emission targets.

Large scale renewables such as solar farms (PV and solar concentration) and wind farms, as well as hydro and tidal technologies that operate outside of city boundaries will need to be integrated with distribution scale systems within cities. This will require smart interfaces between intermittent renewable sources and storage mechanisms to ensure a reliable supply and a stable, balanced system.



District heating and cooling networks provide low carbon energy to communities and combined heat and power plants can export electricity back to the grid.

Upgrading of ageing electrical infrastructure is required to enable cities to transition to a low carbon future.

Combining centralised and decentralised supplies will increase resilience and energy security.

Integration of multiple renewable and low carbon technologies will reduce dependency on fossil fuels.

Figure 4

6 ENERGY SUPPLY – DECENTRALISED ENERGY IN CITIES

Decentralised energy schemes offer affordable way of achieving low carbon energy supply in densely populated cities. Decentralised energy is generally locally and off the conventional electricity grid.

Electricity – Decentralised energy can be provided by small scale renewables or low carbon technologies at a community or household level.

Heating/Cooling – District energy networks can provide heating/cooling for whole communities and even cities. Decentralised energy system can also increase resilience by helping cities cope with fuel price shocks and manage heating and electricity demand more accurately.

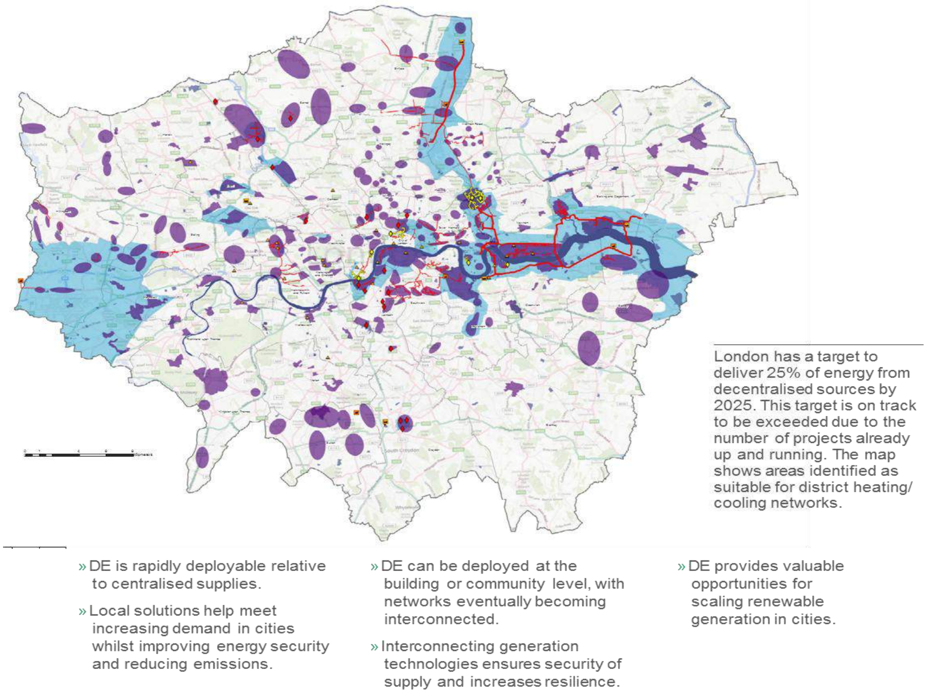


Figure 5

In Hong Kong, the Government has cited that there may be opportunities for the development of small-scale distributed power generation in the recently published Public Consultation document on the Future Development of the Electricity Market (Environment Bureau, March 2015). For example, in a recent joint project between the Hospital Authority and Towngas, a small-scale generator is being planned for construction in a hospital in Tai Po District that would use landfill gas from the North East New Territories Landfill to generate both heat and electricity for the hospital. Another example is that a trigeneration (combined cooling, heating and power CCHP) system fuelled by biofuel (refined waste cooking oil collected from restaurants in shopping malls) was used to generate electricity and harvest the waste heat for space cooling and dehumidification in the Construction Industry Council Hong Kong first Zero Carbon Building in Kowloon East. The Government has considered that development of such distributed generation could be further facilitated if improvement to the grid connection arrangement as stipulated in the Scheme of Control can be made.

Consumer Council in Hong Kong also welcomed the idea of using natural gas as primary fuel for small-scale electricity generation for large buildings with use of the waste heat in their report on Searching New Directions - A Study of Hong Kong Electricity Market (Consumer Council, December 2014). Dependent on the availability of fuel sources, i.e. landfill gas, biofuel or natural gas and providing them in the basic infrastructure to buildings or district scale are key to success of such distributed energy generation.

7 ENERGY SUPPLY – HEAT AND COOLING NETWORKS

Heat and cooling networks represent an affordable, efficient, low carbon, resilient solution to the comfort and hot water needs of domestic and non-domestic buildings in densely-populated areas.

These systems consist of distribution network carrying heated or cooled water from the generation source to the end users thereby avoiding the need for individual systems. These networks can facilitate deployment of a larger amount of renewable heat then by individual stakeholders. Energy sources for heating and cooling networks include conventional options such as local power stations and smaller scale Combined Heat Cooling CHP engines, but also natural sources such as water bodies and geothermal resources; urban infrastructure sources such as underground train epilation shafts, wastewater in sewer pipes and electricity substations; and coupled heating and cooling loads where the rejected heat from a cooled space (such as a data centre) can be captured and used in a heated space.

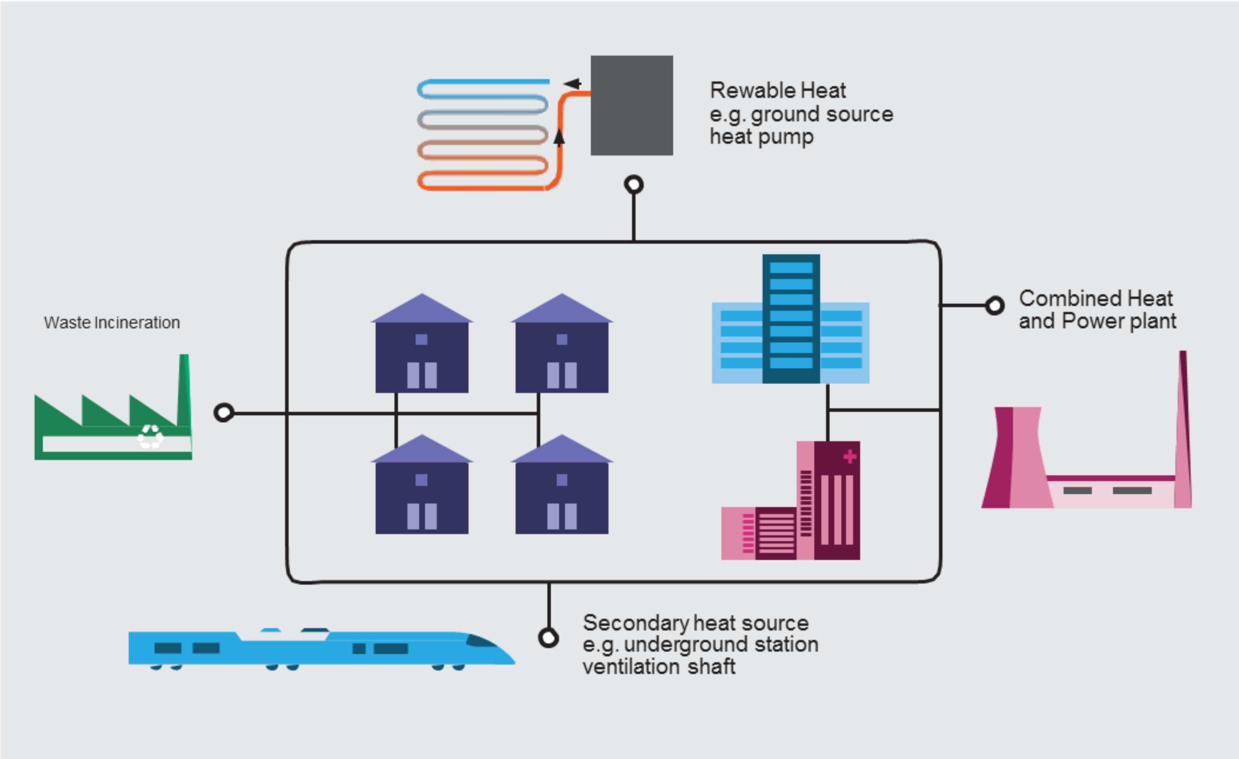


Figure 6

7.1 *District Cooling Systems (DCS) in Hong Kong*

7.1.1 *Kai Tak Development (KTD)*

The KTD DCS is the first of its kind innovative cooling method to be implemented in Hong Kong. It is one of the key initiatives of the 2008-09 Policy Address where the HKSAR Government planned to implement DCS at KTD to promote efficiency and conservation. The KTD is a mixed development with a GFA of over 2.24 million m^2 in the old Kai Tak Airport area. The development includes commercial offices & retail, government offices of various departments, transport infrastructure, community buildings, hotels and both public and private housings. The DCS will serve all the buildings of KTD, with the exception of domestic developments. The design capacity of the DCS is 284MW.

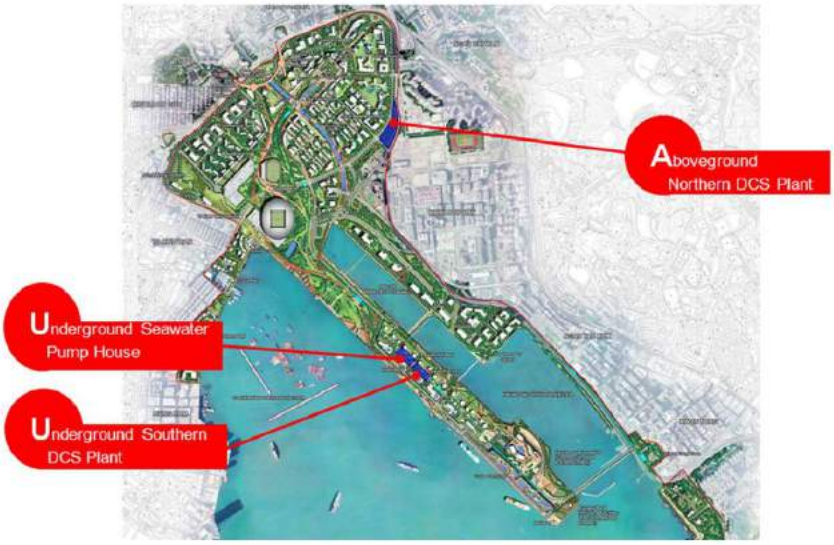


Figure 7 Masterplan of KTD DCS

7.1.2 *West Kowloon Cultural District (WKCD)*

Located at the southern tip of West Kowloon Reclamation Area with an area of 40-hectare, the WKCD will be developed into an integrated arts and cultural district with world-class facilities and iconic architecture for locals and tourists. It is one of the world's largest and most ambitious arts and cultural building programmes that will deliver up to 17 new visual culture and performing arts facilities, open spaces, education, commercial and retail facilities as part of a new cultural hub being developed with funding from the Hong Kong SAR Government. With the introduction of DCS and other sustainable features, it will serve as a showcase of Hong Kong and our commitment to long-term sustainability. The ultimate design capacity of DCS plant is 70MW.

Figure 8 Masterplan of WKCD DCS

7.1.3 *Challenges to DCS development*

Despite the advantages of DCS, there are several challenges for the full implementation of DCS in Hong Kong that make it difficult to reach its full potential:

7.1.3.1 *Site planning and interfaces*

The sites for the construction of DCS plants and pipeworks are extensive and the construction will generally be carried out in phases extending over a long period. Inevitably there will be interfaces with other infrastructure, especially at the seawater pipeworks section in between the basement and seawater pump cells, and with other parties who may be affected by the DCS. Besides, in consideration of heavy capital investment at the beginning stage, staging of system capacity and pipe networking in phase will also lead to carefully planning of the infrastructure system.

7.1.3.2 *Risk of early investment*

The upfront cost of the DCS is significant due to the infrastructure requirements including often; basement plantroom design and the adoption of service corridor/direct buried installation for the DCS piping network. If this part of cost is to be included in the DCS cost, instead of treating as “public works”, the attractiveness of the DCS as a business is not high. In the case of Marina Bay, the common services tunnel was provided by the Singapore Government.

7.1.3.3 *Cooling requirement planning and demand risk*

Under free market operation, the DCS operator is potentially exposed to great demand uncertainty for the following reasons. Since DCS is new in Hong Kong, some potential customers may be unwilling to use the system at all through fear that it might not be reliable. This could be the case in particular with potential customers from private development of the early phase of the development, when the DCS operator will have no established track record. Unless customers sign contracts containing volume guarantees, there is inherent uncertainty in demand due to building design and building use (which determines the internal cooling load of the building). At a time when much of the DCS investment is committed, none of the buildings in the district will have been constructed, and, for many buildings, detailed designs may not be available. The DCS operator may not have very good information about the market price of air conditioning, and thus it may not be able to guarantee that the price it offers potential customers will be attractive to them. Different potential customers may take different views on the likely cost of providing their own cooling - for example, a building operator with a subsidiary company that operates a contract maintenance business would probably be able to run its own cooling plant at a lower cost than one with less experience. Unless provisions or any other sorts of agreements can be negotiated to influence demand for DCS service, demand risk can materialize for the DCS owner and more so for third-party operators/investors if DCS is packaged as an investment case.

7.2 *Tariff Structure*

Tariff structure refers to the way that a fixed total cost of supplying a DCS service is divided into elements that could include a one-off connection charge, an annual connection charge, and one or more unit charges. Tariff structure is important for two reasons. First, by altering the timing of payments to the DCS operator, it alters the distribution of risk between the customer and the operator. Second, because the tariff structure determines the marginal price at which additional units of cooling are supplied, it influences consumption and supply decisions at the margin (i.e. it influences the likelihood that customers will seek to control their consumption of cooling services, and the efforts of the DCS operator to attract new business).

7.3 *DCS Installation within Private Development Premises*

Independent heat exchanger rooms are normally built at private development premises for the installation of heat exchangers between the DCS main loop and the building load. There will be pipework connection to the distribution pumps located in particular DCS pump rooms associated to each building. It will be necessary to coordinate with the future developer of the private development lots such that the DCS systems are in line with the building design. In addition, the cooling load of these buildings shall also be identified during the design and construction to ensure the cooling load will not be over the maximum capability of the heat exchangers and the corresponding chilled water pipework.

7.4 *Incentivizing the Solution*

Successful implementation of the DCS would benefit from some form of the owner (or the Government) support (required due to problems of timing and exposure to demand uncertainty). This support could take the form of incentives or guarantees of some kind.

The main difficulties identified in KTD DCS and WKCD DCS are that the DCS is only marginally profitable under the best case assumptions, and that the operator is potentially exposed to very significant revenue risks from delays in implementing the development programme and from customers being unwilling to connect to the DCS if they are allowed to choose. In principle, therefore, the Government could act to increase revenues, reduce costs, or reduce demand risk. Various options for achieving these three types of support are:

a) Cost to the Government
b) Effect on incentives
c) Link between incentives and performance

7.4.1 *Increasing revenues*

The revenues of the DCS operator can be increased if the Government is able to make the DCS service more valuable, for example by offering a direct financial incentive (subsidy).

There could be a relatively simple way of implementing the incentive; one of the potential benefits of the DCS is that space that would have been occupied by conventional air conditioning plant in the buildings of each potential customer is no longer required for this purpose. Under current planning procedures, this floor area is not included in the calculation of the GFA permitted by the planning and development process. Thus, under current rules, the benefit to the customers that connect to the DCS is that they simply do not have to construct the plant room they are not able to increase the useful floor area of their building. If this rule were changed so that the released floor area could be put to productive use, customers would be provided with a strong incentive to connect to the DCS.

7.4.2 *Reducing costs*

If the DCS operator, in the course of its business operations, has to make payments to the Government, the Government has the opportunity to reduce these payments and thus change the cost base of the DCS operator. This may appear exactly equivalent to subsidy payments that increase revenues, but might nevertheless cost the government less and be easier to implement practically. It might cost less when the opportunity costs of providing government – owned resources are lower than their current charges. It might be easier, from a political perspective, to present Government assistance to the DCS operator by reducing charges it pays to the Government. If charges levied by Government for the provision of publicly-owned resources are not set at the full economic opportunity cost of providing these resources then it might in any case be beneficial to reduce these charges.

7.4.3 *Increasing demand certainty*

If Governmental facilities and the Authority's facilities in KTD and WKCD are able to sign long-term contracts with the DCS operator, possibly before it has begun to build the DCS plant, then the operator has a potentially valuable guarantee of a significant proportion of demand. The extent to which these contracts would remove demand risk would depend on the details of the contract. This would give the operator a guarantee that a significant proportion of its potential customer base would in fact use the DCS rather than other cooling systems. However, it would not mitigate any of the demand uncertainty arising from uncertainty in the timing of the development, or uncertainty in the cooling load of individual buildings once connected to the DCS.

7.4.4 *Mandatory use of DCS*

Demand risk is identified above as having a potentially impact on the value of the DCS. Although DCS owner could guarantee demand from their own facilities relatively easily, guaranteeing demand from other private developments might be more difficult. As an alternative, it is recommended to require all commercial buildings, hotels and non-domestic developments to connect to the DCS (by making it a condition of the lease granted to developers that no alternative cooling sources be permitted on the site). Forcing mandatory connection to the DCS removes all of the uptake risk, although it does not remove all of the volume risk - there would still be potential for delays in the phasing of the district development, and for changes in the cooling demand of individual buildings. Nevertheless, mandatory connection would remove most of the demand risk. However, forcing customers to connect to the DCS precludes any kind of market mechanism acting to set the price of the DCS output equal to the cost of alternative cooling.

8 SMART GRIDS

Smart grids enable responsible energy distribution that is better able to cope with growing demand and new supply technologies.

A smart grid is an electricity network incorporating electricity and communications systems that can intelligently respond to nodes connected to it. Smart grids can also include storage and decentralized generation but their salient feature is the integration of high-speed bi-directional communications between systems and the grid.

Why are smart grids important for cities? Cities are driving increases in electricity demand against the existing, aged and congested grid infrastructure. It is critical that communication-enabled controls are integrated into urban networks to improve their operation and realize a more sustainable interaction with the power end users.

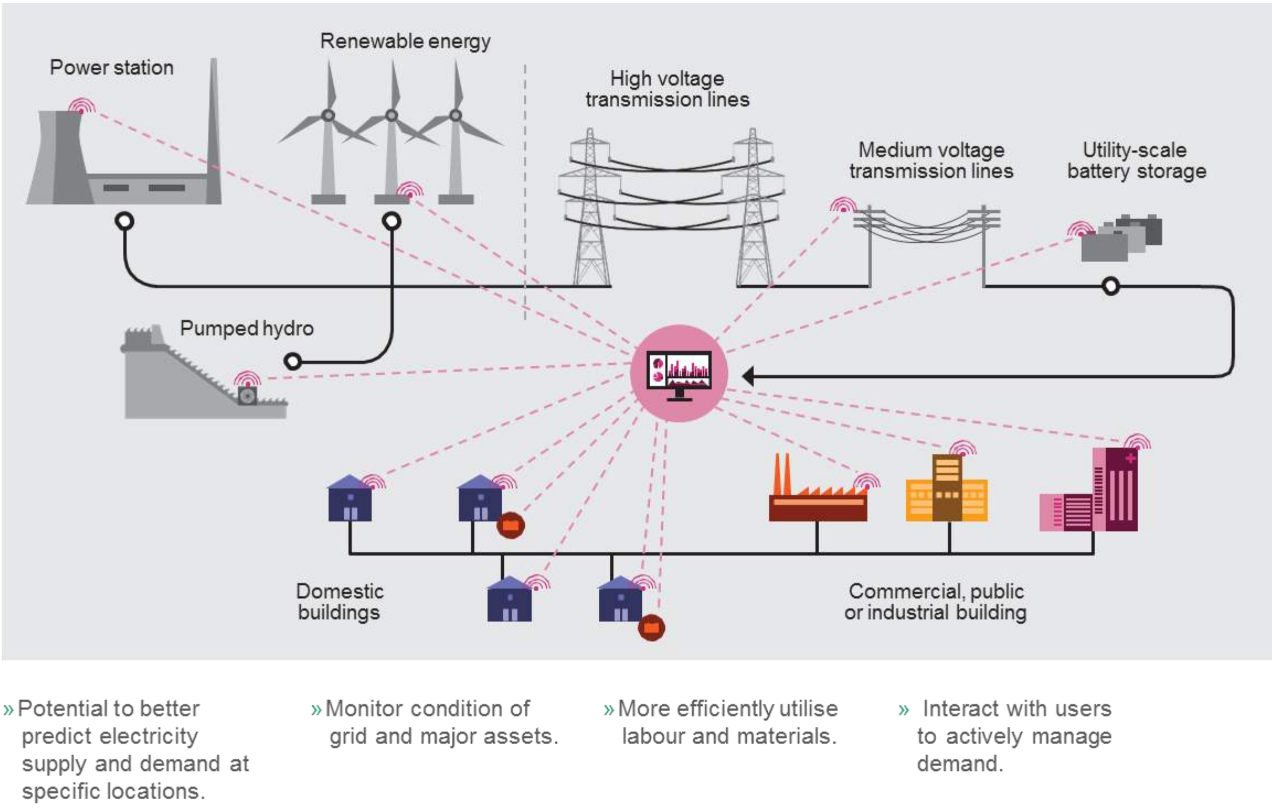


Figure 9

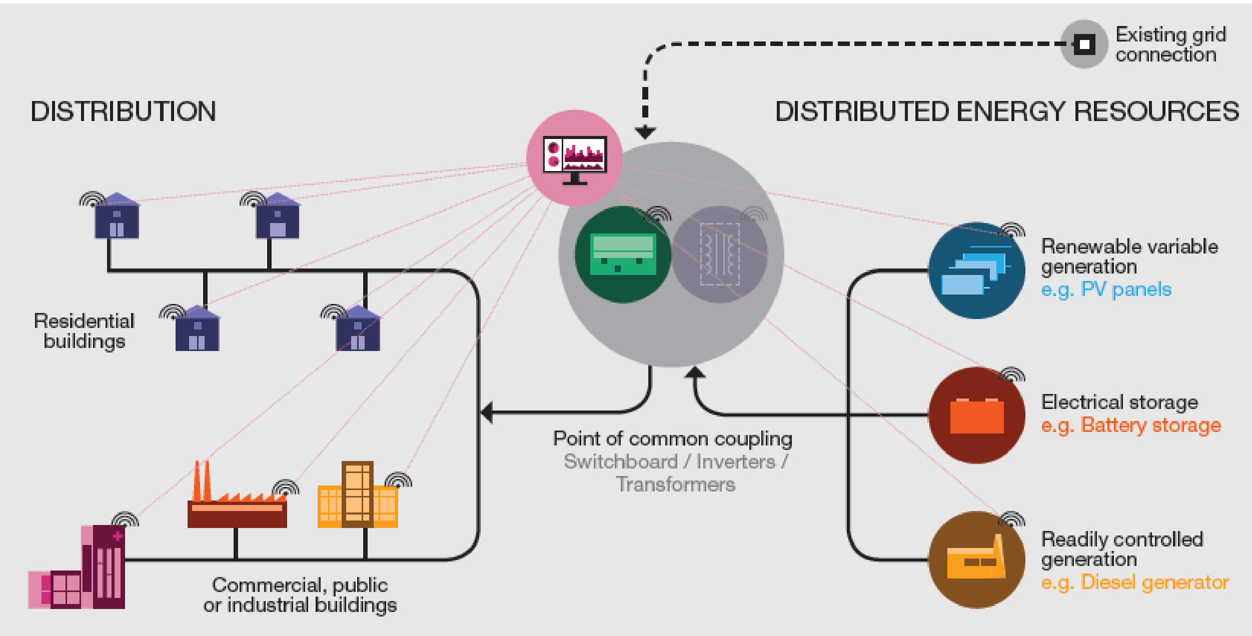
9 MICROGRIDS

A microgrid is a way to simultaneously address energy security, affordability and sustainability through dispersed, locally controlled, independent energy systems tailored precisely to end-user requirements.

A microgrid is a distributed level energy system which includes all the necessary components to operate in isolation of the grid. It is a microcosm of the broader energy network, but at a distributed level. Microgrids incorporate generation, storage and demand management systems so that supply and demand are matched in a safe, effective and reliable way. The smartness of microgrids means that these are compatible with renewable energy generation. Microgrids also alleviate pressure on the grid and enable more efficient supply due to optimized demand and supply balancing and reduced transmission losses.

Microgrids have a range of city applications including high-tech industries, mission critical operations and low carbon commerce.

Cities are increasingly becoming low carbon leaders keen to demonstrate their environmental responsibility while also positioning themselves as pioneers in adoption of sustainable technologies. This could be reflected in maximum deployment of renewable energy and demand side response within microgrids. Cities could accelerate wider adoption using policy and regulation to target users with high loads and available capital. These users might include real estate developers, universities and business parks. Microgrids are compatible with these high performing organizations as they enable maximum onsite sustainable energy contribution while providing the same level of reliability of supply as the conventional grid based supply. The ability to synchronize and dispatch energy to the distribution network or to rapidly reduce demand when Negawatt prices make it commercially attractive are also important features of this type of microgrid.



How can microgrids be part of the solution?

» Enable cities to take ownership of their generation and adoption of renewable energy systems.

» Can be designed for future connection to main grid. Opportunity for suburbs with poor quality or no connection to electricity supply to improve electricity access.

» Enhances resilience against threats including climate change and market volatility.

Figure 10

9.1 *Microgrid Implementation in Asia*

9.1.1 *Micro Energy Grid (MEG): Implementation of smart distributed energy*

A real planning project located in Hansung City, Korea was analyzed for the feasibility to adopt the Micro Energy Grid (MEG) concept. Hansung City is a Korean-Chinese collaboration to develop a mixed-use masterplan for a new sustainable city. The MEG serve all parcels of the 376 hectares (3.76km^2) site with utilities provided via a central energy center (CEC) and total control center (TCC) serving as the heart and brain of the city, respectively.

The MEG concept is developed around five key objectives:

a) Provide efficient, resilient & affordable district Electrical, Heating, & Cooling networks with real-time monitoring & control;
b) Utilize diversified loads across a number of properties to reduce plant size, maximize efficiency and reduce carbon emissions;
c) Reduce peak energy demand, particularly power, resulting in reduced investment in centralized power generation and networks;
d) Harness the energy cascade principle whereby ‘waste’ from power generation, waste treatment or industrial process is reused to increase overall efficiency; and
e) Promote behavior change in demand through feedback, benchmarking and incentives.

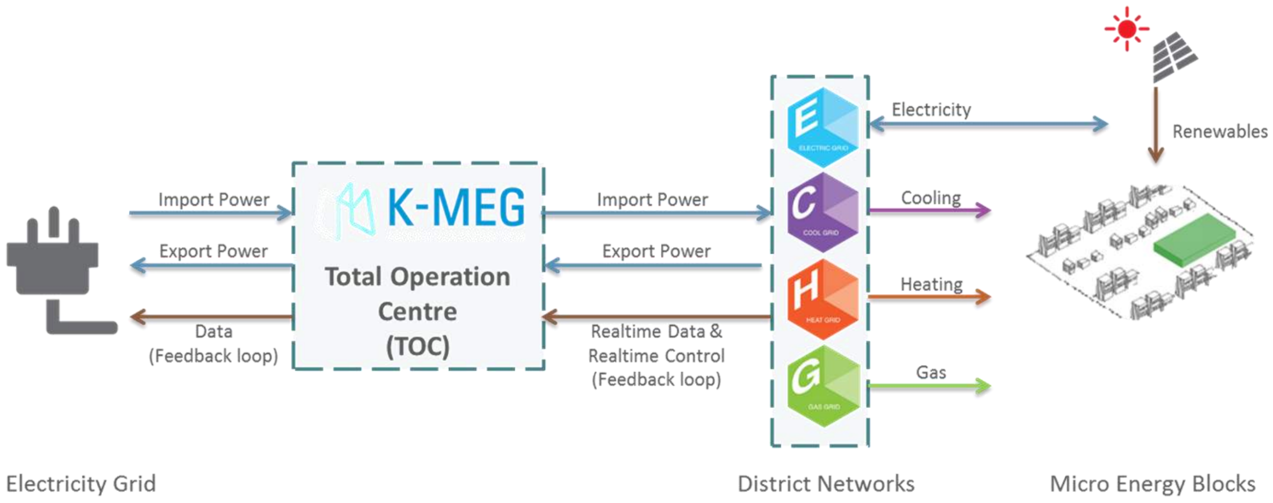


Figure 11 MEG concept

9.1.2 *Setting realistic targets*

Through the project development it was found that the feasibility of a MEG project varies dependent upon the Key Performance Indicators (KPIs), the building mix of development, carbon intensity of local utility energy and the regulatory framework. To drive development in the right direction, strong KPI’s are required at the project outset. Table 1 sets out those applied to the Hansung City development.

Table 1 Hansung City proposed KPIs

	KPIs
Carbon Emission Reduction	25.7%[(1)]
Peak Power Demand	28%[(1)]
On-Site Renewable Energy Generation	2%[(2)]

[(1)] When compared with Business as Usual (BaU) case
[(2)] Of site-wide energy consumption

9.1.3 *Strategy Development*

To meet the stringent KPIs set forth a fully coordinated, combined energy masterplan was developed. The MEG serves all development parcels, including private/public residential, public buildings and commercial properties. This variation in building type and load profiles represents a good mix for the deployment of MEG.

The MEG features a 36MW natural gas generator (sitewide peak demand - 140MW) to produce 'clean' power to the development, this power is fed directly to the district microgrid. In times of low demand the generator continues to run, feeding the excess power to the 20MW of solid state energy storage. The microgrid is supplemented with renewable energy from the 5MW of distributed Photovoltaics located on buildings with low demand during the generating period (i.e. Residential) and a connection to the utility grid for make-up power.

Heat generated as a by-product of electricity generation, which in traditional power stations is lost to the atmosphere, is harnessed and fed to 30MW of absorption chillers in summer to produce chilled water for the district cooling network. The district cooling network is supplemented with high efficiency water cooled chillers. In winter, the 'waste' heat is diverted directly to the district heating network. Renewable energy is further employed to provide local hot water heating to those building types with high hot water demand, such as hospitals and restaurants. The response of the MEG to the changing demand patterns for the peaks summer and winter weeks are indicated in Figure 3.

By providing local distributed energy provisions in the MEG format, it is possible for the Hansung City development to operate under 'island mode' if for any reason there is an interruption in the local power supply. Under this condition a load shedding strategy to reduce non-critical loads will be implemented and monitored from the TCC.

The Smart solutions employed here align with the 'Smart city' concept where the city is continually feeding back information, performing self-diagnostics and improving performance.

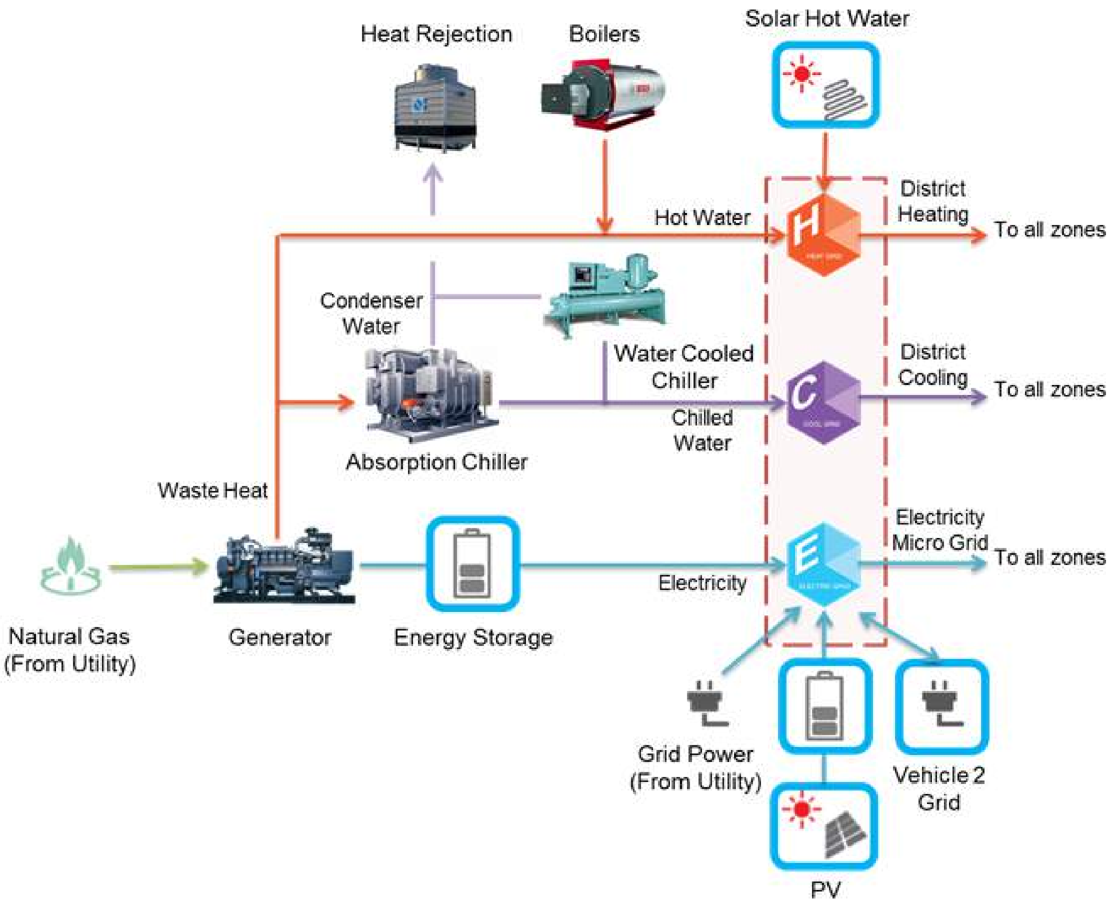


Figure 12 Hansung City MEG Schematic

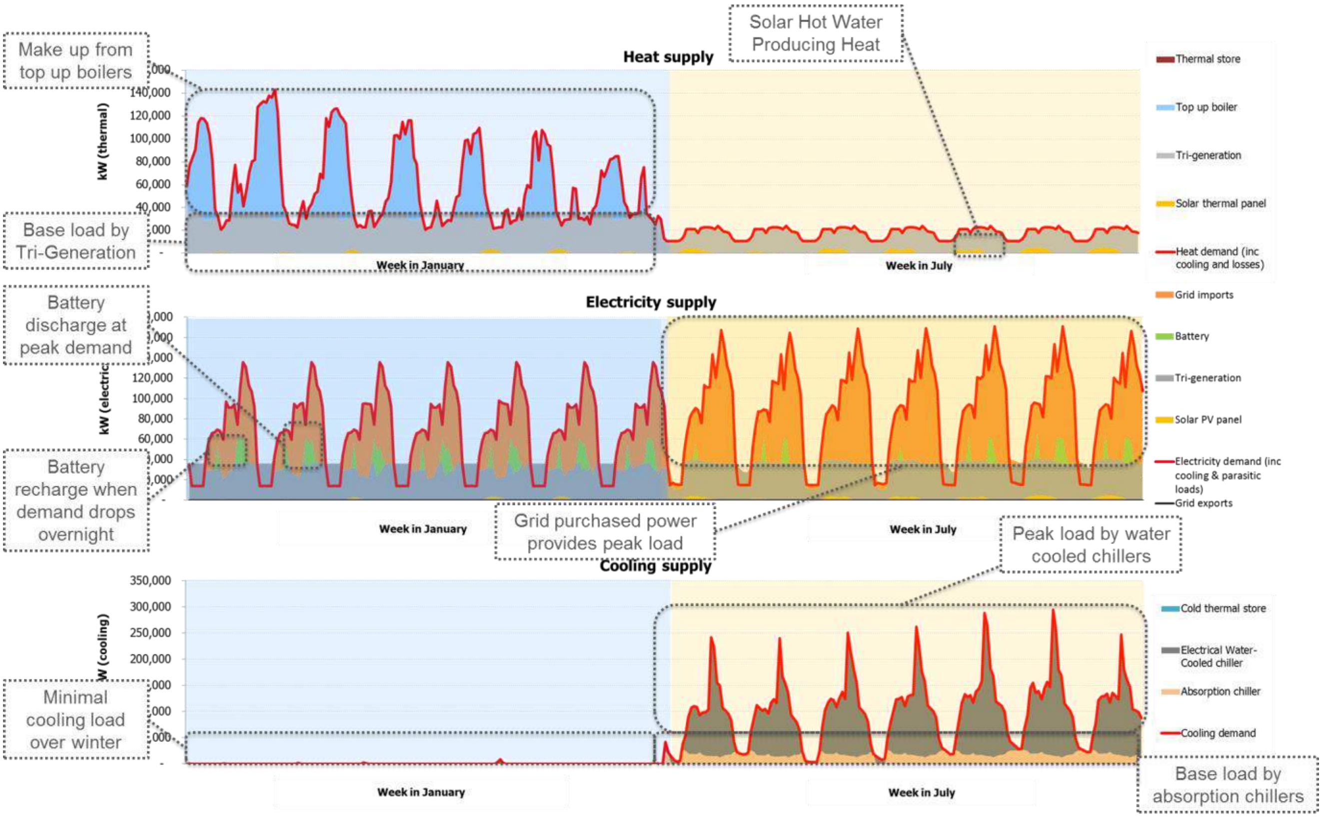


Figure 13 Daily load variations in winter and summer and the response of the MEG

To be fundamentally feasible and acceptable to the local utility provider, the MEG must offer some benefit otherwise resistance from the incumbent industry will exist. A challenge to power companies is maintaining capacity to cater for those times of peak demand, which may occur very rarely. The MEG concept allows for a reduction in the 'peakiness' of the demand by shifting the peak from the traditional peak hours, to the hours of low demand. In the case of Hansung City, the peak demand is reduced by around 28%.

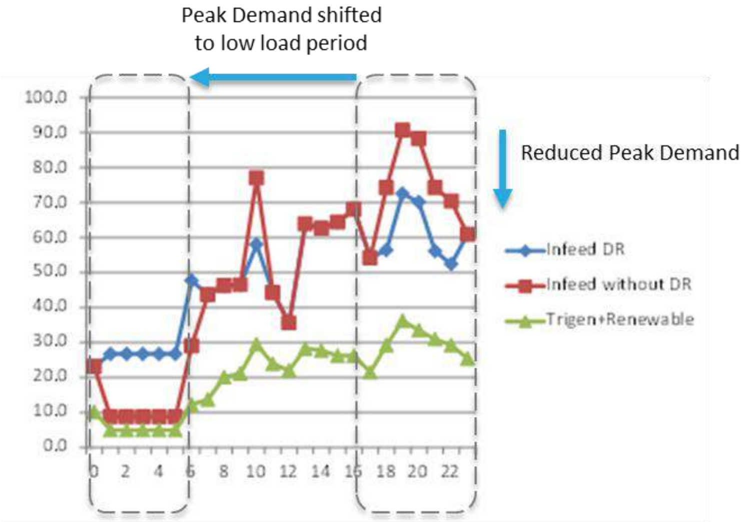


Figure 14 Peak power demand shift

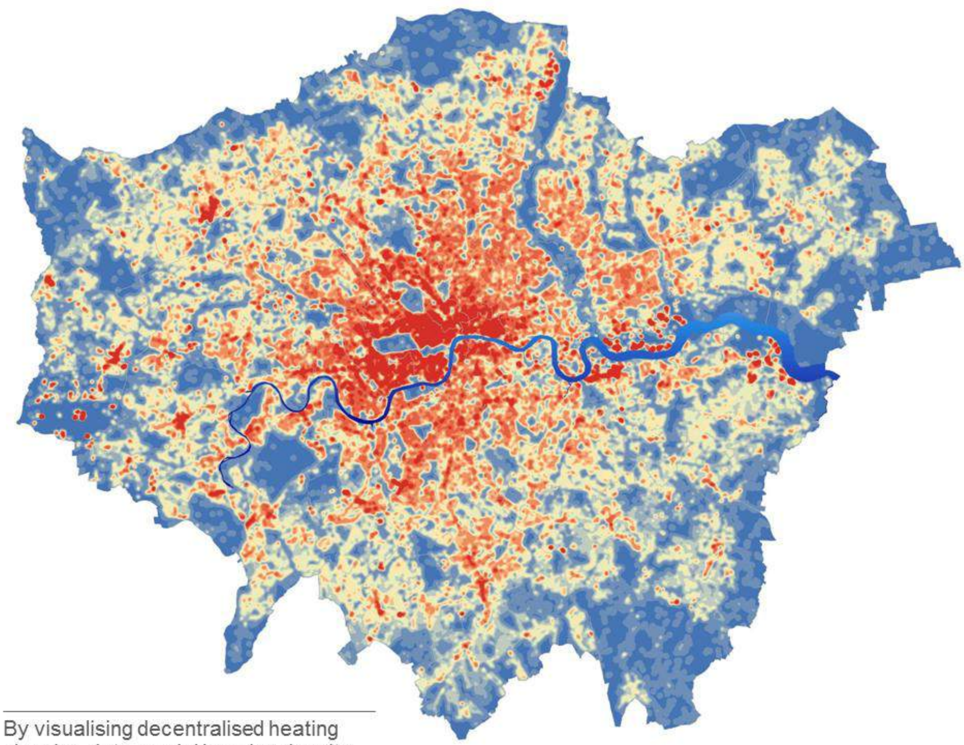
10 DATA IN SMART CITIES

Data enables two-way communications between energy systems facilitating proactive responses and increasing efficiencies.

Maximizing the value of energy data in cities – Cities are generating data every second of every day. Real time data availability is also growing, with the potential for improving energy management and enhancing the efficiency of other city functions. Digital information services e.g. data analytics, and visualization will improve cities capacity to identify risks, opportunities and future requirements for energy, allowing them to respond more effectively to urban challenges such as social equality and fuel poverty. To support cities identifying opportunities and planning for sustainable energy projects, some data-driven planning tools are now available to the public, e.g. the UK National Heat Map, and the London Heat Map. As data proliferates and technologies to utilize it advance, opportunities for more accurate energy balancing will increase, helping to lower costs and carbon and improve reliability of energy supply in cities.

Smart approaches, found on technical expertise, sustainable, integrate thinking and policy considerations, pave the way for city decision-makers to respond effectively to energy related issues that impact climate change.

Smart systems encourage change sin behaviors which prompt new choices and activities, creating a shift in the way we consume and share energy, and hence, contribute to the development of low carbon urban economies and societies. A new generation of integrated hardware, software and network technologies will provide systems with real time information of the world. Advanced analytics will help people to make more intelligent decisions about alternatives and actions that will improve how we use energy and resources. The increasing penetration of smart and data-driven technologies is helping to drive smart cities. Cities must facilitate expansion of the industry through appropriate regulation, incentives and governance.



By visualising decentralised heating planning data, social housing density data and policy indicators, the **London Heat Map** allows stakeholders to identify opportunities and assess the feasibility of decentralised energy deployment in London.

Figure 15

An urban development project in Helsinki features a wide range of informatics strategies and services aimed at reducing the energy demand and carbon footprint of the local community.

Figure 16

11 CONCLUSION

The eco-efficiency in urban construction on Energy in Cities has been discussed. A plan to help cities reduce energy demand, enhance efficiency of supply and increase use of low carbon and renewable energy has been described. The systems and technologies featured in this paper were selected to show that the innovative energy solutions that could enable cities to achieve successful energy transitions. Hong Kong is currently at the important juncture, undergoing the review of the development of Electricity Market and the improvement of the regulatory framework when the current Scheme of Control Agreements expire in 2018. A comprehensive overview of Energy in Cities Concept is vital to give a fuller perspective.

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Innovative installation techniques for PHC piles

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Keywords: PHC piles; environmental friendliness; carbon footprint; daido piles; ground improvement; jacked piles; prebored rock socketed piles; prebored PHC piles.

ABSTRACT: The high population density in Hong Kong necessitates the adoption of heavy infrastructures and high-rise buildings for public and private uses. These heavy infrastructures and high-rise buildings require the support of pile foundations. Among different pile types, the prestressed spun high strength concrete (PHC) pile is probably the most economical. Moreover, it has a smaller carbon footprint than the steel H-pile of comparable design load-carrying capacity. Installation of PHC piles by percussion is probably the most economical. However, noise, vibration and air pollution problems are often inseparable from percussion of piles. Innovative installation techniques for PHC piles to achieve environmental friendliness, quality and rapid construction, and economy are always in demand. Depending on geologic conditions, different innovative installation techniques for PHC piles have been developed and these techniques are presented in this paper.

1 INTRODUCTION

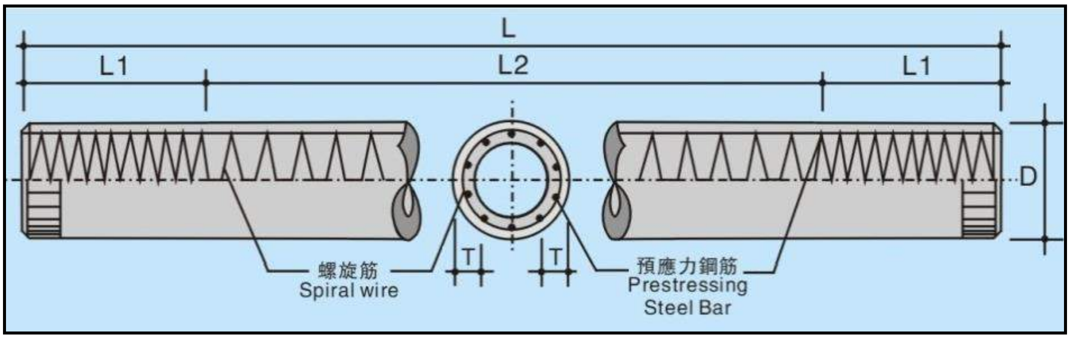
The total area of the Hong Kong Special Administrative Region (HKSAR), China is 2,755km^2, including 1,104km^2 of land and 1,651km^2 of sea. Hong Kong is accommodating a population of approximately 7.1875 million as of mid-2013 and one of the most important financial and trading centers in the world. The average population density of the HKSAR is 6,650 persons/km^2 as of mid-2013. However, most of the population is being housed in 215km^2 of urban development because of steep natural terrain and stringent planning controls. Over 400km^2 have been designated as protected areas including country parks, special areas and conservation zonings. As a result, the most densely populated District Council district is Kwun Tong with a population density of 57,120 persons/km^2. The high concentration of population and economic activities in such a small area exert an intense demand for infrastructures to sustain the rapid growth of Hong Kong. The demand necessitates the adoption of heavy infrastructures and high-rise buildings for public and private uses. These heavy infrastructures and high-rise buildings require the support of pile foundations. Most of these pile foundations are constructed in the vicinity of commercial and residential activities. Innovative installation techniques for piles to achieve environmental friendliness, quality and rapid construction, and economy are thus always in demand. Innovative techniques for the installation of Prestressed spun High strength Concrete (PHC) piles are presented in this paper.

2 THE USE OF PHC PILES IN HONG KONG

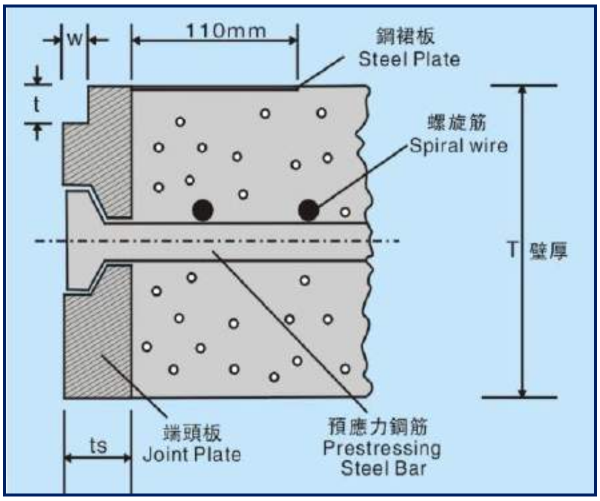
PHC piles are typically manufactured by spinning wet concrete in a formwork with pre-tensioned wires installed. The compressive strength of concrete is approximately 80MPa. The tensile strength of pre-tensioned wires is approximately 1,420MPa. The outer diameter of the pile D varies from 300 to 600mm and the thickness T varies from 70 to 130mm as shown in Figure 1. The design load-carrying capacity of the pile is thus ranging from 900 to 3,500kN. Therefore, many choices of different piles of different design load-carrying capacities are available to satisfy foundation needs. Piles are connected by welding the steel joint plates together to the required total length.

PHC piles are historically known as Daido piles in Hong Kong although the name may be technically incorrect. The original Daido piles were made in Japan where they are installed as replacement piles, as they are inserted into prebored holes followed by grouting of the annular void space between the pile and the prebored hole. However, they are installed by driving in Hong Kong. Since the method of installation has been changed, construction problems such as damage of pile shoe, crushing of concrete near the pile tip, damage of pile head, occurrence of tensile cracks in the pile etc., arise in Hong Kong. Later, Daido piles were manufactured in Hong Kong and similar piles of lower quality manufactured in mainland China also enter the market, resulting in more construction problems due to poor quality unanticipated by the original Daido pile designer. As a result, the use of PHC piles in Hong Kong has diminished for more than a decade.

Recently, the advantageous use of PHC piles in karst formation has stimulated new interest in the use of PHC piles in Hong Kong. However, these piles are still installed by percussion, resulting in noise, vibration and other construction problems, although air pollution problems have been alleviated by the use of hydraulic hammers. It is evident that there is a need for innovative installation techniques for PHC piles in Hong Kong.



(a) Pile details



(b) Joint details

Figure 1 Details of the PHC pile

3 ADVANTAGES AND DISADVANTAGES OF PHC PILES

The design load-carrying capacity of the PHC pile is comparable to that of the steel H-pile. It should be noted that the design load-carrying capacity of a PHC pile of 600mm in diameter and 130mm thick is approximately 3,500kN, similar to that of a Grade 55C 305×305×223kg/m steel H-pile. Moreover, the PHC pile have these advantages over the steel H-pile of similar design load-carrying capacity:

a) The material cost of steel H-pile is approximately HK$1,200/m while that of PHC pile is approximately HK$600/m. Assuming the required pile length is similar, the cost/ton of support load of the PHC pile is approximately a half that of the steel H-pile. Typically, the required length of the PHC pile is shorter than that of the steel H-pile to achieve the similar design load-carrying capacity on site, as the side resistance normally developed along the shaft of the PHC pile is higher than that of the steel H-pile.
b) The bearing stress at the tip of the PHC pile is smaller than that of the steel H-pile as a result of larger bearing area and higher side resistance. Therefore, the PHC pile is particularly useful in areas of karst formation in Hong Kong, such as Yuen Long and Tin Shui Wai, for reduction of foundation stress exerting on the marble rockhead.
c) Taking a carbon footprint of (0.76 ton CO_2)/(ton of structural steel) and (0.155 ton CO_2)/(ton of reinforced concrete), the carbon footprint of the steel H-pile is 0.17 ton CO_2/m and that of the PHC pile is 0.07 ton CO_2/m. Use of more PHC piles can be a significant contribution by the construction industry towards a cleaner and more sustainable environment.
d) There are many high quality PHC pile manufacturers in the vicinity of Hong Kong. It is much easier to satisfy the material requirements of LEED or BEAM by using PHC piles than using steel H-piles in terms of carbon footprint and transport distance of construction materials. Shortening of transportation distance contributes indirectly to the reduction of fossil fuels and air pollution.

Although PHC piles can be cut on site to suit ground conditions, the cut PHC piles cannot be reused, resulting in the generation of construction waste. However, PHC piles are manufactured in different lengths. If the founding level can be reasonably predicted prior to installation, the quantity of construction waste so generated can be minimized.

4 INNOVATIVE TECHNIQUES FOR THE INSTALLATION OF PHC PILES

Depending on ground conditions, three experience-proven installation techniques for PHC piles with minimal vibration and noise are now available on the market. These techniques are presented in the order of the status of acceptance in Hong Kong herein:

4.1 *Prebored Rock Socketed PHC Piles*

When the rockhead is shallow, the design load-carrying capacity of the PHC pile can be derived economically from a rock socket, similar to the prebored rock socketed steel H-pile. A pile bore is prebored to rockhead with the borehole wall supported by a temporary steel casing. A rock socket is drilled using a drilling bit system that can go through the temporary steel casing and extend at the bottom of the temporary steel casing. The structural element of the pile is inserted after cleaning of the pile bore. The pile is grouted while the temporary steel casing is extracted. For the prebored rock socketed steel H-pile, the structural element is the steel H-pile so that it is inserted to the bottom of the rock socket. For the prebored rock socketed PHC pile, the structural element has two components, i.e. the portion above the rock socket and the portion in the rock socket. The portion above the rock socket is a conventional PHC pile. However, the portion in the rock socket warrants a different design to ensure the design load-carrying capacity of the pile can be derived from the bond strength between the pile and the vertical cylindrical surface of the rock socket. Three different types of rock sockets, i.e. Types A, B and C, are depicted in Cheung et al. (2012). All the three different rock socket details use a structural component welded to a 50mm thick steel capping plate which is welded to the steel joint plate at the tip of the PHC pile. Type A rock socket component uses a pre-fabricated reinforcement cage welded to a annular steel capping plate. Type B uses a steel H-section welded to a solid capping plate. Type C uses a pre-fabricated reinforcement cage welded to a solid steel capping plate. A bundle of 50-mm diameter high yield reinforcement bars are inserted into the central hollow space of the PHC pile to the bottom of rock socket for Type A and to the solid steel capping plate for Types B and C to maximize the design load-carrying capacity of the pile.

In Hong Kong, the pile drilling machine for the construction of prebored rock socketed steel H-pile is used for the construction of prebored rock socketed PHC piles, i.e. pile bore of diameter 610mm. As a result, only PHC piles of diameter 500mm and thickness 100mm or 125mm can be inserted into the steel temporary casing. Moreover, 8 50-mm diameter reinforcement bars can be inserted into the hollow space of the 100-mm thick PHC pile and only 7 can be inserted into the 125-mm thick pile, resulting in a design load-carrying capacity of approximately 5,000kN which is comparable to a Grade 55C 305×305×223kg/m steel H-pile. The pile type is recognized by the Buildings Department (BD) of the HKSAR Government as Rock Penetration Composite Piles Types I to III after a successful field test performed in 2006. The new pile type with Type A rock socket component was later successfully adopted to support a church building in Hung Shui Kiu, Yuen Long (Cheung et al. 2012). In Macau, a larger pile drilling machine was used for a school project so that temporary steel casings of 714 mm in diameter could be used to accommodate 600-mm diameter PHC piles. As a result, the design load-carrying capacity of the pile well exceeds that of Grade 55C 305×305×223kg/m steel H-pile at a considerable lower material cost.

4.2 *Jacked PHC Piles*

Jacked piles were originally developed for underpinning and foundation remediation works (White, 1975; Tomlinson and Woodward, 2014). Piles are jacked into the ground continuously by a hydraulic jacking system. The pile installation technique has been widely and routinely adopted in many Asian economies such as China, Japan, Singapore, Malaysia etc. (Li et al., 2003; Li, 2011; Li et al., 2011; 廣東省土木建築學會, 2012). The technology is also known as press-in technology in Japan where extensive research has been conducted by the industry. The International Press-in Association (IPA), an academic organization supported by Giken Seisakusho Co. Ltd. of Japan, was established on 16th February 2007 in Cambridge, England to explicate the underground phenomena and mechanisms encountered by the press-in technology in close coordination with various technical fields such as civil, construction, environmental, geotechnical, instrumentation and mechanical engineering. Financial support is provided to researchers in the form of IPA Research Awards. The author has received three of these research awards. Moreover, technical seminars are organized worldwide biannually to dissimulate new developments of the technology.

Jack piling has many advantages over pile driving. No noise or vibration is generated by the jacking process. As a result, there is practically no limitation on operation hours, resulting in a considerably higher daily production rate than that of pile driving. The hydraulic system is powered by electricity and no black smoke due to incomplete combustion of diesel is generated. Therefore, most adverse impacts of pile percussion on the nearby environment or sensitive receivers are eliminated. The technique makes it possible to install piles in close proximity of existing buildings or sensitive site areas such as underground structures of the subway system (Yeung, 2013). Jack piling is more cost effective and environmentally friendly. In practice, driven piles are often over-driven to ensure the required design load-carrying capacity can be achieved, resulting in unnecessarily longer piles. Jacked piles are hydraulically jacked to the target design load-carrying capacity with no unnecessary extra penetration, resulting in considerable savings in material, labor costs and construction time. More importantly, every jacked pile is fully load tested during construction for better quality assurance. For PHC piles, as no tension is generated in the pile during jacking, gas arc welding can be used to connect sections of pile, resulting in a significant reduction in construction time and potential damage to the pile. Many construction problems associated with percussion of PHC piles are also eliminated.

As the load-carrying capacity of the pile is increased, the reaction weight required is increased proportionally. The dead weight of the pile jacking machine for the installation of piles of design load-carrying capacity of 3,500kN is approximately 10,000kN. Therefore, the technology is not applicable for construction sites of small footprints and/or steep terrain, as it is very difficult to maneuver such a large and heavy pile jacking machine under these circumstances.

Steel H-pile is a small displacement pile and PHC pile is a large displacement pile. Therefore, their engineering behavior is not the same, resulting in differences in the construction equipment and procedure, and termination criteria. Moreover, all the potential disadvantages of displacement piles have to be carefully considered and necessary precautions for proper installation of piles have to be taken during construction.

In Hong Kong, applications of jacked piling are limited to steel H-piles and steel sheetpiles to date (Yeung, 2002; Yue and Ho, 2002; Li et al., 2003; Yang et al., 2006; Li et al., 2011). However, jacking of PHC piles is very common in Macau and China (Yeung and Li, 2012).

4.3 *Prebored PHC Piles*

The technique of installing PHC piles by preboring was introduced to Macau from Japan where the technique was developed. The technique is basically a combination of ground improvement and pile installation. A specialized equipment combining an auger and a soil mixer as shown in Figure 2 is penetrated into the soil to break up the soil to the required depth by an electric motor. A minimal amount of soil is removed during the process. During withdrawal of the equipment, base-forming and side-forming fluids are injected into the soil and mixed well with the soil to increase the tip and side resistance of the soil. The combined auger and soil mixer has to travel up and down a few times during the process.

Figure 2 Auger and mixer used for pile installation

Figure 3 Mechanical splicing of PHC piles

After the combined auger and mixer has been withdrawn, a 12-m long PHC pile is sunk into the liquefied soil by its self-weight. No pile shoe is necessary for the installation. As the PHC pile is hollow, some of the liquefied soil would fill the central hollow space of the pile and overflow. Therefore, the amount of soil displacement generated by the insertion of the pile is minimal, in particular when the surrounding soil is already liquefied by the installation process. The pile is technically a non-displacement pile.

PHC piles are mechanically spliced by screws. The required tightening torque exerted on the screws is measured by a torque meter. The screw tightening sequence is also clearly indicated as shown in Figure 3 and it is strictly followed on site. The quality control and assurance of the mechanical splicing are thus significantly better than welding and no testing of the splicing is required afterwards, so as to shorten the time required for splicing. After the last PHC pile has been installed, the PHC pile is screwed into the ground by the electric motor through the use of a specifically designed helmet. The installation process is complete after the soil has been cured to the required strength. The technology has been applied successfully for the foundation construction of a secondary school in Macau.

The complete construction process is recorded automatically by an on-board computer in real-time for quality control and assurance. The movement of the auger head, the amount of base forming fluid used, the amount of side-forming fluid used etc. are recorded automatically and continuously as a function of time. Typical construction records for a prebored PHC pile are shown in Figure 4.

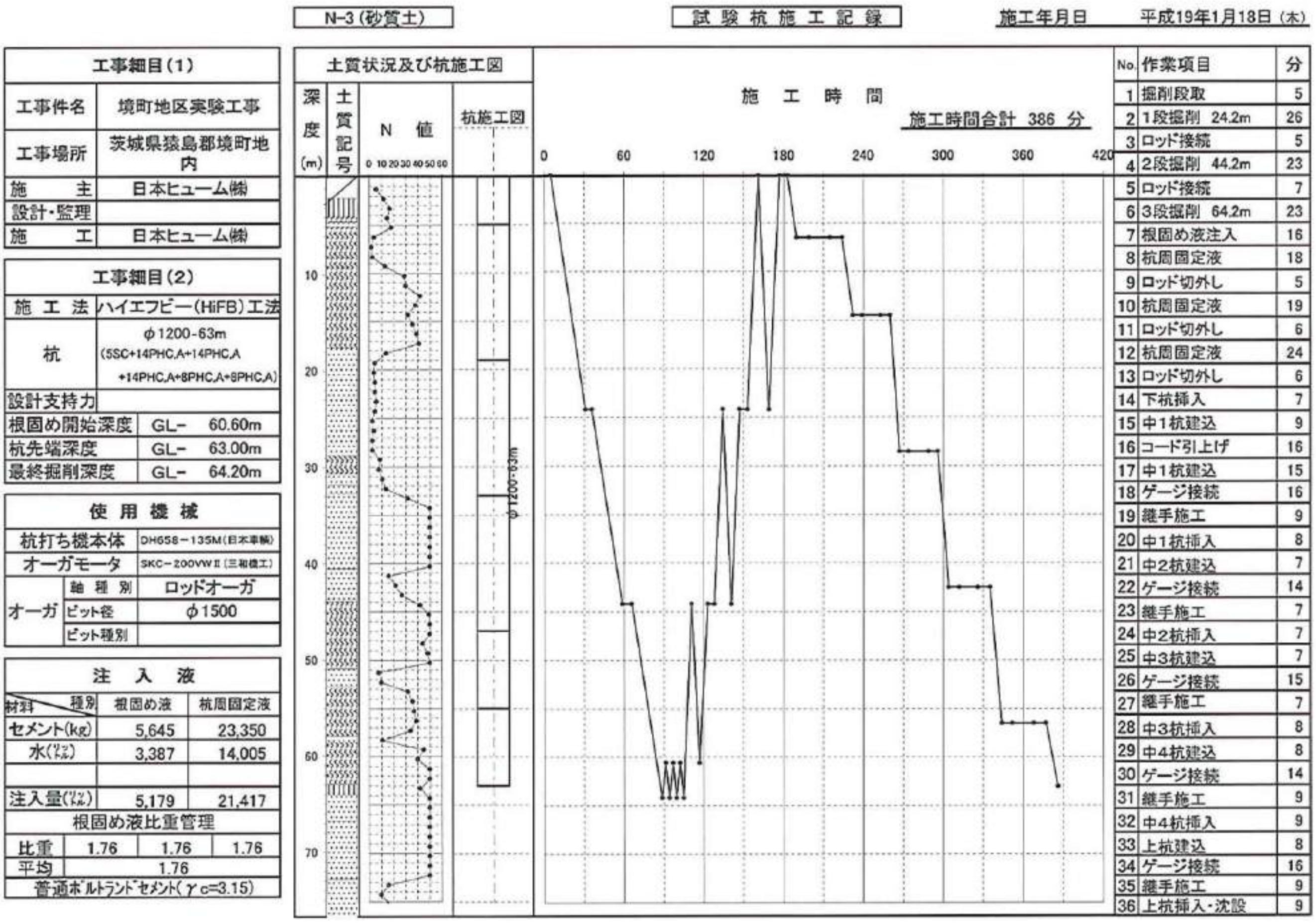


Figure 4 Typical construction records of a prebored PHC pile (in Japanese)

5 OBSTACLES TO INNOVATIONS

The obstacles to innovative technologies in geotechnical works in Hong Kong were identified and frankly voiced by Li and Lo (2012) as follows: (1) government over-control; (2) McDonaldization of university research and teaching; (3) McDonaldization of consulting services; (4) protection of self-interest; and (5) human psychological barrier to innovations. All these obstacles are probably equally applicable to innovations in all other disciplines of civil engineering.

At the time of writing, the prebored rock socketed PHC pile is already a pile type recognized by the BD of the HKSAR Government after a successful field test of the new pile type in 2006 (Cheung et al., 2012). The new pile type was later successfully adopted to support a church building in Hung Shui Kiu, Yuen Long (Cheung et al., 2012). The current non-technical obstacle is the human psychological barrier of the BD approving officer and that of the foundation designer for public works to this new pile type. However, the maximum design load-carrying capacity of the pile is currently limited to 5,104kN on the basis of the pile configuration used in the test pile program, which was a 500-mm diameter PHC pile with a bundle of 50-mm diameter high yield reinforcement bars in the central hollow space of the pile. In Macau, one hundred three (103) prebored rock socketed PHC piles were successfully constructed using 600-mm diameter PHC piles to support a new school building. The maximum design load-carrying capacity of the pile is 6,412kN which is very competitive to the Grade 55C 305×305×223kg/m steel H-pile. Therefore, the remaining hurdle is to overcome this unnecessary bureaucratic obstacle on the technical issue of allowable design load-carrying capacity.

Although the technology of jacking steel H-piles was introduced to Hong Kong approximately 17 years ago (Yeung, 2002, 2014; Li et al., 2003) and it has been employed by a number of public development projects (Chan et al., 2002; Li et al., 2003), it is still not a pile type generally recognized by the BD of the HKSAR Government. Its use for private development projects is being approved on a project by project basis. Jacked steel H-piles have been approved for use in only one private development project in Tai Po to date. The proposal of trying jacked PHC piles for the project was rejected by the BD on the ground that there was no such past experience in Hong Kong. However, thousands of 600-mm diameter PHC piles have been successfully installed by jacking for casino and resort projects in Macau (Yeung and Li, 2012; Yeung, 2014), not to mention even more were successfully completed in mainland China and many other Asian economies. Such ground of rejection is a classic example of unnecessary government bureaucratic over-control and government officers' lack of ability to think out of the box. If there is no trial on a new technology, there will be no approval. When there is no approval on the new technology, there will be no past experience. When there is no past experience of the new technology, there will be no trial on the new technology. This vicious and hopeless circle of arguments will never lead to any technological breakthrough. The implementation of the proven and environmentally friendly technology of jack piling is practically hindered by bureaucratic government procedures.

Prebored PHC piles have not been introduced to Hong Kong yet. However, obstacles for its implementation in Hong Kong are predictable from the terrible past experience of jacked PHC piles. Hopefully it will not take fifteen years to get the first project approved by the BD.

Quality control and assurance of PHC piles may be another obstacle, as there are too many manufacturers of variable quality in the market. It is not an easy task to confirm all the PHC piles in a particular shipment are from the same manufacturer due to the complicated transportation logistics. However, such problems can be easily solved by tracking each pile in real time by the use of a RFID embedded in the pile and a global positioning system (GPS) in conjunction with the internet. The tracking technology is readily available.

The supply of PHC piles in Hong Kong is practically a closed market as controlled by the listing system of the BD. The system may function as quality control and assurance measures if pile quality is being monitored regularly. Unfortunately, this is not the case. There are listed PHC pile manufacturers which have gone out of business for quite some time and the BD has no knowledge about it. Although there is a stringent application procedure to be listed, there is no regular monitoring of the performance of the manufacturers on the list. Poor manufacturers are giving PHC piles a bad reputation, rendering another unnecessary obstacle for the use of PHC piles in Hong Kong. The obstacle can be easily overcome if the BD has a quality monitoring and registration renewal system in place.

The increase in concrete strength or diameter for the PHC pile may require many rounds of bureaucratic approval procedures which may become a deterrent to the development of new technology and/or new high strength material.

6 CONCLUSION

The foundation is one of the most important structural components critically impacting the engineering performance of an infrastructure or a high-rise building. The pile foundation is the most common foundation type for heavy infrastructures and high-rise buildings in Hong Kong. The material cost of the PHC pile is considerably lower than that of the steel H-pile. Moreover, the carbon footprint of the PHC pile is smaller than that of the steel H-pile. If innovative techniques can be developed for efficient and quality installation of PHC piles, the use of PHC piles can be more efficient, economical and environmentally friendly than those of other pile types. Three different experience-proven innovative installation techniques have been presented in this paper. Moreover, obstacles to the implementation of these techniques in Hong Kong and solutions to overcome some of these obstacles are also presented. It is evident from many successful case histories that there are no insurmountable technical obstacles for the implementation of these innovative technologies in Hong

Kong, although there may be some solvable technical issues to be overcome. However, most obstacles arise from administrative blockades imposed by the HKSAR Government. Therefore, it is a matter of the determination of the HKSAR Government to implement these cost-effective, efficient, quality assured and environmentally friendly PHC pile installation techniques in Hong Kong.

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A new approach to determine the polymer content in pre-packed polymer modified mortars

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Keywords: polymer modified mortar; polymer content; thermogravimetric analysis.

ABSTRACT: Polymer modified mortars are widely used in concrete repairing works and patching works as a result of their superior performance compared to conventional mortars. For the sake of quality control, the polymer content of such mortars must be assured. There are two main forms of polymer modified mortar supply in the market, namely polymer latex modified mortar (wet mix) and pre-packed mortar (dry mix). The solid components of polymer latex modified mortar and the latex solution are batched on-site according to the designed mix proportions. On the contrary, all the solid ingredients of pre-packed mortar are blended in the factory and mixed with water on-site. Conventionally, the established method of determining the polymer content in hardened mortars adopted by the public sector in Hong Kong is by measuring the mass loss of acid-digested hardened mortar upon heating to 550^{o}C, such as the method stipulated in the Hong Kong Housing Department's standard. However, the polymer content measured using such method has been found to show large difference from the actual polymer content within the mix. For pre-packed polymer modified mortars (which are majority and commonly used by the construction industry), this problem can be overcome by a new approach based on thermogravimetric analysis to determine the polymer content. The principle of this approach is based on determining the mass loss of the pre-packed mortar sample upon heating to a temperature of 800^{o}C. One advantage of this approach over the conventional approach is that calibration could be performed with respect to the results of thermogravimetry, such that the accuracy of measurement could be much improved. The polymer contents of blended mixes of polymer modified mortar (dry mix) were measured using the proposed new approach. The experimental results verified the validity of this approach.

1 INTRODUCTION

Compared to conventional mortars, polymer modified mortars are advantageous in several ways. The polymer can modify and enhance the cementitious system to achieve the following: Firstly, it possesses plasticizing property, leading to improved workability and reduced shrinkage as less mixing water is required. Secondly, it enhances the bond and adhesion between the mortar and the concrete substrate. Thirdly, it reduces the permeability to water and deleterious chemicals. Finally, it increases the tensile and flexural strength of the mortar (Ohama, 1995). Over the past decades, polymer modified mortars have been widely used in concrete repairing works and patching works. Common usages of polymer modified mortars include cement renders (to apply on surface of walls), masonry mortars (to join bricks or concrete blockwork), tile adhesives (to adhere tiles to floors and walls), sealing slurries (to provide waterproofing barrier for water sensitive elements), repair mortars (to fill cracks or patch repair areas in buildings, pavements, and footpaths), self-levelling mortars (to level floor surfaces), and skim coats (to form thin smooth cover layers for floors and walls) (Zhang, 2005; Wan, 2005).

There are two main forms of polymer modified mortar supply in the market, namely polymer latex modified mortar (wet mix) and pre-packed (or pre-bagged) mortar (dry mix). The solid components (mainly cement and sand) of polymer latex modified mortar and the latex solution such as styrene butadiene rubber (SBR), acrylic and modified acrylic latex are batched on-site according to the designed mix proportions. On the contrary, the solid ingredients of pre-packed mortar including cement, sand, fillers, additives together with pre-dried polymer powder such as poly-ethylene-vinyl-acetate (EVA), poly-styrene-acrylic-ester (SAE) and polyacrylic ester (PAE) are blended in the factory and mixed with water on-site. To enable pre-packing, specialized technology is required for producing pre-dried re-dispersible polymer, i.e. the specially formulated latex polymer is pre-dried by spraying in micro-globules, condensed to powdery form and stabilized, and then blended with mortar binders, fillers and additives which can improve rheological and physical properties of the mortar, and facilitate the re-dispersion of polymer when mixed with water in fresh mortar (Wan, 2005; Nguyen et al., 2014).

For the sake of quality assurance, the type of polymer used in and the polymer content of such mortars must be ascertained. Before the turn of century, wet mix mortar was more common in comparison with dry-mix mortar. As the latex solution was added on-site to the solid ingredients, the burden of verifying the type and content of polymer rested largely on the front-line site supervision, and there were practical difficulties to ensure the right type of polymer was added to the mortar in right contents. As a means of post-application quality verification, methods have been established to identify the polymer and to measure the polymer content in hardened mortars (Cheng, 1989). The method established was adapted by the public sectors in Hong Kong, and details of the method can be found in HKHA Standard 002 : 1990 (Housing Department, 1991).

With the advent of pre-dried re-dispersible polymer production technologies, pre-packed mortars are nowadays the majority and are vastly used by the construction industry. Pre-packed mortars possess the merits of consistent performance and stringent quality control from automated manufacturing process in specialized production plant. In contrast with wet-mix, the burden of verification of polymer types and contents in dry-mix is lessened on-site, but transferred upstream to the quality control of pre-packed mortar producers and confirmation of correct supplies of mortar products. The conventional method of determining the polymer content in hardened mortars, when applied to pre-packed materials, is not in tandem with the shifting of quality control to the upstream. Moreover, possibly due to the differences in polymer and cementitious systems between wet mix and dry mix mortars, the test method may not realistically reflect the polymer content in dry mix mortars, as shown by the measurement results reported in later sections.

To address the above issues, in this research study, the authors propose a new approach to determine the polymer content in pre-packed polymer modified mortars based on thermogravimetric analysis (TGA). Details of the approach are explicated in this paper.

2 CONVENTIONAL EXPERIMENTATION APPROACH

The conventional approach of determining the polymer content in polymer modified mortars was developed in late 1980s and is applicable to hardened mortars (Cheng, 1989; Housing Department, 1991). The method of experimentation is described as follows.

2.1 *Sample Preparation*

From the subject hardened mortar to be tested, 100 g to 150 g of test sample is obtained by successive crushing, rifling and grinding. The size of sample is reduced to passing a 2.4 mm test sieve by using hammer and jaw crusher. The sample is dried in oven at 105℃, and then ground to pass a 150μm test sieve. This shall be the analytical sample. Any metallic iron of the analytical sample is removed using a bar magnet. The analytical sample is further dried in oven at 105℃ for 16 hours and cooled in a desiccator to room temperature. It is then placed in an air-tight bottle and thoroughly mixed by tumbling, rolling or shaking for at least 2 minutes.

2.2 *Test Procedures*

The polymer content is determined by loss on ignition at 550°C after removal of cement by acid-digestion. Organic additives which are not soluble in dilute acid will be included as the polymer content. Dependent on whether the individual constituting raw materials composing the mortar are available, different procedures shall be followed. If the raw materials are not available, the procedures to be followed are essentially as below. The analytical sample is treated with diluted hydrochloric acid at boiling temperature, and the combined solution is soaked on filter paper and then transferred to porcelain crucible. Throughout the procedures, the net mass of the apparatus and mass of apparatus containing the sample are weighed. The crucible containing the filter paper and the solution residue is heated in a muffle furnace to 550°C, and then cooled and weighed. The ignition, cooling and weighing are repeated until the mass is constant.

2.3 *Evaluation of Polymer Content*

Following the above-mentioned procedures, the polymer content is determined as:

$$\text{Polymer content} = [(W_3 - W_1F_1 - W_2) - (W_4 - W_2)F_2]F_3/W$$

In the equation, W is the mass of analytical sample, W_1 is the mass of net mass of dried filter paper, W_2 is the net mass of crucible, W_3 is the mass of crucible containing filter paper and sample residue after drying, W_4 is the mass of crucible and sample residue after ignition, F_1 is the correction factor for the treatment of filter paper, F_2 is the correction factor for sand or blank, and F_3 is the corrector factor for recovery of polymer. Details of evaluation of F_1 and F_2 can be found in Housing Department (1991). The value of F_3 is dependent on the type of polymer, and is recommended to be 1.05 for acrylic styrene copolymer, 1.33 for polyvinyl acetate based copolymer including vinyl acetate/ethylene (VAE) copolymer and vinyl acetate/veova (VA/veova) copolymer, and 1.0 for SBR.

If the raw materials composing the mortar are available, the procedures to be followed are essentially as below. A mortar specimen is prepared by mixing the ingredient materials and cured for 7 days. The polymer content is then determined according to the procedures in Sections 2.1 and 2.2. The polymer content is determined as:

$$\text{Polymer content} = [(W_3 - W_1F_1 - W_2) - (W_4 - W_2)F_2]F_3'/W$$

The variables W, W_1, W_2, W_3, W_4, F_1 and F_2 carry the same meaning as above. The correction factor for the polymer determined F_3' is computed based on the known percentage of polymer added to mortar. Details of evaluation of F_3' can be found in Housing Department (1991).

3 PROPOSED EXPERIMENTATION APPROACH

3.1 *Basis and Principle*

The proposed experimentation approach for pre-packed polymer modified mortars is based on thermogravimetric analysis (TGA) to determine the polymer content. TGA is an analytical technique used to determine a material's thermal stability and identify its fraction of various components by measuring the changes in mass of the material with increasing temperature. The mass loss of sample at different temperatures is recorded by the thermogravimetric curve obtained from the test. To minimize the occurrence and influence of oxidation reactions of sample material, the measurement is performed in an environment of inert gas, such as nitrogen gas and argon gas.

Compared to the conventional approach, the proposed approach has the following merits: (1) it involves simpler procedures of sample preparation and testing; (2) it entails less manual intervention that may introduce errors during the TGA; (3) it provides better information of mass loss at different temperatures as reflected by the thermogravimetric curve; and (4) it enables calibration to be performed to improve the accuracy in determining the polymer content of mortars.

3.2 *Experimental Programme*

During the development stage of the experiment method, polymer powder dry mixed with ordinary Portland cement in various mix proportions, pre-packed dry mix mortars in powder, and hardened pre-packed dry mix mortars were subjected to TGA for comparison of results. In the experimental programme, 8 polymer-cement dry mix samples (laboratory controlled blends), 2 pre-packed mortar powder samples (typical commercial products), and 2 hardened mortar samples (fabricated from typical commercial products) were tested. Table 1 lists the various samples. Besides, for the 2 pre-packed mortar powder samples, the polymer contents were measured using the conventional approach for comparison.

Table 1 Experimental programme

Sample	Description
P0C10	100% cement
P1C9	Mixture of 10% polymer and 90% cement
P2C8	Mixture of 20% polymer and 80% cement
P4C6	Mixture of 40% polymer and 60% cement
P6C4	Mixture of 60% polymer and 40% cement
P8C2	Mixture of 80% polymer and 20% cement
P9C1	Mixture of 90% polymer and 10% cement
P10C0	100% polymer
SA	Pre-packed mortar sample A
SB	Pre-packed mortar sample B
Ha-SA	Hardened sample A mixed with 19% water
Ha-SB	Hardened sample B mixed with 16% water

The polymer-cement dry mix samples were named P(x)C(y), where (x) is the fraction in tenth of the weight of polymer in the mixture, and (y) is the fraction in tenth of the weight of ordinary Portland cement in the mixture. For example, P1C9 means 10% of polymer by weight mixed with 90% of cement by weight. The polymer in powdery form was obtained from the manufacturer. The cement used was Type I Grade 52.5N Portland cement. This group of polymer-cement dry mix samples served as control to examine the resultant mass of ignition residues from the TGA tests with different proportions of polymer.

The pre-packed mortar samples were named SA and SB. They were from two different proprietary pre-packed mortar products (referred to as sample A and sample B) produced by the same manufacturer. Sample A is suited for general usage, while sample B is suited for application to vertical and overhead surfaces. The polymer contents of sample A and sample B as ratio to cement by weight are respectively more than 5.0% and more than 6.5%, which are complying with the relevant requirements in local authorities (Housing Department, 1999). According to the product information from the manufacturer, to form the proper fresh mortar for use, sample A shall be mixed with 19% of water by weight and sample B shall be mixed with 16% of water by weight.

3.3 *Sample Preparation*

In preparing the polymer-cement dry mixes, the polymer powder and the ordinary Portland cement were pre-dried separately in an oven at 105°C and cooled in a desiccator to room temperature. Designated mass of polymer powder and designated mass of cement were weighed, and then mixed in dry environment uniformly. The mixture was dried in an oven at 105°C and cooled in a desiccator before undergoing the TGA.

For the pre-packed mortar samples, the mortar powder received from the manufacturer was dried in an oven at 105℃ and cooled in a desiccator before undergoing the TGA. For the hardened samples, designated mass of mortar powder was weighed and mixed with designated amount of potable water in a mortar mixer according to the aforementioned proportions. The fresh mortars were cast into cube moulds. After at least 1 day, the mortar cubes were demoulded and cured until the age of 7 days. Subsequently, the mortar cubes were broken into fragment by hammer, crushed and ground to powder. The ground powder was dried in an oven at 105℃ and cooled in a desiccator, with or without sieving to 180μm before undergoing the TGA.

To prevent dampening or moistening of the experimentation materials, all materials were stored in a laboratory environment, with regulated temperature and humidity.

3.4 *Test Procedures*

The TGA instrument employed was Perkin Elmer Pyris 1 Thermogravimetric Analyzer and is shown in Figure 1. It adopts a vertical design with the microbalance located above and thermally isolated from the furnace. A precision hang-down wire is suspended from the microbalance down into the furnace. Throughout the heating process, the mass of sample was measured by the microbalance, whose sensitivity and precision were 0.1 μg and 0.001% respectively.

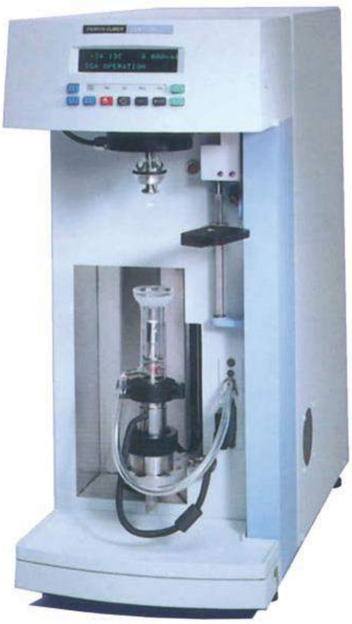

Figure 1 TGA instrument

The maximum temperature of TGA should be set with respect to the material being tested. In this circumstance, the maximum temperature should be such that the mass loss on ignition reflects to the largest degree the polymer content and to the least extent affected by the non-polymer content. In general, most polymer types would be burnt and evaporated under a temperature of 800℃ or less. In contrast, cement and most inorganic ingredients in cementitious mortars would manifest little mass loss at 800℃ in an inert gas atmosphere. Therefore, the maximum heating temperature was designed to be 800℃.

Prior to conducting TGA, the samples were prepared accordance with Section 3.3. Required quantities of samples were weighed to an accuracy of 0.01 mg and placed onto the platinum pan of TGA instrument. The mass of each sample was recorded. The TGA was then carried out to eventually heat the sample to 800℃ at a heating rate of 10℃/min, which is a commonly used rate in performing TGA. Nitrogen gas was used as the inert gas atmosphere. It was envisaged that the residues after ignition would provide reliable basis for estimating the polymer content.

4 RESULTS AND DISCUSSIONS

4.1 *Polymer-Cement Dry Mixes*

The TGA results of the polymer-cement dry mixes samples are tabulated in Table 2. It can been seen that the more proportion of polymer, in other words, the less proportion of cement, in the mixture, the smaller mass of residue on ignition. It is because the loss on ignition of cement is much smaller than polymer, which would be close to completely burnt at 800℃. Figure 2 shows the normalized mass versus temperature curves of the samples. It can be seen that the mass loss throughout the heating process was in gradual stages. At temperature lower than 240℃, the mass loss of samples was not significant. A remarkable loss in mass is observed at a temperature of approximately 350℃. There was more mass loss when the temperature further increased until at about 580℃, the mass became stable. Mass loss is again noticed at about 680℃ to 750℃. Thereafter the mass was stable to the end of TGA. This reflects that different components in the samples were burnt and evaporated at different temperatures.

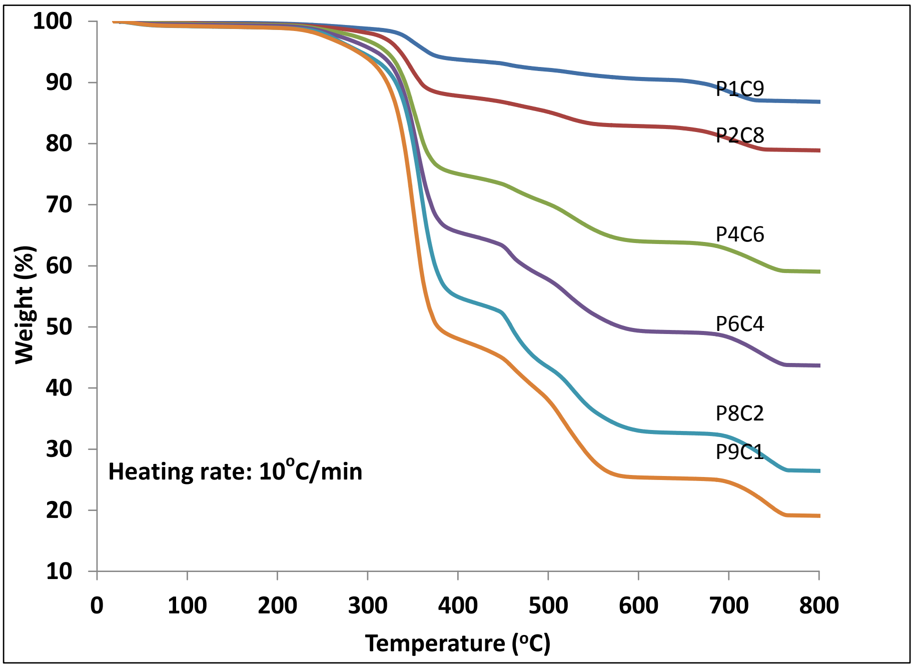

Figure 2 Mass of Polymer-cement dry mix samples during TGA

The loss on ignition of samples is compared with the nominal polymer content. It is found that the variations between the loss on ignition and the nominal polymer content range from +3.2% to -6.4%. Generally speaking, when the polymer content is low, the loss on ignition is larger than the nominal polymer content. In contrast, when the polymer content is high, the loss on ignition is smaller than the nominal polymer content. The source of such variation would be due to incomplete combustion of the polymer and ignition loss of cement. To confirm this point, TGA were conducted for pure cement and pure polymer samples as detailed below, in order to derive the calibration parameters.

Samples P0C10 and P10C0 are respectively pure cement sample and pure polymer powder sample, and were subjected to TGA with the results shown in Figure 3. It is found that the mass of residue of P0C10 is 95.4% (i.e. the loss on ignition is 4.6%), and that the mass of residue of P10C0 is 7.2% (i.e. the loss on ignition is 92.8%). The loss on ignition of cement would be due to trace amount of constituents which are decomposable at high temperature, such as calcium hydroxide, calcium carbonate and calcium sulphate. To investigate the un-burnt residue of polymer, X-ray diffraction (XRD) and energy dispersive X-ray fluorescence spectroscopy (XRF) were conducted. From the XRD scanning results and XRF spectra, minerals or compounds including calcite, silica, and aluminium hydroxide were identified. These would form the residue of ignition.

Table 2 TGA results of polymer-cement dry mixes

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Sample	Nominal polymer content (%)	Mass of residue (%)	Loss on ignition (%)	Variation (4) – (2)	Calculated polymer content (%)	Difference (6) – (2)
P1C9	10	86.8	13.2	+3.2	9.8	-0.2
P2C8	20	78.9	21.1	+1.1	18.9	-1.1
P4C6	40	59.0	41.0	+1.0	41.1	+1.1
P6C4	60	43.7	56.3	-3.7	58.8	-1.2
P8C2	80	26.4	73.6	-6.4	78.4	-1.6
P9C1	90	16.4	83.6	-6.4	89.6	-0.4

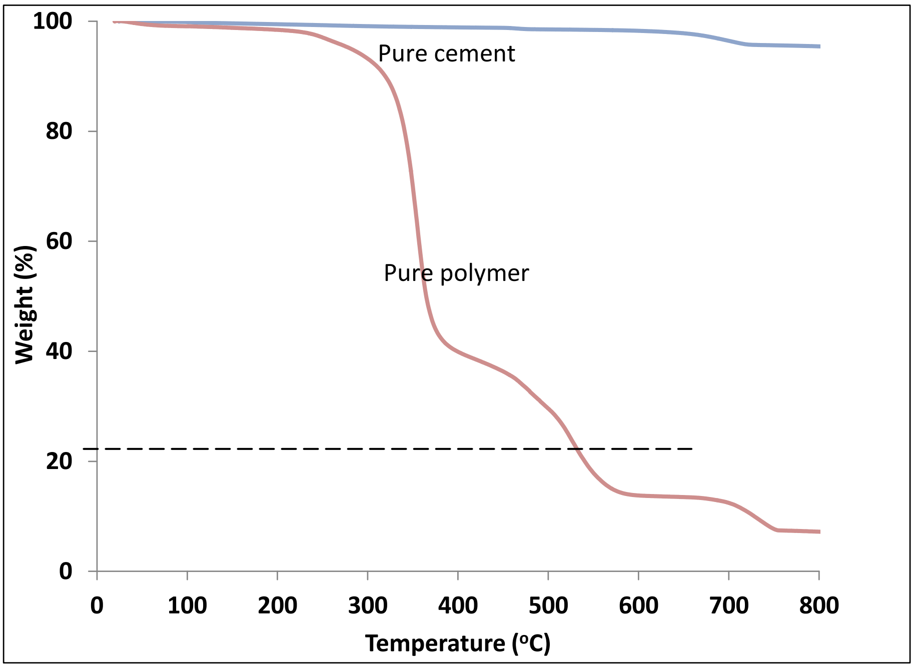

Figure 3 Mass of polymer and cement samples during TGA

To calibrate the test results and calculate the polymer content, the loss on ignition of cement was deducted from and the mass of polymer residue was compensated to the mass loss on ignition of polymer-cement dry mixes samples. The calculated polymer content is compared with the nominal polymer content, and the difference is found to be in the range of +1.1% to -1.6%, which is satisfactory. The difference would be due to experimental errors and non-uniformity of the chemical compositions of cement and polymer in the samples.

4.2 *Pre-packed Mortars*

For pre-packed mortars SA and SB, to examine the repeatability and consistence of test results, the TGA experimentation was done in duplicate, and sample numbers are appended with the post-nominals -(1) and -(2). The results of TGA are tabulated in Table 3. The normalized mass of mortars versus temperature curves are shown in Figure 4. During the TGA, the mass loss of samples up to 200℃ was not significant. Subsequently, a remarkable mass loss is observed at a temperature of approximately 350℃. Afterwards, the mass loss was gradual and became rapid at a temperature of 650℃. At about 750℃, the mass became stable till the end of the TGA. Amongst the duplicated samples, it can be seen that the test results are very consistent. Hence, the average values of the two duplicated samples can be relied on.

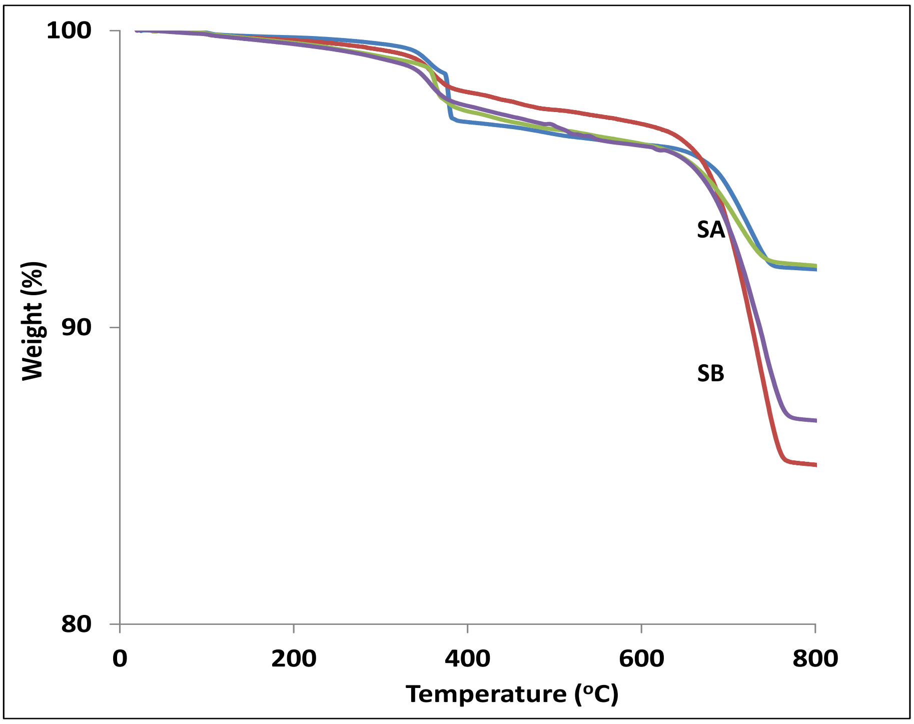

Figure 4 Mass of pre-packed mortar samples during TGA

Table 3 TGA results of pre-packed mortars

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Sample	Polymer content to cement (%)	Nominal polymer content (%)	Mass of residue (%)	Loss on ignition (%)	Calculated polymer content (%)	Difference (6) – (3)
SA-(1)			91.95	8.05		
SA-(2)	> 5.0	2.20	92.07	7.93	2.24	+0.04
SA average			92.01	7.99		
SB-(1)			85.36	14.64		
SB-(2)	> 6.5	2.50	86.85	13.15	2.53	+0.03
SB average			86.10	13.90		

From Table 3, the average mass of residue of SA is 92.01% (i.e. the loss on ignition is 7.99%), and the average mass of residue of SB is 86.10% (i.e. the loss on ignition is 13.90%). To calculate the polymer content, calibration is applied based on the results of the control group of polymer-cement dry mix samples, taking into account the decomposition of additives and other constituents in the mortar mixes at elevated temperature. However, as the TGA of individual raw materials (except polymer and cement) of the pre-packed mortars have not been accomplished, the calibration herein is based on rational assumptions of decomposition mass loss of the raw materials. Hence, the calculated polymer content may be regarded as an interim estimation and shall be refined by further research. At this point, the calculated polymer contents of SA and SB are respectively 2.24% and 2.53% by weight. When computing the differences from the nominal polymer contents, the errors are less than 0.1%. This demonstrates the viability and potentially high accuracy of the proposed approach, as well as its applicability to different types of pre-packed mortars.

As a comparison with the conventional experimentation approach, the same pre-packed mortars SA and SB had been tested according to HKHA Standard 002: 1990 by HOKLAS (The Hong Kong Laboratory Accreditation Scheme) accredited laboratories in real-life construction projects. The polymer contents determined from typical representative samples across different projects were ranging from 0.88% to 2.27% and from 0.95% to 2.15% respectively for SA and SB. These results varied significantly and differed greatly from the actual polymer contents designed and dosed for the mortars which are 2.2% for SA and 2.5% for SB. The major underlying reason of the discrepancies is that the conventional method determines the polymer content from hardened mortars. During curing and hardening of mortar, part of the polymer would be chemically combined with the cementitious matrix (Silva et al., 2002; Kriegel et al., 2003). This influences the loss on ignition of polymer to a large extent. It is conceivable that the proposed experimentation approach using TGA is a better quality assurance testing method than the conventional approach.

4.3 *Hardened Mortars*

The proposed experimentation approach was applied to hardened pre-packed mortars to examine the outcome. Due to the larger variation of test results envisaged for hardened pre-packed mortars, four measurements were taken for hardened mortars SA and SB. Amongst each mortar mix, the TGA experimentation was done in triplicate for the ground powder without sieving, and an additional TGA experimentation was done for the ground powder with sieving. For ease of reference, the sample numbers are appended with post-nominals -(1), -(2), -(3) and -(siev). The results of TGA are shown in Table 4. The normalized mass versus temperature curves for hardened mortar SA and SB are displayed in Figures 5 and 6, respectively. In comparison with mortars in powdery form, the 4 measurements of hardened mortar amongst each mortar mix exhibit larger discrepancies. Therefore, the experimentation with hardened mortars would be less reliable.

Table 4 TGA results of hardened pre-packed mortars

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Sample	Nominal polymer content (%)	Average loss on ignition of powder (%)	Mass of residue (%)	Loss on ignition (%)	Calibrated loss on ignition (%)	Difference (6) – (3)
Ha-SA-(1)	2.20	7.99	78.90	21.10	12.19	+4.20
Ha-SA-(2)			80.35	19.65	11.39	+3.40
Ha-SA-(3)			85.18	14.82	9.93	+1.94
Ha-SA-(siev)			82.81	17.19	10.52	+2.53
Ha-SB-(1)	2.50	13.90	79.55	20.45	15.88	+1.98
Ha-SB-(2)			76.39	23.61	17.31	+3.41
Ha-SB-(3)			84.19	15.81	15.16	+1.26
Ha-SB-(siev)			75.41	24.59	17.44	+3.54

Unlike the foregoing, the mass loss of hardened mortar was significant at around 100℃, which reflects the evaporation of free water. The mass loss gradually increased with temperature, and the rate of mass loss varied with the temperature. This should be caused by the combined effect of loss of bound water in the hydration product of cement, as well as the decomposition of polymer and additives at respective ranges of temperature. At temperature higher than about 750℃, the mass loss occurred at a reduced rate but was still observable. This is in contrast with the behaviour of polymer-cement dry mixes and pre-packed mortars. The reason would be the release of chemically bound water in hydrated cement required a large amount of energy to decompose the calcium-silicate-hydrate gel. Besides, as pointed out by previous research studies that the polymer would be chemically combined with the cementitious matrix (Silva et al., 2002; Kriegel et al., 2003), the evaporation of chemically bound polymer would require much more energy compared to un-reacted polymer. The decomposition of gel and release of chemically bound water and polymer might not have fully taken place even at the end of TGA process.

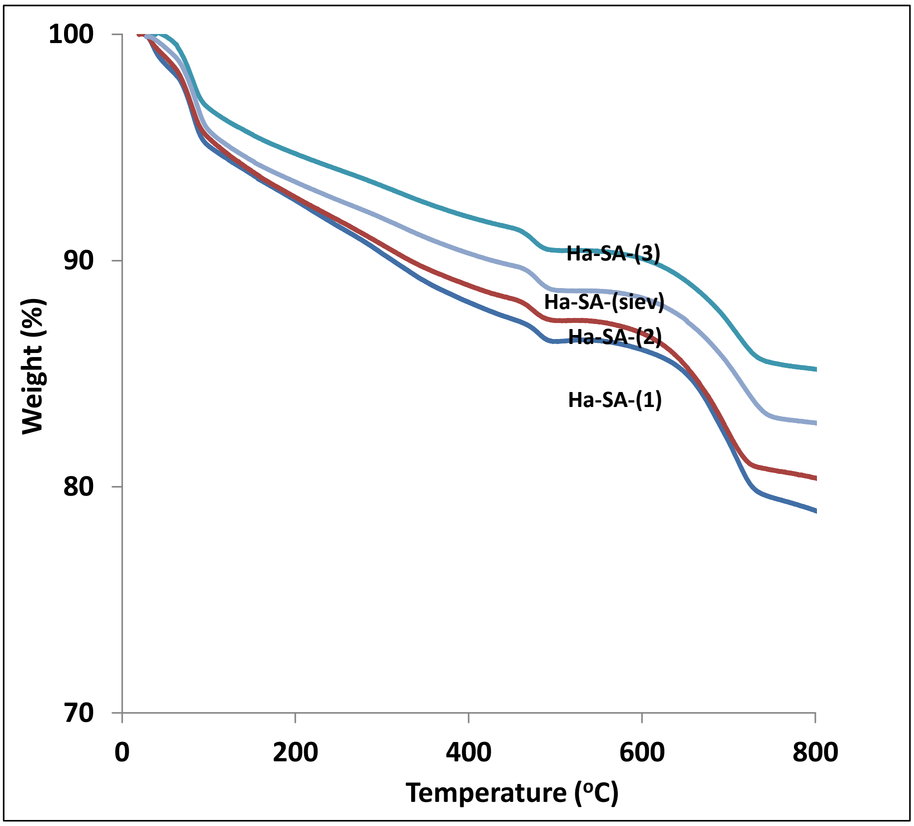

Figure 5 Mass of hardened pre-packed mortar Sample A during TGA

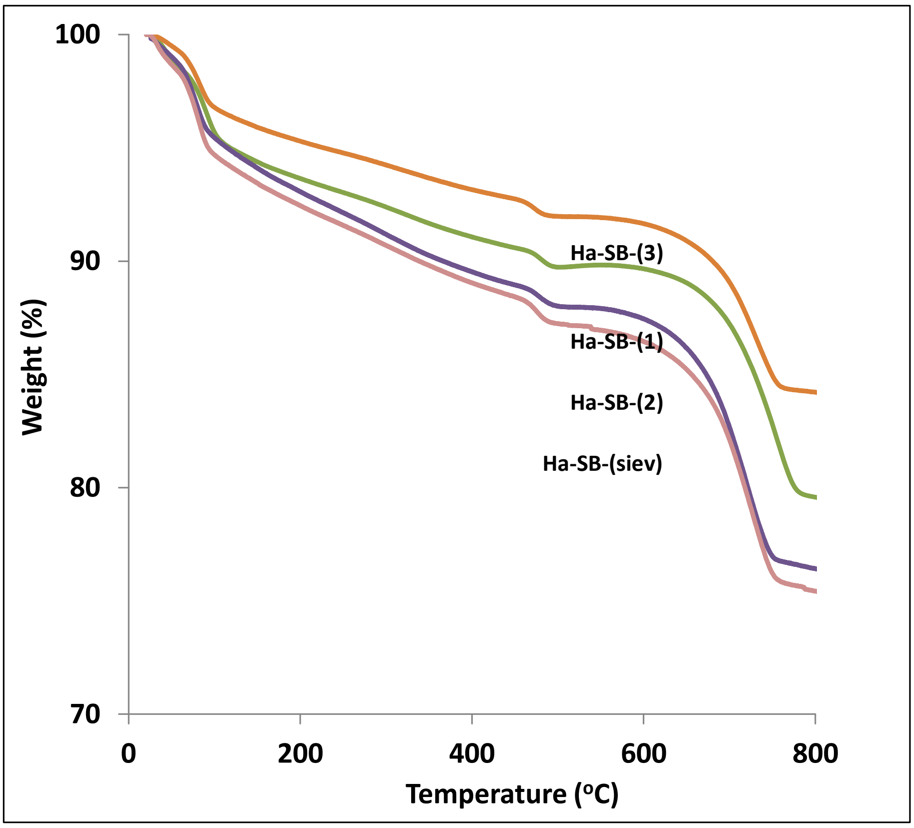

Figure 6 Mass of hardened pre-packed mortar Sample B during TGA

As seen in Table 4, the mass of residue of hardened SA is ranging from 78.90% to 85.18%, and that of hardened SB is ranging from 75.41% to 84.19%. The measured loss on ignition was inclusive of the evaporation of free water and bound water. Therefore, calibration is carried out to eliminate the contribution from free water and bound water evaporation. For hardened SA, the calibrated loss on ignition is ranging from 9.93% to 12.19%, which is higher than that of pre-packed mortar SA in powder form. Both the mass of residue and calibrated loss on ignition values of sieved sample from hardened SA are in the midst of the un-sieved triplicate samples. For hardened SB, the calibrated loss on ignition is ranging from 15.16% to 17.44%, which is again higher than that of pre-packed mortar SB in powder form. The mass of residue of sieved sample from hardened SB is lower than all the un-sieved triplicate samples, whilst the calibrated loss on ignition of sieved sample from hardened SB is higher than all the un-sieved triplicate samples. Moreover, the differences in loss on ignition results between hardened SA and SB and their powder form are considerably large.

As pointed out in the above, the decomposition of calcium-silicate-hydrate gel required large amount of energy and the removal of chemically bound water and polymer might not complete. Due to the large discrepancies of measurements amongst individual hardened mortars, the proportions of bound water and polymer removed during TGA could not be reliably estimated. For this reason, the calibrated loss on ignition did not tally with the loss on ignition of mortar powder. Hence, calculation of polymer content in hardened mortars from TGA results is not recommended. Moreover, while the testing of hardened pre-packed mortars is not at the upstream of quality control, the authors advocate the testing of pre-packed mortar powder by the proposed approach.

5 CONCLUSION

In this study, a new approach to determine the polymer content of pre-packed polymer modified mortar (dry mix) based on thermogravimetric analysis (TGA) has been proposed. The mass of residues and loss on ignition of the mortar sample upon heating to a temperature of 800℃ have been obtained. Calibration can be performed with respect to the results of thermogravimetry to account for factors such as decomposition of mortar additives, so that the accuracy of polymer content calculation can be much improved. Different laboratory-controlled blends of polymer-cement dry mix samples, pre-packed mortar powder samples, and hardened mortar samples have been tested to testify the validity of the proposed experimentation approach. It has been demonstrated that the new approach is able to determine the polymer content of pre-packed mortars with a higher accuracy than the conventional approach, whose repeatability and accuracy may not be desirable from past experience. At this interim stage, the laboratory-controlled dry mix samples and the pre-packed mortar samples contained the same single type of polymer in pre-dried form. Applications of the proposed experimental approach to mortars containing other types of polymers will be conducted in the next stage of study. Further research is needed to establish an inventory for scientific calibration of the TGA results taking into account the plausible statistical variations, with a view to fingerprint the thermogravimetric response of a wide range of raw materials in pre-packed mortars.

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